Upper San Joaquin River Basin Storage Investigation

Initial Alternatives Information Report

Engineering Technical Appendix

A Study By:

RECLAMATION
Managing Water in the West

California Department of Water Resources

In Coordination With:

CALFED BAY-DELTA PROGRAM

Prepared By:

MWH

UPPER SAN JOAQUIN RIVER BASIN STORAGE INVESTIGATION

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ABBREVIATIONS AND ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ac-ft	acre feet
ASBI	American Segmental Bridge Institute
BDPAC	California Bay-Delta Public Advisory Committee
BLM	Bureau of Land Management
CALFED	August 2000 CALFED Bay-Delta Program
CDEC	California Data Exchange Center
CFRF	concrete face rockfill
cfs	cubic feet per second
Corps	United States Department of the Army, Corps of Engineers
CVP	Central Valley Project
cy	cubic yards
Delta	Sacramento-San Joaquin Delta
DEM	digital elevation model
DWR	California Department of Water Resources
EIR	Environmental Impact Report
EIS	Environmental Impact Statement
elevation	elevation in feet above mean sea level
ESRI	Environmental Systems Research, Inc.
FERC	Federal Energy Regulatory Commission
FHAR	Flood Hydrograph and Routing Program
FR	Feasibility Report
FWUA	Friant Water Users Authority
g	gravitational acceleration units
GWh	gigawatt-hours
GIS	geographic information system
hp	horsepower
IAIR	Initial Alternatives Information Report
Investigation	Upper San Joaquin River Basin Storage Investigation
K1	Kerckhoff Powerhouse
K2	Kerckhoff No. 2 Powerhouse
kw	kilowatt
LIDAR	light detecting and ranging
m	meter
msl	mean sea level

MW	megawatts
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
NRDC	Natural Resources Defense Council
OW	outlet works
P&G	Economic and Environmental Principles and Guidelines for Water and Related Resources Implementation Studies
PFR	Plan Formulation Report
PG&E	Pacific Gas and Electric
PH	powerhouse
PMF	probable maximum flood
PHA	peak horizontal acceleration
PT	power transformer
RCC	roller compacted concrete
Reclamation	U. S. Department of the Interior, Bureau of Reclamation
RM	River Mile
ROD	Record of Decision
ROW	river outlet works
SCADA	supervisory control and data acquisition
SJR	San Joaquin River
sq-mi	square mile
SRA	State Recreation Area
STATSGO	State Soil Geographic
TAF	thousand acre-feet
TEWAC	totally enclosed, water/air cooled
TIGER	topologically integrated geographic encoding and referencing
USCS	Unified Soil Classification System
USGS	United States Geological Survey
yd^3	cubic yards

CHAPTER 1. INTRODUCTION

This document is the **Engineering Technical Appendix** (TA) to the Initial Alternatives Information Report (IAIR) for the Upper San Joaquin River Basin Storage Investigation (Investigation). The Investigation is one of five surface water storage studies recommended in the CALFED Bay-Delta Program (CALFED) Programmatic Environmental Impact Statement/Report (PEIS/R) Record of Decision (ROD) of August 2000. It is being performed by the U.S. Department of the Interior, Bureau of Reclamation (Reclamation), and the California Department of Water Resources (DWR). The Investigation is a feasibility study evaluating alternatives to develop water supplies from the San Joaquin River that could contribute to the restoration of, and improve water quality in, the San Joaquin River, and enhance conjunctive management and exchanges to provide high-quality water to urban areas.

The Investigation is being prepared in two phases. Phase 1, which included preliminary screening of initial storage sites, was completed in October 2003. Initially, 17 surface water storage sites were considered, of which 6 were retained for further analysis. Phase 2 began in January 2004 with formal initiation of environmental review processes consistent with Federal and State of California (State) regulations, and will continue through completion of all study requirements. The Investigation will culminate in a Feasibility Report (FR) and supporting environmental documents consistent with the Economic and Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies (P&G) (WRC, 1983), Reclamation directives, DWR guidance, and applicable environmental laws. Reclamation and DWR are coordinating the Investigation with the California Bay-Delta Public Advisory Committee (BDPAC), which provides advice to the Secretary of the United States Department of the Interior (Secretary) regarding the implementation of the CALFED Program, and the California Bay-Delta Authority (CBDA), which provides general oversight and coordination of all CALFED activities.

To facilitate coordination with other agencies and related ongoing studies, preparation of the FR will include two interim planning documents: an Initial Alternatives Information Report (IAIR) and a subsequent Plan Formulation Report (PFR). The IAIR describes without-project conditions and water resources problems and needs; defines study objectives and constraints; screens surface water storage measures; describes groundwater storage measures development; and identifies preliminary water operations rules and scenarios. Retained storage measures and preliminary water operations scenarios will be included in initial alternatives. This IAIR will be used as an initial component of the FR. The PFR will present the results of initial alternatives evaluation, identify refinements of the alternatives, and define a set of final alternatives. A Draft FR will evaluate and compare the final alternatives and identify a recommended plan. A Draft Environmental Impact Statement (EIS) and Environmental Impact Report (EIR) will be included with the Draft FR. Following public review and comment, a final FR/EIS/EIR will be prepared.

STUDY AREA

The study area emphasis for the Investigation encompasses the San Joaquin River watershed upstream of Friant Dam, the San Joaquin River from Friant Dam to the Sacramento-San Joaquin Delta (Delta), and the portions of the San Joaquin and Tulare Lake hydrologic regions served by the Friant-Kern and Madera canals, as highlighted in **Figure 1-1**. The study area includes all potential storage sites under consideration, the region served by the Friant Division of the Central Valley Project (CVP), the eastern San Joaquin Valley groundwater basins, and the portion of the San Joaquin River most directly affected by the operation of Friant Dam. The study area includes a primary study area and an extended study area. The primary study area for evaluations presented in this TA includes the locations of all potential surface water storage sites under consideration.

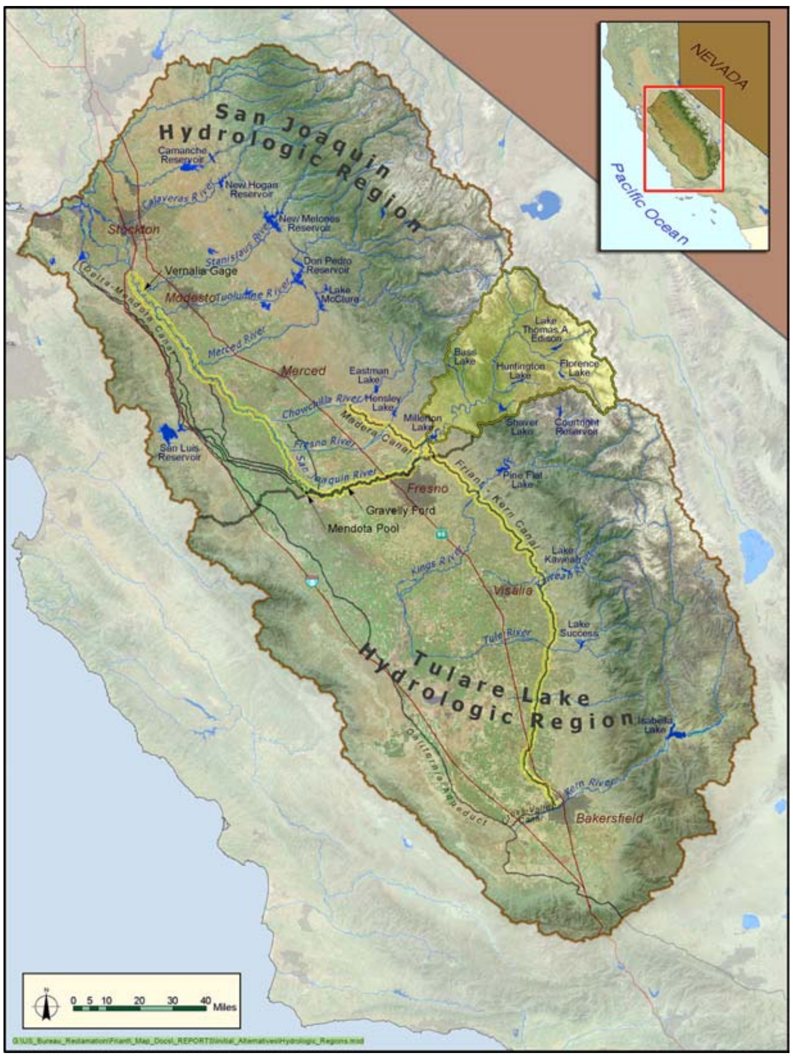

FIGURE 1-1.
UPPER SAN JOAQUIN RIVER BASIN STORAGE INVESTIGATION
STUDY AREA EMPHASIS

SURFACE WATER STORAGE MEASURES CONSIDERED IN THE IAIR

Six potential sites for developing a new surface reservoir or enlarging an existing reservoir were retained from Phase 1 of the Investigation for further consideration in the Investigation. Each site could be configured at various storage sizes, with each configuration identified as a measure. The six surface water storage sites retained from Phase 1 include:

- **Raise Friant Dam.** Enlarging Millerton Lake by raising Friant Dam up to 140 feet.
- **Temperance Flat Reservoir.** Constructing Temperance Flat dam and reservoir at one of three potential dam sites on the San Joaquin River, between Friant and Kerckhoff dams, at River Mile (RM) 274, RM 279, or RM 286.
- **Fine Gold Reservoir.** Constructing a dam and reservoir on Fine Gold Creek to store water diverted from the San Joaquin River or pumped from Millerton Lake.
- **Yokohl Valley Reservoir.** Constructing a dam and reservoir in Yokohl Valley to store water conveyed from Millerton Lake by the Friant-Kern Canal and pumped into the reservoir.

Most of the surface water storage measures retained from Phase 1 would result in a net loss in power generation. In March 2004, Reclamation and DWR held a series of scoping meetings to initiate development of an EIS/EIR. During scoping, power utilities that own and operate hydropower projects in the upper San Joaquin River basin raised concerns about impacts of lost power generation and the ability of retained measures to develop adequate replacement power. These hydropower stakeholders suggested five additional potential reservoir sites that could store water supplies from the upper San Joaquin River without adversely affecting existing hydropower facility operations.

Suggested storage measures include **RM 315 Reservoir** on the San Joaquin River between Redinger Lake and Mammoth Pool, and **Granite Project** (Granite Creek and Graveyard Meadow reservoirs) and **Jackass-Chiquito Project** (Jackass and Chiquito reservoirs) on tributaries to the San Joaquin River upstream of Mammoth Pool. The scoping comments also suggested combining these upstream storage measures with a gravity diversion tunnel from Kerckhoff Lake to a Fine Gold Reservoir.

The locations of the six surface water storage sites retained from Phase 1 and sites suggested during scoping are shown in **Figure 1-2.** This TA presents design and cost information on various configurations of the six surface water storage sites retained from Phase 1. Costs of surface water storage measures suggested during scoping were obtained from previous reports prepared by others, as cited in the IAIR, and are not included in this TA.

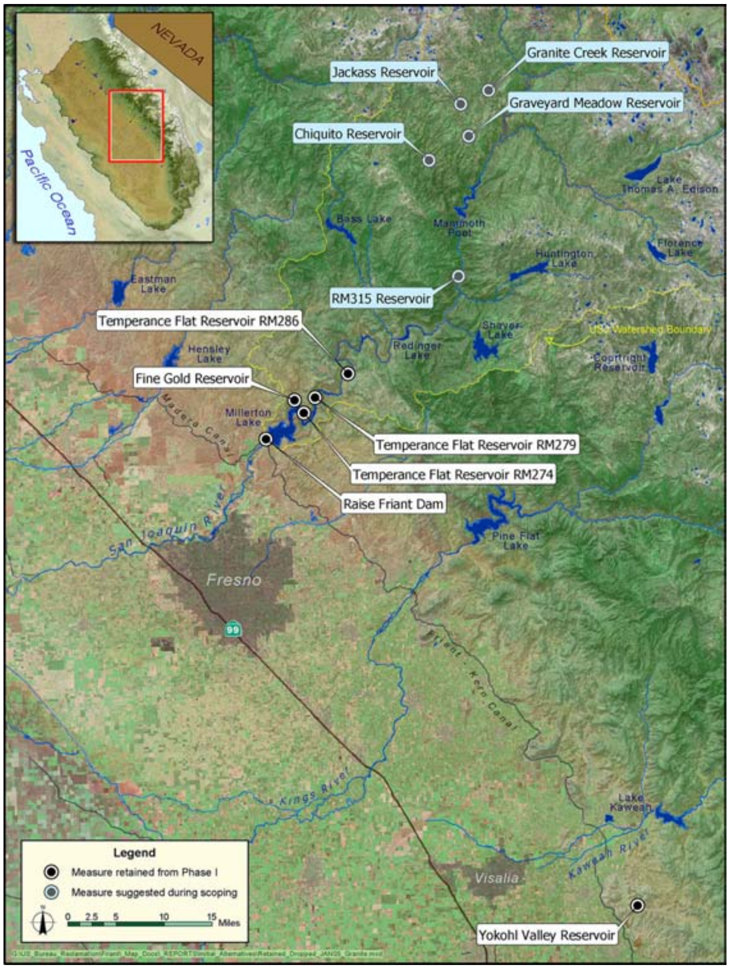

FIGURE 1-2.
SURFACE WATER STORAGE SITES RETAINED FROM PHASE 1 AND SUGGESTED DURING SCOPING

ORGANIZATION OF THIS TECHNICAL APPENDIX

This document is one of several technical appendices to the IAIR. It presents preliminary information on engineering assumptions and designs, and cost estimates for potential surface water storage measures under consideration. Much of the information presented in this document focuses on the general geologic setting at each potential dam site, alternative dam design configurations, facilities for hydropower generation, and preliminary assumptions for retrofitting, relocating, or removing impacted infrastructure. Information regarding potential hydrologic and hydroelectric energy operations for each measure is presented in separate technical documents: **Flood Benefits TA** and **Hydropower TA**.

This introductory chapter explains the purpose and scope of this **Engineering TA**, provides an overview of the surface storage sites retained from Phase 1 of the Investigation, and outlines methods used to develop the technical information presented in the TA.

The next six chapters present technical information for each of the potential dam and reservoir measures retained from Phase 1 of the Investigation. Each site-specific chapter describes relevant physical site characteristics, contemplated structural improvements, resulting reservoir sizes, appurtenant features that would need to be constructed, required relocations of, or modifications to, existing infrastructure, and summaries of construction field costs and construction costs. The last two chapters contain a list of preparers and references used to develop the technical material, respectively.

Five attachments also are included in this appendix. **Attachment A** is the seismic hazard analysis developed in 2002 during Phase 1 of the Investigation. **Attachment B** contains figures of preliminary layouts for potential storage structures and appurtenant features, with representative cross sections of the different dam types considered. **Attachment C** contains cost summary worksheets for the surface water storage measures and related facilities, plus supporting lists of auxiliary mechanical systems that would be included in construction of new powerhouses. **Attachment D** contains the recent probable maximum flood study conducted for consideration in design of a dam at RM 286. **Attachment E** is the recent report of results of geologic drilling investigations at the RM 286 and Fine Gold dam sites.

METHODOLOGY

Information presented in this **Engineering TA** was developed from prior studies, field reconnaissance of the dam and reservoir sites, preliminary designs and costs, and subsequent analysis. Professional experience with similar investigations guided the development of designs and costs.

Engineering and Geology

Designs presented in this appendix were developed at an appraisal level of detail (i.e., prefeasibility). Only standard structure types, consistent with current engineering principles and practices, were considered, with the exception of the Yokohl Valley dam design, which was developed in the early 1970s and has not been updated to current standards. All other design layouts, sections, and dimensions are based on standard practice and experience with similar facilities. With the sole exception of the outdated Yokohl dam design, structures presented in this appendix are considered to be technically viable, based on available information. At the current level of design, no focused efforts were made to optimize the design or minimize cost.

Different dam types were considered at several of the potential dam sites. To avoid unnecessarily complicating the analysis, it was assumed for most surface water storage measures that the dam crest elevation and reservoir gross pool elevation were the same. It is recognized that in actual design and construction, this will not be the case. This assumption was not made for the Raise Friant Dam measures, for which the height of the raise would be measured relative to the existing fixed spillway crest elevation, or for Yokohl Valley Dam, for which only a single dam type was considered.

Throughout this report, left and right directions (e.g., left bank) are used with reference to an orientation facing downstream.

Site Visits

In June 2002, civil engineers and geologists from the Reclamation Mid-Pacific Regional office in Sacramento, California, and Technical Services Center in Denver, Colorado, visited the Friant, Temperance Flat, and Fine Gold dam sites and MWH engineers and geologists visited the Yokohl Valley. During the field trips, dam abutments, possible borrow areas, and site accesses were visually examined and locations of existing and proposed structures were visually assessed. Topography, geology, geotechnical conditions, and utilities were noted. Access routes and possible borrow, staging, and laydown areas were considered. Trip reports for the June 2002 visits are contained in Appendix A of each of the Phase 1 Investigation technical memoranda prepared as appendices to the Phase 1 Investigation Report (Reclamation, 2003d). Additional visits were made to Friant Dam in August 2002 and to Temperance Flat area sites in May 2003. Core samples were obtained for the Fine Gold and Temperance Flat area RM 286 dam sites in summer 2003 (**Attachment E**).

Seismic Analysis

An appraisal level seismic analysis was conducted by Reclamation in August 2002 (Reclamation, 2002b) using readily available information. Peak horizontal accelerations (PHA) from identified faults or background sources were calculated for each potential dam site for return periods of 2,500 years, 5,000 years, and 10,000 years, which correspond to exceedance probabilities of 0.0004, 0.0002, and 0.0001, respectively.

Twenty-two potential fault sources were identified from available information. Faults include the San Andreas Fault, seven western Great Valley faults, seven eastern Sierra Nevada faults, the White Wolf fault of the southern San Joaquin Valley, and six faults of the Sierra Nevada Foothills fault system. No major throughgoing or shear zones have been identified in this area of the Sierra Nevada, and historic seismicity rates are low. However, no on-site seismic investigations were conducted, nor were aerial or satellite imagery examined for other potential faults.

The areal or background seismic source considered was the South Sierran Source Block, a region possessing relatively uniform seismotectonic characteristics that encompasses the entire eastern San Joaquin Valley and the western side of the Sierra Nevada, extending roughly from the watershed of the Consumnes River in the north to the Kern River watershed in the south.

Mapping

Aerial topography acquired by Reclamation in August 2001 using light detecting and ranging (LIDAR) technology was used to produce base maps with 10-foot contour intervals for preparing preliminary designs and calculating quantities for Fine Gold and Temperance Flat area dam sites. United States Geological Survey (USGS) topographic maps were used for analysis of saddle dams at an enlarged Millerton Lake. Storage volume data and illustrative maps were produced using 10-meter grid digital elevation model (DEM) data publicly available from the USGS.

In July 2002, Reclamation geologists conducted geologic mapping of Friant, Temperance Flat, and Fine Gold dam sites, and the Friant saddle dam site, and evaluated potential borrow sources for construction. Details can be found in Reclamation's Appraisal Geologic Study: Storage Options in the Millerton Lake Watershed (Reclamation, 2002a).

Cost Estimates

All features discussed in this report were designed and estimated at the appraisal level. These estimates are not suitable for use as a basis for construction authorization. The construction cost represents a cost estimate to construct each surface water storage measure. It includes individual construction costs for each major feature included in that measure. Where existing features are required to be modified, relocated or abandoned, individual construction costs for these features were developed and are included in the construction cost for each storage measure. Construction costs are cost estimates that include direct costs (major line items that generally have quantities and unit prices applied) and allowances for mobilization, unlisted items, contingencies, and indirects. The allowance for unlisted items adjusts the estimate to account for the minor items that have not been incorporated. Generally, as more details are developed to refine a specific cost estimate, the number of direct cost line items increases, the accuracy of the quantity take-offs increase, and the allowance for unlisted items decrease. Generally, for appraisal estimates the allowance for unlisted items is 15 percent. Contingency costs are provided to accommodate funding needs required after construction starts. These funds are used for overruns on quantities, changed site conditions, order for changes, etc. Indirect costs are provided to accommodate funding requirements for the agency from early project investigations through completion of construction. These funds are used for project investigations, development of environmental documentation, project design data, construction designs and specifications, construction management, and general service activities and facilities.

Cost estimates were developed using the following steps. Major line items to construct the feature were identified and quantified. Unit prices were assigned for the various items identified and the resulting line items cost determined. Line item costs were summed and an allowance of 5 percent was added for mobilization. A 15 percent allowance for unlisted items was added to the resulting subtotal. The sum of the above produces a contract cost which should be comparable to the price proposal received when the project is solicited. Contingencies are then added to produce a field cost, which should represent the cost of the feature after construction is complete. Indirects are then added to produce a construction cost which represents the cost of the feature including all agency overhead requirements. The summations of all construction costs for a given surface water storage measure will produce a construction cost for that measure.

All construction costs in this appendix are reported at July 2004 price levels. Cost estimates originally developed in July 2003, and presented in technical appendices to the Phase 1 Investigation Report in October 2003, were assigned a contingency allowance of 30 percent. Cost estimates produced in July 2004 were developed to a higher degree of detail and assigned a contingency of 25 percent. All estimates from July 2003 were adjusted to account for inflationary effects on construction materials and labor. Reclamation construction cost trend indices were used to determine the appropriate price escalation percentage.

Costs for relocating Powerhouse Road and Bridge for a Temperance Flat Reservoir at an elevation of 1,200 feet above mean sea level (msl) (elevation 1,200) or greater were estimated at an appraisal level of detail in consideration of design requirements. Costs for other road relocations and new bridge construction were estimated with a more general approach by applying the average unit cost of road or bridge relocation from the Powerhouse Road costs (expressed in $/linear foot) to the length of relocated road or bridge that would be required. Required road and bridge lengths were determined by identifying and measuring potential relocation routes on topographic maps developed with a geographic information system (GIS) using 10-meter DEM data and data from the 2000 United States Census Bureau's topologically integrated geographic encoding and referencing (TIGER) system.

For the Raise Friant Dam and Temperance Flat Reservoir measures, costs for constructing replacement or new recreation facilities are excluded from the estimates. However, these costs would be expected to be a relatively small portion of the construction cost. Costs for environmental mitigation are excluded from the construction cost estimates because mitigation requirements presently are unknown. Mitigation requirements typically are defined at a later stage in project evaluation.

Property Acquisition Cost Estimates

Privately owned property in the Millerton Lake, Fine Gold, Temperance Flat, and Yokohl Valley areas was evaluated to determine the approximate acquisition cost or value of land that would be inundated by the storage measures. Land held by the State of California or the United States was not included in the calculations. Acquisition costs for the Millerton Lake area were calculated using market research and "drive-by" appraisals of over 440 properties. GIS was used to identify properties affected by various inundation pools. If an inundation pool touched a developed property, it was the appraiser's opinion that it be considered a "total take." Undeveloped rural acreage was considered "partial take" and the cost was calculated as a percentage of the total based on the inundated area.

For the Temperance Flat and Fine Gold sites, the study conducted in the Millerton Lake area provided data regarding the average cost per acre for "undeveloped rural acreage" (which was defined as a parcel of at least 5 acres with no assessed improvements). An average cost per acre was separately determined for the Yokohl Valley area. Property in the Fine Gold, Temperance Flat, and Yokohl Valley areas was considered to be primarily "undeveloped rural acreage." For each storage measure, the area of a potential inundation pool found to be in private ownership was multiplied by the average cost per acre to determine the total acquisition cost. For all except Raise Friant Dam storage measures, an approximate value was added to the cost of acreage to represent the value of improvements on properties.

Potential costs to acquire existing hydroelectric project assets that would be abandoned or relocated are not included in the cost estimates, nor are potential costs to compensate facility owners for reduced generation. These potential acquisition and damages costs will be quantified at a later stage in the Investigation. Costs to abandon existing facilities and to construct potential replacement power generation facilities, however, have been calculated and included.

Reporting Construction Costs

Summary cost estimates for storage measures are presented in separate chapters for each potential reservoir site. Construction costs to develop a water storage facility were separated from the construction costs for development of replacement hydropower generation facilities. Summary tables in the text of this appendix are based upon detailed worksheets presented in **Attachment C**. All construction costs are expressed in July 2004 dollars.

Consistent with Reclamation practice, numbers are generally rounded to two significant digits. In cases where the first digit is a one, three significant digits are reported with the third digit rounded to the nearest multiple of five. For simplicity of reporting, features with costs less than $10 million are shown to the nearest million.

CHAPTER 2. RAISE FRIANT DAM

This chapter describes engineering features and costs associated with raising Friant Dam. It includes a discussion of site conditions, engineering design considerations, land acquisition, and relocating or abandoning of existing facilities. More detailed descriptions of existing and potential hydroelectric generation facilities summarized in this chapter are included in the **Hydropower TA**.

PREVIOUS STUDIES

Several previous studies examined the potential to provide new water storage at Millerton Lake. The information presented in this appendix builds on those studies.

In March 1930, Hyde Forbes, an engineering geologist, issued a geological report on three potential dam sites on the San Joaquin River for the California Department of Public Works, Division of Water Resources. The report evaluated geologic conditions at the current location of Friant Dam, at Fort Miller (downstream of the confluence of Fine Gold Creek), and near Temperance Flat at River Mile 274 (just upstream of the confluence of Fine Gold Creek). The study found the bedrock at Friant entirely satisfactory as a foundation for a concrete arch or gravity dam; found the Temperance Flat location to be an excellent site and foundation for a concrete arch type dam; and identified the Fort Miller site as the least desirable of the sites from a geologic perspective (Forbes, 1930). The study contributed to efforts that resulted in construction of Friant Dam.

In 1952, 10 years after completion of Friant Dam, Reclamation conducted a study to determine the feasibility of raising Friant Dam. The study included designs and costs for raising Friant Dam by 60 feet and constructing earth saddle dams. Based on a comparison of costs to potential revenue from the increased yield, the study concluded that the raise would be infeasible (Reclamation, 1952).

Reclamation revisited the potential cost for a 60-foot raise at a reconnaissance level in 1975, and developed a cost estimate for an approximate 140-foot raise in 1982.

In 1997, Reclamation reviewed the prior raise studies, updated cost estimates, and reassessed the feasibility of raising Friant Dam by 60 feet and 140 feet. The study concluded that both raises are technically feasible (Reclamation, 1997).

In 2000, a study conducted for the Friant Water Users Authority (FWUA) and Natural Resources Defense Council (NRDC) coalition considered a 20-foot raise of Friant Dam as one of many alternatives for increasing potential water supply to the San Joaquin River (URS, 2000).

SITE DESCRIPTION

Friant Dam and Millerton Lake lie on the San Joaquin River on the border between Fresno and Madera counties, near the community of Friant, about 20 miles northeast of Fresno. The general location is shown in **Figure 1-2**. Millerton Lake and vicinity are shown in **Figure 2-1**.

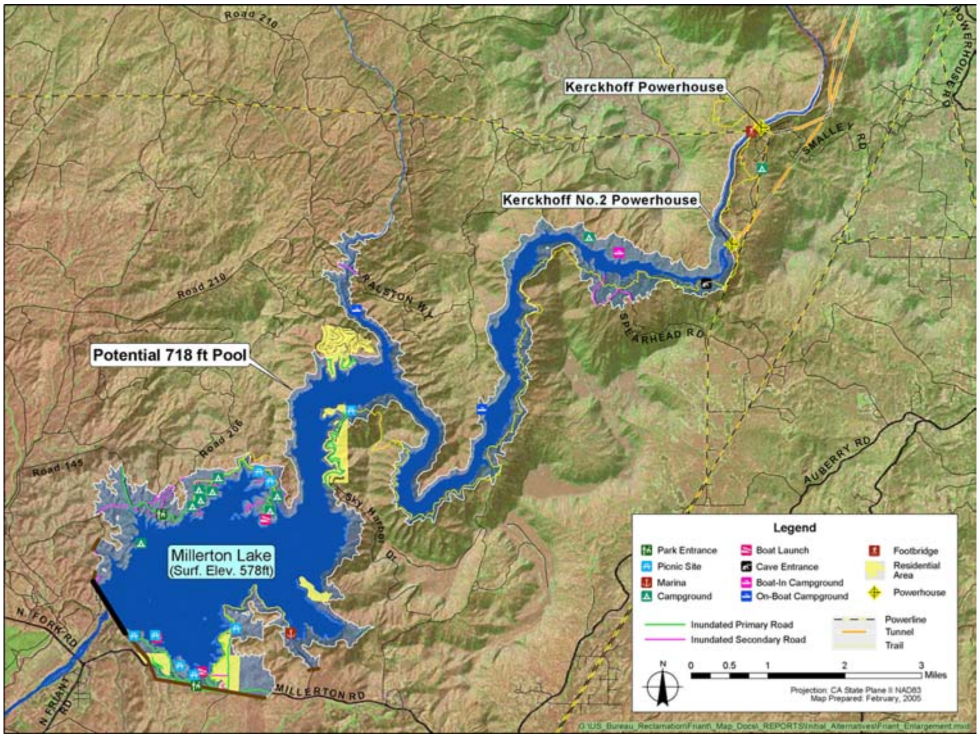

FIGURE 2-1.
POTENTIAL ENLARGEMENT OF MILLERTON LAKE

Topographic Setting

Regional topography consists of the nearly level floor of the San Joaquin Valley rising abruptly to moderately steep, northwest-trending foothills with relatively wide riverine canyons. Farther east, the terrain quickens and the canyons become narrower and steeper. The canyons have been cut by southwest- to west-flowing rivers and associated large tributaries. The San Joaquin River is the main river in the area. The topography of the San Joaquin River basin rises to over elevation 12,000 in the upper watershed portion of the Sierra Nevada mountain range. Elevations in the Millerton Lake area range from about elevation 310 at Friant dam to over elevation 2,100 at the ridges surrounding the upper end of the reservoir.

Friant Dam is located in a section of river that passes through a narrow, southwest-trending bedrock slot in a relatively broad valley at the edge of the San Joaquin Valley. Outside the immediate canyon slot, the right abutment slope rises at a 5:1 horizontal to vertical ratio to about elevation 700. The left abutment slope rises at a lesser inclination, undulating over a broad area along the southern rim of the reservoir, which also rises to slightly over elevation 700.

Geologic Setting

Friant Dam is founded on metamorphic rocks consisting of quartz biotite schist, intruded by aplite and pegmatite dikes and by inclusions of dioritic rocks. The site is located along the western border of the central portion of the Sierra Nevada Geomorphic Province at its boundary with the eastern edge of the Great Valley Geomorphic Province of California. The contact of the metamorphic rocks with the Sierra Nevada batholith lies just east of Friant Dam in Millerton Lake. The Sierra Nevada batholith comprises primarily intrusive rocks, including granite and granodiorite, with some metamorphosed granite such as granite gneiss. Intrusive Sierra Nevada batholith rocks underlie most of Millerton Lake and Friant Dam, Kerckhoff Dam, and the potential Temperance Flat and Fine Gold dam sites. Occasional remnants of lava flows and layered tuff are present in the Millerton Lake area at the highest elevations.

The central Sierra Nevada has a complex history of uplift and erosion. The predominant uplift tilted the western flank of the Sierra Nevada to the west. At the western border, alluvium and sedimentary rocks of the Great Valley Province overtop rocks of the Sierra Nevada. The metamorphic rocks in the Friant Dam area dip steeply downstream to the west, and strike northwesterly. Erosion has resulted in thin alluvial cover.

Friant Dam and the three saddle dams constructed in association with it are founded on metamorphic rock consisting of hard quartz biotite schist, transected by many varying granitic (probably dioritic) dikes. These dikes are mostly aphanitic, but include pegmatitic and porphyritic varieties. Most are less than a few feet thick and locally include a few thin veins of quartz. The parent rock of the schist was derived from marine sediments, and the deformations and physical and chemical alteration that produced the schist principally occurred during emplacement of the Sierra Nevada Batholith. The granite dikes at the site are probably associated with the intrusion. The rock immediately upstream of the dam is the granodiorite of the batholith.


Table Top Formation, near Temperance Flat

Site Geotechnical Conditions

Schist exposed in the river channel immediately downstream of the dam is fresh while weathering is progressively more intense on the valley slopes, ultimately forming an intensely weathered zone 40 to 50 feet thick at the elevation of the dam crest. From descriptions provided on logs of boring during foundation excavation, rock at or below the final foundation surface is moderately to slightly weathered. Due to the weathering profile of the near-surface bedrock, the dam design incorporated a floating target for the final foundation surface. An average thickness of 32 feet of material was removed to achieve a satisfactory foundation. However, the depth of excavation in the area of a fault zone on the left abutment locally approached 70 feet.

Foliation (schistosity) is pervasive at the site and is the primary structural feature of the schist. The attitude of this foliation varies locally, but is fairly uniform within the area of the dam foundation, striking N65 - 75°W (subparallel to the dam axis) and dipping 55° to 85° SW (downstream). The schist readily cleaves along foliation in more weathered intervals of rock. However, this tendency is proportional to the degree of weathering and is absent in fresh rock. Although minor shearing is widespread at the site, only a few faults are specifically documented in the record of construction. Construction drawings and written records of the work indicate discrete sets of joints, "flat seams," steeply dipping faults, and other discontinuities occur within the area of the dam foundation.

A number of trenches and shafts were excavated during dam construction along flat seams and portions of faults. These excavations were backfilled with concrete. An extensive and effective foundation grouting program was performed during construction.

No known adverse geologic/geotechnical conditions exist at the site that would require special consideration for design and/or construction. The foundation bedrock is considered competent for the existing dam, any of the measures for raising the dam, appurtenant structures, and the embankment saddle dam.

Seismic Hazard Analysis

Overall, potential seismic hazard at the site is low. No known through-going faults have been identified in the vicinity of Friant Dam and Millerton Lake. While minor shearing is widespread within the dam site, only a few faults are specifically documented in the record of construction. None are considered active. Probabilistic seismic hazard analysis conducted for the Investigation calculated the PHA for select return periods from known fault and background areal sources. For return periods of 2,500, 5,000 and 10,00 years, the mean peak horizontal acceleration (PHA) near Friant and Temperance Flat was determined to be 0.13, 0.17, and 0.23 gravitational acceleration units(g), respectively. Areal sources were found to be the controlling source of potential earthquakes for these and greater return periods (Reclamation 2002b).

Existing Facilities

Constructed facilities in the Millerton Lake area include Friant Dam, residences, the former Fresno County Courthouse, recreation facilities within the Millerton Lake State Recreation Area (SRA) and San Joaquin River Gorge management area, roads, and Pacific Gas and Electric's (PG&E) Kerckhoff Powerhouses. **Table 2-1** lists facilities in the area upstream of Friant Dam by elevation. Roads, varying considerably in elevation and location, are excluded from the table. Facilities that could be inundated by a 140 - foot raise of Friant Dam, including roads, are depicted in **Figure 2-1**.

TABLE 2-1.
MILLERTON LAKE AREA FACILITIES ABOVE FRIANT DAM

Approximate Elevation (feet above msl)	Approximate Location (SJ River Mile)	Facility
569	269	Boat Ramps Nos. 2-5
585	268	Boat Ramp No. 1
585	269	South Bar Picnic Area
585 – 765	267.5 - 268.5	Lakeview Estates Residential Development (western portion)
585 – 971	271 - 273	Sky Harbor Residential Development
585 – 883	272.5 - 273	Hidden View Residential Development
580	273	Fine Gold Day Use Area
580 – 600	269 - 270	SRA Blue Oak Trail
580 – 650	269 - 270	SRA North Shore Trail
580 – 1,240	273 - 284	San Joaquin River Trail (SRA to SJR Gorge portion)
582	270	Boat Ramp No. 6
585	269.5	Millerton Marina
589	281	Temperance Flat Boat-In Campground
590	268	Rocky Point Campground
590	268.5	North Shore Area Park Entrance
590	269	Dumna Strand Campground
592 – 705	280.5 - 281	Temperance Flat Residences
594 – 640	269 - 269.5	Lakeview Estates Residential Development (eastern portion)
597 – 705	270	Winchell Bay Residential Area
600	268	Former Fresno County Courthouse
600	268	Picnic Facilities at South Shore of Millerton Lake
600	268	South Shore Area Park Entrance
600	269	Fort Miller Campground
600	270	Valley Oak Campground
600	277	On-Boat Camping
605	282.5	Kerckhoff Powerhouse No. 2 Access Tunnel Entrance
620	269	Rocky Point Campground
620	269	Group Campground
630	269	Mono Campground
630	281	Toilet Facility
640	RM1, Fine Gold Ck	N. Finegold On-Boat Camping Area
650	280	Hewitt Valley Environmental Camps
650 – 1,088	269.5 – 270	SRA Buzzards Roost Trail
675	284.5	Kerckhoff Powerhouse Main Floor
680	284.5	BLM Footbridge
680 – 2,120	283.5 - 284.5	San Joaquin River Trail (SJR Gorge portion)
778	283	Substation for Kerckhoff Powerhouse No. 2
889	292.5	Base of Kerckhoff Dam
921	284.5	Surge Chamber for Kerckhoff Powerhouse
960	285	BLM Native American Village (reproduction)
971	292.5	Kerckhoff Dam Crest
1,030	284	BLM Primitive Campground

Key:
BLM – Bureau of Land Management
CK – creek
msl – mean sea level
RM – river mile
SJR – San Joaquin River
SRA – State Recreation Area

Friant Dam and Millerton Lake


Friant Dam

Friant Dam, owned and operated by Reclamation, was constructed between 1939 and 1942. It is a concrete gravity dam that impounds Millerton Lake on the San Joaquin River. Two saddle dams, that close low areas along the reservoir rim, are located on the southern side of the reservoir and serve as embankments for Millerton Road. The reservoir, with a gross storage capacity of 520.5 TAF at elevation 578, is operated for water supply and flood control. Water deliveries, principally for irrigation, are made through outlet works to the Friant Kern and Madera canals. Physical data pertaining to Friant Dam and Millerton Lake are presented in **Table 2-2.**

The spillway consists of an ogee overflow section, chute, and stilling basin at the center of the dam. The spillway is controlled by one 18-foot-high by 100-foot-wide drum gate, and two comparably sized Obermeyer gates. The spillway crest is at elevation 560, and the top of the gates, when closed, is at elevation 578. The flood pool elevation is 585 while the maximum observed water surface elevation was 580, experienced during the January 1997 flood.

Outlets to the Madera Canal are located on the right abutment; outlets to the Friant-Kern Canal are located on the left abutment. A river outlet works is located to the left of the spillway within the lower portion of the dam.

Three powerhouses, owned and operated by the Friant Power Authority, are located on the downstream side of Friant Dam. A powerhouse on each canal generates hydroelectricity as water is released to the Friant-Kern and Madera canals for delivery. A third powerhouse located at the base of the dam adjacent to the spillway generates hydroelectricity as water is released to the San Joaquin River. The combined capacity of the three powerhouses is slightly over 30 megawatts (MW) and between 1986 and 2003 they generated an average of nearly 80 gigawatt-hours per year (GWh/yr). The Friant Kern Canal powerhouse accounts for 60 percent of the capacity and generation, followed in relative contribution by the Madera Canal powerhouse, then the river outlet powerhouse. Additional details can be found in the **Hydropower TA**.

Residential Developments

Several residential areas around Millerton Lake have been established. Three residential developments are located in Fresno County (Lakeview Estates, Winchell Bay, and Sky Harbor); one major development, Hidden View Estates, is located in Madera County. Each of these residential areas includes developed and undeveloped parcels. Other residential sites include two homes in the Temperance Flat area.

TABLE 2-2.
PERTINENT PHYSICAL DATA – FRIANT DAM AND MILLERTON LAKE

General			
Drainage Areas		**Unimpaired Flows of Friant Dam**	
Friant Dam	1,638 square miles	Mean annual runoff (1873-1977)	1,790,300 acre-feet
Mono Creek at Lake Thomas A. Edison	95.2 square miles	Average flow	2,470 cfs
South Fork San Joaquin River at Florence Lake	171 square miles	Min mean daily inflow (10 Oct 1977)	0 cfs
		Max mean daily inflow (23 Dec 1955)	61,700 cfs
Big Creek at Huntington Lake	80.5 square miles	Max instantaneous inflow (23 Dec 1955)	97,000 cfs
North Fork Willow Creek at Bass Lake	50.4 square miles		
Stevenson Creek at Shaver Lake	29.1 square miles	Max mean daily outflow (6 Jun 1969)	12,400 cfs
San Joaquin River at Mammoth Pool Reservoir	1,003 square miles	Min mean daily outflow (20 Oct 1940)	5.5 cfs
San Joaquin River at Redinger Lake	1,295 square miles	**Spillway design flood**	
San Joaquin River at Kerckhoff Diversion	1,461 square miles	Peak inflow	197,000 cfs
San Joaquin River at Mendota	3,943 square miles	Peak outflow	158,500 cfs
Friant Dam and Millerton Lake[1]			
Friant Dam (concrete gravity)		**Millerton Lake**	
Elevation, top of parapet	585.0 feet above msl	**Elevations**	
Freeboard above spillway flood pool	3.25 feet	Minimum operating level[2]	466.1 feet above msl
Elevation, crown of roadway	581.25 feet above msl	Gross pool	578.0 feet above msl
Max height, foundation to crown of roadway	319 feet	Spillway flood pool	585.0 feet above msl
Length of crest		**Area**	
Left abutment, non-overflow section	1,478 feet	Minimum operating level	2,100 acres
Overflow river section	332 feet	Gross pool	4,850 acres
Right abutment, non-overflow section	1,678 feet	Spillway flood pool	5,085 acres
Total length	3,488 feet	**Storage capacity**	
Width of crest at elevation 581.25	20.0 feet	Minimum operating level[2]	130,000 acre-feet
Total concrete in dam and appurtenances	2,135,000 yd^3	Gross pool	520,500 acre-feet
		Spillway flood pool	555,450 acre-feet
Spillway (gated ogee)		**Friant-Kern Canal**	
Crest length		Length	152 miles
Gross	332 feet	Operating capacity below Friant Dam	4,000 cfs
Net	300 feet	Operating capacity at terminus of canal	2,000 cfs
Crest elevation	560 feet above msl		
Discharge capacity (height = 18.0 feet)	83,160 cfs	**Madera Canal**	
Crest gates (1 drum and 2 Obermeyer)		Length	35.9 miles
Number and size	3 @ 100 feet by 18 feet	Capacity below Friant Dam	1,000 cfs
Top elevation when lowered	560 feet above msl	Capacity at Chowchilla River	625 cfs
Top elevation when raised	578 feet above msl		
Outlets			
River outlets (110-inch dia. w/ 96-inch hollow jet valves)			
Number and elevation	4 @ 380 feet above msl		
Capacity at minimum pool	4,000 cfs		
Capacity at gross pool	12,300 cfs		
Diversion outlets, Madera (91-inch dia. w/ 86-inch needle valve)			
Number and elevation	2 @ 446 feet above msl		
Capacity at minimum pool	1,000 cfs		
Capacity at gross pool	4,600 cfs		

Key:
cfs – cubic feet per second
hp – horsepower
kW – kilowatt
msl – mean sea level
yd^3 – cubic yard

Notes:
[1] Elevations given are in vertical datum NGVD 1929.
[2] Minimum operating level generally corresponds with elevation of Madera Canal outlets. Minimum storage for Friant-Kern Canal diversion is about 135 TAF.

Former Fresno County Courthouse

The former Fresno County Courthouse was removed at the time of Friant Dam's construction from an area now within Millerton Lake. The reconstructed stone and brick building now overlooks the lake from a site on the south side of the reservoir.

Recreation Facilities

The Millerton Lake SRA, managed by the California Department of Parks and Recreation, contains numerous recreation facilities on both the north and south sides of the reservoir. These include 10 camping areas, 6 boat ramps, a privately operated marina, 3 picnic and day-use areas, 4 trails, and parking, telephone, and toilet facilities.

The San Joaquin River Gorge, a management area administered by the Bureau of Land Management (BLM), is situated upstream of the SRA. It contains a replicated Native American village, trails, a footbridge across the San Joaquin River, and a primitive campground. The most prominent trail is the San Joaquin River Trail, which connects the South Finegold picnic area in the SRA to the BLM primitive campground off Smalley Road, then crosses the footbridge and climbs the terrain north of the river.

Roads

Several roads in the Millerton Lake area provide access to residential areas and recreation facilities. Millerton Road skirts the south side of the reservoir, connecting the community of Friant with Auberry Road. Winchell Cove Road and Sky Harbor Road extend from Millerton Road north into residential areas. Madera County Roads 206 and 145 on the north side of the lake lead to recreational facilities in the SRA. County Road 216 provides access from north of Millerton Lake into the Hidden View residential area near the confluence of Fine Gold Creek and Millerton Lake.

Kerckhoff Powerhouses and Intakes

Two PG&E powerhouses, Kerckhoff Powerhouse and Kerckhoff No. 2 Powerhouse, are located within a mile of the upstream end of Millerton Lake. Water feeding the powerhouses is diverted from Kerckhoff Lake at Kerckhoff Dam and conveyed through tunnels and penstocks.

Kerckhoff Powerhouse was commissioned in 1920 and is located on the San Joaquin River about a mile upstream of the upper reach of Millerton Lake. The powerhouse has a generation capacity of 38 MW and generated an average of nearly 50 GWh/yr from 1994 through 2002.

Kerckhoff No. 2 Powerhouse is a relatively modern facility, commissioned in 1983 with a capacity of 155 MW. It discharges directly to Millerton Lake and generated more than 500 GWh/yr, on average, from 1994 through 2002. Additional details regarding these powerhouses are provided in the **Hydropower TA**.

POTENTIAL IMPROVEMENTS

Raising the existing concrete gravity dam could be accomplished by placing an overlay on the downstream face of the dam and extending the top of the dam vertically, with either conventional mass concrete or RCC. Potential raises of 25 feet, 60 feet, and 140 feet have been examined. **Figure 2-1** illustrates the largest potential enlargement measure, the 140-foot dam raise. Intermediate raise heights would be possible.

An embankment or RCC saddle dam would be required along the left side of the reservoir. The modified dam would use as much of the existing facilities as possible. Two Obermeyer gates would be removed and reinstalled, but the current drum gate would be replaced with a third Obermeyer gate. Modifications to the existing canal outlet works, river outlet works, and powerhouse would be necessary. It is assumed for purposes of current cost estimates that modified structures would be designed to be similar to their current configurations.

Reservoir Storage and Area

Reservoir area and total capacity data for Millerton Lake associated with raising Friant Dam are summarized in **Table 2-3**. Curves relating elevation to additional storage volume added to Millerton Lake and total reservoir area are shown in **Figure 2-2**, which extends the existing elevation relationships of Millerton Lake up to a 140-foot raise.

TABLE 2-3.
MILLERTON LAKE RESERVOIR AREA AND WATER STORAGE CAPACITY

Measure	Reservoir Area (acres)[1]	Gross Storage Capacity (TAF)	New Storage Capacity (TAF)
Existing	4,963	520	0
25-foot raise	5,667	650	130
60-foot raise	6,384	860	340
140-foot raise	8,119	1,440	920

Key:
DEM – digital elevation model
GIS – geographic information system
TAF – thousand acre-feet
USGS – United States Geological Survey

Note:
[1] Based on GIS data from USGS 10m DEM.

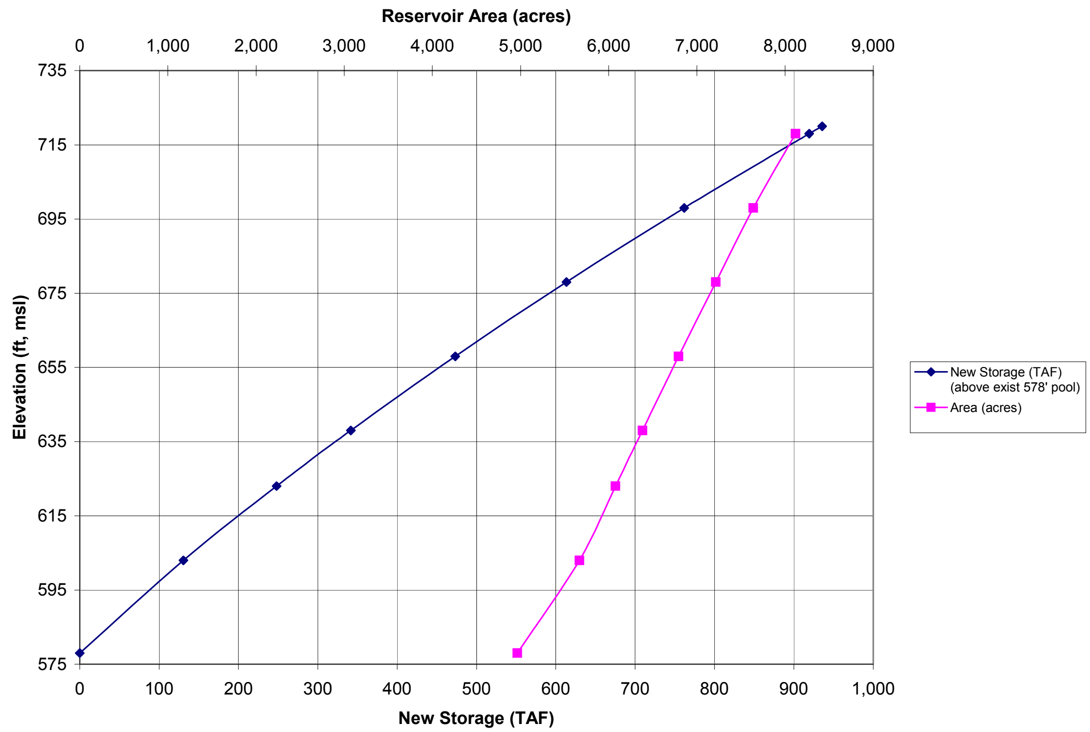

FIGURE 2-2.
MILLERTON LAKE SURFACE ELEVATION VS.
NEW STORAGE AND AREA

Modifications to Friant Dam

The concrete dam would be raised by providing an overlay on the downstream face of the existing dam and extending the top of the dam vertically to the new crest elevation. For purposes of cost estimates, it was assumed that RCC would be used; however, more detailed study would be required to verify this assumption before final designs could be developed. A cross section illustrating typical details for a 25-foot dam raise is shown in **Figure 2-3**.

Facing elements would be included along both the upstream and downstream faces of the overlay. Facing elements would be constructed of conventional concrete to provide a more durable surface on the exposed faces of the dam. A conventional concrete cap would be included along the dam crest, and the spillway crest, guide walls, and stilling basin would be constructed using conventional concrete.

The potential for reactive aggregate was a concern to engineers during construction of Friant Dam. Chemical activity between high-alkali cement and certain components of some concrete aggregate, such as chert, resulted in expansion within the concrete and subsequent cracking. Low-alkali Portland-type cement with a pumicite pozzolan additive was used for most of the dam concrete, but in the early stages of construction, some high-alkali cement was used. Deterioration of the spillway concrete due to alkali aggregate reaction is the most serious problem identified for the existing dam. This issue and its effects on the dam raise will be addressed in future design efforts.

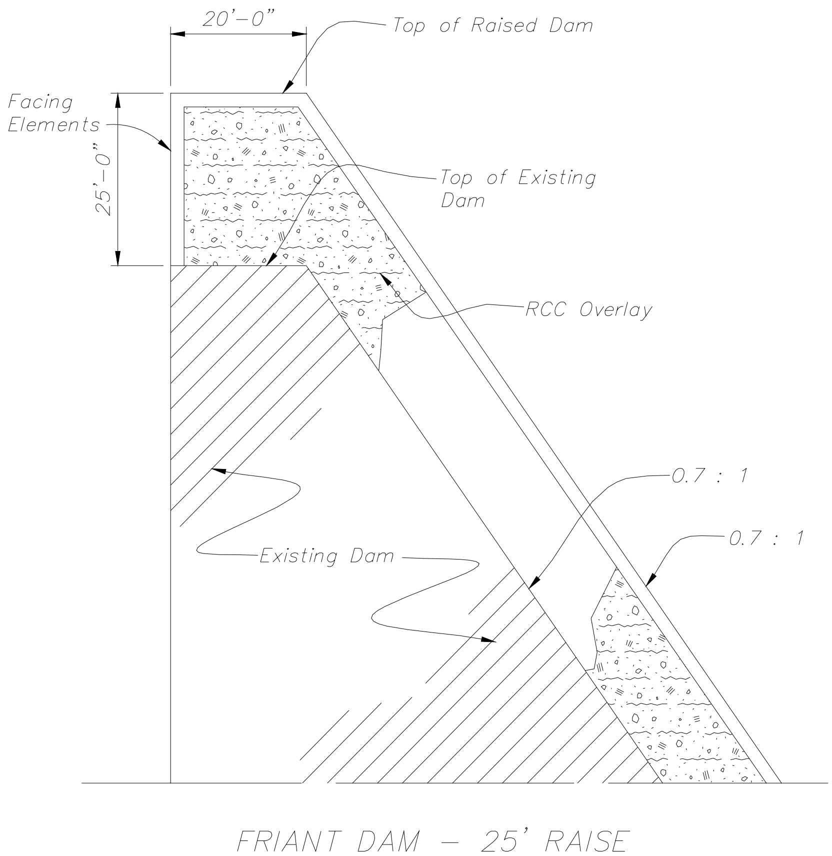

FIGURE 2-3.
FRIANT DAM RAISE SIMPLIFIED CROSS SECTION

Estimates for raising Friant Dam assume that the existing spillway concrete would be completely replaced (Reclamation, 1997). The spillway for the raised structure would be similar to the existing spillway. Two Obermeyer gates would be removed and reinstalled at the top of the raised dam, and the remaining drum gate would be replaced with a third Obermeyer gate.

The existing dam can accommodate about 30 percent of the current probable maximum flood (PMF) volume before overtopping occurs. An abbreviated assessment of overtopping parameters resulted in the determination that a breach of the concrete dam was not likely, and that overtopping and breaching of the existing saddle dams would not result in loss of life. Current studies do not include provisions for increasing spillway capacity. However, if this measure is carried forward, the opportunity or need for providing additional spillway capacity should be considered.

Appurtenant Features

This section describes major appurtenant features that would be associated with enlarging Friant Dam and Millerton Lake.

Saddle Dam

An embankment or RCC saddle dam would be necessary to close off a low area along the left side of the reservoir. The height and length of the saddle dam would depend on the scale of the corresponding raise of Friant Dam. For the 25-foot raise measure, the saddle dam would have a maximum height of 30 feet and crest length of 4,500 feet; a 60-foot raise would require approximately 8,500 linear feet of new saddle dams; and a 140-foot raise would require new saddle dams approximately 9,500 feet in total length, exceeding 100 feet high in some locations.

This appendix assumes that the saddle dam would be a zoned earthfill dam consisting of a central core, filters, drainage elements, slope protection, and riprap.

Outlet Works

Outlet works to the Madera Canal, Friant-Kern Canal, and San Joaquin River would require modification. The current cost estimate provides for extending outlet pipes and for furnishing and installing gates and outlet valves. It is assumed that the redesigned structures would be similar to the existing ones. However, it is possible that higher heads or flows, or any change in reservoir evacuation requirements, might require resizing or a different design of outlet works, canal gates, and valves.

Powerhouses

Existing powerhouses at the base of Friant Dam are owned and operated by the Friant Power Authority. Cost estimate worksheets prepared for the dam modification, presented in **Attachment C2,** assumed that major changes to the existing powerhouses would not be necessary to create additional storage. Approximate costs to increase power-generating capability at Friant Dam, as described in the **Hydropower TA**, assume that replacing the power generating units could increase the generating capacity. Modified or relocated powerhouses would be reconnected to the existing overhead power lines and power grid.

Construction Considerations

The following paragraphs discuss issues of concern related to constructing the dam and reservoir.

Foundations

Foundations for any of the Raise Friant Dam measures would be in sound granitic rock, as described in the geology section. No special foundation considerations are known for the site at this time. Foundation preparation would be typical for each measure.

Reservoir Management and Flood Routing During Construction

Detailed studies were not performed to evaluate diversion during construction. However, the existing spillway has a capacity of about 83,000 cfs, and the outlet works has a capacity of about 17,000 cfs. A 25-year diversion flood has a peak discharge capacity of about 65,000 cfs. For purposes of this study, it was assumed that the existing spillways would remain operational until the last 25 feet of dam raise. The final raise would be scheduled during the normally low-flow periods of the year, and when the modified outlet works would be available. These assumptions would need to be verified during final designs.

Borrow Sources and Materials

Concrete aggregate could be processed from bedrock outcrops in the reservoir area. This same processed material would provide filter material and riprap for the saddle dam. Additional concrete aggregate and other processed materials may be available in limited quantities from alluvial deposits along the San Joaquin River floodplain downstream from the existing dam. Haul distances would be 2 to 3 miles to the dam and 2 miles or less to the saddle dam.

Pervious and semipervious materials could be obtained near the saddle dam by processing the granite colluvial soil to remove oversize material. Impervious material for the core of the saddle dam can be found on agricultural land within 1 to 2 miles of the saddle dam. Because the fine-grained deposits are approximately 4 feet thick, a large surface area would be required to develop an adequate quantity of material. Processed sands and gravels could be supplied by commercial sources and/or crushing and processing quarried rock in the reservoir area.

Construction Site Access

Principal access to the site for construction is via paved roads. Additional roads for construction site access and haul roads from local borrow areas would be required. Rights-of-way for constructing the dam raise and appurtenant facilities would be on or through existing Reclamation and/or other public lands.

Staging Areas

Areas for temporarily laying down construction materials and equipment, and for staging construction likely would be located downstream of the dam on existing Reclamation property.

Lands and Rights-of-Way

All private property subject to inundation would be acquired. A 25-foot raise would begin to inundate portions of four residential developments at Millerton Lake and residences at Temperance Flat. All occupied property in the portion of Lakeview Estates east of the SRA boat launch area would be inundated by a 60-foot raise. A 140-foot raise also would completely inundate the developed portion of Winchell Bay. **Table 2-4** shows the total area that would be inundated by the Raise Friant Dam measures considered and the number of structures that would be affected. Rights-of-way or easements that may be needed to construct new facilities or relocate existing facilities have not been determined.

TABLE 2-4.
RAISE FRIANT DAM RESERVOIR AREA LAND REQUIREMENTS

Description	Raise Friant Dam Measures		
Raise height (feet)	25	60	140
Gross pool elevation (feet above msl)	603	638	718
New storage capacity (TAF)	130	340	920
Estimated inundated area (acres)	704	1,421	3,156
Number of affected structures	58	109	165

Key:
msl – mean sea level TAF – thousand acre-feet

Electric Power Sources

Electric power service at 110 and 220 volts is available at Friant Dam.

Modifications and Relocations of Affected Facilities

Existing facilities that could be inundated by a raise of Friant Dam are indicated in **Figure 2-1.** **Table 2-5** lists upstream facilities that would require relocation *(*i.e., construction at a new location) or abandonment for the three dam raise increments considered. Requirements for relocations of roads and Kerckhoff Hydroelectric Project facilities are described below. Specific requirements for relocating or abandoning recreational facilities have not been determined.

Millerton Area Roads

A 140-foot raise of Friant Dam would inundate virtually all of Winchell Cove Road and substantial segments of Sky Harbor Road and Madera County Road 145. Since these stretches of road do not provide through access, they would be abandoned. Smaller residential roads that spur off of Millerton Road and Road 145 also would be abandoned. A segment of Millerton Road would need to be re-routed for about 2 miles. In all, a 140-foot raise of Friant Dam would inundate slightly under 14 miles of unpaved road and more than 5 miles of paved road, with a smaller raise inundating a lesser amount of roadway. Technical requirements for relocating or abandoning these road segments have not been determined. It is assumed that a similar approach would be used as that described in **Chapters 4 and 5** for relocating Powerhouse Road for the Temperance Flat Reservoir surface water storage measures.

TABLE 2-5.
RELOCATIONS REQUIRED FOR RAISE FRIANT DAM MEASURES

<table>
<tr><th>Features Requiring Relocation or Abandonment</th><th colspan="3">Raise Friant Dam Measures</th></tr>
<tr><td>6 Boat Ramps</td><td rowspan="12">25-foot Friant Raise</td><td rowspan="17">60-foot Friant Raise</td><td rowspan="22">140-foot Friant Raise</td></tr>
<tr><td>3 Picnic or Day Use Areas</td></tr>
<tr><td>Winchell Bay Marina</td></tr>
<tr><td>5 Campgrounds</td></tr>
<tr><td>On-Boat Camping Facility</td></tr>
<tr><td>Blue Oak Trail</td></tr>
<tr><td>Portions of 2 Additional Trails</td></tr>
<tr><td>Portions of Roads</td></tr>
<tr><td>2 SRA Park Entrances</td></tr>
<tr><td>Portions of 4 Residential Developments at Millerton Lake</td></tr>
<tr><td>Portions of Residential Property at Temperance Flat</td></tr>
<tr><td>Historic Courthouse</td></tr>
<tr><td colspan="2">3 Campgrounds</td></tr>
<tr><td colspan="2">Kerckhoff PH No. 2 Access Tunnel Entrance</td></tr>
<tr><td colspan="2">Kerckhoff No. 2 Powerhouse Intake</td></tr>
<tr><td colspan="2">Toilet Facility at Temperance Flat</td></tr>
<tr><td colspan="2">North Finegold On-Boat Camping Facility</td></tr>
<tr><td colspan="3">Hewitt Valley Environmental Camps</td></tr>
<tr><td colspan="3">Buzzards Roost Trail</td></tr>
<tr><td colspan="3">Kerckhoff Powerhouse</td></tr>
<tr><td colspan="3">BLM Footbridge</td></tr>
<tr><td colspan="3">Kerckhoff Powerhouse Intake</td></tr>
</table>

Kerckhoff Project

A 25-foot raise could be accomplished without inundating Kerckhoff No. 2 Powerhouse, although structural protection would be required for the access tunnel to the powerhouse. It is assumed that a concrete wall would be constructed. It appears impractical to consider providing protection from inundation to the facility for any Friant Dam raise substantially greater than 25 feet. A 60-foot or 140-foot raise would completely inundate Kerckhoff No. 2 Powerhouse, requiring abandonment and possible relocation. Requirements for abandoning the existing Kerckhoff No. 2 Powerhouse are described in **Chapter 3**. Specific technical requirements for constructing a relocated Kerckhoff No. 2 Powerhouse at a higher elevation have not been determined. However, analysis of hydropower generation potential documented in the **Hydropower TA** suggests that sufficient remaining head would exist with a 60-foot Friant raise to support construction of a replacement powerhouse at a higher elevation with two 45 MW generating units. A 140-foot raise could support a powerhouse with two 20 MW units. It is assumed that the new powerhouse would use the existing Kerckhoff No. 2 Powerhouse diversion tunnel, but branch off at a higher elevation and discharge to the enlarged Millerton Lake.

Kerckhoff Powerhouse, with the main floor at an elevation of 675 feet, discharges to the San Joaquin River about 1 mile above the uppermost end of Millerton Lake. A 25-foot or 60-foot raise of Friant Dam would not affect the tailwater of Kerckhoff Powerhouse. A 140-foot raise of Friant Dam would completely inundate Kerckhoff Powerhouse. Decommissioning and abandonment requirements are described in **Chapter 3**. Specific construction requirements for relocating Kerckhoff Project generating capacity to a higher elevation have not been determined; however, it appears that raising the elevation of Millerton Lake would leave sufficient generating potential to support relocation as described above for the Kerckhoff No. 2 Powerhouse.

Abandoning the Kerckhoff No. 2 Powerhouse or Kerckhoff Powerhouse also would require abandoning the diversion intake(s) at Kerckhoff Lake. Details are provided in **Chapter 3**.

Construction Costs

Table 2-6 summarizes construction costs for Raise Friant Dam measures. Costs shown for individual project components include field costs and indirect costs of 25 percent of field costs for planning, investigations, designs, and construction management. Acquisition costs for lands in the reservoir area are incorporated into the construction cost of the dam overlay and saddle dams, along with an allowance of 20 percent of lands costs for indirect costs associated with property acquisition in the expanded reservoir area.

Construction costs for individual project components are based on worksheets presented in **Attachments C1** and **C2**. Field costs for the main dam modification and saddle dams, previously calculated at July 2003 price levels, were adjusted to reflect July 2004 prices and a higher contingency allowance. Costs for abandonments were developed in July 2004. All costs listed in **Table 2-6** are therefore at July 2004 price levels.

Costs for abandoning or relocating Kerckhoff Project hydroelectric facilities are addressed as follows. For a 25-foot raise, a placeholder cost was inserted for a two to three foot concrete wall that would be constructed to protect the Kerckhoff No. 2 Powerhouse access tunnel entrance from possible inundation during floods. For the 25-foot raise, both the original Kerckhoff Powerhouse and Kerckhoff No. 2 Powerhouse would continue to operate, as would their respective diversion tunnel intakes at Kerckhoff Dam.

For a 60-foot raise, the Kerckhoff No. 2 Powerhouse would be abandoned in place and inundated by the enlarged Millerton Lake. At Kerckhoff Powerhouse, a replacement powerhouse with two 45 MW generating units would be constructed at a higher elevation to discharge to the enlarged Millerton Lake. It is assumed that the new powerhouse would make use of the existing Kerckhoff No. 2 Powerhouse diversion intake and tunnel. The approximate cost to construct the replacement powerhouse was derived from the estimate prepared for a new powerhouse at Millerton Lake containing four 45 MW units, associated with an elevation 1,300 Temperance Flat Reservoir at RM 286, discussed in **Chapter 5**. The cost estimate prepared for the Temperance Flat RM 286 elevation 1,300 measure was adjusted to account for the lower generating capacity associated with raising the level of Millerton Lake 60 feet. The square root of the generating capacity ratios was used as the scaling factor. Costs for construction of a new surge chamber are included in the estimate.

TABLE 2-6.
CONSTRUCTION COSTS FOR RAISE FRIANT DAM MEASURES
($ MILLION)

Dam Raise Height (feet)	25	60	140
New Storage Capacity (TAF)	**130**	**340**	**920**
Storage Components			
RCC Overlay, Saddle Dams, Reservoir Lands	170	390	970
Concrete Wall to protect Kerckhoff No. 2 Powerhouse Access	2	-	-
Abandon Kerckhoff No. 2 Powerhouse	-	2	2
Abandon and Restore Kerckhoff Powerhouse	-	-	4
Abandon Intake for Kerckhoff Powerhouse	-	-	1
Millerton Road Relocation	28	28	28
Construction Cost, Storage Components	**200**	**420**	**1,005**
Replacement Power Components			
Additional Generation Capacity at Friant Dam (5, 13 or 30 MW)[1]	18	49	115
New Kerckhoff No. 2 Powerhouse (40 to 90 MW)	-	130	88
Construction Cost, Replacement Power Components	**18**	**179**	**203**
Construction Cost[2, 3]	**218**	**599**	**1,208**

Key:
RCC – roller-compacted concrete
TAF – thousand-acre feet
Notes:
[1] Additional generation capacity provided by replacing one or more existing units with new larger units.
[2] All cost estimates are preliminary. Construction cost represents the sum of field costs and indirect costs for planning, engineering, design and construction management, estimated at 25 percent of field costs.
[3] Costs do not include environmental mitigation, new or relocated recreation facilities, acquisition of impacted power facilities, or compensation for lost future power generation.

For the 140-foot Friant Dam raise, both Kerckhoff Powerhouse and Kerckhoff No. 2 Powerhouse would be inundated and abandoned. Kerckhoff No. 2 Powerhouse would be abandoned in place and Kerckhoff Powerhouse would be abandoned and restored. A new powerhouse would be constructed to discharge to the enlarged Millerton Lake. There would be sufficient head to justify two 45 MW generating units. Again, it is assumed that the new powerhouse would use the existing Kerckhoff No. 2 diversion tunnel and intake. However, the intake and diversion tunnel to Kerckhoff Powerhouse would be abandoned. The cost of the replacement powerhouse was approximated using the same method as described in the preceding paragraph.

Approximate costs for upgrading power generation units at Friant Dam to greater capacity are included in **Table 2-7**, as are approximate costs for relocating Millerton Road. Additional study would be needed to determine the costs to relocate recreation facilities and to carry out any required environmental mitigation. Those potential costs are not included in the totals shown.

Lands required for the construction site are public lands held either by Reclamation or other Federal agencies. No acquisition costs have been calculated for these lands since they are already in public ownership. Privately owned lands, including numerous residential sites around Millerton Lake, would need to be acquired for the new reservoir areas associated with a dam raise. An estimate of these costs is included in the construction cost shown for the dam raise (RCC overlay and saddle dams) in **Table 2-7**. However, the costs to acquire any Kerckhoff Project assets that would be abandoned or to compensate their owners for any loss of future energy generation are not included in the table.

Figure 2-4 shows the relationship between new storage capacities that could be developed with a raise of Friant Dam versus construction cost. Costs increase substantially once Kerckhoff No. 2 Powerhouse is inundated and it becomes necessary to replace its generation capacity.

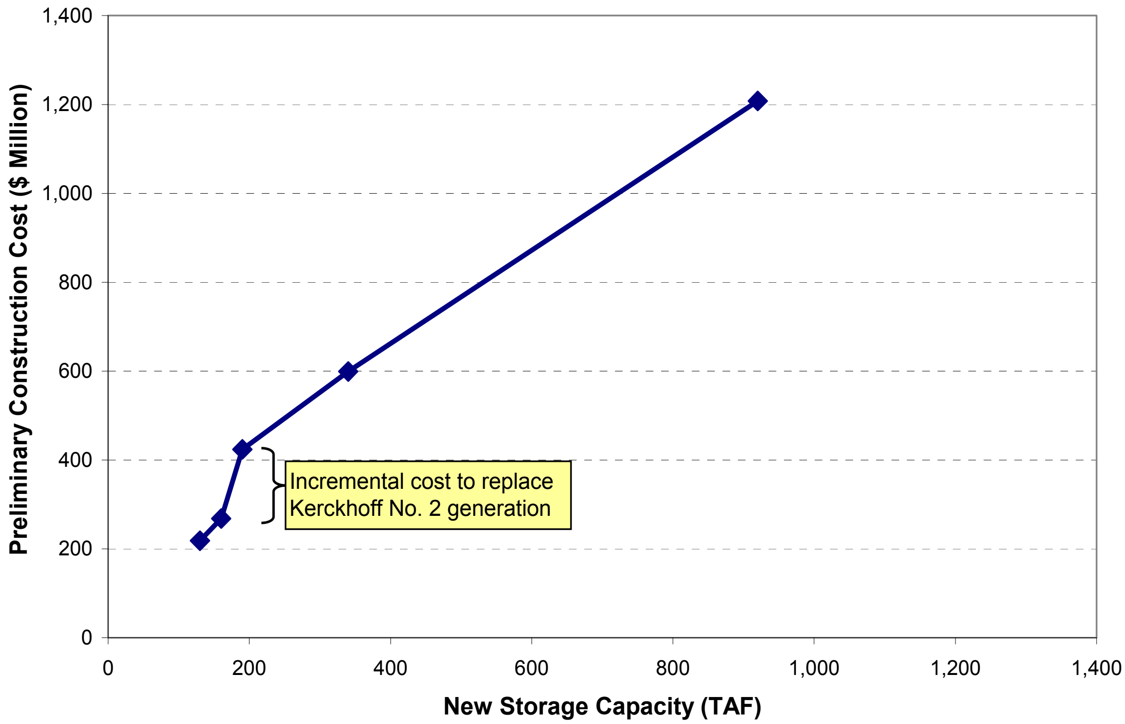

FIGURE 2-4.
CONSTRUCTION COSTS FOR RAISE FRIANT DAM MEASURES VS. NEW STORAGE CAPACITY

CHAPTER 3. TEMPERANCE FLAT RESERVOIR AT RM 274

This chapter describes structures and costs involved in developing a reservoir at RM 274 of the San Joaquin River in the Temperance Flat area. It describes site conditions, engineering considerations associated with design and construction of the dam and appurtenant features, and required relocations or abandonments of existing facilities and land acquisition. Detailed descriptions of existing and potential new hydroelectric generation facilities are included in the **Hydropower TA**.

PREVIOUS STUDIES

In March 1930, Hyde Forbes, an engineering geologist, issued a geological report on three potential dam sites on the San Joaquin River for the California Department of Public Works, Division of Water Resources. The report evaluated geologic conditions at the current location of Friant Dam, at Fort Miller (downstream of the confluence of Fine Gold Creek), and near Temperance Flat at River Mile 274 (just upstream of the confluence of Fine Gold Creek). The study found the bedrock at Friant entirely satisfactory as a foundation for a concrete arch or gravity dam; found the Temperance Flat location to be an excellent site and foundation for a concrete arch type dam; and identified the Fort Miller site as the least desirable of the sites from a geologic perspective (Forbes, 1930).

In planning efforts that preceded construction of Friant Dam, the RM 274 site was considered superior to both the Friant and Fort Miller sites from a water storage perspective. Ultimately, the Friant site was selected because constructing a dam at RM 274 would have required extending delivery canals around or through the area now occupied by Millerton Lake, or constructing a second dam at Friant for diverting water to the canals.

SITE DESCRIPTION

Temperance Flat is a small, bowl-shaped basin in the upper reaches of Millerton Lake, approximately 13 river miles upstream of Friant Dam at about RM 281 of the San Joaquin River. Temperance Flat is located on the border between Madera and Fresno counties, northeast of Auberry Valley and the community of Marshall Station, about 30 miles northeast of Fresno. **Figure 1-1** shows the general location. For purposes of this appendix, the Temperance Flat area refers to the area along the San Joaquin River between the Fine Gold Creek confluence and Kerckhoff Lake, as shown in **Figure 3-1**.

Three potential dam sites are being considered for the Temperance Flat area. A dam at RM 274 or RM 279 would create a reservoir that would inundate Temperance Flat. A dam at RM 286 would store water further upstream. **Figure 3-1** shows the location of these three potential dam sites. The RM 274 dam site would be located in the upper reaches of Millerton Lake, just upstream of the confluence with Fine Gold Creek and about 6 miles downstream of Temperance Flat. **Chapters 4 and 5** discuss the RM 279 and RM 286 sites, respectively.

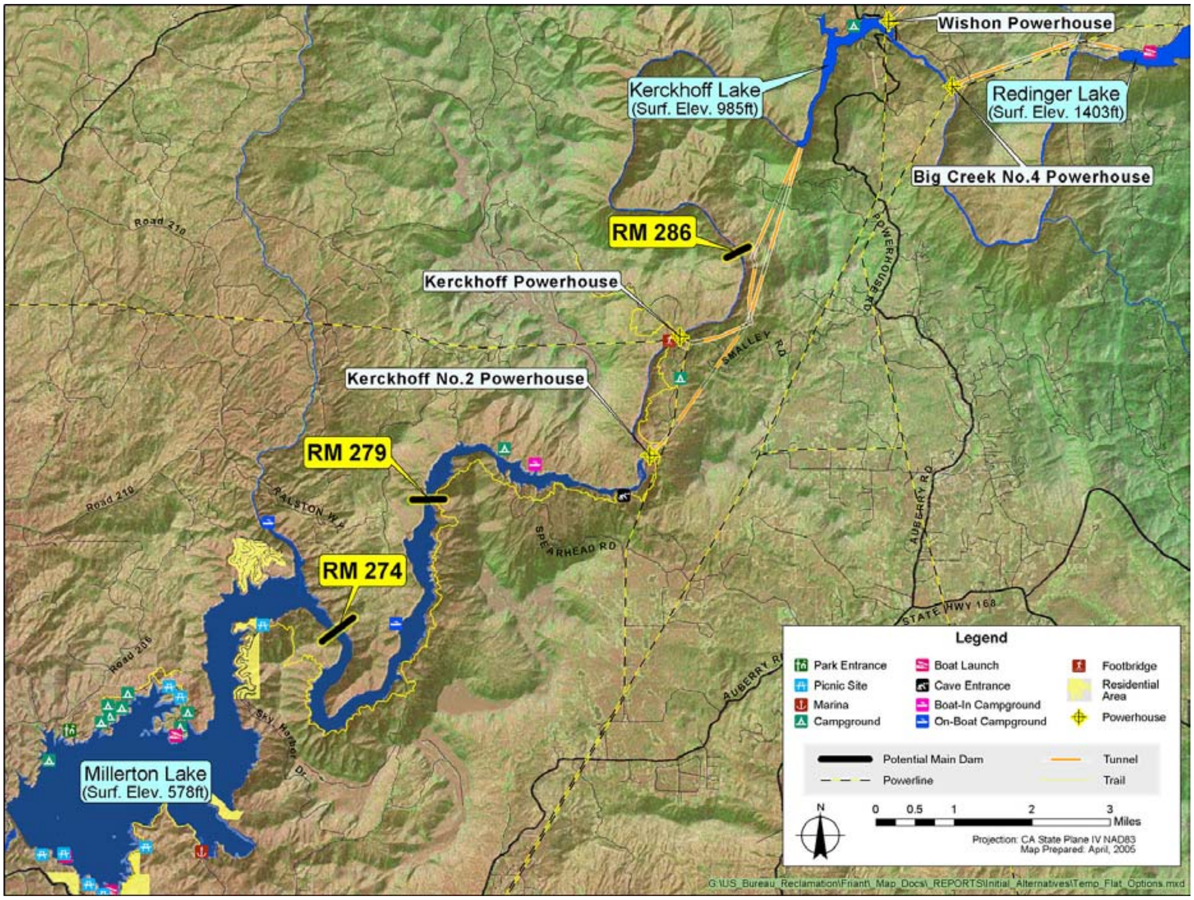

FIGURE 3-1.
POTENTIAL TEMPERANCE FLAT DAM SITES

Topographic Setting

The RM 274 site rises uniformly from elevation 385 in the original San Joaquin River channel. The left abutment rises to elevation 1,582 at Pincushion Mountain and the right abutment rises to elevation 1,473 at an unnamed mountain. A low spot about a mile and a half upstream of the dam site exists along a ridge on the left abutment at elevation 1,120.

Geologic Setting

Both abutments and the channel section are mostly granite and granodiorite, with alluvium in the channel section. The granite is typically hard to very hard where exposed in the bottom of drainages and along the reservoir shoreline. The upper 1 to 10 feet of the granite are intensely weathered to decomposed, and soft to very soft. The granitic bedrock has adequate strength and stability for an embankment, rockfill, concrete gravity, or concrete arch dam structure and for any river diversion feature. The granite also would provide an adequate foundation for a plunge pool or overflow spillway.

Hard, erosion-resistant granite outcrops are scattered on the abutments. Some outcrops are detached blocks of rock up to 25 feet in maximum dimension. A zone of hard, slightly fractured meta-granite or granite gneiss is present near the dam centerline on the left abutment and appears to outcrop in a shallow drainage located upstream of the dam centerline on the right abutment. Alluvium of unknown thickness occurs below the reservoir water surface in the San Joaquin River channel. The alluvium probably ranges from fine-to-coarse grained, with rock blocks up to 25 feet in maximum dimension that detached from the abutment slopes. No unstable wedges, toppling, or slides were observed at the site.

Site Geotechnical Conditions

No known adverse geotechnical conditions that would require special consideration for design and/or construction exist at the site. The foundation bedrock is considered competent for any of the dam types considered and for the potential appurtenant structures. No known faults exist at the RM 274 site or in the immediate vicinity (Reclamation, 2003a).

Seismic Hazard Analysis

Overall, potential seismic hazard potential at the site is low. Areal sources were found to be the controlling source of potential earthquakes for these and greater return periods (Reclamation, 2003b). **Chapter 2** provides mean PHAs calculated for the Temperance Flat area.

Existing Facilities

Constructed facilities in the Temperance Flat area include isolated residences, recreation facilities within the Millerton Lake SRA and the San Joaquin River Gorge management area, roads, and PG&E’s Kerckhoff Project. Located by Kerckhoff Lake are recreational facilities, roads, PG&E’s Wishon Powerhouse, and Southern California Edison’s Big Creek No. 4 Powerhouse. **Table 3-1** lists facilities in the Temperance Flat area, at Kerckhoff Lake, and between Kerckhoff Lake and Redinger Lake. Roads, varying considerably in elevation and location, are excluded from the table. Facilities that could be inundated by a reservoir constructed at RM 274 are indicated in **Figure 3-2.**

Residential Property

The nearest residential development is Sky Harbor, downstream of the RM 274 dam site. Two residences are located in the Temperance Flat area.

Recreation Facilities

Millerton Lake SRA facilities upstream of RM 274 include an on-boat camping facility and the Hewitt Valley Environmental Camps. The San Joaquin River Gorge management area contains a replicated Native American village and a primitive campground. The San Joaquin River Trail follows the southern edge of upper Millerton Lake and the San Joaquin River, connecting the South Fine Gold picnic area in the SRA to the BLM primitive campground off Smalley Road. The trail turns north, crossing the San Joaquin River at a footbridge below Kerckhoff Powerhouse, and climbs the northern side of the San Joaquin River Gorge. Developed recreation facilities at Kerckhoff Lake include a car-top boat launch, day-use area, and campground at Smalley Cove, on the north side of the lake within the Sierra National Forest.

TABLE 3-1.
FACILITIES ABOVE TEMPERANCE FLAT RM 274

Approximate Elevation (feet above msl)	Approximate Location (SJ River Mile)	Facility
580-1,240	273 - 284	San Joaquin River Trail (SRA to SJR Gorge portion)
589	281	SRA Temperance Flat Boat-In Campground
592 - 705	280.5 - 281	Temperance Flat Residences
600	277	SRA On-Boat Camping facility
605	282.5	Kerckhoff Powerhouse No. 2 Access Tunnel Entrance
630	281	SRA New Toilet Facility
650	280	SRA Hewitt Valley Environmental Camps
680	284.5	BLM Footbridge Below Kerckhoff Powerhouse No. 1
675	284.5	Kerckhoff Powerhouse Main Floor
680 – 2,120	283.5 - 284.5	San Joaquin River Trail (SJR Gorge portion)
778	283	Substation for Kerckhoff Powerhouse No. 2
889	292.5	Base of Kerckhoff Dam
900	292.5	Kerckhoff Dam Outlets
921	284.5	Surge Chamber for Kerckhoff Powerhouse
939	301.5	Kerckhoff No. 2 Powerhouse Intake
942	301.5	Kerckhoff Powerhouse Intake
960	285	Native American Village (reproduction)
971	292.5	Kerckhoff Dam Crest
980	295	Powerhouse Road Bridge
980	295.5	Smalley Cove Campground, Picnic Area, Boat Launch
990	294.5	A.G. Wishon Powerhouse
993	283	Surge Chamber for Kerckhoff Powerhouse No. 2
993	296	Big Creek Powerhouse #4
1,030	284	BLM Primitive Campground
1,089	296	Substation for Big Creek Powerhouse No. 4
1,181	301.5	Base of Redinger Dam

Key:
BLM – Bureau of Land Management
msl – mean sea level
SJ – San Joaquin
SJR – San Joaquin River
SRA – State Recreation Area

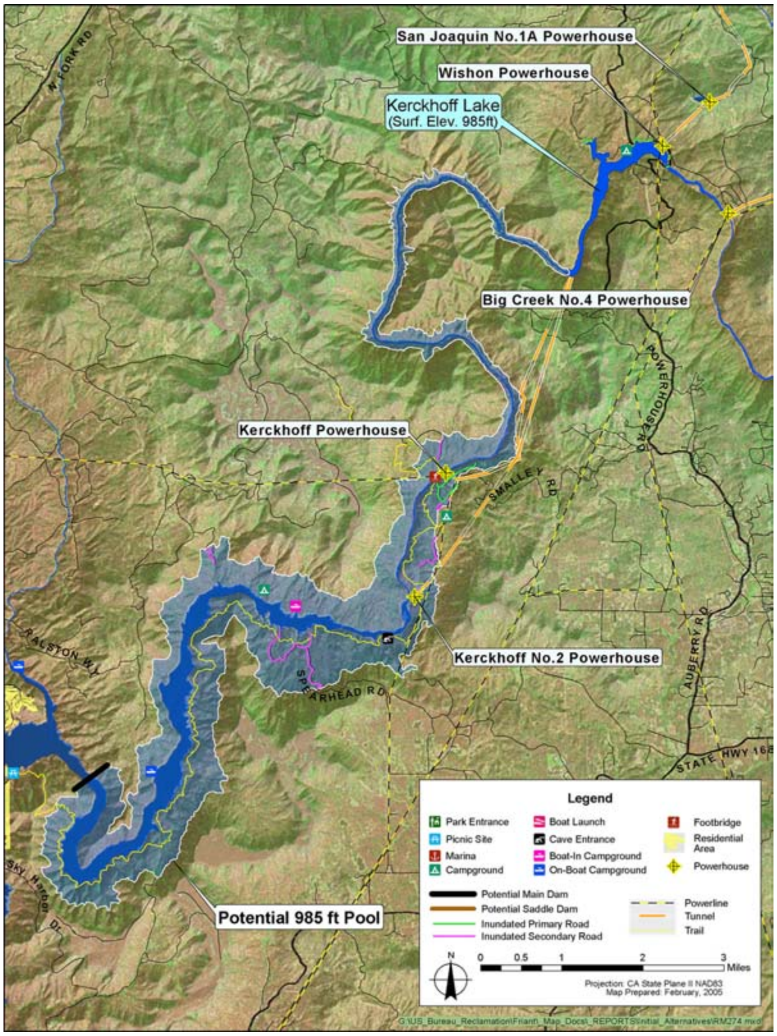

FIGURE 3-2.
POTENTIAL TEMPERANCE FLAT RM 274 RESERVOIR

Roads

Sky Harbor Drive, on the south side of Millerton Lake, provides access to private property in the Sky Harbor development and to the South Finegold picnic area within the SRA.Access to Temperance Flat from Auberry Road is provided by Wellbarn Road, extending to Spearhead Road. Smalley Road, which spurs off Auberry Road, provides access to the Kerckhoff Powerhouse and switchyard, the BLM primitive campground, and San Joaquin River Trail.

Powerhouse Road and Bridge connect Fresno and Madera counties across Kerckhoff Lake. Extending from Auberry Road in Fresno County to Road 222 in Madera County, the road and bridge provide access to Wishon Powerhouse for PG&E staff in Fresno County and to schools in Fresno County for residents in the North Fork area. In **Attachment C**, Powerhouse Road and Powerhouse Bridge are referred to as Auberry Road and Auberry Bridge, respectively. Redinger Lake Road spurs off Powerhouse Road, providing access to the Big Creek No. 4 Powerhouse, and then winding to Redinger Lake.

Hydroelectric Energy Facilities

Kerckhoff Powerhouse and Kerckhoff No. 2 Powerhouse, briefly described in Chapter 2, discharge to the upper portion of Millerton Lake and the San Joaquin River above RM 274. Water is diverted to the powerhouses at Kerckhoff Lake. Kerckhoff Lake storage is controlled at Kerckhoff Dam by outlet works and spillway gates. Outlet works are at elevation 900. The spillway crest is at elevation 971. Spillway gates provide the ability to store water behind the dam to a normal maximum water surface elevation of 985 feet.

A. G. Wishon Powerhouse, the terminal powerhouse of PG&E's Crane Valley Project, discharges to Kerckhoff Lake. Commissioned in 1919, the Wishon Powerhouse has an installed generation capacity of 20 MW. Water is diverted to the powerhouse from Corine Lake, north of Kerckhoff Lake.

Southern California Edison's (SCE) Big Creek No. 4 Powerhouse discharges to the San Joaquin River upstream of Kerckhoff Lake. Commissioned in 1952, it has an installed generation capacity of 100 MW. It is fed by water diverted at Big Creek Dam No. 7, which impounds Redinger Lake.

POTENTIAL IMPROVEMENTS

For a reservoir at RM 274, permanent features that would be constructed include a main dam with an uncontrolled spillway to pass flood flows, a powerhouse to generate electricity, and river outlet works for other controlled releases. Upstream and downstream cofferdams would be required for river diversion and to keep Millerton Lake out of the construction zone. Diversion tunnels to route river flows around the construction zone would be required during construction.

Reservoir Storage and Area

The RM 274 dam site could support a reservoir with storage of more than 2 million acre-feet, without requiring saddle dams. Reservoir surface area could extend up to 8,200 acres. The maximum reservoir size considered practical at this site, based on topographic characteristics, is a 2,100 TAF reservoir resulting from a dam with crest at approximately elevation 1,100.

Curves showing potential net storage capacity and surface area for a reservoir at RM 274 are presented in **Figure 3-3**. Reservoir sizes examined for RM 274 are shown in **Table 3-2**. Net storage volume accounts for existing storage capacity in Millerton Lake and Kerckhoff Lake.

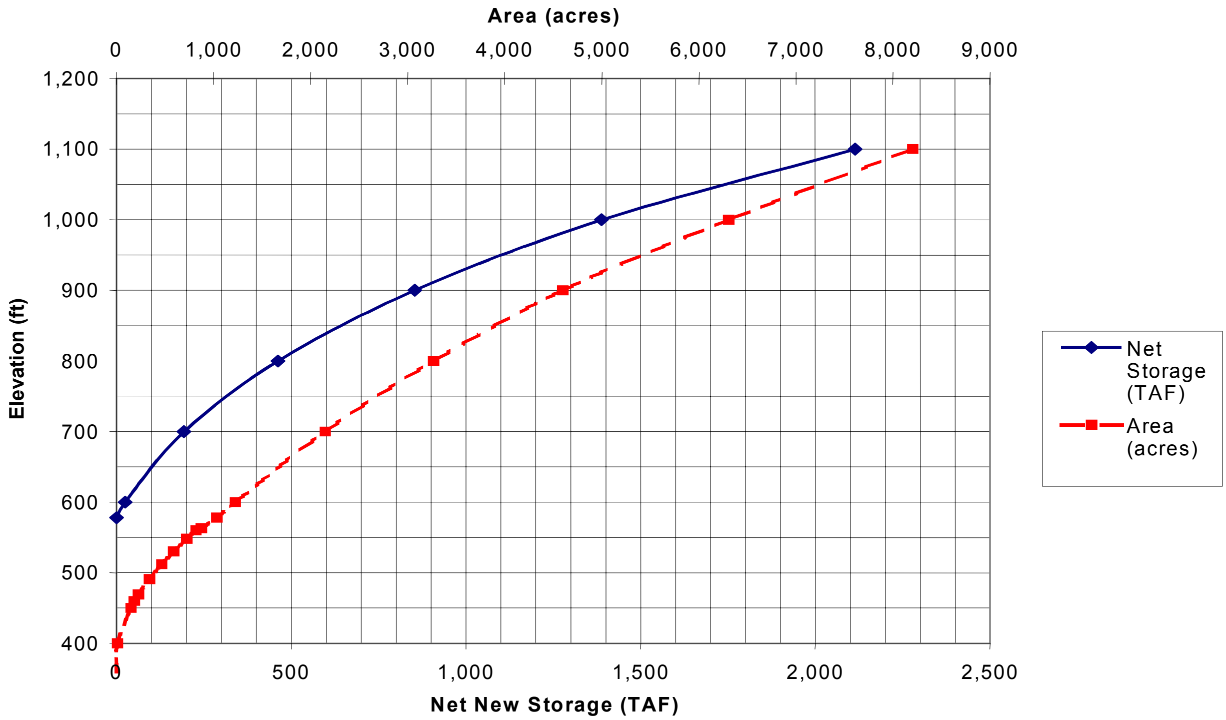

FIGURE 3-3.
TEMPERANCE FLAT RM 274 RESERVOIR SURFACE ELEVATION VS. NEW STORAGE AND AREA

TABLE 3-2.
SURFACE WATER STORAGE MEASURES EVALUATED FOR TEMPERANCE FLAT RM 274

Gross Pool Elevation (feet above msl)	Dam Height (feet)	Dam Type			Reservoir Area (acres)	Gross Storage Capacity (TAF)	Net Storage Capacity (TAF)
		RCC	CFRF	Arch			
800	415		X		3,270	530	460
865	485		X		4,160	790	725
960	575		X		5,620	1,250	1,170
985	600		X		6,050	1,380	1,310
1,100	715		X		8,200	2,190	2,110

Key:
CFRF – concrete face rockfill dam
msl – mean sea level
RCC – roller-compacted concrete
RM – river mile
TAF – thousand acre-feet

Main Dam

The main dam at RM 274 would be a concrete face rockfill dam (CFRF). Other dam types also were considered.

Dam Types Considered

The RM 274 site is suitable for a concrete arch dam or gravity dam (RCC or CFRF). A central-core earthfill dam is not considered economically viable due to the limited availability of plastic, fine-grained materials for the core. An asphaltic-core earthfill dam might be viable, but was not considered due to limited use and experience with this type of dam in the United States.

Foundation conditions are excellent for a concrete arch dam. However, the abutments uniformly rise with relatively flat slopes, producing a wide canyon that would require large volumes of concrete at substantial expense. Therefore, designs and cost estimates were not developed for a concrete arch dam at RM 274. Foundation conditions also are excellent for an RCC gravity dam. Due to limited study resources, however, layouts and cost estimates for an RCC gravity dam were not developed for the RM 274 site. Future studies of dams at this site should consider an RCC dam capable of being overtopped by an extreme flood event. If an RCC dam were to be developed at RM 274, the design would be similar to that for the RM 279 site, described in **Chapter 4**.

Concrete Face Rockfill Dam Design

CFRF layouts, including appurtenances, are shown in **Attachment B3**. A CFRF vertical cross section, perpendicular to the dam axis, is shown in **Figure 3-4**. A more detailed cross section also is included in **Attachment B1**. The design is based on standard practice, as described in Concrete Face Rockfill Dam: II. Design (Cook and Sherard, 1987). The cross section shows a concrete deck, which would act as the impervious water barrier for the dam. Beneath the deck is a layer of silty/clayey sand and gravel (Zone 2), which would provide the placement surface for the concrete deck and a secondary water barrier for seepage passing through joints in the deck. Below Zone 2 is the first of three rock-filled shell zones. The first zone (Zone 3A) would provide a transition to coarser zones below, and would consist of gravel and cobble sizes.

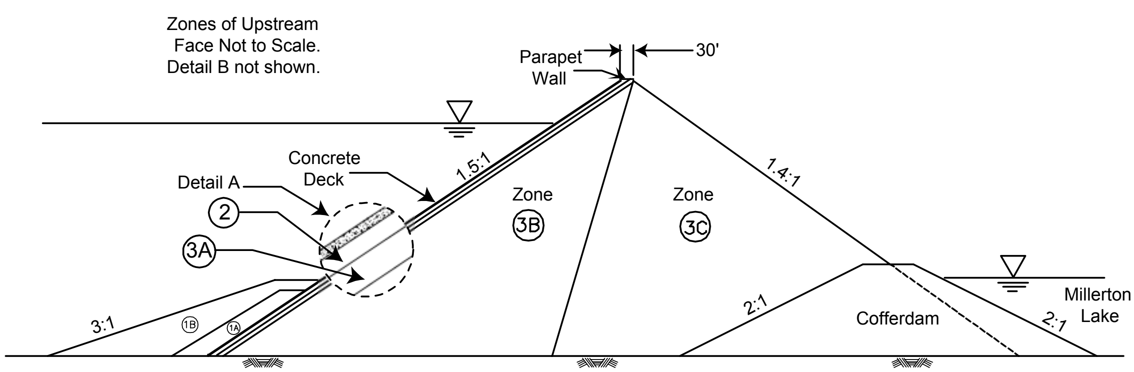

FIGURE 3-4.
CFRF DESIGN CROSS SECTION FOR TEMPERANCE FLAT RM 274 MEASURES

The second shell zone (Zone 3B) would be more compacted to minimize settlement. Zone 3C could be less compacted because it has lower potential for settlement due to its position within the cross section. At the waterside base of the dam, upstream of the concrete deck, primarily fine-grained and impermeable material (Zone 1A) would be placed over the perimeter joint to prevent seepage and minimize joint damage from reservoir debris. A stability zone (Zone 1B) would be placed over Zone 1A to buttress the barrier for greater slope stability.

Foundation grouting would consist of a single row curtain with an average depth of 250 feet, and companion blanket grouting with rows on either side of the curtain. Blanket holes would average 30 feet deep. Spacing of curtain holes would be 30 feet, and the spacing of blanket holes would be 10 feet. Closure pattern grouting is assumed to achieve a complete cutoff. Grouting details are shown on the cross section in **Attachment B1**.

Preliminary Dam Sizes Evaluated

For the RM 274 potential dam site, preliminary designs and construction cost estimates were developed for rockfill dams with crests at elevations 800 and 1,100. The resulting dams would be 415 and 715 feet high, respectively, measured from the existing riverbed at elevation 385. The estimates for a dam with these two crest elevations were then used to create an interpolated cost for a dam with the crest at elevation 960 (575 feet high). An analysis of potential hydroelectric energy generation, documented in the separate **Hydropower TA**, was conducted for reservoirs with net storage volumes of approximately 725 TAF and 1,310 TAF. Although smaller dam sizes could be constructed, the requirement of constructing within Millerton Lake likely would render dam heights below 400 feet uneconomical when compared to other surface water storage measures that produce similar reservoir storage. Topography limits the height of a dam that could be built without extensive saddle dams on the reservoir perimeter to about 700 feet.

Diversion Works

Diversion during construction was based on passing a peak discharge of 65,000 cubic feet per second (cfs), which corresponds to an approximate 25-year return period. Details regarding estimates of peak flows during construction are provided in the construction considerations section of this chapter. Diversion of river flows would be made possible by constructing tunnels through each abutment. A 30-foot diameter tunnel would be constructed through the left abutment, with a 40-foot-diameter tunnel through the right abutment. The capacity of the left abutment tunnel would be about 25,000 cfs during construction, and would later serve as the outlet works for the dam. The capacity of the right abutment tunnel would be about 40,000 cfs. The right abutment tunnel would be plugged following construction.

Upstream and downstream cofferdams would be required for diverting stream flows during construction and to prevent inundation of the site from Millerton Lake. A significant portion of both cofferdams would need to be constructed within the existing reservoir pool at a maximum depth of 175 feet. Cofferdams would be sized for estimated diversion flows, and to allow normal operation of Millerton Lake during construction. The downstream cofferdam would require a minimum crest elevation of 578 feet, and height of about 195 feet. The upstream cofferdam would be higher than the downstream cofferdam. The upstream cofferdam would require a crest at elevation 635 to provide sufficient head to pass the diversion flows, with a corresponding height of approximately 250 feet.

Spillway

The spillway design is based on passing a peak discharge of 145,000 cfs. This would be accomplished using an uncontrolled ogee crest spillway with a crest length of 450 feet, and a head of 20 feet. The spillway for the elevation 800 dam would be located on the right abutment.

However, for the elevation 1,100 dam, the spillway would be located through a saddle on the left reservoir rim at Big Bend, approximately one and a quarter river miles upstream from the dam. This would reduce spillway excavation requirements for the larger reservoir. The alignment for the saddle spillway would cross Fine Gold Drive, near the end of the road, which provides access to several residences and the South Finegold picnic area of the Millerton Lake SRA. A bridge would be required to pass over the spillway.

For both reservoir sizes, downstream channels for the spillway would be excavated through the existing foundation rock. A reinforced concrete apron and training wall would be constructed for the spillway within the first 100 feet upstream from the dam crest and 200 feet downstream from the crest, to control flows within the vicinity of the dam or saddle. Energy would be dissipated by the tailwater at the end of the natural channel, which would be over 100 feet deep depending on the level of Millerton Lake. For future designs, a labyrinth spillway should be considered for raising the crest elevation, providing more storage, and reducing the overall width of the spillway, including the outlet channel.

Recent safety-of-dam studies (Reclamation, 2002d; 2002e) indicate that Friant Dam can safely pass about 30 percent of the PMF before overtopping would occur. A risk assessment of the overtopping condition suggests that the existing concrete gravity dam at Friant can withstand the depth and duration of overtopping without failure. A similar assessment likely would be true for a new RCC dam at the RM 274 site. However, a rockfill dam at this site would likely fail at the same threshold condition. For purposes of this study, the spillway capacity was increased to 145,000 cfs at RM 274 (up from about 85,000 cfs for the existing Friant Dam spillway), to increase the level of threshold before overtopping would occur. Operation plans for a potential new reservoir have not been determined; therefore, flood routings have not been performed. Future studies would need to determine an appropriate inflow design flood for this site. Options for addressing a peak discharge larger than 145,000 cfs with a rockfill dam type include flood forecasting operations, increased spillway capacity, and creating additional surcharge storage. Use of an RCC dam type also should be considered in future studies of this site.

Outlet Works

The left abutment diversion tunnel would be converted to the outlet works. The outlet works layout would consist of a trash-racked intake structure, a water conveyance system, and a series of regulating gates with upstream guard gates. Energy from releases would be dissipated in the tailwater from Millerton Lake (plunge pool). The size of the conveyance system is dictated by diversion during construction, but normal reservoir operation requirements would control the size and number of gates. The capacity of the outlet works was set to closely match the capacity of the existing river and canal outlets at Friant Dam. Bulkheads would be required for the intake structure, and outlet gates within the upstream end of the tunnel also would be needed for dewatering. The control structure for the outlet works would be combined with the powerhouse.

A low-level outlet works with the capability of evacuating the reservoir below elevation 570 was not included in these studies. This range of reservoir level would be within the current operating pool of Millerton Lake, and could only be evacuated if Millerton Lake was drawn down below elevation 435. The need for a low-level outlet works should be considered in future studies. If needed, a tunnel through the abutment could be used for a rockfill dam, which would require placing the downstream cofferdam further downstream to provide room for constructing the outlet end of the tunnel. A low-level outlet for an RCC dam would be constructed through the dam.

Powerhouse

For purposes of preliminary powerhouse design and cost estimation, it was assumed that three turbines of equal size would operate within the head range and discharge capacity available during most of the year, as described in the Hydropower TA. Each turbine would operate independently within specific ranges of reservoir elevations. The powerhouse and outlet works control structure would be located at the downstream portal of the left abutment diversion tunnel. During normal releases, all flows would pass through the turbines. During periods of significant inflow, the outlet works could be engaged to supplement releases with the spillway available as needed.

Construction Considerations

This section discusses issues of concern related to constructing the potential dam, reservoir, and appurtenant features at RM 274.

Foundations

Foundations would be in sound granitic rock, as described in the geologic and seismic section of this chapter. No special foundation considerations are known for this site at this time; foundation preparation would be typical for any of the dams considered.

Flood Routing During Construction

A peak flow frequency analysis was performed to determine diversion requirements during construction (Reclamation, 2002d). The same stream gage used to develop the 100-year snowmelt flood was used for the peak flow frequency analysis. During larger flows, the upstream reservoirs were assumed to pass inflows, a condition similar to current operations. For diversion floods in the range of 10-year to 25-year return periods, it was assumed that the gage record adequately reflects future conditions. Peak flows were calculated for a location just below Kerckhoff Dam, and are considered appropriate for the Temperance Flat area. Results of this study are presented in **Table 3-3**.

The peak discharge of 65,000 cfs, with a return period of approximately 25 years, was used to size the diversion structures for each surface water storage measure at all potential dam sites in the Temperance Flat area (RM 274, RM 279, RM 286).

TABLE 3-3.
PEAK FLOOD FLOWS BELOW KERCKHOFF DAM

Return Period (years)	Peak Flow (cfs)
5	27,500
10	41,600
25	65,100
50	87,300
100	113,900

Key:
cfs – cubic feet per second
Source: Reclamation, 2003a

Borrow Sources and Materials

Rockfill could be quarried from the reservoir area and obtained from excavations required for the dam and appurtenant structures. Earthfill is available in limited quantities. Low-plasticity, fine-grained soil might be available in the Auberry Valley area, and in an area south of Millerton Road near the Millerton Lake SRA entrance. Road cuts in Temperance Flat and the Auberry Valley expose decomposed to intensely weathered granite. Processed sands and gravels, and concrete aggregates could be supplied by commercial sources or by crushing and processing rock quarried in the reservoir area.

Construction Site Access

Access to the dam site is across both public and private land. The site is not directly accessible by existing roads, although Fine Gold Drive terminates downstream of the left abutment, and a jeep trail provides access to higher elevation lands within a mile of the right abutment. Both abutments are accessible by boat from Millerton Lake.

Staging Areas

No specific staging or lay-down areas have been identified for the RM 274 site. This site has a moderate amount of nearby development, due to its location in Millerton Lake. Residential properties and developed recreation facilities fringe the site, making identifying staging areas difficult at the current level of study.

Lands and Rights-of-Way

Two residences are located in the Temperance Flat area would be inundated by a RM 274 reservoir with a surface at elevation 800 feet or greater. Private lands within the reservoir area would need to be acquired. Table 3-4 shows the total area that would be inundated by the RM 274 reservoir measures considered and the amount of private land within that area that would need to be acquired. Rights of way or easements that may be needed to construct new facilities or relocate existing facilities have not been determined and are excluded from the land areas shown.

Electric Power Sources

Electric power, including high voltage power, is available from transmission facilities serving the PG&E Kerckhoff Project. Electric power, including lower voltage power, would be available from existing trunks supplying local residences.

TABLE 3-4.
TEMPERANCE FLAT RM 274 RESERVOIR AREA LAND REQUIREMENTS

Description	RM 274 Reservoir Measures				
Gross pool elevation (feet above msl)	800	865	960	985	1,100
New storage capacity (TAF)	460	725	1,170	1,310	2,110
Estimated inundated area (acres)	2,233	3,127	4,590	5,017	7,172
Estimated public inundated acreage	2,138	2,762	3,775	4,082	5,584
Estimated private inundated acreage	95	365	815	935	1,588

Key:
msl – mean sea level
TAF – thousand acre-feet

Relocations or Abandonments of Affected Facilities

Existing facilities that would be inundated by a reservoir at RM 274 are indicated in **Figure 3-2**. **Table 3-5** lists upstream facilities that would require abandonment or relocation (i.e., construction at a new location), for each of the three dam crest elevations examined.

Facilities that would require abandonment and possible relocation for a reservoir formed at elevation 1,100 include recreational facilities in the Millerton Lake SRA, the San Joaquin River Gorge management area, and at Kerckhoff Lake; an interpretive Native American village in the BLM management area; portions of paved and unpaved roads; Powerhouse Road Bridge; hydropower facilities in the Temperance Flat area and above Kerckhoff Lake. Some, but not all, of these facilities also would be inundated by a reservoir with a lower maximum surface elevation.

Specific requirements for relocating or abandoning recreational and interpretive facilities were not determined. Requirements for relocating or abandoning residential property, roads, and hydroelectric facilities are described below.

Recreation Facilities

A reservoir at elevation 1,100 would inundate nearly the entire portion of the San Joaquin River Trail that starts at the South Finegold picnic area in the SRA and terminates at the primitive campground in the San Joaquin River Gorge. It also would inundate the segment of the trail that crosses the San Joaquin River footbridge. A reservoir at elevation 800 would inundate only slightly less of the trail, leaving a few disconnected segments.

A reservoir developed at RM 274 with a surface at elevation 800 or greater would inundate boat-in and on-boat campgrounds and environmental camps in the SRA. A reservoir at elevation 960 additionally would just inundate the interpretive Native American village. A reservoir at elevation 1,100 would affect the BLM primitive campground in the San Joaquin River Gorge off Smalley Road and at Kerckhoff Lake, a boat launch, picnic area, and the Smalley Cove campground.

TABLE 3-5.
RELOCATIONS REQUIRED FOR TEMPERANCE FLAT RM 274 MEASURES

<table>
<tr><th>Features Requiring Relocation or Abandonment</th><th colspan="3">RM 274 Reservoir Measures</th></tr>
<tr><td>SRA Temperance Flat Boat-In Campground</td><td rowspan="10">Elevation 800</td><td rowspan="15">Elevation 960</td><td rowspan="24">Elevation 1,100</td></tr>
<tr><td>SRA On-Boat Camping Facility</td></tr>
<tr><td>Temperance Flat Residences</td></tr>
<tr><td>Kerckhoff Powerhouse No. 2 Access Tunnel Entrance</td></tr>
<tr><td>SRA New Toilet Facility</td></tr>
<tr><td>SRA Hewitt Valley Environmental Camps</td></tr>
<tr><td>Portions of San Joaquin River Trail</td></tr>
<tr><td>BLM Footbridge</td></tr>
<tr><td>Kerckhoff Powerhouse</td></tr>
<tr><td>Substation for Kerckhoff Powerhouse No. 2</td></tr>
<tr><td colspan="2">Kerckhoff Dam Outlets</td></tr>
<tr><td colspan="2">Surge Chamber for Kerckhoff Powerhouse</td></tr>
<tr><td colspan="2">Kerckhoff No. 2 Powerhouse Intake</td></tr>
<tr><td colspan="2">Kerckhoff Powerhouse Intake</td></tr>
<tr><td colspan="2">BLM Native American Village (reproduction)</td></tr>
<tr><td colspan="3">Kerckhoff Dam Gates</td></tr>
<tr><td colspan="3">Powerhouse Road</td></tr>
<tr><td colspan="3">Powerhouse Road Bridge</td></tr>
<tr><td colspan="3">Smalley Cove Campground, Picnic Area, Boat Launch</td></tr>
<tr><td colspan="3">A.G. Wishon Powerhouse</td></tr>
<tr><td colspan="3">Surge Chamber for Kerckhoff Powerhouse No. 2</td></tr>
<tr><td colspan="3">Big Creek Powerhouse No. 4</td></tr>
<tr><td colspan="3">BLM Primitive Campground</td></tr>
<tr><td colspan="3">Substation for Big Creek Powerhouse No. 4</td></tr>
<tr><td colspan="4">Key:
BLM – Bureau of Land Management RM – river mile SRA – State Recreation Area</td></tr>
</table>

Roads and Bridges

Portions of several roads could be inundated by a reservoir constructed at RM 274, as discussed further below. In total, for a reservoir with a gross pool elevation of 1,100 feet, nearly 5 miles of paved road would be inundated and 11 miles of unpaved road. A lesser amount would be inundated for lower reservoir elevations. Powerhouse Road Bridge also would be affected by a reservoir at elevation 1,100, as discussed below.

Sky Harbor Drive

The spillway for a rockfill dam with crest at elevation 1,100 would be constructed over a ridge on the left abutment and would cross Sky Harbor Drive. Continued use of the road would require constructing a bridge or overpass. Specific requirements for such a feature were not determined. Alternatively, acquiring property presently requiring access from the road would be considered.

A rockfill dam with crest at elevation 800 would not affect Sky Harbor Drive, as its spillway would be constructed on the dam's right abutment. No RCC dam spillway would affect Sky Harbor Drive because the spillway would pass over the top of the dam.

Auberry Road Spur Roads

The end portion of Smalley Road at Kerckhoff Powerhouse would be inundated by a reservoir at RM 274 and would be abandoned. The terminal portion of Spearhead Road that extends from Wellbarn Road to the Temperance Flat area also would be inundated by any reservoir at RM 274 and abandoned. For both roads, the length of road affected would vary with the maximum reservoir elevation.

Powerhouse Road and Bridge

Powerhouse Road and Bridge would be affected by a Temperance Flat Reservoir with a surface at elevation 1,000 or higher. Relocation requirements for the road and bridge for a reservoir with a maximum surface at elevation 1,200 to 1,400 are presented in **Chapter 4**. For a reservoir at elevation 1,100, it is assumed that an approach similar to that described in **Chapter 4** for an elevation 1,200 storage measure would be used. However, the length of road and bridge deck and height of bridge piers would be less. Approximately 9,400 feet of roadway would need to be constructed or improved. The bridge deck length would be approximately 730 feet.

Redinger Lake Road

A portion of Redinger Lake Road starting at Kerckhoff Lake would be inundated by a reservoir extending to elevation 1,100. It is assumed that the inundated portion would be abandoned. The portion of the road descending from Redinger Lake would remain.

Kerckhoff Project

A Temperance Flat RM 274 Reservoir would inundate the existing Kerckhoff Project powerhouses. It is assumed that construction of a new powerhouse below Kerckhoff Dam would use a similar approach to that described in **Chapter 5** for a replacing the Big Creek No. 4 Powerhouse at Redinger Dam. An alternative approach might be to construct a new powerhouse off one of the existing Kerckhoff diversion tunnels at a higher elevation than the existing powerhouses; however, the current analysis assumes that approach would not be taken. Continued use of Kerckhoff Lake water storage for hydroelectric energy generation would be impractical for a new Temperance Flat Reservoir at elevation 960 or greater. Existing outlet works at Kerckhoff Dam are at elevation 900 and the existing diversion tunnel intakes at Kerckhoff Lake are at approximately elevation 940.

Specific requirements for decommissioning the Kerckhoff powerhouses were based on limited general outline drawings and limited powerhouse data. No detailed drawings were available and no site visit was made to inspect the existing facilities. With little information regarding the existing power grid, it was assumed that the existing switchyards at the Kerckhoff powerhouses would remain in place as tie points for the power grid. Subsequently obtained elevation data indicates that this assumption will need to be modified in future analyses.

Kerckhoff No. 2 Powerhouse

Abandoning the Kerckhoff No. 2 Powerhouse would proceed as follows. A hazardous material inventory would be conducted to identify all hazardous materials present at the powerhouse. All hydraulic, lubricating, and insulating oils would be drained and disposed. Any refrigerants, storage batteries, or compressed gas also would be disposed. The step-up transformers would be removed and disposed. A concrete plug would be placed in the intake and draft tubes. The steel surge chamber tank would be disposed and the surge chamber shaft plugged and backfilled.

Kerckhoff Powerhouse

Two alternative approaches to decommissioning were considered: abandoning the facility in place and restoring the site to near natural conditions. The abandon-in-place approach assumes the majority of the physical features would be left in place. This is the same described above for the abandoning the Kerckhoff No. 2 Powerhouse. A hazardous material inventory would be conducted to identify hazardous materials present at the powerhouse. All hydraulic, lubricating, and insulating oils would be drained and disposed. In addition, any refrigerants, storage batteries, or compressed gas would require disposal. The transformers and oil circuit breakers would be removed and disposed. Overhead conductors from the powerhouse to the switchyard would be removed. Finally, concrete plugs would be placed in the intake and draft tubes. The surge chamber would be plugged and backfilled.

For the restore-to-near-natural-conditions approach, all steps for the abandon-in-place approach would be required and most of the physical structures would be removed. Removing and disposing of hazardous waste would be more comprehensive due to demolition of the structures. The estimate for this approach includes additional amounts for removing asbestos and equipment containing mercury. The majority of the mechanical and electrical equipment would be removed. The powerhouse building would be demolished and the concrete disposed in the existing footprint. The powerhouse then would be capped with backfill concrete and the remainder of the void backfilled and graded with earth. Weights of mechanical and electrical equipment to be removed were calculated based on limited outline drawings available in the public domain as part of the Federal Energy Regulatory Commission (FERC) license application.

Kerckhoff Dam and Intakes

Depending on the size of the dam, the reservoir pool behind a new dam at RM 274 could inundate Kerckhoff Dam and submerge the intake structures at Kerckhoff Lake. For a new reservoir with a surface at elevation 800 or 865, the Kerckhoff Dam outlet works and spillway would be unaffected. However, the diversion intakes would be decommissioned, unless a replacement powerhouse that made continued use of one of the diversion tunnels was built.

A reservoir with a surface at elevation 960 would rise up the backside of Kerckhoff Dam to within 11 feet of the spillway crest, submerging the Kerckhoff Dam outlets. Consequently, for an elevation 960 storage measure, the Kerckhoff Dam outlet works would be removed. Intakes for both Kerckhoff powerhouses also would be decommissioned as the diversion structures would not be used. Spillway features could be left intact. A reservoir with a maximum surface at greater than elevation 985 would completely submerge Kerckhoff Dam and require removing the spillway and outlet works equipment, and decommissioning the Kerckhoff Project diversion intakes. Specific features to be removed are identified below.

Spillway Features

Fourteen radial gates extend across most of the length of the existing concrete arch dam. All of the gates and associated equipment would be removed for a new reservoir with a maximum surface at greater than elevation 985. The concrete hoist decks and spillway piers would be removed down to the spillway crest. Rendering Kerckhoff Dam inoperable would involve removal of the following equipment:

- Fourteen 14-foot by 20-foot radial gates
- Five gate hoists
- Rails for gate hoists
- Electric service transformers
- Reinforced concrete spillway hoist decks
- Reinforced concrete spillway piers to elevation 971

Outlet Works Features

The outlet works consists of three concrete-encased steel pipes extending through the dam near the base of the structure. The outlet equipment would be removed, but the holes through the dam would be left open. This would ensure reservoir equalization when the surface of the new reservoir falls below the existing spillway crest and also would provide for passage of flows. Outlet works equipment consists of the following:

- Trashracks
- Three 72-inch diameter slide gates
- Three sets of gate hoists
- Reinforced concrete decks and dam structure above the spillway crest elevation
- Reinforced concrete (assumed) float control house at dam crest

Intakes for Kerckhoff Powerhouses

Abandoning the Kerckhoff powerhouses and diversion tunnels would require the associated intake structures at Kerckhoff Lake to be removed. This is because the intake structures extend above the current reservoir level at Kerckhoff Lake and would present a hazard if left intact. Decommissioning the intake structures would occur as described below. For the elevation 800 storage measure, if Kerckhoff generating capacity were reconstructed at a higher elevation and fed by one of the existing diversion tunnels, the associated intake would be left intact.

Kerckhoff Powerhouse Intake

The intake structure forms the entrance at the portal for the penstock; therefore, only those features extending above the natural ground level would be removed. In addition, a concrete plug would be installed at the entrance to the tunnel. All or portions of the following features would be removed:

- Trashracks
- Two 10-foot by 18-foot wheel mounted gates
- Two sets of gate hoists and stems

- Two 3-foot by 3-foot manually operated gates
- Steel superstructure above elevation 985
- Steel walkway to upper deck
- Reinforced concrete above elevation 970
- Installation of concrete plug

Kerckhoff No. 2 Powerhouse Intake

For the intake of the Kerckhoff Powerhouse diversion, only those features extending above the natural ground level would be removed. A concrete plug would be installed at the entrance to the diversion tunnel. All or portions of the following features would be removed:

- Trashracks
- Two 10 feet 7 inches by 24 feet 6 inches wheel mounted gates
- Two sets of gate hoists and stems
- Steel head frame
- Gate guides
- Installation of concrete plug

Wishon Powerhouse

An RM 274 reservoir with a surface elevation lower than 960 would be below the crest of Kerckhoff Dam and would not affect the Wishon Powerhouse. A RM 274 reservoir with a surface elevation 1,100 would submerge the Wishon Powerhouse. Relocation would involve decommissioning the existing powerhouse and constructing a new one at a higher elevation. Specific construction requirements for a new powerhouse were not determined; however, analysis documented in the **Hydropower TA** indicates that a generating capacity up to 18 MW could be supported.

Specific requirements and cost estimates for decommissioning the Wishon Powerhouse were estimated. For the Wishon Powerhouse, it is assumed that most of the physical structures, including the powerhouse and associated buildings, would be left in place. A hazardous material inventory would be conducted to identify hazardous materials present at the powerhouse. All hydraulic, lubricating, and insulating oils would be drained and properly disposed. In addition, any refrigerants, storage batteries, or compressed gas also would require disposal. Any glass or other material deemed a safety hazard would be removed. Transformers and oil circuit breakers would be removed and disposed. The majority of the mechanical and electrical equipment would be removed. No cost provision was made for removing the penstocks.

The cost estimate for decommissioning the Wishon Powerhouse is based on general outline drawings and limited powerhouse data. No detail drawings were readily available and no site visit was made by estimators to inspect existing facilities. Weights for mechanical and electrical equipment to be removed are based on the limited outline drawings available in the public domain as part of the FERC license application process.

Big Creek No. 4 Powerhouse

As for the Wishon Powerhouse, a reservoir at RM 274 with a surface at elevation 960 or less would not affect the Big Creek No. 4 Powerhouse, but a reservoir with a surface at elevation 1,100 would submerge it. This would require abandonment and possible relocation at a higher elevation. For a Temperance Flat Reservoir at elevation 1,100, which would serve as the tailwater for a replacement powerhouse, analysis documented in the **Hydropower TA** indicates that a replacement powerhouse with generating capacity of 80 MW could be supported.

Decommissioning

For decommissioning the Big Creek No. 4 Powerhouse, it was assumed that most of the physical structure would be left in place. A hazardous material inventory would be conducted to identify all hazardous materials present at the powerhouse. All hydraulic, lubricating, and insulating oils would be drained and properly disposed. In addition, any refrigerants, storage batteries, or compressed gas also would be disposed. Any glass or other material deemed a safety hazard would be removed. Transformers and oil-circuit breakers would be removed and properly disposed. The majority of the mechanical and electrical equipment would be removed. It is assumed that the penstock would be left in place.

Requirements for decommissioning the Big Creek No. 4 Powerhouse were determined using general outline drawings and limited powerhouse data. No detail drawings were available for the purpose of this estimate and no site visit was made to inspect existing facilities. Weights of mechanical and electrical equipment to be removed were calculated from the limited outline drawings available in the public domain as part of the FERC license application process.

Replacement Powerhouse

As indicated above, a new Temperance Flat Reservoir with a gross pool at elevation 1,100 would inundate the existing Big Creek No. 4 Powerhouse. This would present an opportunity to build a replacement powerhouse at a higher elevation, albeit with less generating capacity. Specific requirements for constructing a replacement powerhouse that would discharge to a new Temperance Flat Reservoir with a gross pool at elevation 1,100 were not determined. A design for a replacement powerhouse at an elevation of approximately 1,250 is discussed in **Chapter 4**.

Construction Costs

Costs for constructing RM 274 surface water storage measures at July 2004 price levels are summarized in **Table 3-6**. Costs shown for individual project components include field costs and indirect costs for planning, investigations, designs, and construction management. Indirect costs involved in constructing project components are 25 percent of field costs. Acquisition costs for lands in the reservoir area are incorporated into the construction cost of the main dam, along with an allowance of 20 percent of lands costs for indirect costs associated with property acquisition.

Construction costs are based on worksheets presented in **Attachments C1** and **C3**. Field costs for the dam and appurtenant features, previously calculated at July 2003 price levels, were adjusted to reflect July 2004 prices and a higher contingency. Costs for abandonments and relocations were developed in July 2004. Accordingly, all costs listed in **Table 3-6** are at July 2004 price levels.

Cost estimates were originally developed for dams with CFRF construction and crests at elevations 800 and 1,100, based directly on quantities calculated from preliminary designs. Cost estimates for dam and appurtenant features for crest elevations 865, 960, and 985 were interpolated from estimates for the elevation 800 and 1,100 structures. Where quantities or unit prices for a pay item varied between the elevation 800 and 1,100 costs, corresponding entries for the intermediate elevation estimate used quantities and unit costs that were linearly interpolated from values in the worksheets for the elevation 800 and 1,100 reservoirs.

Cost estimates were not developed for the RCC dam type at RM 274. However, cost estimates were prepared for both RCC and rockfill dam types at RM 279 (see **Chapter 4)**, and the range of variation between RCC and CFRF costs that occurs for the RM 279 site generally would be expected to be similar at RM 274. The original cost worksheets included combined costs for the dam, a spillway, diversion works, a 40 to 60 MW powerhouse, and outlet works. Subsequently, the cost for the powerhouse was isolated and adjusted to reflect a greater generating capacity. Costs shown in **Table 3-6** for the powerhouse at RM 274 reflect the adjusted cost for an 80 MW powerhouse for reservoirs up to elevation 865, and 100 MW powerhouse for greater elevations.

Costs for decommissioning the Kerckhoff powerhouses and their intakes are included in **Table 3-6** for all crest elevations. It is assumed that the powerhouses would be abandoned in place. For the elevation 800 and 865 surface water storage measures, a new replacement powerhouse would be constructed at the base of Kerckhoff Dam, with a single 20 MW generating unit. No design has been prepared for this powerhouse. Instead, it is assumed that the cost to construct a powerhouse at Kerckhoff Dam would be roughly equivalent to the cost to construct a powerhouse at Redinger Dam after adjusting for generating capacity. The adjustment factor is equal to the square root of the ratio of the respective generating capacities.

No replacement powerhouse for the Kerckhoff Project would be constructed for measures with a dam crest elevation of 960 or above. For these measures, indicated costs include removing the Kerckhoff Dam outlet works and spillway gates. Although it would be possible to maintain use of the Kerckhoff Dam spillway gates for an elevation 960 measure, for convenience, the costs for removing the gates were bundled with the costs of disabling the outlet works.

TABLE 3-6.
CONSTRUCTION COSTS FOR TEMPERANCE FLAT RM 274 MEASURES
($ MILLION)

Gross Pool Elevation (feet above msl)	800	865	960	985	1,100
New Storage Capacity (TAF)	**460**	**725**	**1,170**	**1,310**	**2,110**
Storage Components					
CFRF Dam, Spillway, Outlet Works, River Diversion, Reservoir Lands	560	650	790	810	970
Abandon Kerckhoff Powerhouse	2	2	2	2	2
Abandon Intake for Kerckhoff Powerhouse	1	1	1	1	1
Abandon Kerckhoff No. 2 Powerhouse	2	2	2	2	2
Abandon Intake for Kerckhoff No. 2 Powerhouse	1	1	1	1	1
Remove Kerckhoff Dam Outlet Works and Gates	-	-	2	2	2
Abandon Wishon Powerhouse	-	-	-	-	2
Abandon Big Creek No. 4 Powerhouse	-	-	-	-	4
Powerhouse Road Relocation	-	-	-	-	18
Powerhouse Bridge Relocation	-	-	-	-	21
Construction Cost, Storage Components	**566**	**656**	**798**	**818**	**1,023**
Replacement Power Components					
New Powerhouse at RM 274 Dam (80 to 100 MW)	170	170	195	195	195
New Powerhouse at Kerckhoff Dam (20 MW)	59	59	-	-	-
New Wishon Powerhouse (18 MW)	-	-	-	-	46
New Big Creek No. 4 Powerhouse (80 MW)	-	-	-	-	115
Construction Cost, Replacement Power Components	**229**	**229**	**195**	**195**	**356**
Construction Cost[1, 2]	**795**	**885**	**993**	**1,013**	**1,379**

Key:
CFRF – concrete-face rockfill
msl – mean sea level
MW – megawatt
RM – river mile
TAF – thousand acre-feet
Notes:
[1] All cost estimates are preliminary. Construction cost represents the sum of field costs and indirect costs for planning, engineering, design and construction management, which are estimated at 25 percent of field costs.
[2] Costs do not include environmental mitigation, new or relocated recreation facilities, acquisition of impacted power facilities, or compensation for lost future power generation.

Costs for relocating the Wishon and Big Creek No. 4 powerhouses are included for the elevation 1,100 measure. The cost to construct a replacement powerhouse for the Big Creek No. 4 Powerhouse below Redinger Dam was proportionally estimated based on the cost for by smaller replacement Wishon powerhouse. The cost for relocating the Wishon Powerhouse was included based on the cost of constructing an 18 MW powerhouse. The cost to relocate Powerhouse Road and Bridge for a reservoir with at elevation 1,100 was approximated. The route for a relocated road was selected with the aid of a topographic map, and the length of road and bridge determined with GIS. Costs for the relocated road and bridge then were approximated using unit costs per foot derived from cost estimates prepared for relocations of the same road at higher elevations. The basis of those more thorough estimates is described in **Chapter 4**.

Additional study would be needed to determine: the cost to construct a bridge at Sky Harbor Drive over the spillway for the elevation 1,100 reservoir, if required; costs to relocate recreation facilities; and the cost for any required environmental mitigation. Those potential costs are not included in the totals shown.

For all dam sizes, the dam and appurtenant structures would be located on public land. Parcels of land immediately upstream from the construction area and in the potential area of inundation are privately owned and would need to be acquired. **Table 3-6** shows the estimated amount of privately held land that would be acquired for the different reservoir sizes. All costs involved in acquiring private lands in the reservoir area are included in the construction costs listed for the dam and appurtenances. However, the costs to acquire any Kerckhoff Project assets that would be abandoned, or to compensate their owners for any loss of future energy generation, are not included in the table.

Figure 3-5 shows the relationship between new storage capacities that would be developed with a reservoir at RM 274 versus construction cost. As noted, costs would increase substantially when Big Creek No. 4 Powerhouse and Wishon Powerhouse would be inundated and costs are incurred to replace their generation capacity.

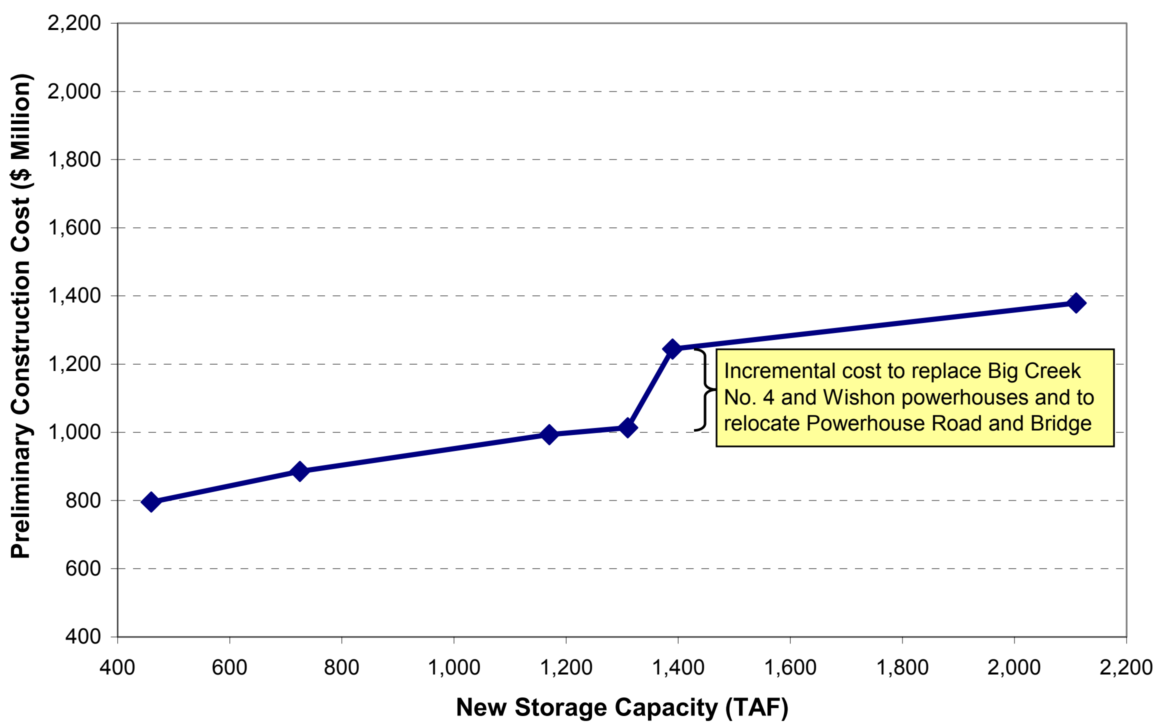

FIGURE 3-5.
CONSTRUCTION COSTS FOR TEMPERANCE FLAT RM 274 MEASURES VS. NEW STORAGE CAPACITY

CHAPTER 4. TEMPERANCE FLAT RESERVOIR AT RM 279

This chapter describes structures and costs involved in developing a reservoir at RM 279 of the San Joaquin River in the Temperance Flat area. This chapter is structured similar to **Chapters 2** and **3**, and describes site conditions, engineering considerations associated with design and construction of the dam and appurtenant features, relocations or abandonments of existing facilities, and required land acquisition. More detailed descriptions of existing and potential new hydroelectric generation facilities are included in the **Hydropower TA**.

PREVIOUS STUDIES

Previous studies of a potential reservoir at Temperance Flat were completed during original planning for development of Friant Dam, as described in **Chapter 3**.

SITE DESCRIPTION

The RM 279 potential dam site is located in the upper reaches of Millerton Lake, approximately six river miles upstream of the confluence with Fine Gold Creek. **Figure 1-1** shows the general location of the Temperance Flat area potential dam sites and **Figure 3-1** shows in greater detail the location of the RM 279 site in relationship to the RM 274 and RM 286 sites.

Topographic Setting

The RM 279 site rises uniformly from elevation 460 in the original San Joaquin River channel at RM 278.9 to elevation 1,080 on the left abutment, then drops slightly through a saddle at elevation 1,040 before continuing to elevation 1,416 at an unnamed mountain. The right abutment rises uninterrupted to elevation 1,566 at an unnamed mountain.

Geologic Setting

Readily observable geologic conditions at the RM 279 site are very similar to those at the RM 274 site. See **Chapter 3** for a description of geologic site conditions.

Site Geotechnical Conditions

Unstable wedges, toppling, or slides were not observed at the RM 279 site. The granitic bedrock has adequate strength and stability for embankment, rockfill, concrete gravity, or concrete arch dam structures and for any river diversion feature. The granite also would provide an adequate foundation for a plunge pool or overflow spillway. No known faults exist at the RM 279 site or in the vicinity (Reclamation, 2002b). No known adverse geologic/geotechnical conditions exist at the site that would require special consideration for design and/or construction.

Seismic Hazard Analysis

Results from analysis of potential horizontal ground acceleration from fault and areal background sources at Temperance Flat are presented in **Chapter 3**.

Existing Facilities

Constructed facilities in the Temperance Flat area include isolated residences, recreation facilities within the San Joaquin River Gorge management area, roads, and the PG&E Kerckhoff Project. At and above Kerckhoff Lake are recreational facilities, roads, the PG&E Wishon Powerhouse, and the SCE Big Creek No. 4 Powerhouse. **Table 4-1** lists facilities in the Temperance Flat area, at Kerckhoff Lake, and between Kerckhoff Lake and Redinger Lake. Roads, varying considerably in elevation and location, are excluded from the table. Facilities that could be inundated by a reservoir constructed at RM 279 are indicated in **Figure 4-1**.

TABLE 4-1.
FACILITIES ABOVE TEMPERANCE FLAT RM 279

Approximate Elevation (feet above msl)	Approximate Location (SJ River Mile)	Facility
580-1,240	273 – 284	San Joaquin River Trail (SRA to SJR Gorge portion)
589	281	SRA Temperance Flat Boat-In Campground
592 - 705	280.5 - 281	Temperance Flat Residences
605	282.5	Kerckhoff Powerhouse No. 2 Access Tunnel Entrance
630	281	SRA New Toilet Facility
650	280	SRA Hewitt Valley Environmental Camps
680	284.5	BLM Footbridge Below Kerckhoff Powerhouse No. 1
675	284.5	Kerckhoff Powerhouse Main Floor
680 – 2,120	283.5 - 284.5	San Joaquin River Trail (SJR Gorge portion)
778	283	Substation for Kerckhoff Powerhouse No. 2
889	292.5	Base of Kerckhoff Dam
900	292.5	Kerckhoff Dam Outlets
921	284.5	Surge Chamber for Kerckhoff Powerhouse
939	301.5	Kerckhoff No. 2 Powerhouse Intake
942	301.5	Kerckhoff Powerhouse Intake
960	285	Native American Village (reproduction)
971	292.5	Kerckhoff Dam Crest
980	295	Powerhouse Road Bridge
980	295.5	Smalley Cove Campground, Picnic Area, Boat Launch
990	294.5	A.G. Wishon Powerhouse
993	283	Surge Chamber for Kerckhoff Powerhouse No. 2
993	296	Big Creek Powerhouse No. 4
1,030	284	BLM Primitive Campground
1,089	296	Substation for Big Creek Powerhouse No. 4
1,181	301.5	Base of Big Creek Dam No. 7 (Redinger Lake Dam)
1,403	301.5	Crest of Big Creek Dam No. 7 (Redinger Lake Dam)

Key:
BLM – Bureau of Land Management msl – mean sea level
SJ – San Joaquin SJR – San Joaquin River SRA – State Recreation Area

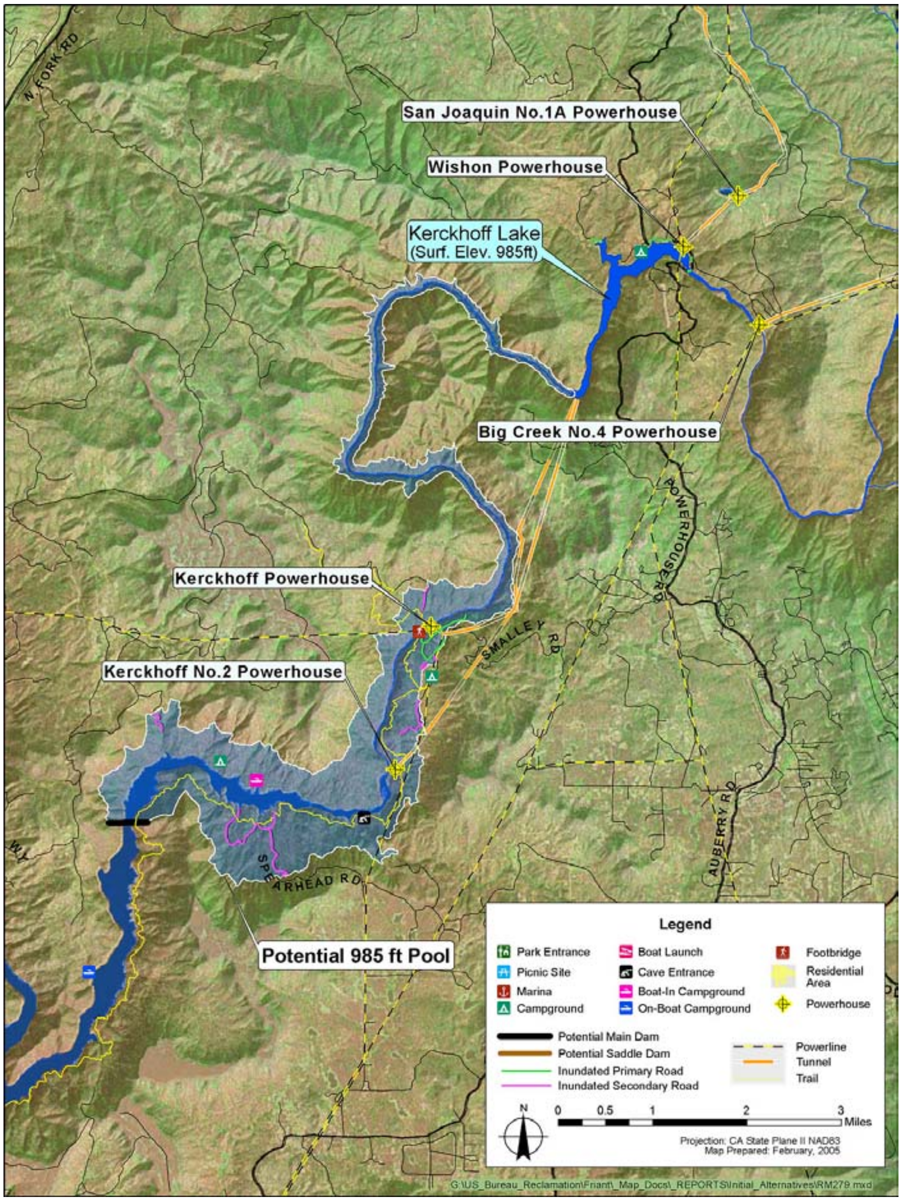

FIGURE 4-1.
POTENTIAL TEMPERANCE FLAT RM 279 RESERVOIR

Residential Property

Two residences are located in the Temperance Flat area.

Recreation Facilities

Recreational facilities upstream of RM 279 include the Hewitt Valley Environmental Camps within the Millerton Lake SRA, and the San Joaquin River Trail, which connects the SRA and BLM San Joaquin River Gorge management area. Within the BLM management area are an extension of the San Joaquin River Trail, footbridge, primitive campground, and a reproduction Native American village. Developed recreational facilities at Kerckhoff Lake include a car-top boat launch, day-use area, and campground at Smalley Cove.

Roads

Roads connecting Auberry Road to Temperance Flat, Powerhouse Road and Bridge, and Redinger Lake Road are within the watershed that could be affected by a reservoir at RM 279. These roads are more fully described in **Chapter 3**.

Hydroelectric Energy Facilities

The Kerckhoff Project, Wishon Powerhouse, and Big Creek No. 4 Powerhouse are hydroelectric energy generation facilities located within the potential inundation area of a reservoir at RM 279. These facilities are described briefly in **Chapters 2** and **3**. Additional details can be found in the **Hydropower TA**.

POTENTIAL IMPROVEMENTS

Permanent features that would be constructed to develop a reservoir at RM 279 include a main dam with an uncontrolled spillway to pass flood flows, a powerhouse to generate electricity, and river outlet works for other controlled releases. Upstream and downstream cofferdams would be required for river diversion and to keep Millerton Lake out of the construction zone. Diversion tunnels to route river flows around the construction zone would be required during construction.

Reservoir Storage and Area

The RM 279 dam site could support a reservoir with storage of up to 2.7 million acre-feet. Reservoir surface area could extend up to 9,300 acres. Curves showing potential new storage capacity and surface area for a reservoir at RM 279 are presented in **Figure 4-2**. Net storage volume accounts for existing storage capacity in Millerton, Kerckhoff, and Redinger Lakes.

Main Dam

The main dam at RM 279 would be either a concrete face rockfill or RCC gravity dam. Other dam types were also considered, as discussed below.

Dam Types Considered

The RM 279 site geology is suitable for concrete arch, RCC, or rockfill gravity dam types and foundation conditions at RM 279 are excellent for a concrete arch dam. However, the abutments are uniform with relatively flat slopes, resulting in a wide canyon that would require large volumes of concrete at substantial expense. Therefore, designs and cost estimates were not developed for a concrete arch dam at RM 279. A concrete thin arch dam could be considered in future studies for cost comparisons with the gravity dams.

A central-core earthfill dam is not considered economically viable due to the limited availability of plastic, fine-grained materials for the core. An asphaltic-core earthfill dam might be viable for the site but was not considered due to limited use and experience with this type of dam in the United States.

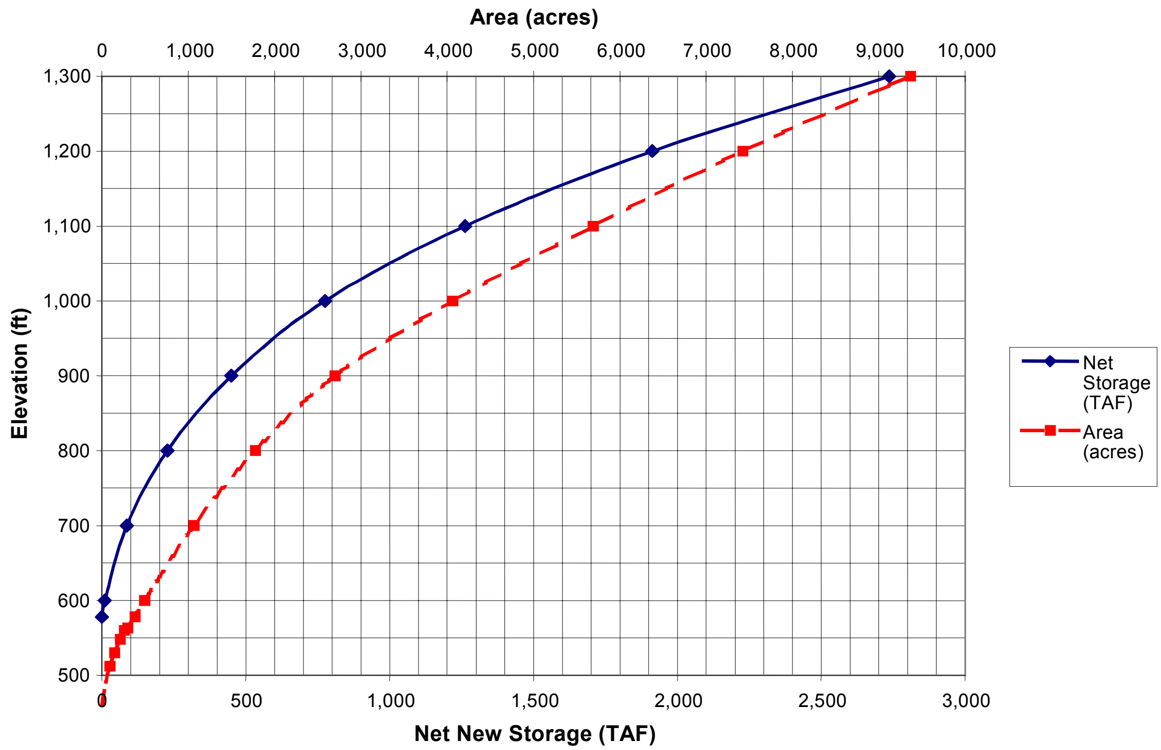

FIGURE 4-2.
TEMPERANCE FLAT RM 279 RESERVOIR SURFACE ELEVATION VS. NEW STORAGE AND AREA

Concrete Gravity Dam Design

The RCC gravity dam layout, including appurtenances, is shown in **Attachment B4**. A representative cross section of the RCC gravity dam is shown in **Figure 4-3** and also presented with additional details in **Attachment B1**. The design is based on standard practice, as described in Design of Gravity Dams (Reclamation, 1976). The design provides for a vertical upstream face and a 0.75H:1V downstream face. Preliminary stability analyses indicate that the design could be refined to employ a steeper downstream slope.

The mass of the dam would be constructed with RCC. The upstream and downstream faces of the dam would be covered with conventional concrete-facing elements to provide durable surfaces. Leveling concrete requirements were calculated for the dam foundation (an average thickness of 1 foot was assumed) and a conventional concrete cap would be provided on the dam crest. The dam crest width and details would be similar to the existing Friant Dam.

Foundation grouting would consist of a single curtain with an assumed spacing of 10 feet. A drainage gallery would be placed in the RCC above the high tailwater elevation. Drainage holes on 10-foot centers would be drilled from the gallery into the foundation, with additional drain holes drilled from the dam crest into the gallery. Depths of the grout and drainage holes that would be drilled into the foundation are shown in **Table 4-2** for the RCC dam, for which preliminary designs were developed.

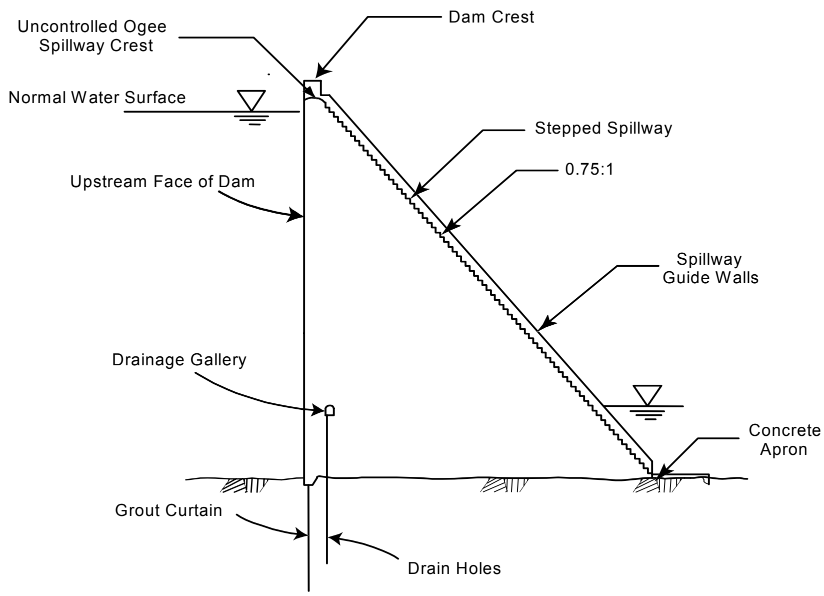

**FIGURE 4-3.
RCC DAM REPRESENTATIVE PROFILE FOR TEMPERANCE FLAT
RM 279 MEASURES**

**TABLE 4-2.
RCC DAM HOLE DEPTHS FOR TEMPERANCE FLAT RM 279**

Dam Crest Elevation (feet above msl)	Grout Hole Depth (feet)	Drainage Hole Depth (feet)
900	150	100
1,100	250	200
1,300	250	200

Key:
msl – mean sea level

Concrete Face Rockfill Dam Design

CFRF layouts, including appurtenances, are shown in **Attachment B4**. A cross section of the CFRF dam is shown in **Figure 3-3** in the RM 274 chapter, and also is presented with additional detail in **Attachment B1**. Design characteristics of a rockfill dam at RM 274 are described in **Chapter 3**. Similar characteristics and assumptions apply to the RM 279 dam design.

Preliminary Dam Sizes Evaluated

For the RM 279 dam site, preliminary designs and construction costs were developed for RCC and rockfill type dams with crests at elevations of 900, 1,100, and 1,300. With the streambed at elevation 460, corresponding dam heights range from 440 feet to 840 feet. Additional cost estimates were initially developed by interpolation at the line item level for dam crests at elevations of 960 and 1,200. An analysis of potential hydroelectric energy generation, documented in the separate **Hydropower TA**, was conducted for reservoirs with net storage volumes of approximately 725 TAF and 1,350 TAF. To be consistent with the hydropower analysis, additional cost estimates were developed by line item interpolation for dams with corresponding crest elevations. The full array of dam sizes for which construction costs were produced or hydropower evaluations were performed is shown in **Table 4-3**, along with the associated reservoir surface area and storage. Although a smaller dam size could be constructed, the requirement of constructing within the existing footprint of Millerton Lake limits the potential new storage that would be created at RM 279 with a smaller dam. Topography places an upper limit on the height of a dam that could be constructed before extensive saddle dams would be required to contain the reservoir.

TABLE 4-3.
SURFACE WATER STORAGE MEASURES EVALUATED FOR TEMPERANCE FLAT RM 279

Gross Pool Elevation (feet above msl)	Dam Height (feet)	Dam Types			Reservoir Area (acres)	Gross Storage Capacity (TAF)	New Storage Capacity (TAF)
		RCC	CFRF	Arch			
900	440	X	X		2,700	470	450
960	500	X	X		3,520	660	650
985	525	X	X		3,860	750	725
1,100	640	X	X		5,690	1,280	1,260
1,115	655	X	X		5,920	1,370	1,350
1,200	740	X	X		7,430	1,940	1,910
1,300	840	X	X		9,370	2,780	2,740

Key:
CFRF – concrete face rockfill dam
msl – mean sea level
RCC – roller-compacted concrete
RM – river mile
TAF – thousand acre-feet

Appurtenant Features

Preliminary, appraisal level designs for appurtenant structures were based on the assumption that Millerton Lake would be continuously operated within the approximate range of elevations 550 to 575. Storage at the RM 279 site could be gravity-fed into Millerton Lake, and downstream releases could use the existing conveyance system at Friant Dam.

Diversion Works

Diversion during construction for all surface water storage measures was based on passing a peak discharge of 65,000 cfs, which approximately corresponds to a 25-year return period. Diversion for the measures would be accomplished by constructing diversion tunnels through each abutment. A tunnel 30 feet in diameter would be constructed through the left abutment, and a 40-foot-diameter tunnel would be constructed through the right abutment.

The capacity of the left abutment tunnel would be about 25,000 cfs during construction, and would later serve as the outlet works for the dam. The capacity of the right abutment tunnel would be about 40,000 cfs. This tunnel would be plugged following construction.

Upstream and downstream cofferdams would be required to divert stream flows during construction and to prevent inundation of the site by Millerton Lake. Cofferdams would be sized for estimated diversion flows, and to allow normal operation of Millerton Lake during construction. The downstream cofferdam would need a crest at a minimum of elevation 578, and height of approximately 120 feet or more. The upstream cofferdam would require a crest at elevation 635 and a height of about 175 feet. A significant portion of both cofferdams would need to be constructed within the existing reservoir pool.

Spillway

The spillway for all surface water storage measures was based on passing a peak discharge of 145,000 cfs. This would be accomplished using an uncontrolled ogee crest spillway with a crest length of 450 feet, and a head of 20 feet.

For the CFRF measures, the spillway would be located on the right abutment. The downstream channel would be excavated through the existing rock abutment, and would daylight into a natural draw that leads back into the reservoir. To control flows within the vicinity of the dam, a reinforced concrete apron and training wall would be constructed within the first 100 feet upstream of the spillway crest and 200 feet downstream.

Energy would be dissipated by the tailwater at the end of the natural channel. Depending on the level of Millerton Lake, the plunge pool could be over 100 feet deep. For future designs, a labyrinth spillway should be considered for raising the crest elevation, providing more storage, and reducing the overall width of the spillway, including the outlet channel.

For the RCC dam measures, the spillway overflow section would be located near the center of the dam. Guide walls would be provided to contain flows within the width of the spillway crest. Energy dissipation would be accomplished as the flow passes over the stepped downstream face of the dam. A concrete cutoff at the toe of the dam would be provided to ensure undercutting does not occur..

Recent safety-of-dam studies indicate that Friant Dam can safely pass about 30 percent of the PMF before overtopping occurs (Reclamation, 2002d; 2002e). A risk assessment of the overtopping condition suggests that Friant Dam can withstand the depth and duration of overtopping without failure. A similar conclusion likely would hold true for an RCC dam at the RM 279 site. However, a rockfill dam would likely fail at this same threshold condition. Consequently, for purposes of this study, the spillway capacity was increased to 145,000 cfs at RM 279 (up from about 85,000 cfs for the existing Friant Dam spillway), to increase the threshold at which overtopping would occur. Future technical analysis would need to determine an appropriate inflow design flood for Temperance Flat Reservoir.

Outlet Works

The left abutment diversion tunnel would be converted to the outlet works and penstock. The outlet works layout for both dam types would consist of a trashracked intake structure, a water conveyance system, and a series of regulating gates with upstream guard gates. Energy from release flows would be dissipated in the tailwater from Millerton Lake (plunge pool). The size of the conveyance system would be dictated by diversion during construction, but normal reservoir operation requirements would control the size and number of gates. The designed capacity of the outlet works was set to closely match the capacity of the existing river and canal outlets at Friant Dam. Bulkheads would be required for the intake structure, and outlet gates within the upstream end of the tunnel also would be provided for dewatering. The control structure for the outlet works would be combined with the powerhouse.

A low-level outlet works with the capability of evacuating the reservoir below elevation 570 was not included in the preliminary designs. Reservoir levels below elevation 570 would be within the current operating pool of Millerton Lake, and could only be evacuated if Millerton Lake was drawn down below elevation 450. The need for a low-level outlet works should be considered in future studies. If considered, a tunnel through the abutment could be used for the CFRF measures, which would require placing the downstream cofferdam farther downstream to provide room for constructing the outlet end of the tunnel. A low-level outlet for the RCC dam measures could be constructed through the dam.

Powerhouse

For purposes of preliminary powerhouse design and cost estimation only, it was assumed that three turbines of equal size would operate within the head range and discharge capacity available during most of the year. Each turbine would operate independently within specific ranges of reservoir elevations. Assumptions regarding power generation and configurations of replacement power options are presented in the **Hydropower TA**.

The powerhouse and outlet works control structure would be located at the downstream portal of the left abutment diversion tunnel. During normal releases, all flows would pass through the turbines. During periods of significant inflow, the outlet works could be used to supplement releases in combination with the spillway, as necessary.

Saddle Dam

For a dam with a crest at elevation 1,300, a saddle dam would be required on the left abutment, downstream of the dam centerline. A narrow saddle in a ridge at this location has a low point at elevation 1,180, leading to the need for a 120-foot-tall saddle dam. Due to the ground surface sloping away from the centerline of the ridge, a saddle dam with a small footprint would be required to minimize the structure’s volume. Therefore, RCC construction was selected for the saddle dam, irrespective of the type of main dam constructed.

Construction Considerations

This section discusses issues of concern related to constructing a potential dam, reservoir, and appurtenant features at RM 279.

Foundations

Foundations for any of the RM 279 dam measures would be in sound granitic rock. As described in the Geologic and Seismic Setting section above, no foundation conditions of special concern are known at this time. Foundation preparation would be expected to be typical for all dams. Excavation for the concrete gravity dams was assumed to extend 10 feet deep to remove overburden and weathered bedrock.

Flood Routing During Construction

A peak discharge of 65,000 cfs with a return period of approximately 25 years was used to size the diversion structures for all Temperance Flat Reservoir surface water storage measures. Additional details are provided in **Chapter 3**.

Borrow Sources and Materials

Rockfill could be quarried from the reservoir area and obtained from excavations required for the dam and appurtenant structures. Earthfill is available in limited quantities. Low-plasticity, fine-grained soil has been identified in the reservoir area at Temperance Flat. Additional quantities of fine-grained soils might be available in the Auberry Valley area, and in an area south of Millerton Road near the Millerton Lake SRA entrance. Road cuts in Temperance Flat and the Auberry Valley expose decomposed to intensely weathered granite. Processed sands and gravels could be supplied by commercial sources and/or by crushing and processing quarried rock in the reservoir area, as could concrete aggregates.

Construction Site Access

Access to the dam site is across both public and private land. Currently, access to the left abutment is by gated Wellbarn Road (Marshal Station) and hiking trail. Access to the right abutment is via Road 210, private road, and jeep trail. Both abutments can be accessed by boat from Millerton Lake.

Staging Areas

Areas for construction use, staging, and lay-down likely would be located at Temperance Flat about 1.5 miles upstream from the dam site, or along the right side of the river about 0.5 to 1.0 miles upstream.

Lands and Rights-of-Way

A reservoir at RM 279 would inundate two residential properties in the Temperance Flat area. Private lands within the reservoir area would need to be acquired. Table 4-4 shows the total area that would be inundated by the RM 279 reservoirs considered and the amount of private land within that area that would need to be acquired. Rights-of-way or easements that may be needed to construct new facilities or relocate existing facilities have not been determined and are excluded from the land areas shown.

TABLE 4-4.
TEMPERANCE FLAT RM 279 RESERVOIR AREA LAND REQUIREMENTS

Description	RM 279 Reservoir Measures					
Gross pool elevation (feet above msl)	900	985	1,100	1,115	1,200	1,300
New storage capacity (TAF)	450	725	1,260	1,350	1,910	2,740
Estimated inundated area (acres)	2,318	3,476	5,307	5,540	7,042	8,982
Estimated public inundated acreage	2,087	2,766	3,957	4,150	4,932	6,440
Estimated private inundated acreage	231	710	1,350	1,390	2,110	2,542

Key:
msl – mean sea level
TAF – thousand acre-feet

Electric Power Sources

Electric power, including grid power, is available from the transmission facilities serving the PG&E Kerckhoff Project. Electric power, including lower voltage electric service, is available from existing trunks supplying local residences.

Utilities

Based on visual inspection of utility markers, no pipeline, communication, or power easements travel through this site. Overhead power lines, originating at the Kerckhoff and Kerckhoff No. 2 powerhouses, run just east of and parallel to Wellbarn Road, and just north of Marshall Station. Signs marking buried phone lines were observed near roads in Temperance Flat.

Relocations, Abandonments, or Modifications of Affected Facilities

Existing facilities that would be inundated by a reservoir at RM 279 are indicated in **Figure 4-1. Table 4-5** lists upstream facilities that would require abandonment or relocation (i.e., construction at a new location) for measures considered. Facilities that would require abandonment or relocation for a reservoir at elevation 1,300 include recreational facilities in the Millerton Lake SRA and San Joaquin River Gorge management area and at Kerckhoff Lake; an interpretive Native American village in the BLM management area; portions of paved and unpaved roads; Powerhouse Road Bridge; and hydropower facilities in the Temperance Flat area and above Kerckhoff Lake. Not all of these facilities would be inundated by a reservoir with a lower maximum surface elevation. Specific requirements for relocating or abandoning recreational and interpretive facilities were not determined. Requirements for relocating or abandoning residential property, roads, and hydroelectric facilities are described below.

TABLE 4-5.
RELOCATIONS REQUIRED FOR TEMPERANCE FLAT RM 279 MEASURES

<table>
<tr><th>Features Requiring Relocation, Modification, or Abandonment</th><th colspan="3">RM 279 Reservoir Measures</th></tr>
<tr><td>San Joaquin River Trail (SRA to SJR Gorge portion)</td><td rowspan="11">Elevation 900</td><td rowspan="15">Elevation 960</td><td rowspan="22">Elevations 1,100; 1,200; and 1,300</td></tr>
<tr><td>Temperance Flat Boat-In Campground</td></tr>
<tr><td>Temperance Flat Residences</td></tr>
<tr><td>Kerckhoff Powerhouse No. 2 Access Tunnel Entrance</td></tr>
<tr><td>SRA New Toilet Facility</td></tr>
<tr><td>Hewitt Valley Environmental Camps</td></tr>
<tr><td>Kerckhoff Powerhouse Main Floor</td></tr>
<tr><td>San Joaquin River Trail (SJR Gorge portion)</td></tr>
<tr><td>BLM Footbridge</td></tr>
<tr><td>Substation for Kerckhoff Powerhouse No. 2</td></tr>
<tr><td>Kerckhoff Dam outlet works</td></tr>
<tr><td colspan="2">Surge Chamber for Kerckhoff Powerhouse</td></tr>
<tr><td colspan="2">Kerckhoff No. 2 Powerhouse Intake</td></tr>
<tr><td colspan="2">Kerckhoff Powerhouse Intake</td></tr>
<tr><td colspan="2">BLM Native American Village (reproduction)</td></tr>
<tr><td colspan="3">Powerhouse Road Bridge</td></tr>
<tr><td colspan="3">Smalley Cove Campground</td></tr>
<tr><td colspan="3">Wishon Powerhouse</td></tr>
<tr><td colspan="3">Surge Chamber for Kerckhoff Powerhouse No. 2</td></tr>
<tr><td colspan="3">Big Creek Powerhouse No. 4</td></tr>
<tr><td colspan="3">BLM Primitive Campground</td></tr>
<tr><td colspan="3">Substation for Big Creek Powerhouse No. 4</td></tr>
<tr><td colspan="4">Key:
BLM – Bureau of Land Management
RM – river mile
SJR – San Joaquin River
SRA – State Recreation Area</td></tr>
</table>

Recreation Facilities

All surface water storage measures evaluated for RM 279 would inundate the Temperance Flat Boat-In Campground in the SRA. A reservoir at elevation 1,300 would inundate several miles of the San Joaquin River Trail, from the dam site to the primitive campground in the San Joaquin River Gorge and the portion of the trail that crosses the San Joaquin River footbridge. A reservoir at elevation 900 would inundate only slightly less of the trail, leaving a few disconnected segments.

A reservoir at elevation 1,100 or greater would inundate the primitive campground in the San Joaquin River Gorge off Smalley Road. A reservoir at elevation 960 would just inundate the BLM interpretive Native American village. A reservoir at elevation 1,100 or greater would submerge the BLM primitive campground. All recreation facilities at Kerckhoff Lake would be submerged by a reservoir at or above elevation 1,100, but would be unaffected by a reservoir at or below elevation 960. Specific requirements for relocating, replacing, or abandoning these recreational facilities have not been determined.

Roads and Bridges

Portions of roads and the Powerhouse Road Bridge could be inundated by a reservoir constructed at RM 279, as discussed further below. In total, for a reservoir with a gross pool at elevation 1,300, 7.5 miles of paved road and 11.5 miles of unpaved road would be inundated. A lesser amount would be inundated for lower reservoir elevations.

Auberry Road Spur Roads

The end portion of Smalley Road at Kerckhoff Powerhouse would be inundated by a reservoir at RM 279 and would be abandoned. The terminal portion of Spearhead Road that extends from Wellbarn Road to Temperance Flat also would be inundated by any reservoir at RM 279 and abandoned. For both roads, the amount of road affected would vary with the maximum reservoir elevation.

Powerhouse Road and Bridge

Powerhouse Road and Bridge would need to be relocated for a Temperance Flat Reservoir with a surface elevation of approximately 1,000 feet or greater.

Powerhouse Road

A reservoir at elevation 1,100 would require approximately 6,000 feet of new roadway construction and 3,400 feet of roadway improvements for Powerhouse Road. Appraisal level road relocation costs were developed for elevation 1,200, 1,300, and 1,400 reservoir levels. The elevation 1,200 and 1,300 reservoir levels are relevant to the RM 279 reservoirs. Drawings in **Attachment B5** refer to the road to be relocated as Auberry Road because it is the extension of Auberry Road on the Fresno County side of the river.

For reservoir elevations at 1,200 and greater, the cross section for Powerhouse Road relocations consists of two 12-foot travel lanes, with 4-foot wide shoulders on both sides, a 3-foot deep ditch at a slope of 3:1 in areas of excavation, and cut and fill slopes of 1.5:1. The vertical alignment would have a maximum grade of 7 percent, and the horizontal alignment was based on a minimum radius of curvature of 150 feet. The road surfacing comprises an 8-inch aggregate base course topped by 4 inches of hot bituminous pavement. A minimum roadway elevation of 1,240 was used, which is approximately 8 feet above the PMF. This measure would require construction of approximately 10,800 feet of new road and a 1,192-foot bridge, as well as approximately 8,300 feet of metal beam guardrails and culverts.

A minimum roadway elevation of 1,340 feet was used for the RM 279 elevation 1,300 reservoir, which is approximately 8 feet above the PMF. Approximately 17,300 feet of new road and a 1,307-foot bridge would need to be constructed, along with approximately 9,000 feet of metal beam guardrails and culverts.

Powerhouse Road Bridge

For relocating Powerhouse Road Bridge to accommodate a reservoir at elevation 1,100, it is assumed that an approach similar to that described below would be used. However, the length of bridge deck and height of bridge piers would be less. The bridge deck length would be approximately 730 feet. Appraisal level bridge designs were developed for elevation 1,200, 1,300, and 1,400 reservoir levels. The elevation 1,200 and 1,300 reservoir levels apply to the RM 279 storage measures.

The bridge alignment and profile were determined during the road relocation design. The bridge roadway profiles were set 40 feet above each of the three reservoir levels. The reservoir crossings are over deep canyons that require high piers, and overall bridge lengths of 1,192 feet, 1,307 feet, and 1,711 feet, respectively. Considering these two factors, and in an effort to minimize the number of piers and therefore minimize the amount of foundation work, a three-span, segmental post-tensioned bridge superstructure was selected for each of the reservoir crossings. The bridge superstructure would consist of a single-cell box girder, which is typically constructed by the balanced cantilever method, and the center span is generally twice the length of the end spans. The pier and abutment footings are assumed to be founded on rock and anchored to bedrock with rock bolts.

The Standard Specifications for Highway Bridges (AASHTO, 2002) and the American Segmental Bridge Institute (ASBI) govern this type of structure design and construction. The overall bridge width can accommodate two lanes of traffic, and therefore will be designed to carry two lanes of HS20-44 loading. The publication Construction and Design of Prestressed Concrete Segmental Bridges (Podolny and Muller, 1982) was used to optimize the span lengths and superstructure cross-section.

The box girder superstructure is post-tensioned in the longitudinal, vertical, and transverse directions. The bottom of the box girder superstructure follows a parabolic shape, and the box girder webs are vertical. The bridge clear width is 32 feet, set to match the road relocation width, which consists of two 12-foot travel lanes, with 4-foot shoulders on both sides. Bridge rails would be 2-feet, 8-inches high, and solid concrete, with New Jersey type rails.

Reclamation's Technical Service Center has previous experience in the design of this type of bridge substructure and superstructure. The design for the Roadway Realignment, Hoover Visitor Facilities, Boulder Canyon Project, Nevada, was issued in November 1988. The bridge design for the visitor's facility consisted of two 265-foot spans founded on rock. For this study, no actual bridge design was conducted; however, the layouts provided in **Figures 22** through **27** of **Attachment B5** generally show how a cast-in-place concrete segmental type of bridge can be designed and constructed to span the canyon as required for the Powerhouse Road Relocation. Drawings in **Attachment B5** refer to the Powerhouse Road Bridge as the Auberry Road Bridge, since Powerhouse Road is an extension of Auberry Road.

For the elevation 1,200 reservoir level, the bridge profile was set at elevation 1,240. The overall length of the bridge is approximately 1,192 feet. The center or main span is approximately 590 feet long, and the end or side spans are approximately 295 feet long. Box girder depths vary from approximately 28 feet, 6 inches, at the piers to 11 feet, 6 inches, at the abutments and midspan. Pier 1 and 2 heights are estimated to be 155 feet, and 90 feet respectively.

For the elevation 1,300 reservoir level, the bridge profile was set at elevation 1,340 feet. The overall length of bridge is approximately 1,307 feet. The center or main span is approximately 650 feet long, and the end or side spans are approximately 325 feet long. Box girder depths vary from approximately 31 feet, 6 inches, at the piers to 12 feet, 9 inches, at the abutments and midspan. Pier 1 and 2 heights are estimated to be 130-feet, and 180-feet respectively.

The only design data provided for the bridge layouts were profiles generated by Autodesk Land Desktop, 2004 software. No drill hole or foundation data were provided to determine the adequacy of the rock foundation. Quantities in the estimate worksheets are based on similar bridge designs, and were calculated based on ratios calculated using the varying widths, heights, and lengths of superstructure and substructure elements. For final design, exploration holes at each abutment and pier and seismic design criteria will be required.

Redinger Lake Road

A substantial portion of Redinger Lake Road would be inundated by a reservoir at elevation 1,100 or greater. It is assumed that the inundated portion, beginning at Kerckhoff Lake, would be abandoned. The portion of the road descending from Redinger Lake would remain.

Kerckhoff Powerhouses

Creation of a reservoir with a dam at RM 279 would submerge the Kerckhoff Powerhouse and Kerckhoff No. 2 Powerhouse. Abandoning these facilities is described in **Chapter 2**, related to a large raise of Friant Dam. A new multiple unit powerhouse that would use an extension of the Kerckhoff No. 2 diversion tunnel is proposed as a major component of one of the potential hydropower facility configurations being considered for a reservoir at RM 279 (i.e. Replacement Power Option 2). Site-specific technical requirements were not developed for constructing such a powerhouse. However, much of the general approach described in **Chapter 5** for constructing a new powerhouse near the existing Kerckhoff No. 2 Powerhouse also likely would apply for a location below RM 279. Analysis documented in the **Hydropower TA** indicates that a replacement powerhouse with three 40 MW generating units could be supported.

Kerckhoff Dam and Kerckhoff Project Diversion Facilities

Depending on the size of the dam, the reservoir pool behind a new dam at RM 279 could inundate existing Kerckhoff Dam outlets and gates and powerhouse intake structures at the dam. Surface water storage measures considered for RM 279 for a hydropower configuration involving a powerhouse at the RM 279 dam only would require decommissioning the outlet works and diversion intakes as described in **Chapter 3**.

The hydropower configuration involving a new powerhouse below RM 279 that discharges to Millerton Lake would make continued use of the Kerckhoff No. 2 Powerhouse diversion intake and tunnel. For this configuration, at reservoir elevations of 1,100 and greater, the intake structure would need to be modified. The description below pertains specifically to reservoir elevations of 1,200 and above, but the same approach likely would be used for a reservoir at elevation 1,100. Reservoir elevations of 1,100 and greater would increase the head in the diversion tunnel and require steel lining in a portion of the tunnel, as described in **Chapter 5**. Any Temperance Flat Reservoir measure with a maximum surface elevation greater than 985 also would submerge Kerckhoff Dam spillway gates and necessitate their removal during the construction period. Specific features identified for removal are discussed in **Chapter 3**.

Kerckhoff No. 2 Powerhouse Intake Modification

The new intake structure would be located just downstream of the original structure along the alignment of the existing tunnel to the Kerckhoff No. 2 Powerhouse, allowing access from the shoreline of the new reservoir at elevation 1,200 or 1,400. The intake structure supports operations and access to the new gates, which can be operated to close off flow to the tunnel for emergency or normal maintenance activities.

Flow through the intake structure is controlled by two 10.75-foot-wide by 24.5-foot-tall high-pressure gates that can be operated under balanced or unbalanced flow conditions. Cost estimates provided for this structure include the hydraulic control system and reinforced concrete structures necessary to operate and support loads from these gates under full reservoir levels.

To allow maintenance of the high-pressure gates without evacuating the reservoir, the design and estimates provide for two 10.75-foot-wide by 24.5-foot-high wheel-mounted gates that can be lowered into position under balanced flow conditions. The water downstream from the gates then can be evacuated, allowing full access to the high-pressure gates and downstream tunnel. Cost estimates include the necessary tracks, gate stems, hoists, reinforced concrete structures, and other miscellaneous equipment for operation and support of the wheel-mounted gates.

Access to the all of the gates would be provided by a vertical concrete-lined shaft excavated into rock extending from an elevation above the planned reservoir level (either elevation 1,200 or 1,400) to the invert of the existing tunnel. A concrete structure would be located at the top of the shaft to house the gate control systems and related equipment and the stairs and hatches for personnel or equipment access to the gates.

The existing tunnel to the Kerckhoff No. 2 Powerhouse would be modified by excavating the concrete lining and adjacent rock to allow construction of a concrete-lined transition from the current 24-foot-diameter tunnel to two rectangular conduits for the two high-pressure gates and two wheel-mounted gates in tandem. A center pier and thickened walls and floor are included in the design to separate the two sets of gates and provide necessary structural support for the gates.

Wishon Powerhouse

A reservoir at RM 279 with a surface at elevations 1,100 to 1,300 would submerge the Wishon Powerhouse and necessitate its decommissioning or relocation. Decommissioning of Wishon Powerhouse is described in **Chapter 3**. Relocation would involve decommissioning the existing powerhouse and constructing a new one at a higher elevation. Specific construction requirements for a new powerhouse were not determined; however, analysis documented in the **Hydropower TA** indicates that a generating capacity of 18 MW could be supported.

Big Creek No. 4 Powerhouse

A reservoir at RM 279 with a maximum surface at elevation 1,100 to 1,300 would submerge the Big Creek No. 4 Powerhouse. This would require decommissioning or relocating the powerhouse. Relocation would involve constructing a new powerhouse at a higher elevation and decommissioning the existing one. Requirements for decommissioning Big Creek No. 4 Powerhouse are presented in **Chapter 3**.

Replacement Powerhouse

For a RM 279 Reservoir at elevation 1,100, which would serve as the tailwater for a replacement powerhouse, analysis documented in the **Hydropower TA** indicates that a replacement powerhouse with up to 40 MW units could be supported. Specific requirements for constructing a replacement powerhouse that would discharge to a new Temperance Flat Reservoir at a gross pool elevation of 1,100 were not determined. However, a design for a replacement powerhouse for a reservoir at an elevation of approximately 1,250 is discussed below. The hydropower analysis suggests that a 30 MW powerhouse with a single generating unit could be supported at the higher elevation.

A replacement for the Big Creek No. 4 Powerhouse would be located approximately 2,500 feet downstream of Redinger Dam. This replacement powerhouse would tap into the existing 15-foot, 6-inch diameter steel penstock pipe that feeds the existing Big Creek No. 4 Powerhouse. This location provides an exposed section of steel pipe that crosses Willow Creek and thus facilitates a more direct and simple connection point to the existing pipe. This location would be suitable for a reservoir with a maximum surface elevation of 1,250 feet.

The replacement powerhouse shown in **Figures 18** through **21B** in **Attachment B5** is divided into two areas, the service bay and the unit bay. The unit bay houses one 13.4 MW generating unit. Water is conveyed to the unit's turbine via a 9-foot-diameter penstock that transitions from the existing 15-foot, 6-inch diameter steel outlet pipe from Redinger Dam.

Turbine selection and unit generation capacity were based on a discharge capacity of 1,000 cfs. Actual flow capacities through the existing Big Creek No. 4 Powerhouse pipeline are about 3,000 cfs. Based on potential flow capacities through the existing pipeline, the replacement powerhouse might support a design generation of 40 to 45 MW. Once actual pipeline flow capacity is verified, options for generation capacities larger than presented can be investigated.

No surge tank is included in this preliminary powerhouse design. The proximity of the turbine to the source reservoir (approximately 2,500 feet) is assumed to provide the necessary surge protection for this facility. Additionally, the penstock profile is believed not to contain knees, which could produce down surge problems. The exact plan and profile of the penstock are unknown, however. If this alternative proceeds to higher-level designs, the designers should perform surge analysis to confirm this assumption.

Turbine

The turbine for the replacement Big Creek No. 4 Powerhouse was sized based on the matrix of options and design data from MWH provided by fax on April 5, 2004. The powerhouse unit was sized with the following characteristics:

- Type – vertical shaft Francis
- Design Head – 280 feet
- Gross head range –150 to 350 feet
- Design flow rate – 1,000 cfs
- Unit speed – 300 revolutions per minute (rpm)
- Design output – 28,300 horsepower (hp)
- Tail water elevation – 1,200 feet minimum

Temperance Flat Reservoir would be the tailwater discharge for the turbine. The reservoir's potential water surface elevation variation is too great for the turbine; therefore, a tailwater weir would be constructed to elevation 1,200 to limit the head variation to a reasonable amount.

Generator

The main generator for the powerhouse was sized at 13,444 kilovolt-amperes (KVA) (12.1 MW), 6,900 volts, 60 cycle, and 276.9 rpms at a 90 percent power factor. The generator would be a vertical-shaft synchronous type with a static excitation system. The enclosure for the generator would be totally enclosed, water/air cooled (TEWAC), which would cool the generator by circulating internal air through an air-to-water heat exchanger. The generator would have control and protective devices to provide for alarm and shutdown of the unit under problem conditions.

Penstock

The powerhouse would use a portion of the existing Big Creek No. 4 Powerhouse penstock, which would be modified to align to the new replacement Big Creek No. 4 Powerhouse. Steel piping for the penstock would be designed in accordance with the following:

- Steel Water Pipe, A Guide for Design and Installation (AWWA)
- Manuals and Reports on Engineering Practice No. 79, Steel Penstocks (ASCE)

All piping is American Society for Testing and Materials (ASTM) A36 Steel plate. This material is a typical grade of steel used for fabricating steel manifolds, penstocks, and outlet works.

The cost estimate prepared for this design and layout assumes that no pipe fabricating shop exists in the vicinity of the various job sites, and any steel pipe sections 16 feet in diameter and greater would not be able to be shipped as a long cylinder. These sizes of pipe would be shipped in half or quarter sections and welded together and pressure-tested in the field.

Guard Valve

The turbine could be isolated from the dam with a 108-inch butterfly valve. Overhead crane access to the butterfly valve would be provided via a 15-foot by 24-foot access hatch. The guard valve estimated for the turbine is a butterfly valve. Large-diameter butterfly valves can be fabricated for the required design pressures. Because the maximum velocity through a butterfly valve should be about 16 feet per second, as recommended by AWWA C504 (AWWA), the size of the butterfly valve would be larger than the inlet to the turbine. The 108-inch diameter butterfly valve could be shop-fabricated and tested in the United States and be transported to the job site as a completed unit.

Structure and Site Layout

The powerhouse shown in **Figures 20** through **22** in **Attachment B5** is about 114 feet long and 70 feet wide in plan. The powerhouse is divided into two areas, the service bay and unit bay. The service bay and unit bay are each 57 feet long at the service yard elevation of 1,253.5. The superstructure extends 60 feet above the service yard to elevation 1,313.50. The substructure foundation elevation is about 1,177 along the unit bay, and elevation 1,160 below the sump at the service bay. All of the structure is assumed to be founded on competent rock.

The turbine setting, maximum and minimum tailwater surface elevations, and topography were used to set floor elevations and service yard elevations for the powerhouse. The minimum tailwater surface used for this layout is elevation 1,200. Due to topographical constraints at this site and Willow Creek, it was necessary to include a tailwater weir in the design to provide the minimum tailwater elevation of 1,200. Submergence requirements for the turbine set the centerline of the turbine at the minimum tailwater elevation of 1,200.

The maximum tailwater surface elevation used for this design is 1,250. The main service bay floor and gate deck are set at elevation 1,254. Bulkhead gates that isolate the draft tubes and turbine from the tailrace channel and river could be lowered and raised from the Gate Deck with a portable gate hoist. The draft tube invert is at elevation 1,181.60 and the draft tube bifurcates into two 9-foot wide sections with a center pier.

The powerhouse service yard is at elevation 1,253.5. No slopes or other features for yard drainage are included in this design and estimate. Clearances around the powerhouse of approximately 50 feet have been provided for truck and other equipment access (see **Figures 18** and **19** in **Attachment B5**). The service yard was oriented to accommodate a more direct alignment of the switchyard with the overhead power lines. Due to site topography and orientation of the service yard and access into the service yard, a large retaining wall is shown adjacent to the Gate Deck for this powerhouse. Access to the service yard has been provided on the northern end, which would provide the shortest and most direct access to the county road located just to the north and east of the service yard.

All service yard, switchyard, and structure excavation is assumed to be in rock. The amount of overburden is assumed to be negligible for the purposes of this level of design and estimate. The rock is assumed to be sound granite and a ½:1 permanent cut slope. No slope protection such as rock bolting or shotcrete was included. The switchyard is 115 feet wide by 180 feet long. The switchyard is shown with a separate fence. Access into and out of the switchyard is provided via two, 20-foot double swing gates.

Mechanical Equipment

Handling of equipment within the unit bay would be accommodated with a 55-ton overhead crane. A rotor set-down area is included in the service bay for assembly and disassembly of unit components. The crane capacity and rail elevation were established based on handling of the rotor/shaft assembly for the generator in the Unit and Service bays. Crane access to all four levels of the service bay would be provided via 10-foot by 14-foot access hatches. Personnel access would be provided via a stairwell and elevator within the service bay area. Equipment access into the powerhouse would be provided at the service bay with a 16-foot wide rollup door. Auxiliary mechanical systems are listed in **Attachment C5** in support of the cost estimate worksheets.

Electrical Equipment

The generator was described above; other components of the electrical system are described in this section.

Bus and Switchgear

A non-segregated phase bus rated at 15,000 volts, 1,200 amperes would transmit power from the generator to the unit breaker located inside the powerhouse and then the bus would continue outside the powerhouse to the switchyard. Several taps are provided off this bus inside the powerhouse to service the unit power transformer (PT) and surge protection cubicle, the power feed to the station service equipment, and the power feed to the unit static excitation system.

Switchyard and Transmission Line

New generation at the replacement Big Creek No. 4 Powerhouse would be 13.4 millivolt-amperes (MVA). It is assumed that the existing 230 kilovolt (kV) power line in the vicinity could accommodate the load from the new powerhouse. Therefore, no new transmission lines are proposed. A new switchyard would be required at the powerhouse. The new switchyard would include a transformer, circuit breaker, and disconnect switches.

Station Service

The double-ended secondary unit substation service power supply would be obtained by tapping off the main unit bus. The station service transformers would step down the voltage from 6,900 to 480Y/277 volts. A 480-volt distribution panel board would be provided as part of the station service equipment to service powerhouse loads inside the powerhouse. A 120/208 volt panel board also would be provided to service lighting, receptacles, and other low voltage powerhouse loads.

Unit Control Board

A duplex control switchboard would be used to control operation of the generating unit. The unit control board would provide controls for starting and synchronizing the main generator and for shutting down the unit. All selector switches, pushbuttons, indicating lights and all unit protective and control devices would be provided by the unit control board. Manual and supervisory control mode type functions would be provided. The supervisory mode would allow operation by a future supervisory control alarm, and data acquisition (SCADA) control system.

Auxiliary Control Boards

Auxiliary control boards would be provided in the powerhouse for operating all auxiliary systems such as hydraulic pumps, water cooling pumps, electrically driven valves, air compressors, etc.

Butterfly Valve Control Board

A control board would be provided for the hydraulically operated butterfly valve. All starters, selector switches, pushbuttons, indicating lights, and all protective and control devices for operation of the valve would be provided by the control board.

Big Creek Dam No. 7

Depending on the size of dam, the reservoir pool behind a new dam at RM 279 could back up to a portion of Big Creek Dam No. 7, which forms Redinger Lake and consequently is sometimes referred to as Redinger Dam. A reservoir at RM 279 with a maximum water surface elevation of 1,100 feet or less would not inundate any portion of the existing dam at Redinger Lake. However, the largest size reservoir considered at RM 279, with a maximum surface at elevation 1,300, would back most of the way up the downstream side of the dam and leave little opportunity to relocate the Big Creek No. 4 Powerhouse. Consequently, for a new reservoir at elevation 1,300 feet, Big Creek Dam No.7 would be inoperable. To accomplish this, the features identified below would be removed.

Spillway Features

Four radial gates exist through the center of the existing concrete arch dam. All of the gates and associated equipment would be removed. The concrete hoist decks and spillway piers (down to spillway crest) also would be removed. Spillway features to be removed would include the following:

- Four 40-foot by 30-foot Radial gates
- Four gate hoists
- Reinforced concrete spillway hoist decks
- Reinforced concrete spillway piers to elevation 1,373
- Reinforced concrete spillway chute walls

Outlet Works Features

The outlet works consists of two concrete-encased steel pipes extending through the dam near the right abutment. The outlets discharge directly into the power penstock. The outlet works equipment would be removed, and the penstock would be plugged downstream of the dam. The holes through the dam would be left open to ensure reservoir equalization and provide passage of flows when the reservoir level falls below the spillway crest elevation at Redinger Dam. This labor on the outlet works would only apply to RM 279 measures with a crest elevation greater than 1,250 that inundate Redinger Dam. For RM 279 measures with crest elevations of 1,250 or less, the outlet works and penstocks of Big Creek Dam No. 7 could still be used for furnishing water to a new powerhouse that would be constructed to replace Big Creek No. 4 Powerhouse. Outlet works features to be removed would include the following:

- Trashracks
- Two 8-foot by 17-foot, 8-inch fixed wheel gates
- Two sets of gate hoists
- Trashrack rake system
- Concrete plug

Sluiceway Features

The sluiceway passes through the dam and discharges along the spillway chute floor. The holes through the dam would be left open to ensure reservoir equalization and provide passage of flows when the reservoir level falls below the spillway crest elevation at Redinger Dam. The trashracks would be left in place to trap debris that could enter and plug the sluiceway. The sluiceway gates would be welded in the open position. This work on the sluiceway would only apply to storage measures with gross pool elevations greater than the crest elevation of Redinger Dam. For measures at lower gross pool elevations, the sluiceway would still be used and left in its current condition.

Construction Costs

Costs for constructing RM 279 surface water storage measures at July 2004 price levels are summarized in **Tables 4-6** and **4-7**. Costs shown for individual project components include field costs and indirect costs for planning, investigations, designs, and construction management. Indirect costs involved in constructing project components are 25 percent of field costs. Acquisition costs for lands in the reservoir area are incorporated into the construction cost of the main dam, along with an allowance of 20 percent of lands costs for indirect costs associated with property acquisition.

Construction costs are based on worksheets presented in **Attachments C1** and **C4**. Field costs for the dam and appurtenant features, previously calculated at July 2003 price levels, were adjusted to reflect July 2004 prices and a higher contingency. Costs for abandonments and relocations were developed in July 2004. All costs are at July 2004 prices.

Cost estimates for RM 279 measures were originally developed in Phase 1 of the Investigation for RCC and CFRF measures with dam crests at elevations 900, 1,100 and 1,300, based on quantities calculated from preliminary designs. Cost estimates for crests at elevations 960 and 1,200 were interpolated at the line item level from the three original estimates. From these preliminary cost estimates, it appears that RCC is the lower cost dam type up to elevation 1,100, after which the CFRF type becomes less expensive. Accordingly, costs for the apparent lower cost dam type are included in **Tables 4-6** and **4-7**. However, cost differences between dam types based upon these preliminary designs are not great enough to conclusively identify the most cost efficient design for all elevations at RM 279.

The **Hydropower TA** analyzes potential generation from surface water storage measures with approximate storage capacities of 725 TAF and 1,350 TAF. To compare energy generation with construction cost, the cost of developing a reservoir for each of these approximate sizes was derived. To accomplish this, the corresponding gross pool elevations of 985 and 1,115 were noted. The field cost of the lower cost dam type with a crest at the noted elevation was then determined by interpolation at the line item level from previously prepared cost estimates of dams at higher and lower elevations.

Tables 4-6 and **4-7** show construction costs for each of two potential powerhouse configurations considered for measures at RM 279: a multiple generating unit powerhouse at the RM 279 dam site (Replacement Power Option 1, **Table 4-6**); and a new multiple unit powerhouse at the end of an extended Kerckhoff No. 2 Powerhouse diversion tunnel, in combination with a small, single unit powerhouse at the RM 279 dam site (Replacement Power Option 2, **Table 4-7**). These potential configurations are discussed in greater detail in the section of this chapter on relocations, abandonments, and modifications of affected facilities. The **Hydropower TA** discusses these configuration options further and analyzes the potential hydroelectric energy generation of each.

Costs for decommissioning the Kerckhoff powerhouses are included in **Tables 4-6** and **4-7** for all RM 279 measures in both hydropower configurations. It is assumed that the powerhouses would be abandoned in place. The Kerckhoff Powerhouse diversion intake also would be abandoned.

All RM 279 storage measures also include the costs to remove Kerckhoff Dam outlet works and spillway gates. This is because the Kerckhoff Dam outlet works are at elevation 900 and would be submerged for all measures evaluated. Although it is recognized that use of Kerckhoff Dam spillway gates actually would be maintained for an elevation 900 measure, for convenience, the costs for removing the gates were bundled with the costs of disabling the outlet works.

Costs for relocating the Wishon and Big Creek No. 4 powerhouses are included for the elevation 1,100 measure. The cost to construct a replacement powerhouse for the Big Creek No. 4 Powerhouse below Redinger Dam was proportionally estimated based on the cost for by smaller replacement Wishon powerhouse. The cost for relocating the Wishon Powerhouse was included based on the cost of constructing an 18 MW powerhouse.Potential costs for reconstructing the Wishon Powerhouse at a specific higher elevation location were not based on designs. A value was based on the cost of constructing a similarly sized replacement Big Creek No. 4 Powerhouse. The cost to construct a replacement Wishon Powerhouse at a higher elevation would be similar to the cost to construct a 12 to 13 MW replacement Big Creek No. 4 Powerhouse at Redinger Dam. The costs to relocate Powerhouse Road and Bridge for a reservoir at elevation 1,100 were approximated as explained in **Chapter 3**. The cost for Powerhouse Road and Bridge relocations for a reservoir at elevation 1,115 were assumed to be similar to those used for elevation 1,100. More thorough cost estimates of the road and bridge relocations were developed explicitly for reservoirs at elevations 1,200 and 1,300, based on design considerations described in a prior section of this chapter.

Cost estimates for the two hydropower facility configurations differ in several respects. Replacement Power Option 1, whose costs are displayed in **Table 4-6**, involves a multiple unit powerhouse at the RM 279 dam site, at the end of the river diversion tunnel on the left abutment. For this configuration, the intake to the Kerckhoff No. 2 Powerhouse would be decommissioned and abandoned.

Replacement Power Option 2, whose costs are displayed in **Table 4-7**, involves extending the Kerckhoff No. 2 diversion tunnel to a new multiple unit powerhouse downstream of the RM 279 dam that would discharge to Millerton Lake. It also would involve a small powerhouse at the RM 279 dam. The cost of a multiunit 120 MW powerhouse and a 15 MW powerhouse at the RM 279 dam was approximated by adjusting the cost estimate prepared for a similar powerhouse configuration near the existing Kerckhoff No. 2 Powerhouse that was developed for RM 286 measures. The adjustment was made by scaling the cost according to the square root of the ratio of the respective generating capacities.

TABLE 4-6.
CONSTRUCTION COSTS FOR TEMPERANCE FLAT RM 279 MEASURES WITH REPLACEMENT POWER OPTION 1 ($ MILLION)

Gross Pool Elevation (feet above msl)	900	985	1,100	1,115	1,200	1,300
New Storage Capacity (TAF)	450	725	1,260	1,350	1,910	2,740
Dam Type	RCC	RCC	CFRF	CFRF	CFRF	CFRF
Storage Components						
RCC Dam, Spillway, Outlet Works, River Diversion, Reservoir Lands	450	650	890	940	1,200	1,550
Abandon Kerckhoff Powerhouse	2	2	2	2	2	2
Abandon Intake for Kerckhoff Powerhouse	1	1	1	1	1	1
Abandon Kerckhoff No. 2 Powerhouse	2	2	2	2	2	2
Abandon Intake for Kerckhoff No. 2 Powerhouse	1	1	1	1	1	1
Remove Kerckhoff Dam Outlet Works and Gates	2	2	2	2	2	2
Abandon Wishon Powerhouse	-	-	2	2	2	2
Abandon Big Creek No. 4 Powerhouse	-	-	4	4	4	4
Powerhouse Road Relocation	-	-	18	18	18	40
Powerhouse Road Bridge Relocation	-	-	21	21	34	38
Construction Cost, Storage Components	**458**	**658**	**943**	**993**	**1,266**	**1,642**
Replacement Power Components						
New Powerhouse at RM 279 Dam (120 MW)	210	210	210	210	210	210
New Wishon Powerhouse (18 MW)	-	-	46	46	46	46
New Big Creek No. 4 Powerhouse (30 to 80 MW)	-	-	115	115	69	69
Construction Cost, Replacement Power Components	**210**	**210**	**371**	**371**	**325**	**325**
Construction Cost[1, 2]	**668**	**868**	**1,314**	**1,364**	**1,591**	**1,967**

Key:
CFRF – concrete face rockfill
RCC – roller-compacted concrete
msl – mean sea level
RM – river mile
MW – megawatt
TAF – thousand acre-feet

Notes:
[1] All cost estimates are preliminary. Construction cost represents the sum of field costs and indirect costs for planning, engineering, design and construction management, estimated at 25 percent of field costs.
[2] Costs do not include environmental mitigation, new or relocated recreation facilities, acquisition of impacted power facilities, or compensation for lost future power generation.

TABLE 4-7.
CONSTRUCTION COSTS FOR TEMPERANCE FLAT RM 279 MEASURES WITH REPLACEMENT POWER OPTION 2 ($ MILLION)

Gross Pool Elevation (feet above msl)	900	985	1,100	1,115	1,200	1,300
New Storage Capacity (TAF)	**450**	**725**	**1,260**	**1,350**	**1,910**	**2,740**
Dam Type	**RCC**	**RCC**	**CFRF**	**CFRF**	**CFRF**	**CFRF**
Storage Components						
RCC Dam, Spillway, Outlet Works, River Diversion, Reservoir Lands	450	650	890	940	1,200	1,550
Abandon Kerckhoff Powerhouse	2	2	2	2	2	2
Abandon Intake for Kerckhoff Powerhouse	1	1	1	1	1	1
Abandon Kerckhoff No. 2 Powerhouse	2	2	2	2	2	2
Remove Kerckhoff Dam Outlet Works and Gates	2	2	2	2	2	2
Abandon Wishon Powerhouse	-	-	2	2	2	2
Abandon Big Creek No. 4 Powerhouse	-	-	4	4	4	4
Powerhouse Road Relocation	-	-	18	18	18	40
Powerhouse Road Bridge Relocation	-	-	21	21	34	38
Construction Cost, Storage Components	**457**	**657**	**942**	**992**	**1,265**	**1,641**
Replacement Power Components						
New Powerhouse at RM 279 Dam (15 MW)	75	75	75	75	75	75
New Powerhouse on Extended Kerckhoff No. 2 Tunnel (120 MW)	150	150	150	150	150	150
Kerckhoff No. 2 Diversion Tunnel Extension	120	120	120	120	120	120
Kerckhoff No. 2 Diversion Tunnel, Steel Liner	-	-	45	45	85	125
Kerckhoff No. 2 Diversion Tunnel, Backfill Concrete	-	-	3	3	3	3
Modify Kerckhoff No. 2 Diversion Intake	-	-	18	18	33	39
New Wishon Powerhouse (18 MW)	-	-	46	46	46	46
New Big Creek No. 4 Powerhouse (30 to 80 MW)	-	-	115	115	69	69
Construction Cost, Replacement Power Components	**364**	**364**	**591**	**591**	**600**	**646**
Construction Cost[1, 2]	**802**	**1,002**	**1,514**	**1,564**	**1,846**	**2,268**

Key:
CFRF – concrete face rockfill
RCC – roller-compacted concrete
msl – mean sea level
RM – river mile
MW – megawatt
TAF – thousand acre-feet

Notes:
[1] All cost estimates are preliminary. Construction cost represents the sum of field costs and indirect costs for planning, engineering, design and construction management, estimated at 25 percent of field costs.
[2] Costs do not include environmental mitigation, new or relocated recreation facilities, acquisition of impacted power facilities, or compensation for lost future power generation.

The cost to extend the Kerckhoff No. 2 diversion tunnel was roughly calculated by applying a unit cost per mile of tunnel to the required extension length. For reservoirs extending to elevation 1,100 and higher, the additional pressure head would likely require portions of the existing Kerckhoff No. 2 diversion tunnel to be lined with steel pipe. These costs are therefore included for Replacement Power Option 2. Additional study would be needed to determine the costs to relocate recreation facilities and to carry out any required environmental mitigation. These potential costs are not included in the totals shown.

For all dam crest elevations, the dam and appurtenant structures would be located on public land. Parcels of land immediately upstream from the construction area and in the potential area of inundation are privately owned and would need to be acquired. Approximate property costs are included in the construction costs shown in **Tables 4-6** and **4-7**. However, the cost to acquire any hydroelectric assets that would be abandoned, or to compensate their owners for any loss of future energy generation, was not included in the table. **Figure 4-4** shows the relationship between new storage capacities that would be developed with a reservoir at RM 279 versus construction cost for both replacement power options. Costs increase substantially once costs are incurred to replace the generation capacity of Big Creek No. 4 Powerhouse and Wishon Powerhouse, and to relocate Powerhouse Road and Bridge to a higher elevation.

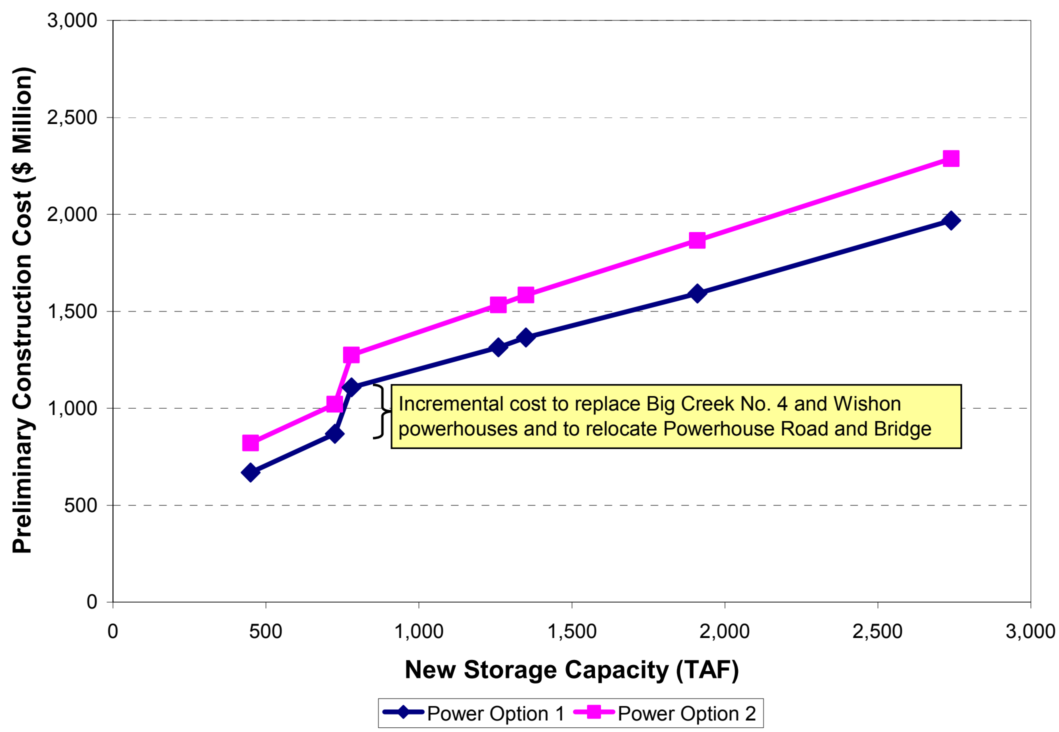

FIGURE 4-4.
CONSTRUCTION COSTS FOR TEMPERANCE FLAT RM 279 MEASURES VS. NEW STORAGE CAPACITY

CHAPTER 5. TEMPERANCE FLAT RESERVOIR AT RM 286

This chapter describes structures and costs involved in developing a reservoir at RM 286 of the San Joaquin River in the Temperance Flat area. This chapter is structured similarly to **Chapters 2** through **4.** It describes site conditions, engineering considerations associated with design and construction of the dam and appurtenant features, relocations or abandonments of existing facilities, and required land acquisitions. More detailed descriptions of existing and potential new hydroelectric generation facilities are included in the **Hydropower TA**.

PREVIOUS STUDIES

No studies of the RM 286 dam site are known to have been conducted prior to this Investigation. Previous studies of a potential reservoir at Temperance Flat are described in **Chapter 3**.

SITE DESCRIPTION

The RM 286 potential dam site is located on the San Joaquin River, upstream of Millerton Lake, approximately 13 river miles upstream of the Fine Gold Creek confluence and 5 river miles above Temperance Flat. **Figure 1-1** shows the general location of the Temperance Flat area sites and **Figure 3-1** shows the location of the RM 286 site in relationship to the RM 274 and RM 279 sites.

The RM 286 site is in the portion of the San Joaquin River that is bypassed by diversions at Kerckhoff Dam to the Kerckhoff powerhouses. Site characteristics and design considerations differ from those for RM 274 and RM 279.

Topographic Setting

The RM 286 site rises uniformly and steeply from elevation 740 in the original San Joaquin River channel at RM 286.1 to elevation 1,400 on the left abutment, and then rises at a slightly less steep slope to about elevation 1,600 before continuing steeply again to elevation 2,100. The right abutment rises steeply and uniformly from the river channel to over elevation 2,000 at an unnamed mountain.

Geologic Setting

Both abutments and the channel section are mostly granite and granodiorite, with alluvium in the channel section. The granite is typically hard to very hard where exposed on steep slopes, in the bottom of drainages, along the San Joaquin River shoreline, and in drill holes in the area.

Site Geotechnical Conditions

Alluvium of unknown thickness occurs in the San Joaquin River channel. The alluvium probably ranges from fine- to coarse-grained, with rock blocks up to 50 feet in maximum dimension that detached from the abutment slopes. No unstable wedges, toppling, or slides were observed at the RM 286 site. The granitic bedrock would provide an excellent dam foundation. It has adequate strength and stability for embankment, rockfill, concrete gravity, or concrete arch structures and any river diversion feature. The granite also would provide an adequate foundation for a plunge pool or overflow spillway.

No specific geologic information was available for the powerhouse site, which would be downstream of the RM 286 dam at the end of the river diversion tunnel that passes through the right abutment. However, geologic conditions are very consistent throughout the proposed dam site area and for a distance of a few miles upstream and downstream of the dam site. Granite dominates the subsurface.

The granitic rock that would serve as the dam foundation is fresh to slightly weathered, slightly to very slightly fractured, hard to very hard, with very few and very widely spaced shears that typically have been healed with quartz. These characteristics are revealed by extensive surface outcrops, core from two drill holes in the area, and exposures at Kerckhoff No. 2 Powerhouse and its adits. Core samples were obtained from drill holes on the left abutment, one along the projection of the dam axis at an elevation of nearly 2,100 feet and another at elevation 1,120, approximately 1,500 feet downstream of the dam axis.

Permeability tests on the core samples show the rock mass to be extremely impermeable below a depth of 20 to 30 feet (Reclamation, 2005; contained in **Attachment E**).

Seismic Hazard Analysis

No known faults exist at the RM 286 site or in the vicinity (Reclamation, 2002b). Areal sources were found to be the controlling source of potential earthquakes for these and greater return periods. Mean PHAs calculated for selected return periods for the Kerckhoff Dam vicinity are described in **Chapter 2**.

Existing Facilities

Constructed facilities in and above the Temperance Flat area include recreation facilities, roads, the PG&E Kerckhoff Hydroelectric Project, PG&E Wishon Powerhouse, and the SCE Big Creek No. 4 Powerhouse. **Table 5-1** lists facilities that could be inundated or affected by a reservoir at RM 286. Roads, varying considerably in elevation and location, are excluded from the table. Facilities that could be inundated by a reservoir constructed at RM 286 also are indicated in **Figure 5-1**. The figure shows the maximum reservoir size contemplated for the site (i.e., the reservoir resulting from a dam with crest at elevation 1,400). No known residences exist in the potential inundation area.

**TABLE 5-1.
FACILITIES POTENTIALLY AFFECTED BY TEMPERANCE FLAT
RM 286 MEASURES**

Approximate Elevation (feet above msl)	Approximate Location (SJ River Mile)	Facility
778	283	Substation for Kerckhoff Powerhouse No. 2
889	292.5	Base of Kerckhoff Dam
900	292.5	Kerckhoff Dam Outlets
921	284.5	Surge Chamber for Kerckhoff Powerhouse
939	301.5	Kerckhoff No. 2 Powerhouse Intake
942	301.5	Kerckhoff Powerhouse Intake
971	292.5	Kerckhoff Dam Crest
980	295	Powerhouse Road Bridge
980	295.5	Smalley Cove Campground, Picnic Area, Boat Launch
992	294.5	A.G. Wishon Powerhouse
993	283	Surge Chamber for Kerckhoff Powerhouse No. 2
1,004	296	Big Creek Powerhouse No. 4
1,089	296	Substation for Big Creek Powerhouse No. 4
1,181	301.5	Base of Big Creek No. 7 Dam (Redinger Dam)
1,403	301.5	Crest of Big Creek No. 7 Dam (Redinger Dam)

Key:
msl – mean sea level
RM – River Mile
SJ – San Joaquin

Recreation Facilities

Developed recreation facilities at Kerckhoff Lake include a car-top boat launch, a day-use area, and a campground at Smalley Cove. Recreational facilities in the Millerton Lake SRA and San Joaquin River Gorge are downstream of the potential dam site.

Roads

Powerhouse Road and Bridge cross Kerckhoff Lake. Redinger Lake Road connects Kerckhoff Lake to Redinger Lake. These roads are more fully described in **Chapter 3**.

Hydroelectric Energy Facilities

The Kerckhoff No. 2 Powerhouse and Kerckhoff Powerhouse are located downstream of RM 286 but are fed by upstream diversion facilities at Kerckhoff Lake. The Wishon Powerhouse discharges to Kerckhoff Lake and is fed by diversions from Corine Lake, northeast of Kerckhoff Lake. Big Creek No. 4 Powerhouse discharges to the San Joaquin River slightly above Kerckhoff Lake and is fed by diversions from Redinger Lake. These facilities are described more fully in **Chapters 2** and **3**. Additional details can be found in the **Hydropower TA**.

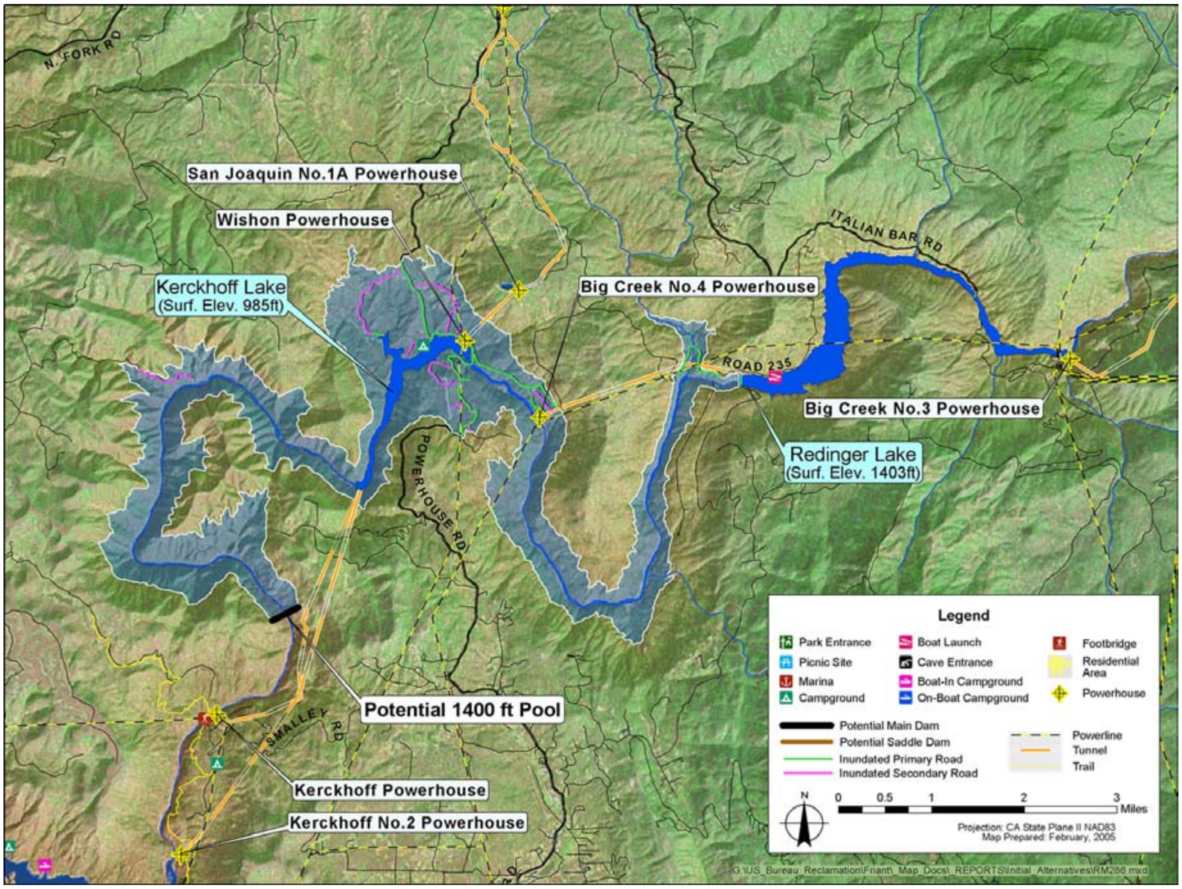

FIGURE 5-1.
POTENTIAL TEMPERANCE FLAT RM 286 RESERVOIR

POTENTIAL IMPROVEMENTS

Permanent features that would be constructed to develop a reservoir at RM 286 include a main dam with an uncontrolled spillway to pass flood flows, a powerhouse to generate electricity, and river outlet works for other controlled releases. Upstream and downstream cofferdams and diversion tunnels would be required for river diversion around the construction zone.

Reservoir Storage and Area

The RM 286 dam site could support a reservoir with storage of nearly 1.4 million acre-feet. Reservoir surface area could extend to more than 6,000 acres. Curves showing potential new storage capacity and surface area for a reservoir at RM 286 are presented in **Figure 5-2**. Net storage volume accounts for existing storage capacity in Kerckhoff and Redinger Lakes.

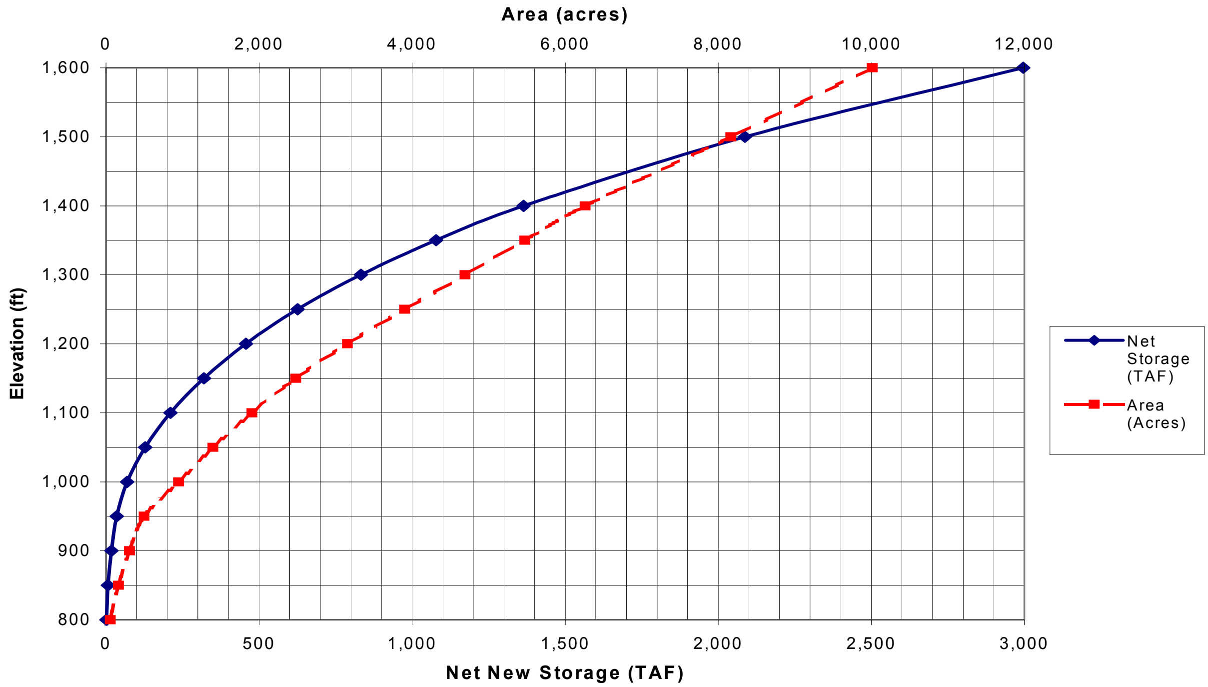

FIGURE 5-2.
TEMPERANCE FLAT RM 286 RESERVOIR SURFACE ELEVATION VS. NEW STORAGE AND AREA

Main Dam

Preliminary layouts and cost estimates were prepared for RCC, CFRF, and concrete arch dam types at RM 286. Other dam types were also briefly considered.

Dam Types Considered

This site is appropriate for concrete arch and RCC or rockfill gravity dam types. A central-core earthfill dam is not considered economically viable due to the limited availability of plastic, fine-grained materials for the core. An asphaltic-core-earthfill dam might be viable for the site but this type was not pursued due to limited use and experience with this type of dam in the United States.

Concrete Arch Dam

Concrete arch dam layouts, including appurtenances, are shown in **Attachment B5** and a representative cross section is shown in **Figure 5-3**. The design is based on standard practice as described in Reclamation's manual, Design of Arch Dams (Reclamation, 1977). Layouts for a dam with a crest elevation 1,200 or 1,400 were developed in 2003. The layout for the dam with a crest at elevation 1,300 was developed and refined in 2004.

Figure 5-3 shows the crown cantilever section of a double curvature arch dam. The dam would be constructed with conventional mass concrete with provisions for initial and final cooling and contraction joint grouting. Individual concrete blocks would be placed in approximate 10-foot lifts within forms. The formed surfaces of the upstream and downstream faces of the dam would provide a more durable surface. Leveling concrete requirements were calculated for the dam foundation (an average thickness of 1 foot was assumed) and a conventional structural concrete cap would be provided for the dam crest. Details such as curbs and parapets on both faces of the dam crest could be included in the design once requirements for access to the dam crest were established.

Foundation grouting would consist of a single curtain with an assumed spacing of 10 feet. A drainage gallery would be placed in the arch about 20 feet above the foundation. Drainage holes on 10-foot centers would be drilled from the gallery into the foundation. Depths of the grout and drainage holes drilled into the foundation are shown in **Table 5-2**. Formed drains would be located near the upstream face of the dam on 10-foot centers (not shown on drawings).

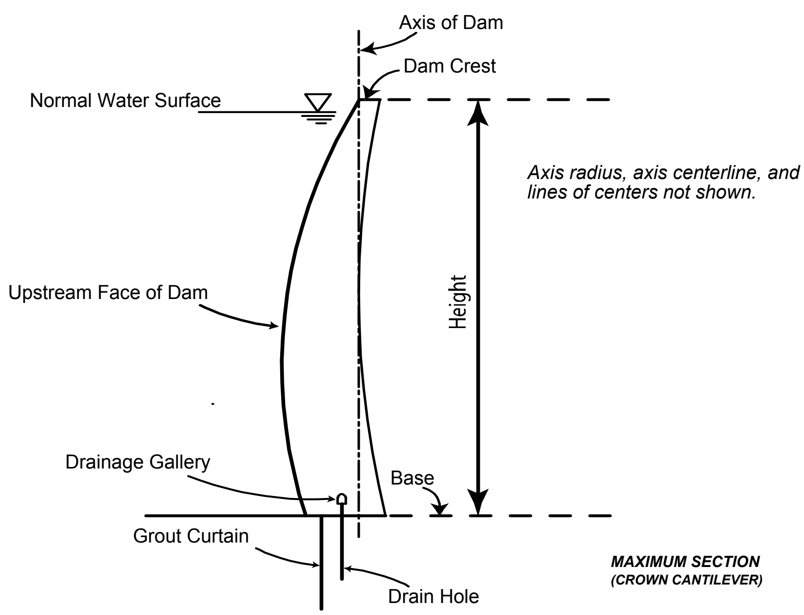

FIGURE 5-3.
ARCH DAM REPRESENTATIVE PROFILE FOR TEMPERANCE FLAT RM 286 MEASURES

TABLE 5-2.
ARCH DAM HOLE DEPTHS FOR TEMPERANCE FLAT RM 286

Dam Crest Elevation (feet above msl)	Grout Hole Depth (feet)	Drainage Hole Depth (feet)
1,200	150	100
1,300	200	150
1,400	250	200

Key:
msl – mean sea level

An initial static stability analysis of the dam was performed for gravity and reservoir loadings using the computer program ADSAS. Results of this study indicated that high arch and cantilever stresses were present in the lower portion of the dam. The geometry was refined until the stresses satisfied Reclamation Bureau criteria for normal loads. The result was a thickening of the lower portions of the dam to reduce stress levels in this area. Design and analysis of the RM 286 elevation 1,300 dam was reviewed by an outside expert in arch dam layout and analysis, and found to be satisfactory for this level of design.

Subsequent to refining the arch shape, the layout was analyzed for the effects of temperature loading to determine the appropriate grouting temperature. Temperature loads were generated using the DAMTEMP program based on data taken from a site in the vicinity of the dam site. Results indicated that the design is adequate for a grout temperature between 40^{o} and 50^{o} F.

Layouts for arch dams with crests at elevations 1,200, 1,300, and 1,400 are contained in **Attachment B5.** The layout for the elevation 1,300 arch dam was developed and refined in 2004. The layouts for the dams with crests at elevations 1,200 and 1,400 are from 2003 and do not reflect refinements made in 2004; they assumed an excavation depth of less than 25 feet. To ensure that cost estimates for the three arch dam sizes are consistent, revised quantities and cost estimates have been developed for the elevation 1,200 and 1,400 dams based on the approach taken with the elevation 1,300 arch dam.

Concrete Gravity Dam

Layouts for RCC gravity dam measures, including appurtenances, are shown in **Attachment B5**. A representative cross section is shown in **Figure 4-3**. The design is based on standard practice as described in Design of Gravity Dams (Reclamation, 1976).

For a dam with crest at elevation 1,300, a straight RCC gravity dam was designed with a 20-foot-wide crest (typical minimum width for RCC dams to allow adequate room for construction equipment), a vertical upstream face, and a downstream face sloping 0.75:1.0 (horizontal to vertical). The downstream slope starts at the upstream edge (axis) of the dam at elevation 1,300. From experience with other Reclamation gravity dams in similar seismic zones, this is an appropriate shape for stability of a straight gravity dam for all loading conditions. In final design, a more efficient shape and more economical design may be obtained by curving the dam in plan and making the downstream face steeper.

The mass of the dam would be constructed with RCC. The RCC mix was estimated with 300 pounds per cubic yard (cy) of cementitious material consisting of 60 percent fly-ash and 40 percent Portland cement. RCC would be placed in 2-foot-high lifts. A half-inch-thick layer of cement mortar was included in the unlisted items and is planned on every RCC lift to improve bond and reduce potential seepage paths through the dam. A 2-foot-thick conventional concrete along the upstream and downstream faces of the dam to provide a more durable exposed surface was included. This concrete would be a structural mix vibrated into place. The face of the dam could be shaped with metal or wood forms or slip-formed with a paving machine.

A conventional concrete cap was included on the dam crest along with spillway guide walls. Crack inducers (metal sheets) at every other RCC lift would be placed from upstream to downstream on 50-foot centers across the canyon to form contraction joints. This would minimize future thermal-induced cracking of the dam from seasonal temperature changes and cooling from heat-of-hydration temperatures as the concrete cures. This cost was included in the unlisted items. Polyvinyl chloride (PVC) water stops embedded along the contraction joints along the upstream side for seepage control would be installed and were included in the unlisted items. A 1-foot-thick layer of structural leveling concrete (six-bag mix) was assumed along the dam-to-foundation contact to smooth irregularities and make a relatively smooth surface for the dam to bear against.

A 4-foot-wide by 7-foot-high drainage gallery was included in the unlisted items and is planned across the entire width of the dam 20 feet from the dam to foundation contact. Foundation drains (4-inch-diameter) would be drilled from the gallery into the foundation on 10-foot centers across the dam. For the elevation 1,300 dam measure, the deepest drain hole would be 120 feet, which is 20 percent of the height of the dam. A foundation grout curtain was included upstream from the foundation drains by drilling 2-inch-diameter holes from the gallery into the foundation on 10-foot centers across the dam and then pressure-grouting the holes with cement grout (assumed 1 sack of cement per foot of hole depth). The deepest grout hole would be 240 feet, which is 40 percent of the height of the dam. Internal drains (6-inch-diameter) were included in the unlisted items and planned in the dam at about 15 feet from the upstream face, drilled vertically from the crest into the drainage gallery on 10-foot centers across the dam. Grout and drain hole depths for a RM 286 dam at all three crest elevations are shown in **Table 5-3**.

TABLE 5-3.
RCC DAM HOLE DEPTHS FOR TEMPERANCE FLAT RM 286

Dam Crest Elevation (feet above msl)	Grout Hole Depth (feet)	Drainage Hole Depth (feet)
1,200	150	100
1,300	240	120
1,400	250	200

Key:
msl – mean sea level

Previous layouts for the elevation 1,200 and 1,400 RCC dams were for a lesser depth of excavation. Revised quantities and cost estimates for these dams based on the additional excavation assumed to be required have been developed. This was done to ensure that cost estimates for the three RCC dam sizes are consistent.

Concrete Face Rockfill Dam

CFRF layouts, including appurtenances, are shown in **Attachment B5**. A representative cross section for a CFRF dam is shown in **Figure 3-1** and **Attachment B1**. The design is based on standard practice as described in Concrete-Face Rockfill Dam: II. Design (Cook and Sherard, 1987). A description of design characteristics and assumptions for a CFRF structure is presented in **Chapter 3**.

Preliminary Dam Sizes Evaluated

Dam sizes at RM 286 for which construction cost estimates were produced are shown in **Table 5-4**, along with associated reservoir surface areas and storage volumes.

**TABLE 5-4.
SURFACE WATER STORAGE MEASURES EVALUATED
FOR TEMPERANCE FLAT RM 286**

Gross Pool Elevation (feet above msl)	Dam Height (feet)	Dam Types			Reservoir Area (acres)	Gross Storage Capacity (TAF)	New Storage Capacity (TAF)
		RCC	CFRF	Arch			
1,200	460	X	X	X	3,150	460	460
1,275	535	X	X	X	4,280	740	725
1,300	560	X	X	X	4.690	860	830
1,400	660	X	X	X	6,260	1,400	1,360

Key:
CFRF – concrete face rockfill dam
msl –mean sea level
RCC – roller-compacted concrete
RM – river mile
TAF – thousand acre-feet

For CFRF dam measures, designs and cost estimates for dam crests at elevations 1,200, and 1,400, and a line item interpolation for a dam crest at elevation 1,300 were developed in July 2003.In July 2004, designs and cost estimates for RCC and concrete arch dam types with a crest at elevation 1,300 were refined to a higher degree of detail. Cost estimates for RCC and arch dam types with crest at elevations 1,200 and 1,400 were also revised for consistency. Costs for an RCC dam with crest at elevation 1,275 that would provide net storage of approximately 725 TAF were calculated by interpolating the July 2004 estimates of contract cost for dams with crests at elevations 1,200 and 1,300.

Dam sizes are constrained more by the presence of existing facilities than by topography. While a smaller dam could be constructed, only a reservoir with a storage capacity less than 100 TAF could avoid inundating the Big Creek No. 4 and Wishon powerhouses. **Figure 5-1** shows the extent of the maximum reservoir size examined (i.e., the reservoir that would result from a dam with a crest at elevation 1,400).

Appurtenant Features

Preliminary designs for the appurtenant structures were based on the assumption that releases from RM 286 would be controlled to keep Millerton Lake at a relatively constant level. Outlets were generally sized to match the current combined outlet capacity (including canal outlet works) at Friant Dam, which is about 22,000 cfs. This capacity would satisfy current Reclamation reservoir evacuation guidelines. Storage at the RM 286 site could be gravity fed into Millerton Lake, and downstream releases could be made using the existing conveyance systems at Friant Dam.

Diversion Works

Diversion during construction for all reservoir measures was based on passing a peak discharge of 65,000 cfs, which corresponds to an approximate 25-year return period. Diversion for the dam would be accomplished by constructing diversion tunnels through each abutment. A tunnel 30 feet in diameter would be constructed through the right abutment, and a 40-foot-diameter tunnel would be constructed through the left abutment. The capacity of the right abutment tunnel would be about 25,000 cfs during construction. The tunnel would later serve as the conduit to the dam's outlet works and powerhouse. The capacity of the left abutment tunnel would be about 40,000 cfs. This tunnel would be plugged following construction.

Upstream and downstream cofferdams would be required for diverting stream flows during construction and preventing tailwater from entering the construction area. Cofferdams would be sized for estimated diversion flows. The downstream cofferdam would have a crest at about elevation 770, and height of about 30 feet. The upstream cofferdam crest would need to be placed at elevation 850 to provide sufficient head to pass the diversion flood, resulting in a cofferdam approximately 110 feet high.

It might be possible to use the existing Kerckhoff Dam upstream to store some of the diversion floodwater during construction and reduce the size of the upstream cofferdam at the site, but this concept was not examined closely.

Future designs should consider reducing the size of the diversion flood for concrete dam measures. The rock foundation is extremely resistant to erosion, and overtopping the concrete dams during construction would not have a significant impact on construction or the schedule. Substantial savings could occur in the size of the diversion tunnel required for a smaller diversion flood.

Spillway

The spillway design for all measures was based on passing a peak discharge of 145,000 cfs. This would be accomplished using an uncontrolled ogee crest spillway with a crest length of 450 feet and a head of 20 feet. For the concrete arch dam measures, the spillway would be divided into two sections, each about half of the total required length. The spillways would be located near each abutment to allow arch stresses from the center of the dam to dip into the abutments with little or no interference from the openings created by the spillways. A flip bucket at the end of each spillway crest would project discharges away from the toe of the dam and onto the massive granite abutments. Future field investigations should determine if a concrete cap and/or rock anchors would be required to protect the impact areas.

The spillway overflow section would be located near the center of the dam for the concrete (RCC) gravity dam measures. Guide walls would be provided to contain the flows within the width of the spillway crest. Energy dissipation would be accomplished as the flow passes over the stepped downstream face of the dam. A concrete cutoff at the toe of the dam was included in the design to prevent undercutting. As the discharge increases, tailwater also would rise and provide additional energy dissipation.

For the CFRF dam type, the spillway would be located on the right abutment for the elevation 1,200 measure, and on the left abutment for the elevation 1,400 measure. The downstream channel would be excavated through the existing rock abutment, and daylight into a natural draw that leads back into the reservoir. A reinforced concrete apron and training wall would be constructed within the first 100 feet upstream from the structure crest and 200 feet downstream from the structure crest to control flows within the vicinity of the dam. Energy would be dissipated by the tailwater that develops in the San Joaquin River at the end of the natural channel. For future designs, a labyrinth spillway should be considered for raising the crest elevation, providing more storage, and reducing the overall width of the spillway, including the outlet channel.

Recent safety-of-dam studies indicates that the existing spillway and outlets of Friant Dam can safely pass about 30 percent of the PMF volume before overtopping occurs (Reclamation 2002d; 2002e). A risk assessment of the overtopping condition suggests that the existing concrete gravity dam can withstand the depth and duration of overtopping without failure. A similar conclusion would be true for the RCC and arch dams at RM 286. However, a rockfill dam would very likely fail at this same threshold condition. Consequently, for purposes of this Appendix, the spillway capacity was increased to 145,000 cfs at RM 286 (up from about 85,000 cfs for the existing Friant Dam spillway) to increase the flow threshold before overtopping would occur. This spillway design flow represents a 500-year flood frequency event, and is equal to 30 percent of the PMF.

Outlet Works and Powerhouse

This section documents an appraisal level design for the outlet works and powerhouse associated with the dam at the end of the 30-foot-diameter diversion tunnel through the right abutment. Design criteria, assumptions, and layout and arrangement of features related to the powerhouse and outlet works are described.

The design was prepared specifically for an RM 286 dam with crest elevation 1,300 and assumed a powerhouse with four 45 MW generating units. The same approach would be used for differing elevations at RM 286. Adjustments in costs for differing generating capacity assumptions are described later in the Construction Costs section of this chapter.

The building containing the powerhouse and outlet works is divided into three areas, the service bay, the unit bay, and the River Outlet Bay. During normal releases, all flows would pass through the turbines.

Outlet Works

The right abutment diversion tunnel would be converted to the combined outlet works and penstock to the new powerhouse. The size of the tunnel system is dictated by requirements for diversion during construction. A transition section from the 30-foot-diameter diversion tunnel to a 25-foot-diameter penstock would be constructed upstream from the downstream portal, and aligned along the back of the combined outlet works control structure and powerhouse. A trashracked intake structure at the upstream portal would be designed to limit approach velocities to 2 feet per second (ft/s) during normal releases and 5 feet/s during maximum releases. Designs include provisions for a bulkhead at the downstream end of the intake structure, and a gate chamber (near the axis of the dam) with isolation gate, for dewatering the tunnel for inspection and maintenance.

The size and number of regulating gates were selected to match the combined capacity of the existing river and canal outlets at Friant Dam. A selective level intake structure was considered but not included in the designs for the following reasons:

- Costs for this type of intake would be significant
- Benefits of a selective level would be minimal because the backwater from Millerton Lake is relatively close to the proposed RM 286 dam site
- If the Kerckhoff No. 2 Powerhouse upgrade option is considered, a mid-level outlet would be constructed in the reservoir

A low-level outlet works that could evacuate the reservoir below elevation 770 was not included. A dead pool of about 20 feet that would hold a relatively small volume of water would remain in the event of a reservoir drawdown.

The River Outlet Bay would contain six 96-inch-diameter pipes extending from the 25-foot-diameter penstock pipe. These discharge pipes would be spaced on 22-foot centers. Each pipe would be fitted with a 96-inch ring follower gate and an 84-inch fixed cone valve. Overhead crane access to the fixed cone valves would be provided via 9-foot by 12-foot access hatches. Total discharge capacity through the river outlet works would be 22,000 cfs (see **Figures 3** through **7** in **Attachment B5** for the arrangement of these features).

Fixed cone valves are typically used for free discharge end-of-the-line service and are readily available from various manufacturers. The guard valves chosen for the outlets are ring follower gates. These gates are fully ported. Reclamation has typically used this type of gate for high head and high velocity service.

Powerhouse

The unit bay would house four 40-45 MW generators with turbines that have a discharge capacity of 1,000 cfs each. The units would be spaced at 45-foot on centers. This spacing is deliberately generous for the purposes of this level of design and estimate. Spacing of the units could be reduced for any future, more detailed layout and design.

The turbine setting, maximum and minimum tailwater surface elevations, and topography were used to set floor elevations and service yard elevations for the powerhouse. The minimum tailwater surface used for this layout would be at elevation 735. Submergence requirements for the turbines set the centerline of the turbine 5 feet below this elevation (elevation 730).

The maximum tailwater surface elevation used for this study was 776. The main service bay floor and Gate Deck were set at elevation 780. Bulkhead gates that isolate the draft tubes and turbines from the tailrace channel and river could be lowered and raised from the Gate Deck with a mobile crane or gantry crane.

Water would be conveyed to the turbines via a 25-foot-diameter penstock pipe that would manifold to four 96-inch-diameter pipes. Each turbine could be isolated from the manifold with a 96-inch spherical valve. Overhead crane access to the spherical valves would be provided via 14-foot by 24-foot access hatches. The draft tube invert is at elevation 713.5 and the draft tube would bifurcate into two 9-foot-wide sections with a center pier.

Turbines

The turbines for the RM 286 powerhouse were sized based on the Phase I Investigation's analysis of potential hydroelectric energy generation at RM 286. Unit flow rates, power outputs, and the number of units were determined from the analysis. The four units of the new RM 286 powerhouse were then selected with the following characteristics:

- Design head – 520 feet
- Head range – 650 to 338 feet
- Design flow rate – 1,000 cfs
- Unit speed – 360 rpm
- Design output – 68,000 hp
- Tailwater elevation – 738 feet minimum

The reservoir has a large variation in water surface elevation for power production. The head range shown above is at the limits of acceptability for Francis-type turbines. The turbine was originally sized for a tailwater elevation of 680. The minimum tailwater elevation shown in **Figure 7** in **Attachment B5** is 735. The difference in these values does not significantly affect the size of the unit and the layout presented in this report is sufficiently accurate for the purposes of these calculations.

Valves

The guard valves estimated for the turbines are spherical valves. Spherical valves are fully ported and designed for high velocities and pressures. Spherical valves are typically used as isolation valves in high head turbine applications. Because they are designed for high velocities, a smaller diameter valve can be used in a larger diameter pipe. Due to the high design pressures and the size of valve required, the weight and cost of these valves is significant. These valves most likely would be supplied by large turbine/pump manufacturers and fabricated by non-domestic suppliers. Due also to the large diameter of these valves, they would be shipped in parts or sections with final assembling and testing done in the field. Other types of less expensive valves such as butterfly-type valves can be fabricated in large diameters, but the large diameter butterfly valve is not made for the required design pressures.

Electrical Equipment

Electrical system components described in this section include the main generator, bus and switchgear, station service, unit control board, and auxiliary control boards. Transmission facilities are discussed later in the section.

<u>Main Generator</u>

The main generators for the powerhouse were sized at 42,700 KVA, 13,800 volts, and 360 rpms at a 90 percent power factor. The generator would be a vertical-shaft synchronous type with a static excitation system. The enclosure for the generator would be TEWAC, which would cool the generator by circulating internal air through an air-to-water heat exchanger. The generator would have control and protective devices to provide for alarm and shutdown of the unit under problem conditions.

Bus and Switchgear

A non-segregated phase bus rate at 15,000 volts, 2,000 amperes, would transmit power from the generator to the unit breaker located inside the powerhouse and then the bus will continue outside the powerhouse to the switchyard. Several taps are provided off this bus inside the powerhouse to service the unit PT and surge protection cubicle, the power feed to the station service equipment, and the power feed to the unit static excitation system.

Station Service

The double-ended secondary unit substation service power supply would be obtained by tapping off the main unit bus. Station service transformers would step voltage down from 13,800 to 480Y/277 volts. A 480-volt distribution panel board would be provided as part of the station service equipment to service powerhouse loads inside the powerhouse, as well as a 120/208 volt panel board to service lighting, receptacles, and other low voltage powerhouse loads.

Unit Control Board

A duplex control switchboard board would be used to control operation of the generating units. The unit control board would provide controls for starting and synchronizing the main generator and for shutting down the unit. All selector switches, pushbuttons, indicating lights, and all unit protective and control devices would be provided by the unit control board. Manual and supervisory control mode type functions would be provided. The supervisory mode would allow operation by a future SCADA control system.

Auxiliary Control Boards

Auxiliary control boards would be provided in the powerhouse to operate all of the auxiliary systems such as hydraulic pumps, water cooling pumps, electrically driven valves, air compressors, etc.

Outlet Works and Powerhouse Shared Equipment and Materials

This subsection provides details regarding equipment and materials required for the combined powerhouse and outlet works facility.

Crane

Handling of equipment throughout the powerhouse would be accomplished with a 150-ton overhead crane. The crane capacity and rail elevation were established based on handling of the rotor/shaft assembly for the generator. Crane access to all five levels of the service bay would be provided via 10-foot by 14-foot access hatches. Personnel access would be provided via a stairwell and elevator within the service bay area. Equipment access into the powerhouse would be provided at the service bay with a 16-foot wide rollup door.

Pipe

Steel piping diameters, thickness, and steel plate material have been selected for manifolds, penstocks, and outlet works in accordance with the following:

- Steel Water Pipe, A Guide for Design and Installation (AWWA)
- Manuals and Reports on Engineering Practice No. 79, Steel Penstocks (ASCE)

All piping is ASTM A36 steel plate. This material is a typical grade of steel used for the fabricating steel manifolds, penstocks, and outlet works.

The cost estimate prepared for this design and layout assumes that no pipe fabricating shop exists in the vicinity of the various job sites and any steel pipe sections 16 feet in diameter or greater would not be able to be shipped as a long cylinder. These sizes of pipe would be shipped in half or quarter sections and welded together and pressure-tested in the field.

Auxiliary Mechanical Systems

A list of auxiliary mechanical systems for operation of the appurtenant features is contained in **Attachment C5** in support of the cost estimate worksheets.

Transmission Facilities

Very little information was available about the existing power grid in the area. The four new 45 MW generators are larger than the existing generators at the Kerckhoff No. 1 and Kerckhoff No. 2 powerhouses. For the present analysis, it was assumed that the existing 115-kV power lines in the vicinity would accommodate the load. Therefore a new 115 kV switchyard would be constructed at the RM 286 dam and powerhouse site. A 1.25-mile transmission line would be constructed to the existing Kerckhoff Powerhouse switchyard. It was assumed that no new equipment would be required at Kerckhoff to make this connection. The new switchyard at RM 286 would include transformers, circuit breakers, and disconnect switches. Due to the lack of information available on the existing power grid, existing switchyards at the Kerckhoff powerhouses were assumed to remain in place as tie points for the power grid.

Site Layout

The powerhouse and outlet works structure shown in **Figures 3** through **8** of **Attachment B5** is approximately 389 feet long and 95 feet wide in plan. The facility is divided into three areas: the service bay, unit bay, and River Outlet Bay. The service bay is 57 feet long at the service yard elevation of 779.5. The unit bay is 194 feet long and the River Outlet Bay is approximately 138 feet long at the yard elevation.

The superstructure extends 64 feet above the service yard to elevation 843.5. The substructure foundation elevation is 708.7 along the Unit and Service bays (excluding the sump) and elevation 727 along the river outlet works. The sump foundation is at elevation 673.7. All of the structure is assumed to be founded on competent rock.

The powerhouse service yard elevation is 779.5. No slopes or other features for yard drainage are included in this design and estimate. Clearances around the powerhouse of 100 feet were provided for crane, truck, and other equipment access. The distance between the switchyard fence and the powerhouse structure was reduced to 50 feet to lessen the magnitude of the required excavation (See **Figure 3** in **Attachment B5**).

All service yard, switchyard, and structure excavation is assumed to be in rock. The amount of overburden is assumed to be negligible for the purposes of this level of design and estimate. A bench is shown at elevation 850 for rock fall protection at the switchyard and service yard. The rock is assumed to be sound granite and a 0.5:1.0 permanent cut slope was used for the cost estimate. No slope protection such as rock bolting or shotcrete was included in the estimate.

The powerhouse penstock would connect to the 30-foot-diameter diversion tunnel portal immediately north of the service yard. An ample distance between the portal and service yard was provided to accommodate construction sequencing requirements for site and structure excavation, continued diversion during construction, and a tie-in between the 25-foot-diameter penstock and diversion tunnel portal.

The switchyard would be 160 feet wide by 220 feet long. The switchyard is shown with a separate fence. Access into and out of the switchyard would be provided via two 20-foot double swing gates.

The cut required for the powerhouse and switchyard shown would be extensive. It is recommended that any future design efforts consider an underground powerhouse such as the current powerhouse at the Kerckhoff No. 2 Powerhouse.

Access Bridge

The cost estimate for the RM 286 powerhouse and outlet works includes a 200-foot-long bridge along the access road to the powerhouse. The bridge clear width would be 16 feet to match the road and shoulder width. The bridge was designed to carry one lane of HS20-44 traffic. The governing code is the Standard Specifications for Highway Bridges (AASHTO, 2002). Access would be provided from this side of the river to provide a more favorable grade and reduce the distance to existing well-maintained roadways in Fresno County.

The bridge superstructure would be supported on reinforced concrete abutments and a single hammerhead type center pier founded on spread footings. Each span of the bridge superstructure would consist of three lines of AASHTO Type IV precast/prestressed concrete beams, with a cast-in-place reinforced concrete deck. The barrier would be a solid concrete Jersey-type barrier. Precast and concrete bridge structures are very durable, and require little or no maintenance.

Future design efforts would require geologic exploration to assess the foundation-bearing capacity and strength. These parameters would assist in a more definite assessment of the bridge support system and associated costs. Also, seismic design criteria would help provide a better estimate for the bridge structure.

Construction Considerations

This section discusses issues related to construction of the potential dam, reservoir, and appurtenant features.

Foundations

Foundations for any of the measures would be in sound granitic rock. The drilling investigation found foundation conditions to be excellent for any dam type (Reclamation, 2005; contained in **Attachment E**). No special foundation considerations are known at this site at this time, and foundation preparation would be typical.

Excavation for both the RCC and concrete arch dam types was assumed to extend to a depth of 25 feet to remove overburden and weathered bedrock. This depth was based on visual observation of core from a drill several hundred feet downstream from the dam (See **Attachment E**). Another consideration in setting the excavation depth was to ensure competent bedrock during overtopping conditions. As there is no evidence that weathering extends to this depth, a more localized geologic investigation program could indicate that the amount of excavation could be greatly reduced.

Flood Routing During Construction

A peak discharge of 65,000 cfs with a return period of approximately 25 years was used to size the diversion structures. The analysis used to determine the diversion flow requirement is described briefly in **Chapter 3**.

Borrow Sources and Materials

Rockfill could be quarried from the reservoir area and obtained from excavations required for the dam and appurtenant structures. Aggregate could be produced by crushing granitic rock obtained from excavations or quarrying in the reservoir area. Earthfill is available in limited quantities. Road cuts in the vicinity and Auberry Valley expose decomposed to intensely weathered granite. Processed sands and gravels could be supplied by commercial sources and/or by crushing and processing quarried rock in the reservoir area.

Construction Site Access

No direct road access exists to the RM 286 dam site. Jeep trails provide access within approximately 1 mile of the dam site at elevations above the river channel, from which the site can be accessed by foot on steeply sloped terrain. Access is across both public and private land.

Staging Areas

A potential construction staging and lay-down area was identified about three-quarters of a mile downstream of the dam site on the left side of the river. An abandoned trailer was found at this location and the area appears to have been previously used for staging.

Lands and Rights-of-Way

Private lands within the reservoir area would need to be acquired. **Table 5-5** shows the total area that would be inundated by the RM 286 reservoir measures considered and the amount of private land within that area that would need to be acquired. Rights-of-way or easements that may be needed to construct new facilities or relocate existing facilities have not been determined and are excluded from the land areas shown.

**TABLE 5-5.
TEMPERANCE FLAT RM 286 RESERVOIR AREA LAND REQUIREMENTS**

Description	RM 286 Reservoir Measures			
Gross pool elevation (feet above msl)	1,200	1,275	1,300	1,400
New storage capacity (TAF)	460	725	830	1,360
Estimated inundated area (acres)	3,155	4,282	4,692	6,262
Estimated public inundated acreage	1,934	2,777	3,047	4,029
Estimated private inundated acreage	1,221	1,505	1,645	2,233

Key:
msl – mean sea level
TAF – thousand acre-feet

Electric Power Sources

Electric power, including grid power, is available from the transmission facilities serving the PG&E Kerckhoff Project. Electric power, including lower voltage service, is available from existing trunks supplying local residences. It was assumed that the existing switchyards at the Kerckhoff powerhouses would remain in place as tie points for the power grid, even if one or both of the Kerckhoff powerhouses was abandoned.

Relocations, Abandonments, or Modifications of Affected Facilities

This subsection of the chapter lists existing facilities that would require relocation, modification, or abandonment for a reservoir at RM 286. Existing facilities that would be inundated by a reservoir with a maximum surface elevation of 1,400 feet are shown in **Figure 5-1**. **Table 5-6** lists all facilities that would be affected by any of the RM 286 surface water storage measures evaluated.

Facilities that would require abandonment or relocation for a reservoir at RM 286 include recreational facilities at Kerckhoff Lake; portions of paved and unpaved roads; Powerhouse Road Bridge; and hydropower facilities above Kerckhoff Lake. Kerckhoff Powerhouse would be abandoned, but a configuration of hydropower generation facilities is under consideration that would involve continued use of a modified Kerckhoff No. 2 Powerhouse and diversion system.

Specific requirements for relocating or abandoning recreational facilities were not determined. Requirements for relocating or abandoning roads and hydroelectric facilities are described below.

TABLE 5-6.
FACILITIES AFECTED BY ALL TEMPERANCE FLAT RM 286 MEASURES

Facilities Requiring Relocation, Modification, or Abandonment
Kerckhoff Dam Outlets
Kerckhoff No. 2 Powerhouse Intake
Kerckhoff Powerhouse Intake
Kerckhoff Dam Gates and Hoists
Powerhouse Road Bridge
Smalley Cove Campground, Picnic Area, Boat Launch
A.G. Wishon Powerhouse
Big Creek No. 4 Powerhouse
Portions of Redinger Lake Road
Substation for Big Creek No. 4 Powerhouse

Kerckhoff Lake Recreation Facilities

All recreation facilities at Kerckhoff Lake would be submerged by a reservoir at RM 286 for all elevations considered in the Investigation. Specific requirements for relocating, replacing, or abandoning these recreational facilities were not determined.

Roads and Bridges

Portions of roads and Powerhouse Road Bridge could be inundated by a reservoir constructed at RM 286, as discussed further below. In total, for a reservoir with a gross pool elevation of 1,400, 6 miles of paved road and 4.5 miles of unpaved road would be inundated. A lesser amount would be inundated for lower reservoir elevations.

Powerhouse Road and Bridge

Appraisal level Powerhouse Road and Bridge relocations were developed for the elevations 1,200, 1,300, and 1,400 reservoir levels (see **Figures 22** through **24** of **Attachment B5**). Drawings in **Attachment B5** refer to the road to be relocated as Auberry Road, as it is the extension of Auberry Road on the Fresno County side of the river.) The design approach to these road and bridge relocations is described in **Chapter 3**, along with a description of the construction requirements for relocations for the elevations 1,200 and 1,300 reservoir levels. A description of requirements for the relocations for the elevation 1,400 measure follows.

Powerhouse Road

For the RM 286 elevation 1,400 measure, a minimum roadway elevation of 1,440 feet was used, which is approximately 8 feet above the PMF. This requires constructing approximately 19,000 feet of new road and a new 1,711-foot bridge, as well as the installation of approximately 8,300 feet of metal beam guardrails, and culverts.

Powerhouse Road Bridge

The bridge profile was set at elevation 1,440 for the RM 286 elevation 1,400 measure. The overall length of the bridge is approximately 1,711 feet. The center or main span is approximately 850 feet long, and the end or side spans are approximately 425 feet long. Box girder depths vary from approximately 41 feet, zero inches, at the piers to 16 feet, 6 inches, at the abutments and midspan. Pier 1 and 2 heights are calculated to be 160 feet, and 310 feet, respectively.

Redinger Lake Road

A substantial portion of Redinger Lake Road would be inundated by a reservoir at elevation 1,200 or higher. It is assumed that the inundated portion, beginning at Kerckhoff Lake, would be abandoned. The portion of the road descending from Redinger Lake would remain.

Kerckhoff No. 2 Powerhouse

A powerhouse at the RM 286 dam location with four generating units was described in the appurtenant features section of this chapter. If such a powerhouse were constructed, as envisioned in Replacement Power Option 1, the existing Kerckhoff powerhouses would be abandoned, along with the diversion facilities at Kerckhoff Lake that feed the powerhouses. Details of decommissioning the powerhouses are provided in **Chapter 2**. Since the abandoned powerhouses would be accessible by land, site security measures also would be required.

Other potential configurations of hydroelectric generation facilities associated with a RM 286 reservoir also are being considered that could make use of existing Kerckhoff Project facilities.

A second configuration being considered, Replacement Power Option 2, involves constructing a new multiple unit powerhouse to replace the existing Kerckhoff No. 2 Powerhouse. The existing Kerckhoff No. 2 diversion tunnel would be used to feed a new powerhouse at approximately RM 283, adjacent to the existing Kerckhoff No. 2 Powerhouse, which would discharge to the upper end of Millerton Lake. The existing Kerckhoff No. 2 Powerhouse would be abandoned in place. The older Kerckhoff Powerhouse would also be abandoned and its site restored to near natural conditions. Abandonment of the existing powerhouses is described in **Chapter 2**. Requirements for constructing the new, replacement Kerckhoff No. 2 Powerhouse is described below.

A third potential hydropower facility configuration, Replacement Power Option 3, involves replacing the Kerckhoff No. 2 Powerhouse turbine and generator with a generating unit of greater capacity than it currently has. This configuration would use the diversion tunnel now serving the Kerckhoff No. 2 Powerhouse but would require a new surge chamber. Kerckhoff Powerhouse, the smaller, older facility upstream of the Kerckhoff No. 2 Powerhouse, would be abandoned, along with its diversion facilities at Kerckhoff Lake. Replacement Power Option 3 would also entail construction of a powerhouse at RM 286 with relatively low generation capacity that could produce energy from water discharges too small for the retrofitted Kerckhoff turbine to accommodate. The turbine and generator replacement for the Kerckhoff No. 2 Powerhouse is detailed below, following the discussion of a new, replacement Kerckhoff No. 2 Powerhouse. Abandoning the Kerckhoff Powerhouse is described in **Chapter 2**. Decommissioning the Kerckhoff Lake diversion intake is discussed in **Chapter 3**.

New Kerckhoff No. 2 Powerhouse

Instead of constructing a powerhouse at the end of the RM 286 flood routing diversion tunnel or retrofitting the Kerckhoff No. 2 Powerhouse with a new generation unit, another potential configuration of hydroelectric generating facilities, Replacement Power Option 2, would consist of replacing the existing Kerckhoff No. 2 Powerhouse with a new powerhouse adjacent to the existing site. Instead of housing a single large capacity generating unit, the new powerhouse would contain multiple smaller capacity generating units to accommodate a wider range of varying heads and flows.

The replacement powerhouse would be located approximately 400 feet upstream of the tailrace for the existing Kerckhoff No. 2 Powerhouse. The close proximity of the replacement powerhouse to the existing one allows the incorporation of existing powerhouse features where possible. The penstock for the replacement powerhouse ties into and uses one of the existing adit tunnels to connect to the existing main tunnel near elevation 769.

The existing tunnel would be used and a new penstock and surge chamber would be connected to the tunnel to match the new alignment to the powerhouse.

Approximately 4,100 feet of the existing penstock tunnel may need to be lined to adequately handle pressure increases as a result of raising the upstream reservoir surface. A steel lining would be provided and the lining would be backfilled with concrete. Although no specific geologic investigations were performed along the existing Kerckhoff No. 2 Powerhouse diversion tunnel alignment, based on available information it was assumed that the existing tunnel is in granite without joints and that operators of the existing tunnel and surge tank have not noticed any problems.

The powerhouse is shown in **Figures 7** through **10** of **Attachment B5.** The powerhouse is divided into two areas, the service bay and the unit bay. The unit bay would house four 45 MW generators with turbines that have a discharge capacity of 1,000 cfs each. The units would be spaced at 45-foot centers. This spacing is deliberately generous for this level of design and estimate. Spacing of the units may be reduced for any future, more detailed layout and design.

The turbine setting, maximum and minimum tailwater surface elevations, and topography were used to set floor elevations and service yard elevations for the powerhouse. The minimum tailwater surface used for this layout was elevation 543. The submergence requirements for the turbines set the centerline of the turbine 12.5 feet below this elevation (elevation 530.5).

The maximum tailwater surface elevation used for this study was 603. The main service bay floor and Gate Deck were set at elevation 605.5. Bulkhead gates that isolate the draft tubes and turbines from the tailrace channel and river could be lowered and raised from the Gate Deck with a mobile crane or gantry crane.

Water would be conveyed to the turbines via a 16-foot-diameter penstock pipe that would manifold to four 96-inch-diameter pipes. Each turbine could be isolated from the manifold with a 96-inch spherical valve. Overhead crane access to the spherical valves would be provided via 15-foot by 24-foot access hatches. The draft tube invert would be at elevation 514 and the draft tube would bifurcate into two 9-foot wide sections with a center pier.

Turbines

The turbines for the new powerhouse near the existing Kerckhoff No. 2 Powerhouse location were sized based on a matrix developed from the Investigation's Phase 1 hydropower analysis. The matrix listed unit flow rate, power output, and the number of units for each configuration. The four units of the new powerhouse near the existing Kerckhoff No. 2 Powerhouse location were selected with the following characteristics:

- Type – vertical shaft Francis
- Design head – 604 feet
- Gross head range – 457 to 845 feet
- Design flow rate – 1,000 cfs
- Unit speed – 360 rpm
- Design output – 62,350 hp
- Tailwater elevation – 543 feet minimum

The reservoir has a large variation in water surface elevation for power production; therefore, the head range is at the limits of acceptability for Francis turbines. The turbines were set at an elevation that should preclude damaging cavitation at minimum tailwater elevation with a modern hydraulic design.

Guard Valves

The guard valves that would be used for the turbines are spherical valves. Spherical valves are fully ported and designed for high velocities and pressures. Spherical valves are typically used as isolation valves in high head turbine applications. Because they are designed for high velocities, a smaller diameter valve can be used in a larger diameter pipe. Due to the high design pressures and the size of valve required, the weight and cost of these valves is substantial. These valves would most likely be supplied by large turbine/pump manufacturers and fabricated by non-domestic suppliers. Due also to the large diameter of these valves, they would be shipped in parts or sections with final assembling and testing to be done in the field. Less expensive valves such as butterfly-type valves can be fabricated for large diameters, but the large diameter butterfly valve is not made for the required design pressures.

Penstock

The new steel penstock would attach to the existing tunnel through the existing construction adit. At this location, the steel lining would begin where the rock above and around this location resists the hydraulic forces. This is standard Reclamation practice.

The same pipe design specifications and assumptions would apply to the new penstock as those described earlier in this chapter for the RM 286 outlet works and powerhouse under Appurtenant Features, Outlet Works, and Powerhouse Shared Equipment and Materials.

Surge Tank

As the project raises the upstream reservoir elevation, the static head in the surge tank, and upsurges also would rise. Surge analysis indicated that the elevation of the tank top must rise to 430 feet above the surrounding ground surface if the tank were to remain at its current location. To reduce the required height of the structure, the new tank would be located at a higher elevation a short distance away from the existing surge tank.

Figure 4 in **Attachment B5** shows the plan and section of the new surge tank. It was assumed for the purposes of this layout that the geology is granite and without joints. The new tank would connect to the existing tunnel with a 20-foot-diameter adit and riser shaft; this matches the diameter of the existing riser. Design assumptions and excavation process discussed for the surge tank for the Kerckhoff No. 2 Powerhouse retrofit configuration also would apply to the new powerhouse near the Kerckhoff No. 2 Powerhouse location. The existing tunnel would be plugged upstream of the knee and the existing riser immediately above the existing tunnel.

Surge Analysis

The surge analysis discussed for the Kerckhoff No. 2 Powerhouse turbine retrofit also was conducted for the hydropower facility configuration involving a new powerhouse near the existing Kerckhoff No. 2 Powerhouse location. The transient analysis determined that surges in the new tank ranged from elevation 909 to elevation 1,440. With freeboard, the tank bottom would be set at elevation 900 and the tank top at elevation 1,450.

Surge analysis results and variables were the same as reported for the Kerckhoff No. 2 Powerhouse turbine retrofit, with the following exception: maximum downsurge (start-up): 909 feet (tank water surface). In all other respects, assumptions, process, and results of the transient analysis were the same as for the K2 turbine retrofit configuration discussed earlier in this chapter.

Crane

Handling equipment within the unit bay would be accommodated with a 150-ton overhead crane. A rotor set-down area would be included in the unit bay for assembly and disassembly of unit components. Handling equipment within the service bay would be accomplished with a 75-ton overhead crane. The crane capacities and rail elevations were established based on handling of the rotor/shaft assembly for the generator in the unit bay. Crane access to all four levels of the service bay would be provided via 10-foot by 14-foot access hatches.

Auxiliary Mechanical Systems

Mechanical systems for operation of the new powerhouse are detailed in **Attachment C5** in support of the cost estimate worksheets.

Electrical Equipment

The same type of electrical system components (main generator, bus and switchgear, station service, unit control board, and auxiliary control boards) would be provided as those described for the RM 286 powerhouse, in the Appurtenant Features section of this chapter, with two exceptions. As explained below, main generator sizing would differ, and spherical valve control boards would be provided. SCADA also is briefly discussed.

<u>Main Generator</u>

The main generator for the powerhouse would be sized at 45,600 KVA, 13,800 volts, 60 cycles and 360 rpms at a 90 percent power factor. In other respects the generator type, cooling enclosure, and controls would be similar to those described for the RM 286 powerhouse in the Appurtenant Features section of this chapter.

<u>Spherical Valve Control Boards</u>

Control boards would be provided for the hydraulically operated spherical valves. All starters, selector switches, pushbuttons, and indicating lights and all protective and control devices for the operation of the valves would be provided by the control boards.

<u>Supervisory Control Alarm and Data Acquisition</u>

Specific SCADA requirements cannot be determined at this time. Any costs associated with providing a SCADA system would be low relative to other equipment and are therefore considered under "unlisted items."

Transmission

Available information indicated that there are several lines leaving the existing switchyard. Use of the existing 115 kV power lines in the vicinity to transport the power is assumed adequate for the purposes of this estimate. Existing switchyard structures at the existing Kerckhoff No. 2 Powerhouse would therefore remain in place. A new switchyard for the new powerhouse would tie into the existing switchyard structures.

The four new 45 MW generators would provide slightly more power than the existing 140 MW generator. A new transformer would be installed based on the assumption that existing transformer capacity is insufficient for the new load.

Site Layout

The powerhouse shown in **Figures 7** through **10** of **Attachment B5** is approximately 305 feet long and 95 feet wide in plan. The service bay is 108 feet long and the unit bay is approximately 200 feet long at the service yard elevation of 605.

The superstructure would extend 64 feet above the service yard to elevation 669. The substructure foundation elevation would be approximately 509 along the unit bay and 474 below the sump at the service bay. All of the structure is assumed to be founded on competent rock.

The powerhouse Service Yard elevation would be 605. No slopes or other features for yard drainage are included in this design and estimate. Clearances around the powerhouse of 100 feet were provided for crane, truck, and other equipment access. The distance between the switchyard fence and the powerhouse structure was reduced to approximately 30 feet to reduce the magnitude of the required excavation (see **Figures 5** and **6** of **Attachment B5**).

Access to the service yard would be provided on the northern end of the service yard. Personnel access would be provided via a stairwell and elevator within the service bay area. Equipment access into the powerhouse would be provided at the service bay with a 16-foot-wide rollup door. Analysis of the layout depicted in this report indicates that length of access road and amount of excavation could be significantly reduced by providing access from the south side of the service yard. Future designs and layouts should investigate access from the south to reduce costs. Rearrangement of the powerhouse, switchyard, and Service Yard would be necessary to accommodate access from the south.

All Service Yard, switchyard, and structure excavation is assumed to be in rock. The amount of overburden is assumed to be negligible for the purposes of this level of design and estimate. The rock is assumed to be sound granite and a 0.5:1.0 permanent cut slope has been used. No slope protection such as rock bolting or shotcrete was included.

The powerhouse penstock would connect to the existing 25-foot-diameter adit tunnel through the tunnel portal immediately northeast of the service yard. Any potential need to maintain operation of the existing Kerckhoff No. 2 Powerhouse during construction of the new powerhouse was not considered in regard to location and orientation of new features.

The switchyard would be 180 feet wide by 210 feet long. The switchyard is shown with a separate fence. Access into and out of the switchyard would be provided via two 20-foot double swing gates.

Replace Kerckhoff No. 2 Powerhouse Turbine and Generator

A dam with reservoir maximum surface elevation of 1,400 feet would increase the head available for generation by approximately 400 feet. The existing turbine at the Kerckhoff No. 2 Powerhouse is not designed to operate with the new head. Consequently, continued use of the Kerckhoff No. 2 Powerhouse would require replacing the turbine and installing a new surge tank. The generator also would be replaced, resulting in a 182 MW unit. To serve as a guard valve, a ring follower gate would be installed upstream of the new turbine for unit isolation within the existing powerhouse. A new steel penstock would be assembled and inserted inside the existing penstock; the existing penstock would serve as a casing.

Based on the limited information available, other major infrastructure at the site, including the switchyard, power transmission lines, access roads, and tunnels, are assumed adequate to meet the needs of the turbine replacement option for the existing Kerckhoff No. 2 Powerhouse.

Tunnel Lining

The capacity of the existing diversion tunnel to resist the increased hydraulic loading caused by raising the upstream reservoir level was not analyzed. However, it appears that portions of the existing tunnel alignment may have insufficient overburden to provide adequate resistance for the increased pressure. Approximately 4,100 feet of the existing penstock tunnel may need to be lined to adequately handle pressure increases as a result of raising the reservoir surface. A steel lining would be provided for the relevant sections of tunnel and the lining would be backfilled with concrete.

No specific geologic investigations were performed at the existing Kerckhoff No. 2 Powerhouse site. Based on available information, it was assumed that the existing tunnel is in granite without joints, and that operators of the existing tunnel and surge tank have not noticed any problems.

Turbine Replacement

Replacing the turbine would be approached similarly to that of the Roosevelt Powerplant turbine replacement. The Roosevelt Dam and Powerplant are located near Payson, Arizona; the dam was raised 77 feet in 1996. The turbine spiral case was exposed by removing the surrounding concrete and the top of the spiral case was cut off. The new spiral case, which was smaller than the old spiral case, was grouted inside the remains of the old case. The original draft tube was reused. Even though the head increase contemplated for a reservoir at RM 286 is significantly greater than that of Roosevelt Dam, it is thought that this method of replacing the turbine with one more suited to the new head is a viable option and was considered in this appraisal level study.

Replacing the turbine would require excavating and removing various portions of the existing reinforced concrete powerhouse structure. The amount of concrete to be removed is considered relatively small. However, removal methods would be limited to those that produce relatively accurate excavated surfaces to retain the structural integrity of the powerhouse that will remain once the existing unit is removed. Methods used for past similar projects have required extensive pre-drilling along accurately defined lines and angles. Blasting is typically not allowed for this type of excavation and removal. This restriction is assumed to apply to turbine replacement at the Kerckhoff No. 2 Powerhouse.

Numerous access tunnels are available at the existing powerhouse. These existing tunnels were assumed adequate to provide access for personnel and equipment required to perform the necessary concrete excavation and removal.

Guard Valve / Ring Follower Gate

The existing valve would not be adequate for the increase in pressure for the new unit; therefore, a ring follower gate would be used as a guard valve for the turbine. Reclamation has typically used this type of gate for high head and high velocity service. Ring follower gates are fully ported and have short bodies, but are very tall. Most of the gate body would be encased in concrete with only the hydraulic cylinder being exposed. The valve house required for this valve would be longer and taller than the existing one. A new shaft would need to be constructed for the new valve house. The shaft, or part of it, would remain for access to the new valve.

A 144-inch-diameter ring follower gate would be installed upstream of the new turbine for unit isolation within the existing powerhouse. Additional rock excavation would be required within the existing valve area to accommodate the deeper structure required for the ring follower gate.

A substantial amount of time would be needed to create the design and prepare drawings for the gate. The gate could be manufactured in the United States. Because of the size of the gate, it would have to be shipped in parts or sections with final assembling and testing to be conducted in the field.

Penstock and Piping

The existing steel penstock would not be adequate for the increase in pressure for the new unit, but it could be used as a casing for the new replacement penstock. The new penstock would be installed inside the existing penstock. The annulus between the two pipes would be filled with grout. The new penstock would be designed for full internal pressure.

The same pipe design specifications and assumptions would apply to the new penstock as those described earlier in this chapter for the RM 286 outlet works and powerhouse under Appurtenant Features, Outlet Works and Powerhouse Shared Equipment and Materials.

Surge Tank

As the project raises the upstream reservoir elevation, the static head in the surge tank, and upsurges also would rise. Surge analysis indicated that the elevation of the tank top must rise to 430 feet above the ground surface if the tank were to remain at its current location. To accommodate this surge requirement, a new surge tank would be located further up the existing hillside at elevation 1450. **Figure 3** in **Attachment B5** shows the plan and section of the new surge tank. The new tank would connect to the existing tunnel with a 20-foot-diameter adit and riser shaft. This matches the diameter of the existing riser.

It is assumed that the adit and shafts would require little support and would be unlined, similar to the existing tunnel and shaft. Construction access would likely be through the existing portal near the knee. Miners would excavate the adit from the existing tunnel and then excavate the riser shaft by boring upward to the top of the tank. The tank would be enlarged by drilling and blasting (slashing) down, with tank waste falling through the raised-bore hole. Miners would remove the entire riser shaft and the tank waste through the newly excavated adit and existing tunnel.

Surge Analysis

New surges and the new surge tank location were evaluated in an analysis. The surge analysis kept the tank's connection point in the tunnel at its current location and near the knee in the penstock. This provides water to the penstock when the operators open the wicket gates and accepts water when the wicket gates close automatically during load rejection. The analysis assumed a 10-second wicket gate opening and closing time. Four cases were studied for completeness:

- Load rejection at minimum reservoir elevation
- Load rejection at maximum reservoir elevation
- Load acceptance at minimum reservoir elevation
- Load acceptance at maximum reservoir elevation

Transient analysis determined that surges in the new tank ranged from elevation 912 to elevation 1,440. With freeboard, the tank bottom would be set at elevation 900 and the tank top at elevation 1,450. Analysis results and variables were as follows:

- Water surface increase: from elevation 985 to elevation 1,400
- Design maximum flow rate: 4,000 cfs
- Design turbine shutdown rate: 10 seconds
- Minimum water surface: 1,000 feet
- Maximum upsurge (load rejection): 1,440 feet (tank water surface)
- Maximum downsurge (startup): 912 feet (tank water surface)
- Maximum pressure at top of turbine penstock: 696 feet

Crane

A new, 400-ton overhead crane would replace the existing overhead crane to accommodate handling of new rotor/shaft assembly for the replacement generator. The weight of this assembly is 659,000 pounds; the replacement crane span would be approximately 85 feet. A new overhead crane is included based on the assumption that the replacement generator rotor/shaft assembly weight would be greater than the existing generator rotor/shaft assembly weight.

It is assumed that the crane hook limits and restrictions due to the height and interior space of the existing structure would be compatible with the new crane. Existing space within this powerhouse is assumed to be adequate for handling the new rotor/shaft assembly.

Auxiliary Mechanical Systems

Other mechanical equipment, including draft tube bulkhead gates and guides, intake trashrack, heating/ventilating, and powerhouse equipment, is assumed to be satisfactory and would not need to be replaced.

Electrical Equipment

The same type of electrical system components (main generator, bus and switchgear, station service, unit control board, and auxiliary control boards) would be provided as those described for the RM 286 powerhouse in the Appurtenant Features section of this chapter with three exceptions: as explained below, main generator sizing and bus amperage would differ, and a turbine isolation gate control board would be provided.

Main Generator

The main generator for the retrofitted powerhouse is sized at 182,300 KVA, 13,800 volts, 60 cycle, and 240 rpms at 90 percent power factor. In other respects, the generator type, cooling enclosure, and controls would be similar to that described for the RM 286 powerhouse in the Appurtenant Features section of this chapter.

Bus and Switchgear

An isolated phase bus rated 15,000 volts and 10,000s ampere would transmit power from the generator to the unit breaker located inside the powerhouse. The bus would then continue outside the powerhouse to the switchyard. Several taps would be provided off this bus inside the powerhouse to service the unit PT and surge protection cubicle, the power feed to the station service equipment, and the power feed to the unit static excitation system.

Turbine Isolation Gate Control Board

A control board would be provided for the hydraulically operated turbine isolation gate. All starters, selector switches, pushbuttons, and indicating lights and all protective and control devices for operating the valve would occur through the control board.

Transmission

Readily available information indicates that several lines leave the existing switchyard. The existing 115 kV power lines in the vicinity are assumed to accommodate the load.

The new 180 MW generator provides slightly more power than the existing 140 MW generator. It is assumed that the existing transformer would not be sufficient for the new load. Consequently, a new transformer would be provided.

Kerckhoff Powerhouse

Kerckhoff Powerhouse would be abandoned if a dam were constructed at RM 286. Two alternative approaches to decommissioning were considered: abandon in place or restore to near-natural conditions. Decommissioning details were presented in **Chapter 2**. Security measures also would be required, since the powerhouse would be accessible by land.

Kerckhoff Dam and Intakes

The reservoir pool behind a new dam at RM 286 would inundate the existing Kerckhoff Dam and hydropower intake structures. If the Kerckhoff No. 2 Powerhouse were retrofitted with a new turbine or replaced with a new powerhouse near the existing location at the upper end of Millerton Lake, then the Kerckhoff No. 2 Powerhouse intake structure at Kerckhoff Dam would need to be modified, as described in **Chapter 4**. The intake for the Kerckhoff Powerhouse, however, would be decommissioned, as described in **Chapter 3**. If the Kerckhoff No. 2 Powerhouse were decommissioned and only a powerhouse at the RM 286 dam were developed, then the Kerckhoff No. 2 Powerhouse diversion tunnel would be abandoned and all working components of Kerckhoff Dam and intakes would be decommissioned, as described in **Chapter 3**.

Wishon Powerhouse

A reservoir at RM 286 would submerge the Wishon Powerhouse and necessitate its decommissioning or relocation. Decommissioning of the Wishon Powerhouse is described in **Chapter 3**. Relocation would involve decommissioning the existing powerhouse and constructing a new one at a higher elevation. Specific construction requirements for a new powerhouse were not determined; however, analysis documented in the **Hydropower TA** indicates that a generating capacity of between 14 and 16 MW could be supported.

Big Creek No. 4 Powerhouse

A reservoir at RM 286 would submerge the Big Creek No. 4 Powerhouse. This would require decommissioning or relocating the powerhouse. Relocation would apply to a new reservoir with a gross pool elevation at or below 1,250 feet. Relocation would involve decommissioning the existing powerhouse and constructing a new powerhouse at a higher elevation. Requirements for decommissioning the Big Creek No. 4 Powerhouse are presented in **Chapter 3**. Requirements for constructing a replacement powerhouse are provided in **Chapter 4**.

Big Creek Dam No. 7

Depending on the size of a dam at RM 286, the new reservoir pool would back up to or submerge Big Creek Dam No. 7, which forms Redinger Lake. A reservoir with a maximum surface elevation of 1,400 feet would submerge the existing Big Creek No. 7 Dam at Redinger Lake and make the intake structures for Big Creek No. 4 Powerhouse unusable. In such case, the dam would be decommissioned. Specific features that would be removed were identified in **Chapter 4**. If the Big Creek No. 4 Powerhouse were relocated to a higher elevation, the intake would be left intact.

Construction Costs

Costs for constructing RM 286 storage measures at July 2004 price levels are summarized in **Tables 5-7** through **5-9**. Costs shown for individual project components include field costs and indirect costs for planning, investigations, designs, and construction management. Indirect costs involved in constructing project components are 25 percent of field costs. Acquisition costs for lands in the reservoir area are incorporated into the construction cost of the main dam, along with an allowance of 20 percent of lands costs for indirect costs associated with property acquisition.

Construction costs are based on worksheets presented in **Attachments C1** and **C5**. Field costs for the dam and appurtenant features, previously calculated at July 2003 price levels, were adjusted to reflect July 2004 prices and a higher contingency. Costs for abandonments and relocations were developed in July 2004. Accordingly, all costs listed are at July 2004 prices.

Cost estimates were originally developed in Phase 1 of the Investigation for RCC, concrete arch, and CFRF types at RM 286 with dam crests at elevations 1,200 and 1,400, based on quantities calculated from preliminary designs. Cost estimates for elevation 1,300 dam measures were interpolated at the line item level from the costs for crests at elevations 1,200 and 1,400. From these preliminary cost estimates, it appears that RCC is typically the lowest cost dam type at RM 286, except at elevation 1,200, where the concrete arch design has the lowest cost. Accordingly, dam costs provided are represented by the cost for an RCC type dam. Cost differences between dam types based upon these preliminary designs, however, are not great enough to conclusively state that the RCC dam type is the most cost efficient at RM 286 for all sizes.

Tables 5-7 through **5-9** show construction costs for each of the three potential powerhouse configurations considered for a storage measure at RM 286: a multiple generating unit powerhouse at the RM 286 dam (Replacement Power Option 1, **Table 5-7**); a new multiple unit replacement Kerckhoff No. 2 Powerhouse (Replacement Power Option 2, **Table 5-7**); and a replacement turbine and generator for the existing Kerckhoff No. 2 Powerhouse, in combination with a small powerhouse at the RM 286 dam (Replacement Power Option 3, **Table 5-9**).

Cost estimates are provided for a 725 TAF reservoir size to provide consistency with the analysis contained in the **Hydropower TA**. A 725 TAF RM 286 reservoir would have an approximate gross pool elevation of 1,275. Field costs for an RCC dam with crest at elevation 1,275 were interpolated from July 2004 contract costs in cost estimates for the relevant features at higher and lower elevations.

TABLE 5-7.
CONSTRUCTION COSTS FOR TEMPERANCE FLAT RM 286 MEASURES WITH REPLACEMENT POWER OPTION 1 ($ MILLION)

Gross Pool Elevation (feet above msl)	1,200	1,275	1,300	1,400
New Storage Capacity (TAF)	**460**	**725**	**830**	**1,360**
Storage Components				
RCC Dam, Spillway, River Diversion, Reservoir Lands	320	360	370	560
River Outlet Works at RM 286	70	88	88	105
Abandon Kerckhoff No. 2 Powerhouse	2	2	2	2
Abandon Intake for Kerckhoff No. 2 Powerhouse	1	1	1	1
Abandon and Restore Kerckhoff Powerhouse	4	4	4	4
Abandon Intake for Kerckhoff Powerhouse	1	1	1	1
Remove Kerckhoff Dam Outlet Works and Gates	2	2	2	2
Abandon Wishon Powerhouse	2	2	2	2
Abandon Big Creek No. 4 Powerhouse	4	4	4	4
Remove Redinger Dam Operating Equipment	-	-	-	8
Powerhouse Road Relocation	18	36	40	55
Powerhouse Road Bridge Relocation	34	35	38	49
Construction Cost, Storage Components	**458**	**535**	**552**	**793**
Replacement Power Components				
New Powerhouse at RM 286 Dam (150 to 180 MW)	115	120	120	125
RM 286 Switchyard and Transmission Line	18	18	18	18
New Wishon Powerhouse (14 to 16 MW)	46	46	46	46
New Big Creek No. 4 Powerhouse at Redinger Dam (30 MW)	69	69	69	-
Construction Cost, Replacement Power Components	**248**	**253**	**253**	**189**
Construction Cost[1, 2]	**706**	**788**	**805**	**982**

Key:
msl – mean sea level
RM – river mile
MW – megawatt
TAF – thousand acre-feet
RCC – roller-compacted concrete

Notes:
[1] All cost estimates are preliminary. Construction cost represents the sum of field costs and indirect costs for planning, engineering, design and construction management, estimated at 25 percent of field costs.
[2] Costs do not include environmental mitigation, new or relocated recreation facilities, acquisition of impacted power facilities, or compensation for lost future power generation.

TABLE 5-8.
CONSTRUCTION COSTS FOR TEMPERANCE FLAT RM 286 MEASURES WITH REPLACEMENT POWER OPTION 2 ($ MILLION)

Gross Pool Elevation (feet above msl)	1,200	1,275	1,300	1,400)
New Storage Capacity (TAF)	460	725	830	1,360
Storage Components				
RCC Dam, Spillway, River Diversion, Reservoir Lands	320	360	370	560
River Outlet Works at RM 286	70	88	88	105
Abandon Kerckhoff No. 2 Powerhouse	2	2	2	2
Abandon and Restore Kerckhoff Powerhouse	4	4	4	4
Abandon Intake for Kerckhoff Powerhouse	1	1	1	1
Remove Kerckhoff Dam Outlet Works and Gates	2	2	2	2
Abandon Wishon Powerhouse	2	2	2	2
Abandon Big Creek No. 4 Powerhouse	4	4	4	4
Remove Redinger Dam Operating Equipment	-	-	-	8
Powerhouse Road Relocation	18	36	40	55
Powerhouse Road Bridge Relocation	34	35	38	49
Construction Cost, Storage Components	**457**	**534**	**551**	**792**
Replacement Power Components				
New Kerckhoff No. 2 Powerhouse (170 to 200 MW)	175	180	180	190
Kerckhoff No. 2 Diversion Tunnel, Steel Liner	85	115	125	165
Kerckhoff No. 2 Diversion Tunnel, Backfill Concrete	3	3	3	3
Modify Kerckhoff No. 2 Diversion Intake	33	36	39	45
New Wishon Powerhouse (14 to 16 MW)	46	46	46	46
New Big Creek No. 4 Powerhouse at Redinger Dam (30 MW)	69	69	69	-
Construction Cost, Replacement Power Components	**411**	**449**	**462**	**449**
Construction Cost[1, 2]	**868**	**983**	**1,013**	**1,241**

Key:
msl – mean sea level
RM – river mile
MW – megawatt
TAF – thousand acre-feet
RCC – roller-compacted concrete

Notes:
[1] All cost estimates are preliminary. Construction cost represents the sum of field costs and indirect costs for planning, engineering, design and construction management, estimated at 25 percent of field costs.
[2] Costs do not include environmental mitigation, new or relocated recreation facilities, acquisition of impacted power facilities, or compensation for lost future power generation.

TABLE 5-9.
CONSTRUCTION COSTS FOR TEMPERANCE FLAT RM 286 MEASURES WITH REPLACEMENT POWER OPTION 3 ($ MILLION)

Gross Pool Elevation (feet above msl)	1,200	1,275	1,300	1,400
New Storage Capacity (TAF)	**460**	**725**	**830**	**1,360**
Storage Components				
RCC Dam, Spillway, River Diversion, Reservoir Lands	320	360	370	560
River Outlet Works at RM 286	70	88	88	105
Abandon and Restore Kerckhoff Powerhouse	4	4	4	4
Abandon Intake for Kerckhoff Powerhouse	1	1	1	1
Remove Kerckhoff Dam Outlet Works and Gates	2	2	2	2
Abandon Wishon Powerhouse	2	2	2	2
Abandon Big Creek No. 4 Powerhouse	4	4	4	4
Remove Redinger Dam Operating Equipment	-	-	-	8
Powerhouse Road Relocation	18	36	40	55
Powerhouse Road Bridge Relocation	34	35	38	49
Construction Cost, Storage Components	**455**	**532**	**549**	**790**
Replacement Power Components				
New Powerhouse at RM 286 Dam (40 to 60 MW)	130	145	145	155
RM 286 Switchyard and Transmission Line	18	18	18	18
Kerckhoff No. 2 Turbine Generator Replacement (140 to 186 MW)	68	70	70	78
Kerckhoff No. 2 Penstock, Ring Follower Gate	28	28	28	28
Kerckhoff No. 2 Diversion Tunnel, Steel Liner	85	115	125	165
Kerckhoff No. 2 Diversion Tunnel, Backfill Concrete	3	3	3	3
Modify Kerckhoff No. 2 Diversion Intake	33	36	39	45
New Wishon Powerhouse (14 to 16 MW)	46	46	46	46
New Big Creek No. 4 Powerhouse at Redinger Dam (30 MW)	69	69	69	-
Construction Cost, Replacement Power Components	**480**	**530**	**543**	**538**
Construction Cost[1, 2]	**935**	**1,062**	**1,092**	**1,328**

Key:
msl – mean sea level
MW – megawatt
RCC – roller-compacted concrete
RM – river mile
TAF – thousand acre-feet

Notes:
[1] All cost estimates are preliminary. Construction cost represents the sum of field costs and indirect costs for planning, engineering, design and construction management, estimated at 25 percent of field costs.
[2] Costs do not include environmental mitigation, new or relocated recreation facilities, acquisition of impacted power facilities, or compensation for lost future power generation.

Cost estimates for all RM 286 storage measures and replacement power options include appraisal level costs for decommissioning and abandoning: the Kerckhoff Powerhouse and its intake structure; Kerckhoff Dam, gates, hoist, and outlet works; Wishon Powerhouse; and Big Creek No. 4 Powerhouse. Costs also include the cost to construct a replacement Wishon Powerhouse at a higher elevation, as well as the cost of constructing a 30 MW replacement Big Creek No. 4 Powerhouse for reservoir measures up to and including gross pool elevation 1,300. The cost to construct a replacement Big Creek No. 4 Powerhouse below Redinger Dam was originally calculated for a 12 MW replacement facility at an elevation of approximately 1,250 feet. The cost for a 30 MW facility at or above 1,100 feet was approximated by rescaling the estimate prepared for a 12 MW facility. The square root of the generation capacity ratios was used as the scaling factor. It was assumed that the cost to construct the facility at an elevation of approximately 1,250 feet would be roughly comparable to the cost of construction at a lower elevation.

For all three replacement power options, appraisal level cost estimates for relocating Powerhouse Road and Bridge were explicitly prepared for measures at gross pool elevations of 1,200, 1,300, and 1,400. Field costs of relocating Powerhouse Road and Bridge for a reservoir at elevation 1,275 were approximated from curves drawn from cost estimates at the other elevations.

Cost estimates for the three replacement power options differ according to which hydropower components are included. For Replacement Power Option 1, the powerhouse would be located at the end of the diversion tunnel through the right abutment at the RM 286 dam site. Cost estimate worksheets in **Attachment C5** were developed for a four-unit powerhouse with 180 MW of generating capacity, consistent with the hydropower analysis results for an RM 286 reservoir with a net storage volume of close to 1,360 TAF. For further consistency with the hydropower analysis, the cost for a 160 MW facility associated with a 725 TAF reservoir at elevation 1,275 was approximated by adjusting for generating capacity. The same process was used to determine costs for a 150 MW facility at elevation 1,200 and a 170 MW facility at elevation 1,300.

A similar process was used for Replacement Power Option 2, involving a replacement Kerckhoff No. 2 Powerhouse. Cost estimate worksheets were specifically developed for a four-unit powerhouse with 180 MW of generating capacity; cost for that facility was then adjusted for generating capacities from 170 to 200 MW. For Replacement Power Option 3, a specific cost estimate was prepared for a 186 MW replacement turbine. That cost was adjusted for generating capacities down to 140 MW.

Analysis of the two hydropower configurations involving the new powerhouse at Millerton Lake and a retrofitted turbine at Kerckhoff No. 2 Powerhouse, respectively, also required that costs be determined for a river outlet works at the RM 286 dam, separate from a powerhouse. Cost estimates produced in July 2004, which were developed for a combined outlet works and powerhouse were separated to identify costs for an outlet works and for a 40 to 60 MW powerhouse at RM 286. These costs were derived from July 2003 cost estimates for CFRF dam types. Costs for powerhouse and outlet works subtotaled in those cost estimates were adjusted by apportioning costs for valves and pipes between powerhouse and outlet works subtotals. The separated cost for a 40 to 60 MW powerhouse is shown in **Table 5-9**. Details of the derivation are documented in **Attachment C1**.

Additional study would be needed to determine the costs to relocate Kerckhoff Lake recreation facilities and for any required environmental mitigation. Those potential costs are not included in the totals shown in the tables.

For all dam crest elevations, the dam and appurtenant structures would be located on public land. Parcels of land immediately upstream from the construction area and in the potential area of inundation are privately owned and would need to be acquired. Costs for acquiring private lands in the reservoir area are included in the construction costs shown for the dam and spillway in **Tables 5-7** through **5-9**. However, the cost to acquire any hydroelectric assets that would be abandoned, or to compensate their owners for any loss of future energy generation are not included in the tables. Construction costs of the competing replacement power options at the various reservoir sizes are shown graphically in **Figure 5-4**.

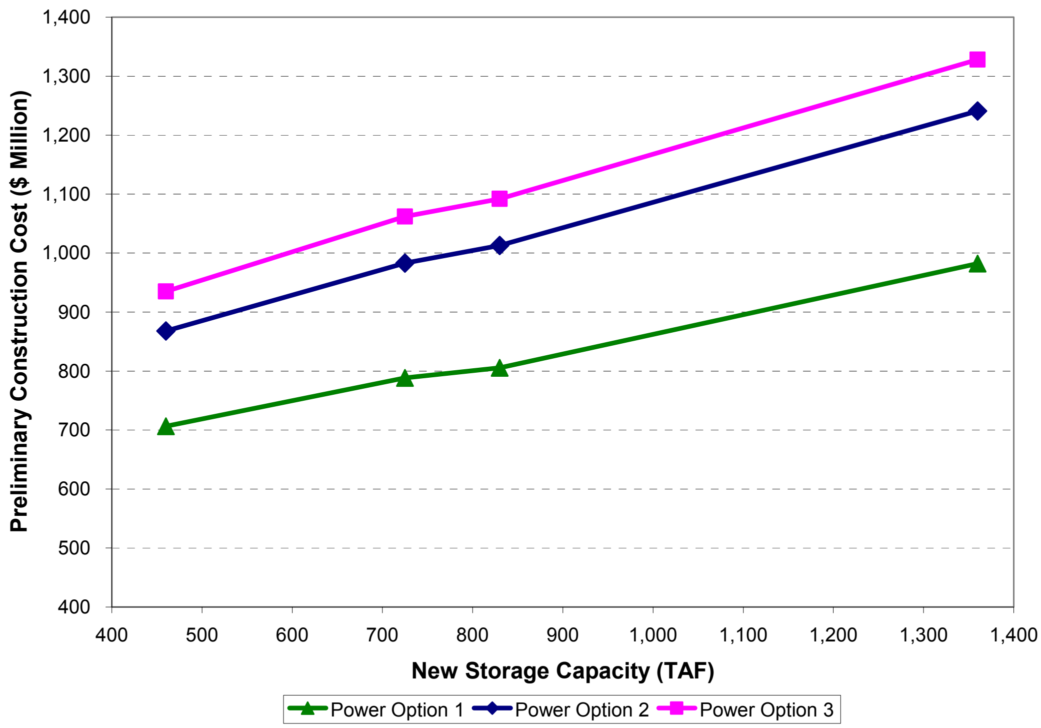

FIGURE 5-4.
CONSTRUCTION COSTS FOR TEMPERANCE FLAT RM 286 MEASURES VS. NEW STORAGE CAPACITY

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CHAPTER 6. FINE GOLD RESERVOIR

This chapter describes structural features and costs that would be involved in developing a reservoir on Fine Gold Creek. This chapter is structured similar to that of **Chapters 2** through **5**. It describes site conditions, engineering considerations associated with design and construction of the dam and appurtenant features, relocations or abandonments of existing facilities, and required land acquisitions. More detailed descriptions of potential new hydroelectric generation facilities are included in the **Hydropower TA**.

PREVIOUS STUDIES

In 1988, Parsons, Binkerhoff, Quade, and Douglas, Inc., proposed a pump storage system for a reservoir at Fine Gold. In February 1991, WAVE Engineers, Inc., prepared an Initial Operation Study for the Fine Gold Water Conservation Project on behalf of Madera Irrigation District. The study calculated the potential yield and power production of a 350 TAF reservoir on Fine Gold Creek (WAVE Engineers, 1991).

In 2000, a study conducted for the FWUA and NRDC coalition considered a 400 TAF reservoir at Fine Gold Creek as one of many alternatives for increasing potential water supply to the San Joaquin River (URS, 2000). In January 2002, DWR prepared a Draft Reconnaissance Study of Fine Gold Creek Reservoir, which revisited the previous Initial Operation Study.

SITE DESCRIPTION

Fine Gold Creek is located in Madera County near the community of Friant, about 23 miles northeast of Fresno. The general location is shown in **Figure 1-1**. The dam site is situated on the Fine Gold Creek arm of Millerton Lake, three quarters of a mile upstream from the creek's former confluence with the San Joaquin River. Friant Dam lies about 5 miles downstream of the confluence.

Topographic Setting

The Fine Gold Reservoir area is in a broad upland drainage basin with numerous hills and ridges separated by small, steep-sided drainage basins, such as Willow Creek, feeding into Fine Gold Creek.

The proposed dam site is located in the valley of the Fine Gold Creek arm of Millerton Lake. Fine Gold Creek flows generally southward. Elevations in the immediate area range from approximately elevation 520 at the proposed dam site to just over elevation 2,000 on the adjacent ridges. The ground surface at the right abutment rises steeply at a 2:1 horizontal to vertical ratio to about elevation 800, then flattens somewhat up to elevation 1,160, where it continues through a series of saddles to elevation 1,847 at Hulbert Mountain. The left abutment rises at an inclination of about 2:1 to about elevation 650, then flattens somewhat up to about elevation 1,100, before ascending through a series of saddles and hilltops to Crook Mountain at elevation 2,006.

Geologic Setting

Based on drill hole data and limited surface observations, the right abutment is underlain primarily by mica schist metasedimentary rock intruded by numerous granitic and aplite dikes, whereas the left abutment is underlain by granitic rock. The contact between these two rock units is an intrusive contact that was not crossed by drill holes. The mica schist on the right abutment is moderately weathered, moderately hard to moderately soft, intensely fractured mica schist (metasedimentary rock) intruded by numerous, very hard, granitic and aplite dikes. Rock mass permeability of the right abutment appears to be low in the mica schist (0.5 to 3 gpm) and higher (5 to 20 gpm) at contacts between the schist and dikes.

On the left abutment, at a small topographic saddle, a drill hole commencing at a ground surface elevation of 1,040 encountered a thick interval of decomposed to intensely weathered granite from the ground surface to approximately 130 feet deep and moderately to slightly weathered granite below 130 feet. Rock mass permeability of the left abutment was (0.2 to 2 gpm).

Downstream of the proposed dam axis, the steep, water-scoured shoreline of the Fine Gold Creek arm of Millerton Lake exposes discontinuous zones of gray to brown foliated metamorphic rock lenses within the more widespread granitic rock. The metamorphic rocks are typically intruded by light gray granitic dikes.

Alluvium of unknown thickness occurs below the existing reservoir water surface in the Fine Gold Creek channel. The alluvial material probably ranges from fine- to coarse-grained sands, with rocks that detached from the abutment slopes ranging up to 25 feet in maximum dimension. Unstable rock wedges, rock toppling, or evidence of landslides were not observed.

At the saddle dam site higher in the watershed, a drill hole located 800 ft into the potential reservoir area from the dam axis found very intensely weathered granite to a depth of 15 ft. Moderately weathered to slightly weathered granite was found from 15 to 35 ft. Beyond 35 ft the rock is slightly weathered to fresh granite.

Based on limited surface observations and data from an exploratory drill hole located 800 ft into the potential reservoir area from the proposed saddle dam axis, the two saddle dam abutments are underlain entirely by granite. The drill hole encountered very intensely weathered granite from the ground surface to 15 feet deep, moderately to slightly weathered granite from 15 to 35 feet deep, and slightly weathered to fresh granite below 35 feet. Rock mass permeability appears to be low (0 to 3 gpm).

Based on exploratory drill hole data and limited surface observations, the borrow area is underlain by saprolite (soil produced by in place weathering of rock), alluvium, or decomposed to intensely weathered granite to depths ranging from 15 to 55 feet. Below these depths is hard, slightly weathered to fresh granite. Drill holes typically encountered soil in the upper 15 to 55 feet classified as Silty Sand to Sandy Silt (SM-ML) according to the Unified Soil Classification System (USCS).

Details of the most recent observations of geologic conditions at the Fine Gold dam and reservoir site are provided in **Attachment E**.

Site Geotechnical Conditions

The proposed main dam site appears to be suitable for either a concrete or embankment dam. Deep weathering on the upper left abutment suggests that an embankment dam would be required in this area. Rock units and properties differ between the two abutments and will likely require more extensive geologic field investigations than would a site with more uniform geology. Relatively high permeability at the dike/schist contacts on the right abutment is a geotechnical concern that also requires additional study (Reclamation, 2005; included as **Attachment E**).

No unusual or problematic subsurface conditions were encountered at a drill hole located 800 ft into the potential reservoir area from the proposed saddle dam axis. The shallow weathering profile in the granite foundation indicates the site is suitable for an embankment dam and may be suitable for a roller compacted concrete (RCC) dam with a modest amount of additional excavation (Reclamation, 2005).

Seismic Hazard Analysis

No major through-going faults or shear zones have been identified in this area of the Sierra Nevada and historic seismicity rates are low. No known faults exist at the Fine Gold site. Ancient faults were encountered at Friant Dam during its construction, but they were inactive. Overall, potential seismic hazard potential at the site is low. Areal sources were found to be the controlling source of potential earthquakes for these and greater return periods (Reclamation, 2002b). Mean PHAs for the Fine Gold area are provided in **Chapter 2**.

Existing Facilities

Existing constructed facilities in the area include residences, roads, and the North Fine Gold On-Boat camping area. Facilities that could be inundated by a reservoir constructed at Fine Gold Creek shown are in **Figure 6-1.** The figure shows the maximum reservoir size contemplated for the site (i.e., the reservoir resulting from a dam crest at elevation 1,100).

Residential Property

Hidden Lake Estates, a residential development, overlooks Millerton Lake just downstream of where Fine Gold Creek joins the lake. It includes both developed and undeveloped parcels. Up to about 10 residences are upstream of the proposed dam site in the Fine Gold Creek watershed.

Roads

County roads 210 and 216 provide access from north of Millerton Lake into Hidden Lake Estates. Ralston Way and Hidden Lake Boulevard are also in the Fine Gold watershed.

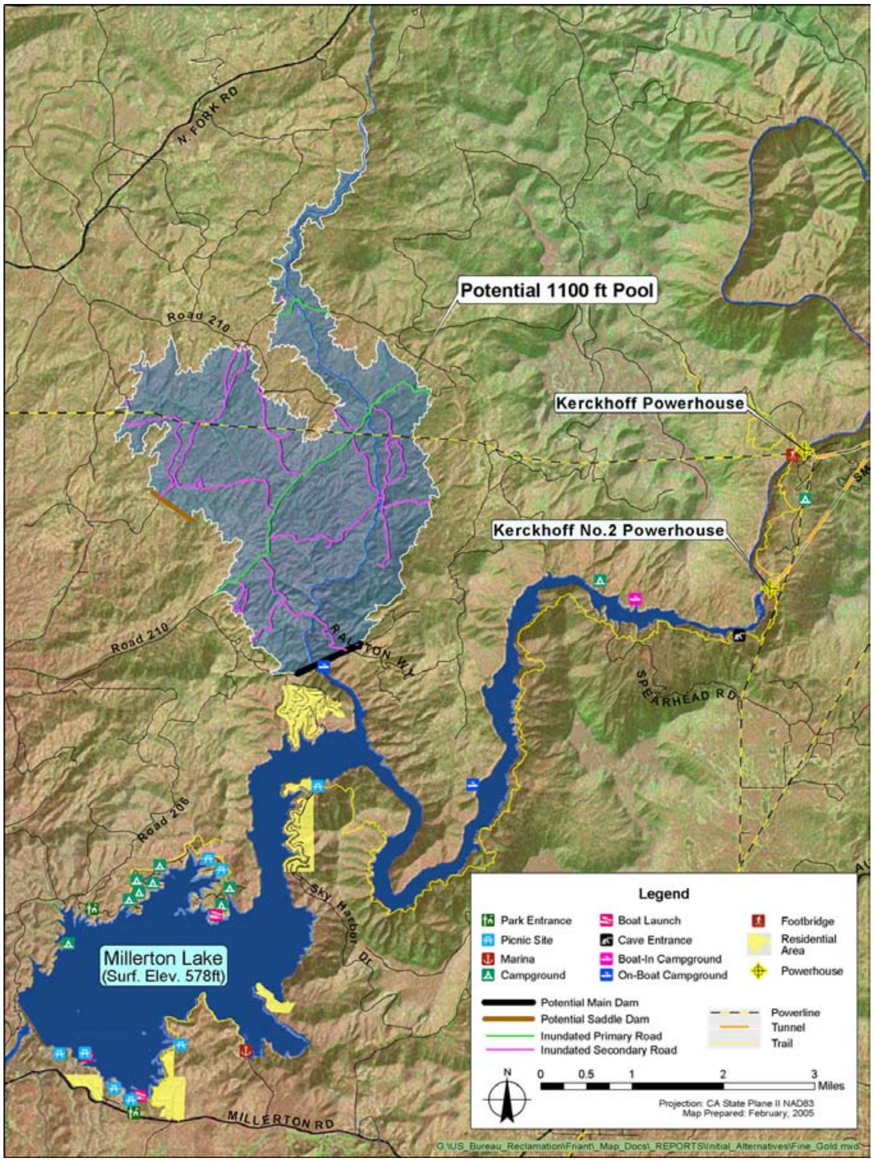

FIGURE 6-1.
POTENTIAL FINE GOLD RESERVOIR

POTENTIAL IMPROVEMENTS

Fine Gold Reservoir would be an off-stream storage facility. Water could be pumped from Millerton Lake for temporary storage in the new reservoir and then released back into Millerton when the water is needed for release from Friant Dam to the San Joaquin River, or for delivery through the Madera Canal or Friant Kern Canal. Fine Gold Creek would provide supplemental inflows. The approach to storage development is examined in this appendix.

An alternative approach for storing water off-stream in a Fine Gold Reservoir would involve making diversions from Kerckhoff Lake or from the San Joaquin River. One variation of this approach would involve gravity diversions. Water stored in Fine Gold Reservoir could be released later to Millerton Lake through a hydroelectric powerhouse. A second variation would involve pumping water from Kerckhoff Lake or the San Joaquin River to a larger Fine Gold Reservoir with a higher maximum water surface. In this case, some energy could be produced on release back to the point of diversion; other energy could be produced from releases to Millerton Lake. These concepts for filling Fine Gold Reservoir by diversion from Kerckhoff Lake or the San Joaquin River were proposed by stakeholders in spring 2004 during the scoping. No facility layouts or cost estimates were developed for these alternative approaches. Consequently, requirements for implementing these diversion concepts are not provided in this appendix.

Permanent features that would be constructed for storing water from Millerton Lake include a main dam with an uncontrolled spillway to pass flood flows, and river outlet works for some controlled releases. A dual-use pumping powerhouse and powerhouse would be required to pump water into Fine Gold reservoir from Millerton Lake and to generate electricity from releases back to Millerton Lake. Upstream and downstream cofferdams would be required for diverting Fine Gold Creek flood flows and keeping Millerton Lake out of the construction zone. Diversion tunnels to route flood flows around the construction zone also would be required. A saddle dam would be required for any reservoir with a surface elevation greater than approximately 1,000 feet.

Reservoir Storage and Area

The Fine Gold Creek dam site could support a reservoir with storage of up to 750 TAF with construction of a saddle dam. Reservoir surface area could extend over 5,000 acres. Curves showing potential net storage capacity and surface area for a reservoir on Fine Gold Creek are presented in **Figure 6-2**. Reservoir sizes examined in this appendix are shown in **Table 6-1**. Net storage volume accounts for approximately 190 acre-feet of existing storage in Millerton Lake above the proposed Fine Gold dam axis.

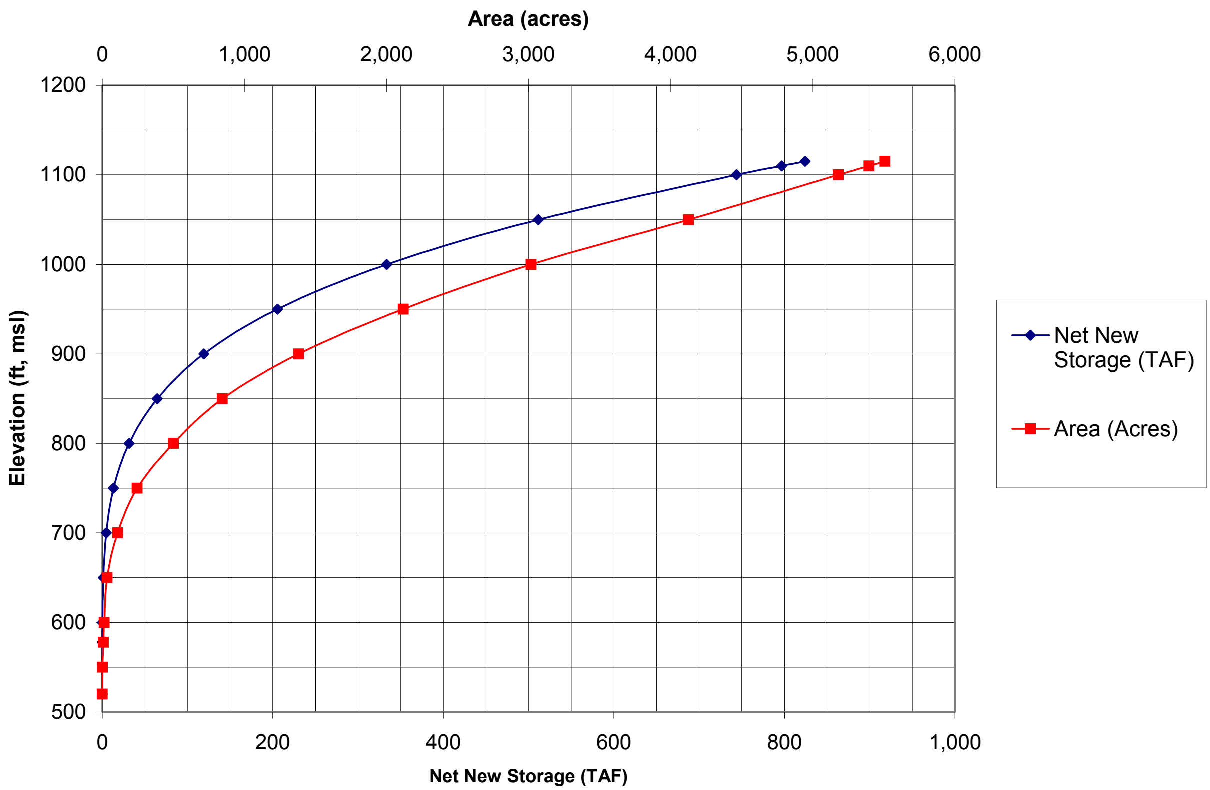

FIGURE 6-2.
FINE GOLD RESERVOIR SURFACE ELEVATION VS. NEW STORAGE AND AREA

TABLE 6-1.
SURFACE WATER STORAGE MEASURES EVALUATED FOR FINE GOLD RESERVOIR

Gross Pool Elevation (feet above msl)	Dam Height (feet)	Dam Types			Reservoir Area (acres)	Gross Storage Capacity (TAF)	Net Storage Capacity (TAF)
		RCC	CFRF	Arch			
900	380	X	X		1,380	120	120
1,100	580	X	X		5,180	740	740
1,110	590	X	X		5,400	800	800

Key:
RCC – roller-compacted concrete
CFRF – concrete face rockfill dam
msl – mean sea level
TAF – thousand acre-feet

Main Dam

The main dam on Fine Gold Creek, as described in this appendix, would be either a concrete face rockfill or RCC gravity dam. Other dam types were also considered.

Dam Types Considered

RCC and rockfill gravity dams are well suited for the Fine Gold Creek site. The foundation conditions are also excellent for a concrete arch dam. However, the abutments are uniform with relatively flat slopes, resulting in a wide canyon that would require potentially large volumes of concrete. Therefore, a conventional concrete arch dam was not considered for preliminary designs. Layouts and cost estimates were not prepared for a concrete thin arch dam, but this dam type could be considered in future studies for cost comparisons.

A central-core earthfill dam is not considered economically viable due to the limited availability of plastic, fine-grained materials for the core. An asphaltic-core earthfill dam might be viable for the site but was not considered due to limited use and experience with this type of dam in the United States.

Dam Sizes Considered

Two potential dam crest elevations were considered for developing initial engineering calculations and cost estimates: the lower one at elevation 900, and the other at elevation 1,100. The streambed at the proposed dam site is at elevation 520, resulting in dam heights of 380 feet and 580 feet, respectively. Analysis of hydroelectric energy generation potential and pumping energy requirements was conducted for a reservoir of 800 TAF. **Table 6-2** shows dam sizes for which preliminary layouts and construction cost estimates were produced or for which hydropower analyses were conducted along with the surface area and storage for the resulting reservoir. **Figure 6-1** shows the potential location of the dam and the area that would be inundated by the Fine Gold elevation 1,100 measure.

Concrete Gravity Dam Design

The RCC gravity dam layout, including appurtenances, is shown in **Attachment B6**. A representative cross section of the RCC gravity dam is shown in **Chapter 4**, **Figure 4-3**, and is presented with additional details in **Attachment B1**.

A preliminary design for an RCC dam is described in **Chapter 4** for the RM 274 site. Fundamentally, the same design would be used for the Fine Gold Creek site, although dimensions and quantities would differ. The design provides for a vertical upstream face and a 0.75H:1V downstream face. Preliminary stability analyses indicate that the design could be refined to employ a steeper downstream slope, especially for the dam with the lower crest elevation.

In developing preliminary cost estimates, excavation to remove overburden and weathered rock for the concrete gravity dam type was assumed to extend to an average depth of 4-ft. Due to access constraints, drilling could not be conducted in the vicinity of the channel to check this assumption. Assumed depths of grout and drainage holes that would be drilled into the foundation are shown in **Table 6-2**.

TABLE 6-2.
HOLE DEPTHS FOR RCC DAM AT FINE GOLD

Dam Crest Elevation (feet above msl)	Grout Hole Depth (feet)	Drainage Hole Depth (feet)
900	150	100
1,100	250	200

Key:
msl – mean sea level

Concrete Face Rockfill Dam Design

CFRF layouts, including appurtenances, are shown in **Attachment B6**. A representative cross section of the CFRF option is shown in **Figure 3-3** in the RM 274 chapter, and also is presented with additional detail in **Attachment B1**. Design characteristics of a CFRF structure at RM 274 are described in **Chapter 3**. Similar characteristics and assumptions apply to the Fine Gold Creek dam design, including grout hole spacing and depth.

Appurtenant Features

In addition to the dam, low-level outlet works are planned for water deliveries and reservoir evacuation capability. Since this site is off-stream from the San Joaquin River, a pumping powerhouse would be required to fill the reservoir. Outlet works and power generation capabilities would be integrated into the pumping plant and would discharge to Millerton Lake. An uncontrolled spillway would pass flood flows.

Appraisal level designs for the appurtenant structures were based on the assumption that Millerton Lake would be continuously operated within an approximate range of elevations 550 to 575. Storage at the Fine Gold site could be fed by gravity into Millerton Lake.

Diversion Works

Upstream and downstream cofferdams are required for diverting Fine Gold Creek flows during construction and preventing inundation of the site from Millerton Lake. Because the dam site is located within the influence of Millerton Lake, a downstream cofferdam would be necessary to allow normal operation of Millerton Lake during construction. An upstream cofferdam at a higher crest elevation than the downstream cofferdam would be required to pass diversion flows.

Diversion during construction for all dam options was based on passing a peak discharge of 5,000 cfs, which corresponds to an approximate 4 percent chance flood event (25-year return period). Development of the flood frequency table used to determine requirements for flood routing during construction is discussed further in the Construction Considerations subsection of this chapter. Downstream cofferdams would need to be placed to elevation 578 (approximate height 60 feet) for all options to accommodate normal reservoir operations for Millerton Lake.

Diversion Tunnel and Upstream Coffer Dam - Rockfill Dam

Diversion for the rockfill dam options would be accomplished by constructing a 14-foot-, zero-inch-diameter tunnel through the left abutment. After the dam was completed, the tunnel would serve as the reservoir outlet works.

To pass the specified flood flow, an upstream cofferdam for the rockfill dam options would need to be placed to elevation 655 (approximately 80 feet high).

Diversion Tunnel and Upstream Coffer Dam – Concrete Gravity Dam

Diversion for the concrete dam options would be accomplished by installing two 10-foot-, zero-inch-diameter steel conduits through the base of the diversion cofferdams and the main RCC dam. This installation would be removed following construction except where it passes below the RCC dam where it would be plugged.

The upstream cofferdam for the RCC dam options would need to be placed to elevation 590 (approximately 25 feet high).

Spillway

Design of the spillway for all options was based on passing a peak discharge of 30,000 cfs. This would be accomplished using an uncontrolled ogee crest spillway with a crest length of 150 feet, and a head of 15 feet.

CFRF Spillway Design

For the rockfill dam options, the spillway would be located on the left abutment. The downstream channel would be excavated through the existing rock abutment, and daylight into a natural draw that leads back into the reservoir. A reinforced concrete training wall would be constructed within the first 100 feet of the channel to control the flows within the vicinity of the dam. Energy dissipation would be controlled by the tailwater at the end of the natural channel, which would be in the range of 30 to 50 feet deep, depending on the level of Millerton Lake. For future designs, a labyrinth spillway should be considered for raising the crest elevation, providing more storage, and reducing the overall width of the spillway, including the outlet channel.

Spillway Design for RCC Dam

For the concrete dam options, the spillway overflow section would be located near the center of the dam. Guide walls would be provided to contain the flows within the width of the spillway crest. Energy dissipation would be accomplished as the flow passes over the stepped downstream face of the dam. A concrete cutoff would be planned at the toe of the dam to ensure undercutting does not occur. The depth of tailwater also is expected to be 30 to 50 feet for this option.

Outlet Works

The outlet works layout for both dam types would consist of a trashracked intake structure, a water conveyance system, and a regulating gate with an upstream guard gate. The energy from releases would be dissipated in the tailwater from Millerton Lake (plunge pool). The size of the conveyance system is dictated by diversion during construction (both CFRF options), reservoir evacuation (high RCC dam), or normal reservoir operation requirements (low RCC dam).

Bulkheads would be required for the intake structure, and the CFRF options would require an outlet within the upstream end of the tunnel. The number and size of regulating gates and guard gates for the low and high dam options were based on satisfying normal reservoir release requirements and reservoir evacuation requirements, respectively.

The spillway, in combination with outlet works discharges and reservoir surcharge capacity, would be capable of safely passing both the general storm PMF (peak inflow of 33,200 cfs, and 5-day volume of 59,400 acre-feet) and/or the local storm PMF (36,100 cfs peak inflow and 2-day volume of 17,100 acre-feet).

Pumping and Generating Powerhouse

The majority of water for filling Fine Gold reservoir would be pumped from Millerton Lake. Releases made when storage levels in the new reservoir were high would be used to generate hydroelectric energy.

Three pumps and one pump turbine would be required for the head range that may be encountered during normal operations. The powerhouses and turbines would be housed in a powerhouse at the toe of the RCC dam options, and at the downstream end of the diversion tunnels for the CFRF options. The combination of reservoir head and capacity, and current operating procedures at Friant Dam were used to select the number and size of pumps and to set a reasonable operating range for power generation. Power generation with lower reservoir levels does not appear to be economically feasible, so less expensive “pure pumps” were provided for those ranges of reservoir level where power generation benefits would be minimal. At the lower reservoir levels, releases would be made through the outlet works bypass valves. **Table 6-3** shows the configuration of pumps and pump turbines for the two dam levels.

**TABLE 6-3.
PUMP AND TURBINE CONFIGURATIONS FOR
FINE GOLD SURFACE WATER STORAGE MEASURE DESIGNS**

Facility	Dam Crest Elevation 900Head Range (feet)	Dam Crest Elevation 1,100 Head Range (feet)
No. 1 pump	25 to 55	50 to 90
No. 2 pump	55 to 100	90 to 160
No. 3 pump	100 to 180	160 to 290
No. 1 pump turbine	180 to 330	290 to 530

Power generated and sold from the project would offset much of the costs for the power required to fill the reservoir by pumping. The assumed generating capacity of the Fine Gold powerhouse is about 40 MW for the low dam and 60 MW for the high dam. The new powerhouse could be connected to the existing grid in the Friant/Kerckhoff region.

Saddle Dam

For the dam option with a crest at elevation 1,100, a saddle dam would be required on the right reservoir rim. The saddle dam would be located about two miles northwest of Fine Gold Dam site and would block a topographic low where the reservoir rim dips to about elevation 1,038.

The saddle dam would be about 100 feet high and 3,200 feet long. The cross section would consist of a central core, upstream and downstream filter zones with an additional transition zone, riprap, and downstream slope protection.

Construction Considerations

This section discusses issues related to construction of the potential dam, reservoir, and appurtenant features.

Foundations

As indicated previously, foundations at the proposed main dam site appear suitable for either a concrete or embankment dam. However, rock units and engineering properties differ between the right and left abutments. The complex geology and current lack of detail on the contact between the rock units will likely require additional geologic investigation and may require special foundation treatment (Reclamation, 2005, included asn **Attachment E**).

In addition, decomposed or intensely weathered rock to a depth of 120 ft at a small topographic saddle on the upper margin of the left abutment of the main dam site, at an approximate ground surface elevation of 1,100, suggests that an embankment dam, not concrete, would be required for a dam with a crest that would extend as far as the saddle. However, observations of abundant, hard granite outcrops on the principal portion of the left abutment indicate that the extremely deep weathering is thought to be restricted to the saddle, not the main abutment slope (Reclamation 2005; see **Attachment E**).

For the proposed saddle dam higher in the watershed, a drill core on the reservoir side of the dam axis indicates that only routine foundation preparation would be required for an embankment saddle dam. The site may be made suitable for an RCC saddle dam with a modest amount of additional excavation, to a depth of approximately 15 ft.

Flood Routing During Construction

A frequency curve of peak flow values for the basin was developed as part of flood studies to determine diversion flow requirements during construction (Reclamation, 2002). These data were developed using the USGS National Flood Frequency program. Results of this study are presented in **Table 6-4**. A peak discharge of 5,000 cfs was used to size the diversion structures for each option with a return period of about 25 years.

TABLE 6-4.
PEAK FLOOD FLOWS FOR FINE GOLD CREEK

Return Period (years)	Peak Flow (cfs)
2	960
5	2,160
10	3,110
25	5,160
50	6,710
100	8,910

Borrow Sources and Materials

Crushed aggregate could be obtained from hard, slightly weathered to fresh granitic rock that is present at depths ranging from 15 to 55 feet. Hard, fresh granite could be quarried most economically near CR 210 where depth of excavation appears shallowest. Saprolite soil and decomposed to very intensely weathered granite occurring over most of the reservoir area would provide a source of silty sand (SM) fill with low plasticity fines.

Construction Site Access

Access to the right abutment through the uplands and the upper left abutment is restricted by private land and/or roads. Access to other areas at the dam site is generally over public roads or across public lands. Significant portions of the reservoir areas can be accessed only by private roads or across private land.

Based on visual inspection of utility markers, no pipeline, communication, or power easements travel through the dam site. Types and locations of utilities serving Hidden Lake Estates above elevation 1,100 near the dam site were not identified. Typical utilities and easements serving residences are located in the reservoir area.

Staging Areas

Areas for construction use, staging, and laydown likely would be located upstream of the dam site within the reservoir area.

Lands and Rights-of-Way

Up to about 10 residences are upstream of the proposed dam site in the Fine Gold Creek watershed. Hidden Lake Estates is situated below the dam site and would not be inundated. All lands within the reservoir area are private and would need to be acquired. Table 6-5 shows the amount of private land that would need to be acquired for the reservoir options. Rights-of-way or easements that may be needed to construct new facilities or relocate existing facilities have not been determined and are excluded from the land areas shown.

TABLE 6-5.
FINE GOLD RESERVOIR AREA LAND REQUIREMENTS

Description	Fine Gold Reservoir Measures			
Gross pool elevation (feet above msl)	900	1,020	1,100	1,110
New storage capacity (TAF)	120	400	740	800
Estimated inundated area (acres)	1,373	3,438	5,298	5,407
Estimated public inundated acreage	0	0	0	0
Estimated private inundated acreage	1,373	3,438	5,298	5,407

Key:
msl – mean sea level
TAF – thousand acre-feet

Electric Power Sources

Electric power, including high voltage grid power, is available from the Kerckhoff branch. Electric power, including lower voltage electrical service, is available from existing trunks supplying local residences.

Relocations and Abandonments of Affected Facilities

Residences and portions of roads would be subject to inundation by Fine Gold Reservoir and would need to be abandoned or relocated. Requirements for relocating these facilities were not determined.

Roads

Portions of County roads 210 and 216, Ralston Way, and Hidden Lake Boulevard would be subject to inundation by the elevation 1,100 reservoir measure. In total, for a reservoir with a gross pool at elevation 1,100, approximately 4.5 miles of paved road and 17 miles of unpaved road would be inundated. A lesser amount would be inundated for a reservoir option at a lower elevation.

Road 210

Road 210 connects Hildreth with the communities of O’Neals and Indian Springs. Portions of the road would need to be relocated to prevent Hildreth’s isolation. To accomplish this for the elevation 900 reservoir option, it was calculated that approximately 2.4 miles of new road, and two relatively short bridges totaling approximately 500 feet in length may need to be constructed. This would preserve the most direct connection routes to both Indian Springs and O’Neals. For the larger, elevation 1,100 reservoir option, 1.5 miles of road and one span of new bridge approximately 500 feet in length would need to be constructed. The larger reservoir inundation area would prevent the most direct connection to Indian Springs from being preserved, but the assumed relocation route would preserve the connection to O’Neals, from which wider access could be obtained.

Construction Costs

Costs for constructing Fine Gold Reservoir options at July 2004 price levels are summarized in **Table 6-6**. Field costs for the dam and appurtenant features are based on worksheets presented in **Attachment C6,** originally calculated at July 2003 price levels. Those costs were adjusted to reflect July2004 prices and a higher contingency. Costs include indirect costs and costs for acquiring private lands. Indirect costs for planning, designs, and construction management of structural components are equal to approximately 25 percent of field costs. An allowance of 20 percent of lands costs also has been included for indirect costs associated with property acquisition.

Costs for relocating Road 210 and constructing 500 feet of bridge were approximated and are included in the cost estimate. Additional study would be needed to determine the costs for any required environmental mitigation. Those potential mitigation costs are not included in the totals shown.

TABLE 6-6.
CONSTRUCTION COSTS FOR FINE GOLD RESERVOIR MEASURES ($ MILLION)

Gross Pool Elevation (feet above msl)	900		1,020	1,100		1,110
New Storage Capacity (TAF)	120		400	740		800
Dam Type	RCC	CFRF	CFRF	RCC	CFRF	CFRF
Components						
Dam, Appurtenant Features, Reservoir Lands[1]	240	240	430	640	590	610
Road 210 Relocation	28	28	23	18	18	18
Road 210 Bridges	15	15	15	15	15	15
Construction Costs[2,3]	**283**	**283**	**468**	**673**	**623**	**643**

Key:
CFRF – concrete face rockfill
msl – mean sea level
RCC – roller-compacted concrete
TAF – thousand acre-feet

Notes:
[1] Appurtenant features include spillway, diversion during construction, outlet works, and pumping-generating plant.
[2] All cost estimates are preliminary. Construction cost represents the sum of field costs and indirect costs for planning, engineering, design and construction management, estimated at 25 percent of field costs.
[3] Costs do not include environmental mitigation, new or relocated recreation facilities.

Land required for developing the reservoir is apparently in private ownership, and would have to be acquired. Private land within the proposed reservoir includes up to about 10 residences for the 800 TAF reservoir, and two residences for the 120 TAF reservoir. **Figure 6-3** shows the relationship between construction cost and new storage capacity for Fine Gold Reservoir .

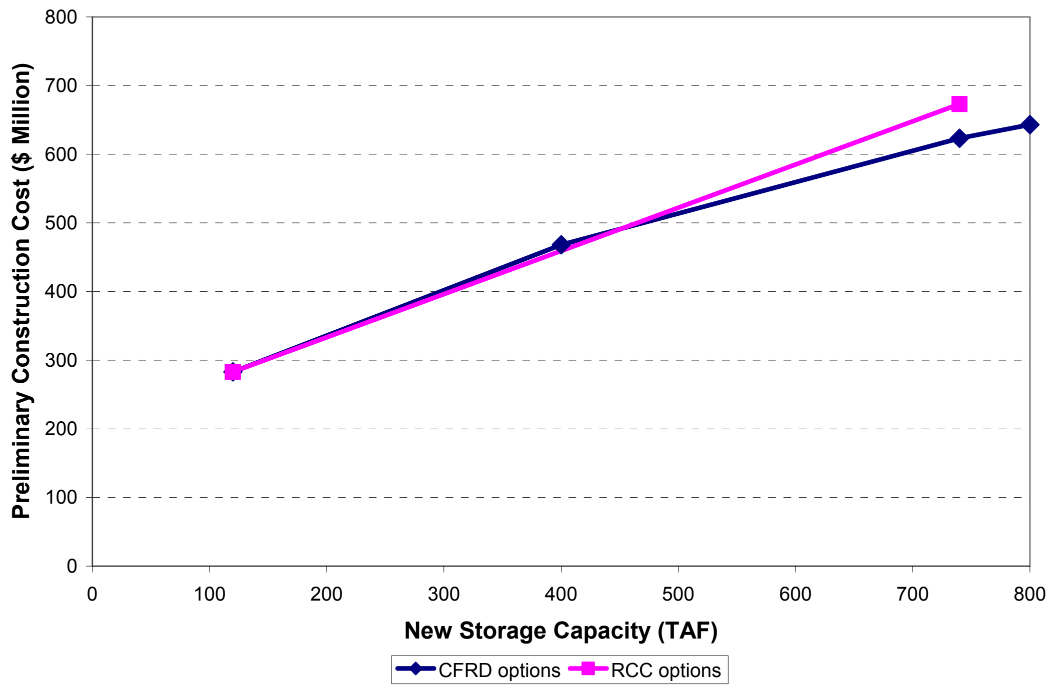

FIGURE 6-3.
CONSTRUCTION COSTS FOR FINE GOLD RESERVOIR MEASURES VS. NEW STORAGE CAPACITY

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CHAPTER 7. YOKOHL VALLEY RESERVOIR

This chapter describes structural features and costs that would be involved in developing a reservoir at Yokohl Valley. This chapter is structured similar to that of **Chapters 2** and **3**. It describes site conditions, engineering considerations associated with design and construction of the dam and appurtenant features, relocations or abandonments of existing facilities, and required land acquisitions. More detailed descriptions of existing and potential new hydroelectric generation facilities are included in the **Hydropower TA**.

PREVIOUS STUDIES

In April 1958, Reclamation personnel conducted a brief reconnaissance of 5 alternative dam sites for a potential reservoir at Yokohl Valley. The site currently being considered was identified as the preferred site. A reconnaissance level cost estimate was prepared for an earthen dam and reservoir of approximately 450 TAF. In February 1964, Reclamation prepared a cost allocation estimate for the reservoir.

The prior cost estimate was updated IN 1972. Reclamation conducted a geologic site visit IN 1975, recalculated quantities of materials that would be needed to construct the dam, and prepared a new cost estimate for a 450 TAF Yokohl Reservoir. In June 1980, Reclamation developed maps illustrating proposed borrow areas for constructing Yokohl Dam and Reservoir, and surface geology for Yokohl Reservoir and tunnel alignment. Hydrologic conditions were evaluated in the Corps' Kaweah River Basin Hydrology Report of August 1990.

In 2000, a study conducted for the FWUA and NRDC Coalition considered a reservoir on Yokohl Creek as one of many initial alternatives for increasing potential water supply to the San Joaquin River. The study entertained the concept of a dam up to 320 feet high at Yokohl Valley with an 8-mile-long, 10-foot-diameter diversion tunnel to divert excess water from Lake Kaweah. The study indicated that such a reservoir could store nearly 950 TAF of water (URS, 2000). The study contained no designs for this larger dam option, and limited technical information. Elevation differences between Lake Kaweah and Yokohl Valley indicate that filling a reservoir at Yokohl greater than approximately 120 TAF with water diverted from Lake Kaweah would require pumping.

SITE DESCRIPTION

Yokohl Valley Dam and Reservoir would be located in Tulare County, near the community of Exeter, about 16 miles east-southeast of Visalia. The dam site is located in Yokohl Valley about 8 miles southeast of the confluence of Yokohl Creek with the Kaweah River. The general project location is shown in **Figure 1-1**.

Topographic Setting

Regional topography consists of the nearly level floor of the San Joaquin Valley rising abruptly to moderately steep, northwest-trending foothills with rounded canyons. Elevations in the immediate area range from about elevation 530 to over elevation 1,300.

Farther east, the terrain steepens and the canyons become more incised. The canyons have been cut by southwest- to west-flowing rivers and associated large tributaries. The Kaweah River is the main river in the area. Yokohl Creek is a west- to northwest-flowing tributary to the Kaweah. Its confluence with the Kaweah River is about 8 miles downstream of Terminus Dam.

The proposed dam site is located about 4 miles south-southeast of the mouth of Yokohl Valley. At the proposed dam site, the valley floor is relatively broad (about 2,000 feet). The streambed at the proposed dam site is approximately at elevation 550. The left abutment rises at a moderately steep 3:1 slope (horizontal:vertical), while the right abutment is about 5:1, steepening to 3:1 above the proposed crest elevation. The left ridge rises to about elevation 1,200 and the right ridge to about elevation 1,500.

Geologic Setting

Yokohl Valley is located near the boundary of the Sierra Nevada Geomorphic Province and the San Joaquin Valley portion of the Great Valley Geomorphic Province. The Great Valley basin is filled with thick accumulations of marine (at depth) and non-marine sediments shed largely from the Sierra Nevada mountain range. Recent alluvium of lake and river origin blanket most of the present-day surface, while dissected remnants of Pleistocene alluvial fans rim the valley margin.

The Sierra Nevada range is characterized by batholiths of Mesozoic granitic rock and Paleozoic roof pendants of the Calaveras Complex and related rocks. The Sierra Nevada foothills take the form of outliers of low to irregular hills of Mesozoic granitic, and late Paleozoic to Mesozoic basic and ultrabasic rock (ophiolites) of the “serpentine belt” of the Kings-Kaweah suture, and other associated Mesozoic metamorphic rocks.

The west- to northwest-trending Yokohl Valley is located in what may be an erodible zone along a geologic contact between granitic rocks and a roof pendant of pre-Cretaceous metasedimentary rock. At the dam site, an undated Reclamation geologic map shows that pre-Cretaceous metagabbro and Mesozoic ultrabasic intrusive (serpentinite and talcose serpentinite) rocks are found in both proposed dam abutments. Pre-Cretaceous amphibolite also is found in the right abutment.

The perimeter of the potential reservoir is surrounded by Mesozoic granitics (quartz diorite), basic and ultrabasic intrusive rocks, and pre-Cretaceous metasedimentary rocks. Clayey slopewash (colluvium) and talus blanket much of the lower valley slopes.

Relatively thick Pleistocene and recent river alluvium deposits of sand, gravel, and possible silt are found beneath the floor of Yokohl Creek. Alluvium in the lower Yokohl Valley (downstream of the dam site) ranges from 170 to 275 feet thick. At Gill Ranch (upstream of the dam site), the alluvium is about 30 to 50 feet thick. The borehole advanced under the downstream toe of the potential dam, as part of the geologic investigation, extended to a depth of 87 feet below ground surface without encountering bedrock.

Site Geotechnical Conditions

Geologic mapping conducted by Reclamation as part of its geologic investigation shows that the dam site, two saddle dams, and tunnel connecting to the San Joaquin Valley floor would be founded largely on ultrabasic rocks variably altered to serpentine. The serpentine is dark green and massive, and was considered sound. It locally grades to dark to light green schistose to sub-schistose serpentinized rock. In the abutment areas, the serpentinite forms bold to inconspicuous outcrops that are lightly to moderately weathered and moderately jointed.

Jurassic meta-gabbro is found in both abutments and as a cap to the right abutment ridge. It is found as irregular to crudely linear, north- to northwest-trending, steeply to moderately eastward-dipping intrusive bodies. Minor amounts of talcose serpentine, talc, talc schist, chert, and amphibolite are found as inclusions in the serpentine, or along serpentine contacts. The softer talcose/schistose materials occur as infrequent, variably sheared stringers that are typically covered by slopewash.

Pre-Cretaceous metavolcanics rocks consisting mostly of metabasalt and amphibolite also are found near the dam site. The metabasalt is hard, gray to dark green, and fine-textured to locally porphyritic, forming lightly weathered to fresh craggy outcrops. The amphibolite is hard, dark green, lightly weathered, mostly fine-textured, and massive, grading locally to schistose. It is typically found as lenses in, or associated with, other rock types.

Seismic Hazard Analysis

No significant faults have been identified in the vicinity of the potential dam and reservoir sites. Overall, potential seismic hazard potential at the site is low. Areal sources were found to be the controlling source of potential earthquakes for these and greater return periods. For return periods of 2,500, 5,000 and 10,000 years, the mean PHA was determined to be 0.13-, 0.18- and 0.23-g, respectively.

Existing Facilities

Terminus Dam and Lake Kaweah are located north by northeast of the dam site on the Kaweah River approximately 8½ miles upstream of its confluence with Yokohl Creek. The Friant-Kern Canal passes within a mile and a half of the proposed reservoir margin, to the west of Yokohl Valley.

Existing facilities in the immediate area of the proposed dam and reservoir site include a few residences and cattle ranching structures (*e.g.*, shelters, feed storage, corrals, fences), Yokohl Drive, and private, unpaved roads apparently providing access to grazing lands. One of the residential sites is downstream of the potential dam site; two are within the potential reservoir area. A high voltage power transmission corridor crosses the reservoir area from north to south and consists of two parallel transmission lines and towers. **Figure 7-1** shows the potential inundation area of the maximum reservoir size contemplated in the Investigation and existing facilities lying within it.

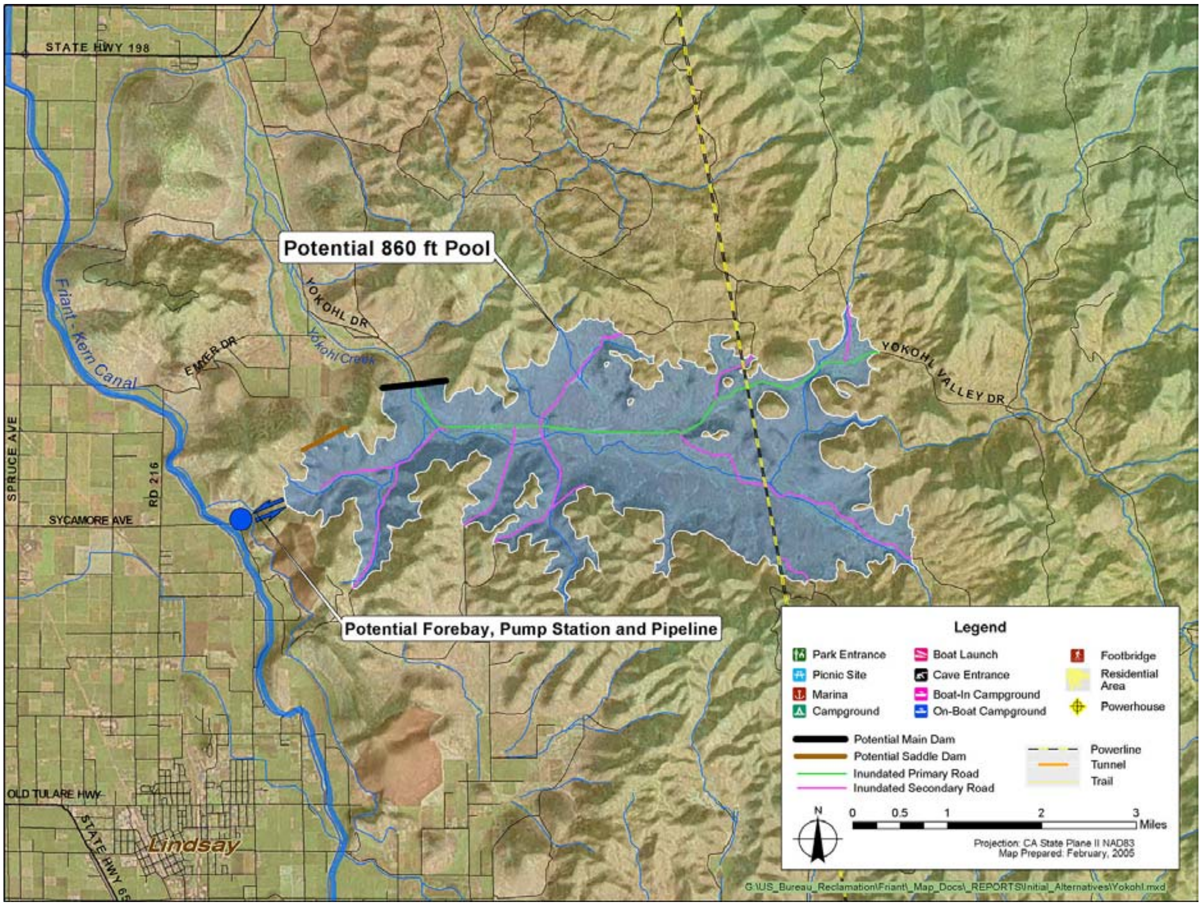

FIGURE 7-1.
POTENTIAL YOKOHL VALLEY RESERVOIR

POTENTIAL IMPROVEMENTS

A 260-foot-high earthfill dam would be constructed at Yokohl Valley creating a reservoir with a storage capacity of approximately 450 TAF. Water would be pumped from the Friant-Kern Canal and supplemented by local runoff in Yokohl Creek. Water stored in Yokohl Valley Reservoir would be conveyed back to the Friant-Kern Canal, to supplement deliveries from Millerton Lake or to offset releases from Millerton Lake to the San Joaquin River. Appurtenant features are described in a subsequent subsection of this chapter.

Reservoir Storage and Area

Yokohl Valley Reservoir, as proposed in 1975 by Reclamation, would cover 4,550 acres and have a storage capacity of approximately 450 TAF. The normal maximum water surface would be at elevation 791 and the maximum water surface during flood conditions would be at elevation 798. A reservoir with a storage volume of approximately 800 TAF would rise to approximately elevation 860. Curves showing reservoir storage and area versus elevation are shown in **Figure 7-2**.However, no specific design has been developed for this larger 800 TAF reservoir.

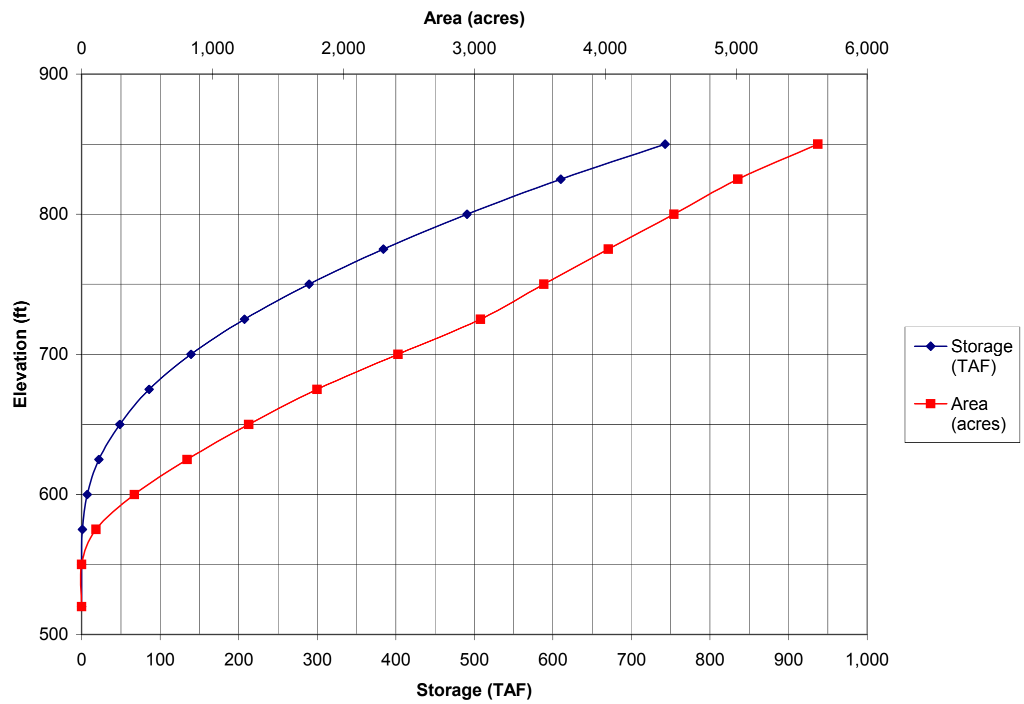

FIGURE 7-2.
YOKOHL VALLEY RESERVOIR SURFACE ELEVATION VS. NEW STORAGE AND AREA

Main and Saddle Dams

As proposed by Reclamation in 1975, Yokohl Valley Dam would be a 260-foot zoned earthfill structure 2,960 feet in length with a crest 30 feet wide at an elevation of 805 feet above mean sea level (elevation 805). The axis of the dam would extend east-west across Yokohl Creek.

The upper portion of the upstream dam face would be sloped at 2½:1, while the lower portion would be at 3½:1. The downstream dam face would be at 3:1 in the upper portion and 3½:1 in the lower portion. A 1975 drawing of the dam design did not include filter material on the downstream side of the impervious core. Accordingly, the dam design would need to be updated to current standards.

In addition to the main dam, two small saddle dams in the hills southwest of the dam would be required to contain the reservoir.

About 12.6 million cy of earth materials would be required to construct the embankment and associated small saddle dams for the 450 TAF reservoir. Of that, about 9.6 million cy of impervious material would be required, 2.8 million cy of rockfill, 0.1 million cy each of sand/gravel blanket materials, and rock slope protection material.

A conceptual Yokohl Valley Reservoir with 800 TAF of storage was analyzed for potential yield and hydroelectric energy generation and pumping requirements. However, as indicated above, no dam design was developed for this larger storage option.

Appurtenant Features

In addition to saddle dams, other appurtenant features would include a spillway, outlet works, a pumping/generating plant, and a tunnel or pipeline for conveyance between the reservoir and Friant-Kern Canal.

Spillway and Outlet Works

The spillway would be located on the left abutment and would be an ungated ogee type with a capacity of 1,220 cfs.

Outlet works for flood releases of up to 700 cfs to Yokohl Creek were included in the 1975 design. A 1,480-foot-long, 6.5-foot-diameter tunnel would connect to a gate chamber, from which flow through a 4-foot-diameter penstock would be controlled with a slide gate. The penstock would extend 210 feet to a control house equipped with a 10-inch-diameter needle valve for releases of up to 45 cfs, and a 4-foot-diameter slide gate for flood control releases. Releases would proceed through a stilling basin to the downstream portion of Yokohl Creek.

Conveyance and Pumping/Generation Facilities

Yokohl Valley Reservoir would be filled with water from the Friant-Kern Canal. This would require constructing an intake channel or forebay to connect with the Friant-Kern Canal, a pump-generation station, switchyard, lined tunnel, and an inlet-outlet structure at the reservoir.

As considered by Reclamation in 1972, an intake channel would be 300 feet in length, with a 20-foot bottom width. A pumping powerhouse would contain four units, each capable of operating at 250 cfs with 400 feet of pumping head. A 7,600-foot-long, 11-foot-diameter, concrete-lined tunnel would convey the pumped water to the reservoir.

The current cost estimate assumes an intake channel would be constructed, but differs from assumptions in the 1970s regarding pumping capacity and tunnel diameter. The current assumption is that a 120 MW pumping and generating station would be constructed that is capable of delivering 2,000 cfs and generating at 2,800 cfs. The concrete-lined tunnel connecting to the reservoir would be 15 feet in diameter.

A potential site for a forebay is located on the east side of the Friant-Kern Canal, about three quarters of a mile northeast of the small community of Tonyville. It is a relatively level, roughly triangular parcel of agricultural land within a small valley at the base of adjacent low mountains. Based on USGS topographic maps (20 -foot contour intervals), it appears that the forebay could potentially cover about 15 to 20 acres. This would be adequate for the required submergence of the pump/generator equipment to ensure good inflow and outflow conditions at the pump/generating station and in the Friant-Kern Canal, and to maintain the hydraulic grade in the Friant-Kern Canal. Requirements for emergency dewatering of the tunnel without disturbing the hydraulics and hydraulic gradient of the Friant-Kern Canal were not considered in this analysis. Water from this forebay would be pumped into the reservoir through the adjacent mountain via an approximately 1-to 1½-mile long tunnel.

As part of the FWUA/NRDC study, a gravity tunnel from Lake Kaweah was proposed to provide excess floodwater from the Kaweah River (URS, 2000). An 8-mile-long, 10-foot-diameter diversion tunnel would be required to divert water from Kaweah Lake.

The Kaweah diversion concept would not be able to store 450 TAF of storage, however, without pumping. The pool elevation of Lake Kaweah is elevation 694, while the planned normal pool for Yokohl Reservoir is elevation 791. Implementation of the Kaweah diversion concept would require lowering the Yokohl Valley Reservoir normal pool to elevation 694, thereby reducing storage to about 121 TAF. Alternatively, pumping facilities would need to be constructed, in addition to the tunnel itself. Some length of pressure pipeline also would be required to connect with, replace, or line the diversion tunnel.

Construction Considerations

This section discusses issues related to construction of the potential dam, reservoir, and appurtenant features.

Foundations

It is anticipated that the dam foundation would be in relatively hard rock with relatively tight, medium to closely spaced fractures and joints. Pre-split drilling and light blasting might be required for excavation. Some soft, sheared zones could be encountered, but they could be backfilled with lean concrete for minor dental preparation of the foundations.

The spillway would be founded in generally sound serpentine alternating with hard meta-gabbro. Spillway excavation would be in material composed of about 75 percent rock consisting of lightly to moderately weathered, moderately jointed rock with occasional intensely fractured zones. Slope-wash and residual soils are expected to be about 2 to 5 feet thick. The apron of colluvium at the base of the slope is expected to be about 4 to 15 feet thick.

Flood Routing During Construction

A tunnel or pipe for diversion of Yokohl Creek flood flows during the construction period could be required. It might be possible to use the outlet works for this purpose. Flood data were not readily available for determination of diversion requirements.

Borrow Sources and Materials

A 1980 map illustrating potential borrow sources suggests that sufficient impervious, pervious, and riprap materials are located within 2 miles upstream and downstream of the proposed dam site. It is not clear, however, whether a detailed evaluation of borrow source volumes was conducted at that time.

Portland cement is available from nearby commercial sources, including six producers within a few hundred miles of the project site. Bulk transport to the site could be conducted either by truck or, preferably, railcar. Pozzolan is available from producers in Stockton or Sacramento.

Construction Site Access

Yokohl Drive provides direct access to the proposed dam site.

Staging Areas

Potential staging and laydown areas are located immediately upstream and downstream of the project site.

Lands and Rights-of-Way

Two rural residential sites are situated within the potential reservoir area on either side of Yokohl Drive and would need to be acquired. All lands within the reservoir area are private and would need to be acquired. Table 7-1 shows the amount of private land that would need to be acquired for the reservoir options. Rights-of-way or easements that may be needed to construct new facilities or relocate existing facilities have not been determined and are excluded from the land areas shown.

TABLE 7-1.
YOKOHL VALLEY RESERVOIR AREA LAND REQUIREMENTS

Description	Yokohl Valley Reservoir Measures	
Gross pool elevation (feet above msl)	790	860
New storage capacity (TAF)	450	800
Estimated inundated area (acres)	4,292	5,864
Estimated public inundated acreage	0	0
Estimated private inundated acreage	4,292	5,864

Key:
msl – mean sea level
TAF – thousand acre-feet

Electric Power Sources

Electrical power is likely to be available from sources in Exeter or along Highway 198.

Relocations of Affected Facilities

Relocation would be required for the power transmission infrastructure in the potential reservoir area, and for Yokohl Drive.

Electric Transmission Lines

Approximately 2 miles of high voltage power transmission lines, supported by towers, would be inundated by the 450 TAF reservoir. A new route around the reservoir would involve constructing approximately 6 miles of transmission corridor. For a 800 TAF reservoir, approximately 8 miles would be needed.

Yokohl Drive

Yokohl Drive, which runs the length of Yokohl Valley, would need to be relocated. For a 450 TAF reservoir, approximately 16.5 miles of Yokohl Drive would be inundated, and about one half mile more of the road for a 800 TAF reservoir. Relocating the road would require improving about 7 miles of existing road for the smaller reservoir size. The larger reservoir would require nearly 8 miles of improved road and close to 1 mile of new road.

Construction Costs

Costs for constructing Yokohl Valley Reservoir at July 2004 price levels are summarized in **Table 7-2**. Field costs for Yokohl Valley dam and appurtenant features for a 450 TAF reservoir are based on worksheets presented in **Attachments C1** and **C7**. The worksheets in **Attachment C7**, developed in 2003, are based on the 1975 Reclamation cost estimate, revised to reflect July 2003 material costs. The cost estimate for the dam reflects the 1975 design and does not include the cost of filter material that would be required under current engineering standards.

Costs for items not detailed in the 1975 Reclamation cost estimate, but included in the worksheets (i.e., intake channel, pump station, switchyard, and tunnel), are based in part on parameters identified in 1972 Reclamation documentation and supplemented with knowledge about similar types of projects and general project conditions. The construction cost estimate developed in July 2003 has been adjusted to reflect July 2004 prices and a higher contingency allowance. Indexing to July 2004 prices was accomplished using Reclamation construction cost trends.

Costs for removing and relocating powerlines in Yokohl Reservoir were calculated in 1974 and updated to July 2004 prices. Approximate costs for relocating Yokohl Drive also are included in **Table 7-2**. Costs for potential environmental mitigation were not determined and, therefore, are not included in the totals shown. Costs for the dam, appurtenances, and reservoir presented include indirect costs and lands costs. Indirect costs for planning, designs, and construction management of structural components are 25 percent of field costs. Land costs were calculated by multiplying the privately held reservoir inundation area by a representative cost per acre and adding a value for improvements on the few residential properties in Yokohl Valley. An allowance of approximately 20 percent of land costs was included for indirect costs associated with property acquisition.

TABLE 7-2.
CONSTRUCTION COSTS FOR YOKOHL VALLEY RESERVOIR MEASURE
($ MILLION)

Gross Pool Elevation (feet above msl)	**790**
New Storage Capacity (TAF)	**450**
Components	
Dam, Appurtenant Features, Reservoir Lands[1]	400
Yokohl Drive Relocation	56
Transmission Lines Relocation	12
Construction Cost [2,3,4,5]	**468**

Key:
msl – mean sea level
TAF – thousand acre-feet

Notes:
[1] Appurtenant features include spillway, diversion during construction, outlet works, pumping-generating plant, and conveyance between the Friant Kern Canal and the reservoir.
[2] Costs for the 450 TAF size are based on an index-adjusted cost estimate prepared in 1975 and are likely low because design features do not include current standards.
[3] Costs for the 800 TAF size have not been estimated.
[4] All cost estimates are preliminary. Construction cost represents the sum of field costs and indirect costs for planning, engineering, design and construction management, estimated at 25 percent of field costs.
[5] Costs do not include environmental mitigation or potential recreation facilities.

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CHAPTER 8. LIST OF PREPARERS

NAME	ROLE
Bureau of Reclamation	
Brian Zewe	Project Manager
Chuck Howard	Regional Geologist, retired
Joel Sturm	Regional Geologist
Greg Mongano	Geologist
Eric Vance	Geologist
Robert Williams	Geologist
L. Hobbs	Geologist
Alan Stroppini	Lead Engineer
Clarence Duster	Geotechnical Engineer, retired
Mark Pabst	Geotechnical Engineer
Steve Higinbotham	Civil Engineer, retired
Bill Thompson	Civil Engineer
Mike O'Shea	Civil Engineer
Dave Hinchliff	Civil Engineer
P.M. Ruchti	Civil Engineer
Mark Leavitt	Civil Engineer
Dave Hulse	Mechanical Engineer
Wayne Delzer	Mechanical Engineer
Rick Frisz	Mechanical Engineer
John Grass	Mechanical Engineer
Alex Ritt	Mechanical Engineer
Richard Noi	Electrical Engineer
Lisa Gamuciello	Electrical Engineer
Kenneth Bullard	Hydraulic Engineer
Robert Swain	Hydraulic Engineer
Robert Baumgarten	Estimating Team Leader
Dan Donaldson	Estimator
DWR	
Waiman Yip	Project Manager
MWH	
William Swanson	Project Manager
Stephen Osgood	Planner
Ryan Murdock	Water Resources Engineer
Foster Pelton	Hydropower Engineer
Jamil Ibrahim	Environmental Planner
Rebecca Fedak	Planner
James Darke	GIS Analyst
Steve Irving	GIS Technician
Emily McAlister	Technical Editor
Leonora Antonio	Document Coordinator

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