North-of-the-Delta Offstream Storage Investigation

Feasibility Report (Working Copy)

Appendices A through H

Prepared For Reclamation By URS Corporation

U.S. Department of the Interior
Bureau of Reclamation
Mid-Pacific Region
Sacramento, California

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APPENDIX B

Engineering

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B.1 INTRODUCTION

This Engineering, Design, and Cost Estimate appendix presents information related to the preliminary designs and cost estimates for the measures included in the alternative plans developed for North-of-the-Delta Offstream Storage (NODOS). This information was derived from previous studies by the United States Department of the Interior, Bureau of Reclamation (Reclamation), and the Department of Water Resources (DWR).

B.1.1 Chronology

The NODOS concept has been studied by DWR since 1957, and by Reclamation since 1964. Traditionally, reservoirs are created by constructing dams on major streams (on stream storage). An offstream storage reservoir is typically constructed on a small, generally seasonal stream that contributes a minor share of the water supply to the reservoir. Offstream storage involves diverting water from a major stream and transporting the water through a conveyance system to the reservoir.

The earliest published reference to a NODOS project is found in DWR Bulletin 3, the California Water Plan 1957, which mentions a 48,000 acre-foot (AF) Sites offstream storage reservoir on Stone Corral and Funks Creeks to be supplied by a then-proposed Tehama-Colusa (TC) Canal.

DWR’s Bulletin 109, Colusa Basin Investigation 1964, evaluated potential flood control projects and considered two separate reservoirs of 5,800 and 7,600 AF on Stone Corral and Funks Creeks, respectively. An update of this report in 1990 found these reservoirs economically unjustified for flood control alone.

Consideration of larger projects at the Sites location were first documented in December 1964 in Reclamation’s West Sacramento Canal Unit Report, which studied the feasibility of extending the TC Canal, via a new West Sacramento Valley Canal, into Solano County near Fairfield. To develop additional water supply to support this canal extension plan, a 1.2 million acre-feet (MAF) Sites Reservoir was proposed. This study did not evaluate the potential of Sites as a stand-alone project, but only as part of the extended canal system. Reclamation unsuccessfully attempted to obtain funds for a full feasibility study of Sites Reservoir in 1977 and documented its findings in a report published in 1981.

In March 1990, CH2M HILL prepared a long-range plan for the Glenn-Colusa Irrigation District (GCID) that included an 870,000 AF Sites Reservoir with a normal water surface elevation (WSE) of 460 feet. Based on information contained in Reclamation’s 1964 report, GCID determined that an 870,000 AF Sites Reservoir was beyond its financial capability.

In 1993, CH2M HILL published a report, *Meeting California’s Water Needs in the 21st Century*, which presented a conceptual Westside Storage and Conveyance System. The report mentioned a Sites/Colusa Reservoir with a feeder pipeline from Lake Oroville.

A July 1995 draft report by the Colusa Basin Drainage District on its proposed Water Management Program recommended a 62-foot-high dam on Funks Creek that would impound 9,500 AF in Golden Gate Reservoir. The project benefits listed were flood control and modest springtime irrigation yield.

In late 1995, DWR received numerous requests from water interests, including the Northern California Water Association (NCWA), for information regarding the potential of an offstream storage reservoir at the Sites location. In response to this renewed interest, DWR reviewed historic documents on the project to assess its potential to augment local and statewide water supplies during drought periods. DWR conducted a brief investigation of environmental literature, studies, aerial photos, and conducted limited field work in the project area. DWR published its findings in a July 1996 report entitled *Reconnaissance Survey – Sites Offstream Storage Project*.

DWR's 1996 report briefly summarized Sites project planning information and updated earlier cost estimates to 1995 levels. No insurmountable problems were identified that would prevent further evaluation of the project. Rather, DWR found that the project had several unique characteristics that made it an attractive candidate for further feasibility-level investigations. The project has a significantly lower cost per unit of storage than most other storage sites and the area is sparsely populated and contains relatively few environmentally sensitive species. The geography of the potential reservoir location permits a range of storage options for consideration.

In July 2000, a *NODOS Progress Report* was published to summarize the work conducted since 1997. The document provided California Federal Bay-Delta Program (CALFED) agencies and the public with information about projects under evaluation. Engineering and geologic investigations conducted at the probable locations of the Golden Gate and Sites Dams indicated they were suitable for construction of dams that could impound a 1.8-MAF Sites Reservoir.

In August 2000, DWR's Division of Engineering (DOE) prepared an *Engineering Progress Report on Feasibility Studies for Sites Reservoir*. The report documented progress on feasibility-level studies for Sites Reservoir facilities. It summarized the conceptual design for Golden Gate and Sites Dams, spillway, inlet/outlet works, and the pumping/generating plant directly serving the Sites Reservoir. The report deemed detailed geologic mapping and investigation of the faults, for further studies, essential, because of potential seismic activity at the dam sites.

In June 2002, DOE released another report, *Materials Investigation, Testing, and Evaluation Program for Sites Reservoir*. The investigation objectives included identification of the types of available on-site construction materials, examination of their potential uses, and performance of limited testing and evaluation to determine their suitability for use in the dams and appurtenant structures.

In October 2002, a *NODOS Scoping Report* was published by DOE, summarizing the public concerns, evaluating the magnitude of concerns, and helping decision makers determine the range of alternatives for NODOS.

In April 2003, URS Corporation (URS) and CH2M HILL prepared an *Engineering Feasibility Report on Modifications to GCID Fish Screen and Enlargement of Main Canal*. The study evaluated facility improvements, operating concepts, capital costs, and overall feasibility of expanding GCID's Main Pump Station and Main Canal in order to convey Sacramento River diversions to the proposed Sites Reservoir.

In May 2003, URS and CH2M HILL prepared a *Final Engineering Feasibility Report for TC Canal Alternatives* for NODOS. The study evaluated alternatives for the TC Canal intake facilities including enlargement of the canal itself to meet the needs of the Sites Reservoir project.

In May 2003, CH2M HILL prepared, Stony Creek Alternative for Conveyance to Sites Reservoir NODOS Evaluations. The purpose was to conduct feasibility-level evaluations of alternative routes and associated conveyance facilities from the Black Butte Dam Afterbay to the TC Canal near Orland.

In June 2003, DOE released another report, *Sites Reservoir Engineering Feasibility Study, Pumping Plants, and Appurtenant Facilities*. The purpose of the study was to evaluate the engineering feasibility for the design and construction of a pumping plant, inlet/outlet channel, tunnel, reservoir inlet/outlet structure, emergency release structure, and an ungated spillway. The study found each of these appurtenances to be technically feasible.

In July 2003, DOE released *Geologic Feasibility Report, Sites Reservoir Project*. The report summarized all of the geologic information that had been developed to that time. The report found that the proposed project was geologically feasible and that there were no geological fatal flaws.

In August 2003, DOE released *Conveyance, Colusa Basin Drain, and Funks Reservoir Modification*. The report focused on three tasks for the Sites project: conveyance from the Sacramento River to Funks Reservoir; utilizing the Colusa Basin Drain as a source of supply; and modification of Funks Reservoir to meet the requirements of the Sites Reservoir project.

In September 2005, DOE released a *Feasibility Study for Reverse Flow and Pumping-Generating Plants*. The report considered the feasibility and benefits of a reverse-flow conveyance system from Funks Reservoir to the Sacramento River.

In May 2006, URS prepared a *NODOS Investigation Initial Alternatives Information Report* (IAIR) for Reclamation. The purpose of the investigation was to identify and screen alternatives for NODOS.

In June 2008, CH2M HILL prepared a feasibility study for DWR for the fish screen facility at the proposed diversion location for NODOS.

In July 2008, DOE prepared an addendum amending the August 2003 *Feasibility Study Report*. The addendum updated the costs of the pumping-generating plants for pumping capacities of 1,500 and 2,000 cubic feet per second (cfs), and a complete site plan of the pumping-generating plant, in concert with CH2M HILL's updated *Fish Screen Facility Study*.

In September 2008, URS prepared the *NODOS Plan Formulation Report* (PFR) for Reclamation, describing the formulation, evaluation, and comparison of initial alternatives plans that address NODOS Investigation planning objectives.

B.1.2 Proposed Alternatives

On January 4, 2011, the DWR and Reclamation Project Team (Team) met to discuss and review 19 storage and conveyance options being evaluated for NODOS. Based upon the meeting, the Team selected three of the options to be the alternatives carried forward for evaluation in the NODOS ADEIR/EIS. Table B.1-1 outlines key aspects of these alternatives.

Table B.1-1. Proposed NODOS Alternative Projects

	Alternative		
	A	**B**	**C**
Screening Study Identifier	R12C3	R18C3 (No Pump)	R18C3
Storage Capacity			
Sites Reservoir	1.2 MAF	1.8 MAF	1.8 MAF
Conveyance Capacities To Sites Reservoir			
Tehama-Colusa Canal	2,100 cfs	2,100 cfs	2,100 cfs
Glenn Colusa Canal	1,800 cfs	1,800 cfs	1,800 cfs
Delevan Pipeline	2,000 cfs	0 cfs	2,000 cfs
Conveyance Capacities from Sites Reservoir			
Tehama-Colusa Canal	800 cfs	800 cfs	800 cfs
Glenn Colusa Canal			
Delevan Pipeline	1,500 cfs	1,500 cfs	1,500 cfs

cfs = cubic feet per minute
MAF = million-acre feet

B.1.3 Project Description

Figure B.1-1 is a schematic representation of the principal components of the proposed NODOS project showing direction of flow, and storage volumes. Most of the facilities shown are common to all three project alternatives currently under consideration. The proposed project would include water conveyance from the Sacramento River through existing, expanded, or new facilities and water storage in the Sites Reservoir. Water would be diverted to Sites Reservoir from the Sacramento River, primarily in winter months. The stored water would be released from Sites Reservoir in summer months to deliver water to local water users through existing facilities such as the GCID and TC Canal, or to return water to the Sacramento River using a new conveyance system. Water would be provided from Sites Reservoir in exchange for water that otherwise would have been released from Shasta Lake. The exchanged water would then remain in Shasta Lake for beneficial use later in the summer. Shasta Lake water could help cool the upper river for fishery maintenance purposes, and be used downstream for agricultural, environmental, and urban purposes.

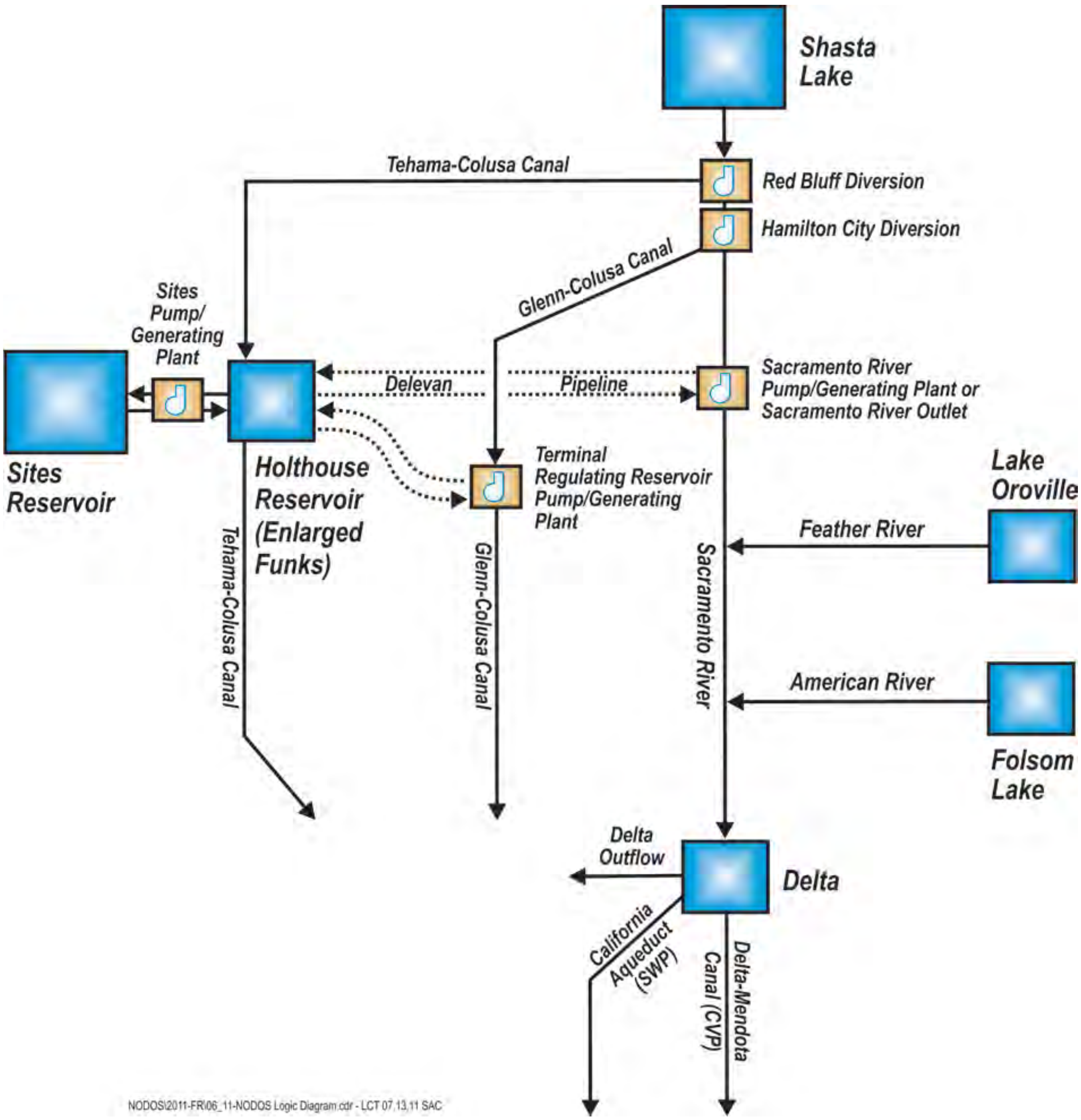


Figure B.1-1. NODOS Project – Schematic Layout

Sites Reservoir would be located approximately 10 miles west of Maxwell, California. The reservoir would be formed by constructing Sites Dam approximately 290 feet high on Stone Corral Creek and Golden Gate Dam approximately 310 feet high on Funks Creek. Sites dam would range in height from 290 feet for the 1.81-MAF reservoir to 250 feet for the 1.27-MAF reservoir. Golden Gate Dam would range in height from 310 feet for the 1.81-MAF reservoir to 270 feet for the 1.27-MAF reservoir. Nine saddle dams ranging up to 130 feet high would also be built along the reservoir's northern boundary for the 1.81-MAF reservoir. Fewer saddle dams would be required for the 1.27-MAF reservoir.

The proposed Sites Reservoir water control features (appurtenances) include water intake and outlet structures, the Sites Pumping-Generating Plant (SPGP) located downstream of the Golden Gate Dam site on Funks Creek, and an emergency signal spillway located at Sites Saddle Dam 6. Sites Dam and Golden Gate Dam would have low-level outlet works capable of releasing stream maintenance flows into Stone Corral Creek and Funks Creek after construction is completed. These low-level outlet works will be incorporated as permanent facilities in the creek diversion systems installed at both dam sites to pass winter storm runoff through the construction sites.

Presently, the existing 40-foot-high Funks Dam forms a 2,250-AF reservoir 1 mile downstream of the Golden Gate Dam site. This reservoir was constructed by Reclamation and is part of the TC Canal system. Funks Reservoir serves as a re-regulating reservoir to stabilize flows in the canal below Funks Reservoir as diverters come online and off line. The existing Funks Reservoir will be expanded to form the Holthouse (Enlarged Funks) Reservoir by constructing a new dam and reservoir to the east and breaching the existing Funks Dam so that the new and existing reservoirs act as one unit with an enlarged active storage capacity of approximately 6,500 AF. In addition to facilitating water inflow and outflow water management, that capacity provided pump-back storage capability that would enhance project power generation, particularly during daily on-peak periods. For all of the water source options considered, imported water entering Sites Reservoir would pass through Holthouse Reservoir, which would serve as a forebay/afterbay to the SPGP.

Development of Sites Reservoir with a diversion capability from the Sacramento River would require at least:

- Minimal modification of the TC Canal and GCID Canal intakes.
- Construction of a terminal regulating reservoir and pumping-generating plant on the GCID Canal.
- Inter-connection from the GCID Canal to the TC Canal.
- Construction of the proposed Sacramento River Pumping/Generating Plant for Project Alternatives A and C, which would divert up to 2,000 cfs from the Sacramento River, through a new pipeline. Project Alternative B would not include Sacramento River pumping as a project feature, but seasonal releases would still be made back to the Sacramento River through the new Delevan Pipeline.

Winter flows diverted into these canals and pipeline would be conveyed into Holthouse Reservoir and then pumped into Sites Reservoir. These modified or new facilities would allow winter diversions of water from the Sacramento River when downstream criteria are met. Total diversion capacity from the Sacramento River for the currently proposed source and conveyance alternatives would not exceed 5,900 cfs for Alternatives A and C, or 3,900 cfs for Alternative B with no Sacramento River pumping.

Table B.1-2 summarizes the major features of NODOS alternatives as included in the PFR.

When water is released from Sites Reservoir, it would be routed through reversible pump-turbine-generators to generate clean hydroelectric power. These releases could help offset energy costs associated with pumping and provide firm power to support wind and solar generation. Holthouse Reservoir is sized to provide approximately 6 hours per day of on demand generation during peak power periods.

As currently envisioned, water would be pumped on a seasonal basis into Sites Reservoir during periods of relatively lower energy cost and released through the hydroelectric generation facilities during times of higher energy value. Volume regulation would occur in Holthouse Reservoir on a daily basis. Pumping and release would each take place approximately five months a year. During the approximately two months per year that water is not stored or released to accomplish major project goals, daily pump-back operations potentially would be performed to enhance the peak power generating capability to help offset the power usage and provide some ancillary power benefits to enhance the reliability of California’s electric grid. This would also increase the economic return on the project without losing control of the water impounded by Sites Reservoir.

B.1.4 Organization

Section B.2 of this appendix presents the NODOS project settings, and a summary of the geotechnical investigation conducted for the project. Section B.3 discusses project design considerations to date. Section B.4 presents the summary of cost estimates for the three NODOS project alternatives discussed in the PFR. Section B.5 discusses the construction schedule and Section B.6, the construction equipment utilization and workforce. Finally, Section B.7 lists the reference materials considered in this appendix.

Table B.1-2. Summary of the Major Features of NODOS Plans in the PFR

Facility	Feature	Alternative A	Alternative B	Alternative C
Sites Reservoir	Gross Storage Capacity Water Surface Elevation Dam Crest Elevation Minimum Operating Pool Inundation Area Inlet/Outlet Type	1.27 MAF 480 feet msl 500 feet msl 320 feet msl 12,500 acres Multi-level inlet/outlet tower A low-level inlet/outlet structure	1.8 MAF 520 feet msl 540 feet msl 320 feet msl 14,000 acres Multi-level inlet/outlet tower A low-level inlet/outlet structure	1.8 MAF 520 feet msl 540 feet msl 320 feet msl 14,000 acres Multi-level inlet/outlet tower A low-level inlet/outlet structure
Golden Gate Dam (Sites Reservoir)	Location Type Crest Length Maximum Height Embankment Volume	Funks Creek Earth/Rockfill Embankment 1,450 feet 260 feet 5,987,000 cubic yards	Funks Creek Earth/Rockfill Embankment 2,120 feet 310 feet 10,590,000 cubic yards	Funks Creek Earth/Rockfill Embankment 2,120 feet 310 feet 10,590,000 cubic yards
Sites Dam (Sites Reservoir)	Location Type Crest Length Maximum Height Embankment Volume	Stone Corral Creek Earth/Rockfill Embankment 725 feet 250 feet 2,853,000 cubic yards	Stone Corral Creek Earth/Rockfill Embankment 850 feet 290 feet 3,877,000 cubic yards	Stone Corral Creek Earth/Rockfill Embankment 850 feet 290 feet 3,836,000 cubic yards
SSD for Sites Reservoir	Location Type SSD Numbers	North End of reservoir from Funks Creek to Hunters Creek Earth/Rockfill Embankments 1, 6, 8b – <5 feet to 25 feet high 3, 5, 8a -50 feet to 85 feet high	North End of reservoir from Funks Creek to Hunters Creek Earth/Rockfill Embankments 1, 4, 9 -40 to 50 feet high 2, 3, 5, 6, 7, 8 - 70 to 130 feet high	North End of reservoir from Funks Creek to Hunters Creek Earth/Rockfill Embankments 1, 4, 9 -40 to 50 feet high 2, 3, 5, 6, 7, 8 - 70 to 130 feet high
Emergency Spillway (Sites Reservoir)	Location Diameter Inlet Elevation	Saddle Dam 6 7-foot RCP 486.5 feet (top of PMF storage)	Saddle Dam 6 7-foot RCP 525.5 feet (top of PMF storage)	Saddle Dam 6 7-foot RCP 525.5 feet (top of PMF storage)
Sites Reservoir Inlet/Outlet Works	Type Capacity Size	Multi-level Inlet Tower and Low-Level Outlet 23,000 cfs (emergency release) 30-foot-diameter concrete and steel-lined pressure tunnel	Multi-level Inlet Tower and Low-Level Outlet 23,000 cfs (emergency release) 30-foot-diameter concrete and steel-lined pressure tunnel	Multi-level Inlet Tower and Low-Level Outlet 23,000 cfs (emergency release) 30-foot-diameter concrete and steel-lined pressure tunnel

Table B.1-2. (Continued)

Facility	Feature	Alternative A	Alternative B	Alternative C
Sites Pumping-Generating Plant	Location Flow Capacity Pumping Head Generating Capacity	Downstream from Golden Gate Dam 5,900 cfs pumping 5,100 cfs generating 295 feet 123 MW at 5,100 cfs	Downstream from Golden Gate Dam 3,900 cfs pumping 5,100 cfs generating 335 feet 123 MW at 5,100 cfs	Downstream from Golden Gate Dam 5,900 cfs pumping 5,100 cfs generating 335 feet 123 MW at 5,100 cfs
Funks Dam (Existing)	Maximum Height Max WSE Surface Area Total Capacity Remaining Storage	40 feet 205 feet msl 232 acres 2,250 AF 1,170 AF	40 feet 205 feet msl 232 acres 2,250 AF 1,170 AF	40 feet 205 feet msl 232 acres 2,250 AF 1,170 AF
Holthouse Reservoir (Enlarged Funks)	Maximum Height Max WSE Surface Area Total Capacity Remaining Storage	45 feet 205 feet msl 6,500 AF 6,500 AF	45 feet 205 feet msl 6,500 AF 6,500 AF	45 feet 205 feet msl 6,500 AF 6,500 AF
New Delevan Pipeline from Sacramento River to TC Canal	Flow Capacities Length Size From/To	2,000 cfs pumping 1,500 cfs releasing 13 miles Two 12-foot-diameter RCP Sacramento River to Holthouse Reservoir	No Pumping 1,500 cfs releasing 13 miles Two 12-foot-diameter RCP Sacramento River to Holthouse Reservoir	2,000 cfs pumping 1,500 cfs releasing 13 miles Two 12-foot-diameter RCP Sacramento River to Holthouse Reservoir
Sacramento River Pumping-Generating Plant	Location Flow Capacities Fish Screens Required	West side of Sacramento River, near Highway 45 2,000 cfs pumping 1,500 cfs releasing Yes	West side of Sacramento River, near Highway 45 No Pumping 1,500 cfs releasing No	West side of Sacramento River, near Highway 45 2,000 cfs pumping 1,500 cfs releasing Yes
TC Canal (Existing)	Flow Capacity Improvements Required	2,100 cfs Improvements to existing headworks, fish screens, and canal	2,100 cfs Improvements to existing headworks, fish screens, and canal	2,100 cfs Improvements to existing headworks, fish screens, and canal
GCID Canal (Existing)	Flow Capacity Improvements Required	1,800 cfs Raise headgate structure and fish screens	1,800 cfs Raise headgate structure and fish screens	1,800 cfs Raise headgate structure and fish screens

Table B.1-2. (Continued)

Facility	Feature	Alternative A	Alternative B	Alternative C
TRR	Capacity Footprint Depth Maximum Embankment Height	2,000 AF 200 acres 17 feet 21 feet	2,000 AF 200 acres 17 feet 21 feet	2,000 AF 200 acres 17 feet 21 feet
TRR Pumping-Generating Plant	Location Capacity Pumping Head (Net) Generation	TRR Reservoir 1,890 cfs pumping 1,500 cfs generating 98 to 114 feet 9.8 MW	TRR Reservoir 1,890 cfs pumping 1,500 cfs generating 98 to 114 feet 9.8 MW	TRR Reservoir 1,890 cfs pumping 1,500 cfs generating 98 to 114 feet 9.8 MW
TRR Pipeline	Location Flow Capacities Length Size From/To	TRR Reservoir 1,890 cfs pumping 1,500 cfs releasing 5 miles Two 12-foot-diameter RCP TRR Reservoir to Holthouse Reservoir	TRR Reservoir 1,890 cfs pumping 1,500 cfs releasing 5 miles Two 12-foot-diameter RCP TRR Reservoir to Holthouse Reservoir	TRR Reservoir 1,890 cfs pumping 1,500 cfs releasing 5 miles Two 12-foot-diameter RCP TRR Reservoir to Holthouse Reservoir

AF = acre-foot
cfs = cubic feet per second
GCID = Glenn-Colusa Irrigation District
MAF = million acre-feet
msl = mean sea level
MW = megawatt
NODOS = North-of-the-Delta Offstream Storage
PFR = Plan Formulation Report
PMF = permissible maximum flood
RCP = reinforced concrete pipe
SSD = Saddle Dams
TC = Tehama-Colusa
TRR = Terminal Regulating Reservoir
WSE = water surface elevation
< = less than

B.2 SETTING

B.2.1 Location

The NODOS project is located in the Coast Range foothills and lowlands along the western edge of the northern Sacramento Valley (Figure B.2-1). The United States Geological Survey (USGS) watersheds and subbasins containing the proposed off-stream reservoir include Lower Grapevine, Funks, Howard, Upper Grapevine, Antelope, Upper and Lower Hunters Creeks, and McDowell Canyon.

The key feature of the NODOS project, the proposed Sites Reservoir, is located in north-central Colusa County and south-central Glenn County, approximately 10 miles west of the community of Maxwell. Two reservoirs are under consideration at the same site, one with a maximum capacity of 1.81 MAF inundating approximately 14,000 acres, the other with a maximum capacity of 1.27 MAF inundating an area of approximately 12,500 acres. The proposed reservoir inundation areas include most of Antelope Valley and the small community of Sites. The reservoir is in the Funks Creek and Stone Corral Creek watersheds and includes the eight associated USGS subbasins.

B.2.2 Topography

The physical topography of the watersheds draining the east side of the Coast Range toward the Sacramento Valley is diverse. The topography ranges from steep, rugged, mountainous terrain within the upper watersheds to rolling foothills in the project area, to relatively flat alluvial terrain as the watersheds enter the Sacramento Valley. Elevations range from less than 40 feet on the valley floor to more than 8,000 feet along the Coast Range divide.

The proposed Sites Reservoir would be situated between the Sacramento Valley to the east and the mountainous portion of the Coast Range on the west. The Coast Range Mountains are a series of rugged, north-south trending ridges dissected by narrow canyons containing steep gradients, and entrenched streams. A relatively narrow band of steep rolling foothills, approximately 2 to 3 miles wide, separates the proposed reservoir area from the Sacramento Valley. Antelope Valley, the primary inundation area of the proposed Sites Reservoir, lies between this narrow band of foothills and the more mountainous Coast Range. This relatively narrow north-south trending valley is approximately 13 miles long and up to 2 miles wide. Elevation of the Antelope Valley floor ranges from 320 to 400 feet above mean sea level (msl), while the foothills separating the valley from the Sacramento Valley reach a maximum elevation of 1,300 feet. Elevations along the west side of Antelope Valley increase rapidly with several peaks within 2 miles of the valley margin above 2,000 feet.

B.2.3 Climate

The climate of the watersheds draining into the western Sacramento Valley is typical Mediterranean. Winters are rainy and relatively mild with only occasional freezing temperatures at the lower elevations; summers are comparatively dry and hot.

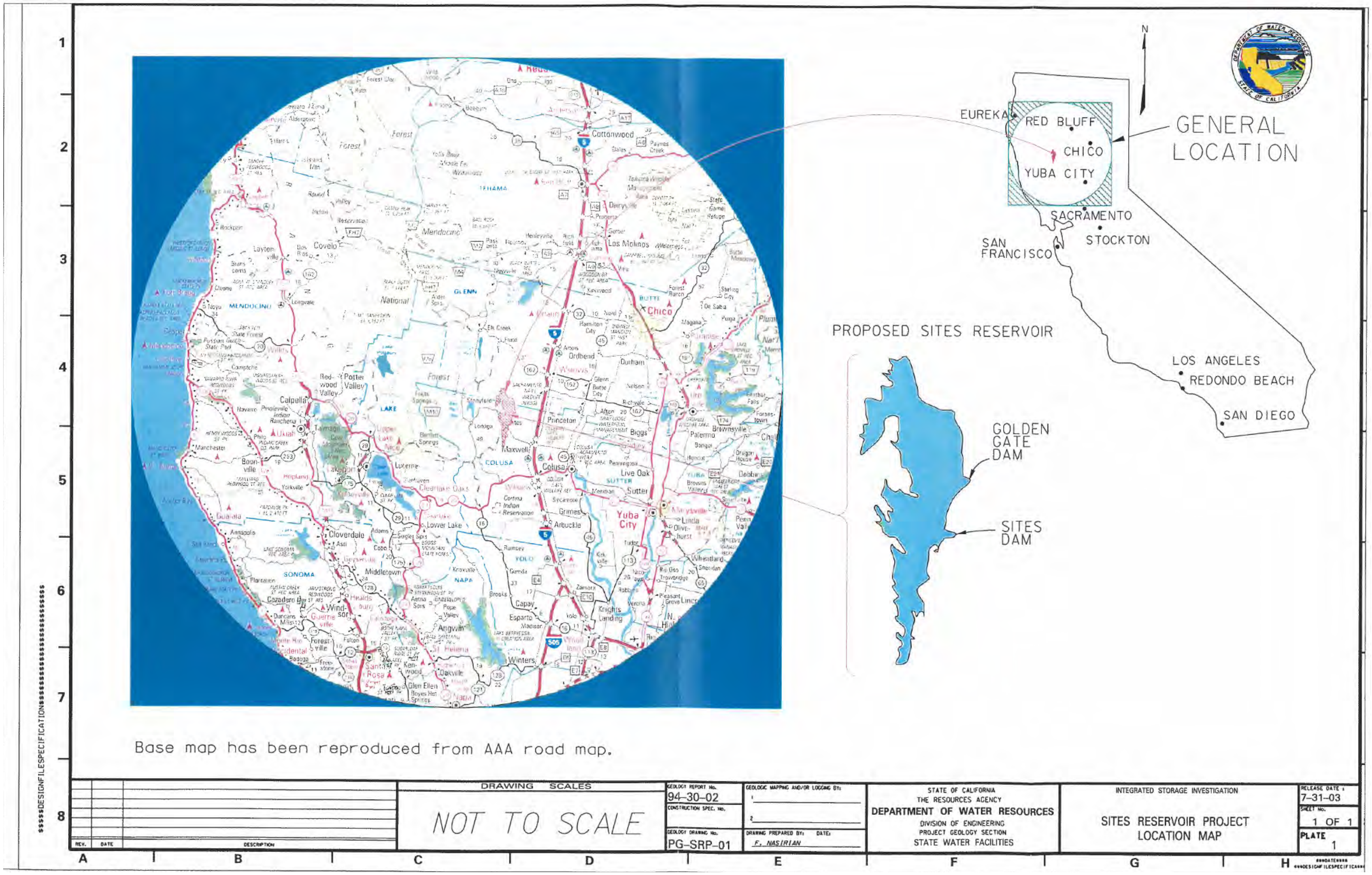


Figure B.2-1. Sites Reservoir Project – Location Map

The rainy season normally begins in September and continues through March or April. Rains may continue for several days at a time, but are usually gentle. Summer rains are rare, as are thunderstorms and hailstorms. Thunderstorms occur approximately 10 days per year in the Sacramento Valley, occasionally producing high-intensity rainfall of short duration. Most precipitation is associated with migrant storms that move across the area during winter. Snow is the dominant form of precipitation above 5,000-foot elevations and persists on north- and east-facing slopes into the early summer.

High temperatures occur during July, August, and September, with temperature readings commonly in excess of 100 degrees Fahrenheit (ºF). Fog of varying density and duration is common within the Sacramento Valley during winter. However, due to the physical topography, dense or persistent fog is much less common in the project areas. Winds occur seasonally, with dry, north winds common during the summer and fall, while winds from the south are frequently associated with winter storm events. Winds in excess of 60 miles per hour (mph) may occur; however, these events are relatively uncommon and of short duration. Average wind speed at Red Bluff is 8.8 mph, with the strongest winds reported during the winter months. Gross evaporation, the depth of water lost to the atmosphere, averages approximately 70 inches per year in the foothill region.

B.2.4 Vegetation

Nearly 89 percent of the proposed Sites Reservoir watershed area is annual grassland community with the remaining areas dominated by Blue Oak woodland and small amounts of chaparral, riparian wetlands, cultivated grain, and non-vegetated areas. The Blue Oak woodland gives way to Foothill pines above the crest elevations of the two principal dams and large chaparral stands emerge on the southern exposures and shallow soils above the dam crest elevations.

B.2.5 Watershed Hydrology

The watershed contributing inflow directly to Sites Reservoir drains approximately 83 square miles. Average annual precipitation within the basin is approximately 18 inches and occurs almost exclusively in the form of rain. Snow occurs annually at the higher elevations and occasionally within the reservoir areas. Some areas within western Glenn County that range in elevation from 5,000 to 7,000 feet frequently receive between 60 and 75 inches of precipitation per year, primarily in the form of snow.

Streams draining into the proposed reservoir are ephemeral with little or no flow from July to October. However, these streams tend to respond rapidly to significant rainfall events. Flash flooding with substantial overland flow has been observed. Flow recorded at the stream gauge on Stone Corral Creek near Sites is representative of the flow variability of these small ephemeral streams. Annual discharge varied from 0 AF in 1972, 1976, and 1977 to 39,930 AF in 1963, and averaged 6,500 AF. Monthly volumes in excess of 15,000 AF have been documented.

FUNKS CREEK

Funks Creek has 43 square miles of drainage area. No stream gauge has ever been placed on Funks Creek, so no accurate estimate of 100-year discharge is available. However, as the topography and soil composition of the watershed are similar to those of Stone Corral Creek, where stream flow records are available, and given the comparable drainage areas of the two watersheds, it may be reasonably assumed that the 100-year discharge into Funks Creek would be similar to that of Stone Corral Creek.

STONE CORRAL CREEK

The drainage area of the Stone Corral Creek watershed is 38.2 square miles. The 100-year discharge was established in a 1987 Department of Water Resources (DWR) Colusa Basin flood flow frequency analysis at 7,870 cfs (March 10, 1987, Colusa Basin Flood Flow Frequency Analysis). This value was based on 25 years of discharge measurements collected near the town of Sites, with interruption, from 1958 through 1985.

B.2.6 Flood Hydrology

Flood Inflows and Volumes

At the request of DWR's Division of Planning and Local Assistance (DPLA), the DWR DOE performed a probable maximum flood (PMF) analysis for the Sites Reservoir watershed. The goal of the study was to determine the performance of the reservoir in response to inflows from a probable maximum precipitation (PMP) event. The study focused on the 1.81 MAF reservoir size and the assumptions of the study included a completely full reservoir with a WSE of 520 feet at the start of the storm, a dam crest elevation of 540 feet, and a reservoir inundation area of 14,000 acres. The results of this analysis are also applicable to the 1.27 MAF reservoir, which is in the same location as the larger reservoir and has very similar drainage areas.

The PMF analysis for the 1.81 MAF reservoir determined that the PMP the watershed would experience during a three-day event was 20.63 inches. The total volume of inflow into the reservoir would be 78,422 AF with a peak inflow of 68,500 cfs during the storm. Storing the full runoff in the reservoir above the normal maximum water level would raise the WSE by approximately 5.2 feet. After the PMP event, the freeboard left in the 1.81 MAF reservoir would be approximately 14.8 feet (Sites Reservoir PMF analysis from August 2004.)

As mentioned above, the storm inflow volume for the 1.27 MAF reservoir would be approximately the same for the 1.27 MAF reservoir. The 1.27 MAF reservoir would have a maximum normal operating elevation of 480 feet (based upon the available area capacity curve for the reservoir site) and a dam crest elevation of 500 feet. Storing the full runoff in the reservoir above the normal maximum water level would raise the WSE by approximately 6.25 feet. After the PMP event, the freeboard left in the 1.27 MAF reservoir would be approximately 13.75 feet.

Emergency Signal Spillway Design Flood

Preliminary design of the Sites Reservoir emergency spillway was performed in accordance with the state of practice for dam appurtenant structures. The emergency signal spillway design conforms to existing Division of Safety of Dams (DSOD) criteria. The elevation, type, and capacity of the emergency spillway, are designed for a highly unusual set of circumstances.

The proposed Sites Reservoir in either the 1.8 MAF or 1.27 MAF configuration would accommodate full storage of the design flood inflow with adequate residual freeboard to the dam crest. The emergency spillway would be required primarily for the improbable case where the pumping plant would continue pumping into the reservoir despite the reservoir being at maximum pool. The emergency spillway selected for the feasibility study would consist of one 7-foot-diameter reinforced concrete pipe, which would be located at Saddle Dam 6 for both reservoir alternatives. However, the spillway configuration would be different for the two reservoirs, as discussed further in Section B.3. The pipe size selection is based upon maintenance and inspection considerations.

B.2.7 Geology

General

This section summarizes the geology and geotechnical conditions of the features under consideration for the proposed NODOS project. The information in this chapter was summarized from the *July 2003 Project Geology Report No. 94-30-02, Geologic Feasibility Report, Sites Reservoir Project, Appendix to Engineering Feasibility Report*. Project Geology Report No. 94-30-02, in turn, provided a general summary of the detailed geologic reports prepared by the DOE and the DPLA, Northern District, Geology Section (Northern District). Discussions include the preliminary geologic investigations that were conducted mostly by the Northern District for the proposed project features. The aforementioned, detailed, geologic reports are referenced at the end of this technical memorandum.

Geologic conditions at the two main dam sites (Golden Gate and Sites), the saddle dam sites, and all other project facilities provide good quality rock for their respective foundations. Three important geology-related issues associated with the feasibility study were:

- Determining that geologic foundation conditions for the proposed project facilities are adequate

- Determining seismic sources for the project and fault activity at the Golden Gate and Sites Dam sites

- Evaluating sources for sand and gravel for construction materials

The data and analyses presented in this summary were used in developing preliminary design criteria for the NODOS project.

Geologic Setting

The NODOS project is located in the foothills of the northwestern edge of the Sacramento Valley, within the border area of the Great Valley and Coast Range geomorphic provinces (Figure B.2-2). The project area is underlain by Upper Cretaceous sedimentary rocks of the Cortina and Boxer Formations and alluvial deposits of the Sacramento Valley. Surficial geologic units in the project area include Pliocene to Pleistocene age deposits of the Tehama Formation, Quaternary older alluvial terrace deposits, and Holocene (Recent) alluvium, colluvium, and landslide deposits (Figure B.2-3). Structurally, the reservoir site is bounded by the Green Valley thrust and Stony Creek faults to the west, and the Corning and Willows faults to the east. Passing beneath the proposed reservoir are westerly dipping fault planes of the Funks and Bear Valley segments of the Great Valley thrust fault, while the Fruto Syncline, Sites Anticline, and Salt Lake thrust fault pass through the project site in a generally north-south trend.

The proposed Golden Gate Dam site is located in a stream-cut, watergap on Funks Creek within the Venado sandstone member of the Upper Cretaceous Cortina Formation. The proposed Sites Dam site is located in a stream-cut, watergap on Stone Corral Creek, within the Boxer and Cortina Formations. At these dam sites, the Cortina and Boxer Formations are part of a series of an east-dipping, Great Valley sequence of rocks exposed in the foothills bordering the eastern Coast Ranges (Figure B.2-3). Directly west of the dam sites, at the saddle dam sites and in the reservoir, these rocks are folded about the axes of the north-trending Sites anticline and Fruto syncline. East of the Golden Gate and Sites Dam sites, the rocks progressively flatten beneath the western Sacramento Valley margin. The two primary rock units in both the Cortina and Boxer Formations are sandstone and mudstone. The sandstones are commonly ridge-formers and the mudstones are generally expressed as topographic lows.

In addition to the folds, numerous faults offset and deform the bedrock strata in the study area. Two primary sets of surface faults were mapped in the vicinity of the dam sites:

- Northeast-striking, high-angle faults that obliquely cut the north-striking bedrock units, and consistently displace stratigraphic contacts in a right-lateral strike-slip sense. Specific examples of these structures include the informally named GG-1, GG-2, GG-3, and S-2 faults, all of which pass directly through or near the proposed Golden Gate and Sites Dam (Figure B.2-3).

- North-striking faults which are generally parallel to bedding. The most laterally continuous example of these structures is the steeply east-dipping Salt Lake thrust fault, which is parallel to, and directly east of, the axis of the Sites anticline in the proposed reservoir area. This fault trends through the middle of the reservoir and through the proposed Sites Saddle Dam 2 site.

Geotechnical Investigations

Geologic exploration for investigation at the various NODOS project features consisted of geologic mapping, drilling and water pressure testing, seismic refraction surveys, trenching and test pits, and laboratory testing. Individual exploration techniques are discussed below.

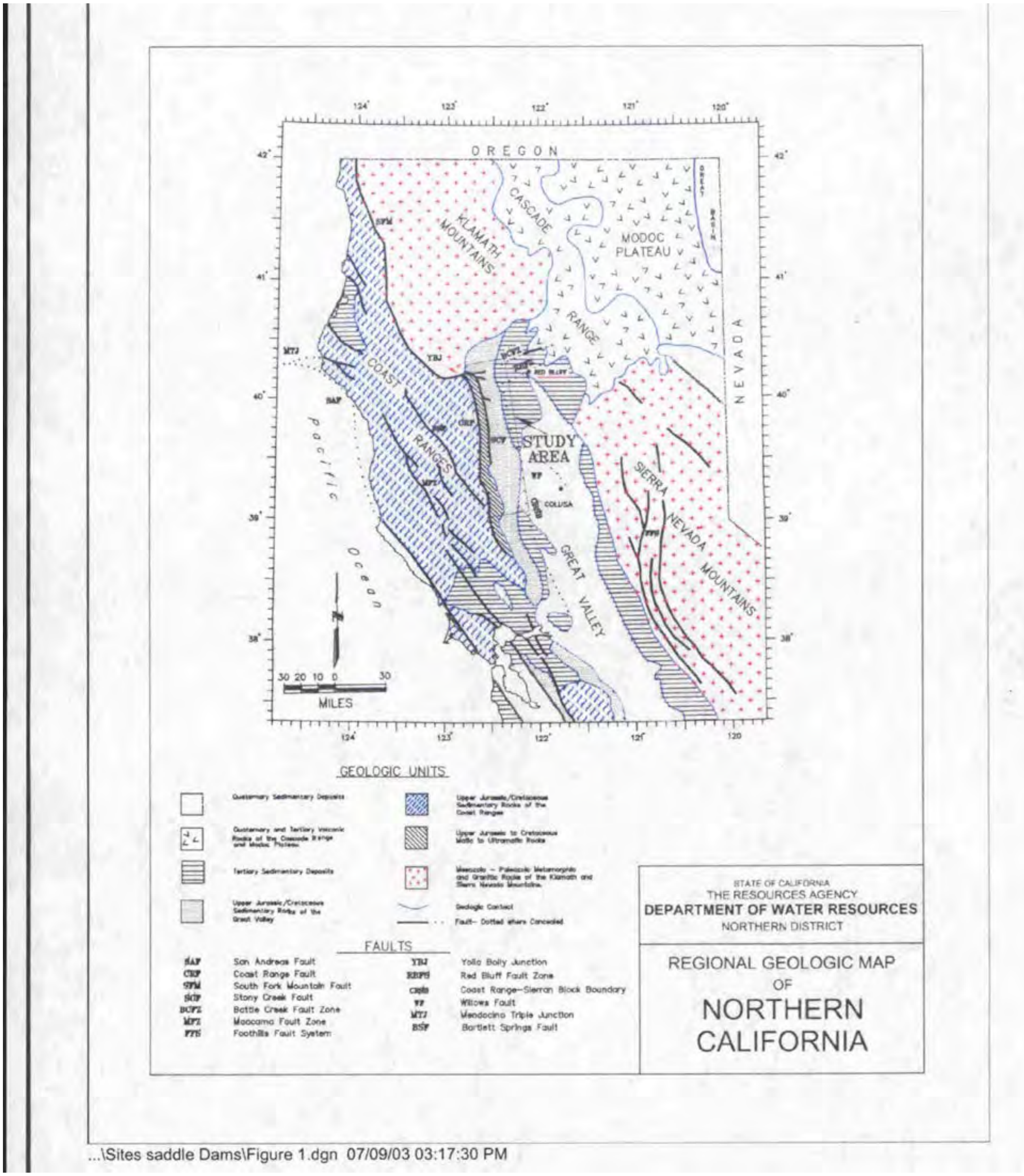
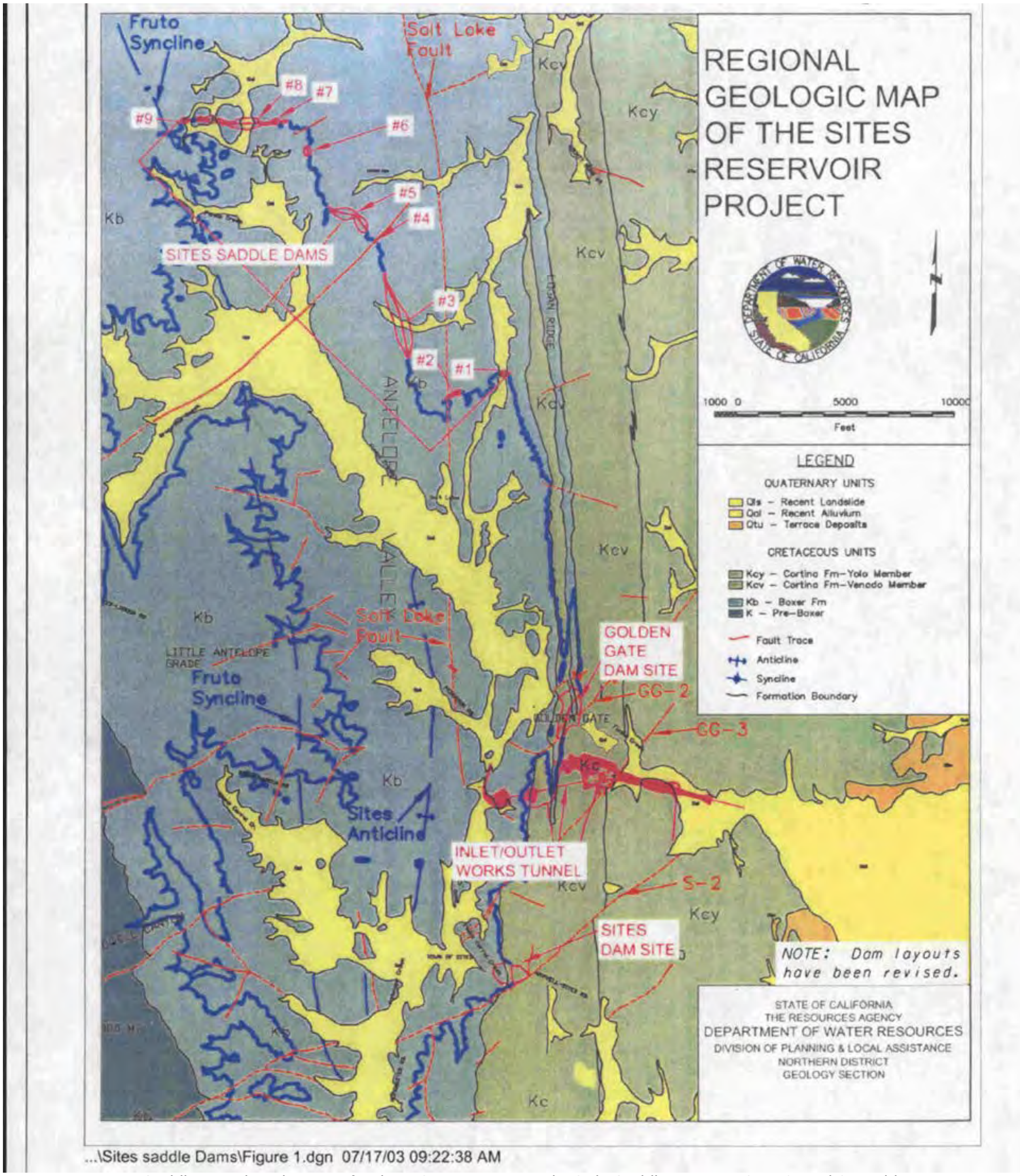


Figure B.2-2. Regional Geologic Map of Northern California



Note: Saddle Dam locations are for the 1.81 MAF Reservoir. Only Saddle Dams 1, 3, 5, 6, and 8 would be required for the 1.27 MAF Reservoir.

Figure B.2-3. Regional Geologic Map of the Sites Reservoir Project

- DWR's Division of Land and Right-of-Way (DLRW) produced new topography from a 1:7200 aerial flight in November 1998. Contours were developed at 2-foot intervals. Most of the geologic maps for the project utilized this topographic base map.

Numerous sources of previous geologic mapping were utilized in developing site maps for individual features of the NODOS project. The first published source of geologic mapping that covers the entire project area is the USGS Oil and Gas Investigations Map OM-210, entitled *Geologic Map of the Lodoga Quadrangle, Glenn and Colusa Counties, California*. The mapping was performed by R.D. Brown and E.I. Rich in 1961 at a scale of 1:48,000. This detailed regional geologic mapping covers the entire reservoir area and was the geologic base map for all of the studies performed by Reclamation and DWR since 1963.

In 1963 and 1969, Reclamation performed feasibility level geologic mapping at a scale of 1 inch = 200 feet. In addition, in 1979 and 1980, Reclamation performed additional geologic site mapping utilizing metric scales and topography. Geologic mapping and subsurface exploration for the NODOS project have been ongoing since 1997, by numerous DWR geologists. Geologic mapping for the reservoir site was primarily conducted by Northern District personnel between 1997 and 2001, with contributions by DOE engineering geologists.

Most of the subsurface exploration to date has been concentrated at the Sites, Golden Gate, and saddle dam sites. However, some reservoir and fault mapping, along with trenching associated with fault studies carried out by William Lettis & Associates Inc. (WLA), were conducted independent of individual dam site studies. Approximately 72 exploration borings were drilled in the project area by DWR and Reclamation, using a combination of rotary coring and hollow-stem auger methods. Standard penetration tests (SPT), water pressure/packer tests were performed in the boreholes as appropriate, and samples collected for laboratory testing. WLA excavated 31 trenches and test pits at 14 locations for their faulting and seismicity studies, while DWR and Reclamation excavated 10 test pits for construction material evaluations. Ten seismic refraction arrays were surveyed for the project, six at the Golden Gate Dam site and four for the Funks Reservoir enlargement and new Sacramento River conveyance (Delevan pipeline) alignment.

Laboratory testing was performed on rock and soil samples collected from test pits, drill holes, and the nearby Sites Quarry for potential construction materials use. The testing was performed by the United States Army Corps of Engineers (USACE), Reclamation, and DWR over different periods of exploration. Testing of rock included specific gravity; absorption; compressive, shear, and tensile strengths; and Los Angeles abrasion losses. Petrographic analyses of weathered and fresh Venado sandstone were performed by the USACE in 1962 and 1972. Soil samples were tested for classification, density, permeability, and shear strength; soil samples from auger holes along the proposed conveyance alignment were also tested for resistivity, sulfates, and chlorides. Results of laboratory testing can be found in *Material Investigation, Testing, and Evaluation Program* (DWR, 2002).

GEOLOGIC CONDITIONS

Sites Reservoir

The proposed Sites Reservoir would lie on alluvial and colluvial deposits and interbedded mudstone, sandstone, and conglomerate bedrock of the Cretaceous Boxer Formation (Figure B.2-3). However, a comprehensive geologic study of the entire reservoir has not as yet been completed. To date, most of the exploration has focused on the Sites, Golden Gate, and to a lesser extent, the saddle dam sites. Figure B.2-4 provides an explanation of symbols for individual dam site geologic maps.

Golden Gate Dam will be founded on sandstone (Kcvs) and mudstone (Kcvsm) of the Cortina Formation that generally strike north-south and dip 50º downstream to the east (Figures B.2-5A, B.2-5B, B.2-6, and B.2-7)

The sandstone is moderately to well indurated, and thin to thick bedded with mudstone interbeds up to 5 feet thick. Near the surface, the sandstone is intensely to moderately weathered, moderately hard, moderately strong, and closely to moderately fractured. At depth, the sandstone improves from slightly weathered to fresh, hard, strong, and moderately to slightly fractured. Fractures are generally associated with jointing, and are mostly healed with calcium carbonate. Overlying the bedrock, within the Funks Creek channel, is recent alluvium (Qal) consisting of silty and poorly graded sand and gravel deposits up to 17 feet thick. There are also up to 25 feet of older, sandy, and lean clay alluvial terrace (Qoal) deposits along the channel banks.

The mudstone unit at the site is moderately indurated, thinly laminated to thinly bedded with thin sandstone interbeds, and not exposed on the surface within the dam footprint. Nearby outcrops show that weathered mudstone is friable to low hardness, weak, closely fractured, brittle, and susceptible to slaking when exposed to air or moisture. At depth, fresh mudstone is moderately hard, moderately strong, and slakes only slightly after prolonged exposure to air. Bedding plane fractures predominate, while joint fractures are relatively short and discontinuous; mudstone bedding plane fractures commonly exhibit some plastic deformation in the form of slickensides or internal shearing.

Shears associated with bedding were noted in the sandstone and mudstone units, with zones up to 3.7 feet thick. The shears commonly contained up to 10 percent clay gouge with a variable percentage of calcite healing.

Two primary joint sets were identified from surface mapping at the site (Figures B.2-5A and B.2-5B). Set A, trending nearly normal to the ridges and bedding, has a strike ranging from N68°E to N86°E, and dips steeply from 67° north to 79° south. Set B is nearly parallel to bedding, and strikes north-south to N25°W, dipping 53° to 79° east. Analysis of the field mapping also showed a minor joint set, Set C ranging in strike from N48°-58°W and dipping 54° to 74° south.

Two near-vertical, northeast-trending faults are located at the Golden Gate Dam site, GG-1 and GG-2 (Figures B.2-5A and B.2-5B). Fault GG-1 is located approximately 150 to 400 feet upstream of the left abutment exhibiting right-lateral displacement of

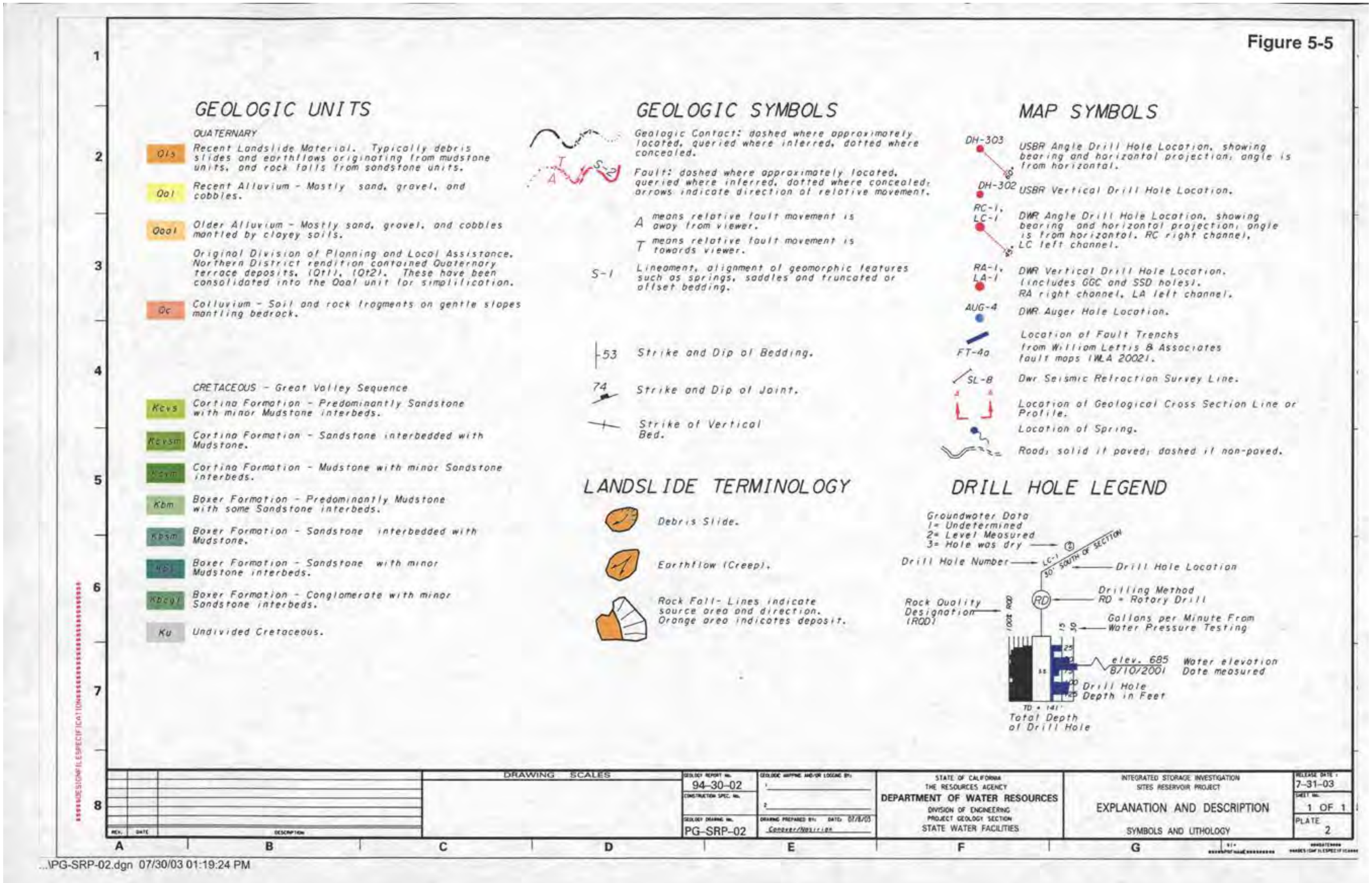


Figure B.2-4. Explanation and Description – Symbols and Lithology

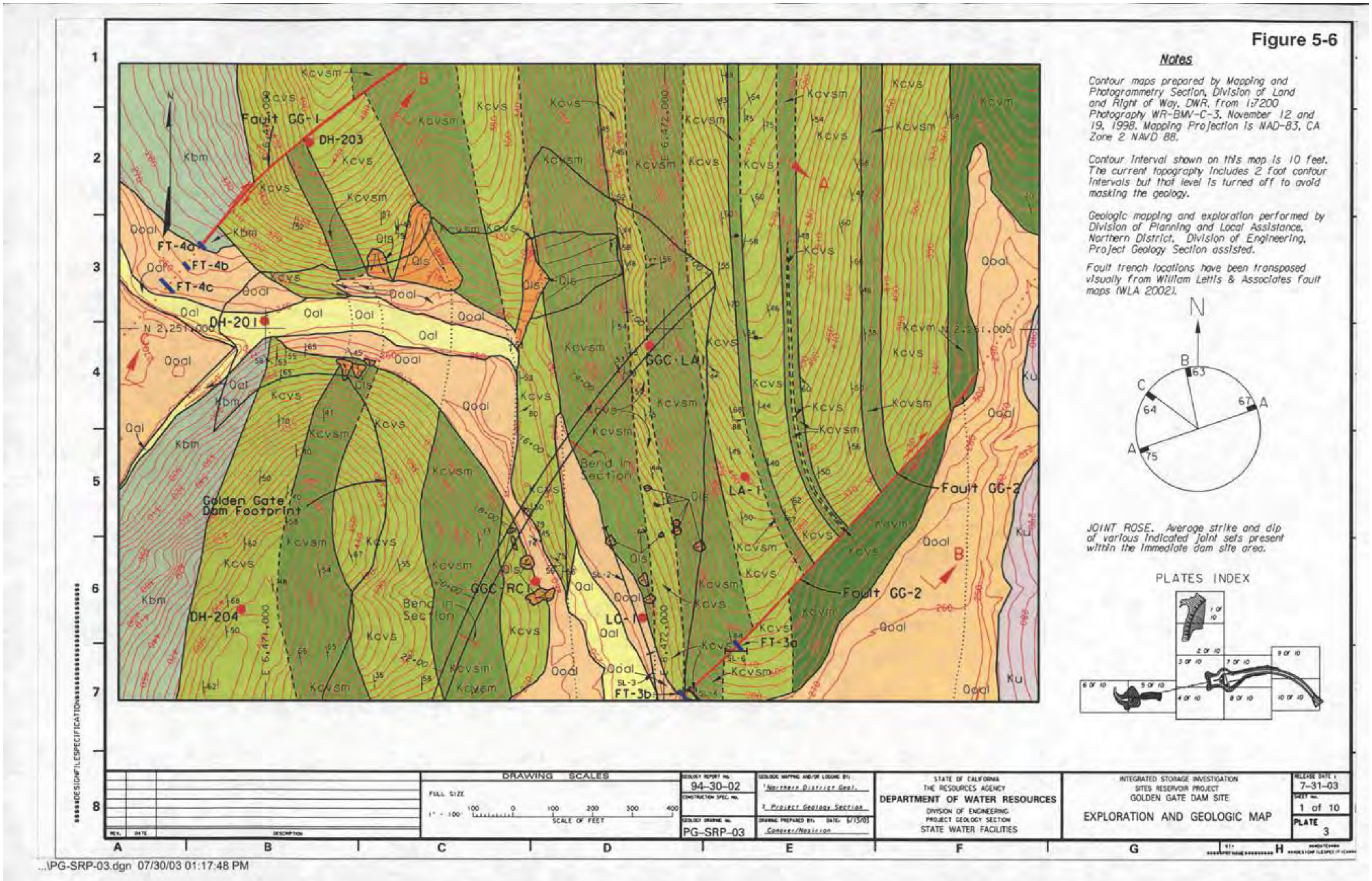


Figure B.2-5A. Golden Gate Dam Site – Exploration and Geologic Map

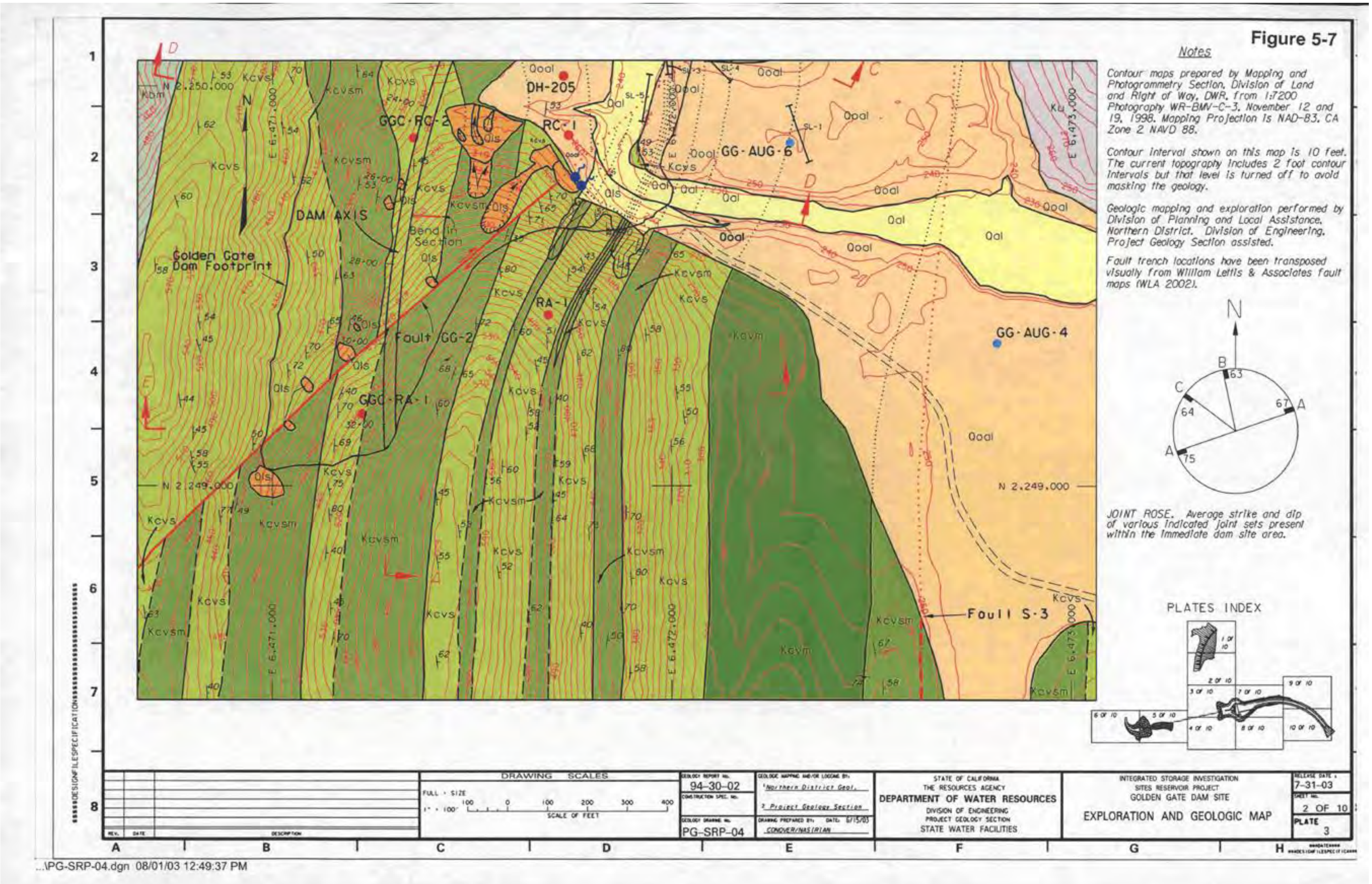


Figure B.2-5B. Golden Gate Dam Site – Exploration and Geologic Map

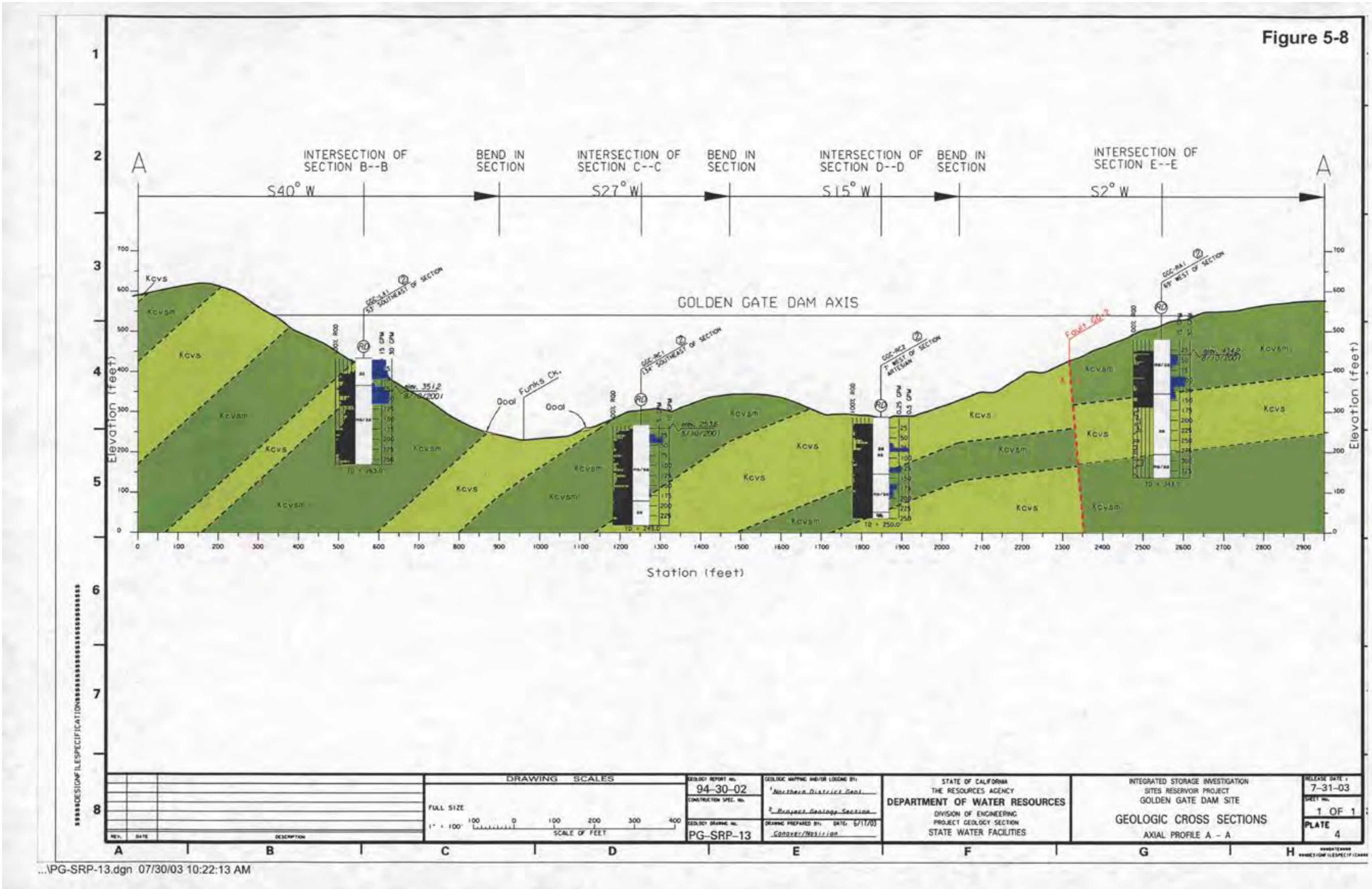


Figure B.2-6. Golden Gate Dam Site – Geologic Cross-Sections – Axial Profile A-A

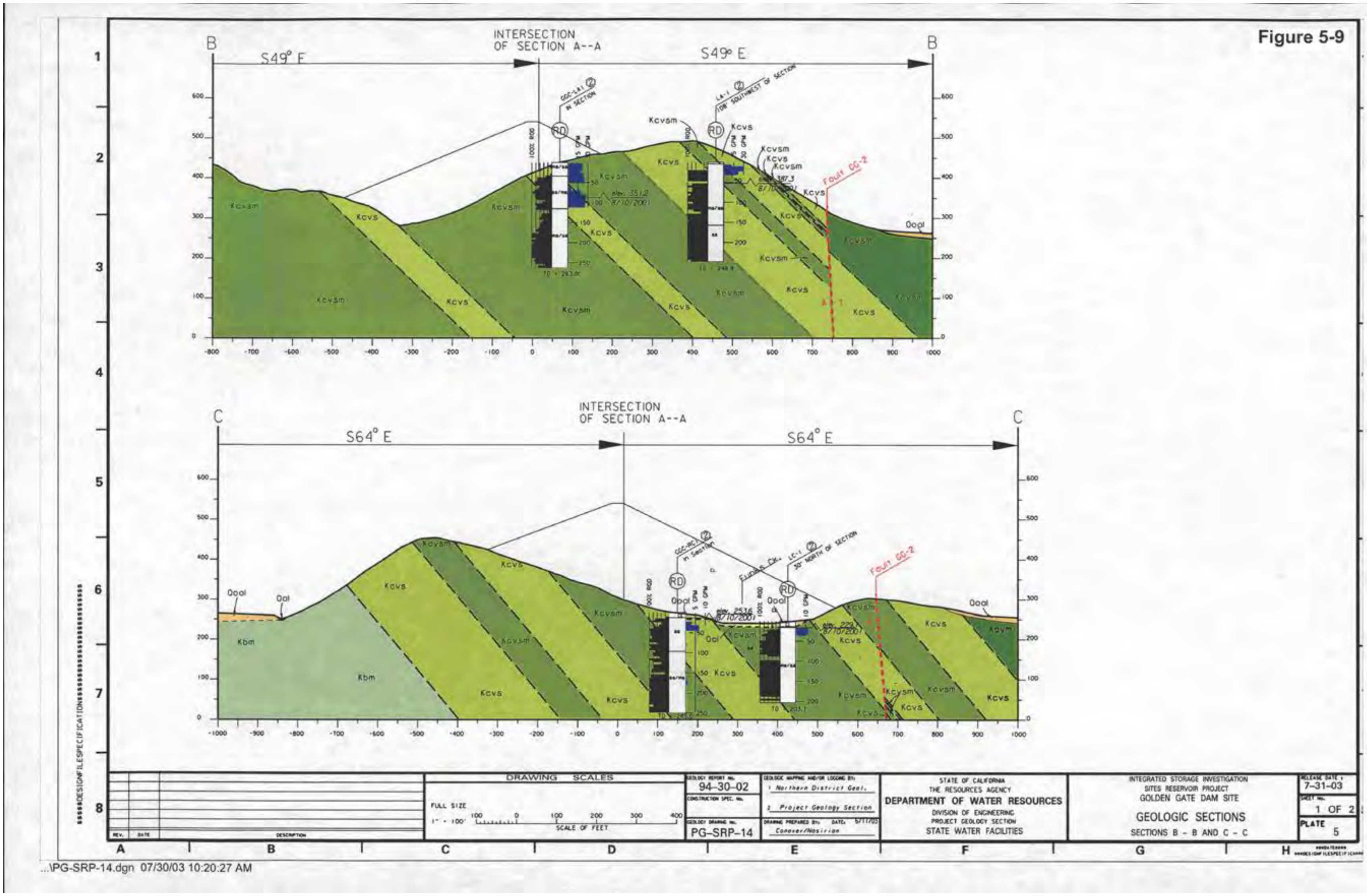


Figure B.2-7. Golden Gate Dam Site – Geologic Cross-Sections – Sections B-B and C-C

up to 250 feet. The second fault, GG-2, is located approximately 650 feet southeast (downstream) of the left abutment, passing under the downstream footprint, and traversing across the upper right abutment. GG-2 trends approximately N50°E, and displays a maximum right-lateral separation of approximately 1,300 feet.

Some colluvium, minor landslides, rockfalls, and surficial slumps are present within the footprint of the proposed dam (Figures B.2-5A and B.2-5B). They do not appear to pose any construction problems, or ultimate dam safety/foundation problems. Current rock falls are associated with older quarry operations. Stripping of these materials would be required to reach a suitable foundation.

Water pressure testing in exploration drill holes generally indicated that the slightly weathered and fresh foundation rock would be fairly tight, with some localized zones of potential high hydraulic conductivity near the surface. Grouting would be required.

Groundwater data was collected from monitoring wells installed during exploration drilling. Water levels in the wells ranged from elevations of 357 to 382 feet in the left abutment, 231 to 254 feet in the channel, and 392 to 437 feet in the right abutment. Based on the groundwater elevations above, dewatering would be required, as some of the dam foundation and cut-off wall excavation would be below the water table.

Sites Dam Site

Sites Dam will be founded on Cretaceous sedimentary rocks of the Cortina and Boxer Formations that generally trend north-south and dip 50º downstream to the east (Figures B.2-8A, B.2-8B, B.2-9, and B.2-10). The Cortina Formation consists of sandstone (Kcvs) and interbedded sandstone and mudstone (Kcvsm) of the Venado member, while mudstone (Kbm) with sandstone interbeds (Kbs) make up the Boxer Formation rocks at the site. The Stone Corral Creek channel is comprised of up to 10 feet of recent alluvium (Qal) and colluvium consisting of silty and poorly graded sands and gravels. Up to 15 feet of older alluvial (Qoal) deposits of sandy and lean clay and silt with gravel underlying silty and clayey gravels, mantle the dam abutments. In some areas, there is no soil cover and sandstone bedrock is exposed at the surface. Some landslides and surficial slumps were mapped within the footprint of the proposed dam, with the Boxer Formation more susceptible to instability than the Cortina Formation.

The Cortina Formation sandstone at the site is thin to very thick bedded with thin interbeds of mudstone ranging from laminar to 5 feet thick, and moderately to well indurated. At and near the surface the sandstone is intensely to moderately weathered, moderately hard, moderately strong, and closely to moderately fractured. At depth, the sandstone is slightly weathered to fresh, hard to very hard, strong, and closely to slightly fractured to massive. Fractures are mostly associated with jointing, and are commonly healed with calcite.

The Boxer Formation at the site is approximately 60 to 80 percent mudstone with 20 to 40 percent interbedded sandstone. The Boxer mudstone unit is low to moderately hard, weak to moderately strong, closely to moderately fractured, and very thin to thinly bedded. Fractures in the mudstone are mostly associated with bedding, and commonly exhibit some plastic deformation in the form of slickensides or internal shearing. The mudstone is susceptible to slaking when exposed to air and/or water.

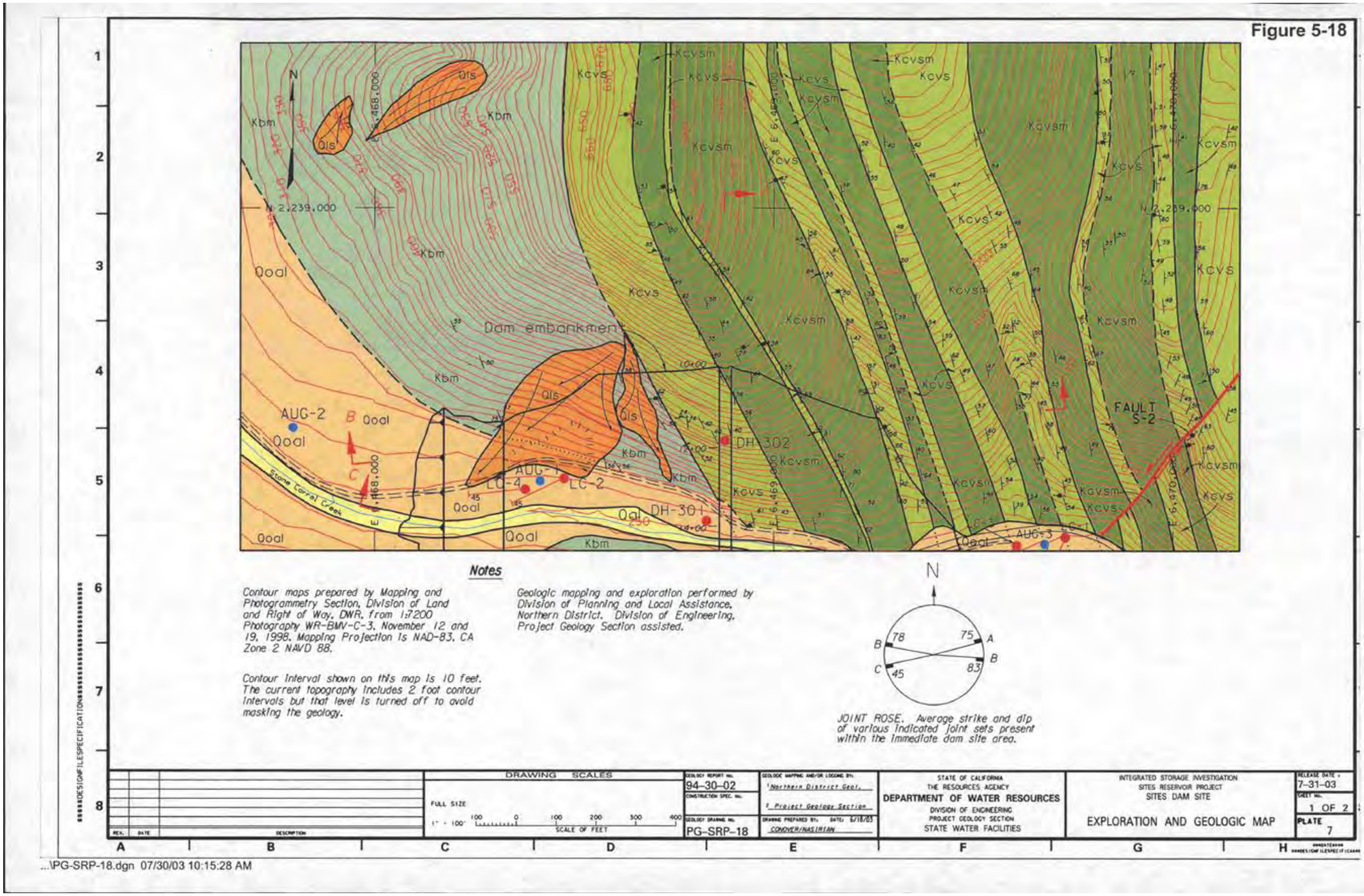


Figure B.2-8A. Sites Dam Site – Exploration and Geologic Map

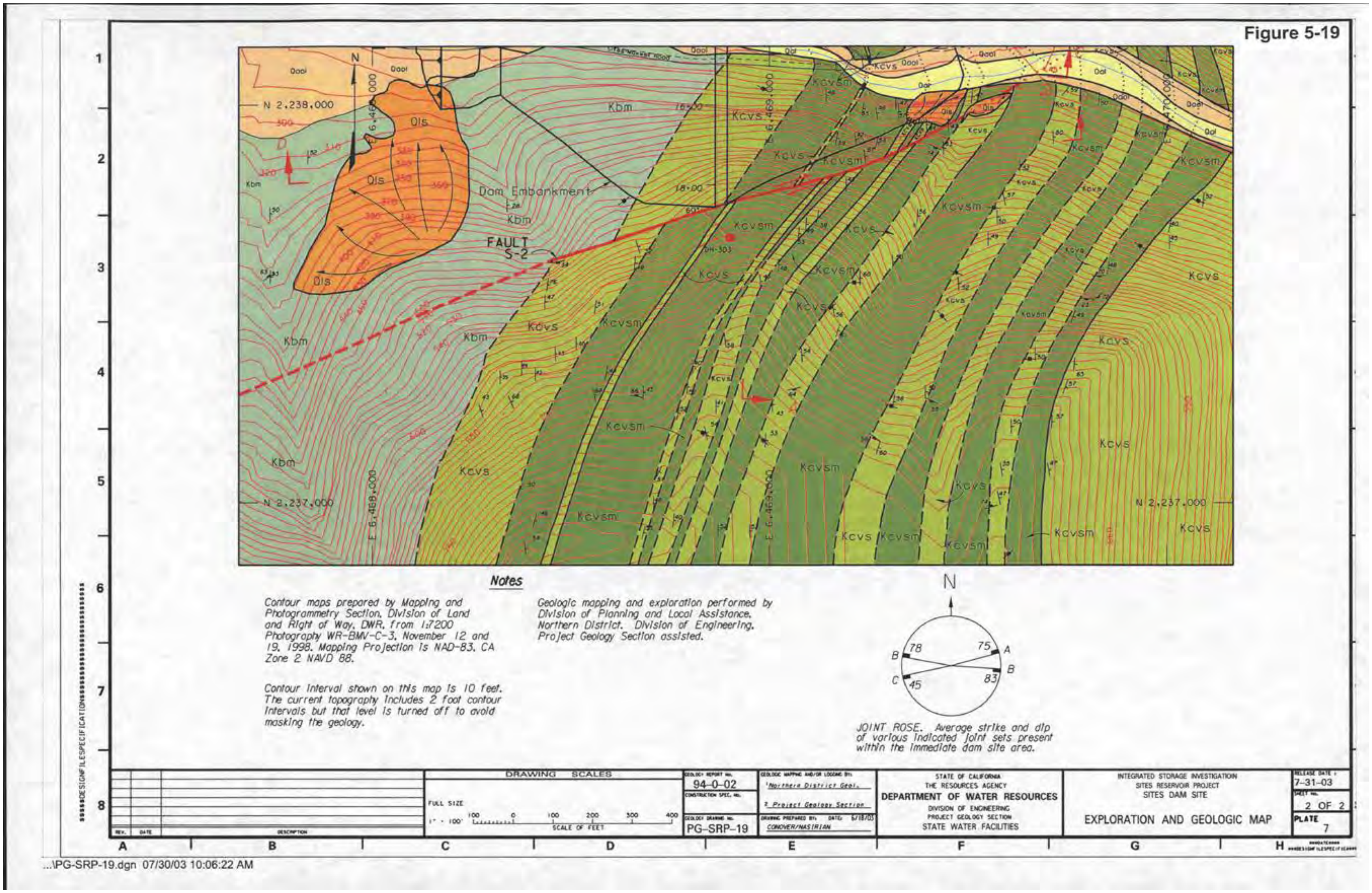


Figure B.2-8B. Sites Dam Site – Exploration and Geologic Map

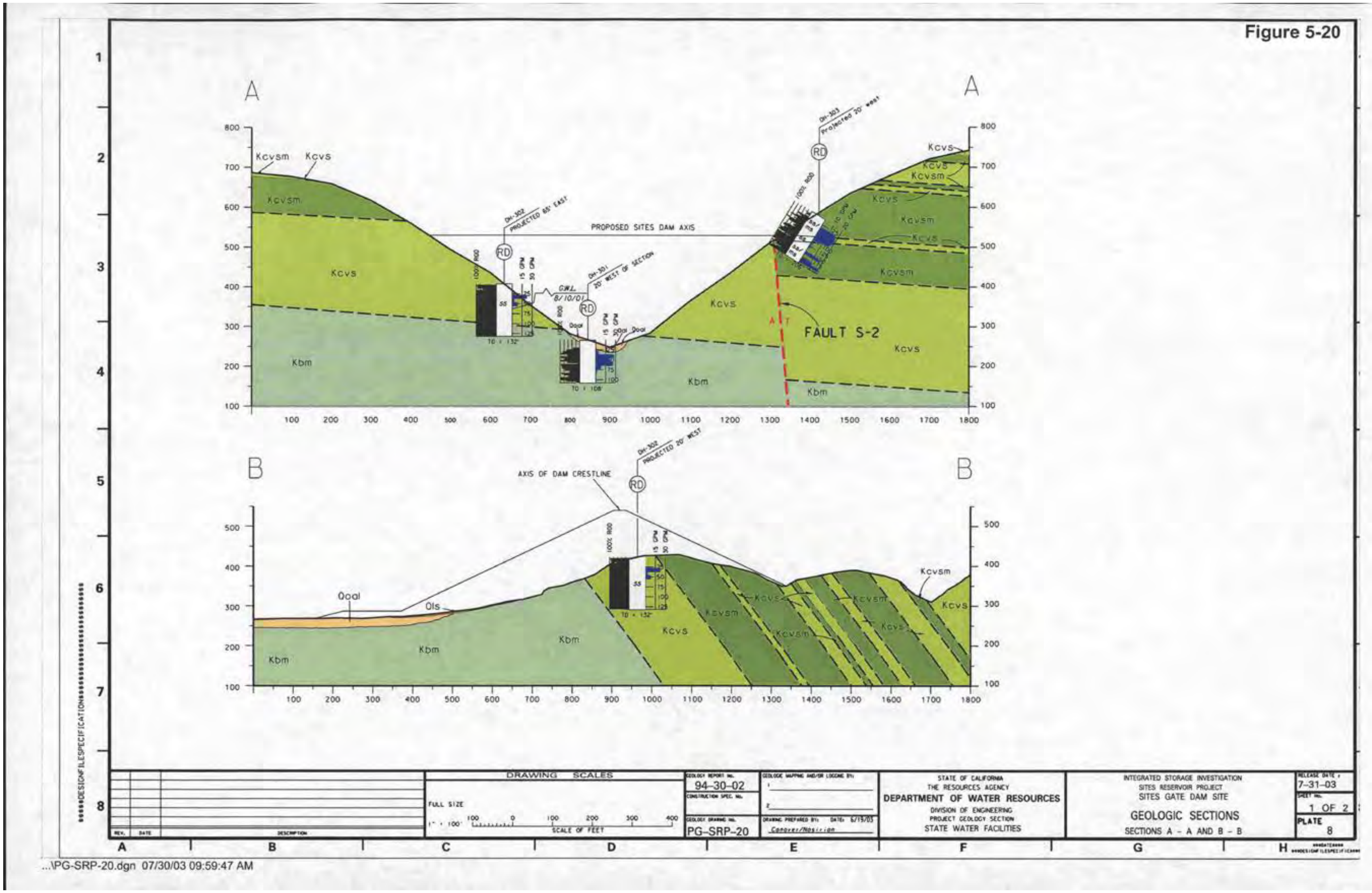


Figure B.2-9. Sites Dam Site – Geologic Sections – Sections A-A and B-B

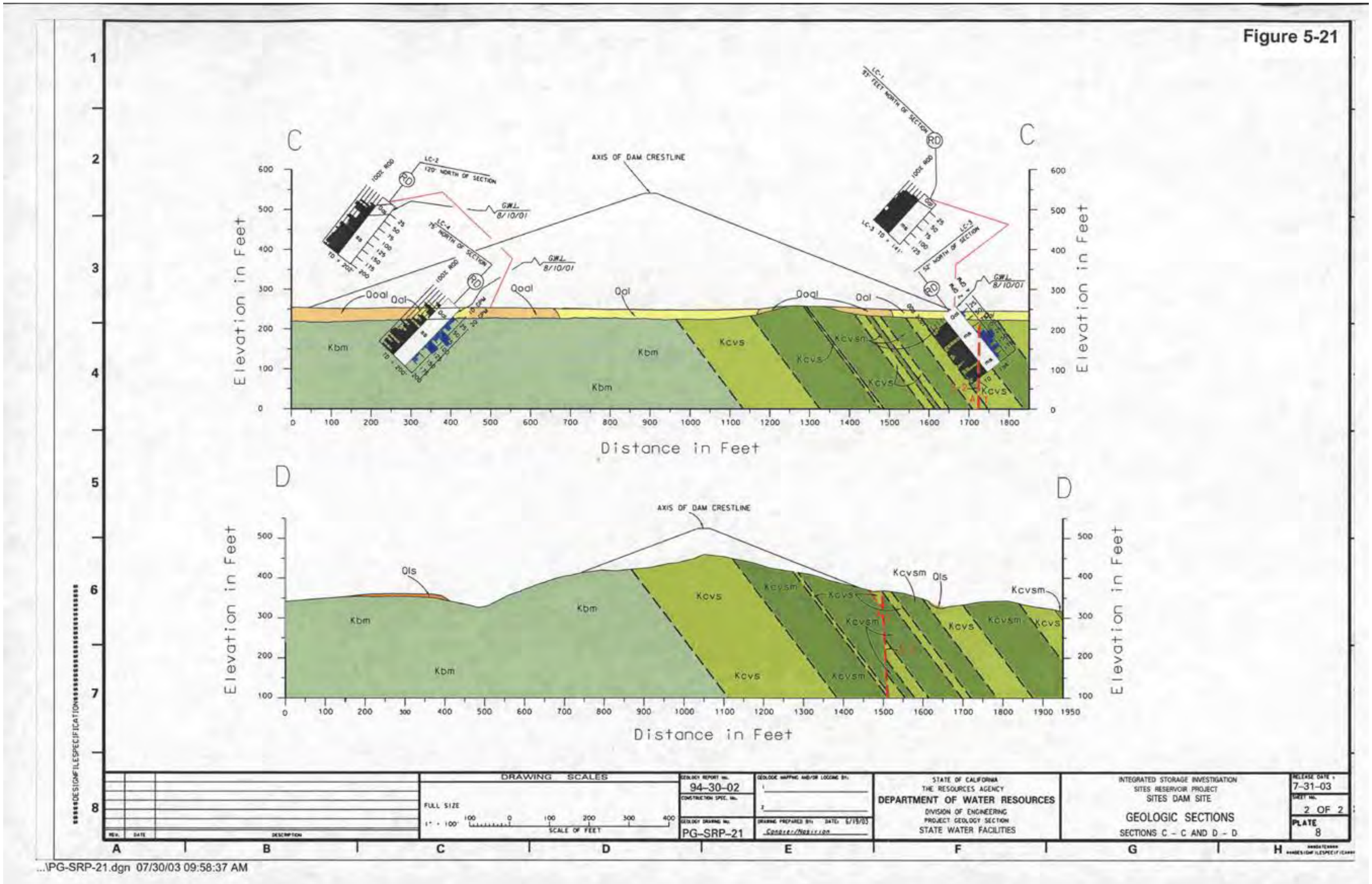


Figure B.2-10. Sites Dam Site – Geologic Sections – Sections C-C and D-D

Geologic mapping at the site identified two primary joint sets (Figures B.2-8A and B.2-8B). Set A trends nearly normal to the topographic ridges and bedding, striking between N57°E to N80°E and dipping 69° to 80° north. Joint Set B ranges in strike from N72°W to N86°W, and dips steeply from 78° north to 83° south. A minor joint set, Set C, was also identified striking approximately N74°E and dipping approximately 45° north.

Fault S-2 is a 2.5-mile-long, northeast-trending, tear fault. It is located approximately 900 feet downstream of the left abutment of the proposed Sites Dam, 700 feet downstream of the axis, and approximately 100 feet downstream of the toe on the right abutment (Figures B.2-8A and B.2-8B). The trend of Fault S-2 changes from approximately N60°E at the right abutment toe, to N70°E midway up the right abutment. In their fault trench studies, WLA determined that Fault S-2 is a narrow, sub-vertical bedrock shear zone that has a maximum right-lateral separation of approximately 550 feet.

Several landslides and slumps were identified in both the right and left abutments (Figures B.2-8A and B.2-8B). Two landslides were mapped on the left abutment, in Boxer Formation mudstones, within the footprint of the dam, upstream of the axis. Both are considered small, do not exceed a depth of 30 feet, and will be removed during excavation of the abutment. A larger slide of just under 6 acres was mapped on the right abutment, approximately 600 feet upstream of the axis, within the upstream portion of the dam footprint (Figures B.2-8A and B.2-8B). A toe bulge of up to 3 feet was observed in the western edge of the slide, but no bulge was detected to the east (inside the footprint). The depth of this slide does not appear to exceed 35 feet. Two additional minor landslides were mapped at the downstream toe of the right abutment, within the Cortina Formation.

Water pressure testing in exploration drill holes at the Sites Dam generally indicated that the slightly weathered and fresh foundation bedrock is fairly impermeable, with some localized zones of potentially high hydraulic conductivity closer to the surface. High water takes generally occurred to depths of 40 to 60 feet below the proposed excavation surface, with the rock mostly tight below 60 feet.

Groundwater ranged from elevations of 343 feet in the left abutment, to approximately 260 feet in the channel (representative of surface water elevations in Stone Corral Creek), and to approximately 450 feet in the right abutment.

Sites Reservoir Saddle Dam Sites and Emergency Signal Spillway

The nine saddle dams and emergency spillway proposed for the 1.81 MAF Sites Reservoir are located in the northern part of the reservoir, as shown on Figure B.2-11. The 1.27 MAF reservoir would have fewer saddle dams because the maximum WSE is 40 feet lower. Saddle dams will be constructed on interbedded mudstone, sandstone, and conglomerate sedimentary rocks of the Boxer Formation (Figure B.2-11). Overlying the bedrock is a lean to fat clay with a sand, colluvial cover (Qc) typically ranging from 5 to 25 feet in depth. The geologic rock units strike roughly north-south with a dip that ranges from west to east depending on the location of the saddle dam with respect to the Fruto syncline, which trends just west of Saddle Dam 9. Bedding associated with the eastern limb of the syncline dips at approximately 20° west at Saddle Dam 8, and flattens to 5° west at Saddle Dam 9. Bedding dip direction

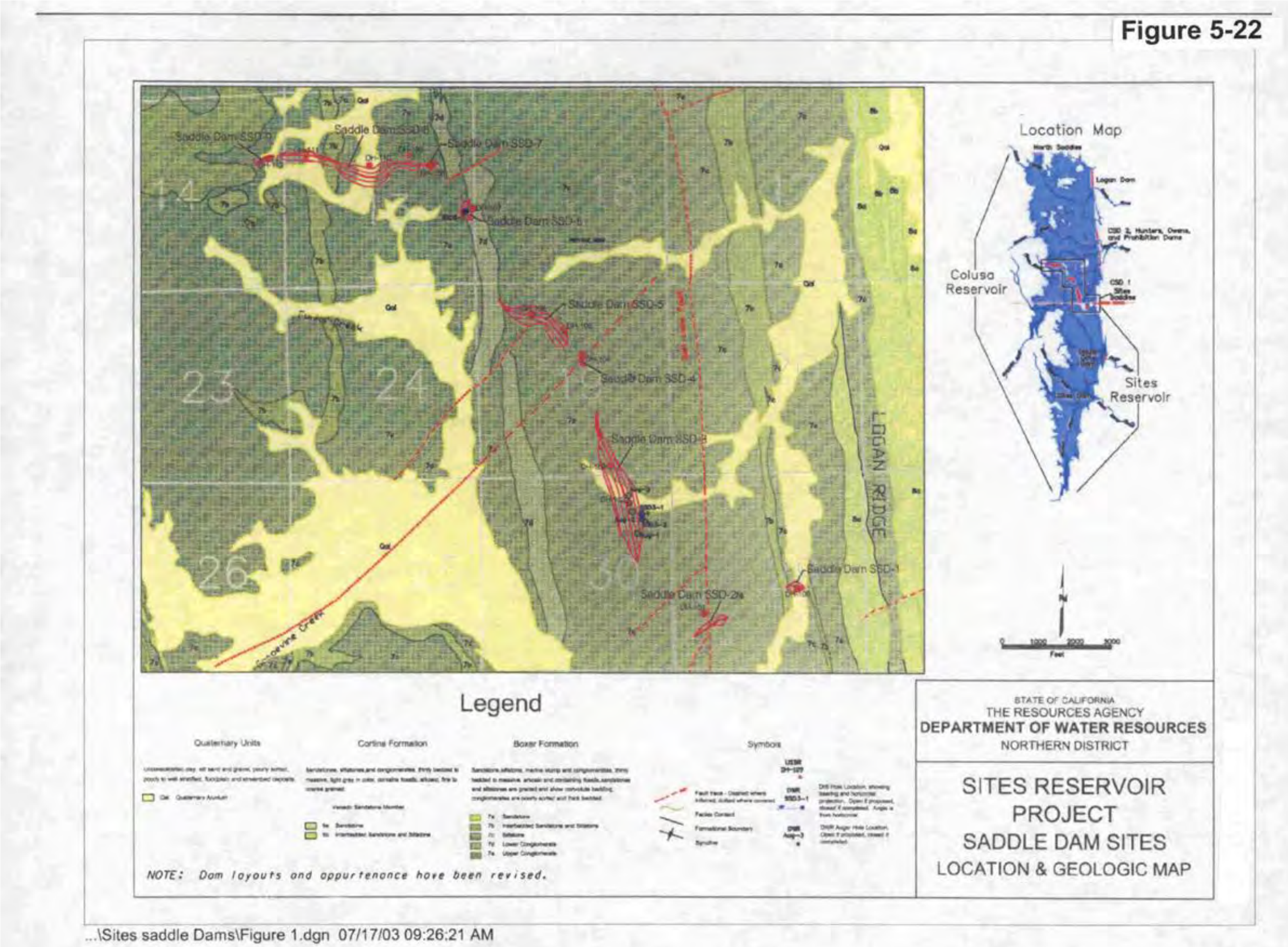


Figure B.2-11. Sites Reservoir Project – Saddle Dam Sites – Location and Geologic Map

gradually changes from west to east, near the intersection of the Salt Lake fault, and possibly the northern extension of the Sites anticline in the vicinity of Saddle Dam 1 and Saddle Dam 2.

Bedrock at the saddle dam sites is generally approximately 70 percent mudstone (Kbm), 25 percent sandstone (Kbs), and 5 percent massive conglomerate (Kbcgl). The mudstone ranges from decomposed to fresh, intensely to slightly fractured, friable to moderately hard, weak to moderately strong, and laminar to thinly bedded. The sandstone is thin to very thick bedded, decomposed to fresh, intensely to moderately fractured, low hardness to hard, and weak to very strong. Bedding trends between N30°W and N20°E, dipping approximately 65° to 70° east to 20° to 40° west.

The conglomerate exposed on the left abutment of Saddle Dam 5 is mostly low hardness to hard, weak to strong, intensely to closely fractured, very thick-bedded and contains 45 percent sandstone and 5 percent mudstone interbeds.

The Salt Lake fault, judged capable of generating up to 16 inches of reverse displacement in a single event (WLA, 2002), has been projected through the left abutment of Saddle Dam 2 (Figure B.2-11). A northeast trending tear fault is projected through the center of Saddle Dam 4 (Figure B.2-11), but has not been verified. At least two northeast-trending shear zones, referred to as lineaments LSSD5-4 and LSSD5-3, are projected through Saddle Dam 5. Lineament LSSD5-4 is projected through the middle of the dam axis, while lineament LSSD5-3 trends through a proposed slurry wall (Figure B.2-11).

Minor surficial landslides and slumps were mapped on the right abutment and upstream of the left abutment of Saddle Dam 1, but are not considered to be a problem.

Sites Reservoir Inlet/Outlet Works

Two alignments were considered for the inlet/outlet tunnel from the SPGP location. The two alignments are referred to as the Long Tunnel (approximately 4,000 feet) and the Short Tunnel (approximately 2,500 feet), both with a gate shaft option or multi-level outlet works option. Both the Long and Short Tunnels are designed to be 30 feet in diameter. Initially, the Long Tunnel alignment along with the gate shaft option was investigated because it avoided the intersection of Fault GG-2 in the tunnel and only encountered the fault in the intake channel in the reservoir (Figures B.2-12A through B.2-12H). The geologic profile along this alignment is illustrated in Figures B.2-13A and B.2-13B. The Short Tunnel alignment alternative would encounter Fault GG-2 approximately 600 feet downstream of the proposed intake tower.

The proposed 30-foot-diameter concrete-lined tunnel for both the Long and Short Tunnel alignments would be through the Cortina and Boxer Formations characterized by interbedded sandstones (Kcvsm) and mudstones (Kbm). Maximum cover for the Long and Short Tunnel alignments would be approximately 550 and 400 feet, respectively. Geologic mapping indicates that approximately 500 feet, or 20 percent, of the Short Tunnel alignment would be in the Boxer Formation, which consists of approximately 70 percent mudstone. The remaining 2,000 feet, or 80 percent, of the

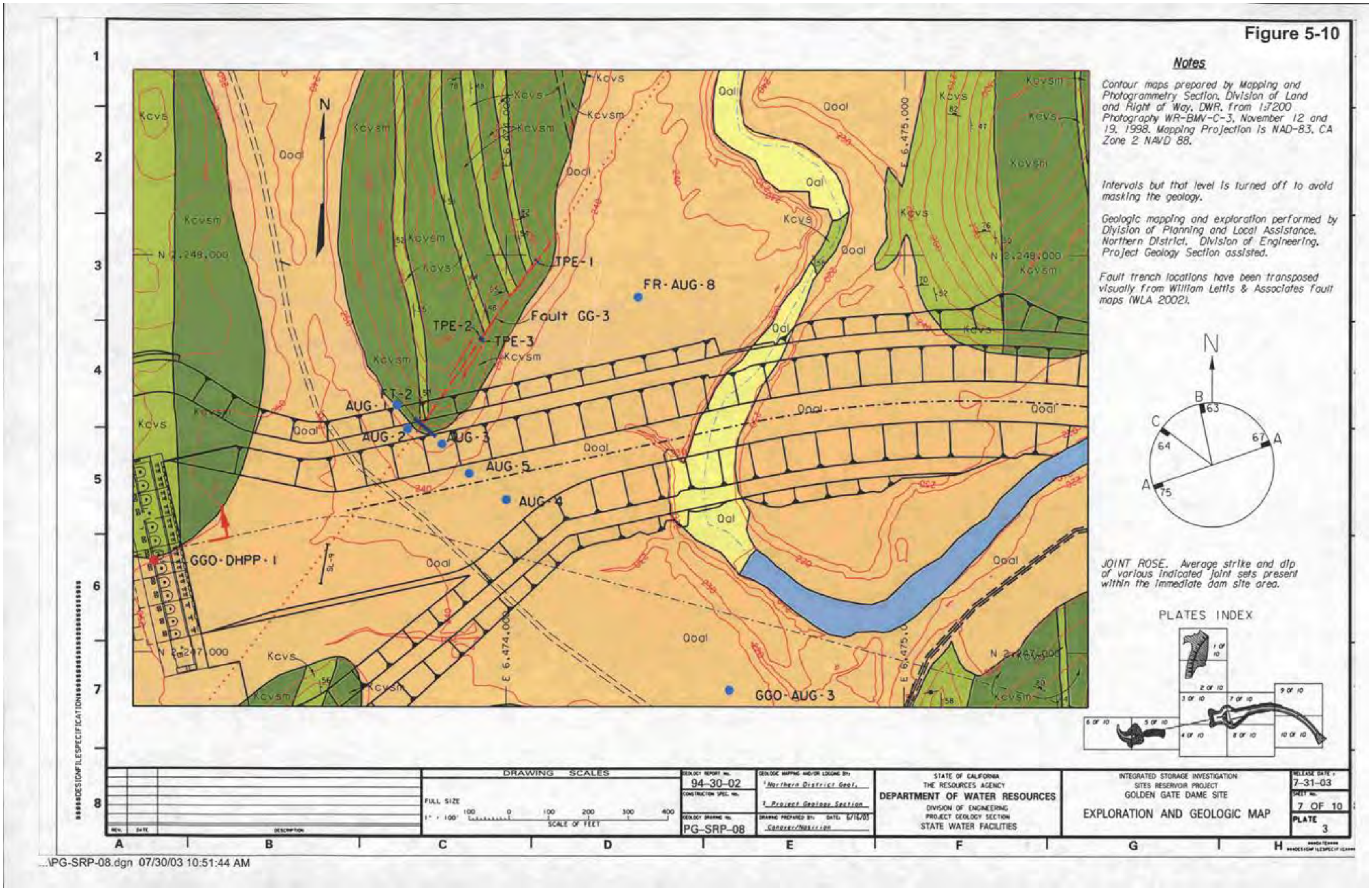


Figure B.2-12A. Golden Gate Dam Site – Exploration and Geologic Map

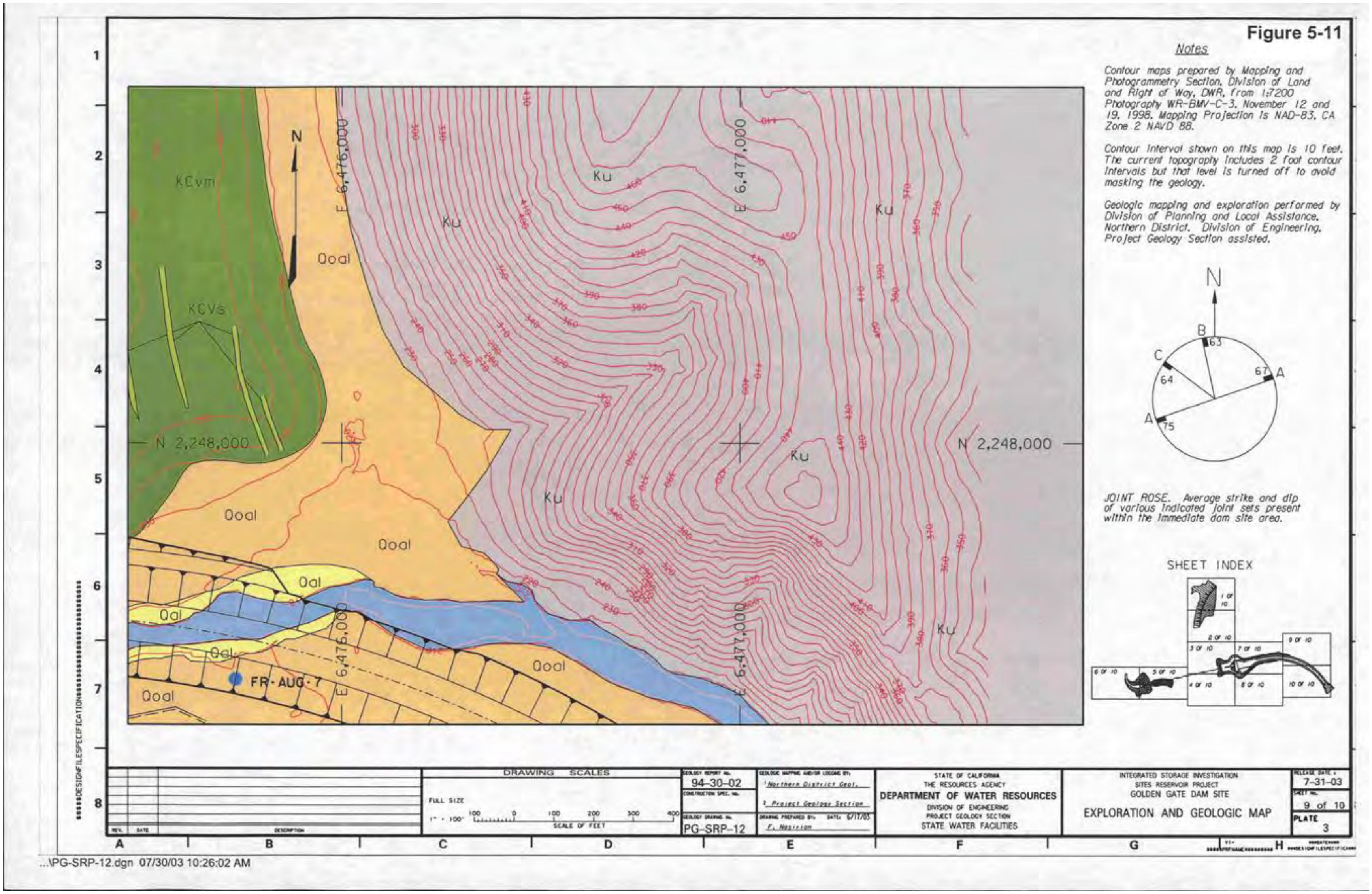


Figure B.2-12B. Golden Gate Dam Site – Exploration and Geologic Map

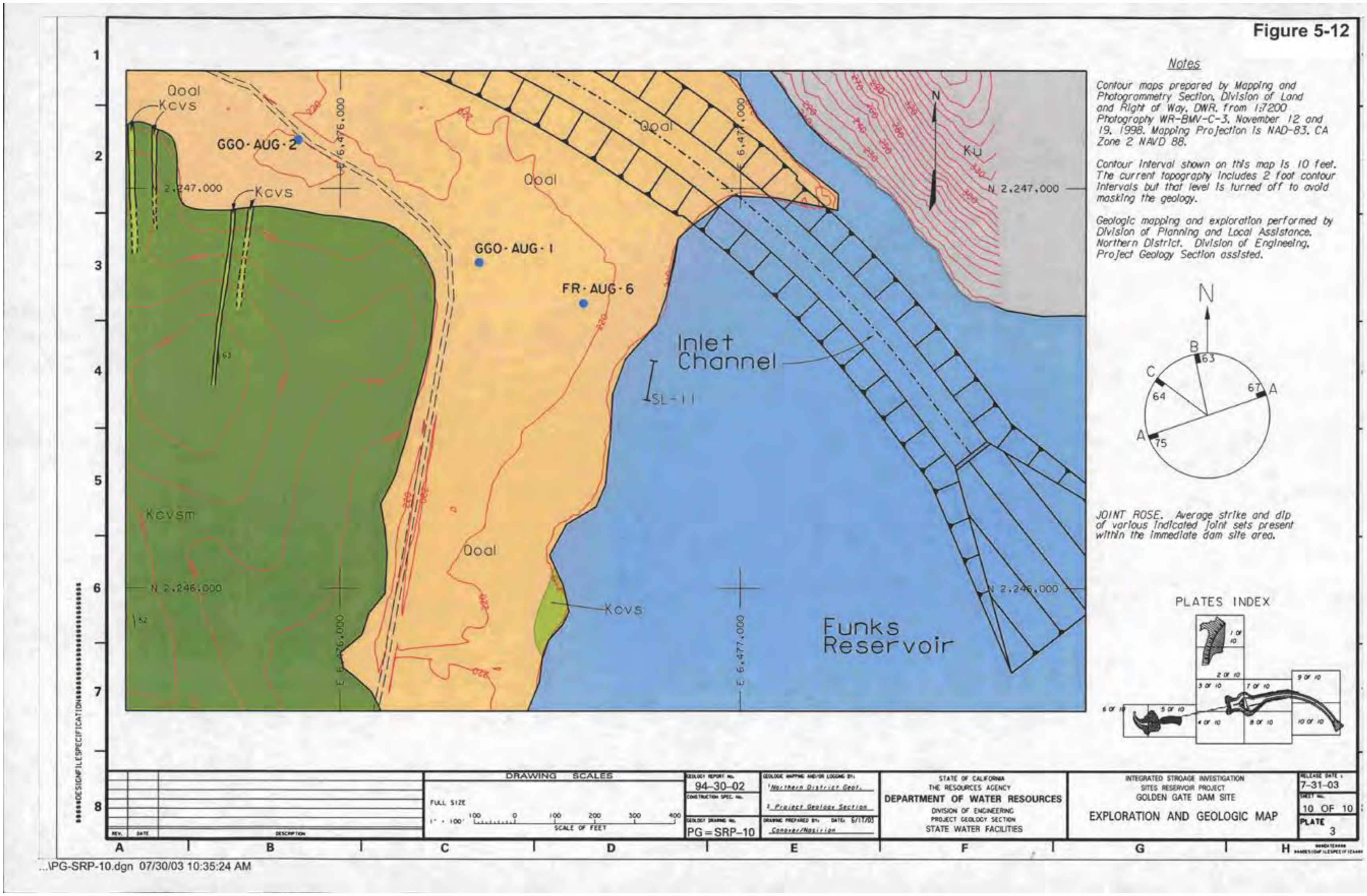


Figure B.2-12C. Golden Gate Dam Site – Exploration and Geologic Map

Figure B.2-12D. Golden Gate Dam Site – Exploration and Geologic Map

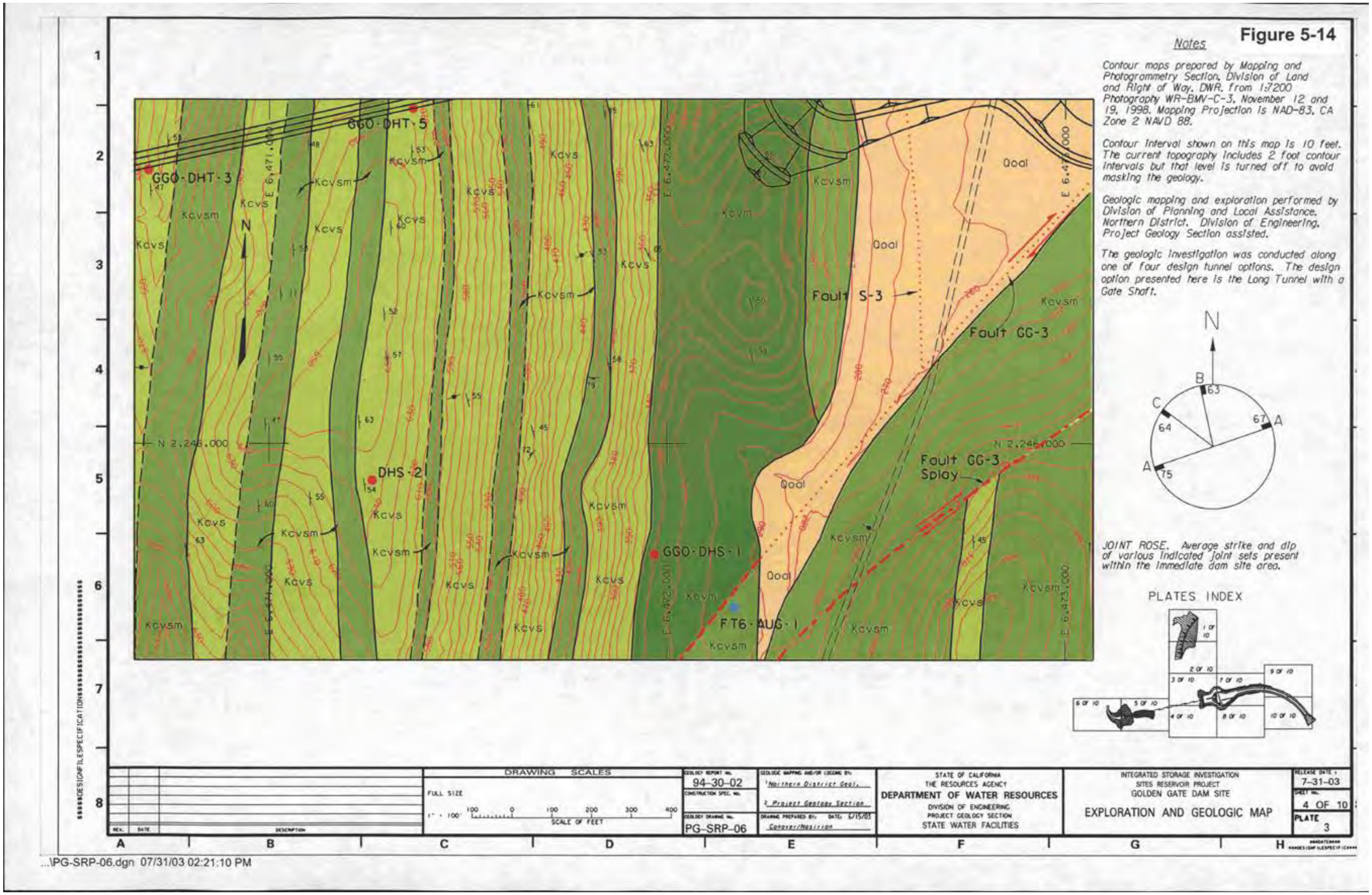


Figure B.2-12E. Golden Gate Dam Site – Exploration and Geologic Map

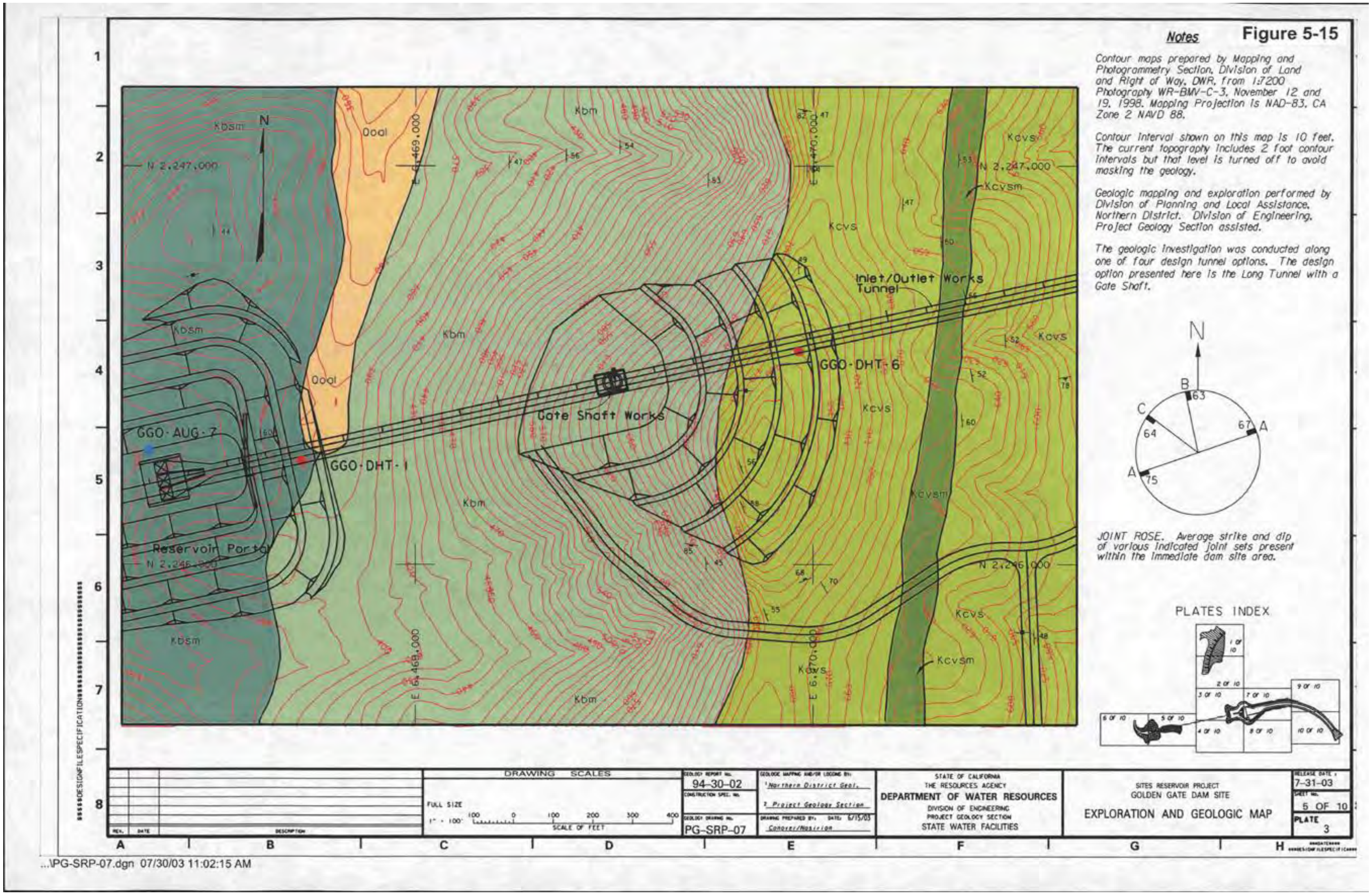


Figure B.2-12F. Golden Gate Dam Site – Exploration and Geologic Map

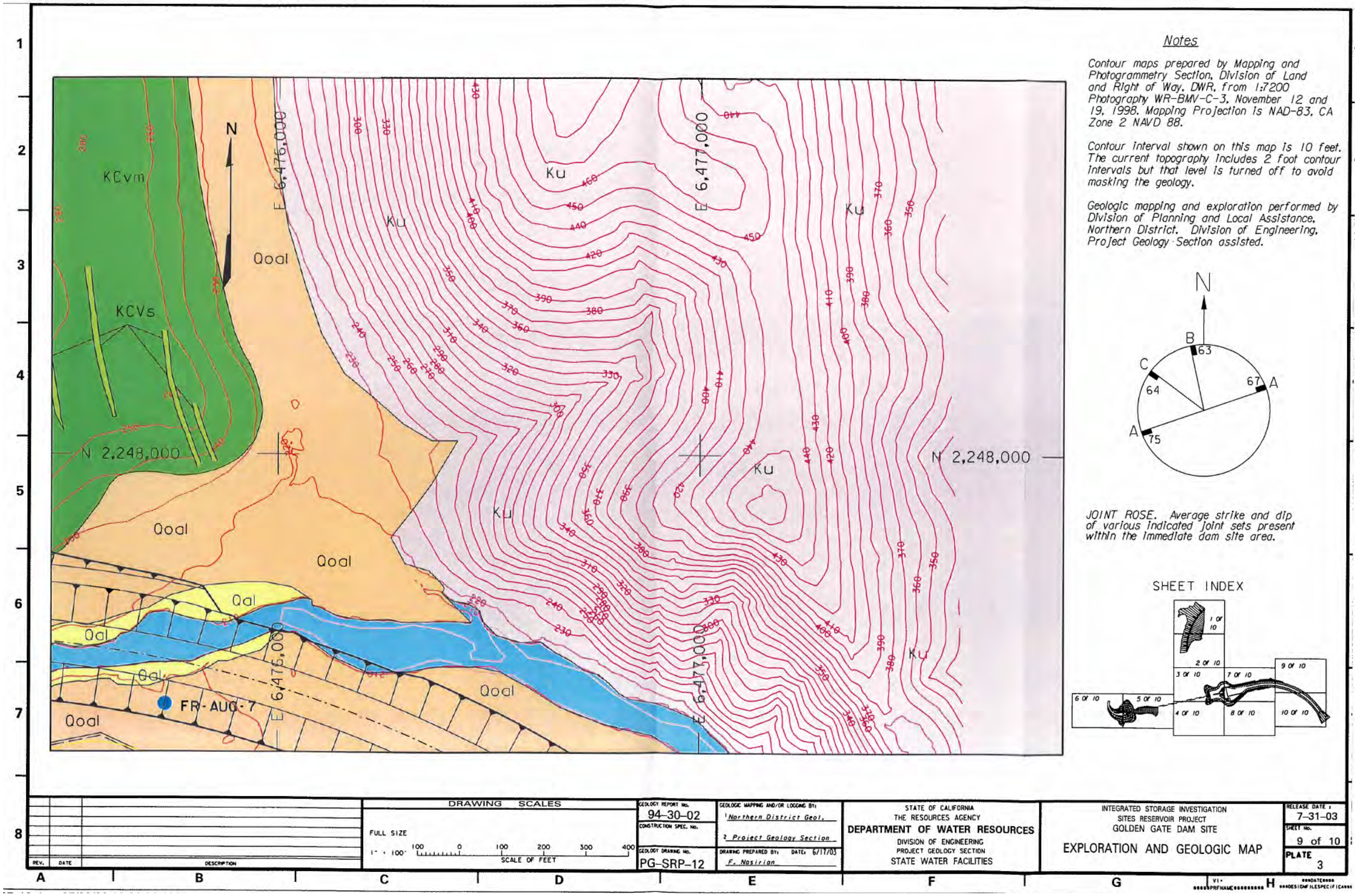


Figure B.2-12G. Golden Gate Dam Site – Exploration and Geologic Map

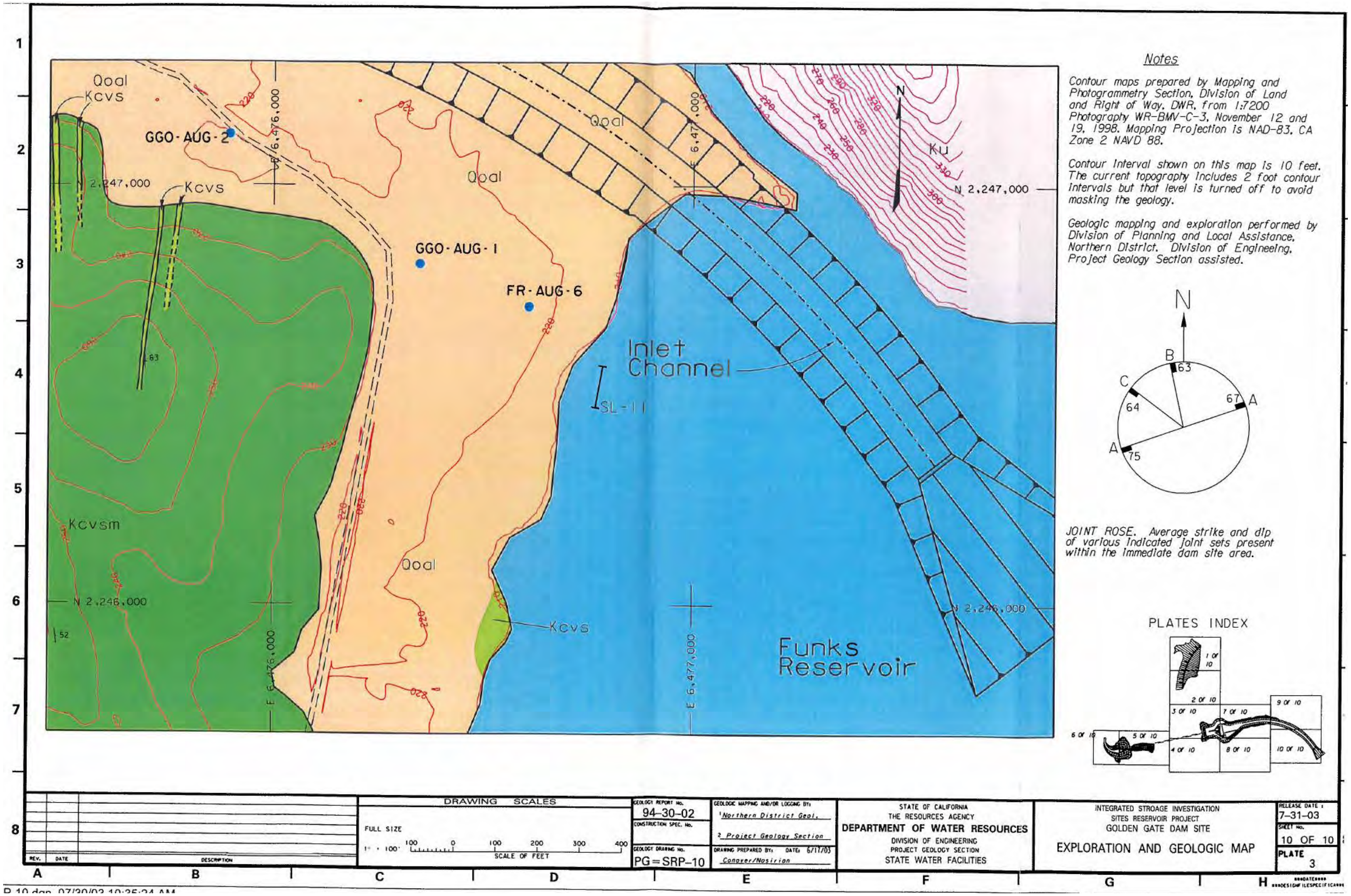


Figure B.2-12H. Golden Gate Dam Site – Exploration and Geologic Map

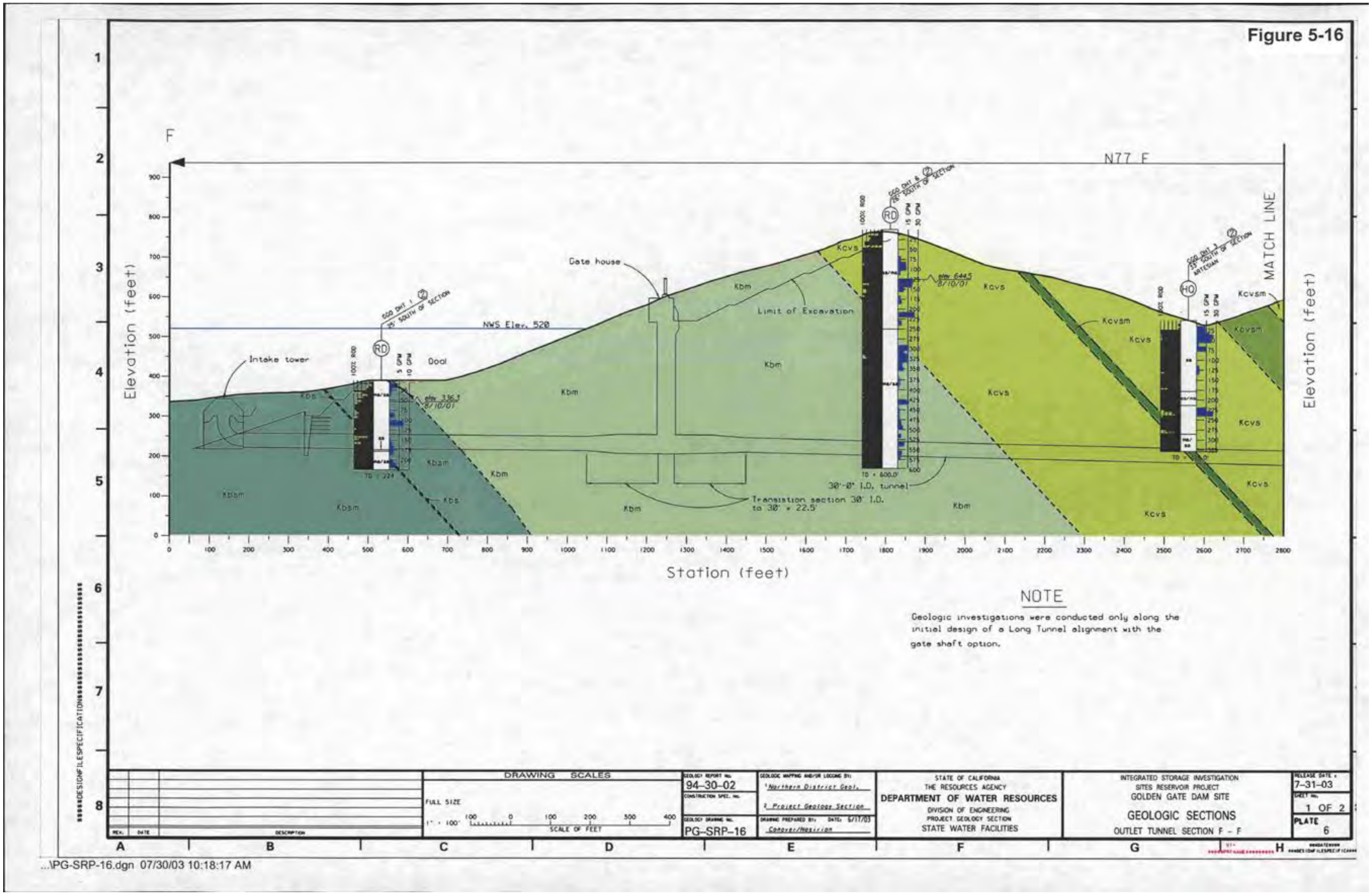


Figure B.2-13A. Golden Gate Dam Site – Geologic Sections – Outlet Tunnel Section F- F

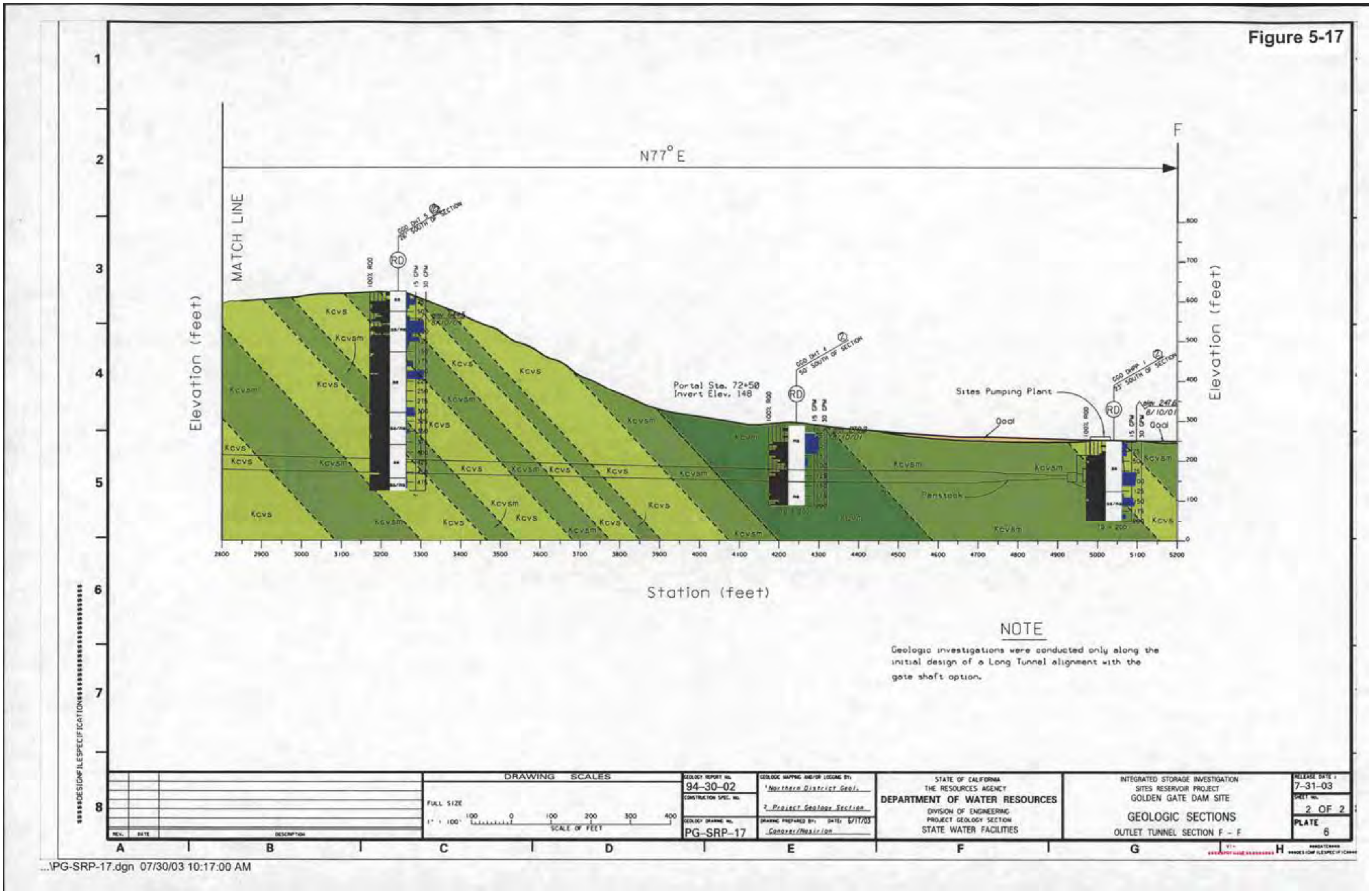


Figure B.2-13B. Golden Gate Dam Site – Geologic Sections - Outlet Tunnel Section F-F

alignment would encounter the Cortina Formation, which is approximately 70 percent laminated to thinly-bedded mudstone with very-thinly- to thickly-bedded sandstone interbeds. The Cortina Formation consists of very thinly- to very thick-bedded sandstone with laminated to thinly-bedded mudstone.

Fresh sandstone is generally hard and strong, and fresh mudstone is moderately hard and moderately strong. Sandstone and mudstone core samples from exploration tunnel drill holes were submitted for testing. Table 8 of the Project Geology Report 94-30-02 lists the test results. Based on preliminary drill hole information, tunneling conditions through these sedimentary rocks would be good. Strike of the bedding is roughly north-south, nearly normal to the tunnel alignment, with an average dip of approximately 50º east. Drill hole logs indicate that some steeply-dipping (70^{o}) shears may be present at tunnel grade.

Some instability or minor overbreak may occur in the crown associated with laminated bedding of the mudstone. Moderate overbreak may occur along the tunnel walls where shears and associated fractured rock are present. Rock quality designation (RQD) values indicate that tunnel support requirements would utilize light to moderate weight steel sets on approximately 4-foot centers. Local areas of fractured rock associated with shearing may require heavier steel sets and/or closer spacing.

All five of the tunnel exploration drill holes showed groundwater levels well above tunnel grade, indicating that groundwater will be encountered during tunnel excavation. Figures B.2-13A and B.2-13B illustrate drill hole groundwater levels. One drill hole encountered artesian flows estimated to be approximately 10 gallons per minute (gpm), indicating that some isolated, confined, high groundwater flows will be encountered during the tunnel excavation. In addition, water pressure testing was performed in the tunnel drill holes to estimate permeabilities for the tilted strata, especially where they intersect the tunnel alignment. Water pressure tests at tunnel grade in the five exploration holes showed no water losses, indicating that the rock in the tunnel at these locations is fairly impermeable. Subsequent references to Funks Reservoir refer to the existing reservoir.

SPGP, Approach Channel, and Funks Reservoir

The evaluation described below was prepared at a time when the existing Funks Reservoir was to be incorporated into the NODOS project within its existing limits. However, to enhance power generation opportunities, the Holthouse (Enlarged Funks) Reservoir facility has since been developed. Holthouse Reservoir would involve constructing a new dam east of the existing Funks Dam and breaching the existing dam to form a larger reservoir with an active storage of approximately 6,500 AF. The SPGP channel would be deepened and extended eastward beyond the limits shown on the figures referenced below to connect into the expanded reservoir. Additional geotechnical investigation would be required as part of future development work for the new dam and along the extended channel alignment.

The proposed SPGP, the associated approach channel from Funks Reservoir, and Funks Reservoir itself, are located on the eastern side of the primary ridge at the right abutment of the Golden Gate Dam. The curved approach channel would extend approximately 5,000 feet from Funks Reservoir (Figures B.2-12E through B.2-12H).

Sandstone with interbedded mudstone of the Cortina Formation (Kcvsm) will comprise the foundation.

The strike of the bedding is generally north-south with a dip of approximately 50° east. The sandstone and interbedded mudstone are anticipated to be fresh and hard at invert and should provide excellent bearing capacity for the support of the structures. The older alluvium (Qoal) and recent alluvium (Qal) along the alignment for the approach channel ranges in depth from as shallow as 6 feet to the east at Funks Reservoir to approximately 35 feet at the west end. The soils are primarily lean clay and silt with some gravel interbeds. These soils may be erodible; therefore, the channel would likely require some type of protection.

Only one exploration borehole was drilled at the SPGP site. The rock is approximately 90 percent sandstone (Kcvs) with some minor mudstone interbeds. The sandstone is mostly hard and strong and slightly fractured to massive. The hardness and strength along with the excellent RQD values for the sandstone indicate that, below a depth of approximately 50 feet, blasting will be required. Depth-to-groundwater is approximately 13 feet.

Auger holes were advanced to the top of bedrock along the straight alignment for the approach channel. The current design would encounter approximately 35 to 50 feet of interbedded sandstone and mudstone (Kcvsm). Seismic velocities generated from seismic lines SL-10 and SL-11, in the vicinity of the approach channel, ranged between 8,000 and 9,000 feet per second (fps). Some blasting in the lower 5 to 15 feet of the excavation may be required in the harder, fresh sandstone.

Fault GG-3 trends (approximately N30°E) diagonally across the approach channel approximately 450 feet west of the pumping plant site (Figure B.2-12E). Fault trenches excavated by WLA indicate that the "GG-3 fault is a narrow (less than 2 feet wide), sub-vertical bedrock shear zone" (WLA, 2002). GG-3 may act as a groundwater barrier in the bedrock exposed in the approach channel.

Permanent cuts in the alluvial soils should be stable at slopes with a ratio of 2 horizontal to 1 vertical (2H:1V), or possibly at 1.5H:1V slopes upon further investigation and testing. Weathered bedrock slopes should be stable at 1H:1V, and fresh rock slopes at 0.5H:1V along the approach channel. Groundwater will be encountered in the excavation for the approach channel at a depth of approximately 25 feet or higher; therefore, dewatering will be required. Clearing will be minimal at the pumping plant and along the approach channel, as the only vegetation is light grasses and scattered pockets of riparian growth in the Funks Creek channel.

Conveyance Alternatives

Preliminary design of conveyance facilities for the NODOS project includes the 13-mile Delevan pipeline from the SRPGP to Holthouse Reservoir. The pipeline alignment was characterized using a number of auger borings, with SPT, and seismic refraction surveys. Geologic soil units traversed by the proposed conveyance alignment include, from east to west, Sacramento River channel deposits, Modesto Formation, Basin Deposits, Riverbank Formation, Tehama Formation, and Red Bluff Formation. Soils types encountered range from lean clay to poorly graded sand. Cretaceous age mudstone of the Cortina Formation was encountered at relatively shallow depths (16 feet) within the western-most 2 miles of the conveyance alignment. The mudstone was generally decomposed to intensely weathered between 16 and 52 feet, and is considered rippable to that depth. All of the soil units may be excavated using common methods (Figures B.2-14A through B.2-14C).

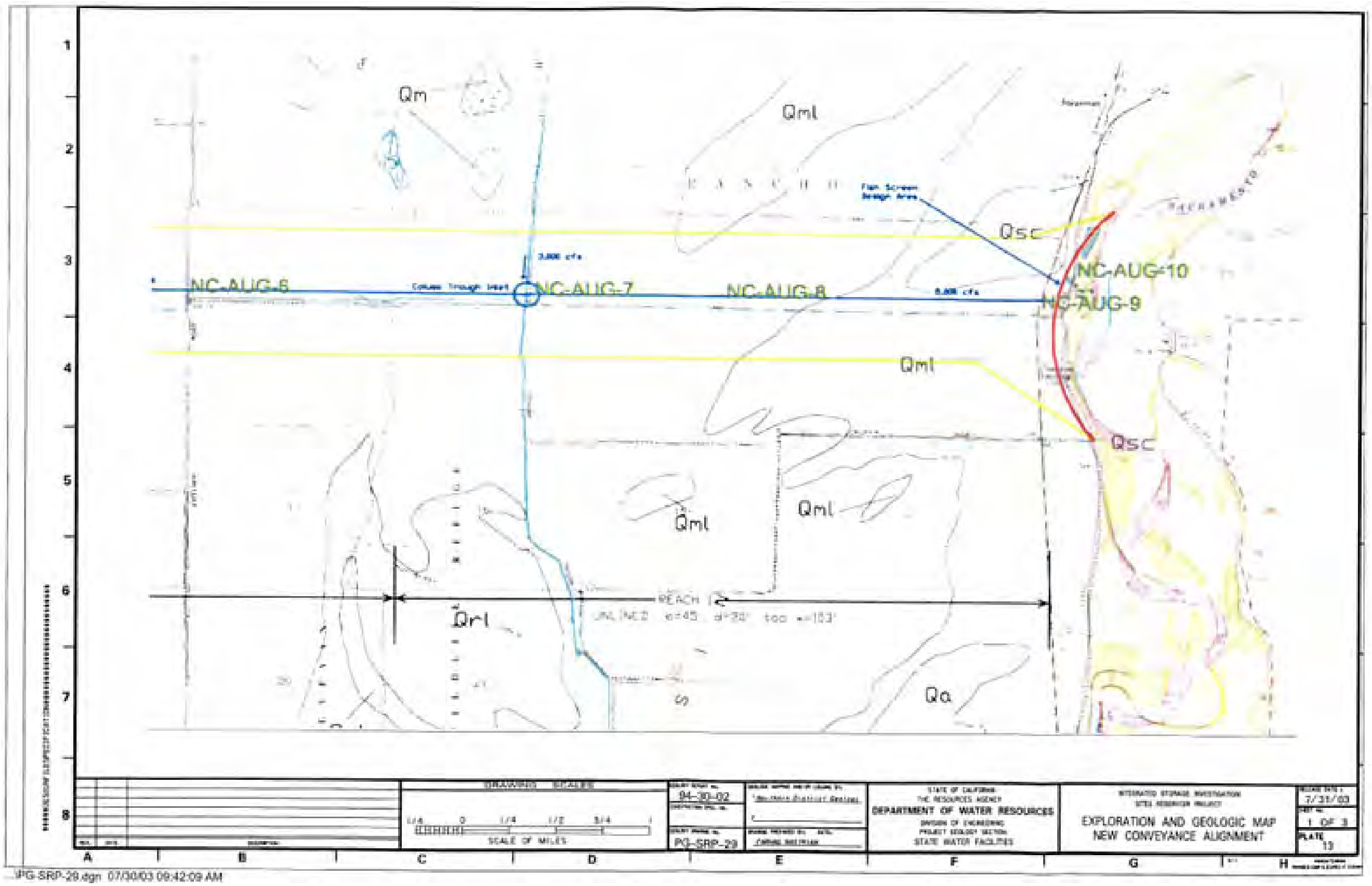


Figure B.2-14A. Exploration and Geologic Map – New Conveyance Alignment

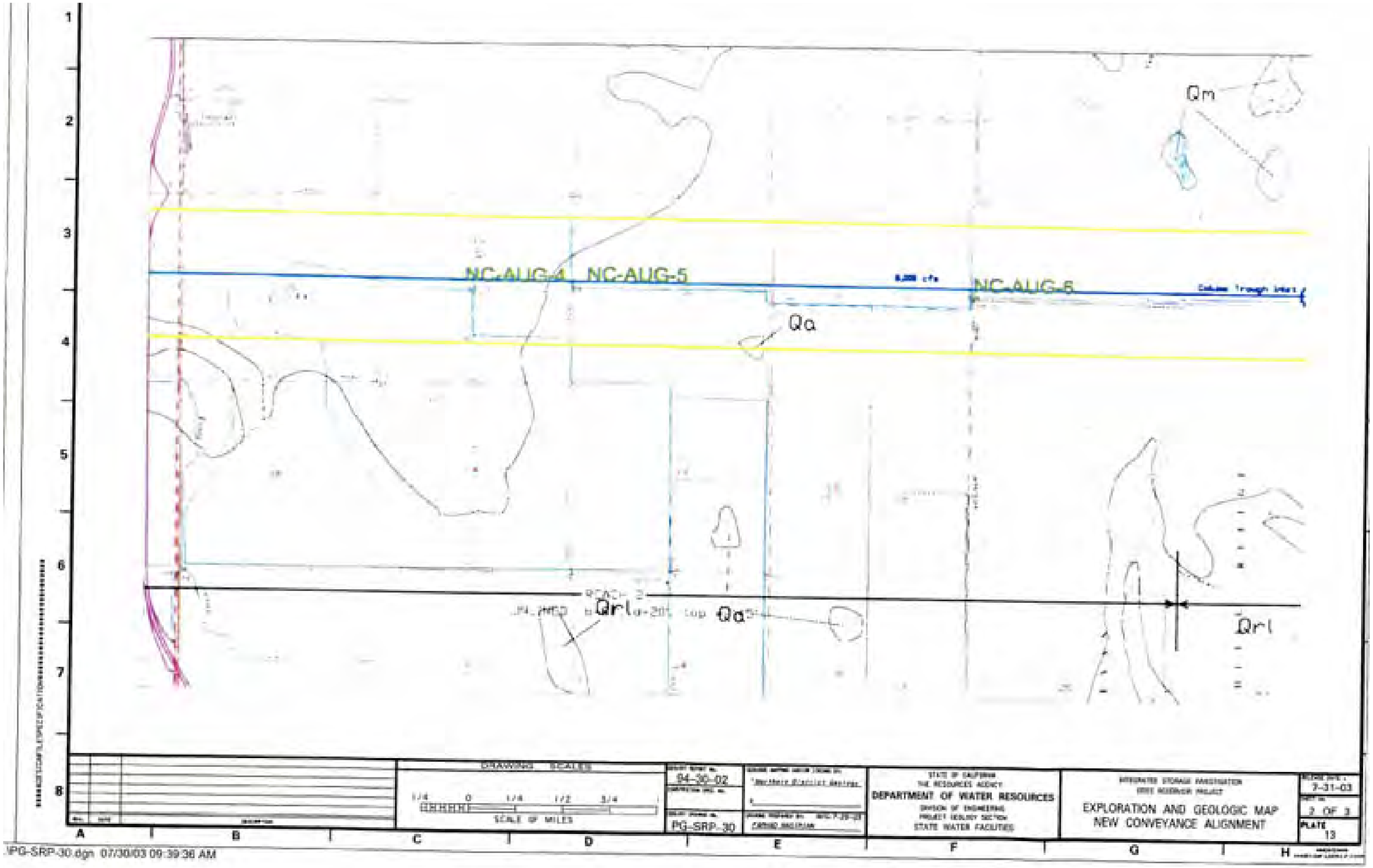


Figure B.2-14B. Exploration and Geologic Map – New Conveyance Alignment

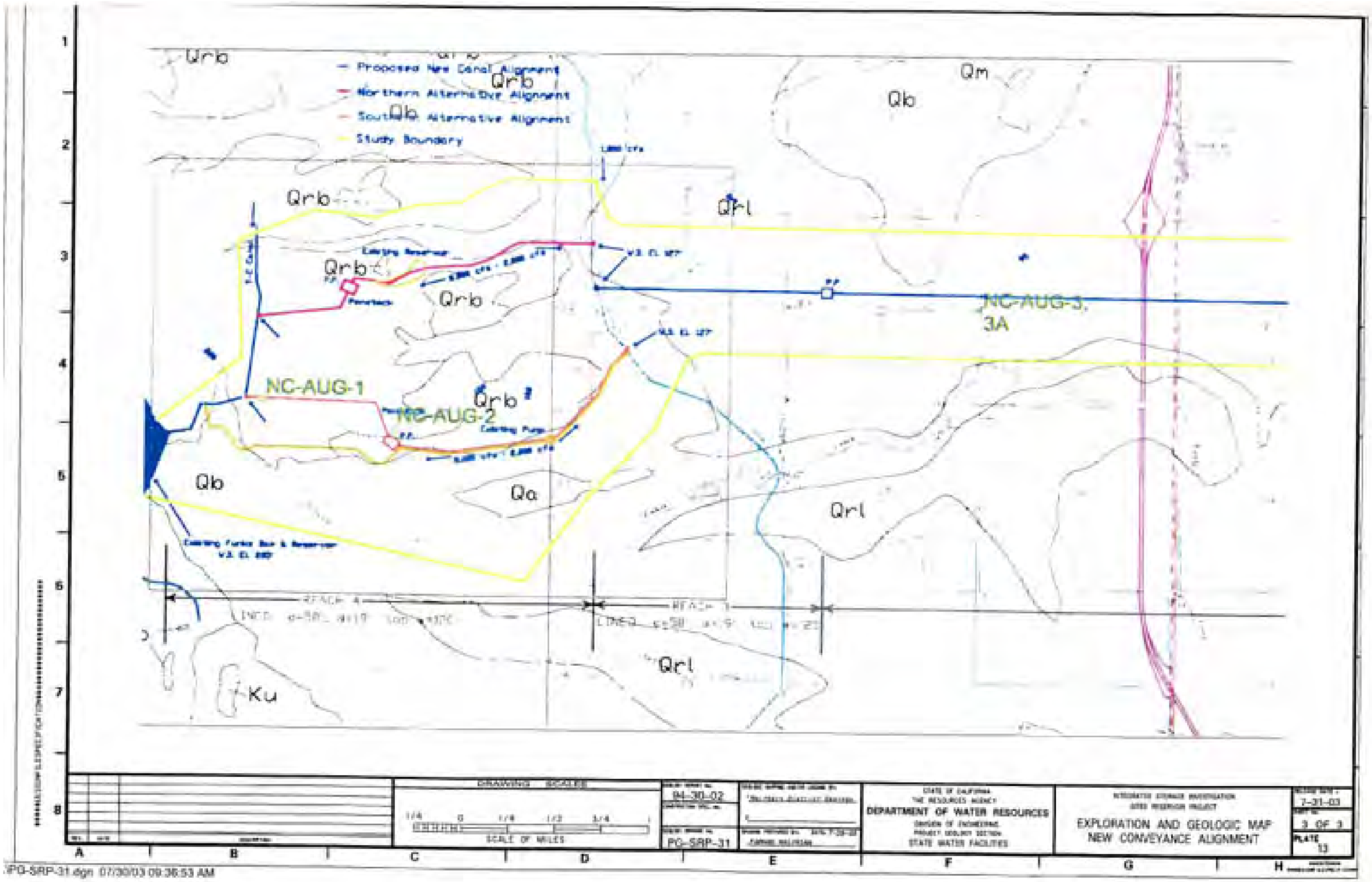


Figure B.2-14C. Exploration and Geologic Map – New Conveyance Alignment Project Area

Along this 13-mile alignment, groundwater was encountered at relatively shallow depths in all of the auger holes, ranging from 5.5 to 9 feet below the surface. The shallow groundwater depths indicate that dewatering will be required for excavation of most of the conveyance alignment. Temporary slopes for the pipeline excavation in saturated soils should be no steeper than 1.5H:1V, but may require laying back to 2H:1V if instability is a problem.

B.2.8 Seismicity

General

This section describes faulting and seismicity for the project features under consideration for the proposed NODOS project. Information is summarized from *Project Geology Report No. 94-30-02*, dated July 2003, *Geologic Feasibility Report, Sites Reservoir Project, Appendix to Engineering Feasibility Report. Project Geology Report No. 94-30-02*, in turn, provided a general summary of the detailed fault and seismic hazard reports prepared by the Division of Planning and Local Assistance, Northern District, Geology Section, and by WLA. Discussions include findings from the 1999 Phase I, Fault and Seismic Hazards Investigation by Northern District, and the Seismotectonic Evaluation, Phase II Fault and Seismic Hazards Investigations conducted by WLA for the NODOS project. The study also included data from previous mapping and studies conducted by the USGS and Reclamation. The aforementioned detailed geologic reports are referenced at the end of this report.

Analysis of faulting and seismicity data for the two main dam sites (Sites and Golden Gate), the saddle dam sites, and all other project facilities show that displacement along Quaternary-age faults within the reservoir site could be activated by regional seismic sources such as the San Andreas or Great Valley faults.

Faulting

Faults that might have an effect on the proposed reservoir and structures can be categorized into regional and project site faults. Regional faults, such as the San Andreas fault, Cascadia Subduction Zone, and the Great Valley fault (also known as the Coast Ranges-Sierran Block Boundary Zone) are considered active seismic sources for earthquakes that could affect the project area.

The Great Valley thrust fault has been divided into two segments in the area of the project, the Funks Segment and the Bear Valley Segment (Figure B.2-15); both segments are blind thrusts with no surface exposure. A seismic event along one or more of the regional faults could result in sympathetic displacement on any of the project site faults.

Six faults have been mapped within the Sites Reservoir boundaries that could have a significant impact on one or more of the proposed structures (Figures B.2-3 and B.2-16). Two sets of surface faults have been mapped in the vicinity of the dam sites. The first set is characterized as northeast-striking, high angle faults that obliquely cut the north-striking bedrock units, and consistently displace stratigraphic contacts in a right-lateral strike-slip sense. This fault set includes the informally named S-2, GG-1, GG-2, and GG 3 faults, which traverse through or near the proposed Sites and

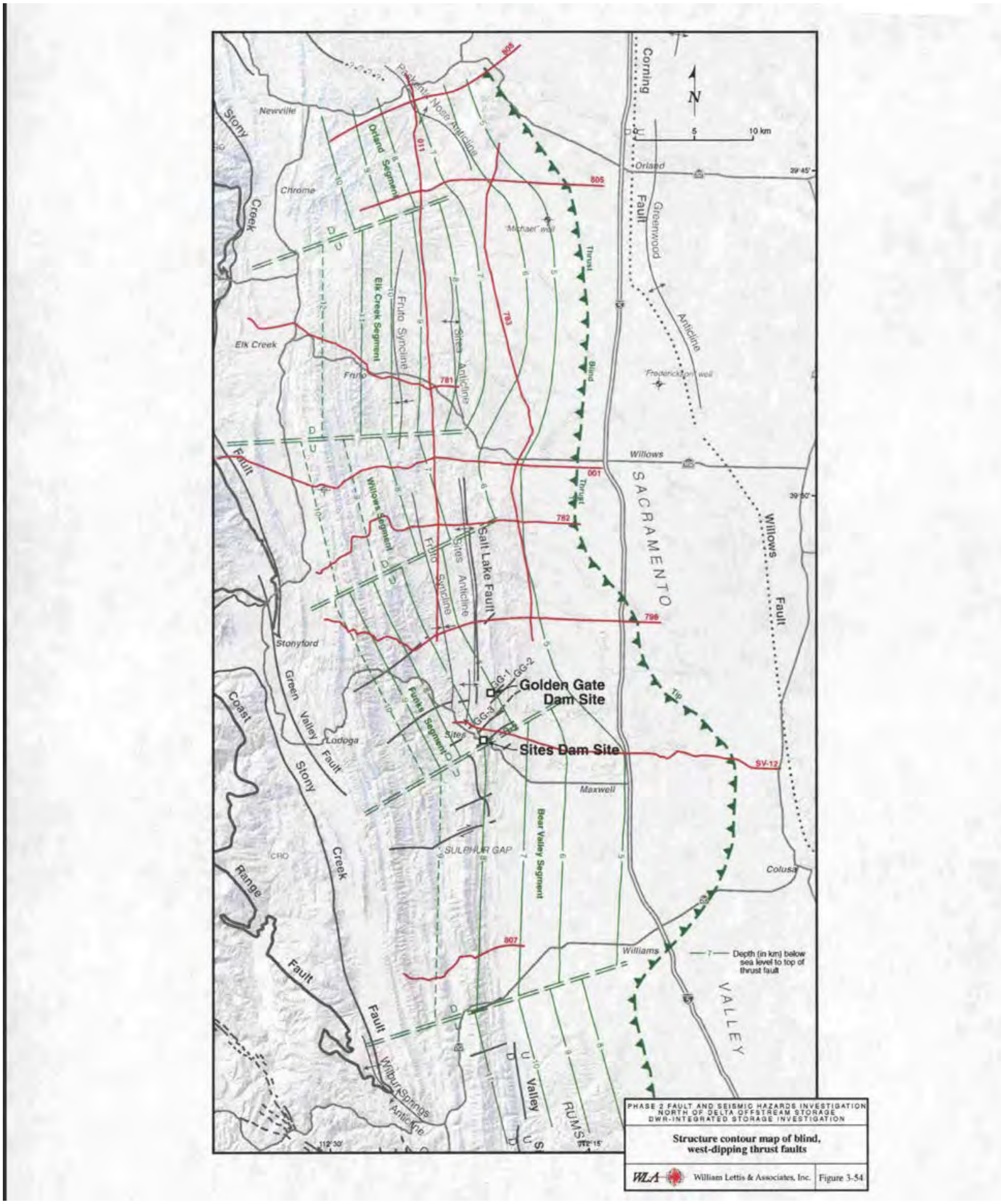


Figure B.2-15. Structure Contour Map of Blind West-Dipping Thrust Faults

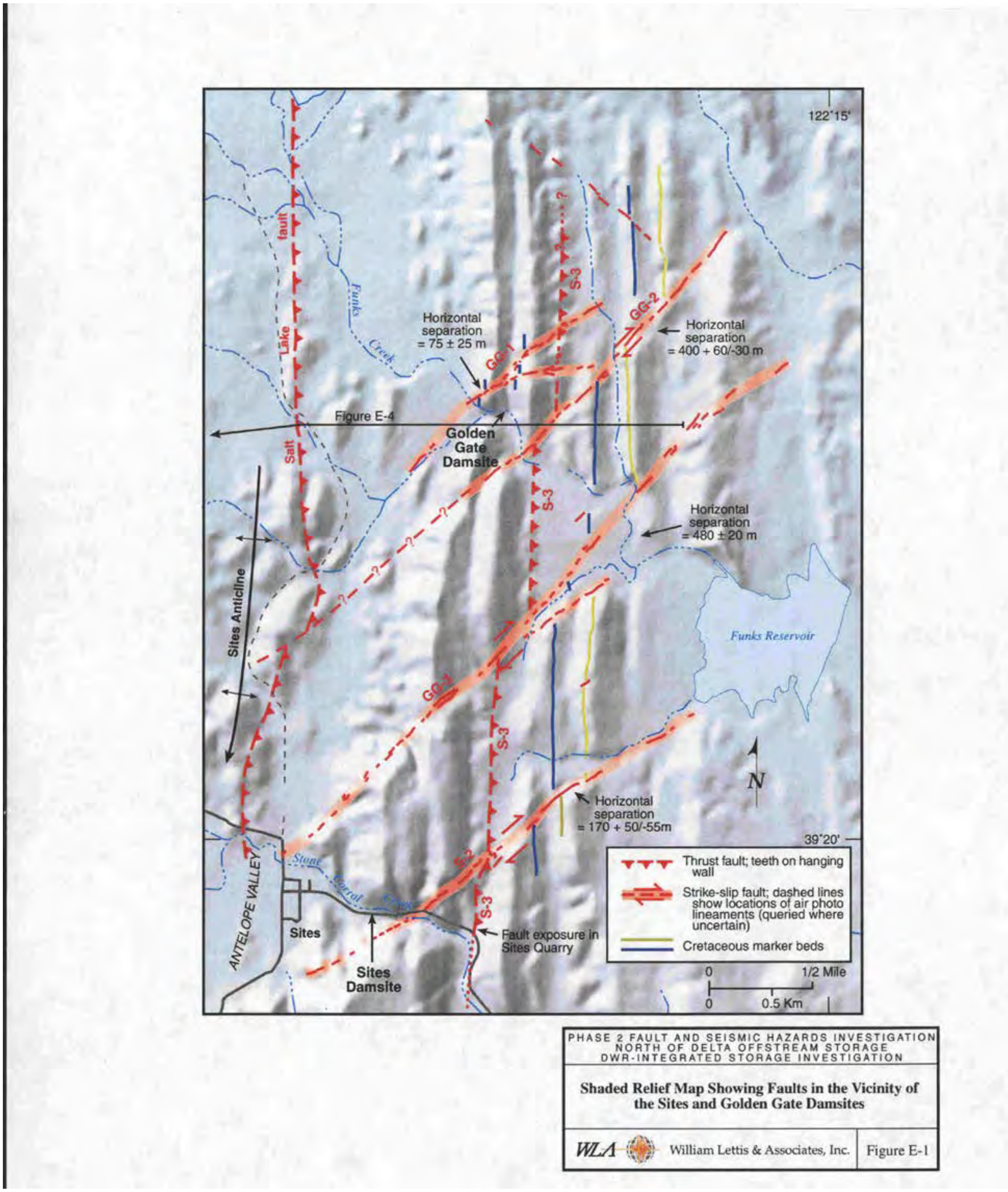


Figure B.2-16. Shaded Relief Map Showing Faults in the Vicinity of the Sites and Golden Gate Dam Sites

Golden Gate Dam sites. The second set of faults, characterized as north-striking structures that are generally parallel to bedding, include the east-dipping Salt Lake thrust and S-3 faults. The Salt Lake thrust fault lies within approximately 1 mile of both the Sites and Golden Gate Dam sites, while the S-3 fault has been mapped as passing through the inlet/outlet tunnel immediately west of the pumping plant site. WLA believes that the westward-dipping Funks Segment of the Great Valley fault underlies the surface faults and ramps up to and intercepting the Salt Lake thrust fault (Figure B.2-17). Faults S-2, GG-1, GG-2, and GG-3 are interpreted as tear faults associated with the Funks Segment southern structural boundary.

Exploration trenches have shown that surface rupture may occur along the Salt Lake fault during earthquakes on the Funks Segment of the Great Valley fault. The Funks Segment is considered the most likely seismic source for the NODOS project. Studies concluded that approximately 4.5 to 16 inches of reverse displacement might occur during a single surface rupturing event. Paleoseismic data supports only minor movement along the northeast-striking dextral or tear faults GG-1, GG-2, GG-3, and S-2. Analyzing the paleoseismic data with an assumed 3.3 feet of slip and a maximum magnitude earthquake (Mw 6.6) along the Funks Segment, and using three different fault movement models, WLA concluded that displacement along the tear faults would not exceed 8 inches, and is likely be lower (approximately 2.4 to 4 inches).

Seismicity

Historic regional and localized earthquake activity was analyzed by both DWR's Northern District and WLA. The purpose of the earthquake study was to identify potential seismic sources capable of generating significant ground motion at the project site, and to further characterize the seismotectonic setting of the area. Table B.2-1 lists potential regional seismic sources and include: the Bartlett Springs, Coast Range, and San Andreas fault zones to the west; the Cascadia subduction zone to the northwest; the Sierra Nevada range to the east; and the Great Valley fault beneath the proposed reservoir site. Moderate to strong earthquakes have been reported in northern California since the mid-1800s. Some of the more prominent events that probably shook the project area include the following: magnitude (M) 6.2 1898 Sonoma County; M 6.5 1898 Mendocino County; M 6.6 1954 Arcata; M 5.7 1940 Chico; M 6 1889 Antioch; three M 5.5 – 6.4 1892 Winters-Vacaville earthquakes; and the M 7.8 1906 San Francisco earthquake on the San Andreas fault zone. The Winters-Vacaville earthquakes of 1892 are of the most importance to this study, as they have been associated with a blind, west-dipping segment of the Great Valley fault (Figures B.2-18 through B.2-21).

The Funks and Bear Valley segments of the Great Valley fault are the closest seismogenic faults to the NODOS project that are considered capable of triggering surface displacement at one or more of the proposed structures. Indirect evidence of late Quaternary activity along the Funks segment includes:

- Paleoseismic trenching studies of the Salt Lake thrust fault have shown that at least one, and probably three or more, surface ruptures have occurred in the past 30,000 to 70,000 years; the Salt Lake thrust fault has been interpreted by WLA as terminating down-dip against the Funks segment of the Great Valley fault.

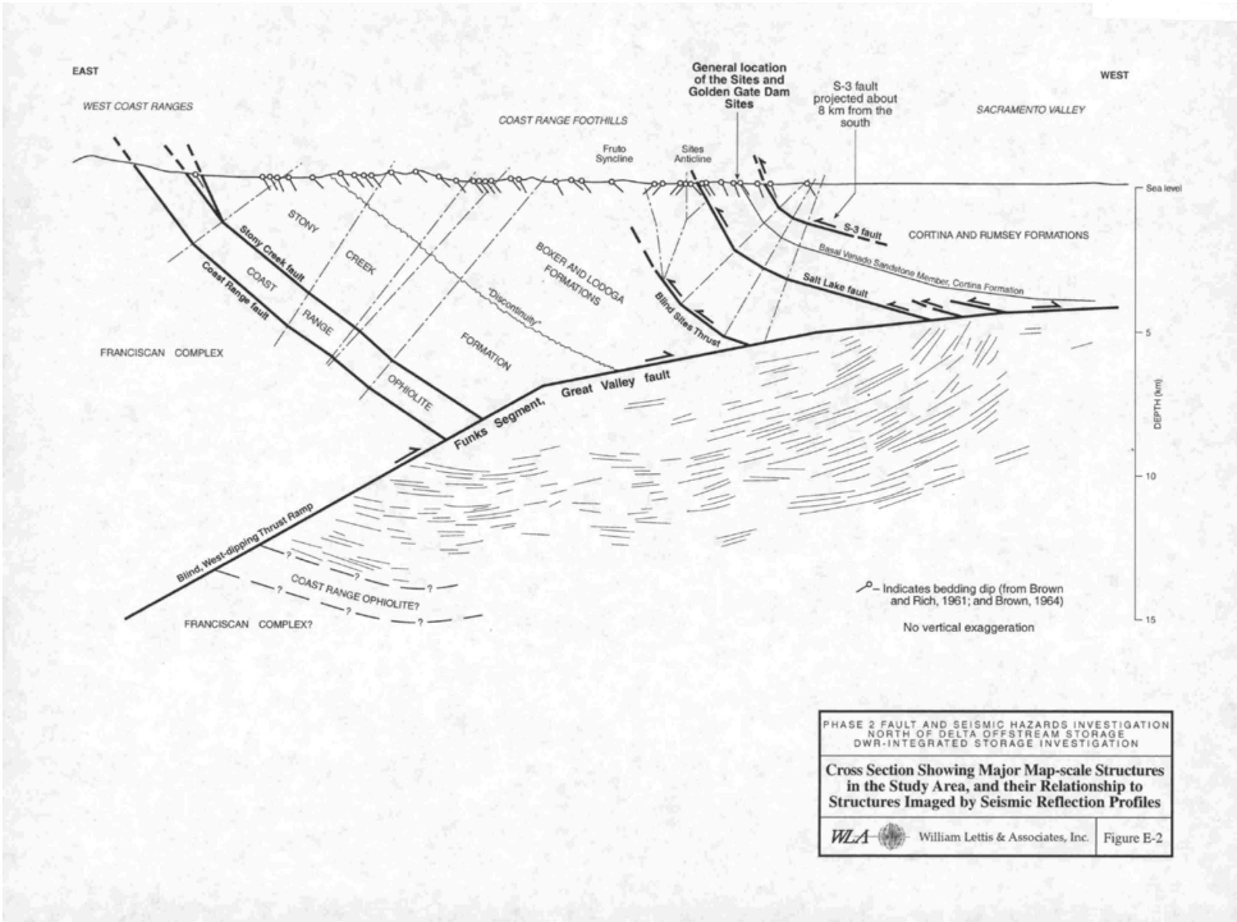


Figure B.2-17. Cross-Section Showing Major Map Scale Structures in the Study Area Reflection Profiles.

Table B.2-1. Faults and Geologic Structures Investigated in this Report

Fault Name	Fault Type	Activity	Seismic Source Characterization				Closest Approach		Comments
			Fault Length	Fault Width	Rupture Area	Maximum Magnitude	Sites	Golden Gate	
GG-1	Right Lateral, Strike Slip	No Holocene Activity	1.1 mi (1.8 km)	Not a seismic source			3.1 mi (4.1 km)	<0.5 mi (<1 km)	GG-1, GG-2, GG-3 and S-2 are interpreted to be shallow tear faults along Funks/Bear Valley Segment boundary. Conservatively assumed to be sources of aftershocks. Possible surface-rupture hazards.
GG-2	Right Lateral, Strike Slip	No Holocene Activity	3.7 mi (5.9 km)	Faults GG-2, GG-3 are S-2 are considered potential sources of shallow aftershocks. Maximum earthquake magnitude for these structures is M_w 5.4.			1.7 mi (2.3 km)	<0.5 mi (<1 km)	
GG-3	Right Lateral, Strike Slip	No Holocene Activity	3.0 mi (4.8 km)				0.4 mi (0.7 km)	<0.5 mi (<1 km)	
S-2	Right Lateral, Strike Slip	No Holocene Activity	2.4 mi (3.9 km)				<0.5 mi (<1 km)	2.2 mi (3.5 km)	
Salt Lake Fault	Thrust	Multiple Late Quaternary Surface Ruptures	12 mi (20 km)	Not a seismic source			1.5 mi (2.4 km)	1.7 mi (2.7 km)	Interpreted to accommodate triggered, aseismic slip
S-3	Thrust	No Holocene Activity	≥4.25 mi (6 km)	Not a seismic source			0.9 mi (1.5 km)	600 ft (200 m)	May accommodate triggered aseismic slip
Funks Segment, Great Valley Fault	Blind Thrust	Late Quaternary Activity	11 mi (17 km)	14 mi (22 km)	146 mi^2 (374 km^2)	M_w 6.6	4.0 mi (6.5 km)	3.6 mi (5.8 km)	Indirect evidence of late Quaternary activity
Bear Valley Segment, Great Valley Fault	Blind Thrust	Assumed to be Active	14.4 mi (23 km)	14.4 mi (23 km)	207 mi^2 (529 km^2)	M_w 6.8	4.8 mi (7.7 km)	4.4 mi (7.0 km)	Conservatively assumed to be active
San Andreas Fault	Strike Slip	Active	650 mi (1,050 km)	Maximum magnitude = M_w 8 (WLA, 1997)			70 mi (113km)	70 mi (113 km)	Assumes maximum earthquake will rupture 272 mi (435 km)
Maacama Fault	Strike Slip	Active	84 mi (135 km)	Maximum magnitude = M_w 6.5 (WLA, 1997)			45 mi (72 km)	45 mi (72 km)	Too far from sites to dominate hazard
Bartlett Springs Fault	Strike Slip	Active	70 mi (113 km)	9.4 mi (15 km)	117 mi^2 (306 km^2)	M_w 6.6	20 mi (32 km)	22 mi (32 km)	Maximum rupture length of proximal Coyote Rocks Segment is 11 mi (18 km)
Stony Creek Fault	Thrust (?)	Not Active	63 mi (100 km)	Not characterized			10 mi (16 km)	11 mi (18 km)	Interpreted to be a deformed Mesozoic fault

Table B.2-1. (Continued)

Fault Name	Fault Type	Activity	Seismic Source Characterization				Closest Approach		Comments
			Fault Length	Fault Width	Rupture Area	Maximum Magnitude	Sites	Golden Gate	
Coast Range Fault	Normal	Not Active	Not a continuous fault trace	Not characterized			12.4 mi (20 km)	12.4 mi (20 km)	Interpreted to be a deformed Mesozoic fault
Green Valley Thrust Fault and related faults	Thrust	Not Active	11 mi (17 km)	Not characterized			8 mi (12.5 km)	9 mi (15 km)	Bedding-parallel thrust fault confined to the upper 3 mi (5 km) of the crust
Paskenta Fault	Normal	Not Active	28 mi (45 km)	Not characterized			25 mi (41 km)	23 mi (37 km)	Interpreted to be a deformed Mesozoic fault
Rumsey Hills Fault	Blind Thrust	Active	16 mi (25 km)	Not characterized			28 mi (45 km)	30 mi (49 km)	Too far from dam sites to dominate hazard
Sweitzer Fault	Thrust	Active	11 mi (17 km)	Not a seismic source			28 mi (45 km)	30 mi (49 km)	May accommodate triggered aseismic slip
Valley Side Fault	Thrust/Reverse	Active	10.5 mi (17 km)	Not a seismic source			16 mi (26 km)	18.6 mi (30 km)	May accommodate triggered aseismic slip
Black Butte Fault	Bedrock Escarpment	Not a Fault	10.5 mi (17 km)	Not characterized			30 mi (48 km)	27 mi (44 km)	
Southern Reach, Corning Fault	Oblique-Reverse	Active	13 mi (21 km)	13 mi (21 km)	182 mi^2 (462 km^2)	M_w 6.7	20 mi (32 km)	18 mi (29 km)	Associated with clusters of seismicity
Cascadia Subduction Zone	Megathrust Fault	Active	620mi (1,000 km)	74.5 mi (120 km)	46,310 mi^2 (120,000 km^2)	M_w 9	100 mi (160 km)	100 mi (160 km)	Geologic evidence for giant Cascadia earthquakes
Intraplate (Gorda slab) Faults	Probable Strike Slip	Active	Source dimensions not directly observed; maximum magnitude adopted from empirical observations			M_w 7.5	53 mi (85 km)	53 mi (85 km)	1922 Gorda plate earthquake estimated to be M_s 7.6

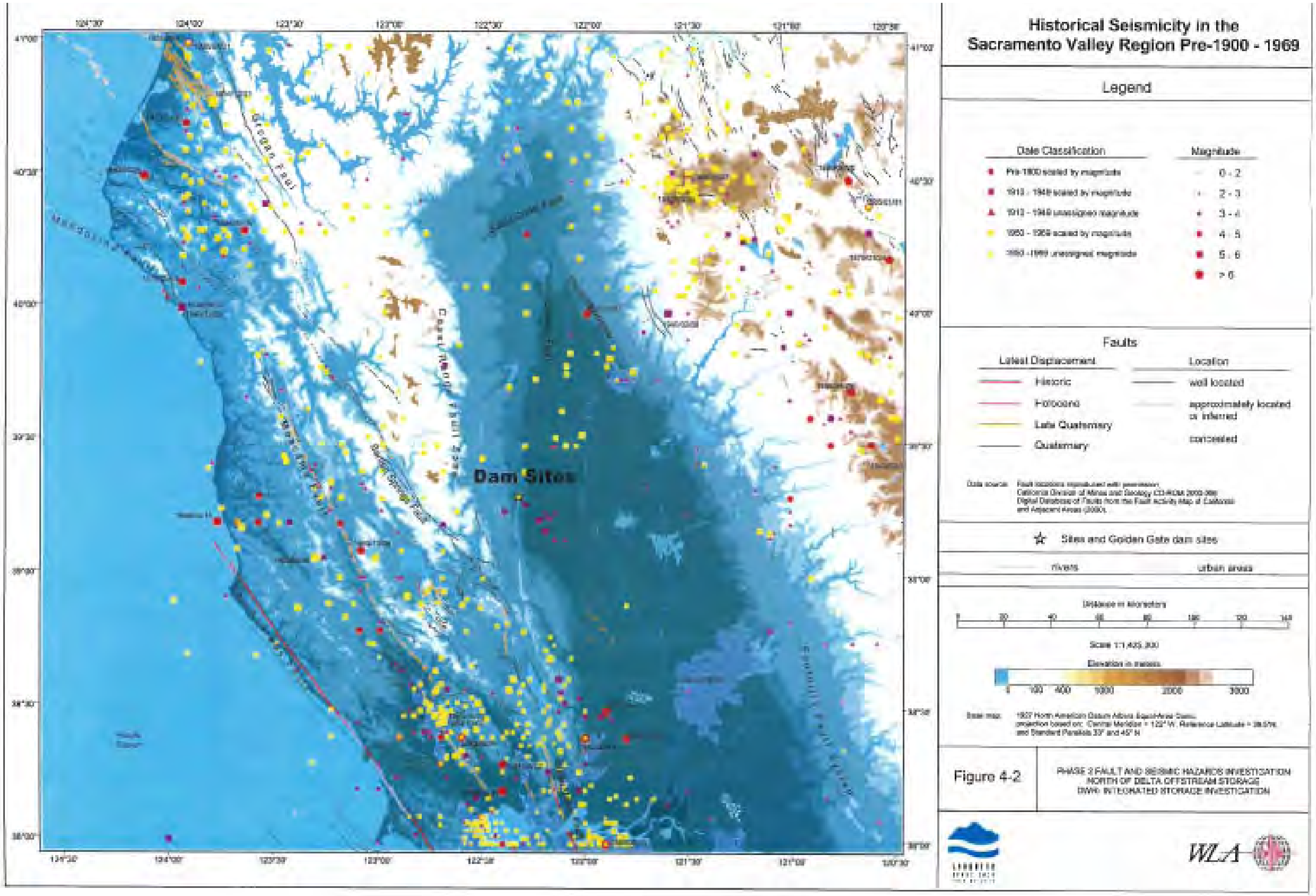


Figure B.2-18. Historical Seismicity in the Sacramento Valley Region, Pre-1900 – 1969

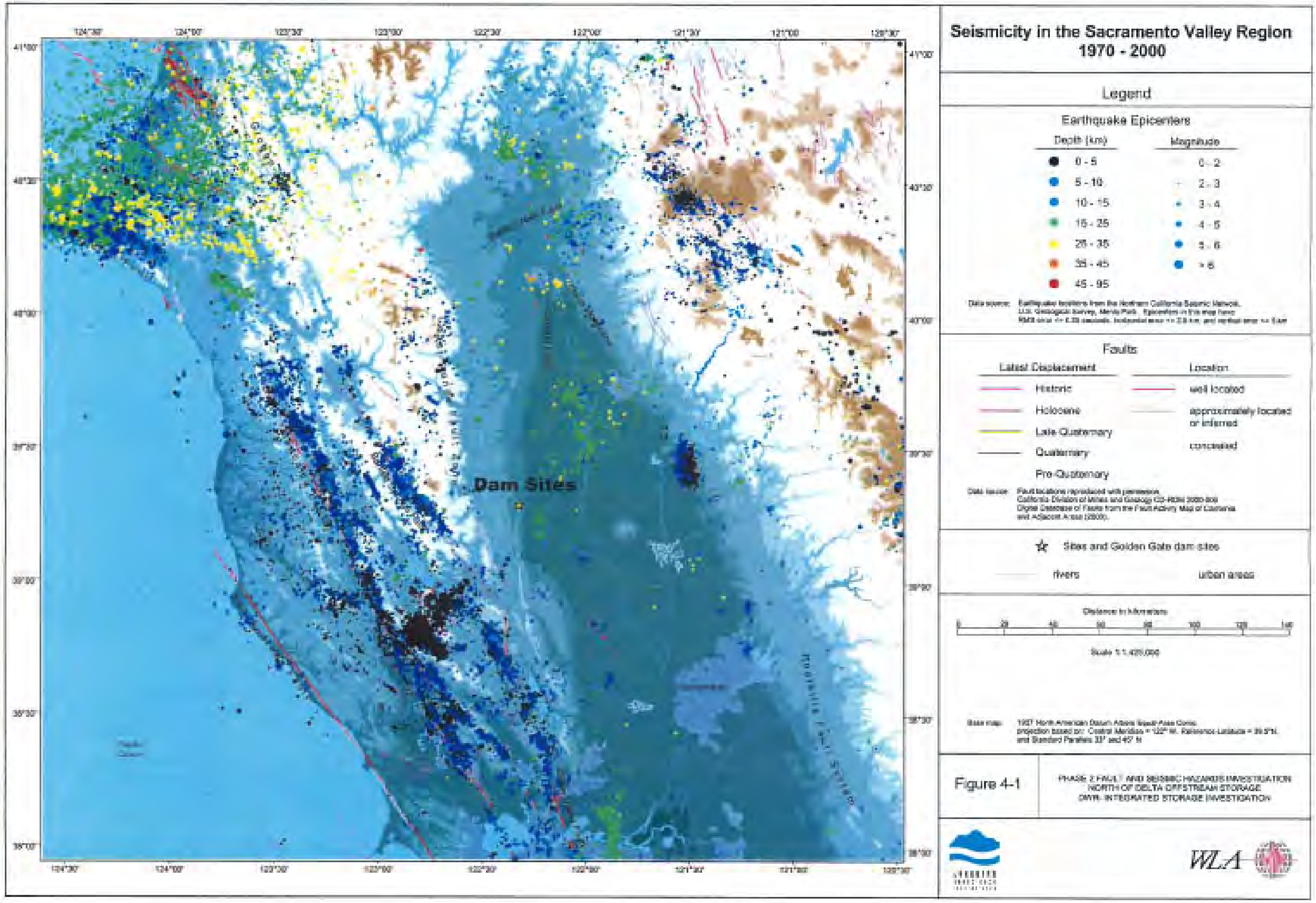


Figure B.2-19. Seismicity in the Sacramento Valley Region, 1970 – 2000

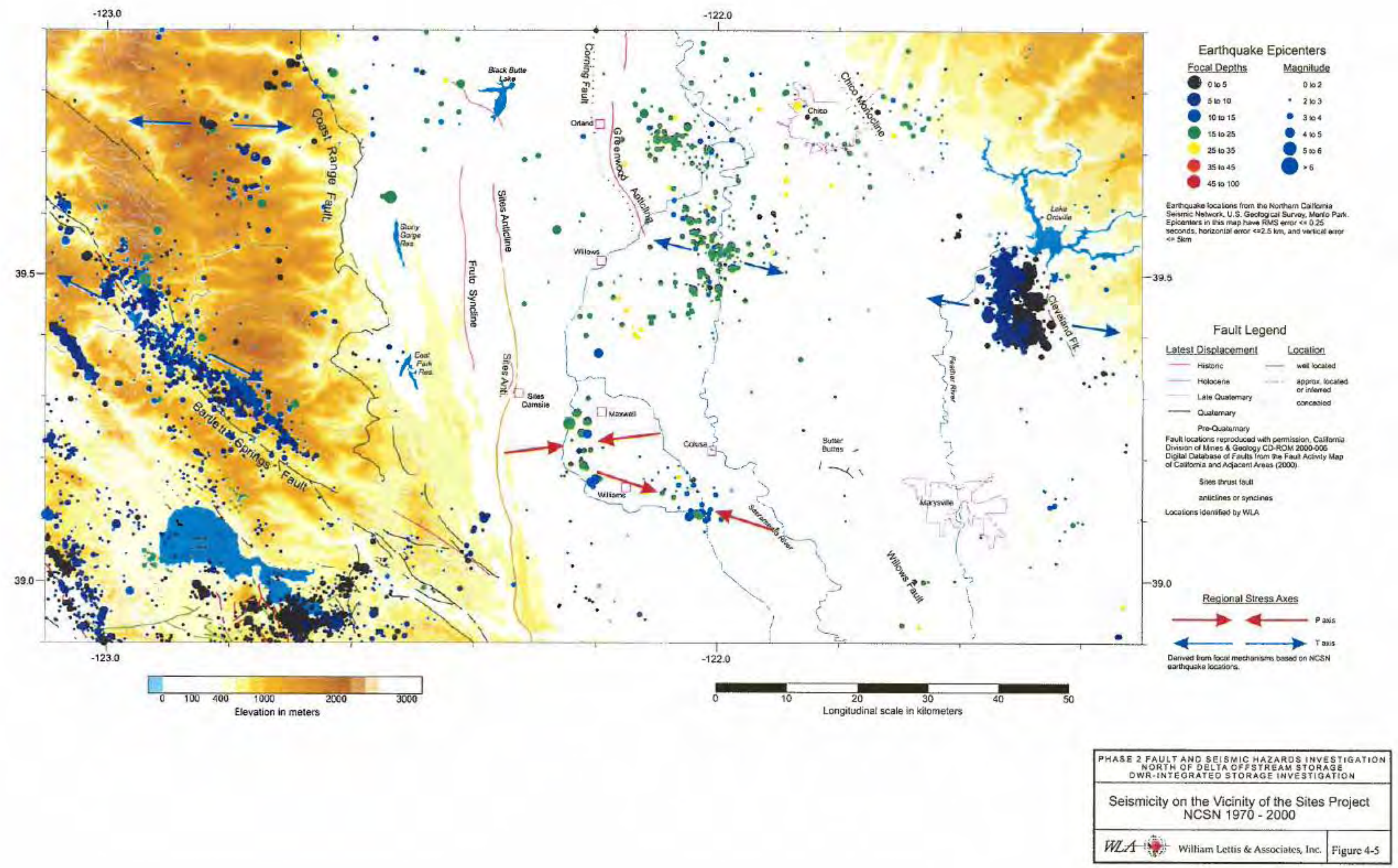


Figure B.2-20. Seismicity in the Vicinity of the Sites Project – Northern California Seismic Network, 1970 – 2000

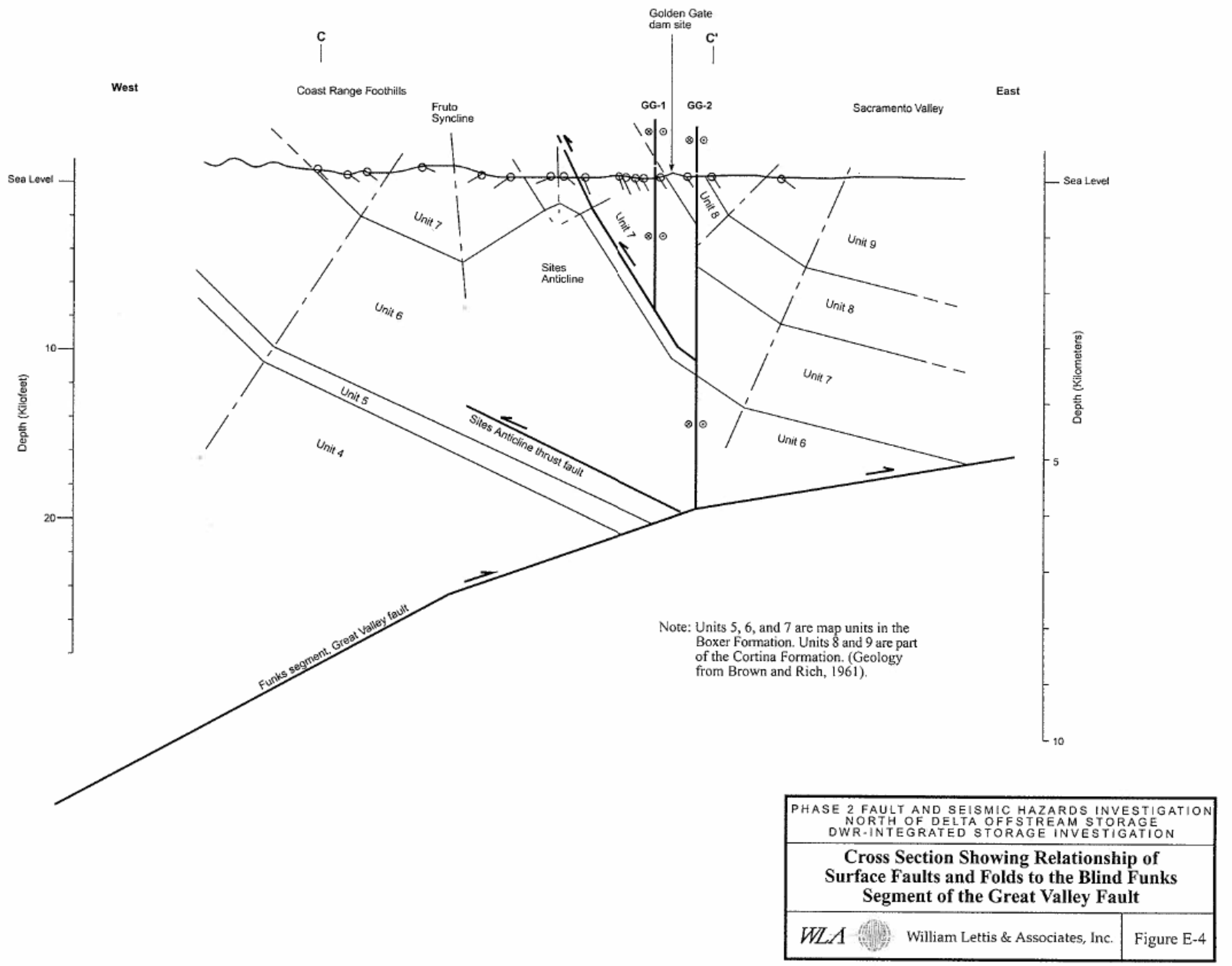


Figure B.2-21. Cross-Section Showing Relationship of Surface Faults and Folds to the Blind Funks Segment of the Great Valley Fault

- Morphometric analysis of regional topography showing evidence of localized late Quaternary uplift coincident with the Sites anticline and Salt Lake thrust fault, both of which are located above the Funks segment.

- Morphometric analysis of stream drainages across Coast Range foothills showing localized fluvial incision and channel morphology consistent with active surface uplift above the Funks segment.

Locally, the northeast-striking tear faults (S-2), GG-1, GG-2, and GG-3 have been interpreted to terminate downward against the Funks segment. It is thought that the tear faults move sympathetically during large magnitude earthquakes on the Funk segment thrust ramp, and do not behave as independent seismic sources. However, WLA has concluded that the northeast-striking faults may be a source of aftershocks following an earthquake on the Funks or Bear Valley segments of the Great Valley fault. Calculations and seismic reflection data analyses by WLA show that the tear faults would have a maximum rupture depth of 3.1 miles, and maximum earthquake magnitudes of Mw 5.3 to 5.4; the rupture depth for GG-1 was calculated at 1 to 2 miles, based on its short surface trace. Maximum surface displacement along the northeast-striking tear faults were calculated to range from 2.4 to 8 inches. WLA's ground motion analysis, using methods by Abrahamson and Silva (1997) and Sadigh et al. (1997), accounting for fault rupture directivity, determined that a peak ground acceleration of 0.8 g (acceleration due to gravity) would be generated from a maximum credible (MCE) of Mw 6.8 on the Bear Valley segment of the Great Valley fault. Analytical results may be slightly less for individual sites and structures within the project area (Figures B.2-22 and B.2-23).

A similar study was not conducted for the Bear Valley segment of the Great Valley fault south of the reservoir site. However, WLA concluded that the Bear Valley segment is active based on its location within a recognized zone of late Cenozoic tectonic activity and between two other active segments of the Great Valley Fault (Funks segment to the north and the tectonically active Rumsey Hills-Dunnigan Hills region to the south). The Bear Valley segment is considered the controlling seismic source for both major dam sites within the project area, based on comparative earthquake response spectra within a 31-mile radius of the reservoir site. Analyses of surface geologic structures and seismic reflection data indicate that the Bear Valley segment is 14 miles long, 3 to 6 miles deep, strikes north-south, and dips approximately 21º west, has a rupture width of 14 miles, and rupture area of 207 square miles. Further analysis by WLA shows an MCE, or design basis earthquake, magnitude of Mw 6.8 located 4.8 and 4.4 miles, respectively, from the Sites and Golden Gate Dam sites (Figures B.2-20 and B.2-21).

Reservoir-Triggered Seismicity

Reservoir-triggered (or reservoir-induced) seismicity, which is the triggering of earthquakes by the physical processes associated with the impoundment of reservoirs, is considered hazardous to many large dams. Because of the potential for damage, reservoir-induced seismicity should be considered in the design of most dams.

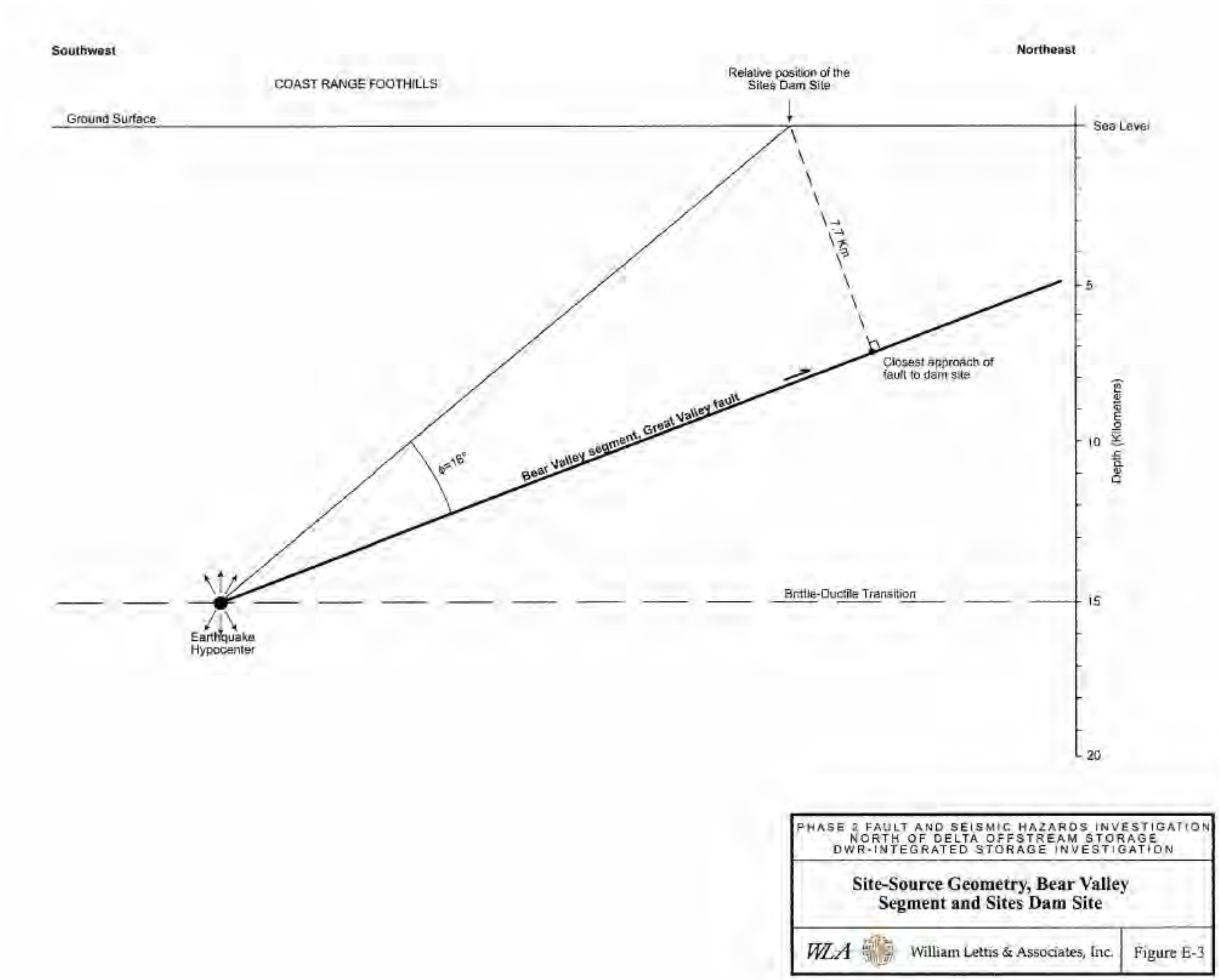


Figure B.2-22. Site-Source Geometry, Bear Valley Segment and Sites Dam Site

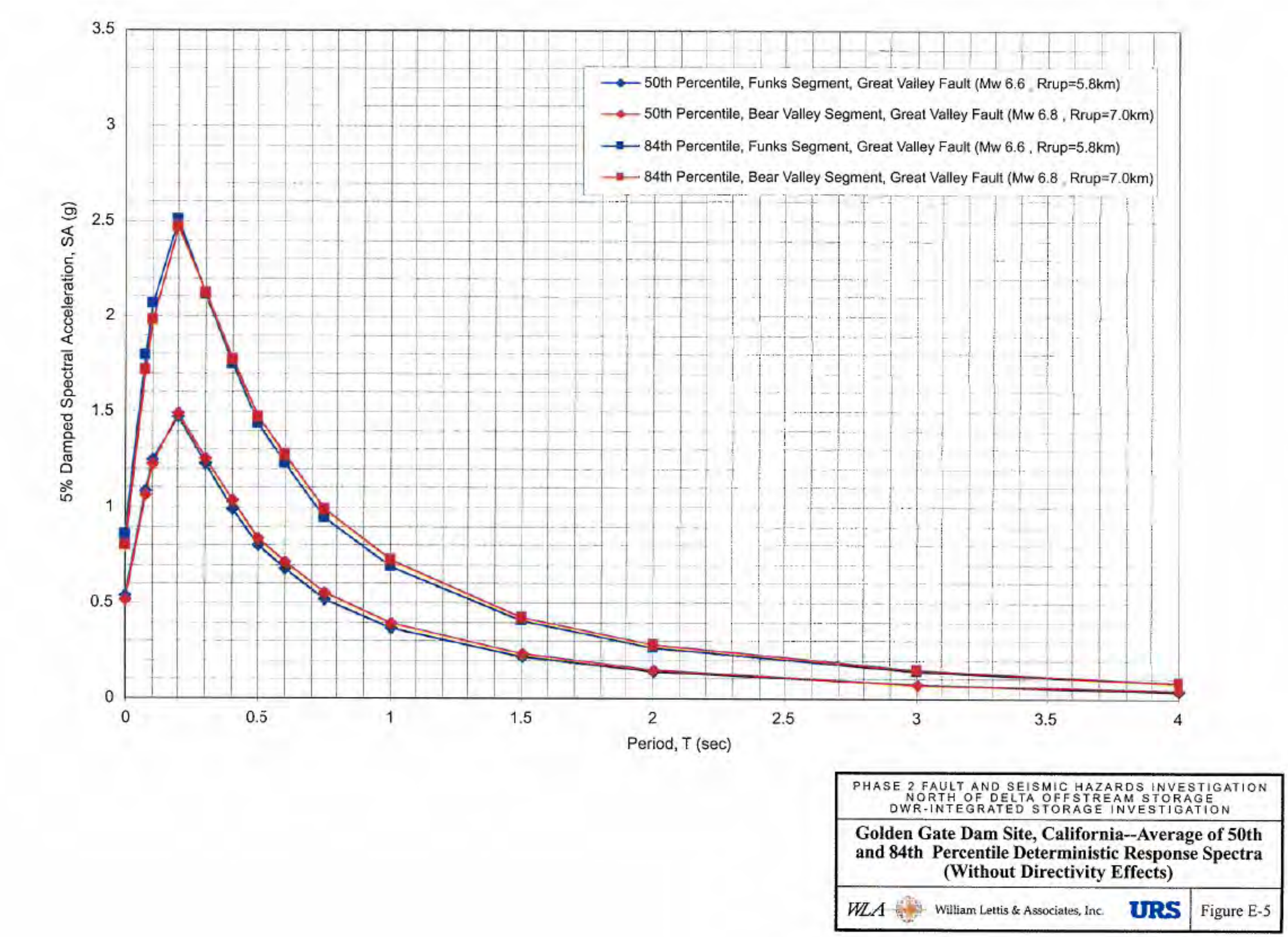


Figure B.2-23. Bear Valley and Funks Segments, California – Average of 50th and 84th Percentile Deterministic Response Spectra (without Directivity Effects)

While reservoir-triggered seismicity has been intensely studied, it is not clear what physical mechanism is responsible for triggering an earthquake, and under what circumstances. A correlation, however, has been observed between dam height and increased seismicity. Two mechanisms are thought to act in triggering fault movement, either in concert or independently. Those mechanisms are a change in crustal elastic stress as the reservoir goes through fill and release cycles, and an increase in fluid pore pressure at the depths that earthquakes originate; the increased pore pressure at depth reduces the effective stress acting on the fault plane.

Earthquakes triggered from reservoir impoundment have been identified both worldwide, and within California. The largest known reservoir-triggered earthquake was a magnitude Mw 6.3, occurring in western India in December 1967. Several possible cases of reservoir-triggered seismicity have been evaluated in northern California. The most prominent of which is the ML 5.7 Oroville earthquake on August 1, 1975, with an epicenter 31 miles east of the Sites Reservoir project.

Given the geologic and tectonic setting of the Sites Reservoir site, the likelihood of reservoir-triggered seismicity is reduced relative to other northern California reservoirs, including Lake Oroville. The general reasons for this are:

- The reservoir site is located within a zone of active compression where the maximum principal stress is expected to be horizontal.

- Bedrock fracture permeability is expected to be less in this region of folding and thrusting, reducing the probability of pore fluid pressure fluctuation.

B.2.9 Construction Materials

MATERIALS INVESTIGATIONS

Construction materials, for use in embankment dams and levee protection, have been investigated in the NODOS project area since the 1960s. Previous materials investigations have been performed by the Reclamation and the USACE and were reviewed as part of the current DWR investigation. Reclamation material investigations in 1964 and 1980 included an evaluation of material sources for the construction of dams at the Sites Reservoir site. The USACE investigations focused on evaluating the suitability of Venado sandstone for use as riverbank protection projects on the Sacramento River. The USACE investigations primarily consisted of sampling and testing Venado sandstone at quarry sites downstream of the proposed Sites Dam and were not formally documented in a report. In addition, an extensive evaluation of the shear strength properties of the Venado sandstone is presented in a University of California, Berkeley report published by Becker et al. in 1972.

Because material requirements for the Sites Reservoir dams constitute a major component of the overall NODOS project, the primary focus of the preliminary materials investigation program was to identify and evaluate materials sources for construction of the proposed dams. The current DWR construction materials investigation program consisted of an assessment of available on-site and off-site material sources, laboratory testing, and an evaluation of the suitability and engineering properties of the available materials. The assessment included a review of published data, field investigations, and material sampling with materials testing performed at DWR's Soils and Concrete Laboratory. An evaluation of the engineering properties and suitability of the available materials for construction of the proposed dams is documented in the report *North-of-the-Delta Offstream Storage Investigation, Sites Reservoir Feasibility Study, Materials Investigation, Testing, and Evaluation Program* (DWR, 2002).

Material Sources

General

The construction materials investigation program examined the following materials available in or near the proposed Sites Reservoir project area: alluvial deposits (Recent and older alluvium), and Venado sandstone of the Cortina Formation (fresh and weathered).

Mudstone of the Boxer Formation

These material sources were investigated, tested, and evaluated to examine their suitability for use as the following types of construction materials: impervious core; rockfill and riprap; random fill; filter, drain, and transition; and concrete aggregate.

Impervious Core

A large amount of potential impervious material exists within or near the Sites Reservoir project. Previous studies by Reclamation identified four main areas of alluvial deposits in the reservoir area encompassing roughly 36 million cubic yards of material. Figure B.2-24 illustrates the extent of these deposits. Additional impervious materials are located within required excavation areas for the appurtenant structures and Funks Reservoir enlargement. These required excavation areas would be utilized to the maximum extent practicable. Additional quantities of impervious materials are located in potential borrow sites located within 1 mile of each of the 11 dam sites. Figure B.2-24 illustrates the locations of these potential borrow areas. The impervious materials are suitable for use in the proposed embankment dams, and are generally classified as low to medium plasticity clays, with lesser amounts of high plasticity clays, and clayey sands.

Rockfill and Riprap

The best available source of clean rockfill material within the project area is fresh Venado sandstone. Sandstone quarry areas have been identified within 1 mile of both the Golden Gate and Sites Dam sites, and are presented in plan view in Figure B.2-24. Sufficient quantities of fresh sandstone for rockfill material can be obtained from these quarries to construct the proposed embankment dams. Future design investigations should also evaluate the advantages and disadvantages of developing one centrally located quarry for both Golden Gate and Sites Dams instead of developing a quarry for each dam.

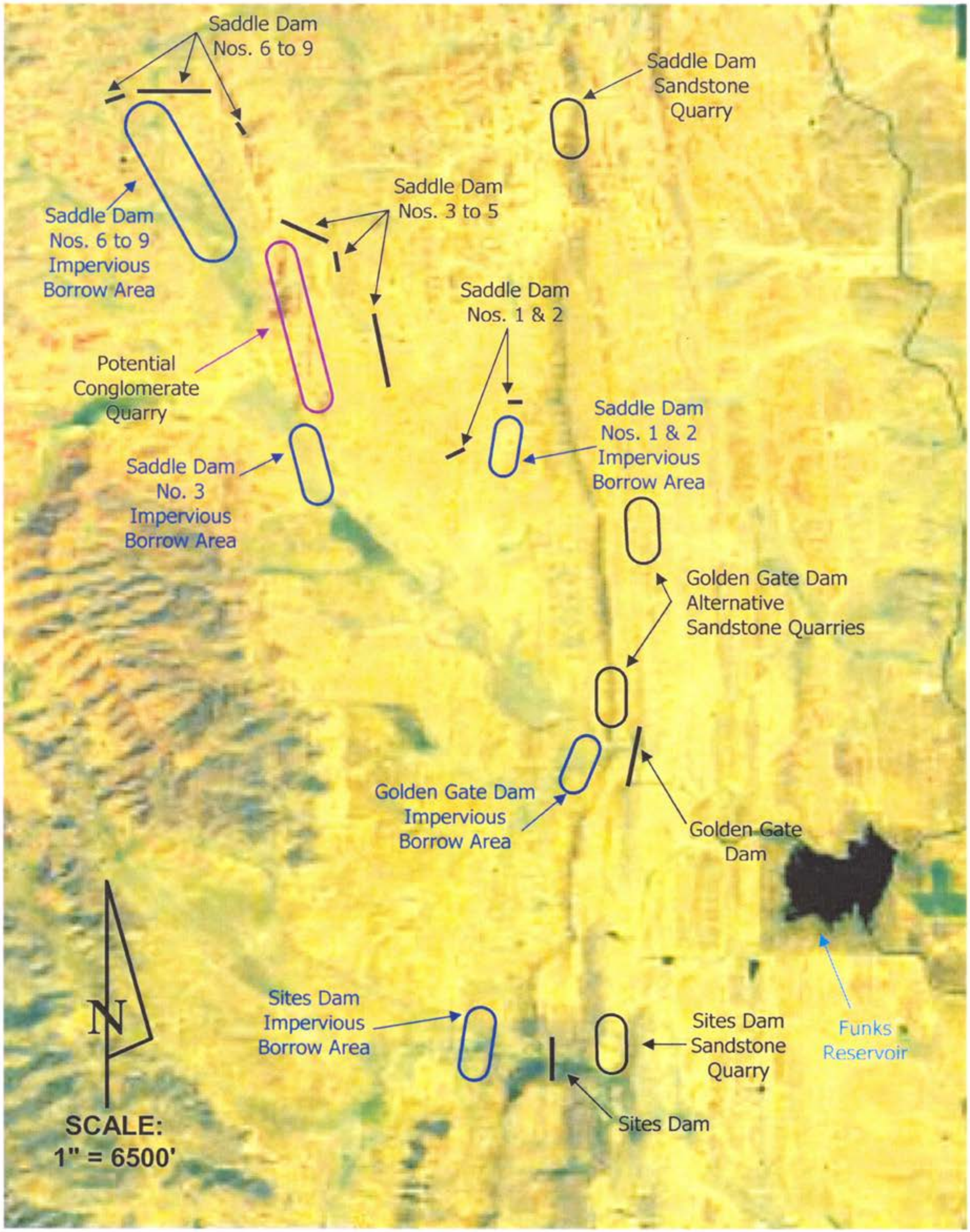


Figure B.2-24. Sites Reservoir – Proposed Impervious Borrow Areas and Quarries

Figure B.2-24 also presents a proposed sandstone quarry location for construction of the saddle dams. The haul distance from this proposed quarry is roughly 3 to 4 miles from the saddle dam sites. A potential alternate source of rockfill and riprap material for construction of the saddle dams is a ridge of conglomerate located within the reservoir area near Saddle Dam 3 (Figure B.2-24). Although not evaluated as part of the materials investigation program, this potential rockfill source offers a shorter haul distance to the saddle dams (1 to 2 miles). This rockfill source would cause less environmental impacts in comparison to the proposed sandstone quarry, because the ridge of conglomerate is located within the reservoir area and the potential sandstone quarry is not. Because the suitability of the conglomerate cannot be confirmed at this time, it was assumed that development of the sandstone quarry would be required for construction of the saddle dams.

Random

It is anticipated that two general types of random materials would be generated during construction depending upon the source of the material. One type of random material would be comprised of predominately weathered sandstone from the Cortina Formation, while the other type would be predominately mudstone from the Boxer Formation. Mudstone from the Boxer Formation would tend to be “soil like” after excavation and compaction operations, as it is a low-strength rock and has a propensity to break down when exposed to air and water. The weathered Cortina Formation would tend to be a dirty rockfill.

At the Sites and Golden Gate Dam sites, random embankment material would be comprised of materials unsuitable for use as clean rockfill. Materials would consist of weathered sandstone, mudstone, slopewash, etc., from excavations for the dam foundations, appurtenant structures, and rockfill quarries. Material from clearing and grubbing operations would not be used in any embankment structure. Random material generated during construction of these dams would have haul distances of less than 1 mile.

Random material would be generated from the Boxer Formation during construction of the saddle dams and designated borrow areas. Random material borrow areas for construction of the saddle dams have not been identified, but would be located within the reservoir area with haul distances of less than 1 mile. Sufficient quantities are available for construction of the saddle dams. Although the Boxer Formation material would function more as an upstream and downstream shell zone in the saddle dam sections, the term “random” is used for this material zone to be consistent with the terminology used at Sites and Golden Gate Dams.

Filter, Drain, and Transition Materials

Deposits of sand and gravel of sufficient quantity for construction of the Sites Reservoir dams are not available in the project area. Therefore, alternative sources of filter, drain, and transition materials were examined as part of the preliminary materials investigation. Laboratory testing indicated that crushed, fresh Venado sandstone may be suitable as filter, drain, and transition materials. However, it was not extensively tested as part of the materials investigation because a particle breakage evaluation would have been required. This particle breakage evaluation will require test quarries and fills and was considered beyond the scope of the preliminary investigations. Because the suitability of the Venado sandstone cannot be confirmed at this time, it was assumed that filter, drain, and transition materials for the proposed embankment dams would be imported from the closest off-site sand and gravel deposit. This off-site deposit was identified as an old channel on Stony Creek, between Orland and Willows (Figure B.2-25). The channel is approximately 30 to 35 road miles from the project area, and has an estimated material availability of 160 million cubic yards, far exceeding the construction requirement.

Concrete Aggregate

Similar to the approach used for filter, drain, and transition materials, crushed Venado sandstone and off-site sand and gravel deposits were examined as potential sources of concrete aggregate. Preliminary testing performed on crushed samples of Venado sandstone indicates that it marginally meets concrete aggregate suitability criteria. Verification of the suitability of the Venado sandstone for use as concrete aggregate would be the focus of future investigations.

Construction Water

Construction water will be obtained from Funks Creek and Stone Corral Creek supplemented by off-site sources if required. No water quality testing has been performed on these sources to date.

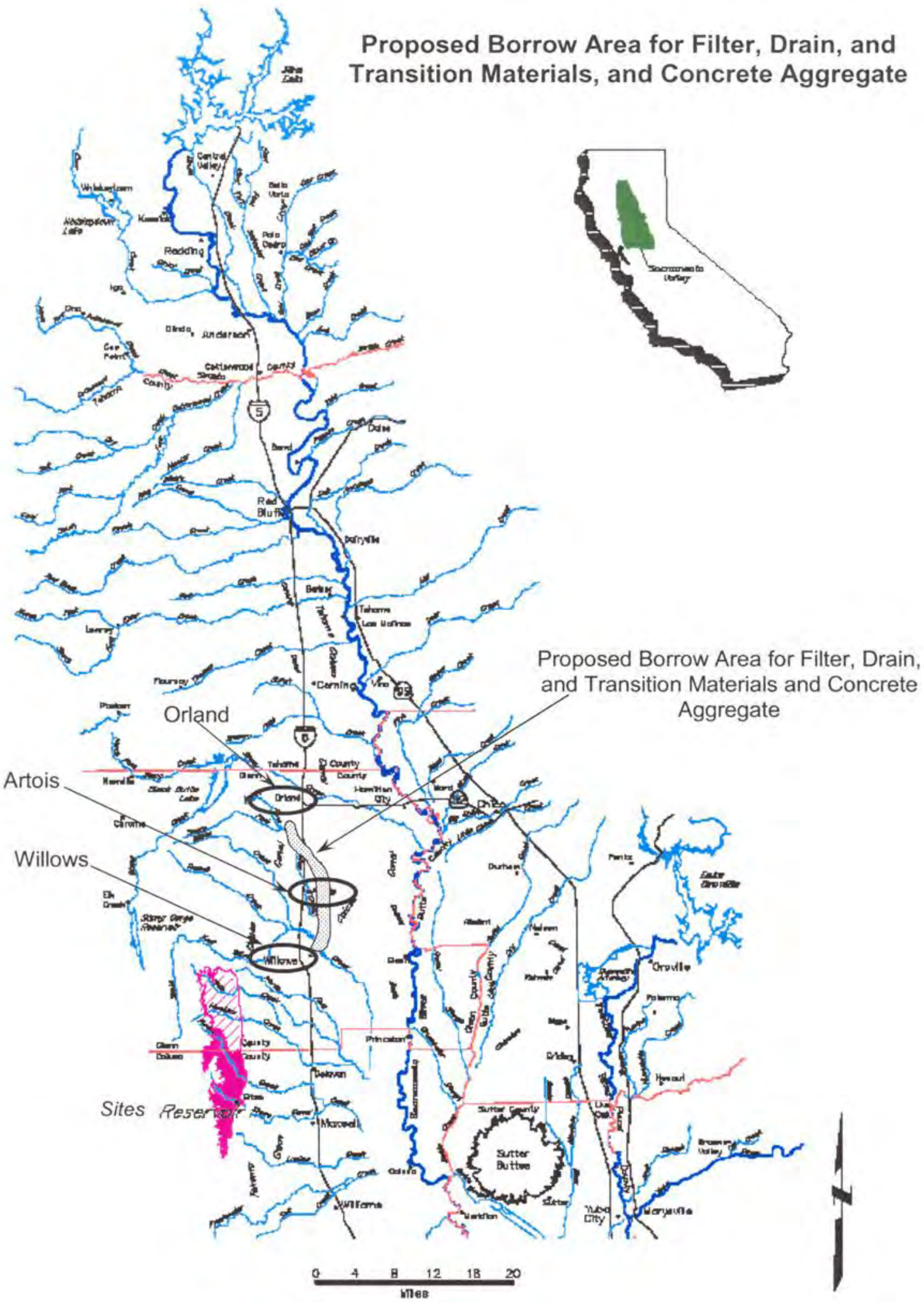


Figure B.2-25. Proposed Borrow Area for Filter, Drain, and Transition Materials, and Concrete Aggregate

B.3 DESIGN CONSIDERATIONS

B.3.1 General

This section of Appendix B describes the engineering aspects of the alternative features currently under consideration for the proposed NODOS project. This section includes a general summary of the PFR, and the preliminary feasibility reports prepared by DWR's DOE and its consultants, and the Technical Memoranda (TM) developed post-PFR. Additional information regarding the technical analyses and designs is available in the references listed in Section B.5 of this appendix. Currently, three project alternatives are being considered, as outlined in Table B.3-1.

Table B.3-1. Project Alternatives Carried Forward

	Alternative A	Alternative B	Alternative C
Screening Study Identifier	R12C3	R18C3	R18C3
Sites Reservoir Size	1.27 MAF	1.81 MAF	1.81 MAF
Conveyances			
TC Canal	2,100 cfs	2,100 cfs	2,100 cfs
GCID Canal	1,800 cfs	1,800 cfs	1,800 cfs
New Delevan Pipeline			
Diversion	2,000 cfs	0 (No Pumping)	2,000 cfs
Release	1,500 cfs	1,500 cfs	1,500 cfs

cfs = cubic feet per second
GCID = Glen-Colusa Irrigation District
MAF = million-acre-feet
TC = Tehama-Colusa

Figure B.1-1 shows a schematic diagram of the potential NODOS project components and direction of flow. Figure B.3-1 is a map of the NODOS project facilities.

Alternative A and Alternative C would fill Sites Reservoir using water from the existing TC Canal, GCID Canal, and the Sacramento River during the winter and spring when water is available for diversion. The water from the Sacramento River would come through the Delevan Pipeline Intake Facility, which includes the Sacramento River Pumping/Generating Plant and associated fish screen facility (SRPGP), and the Delevan Pipeline from the River to Holthouse Reservoir. Releases from Sites Reservoir to the TC Canal, the GCID Canal, and to the Sacramento River through the Delevan Pipeline would be made through Holthouse Reservoir during the summer and fall as required. The releases to the Sacramento River would be used to generate hydroelectric power at the SRPGP.

Alternative B includes no new pumping from the Sacramento River; under this alternative, Sites Reservoir would be filled using water from the existing TC Canal and the GCID Canal. However, Alternative B still would include the Delevan Pipeline to the Sacramento River to provide the capability to make summer releases to the Sacramento River from Sites Reservoir. Releases to the Sacramento River would be made through an outlet energy dissipating structure constructed on the river bank at the same location where the SRPGP would be constructed for Alternative A and Alternative C. No power generation capability is included at the Sacramento River release point for Alternative B.

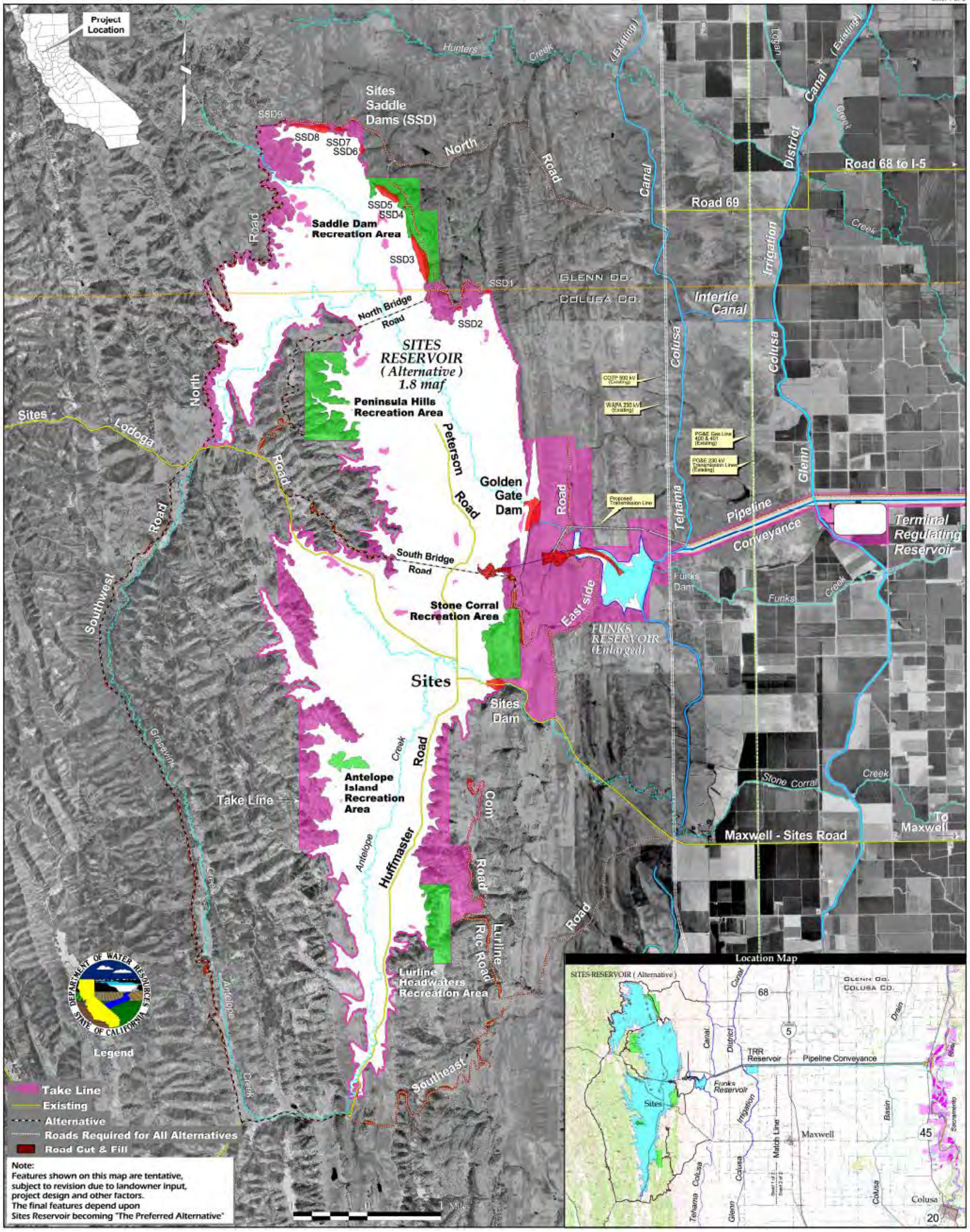


Figure B.3-1. Project Description Map

Under all three project alternatives, whenever releases are made from Sites Reservoir, hydroelectric power can be generated using pump-turbine equipment incorporated into the design of the SPGP. In addition, for Alternative A and Alternative C, releases to the Sacramento River can also be used to generate hydroelectric power using pump-turbine equipment incorporated into the design of the SRPGP. For Alternative B, there is no pumping plant at the river and releases would be made directly to the river through an energy dissipating structure with no power generation. Hydroelectric power can also be generated as releases are made to the GCID Canal using turbines incorporated into the design of the terminal regulating reservoir (TRR) pumping plant located near the connection point between the project and the GCID Canal.

Holthouse Reservoir is a common feature for all project alternatives that serves as a forebay and afterbay pond for the SPGP. Water supplied to the project from the canals and the Sacramento River is conveyed to Holthouse Reservoir for pumping into Sites Reservoir. Water released from Sites Reservoir flows into Holthouse Reservoir where it is distributed out to the canals and the Sacramento River through various project conveyances. Holthouse Reservoir will include a new dam located downstream (east) of the existing Funks Dam. The existing dam will be breached so that the existing pool and the new pool will function as one reservoir. Sediment that has accumulated in the existing Funks Reservoir would be removed as part of project construction wherever it interferes with the construction of new project components.

Holthouse Reservoir would have a minimum active storage of approximately 6,500 AF that permits the SPGP to function as a pumped-storage project for up to six hours per day while simultaneously collecting and storing inflows from the canals and Sacramento River. All water would be pumping into Sites Reservoir from Holthouse Reservoir during off-peak power periods on a daily basis.

The principal storage feature of the NODOS project is Sites Reservoir, which will have a maximum active storage capacity of 1.27 MAF or 1.81 MAF depending on the project alternative selected for further evaluation. Water in Sites Reservoir would be contained by the Golden Gate and Sites Dams located on the eastern edge of the reservoir, and by a series of Sites Saddle Dams located along the northern rim of the reservoir. Nine saddle dams would be required for the 1.81 MAF reservoir, but only six saddle dams would be required for the 1.27 MAF reservoir because the maximum reservoir water level would be 40 feet lower.

Water would be pumped by the SPGP from Holthouse Reservoir to Sites Reservoir through an inlet/outlet works located south of Golden Gate Dam. Releases from Sites Reservoir would be made through the same inlet/outlet works. Releases can be made through gated outlets located at various elevations in the vertical outlet tower to accommodate any reservoir water level and to regulate outlet water temperature. The inlet/outlet works concept would be the same for the 1.81 MAF and 1.27 MAF reservoirs.

Sites Reservoir construction would require relocating two county roads (Maxwell-Lodoga Road and Huffmaster Road) and the community of Sites. Although recreational use has not been considered as a primary project purpose, five potential recreational facility locations have been evaluated. The Stone Corral Recreation Area and the Antelope Island Recreation Area appear most suitable and most flexible to accommodate the two reservoir sizes under consideration. PFR Appendix E provides additional information about the recreation evaluation.

Various alternatives for conveying water to and from Sites Reservoir from the Sacramento River and other tributary sources were evaluated and screened in the PFR, providing a short list of conveyance management measures to carry forward in the NODOS investigation. Subsequent evaluation of the short list lead to the selection of the GCID Canal, the TC Canal, and the Delevan Pipeline as the preferred conveyances for water drafted from the Sacramento River.

Other tributary conveyances evaluated using sources other than the Sacramento River include a new pipeline from the Colusa Basin Drain (CBD) and a new pipeline from Stony Creek, originating at the Black Butte afterbay and connecting to the TC Canal below Orland. Neither was recommended for further consideration, based on their apparent inefficiency.

B.3.2 Red Bluff Diversion Dam Fish Passage Improvement Project

B.3.2.1 General

The TCCA Fish Passage Improvement Project is being constructed at the Red Bluff Diversion dam (RBDD). Construction began in 2009 and is expected to be completed in 2012. The project (shown in schematic form on Figure B.3-2) includes a fish screen facility, a pumping plant at the Mill Site (known as the Red Bluff Pumping Plant), canal, siphon, a forebay, switchyard, and a bridge across Red Bank Creek.

The fish screen structure is designed to meet National Oceanic and Atmospheric Administration-National Marine Fisheries Service and California Department of Fish and Game criteria for diversion flows of 80 to 2,500 cfs.

The new pumping plant will accommodate up to11 pumps providing a total capacity of 2,500 cfs. Initially, however, only nine vertical axial-flow pumps will be installed. The nine pumps include seven 250-cfs pumps and two 125-cfs pumps providing an initial capacity of 2,000 cfs. The two remaining unused pump bays will accommodate the future installation of two 250-cfs vertical axial-flow pumps, which would raise the total capacity to 2,500 cfs after the pumps are installed.

B.3.2.2 NODOS Component of the Red Bluff Diversion Dam Fish Passage Improvement Project

NODOS project operation will require the installation of one 250 cfs pump in one of the two spare pump bays at the Red Bluff Pumping Plant to bring the total capacity of the plant to 2,250 cfs. The NODOS project scope is limited to the additional pump's installation and operation impacts and responsibilities.

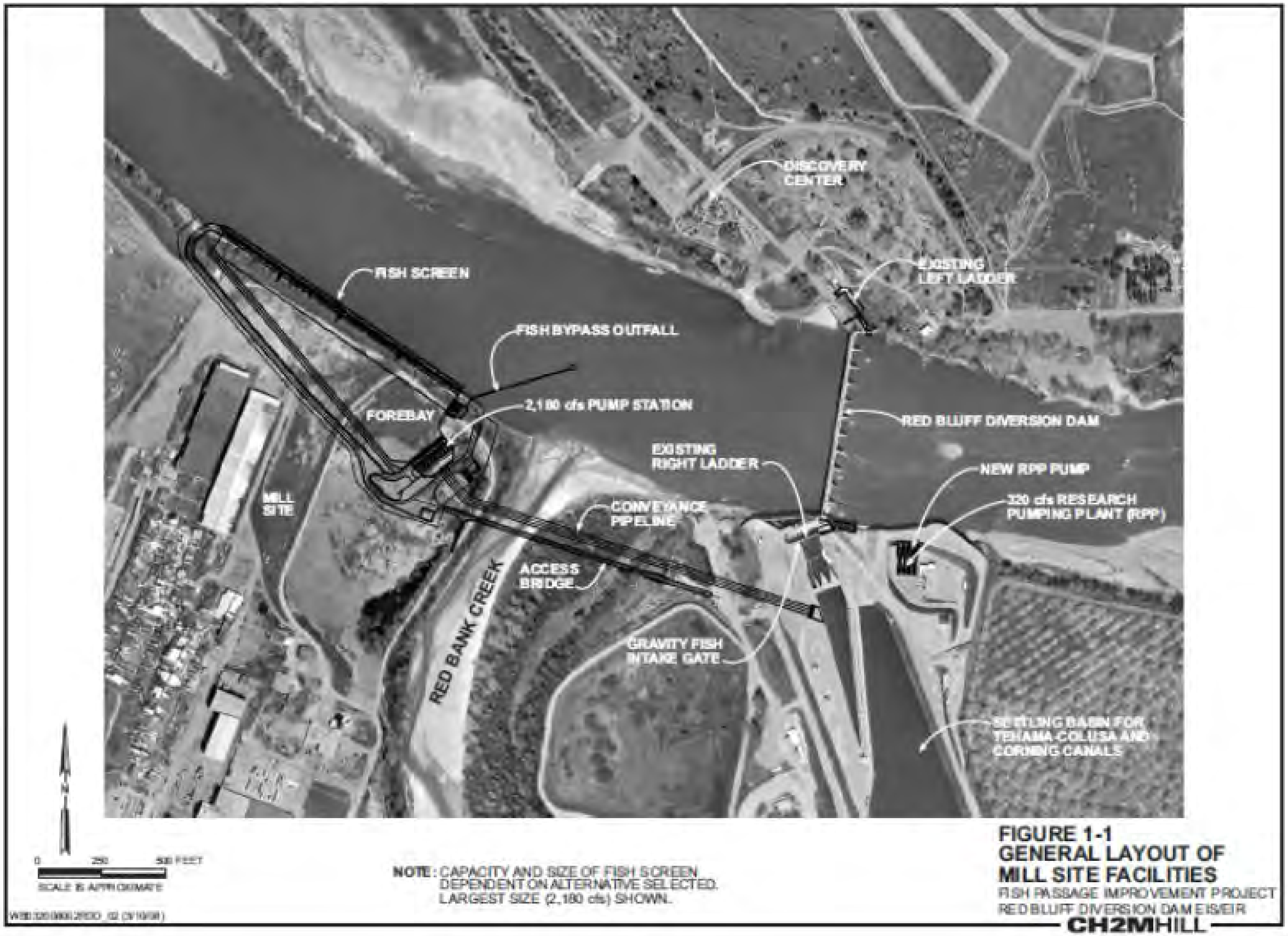


Figure B.3-2. General Layout of Fish Passage Improvement Facility

B.3.2.3 Pump Installation, Operation, and Maintenance

Pump Installation

Installation of the 250-cfs vertical axial flow pump unit would require the following construction activities:

- The spare pump bay to receive the NODOS project pump already includes an existing 84-inch-diameter steel pipe embedded in the pumping plant back wall. The pipe includes a blind flange on the afterbay side of the pumping plant back wall to prevent water from draining back into the forebay. The blind flange would be removed and replaced with an 84-inch butterfly valve. A new 84-inch-diameter flanged steel pipe spool (approximately 3 feet long) would be connected to the butterfly valve and terminate with a new 84-inch flap gate. Permanent supports would be required beneath the butterfly valve and flap gate.

- Dewatering the afterbay would likely be required. Therefore, installation of the pump should be performed during the non-irrigation season to minimize interruptions to the irrigation delivery system. A mobile crane would be required to install the piping and appurtenances.

- Install pumping plant unit bay stoplogs, using a mobile crane, to accommodate dewatering the pump bay.

- Inspect pump bay and remove all sediment. Access to the bottom floor of the pumping plant is provided at each bay via 4´6” x 7´0” access hatches and ladders.

- Remove roof hatches over the pump unit bay using a mobile crane.

- Install the pump in accordance with the pump manufacturer’s written installation instructions, including constructing the pump pedestal and connecting the pump discharge nozzle to the discharge pipe via a new flexible coupling.

- Install electrical conductors and supervisory control and data acquisition system.

Operations

The Red Bluff Pumping Plant includes a control system to provide remote manual and remote auto control of pumps and associated appurtenances. The pumping plant and associated gravity conveyance system are designed to deliver water to the existing 17-acre settling basin located to the west of the RBDD. Once in the settling basin, water would flow to Check No. 1 on the TC Canal and the Corning Pumping Plant, similar to the current RBDD operation. The additional pump at the pumping plant would allow for normal operational diversions up to 2,160 cfs for each NODOS action alternative in winter and spring months, including up to 2,100 cfs for diversion to the proposed Sites Reservoir and an additional 50 to 60 cfs for maintaining existing winter and spring flow operations of the TC Canal. The difference between the installed capacity (2,500 cfs) at the pumping plant and the operation diversions (2,160 cfs) is spare pumping capacity.

Maintenance

It is anticipated that the following basic preventive measures would be undertaken on a regular basis to maintain the NODOS project vertical axial-flow pump and its appurtenances that would be installed as part of the NODOS project. These activities would occur as part of the regular maintenance activities for the Red Bluff Pumping Plant.

- Wash down or pressure wash as necessary
- Check for rust/corrosion, annually; maintain all coatings
- Visually inspect for damage or wear, monthly
- Assess fluids and lubrication; address as necessary
- Inspect pumping plant trashracks daily and remove debris as necessary
- Visually inspect butterfly valves and flap gates, monthly

The additional NODOS project pump would not increase the frequency of maintenance activities required at the pumping plant, nor would it require additional personnel to perform pump maintenance. However, the volume and timing of non-TCCA water diversions, through any of the pumps, could impact the sediment load distributed to the TCCA system (i.e., the pumping plant forebay and settling basin). Increased sedimentation associated with non-TCCA water diversions may require more frequent dredging within the pumping plant forebay than prior to the NODOS pump installation and operation.

B.3.3 Sites Reservoir

B.3.3.1 General

Figure B.3-3 presents the reservoir area-capacity curve for the Sites Reservoir site. Reservoir capacities of 1.81 MAF and 1.27 MAF are currently under consideration at the site. Table B.3-2 summarizes reservoir parameters for each capacity.

Table B.3-2. Sites Reservoir Sizes Under Consideration

	Large Reservoir	Small Reservoir
Active Storage Capacity	1.81 MAF	1.27 MAF
Maximum Operating Water Elevation	520 feet	480 feet
Crest Elevation (Without Camber)	540 feet	500 feet
Inundation Area	14,200 acres	12,400 acres
Minimum Operating Water Elevation	340 feet	340 feet
To of Dead Pool	320 feet	320 feet

MAF=million-acre-feet

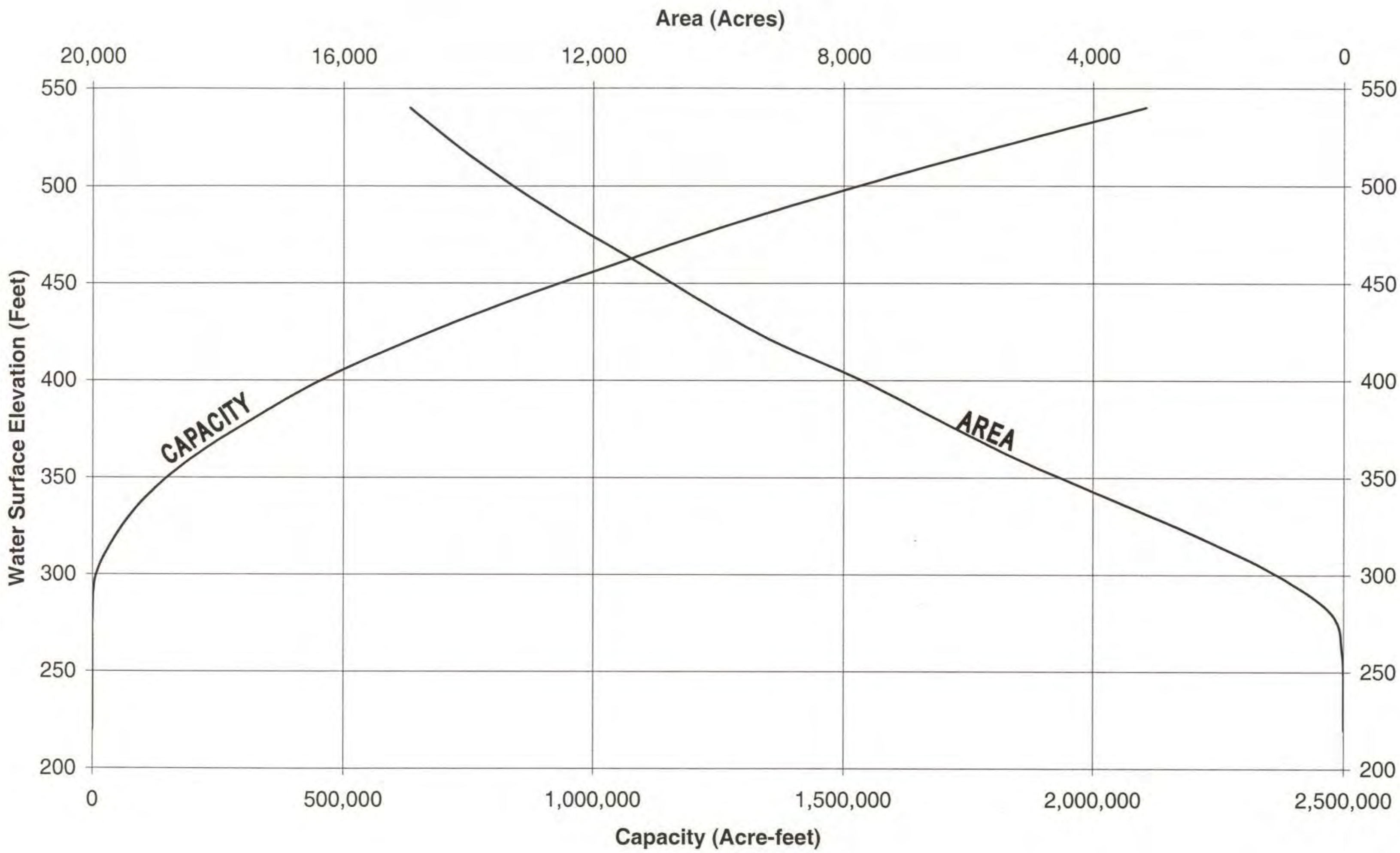


Figure B.3-3. Sites Reservoir – Area versus Capacity Curve

B.3.3.2 Maximum Feasible Reservoir Elevation

Preliminary feasibility studies conducted by DWR focused on constructing Sites Reservoir to provide the greatest water supply yield. Selection of the larger 1.81 MAF reservoir with WSE of 520 feet was based upon review of reservoir rim topography, site geology, the presence of geologic features trending through the reservoir rim, and a cursory evaluation of the relationship between embankment volume and reservoir storage for a range of WSEs from 480 to 560 feet. A review of the reservoir rim indicated that WSEs above 540 feet would likely require treatment of the lower saddle areas along the relatively steep ridges that form the eastern side of the reservoir where seepage paths would be relatively short, in order to control adverse seepage out of the reservoir at these locations. This treatment, combined with the increase in dam embankment material volume in relation to reservoir surface elevation (Figure B.3-4), would likely result in larger unit costs per AF of storage for reservoir elevations above 540 feet. Therefore, reservoir alternatives below elevation 540 feet are considered more economical on a unit cost basis. Consequently, a maximum WSE of 520 feet was selected to ensure that the proposed size of Sites Reservoir would be technically feasible and not unduly expensive.

B.3.3.3 Sites Reservoir Dams – General

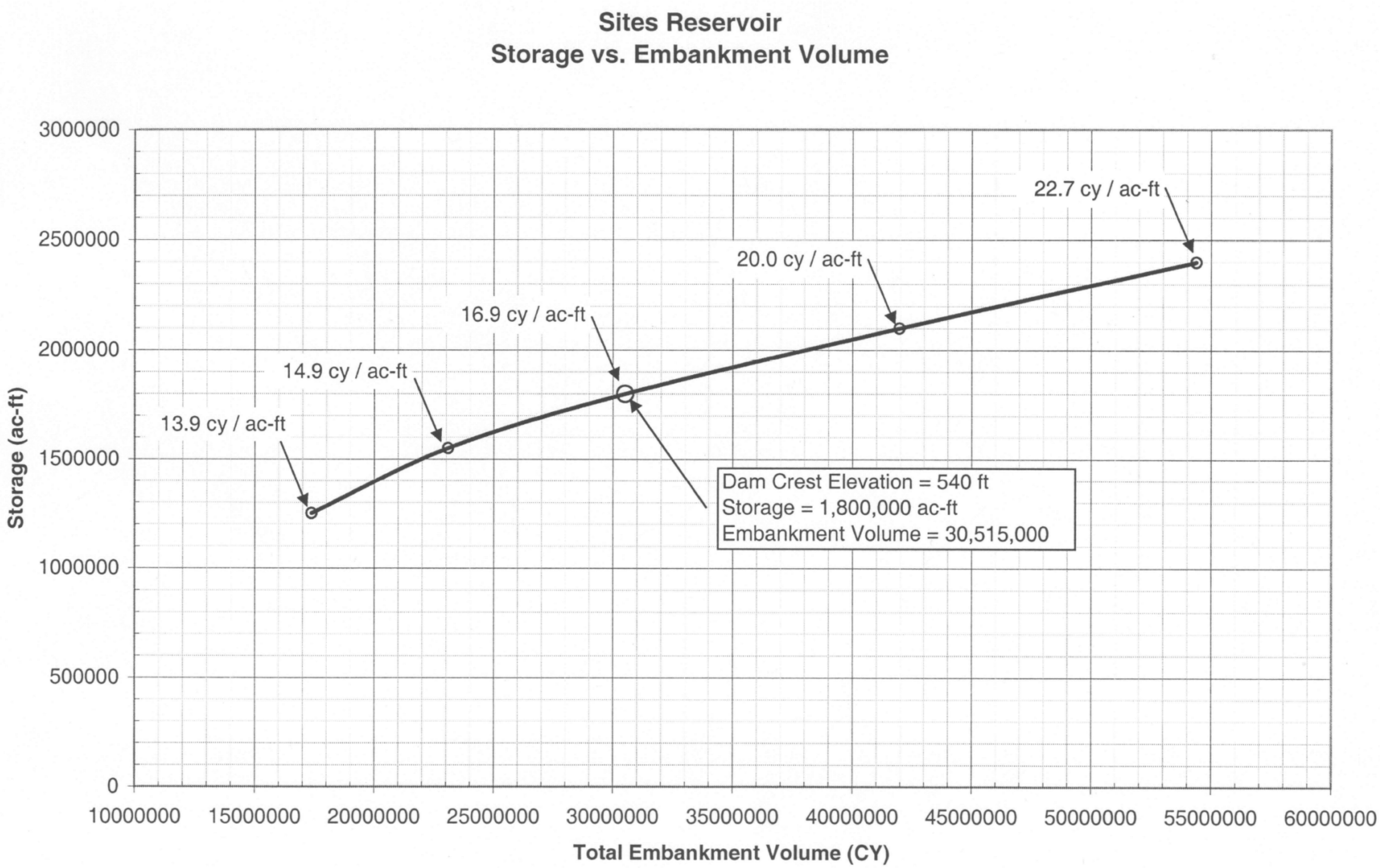
The two Sites Reservoirs currently under investigation would require the construction of Golden Gate Dam on Funks Creek, Sites Dam on Stone Corral Creek, and a series of Sites Reservoir Saddle Dams on the northern end of the reservoir between the Funks Creek and Hunters Creek watersheds. The number of saddle dams required depends upon the size of the reservoir. The main dams and saddle dams discussed in this report are all zoned earth and rockfill embankment dams because previous investigations have indicated that this type of dam would be the most economical considering the remoteness of the site and the types of materials available for dam construction. Future investigations, however, may show that roller-compacted concrete (RCC) dams could economically competitive if the quality and quantity of rock material available in the project region is found to be adequate for use in an RCC mix. Tables B.3-3 and B.3-4 present a summary of dam characteristics required to impound Sites Reservoir for the two reservoir sizes currently under consideration.

Table B.3-3. Required Dams Volumes For 1.81 MAF Sites Reservoir

Dam	Maximum Height Above Streambed[a] (feet)	Crest Length (feet)	Total Embankment Volume (cubic yards)
Golden Gate Dam	310	2,250	10,590,000
Sites Dam	290	850	3,836,000
Saddle Dam 1	50	490	93,000
Saddle Dam 2	80	420	86,000
Saddle Dam 3	130	3,810	3,577,000
Saddle Dam 4	40	270	18,000
Saddle Dam 5	100	2,290	1,505,000
Saddle Dam 6	70	530	144,000
Saddle Dam 7	75	1,040	196,000
Saddle Dam 8	105	2,990	1,915,000
Saddle Dam 9	45	340	49,000
		Total	**22,000,000**

[a] Maximum height above streambed is measured from the downstream toe, except for Saddle Dam 2 which has a height of 80 feet at the upstream toe.

MAF = million acre-feet



From "Sites Reservoir - Addendum to Embankment Volume and Reservoir Storage Assessment," May 7, 2002.
Embankment volumes shown in this figure are cursory level quantity estimates.

Figure B.3-4. Sites Reservoir – Storage versus Embankment Volume

Table B.3-4. Required Dams Volumes For 1.27 MAF Sites Reservoir

Dam	Maximum Height Above Streambed[a] (feet)	Crest Length (feet)	Total Embankment Volume (cubic yards)
Golden Gate Dam	270	2,250	5,987,000
Sites Dam	250	850	2,853,000
Saddle Dam 1	10	490	1,400
Saddle Dam 3	90	3,810	1,365,000
Saddle Dam 5	60	2,290	398,000
Saddle Dam 6	10	530	9,000
Saddle Dam 8a	65	2,990	390,000
Saddle Dam 8b	5	340	15,000
		Total	**11,018,400**

[a] Maximum height above streambed is measured from the downstream toe.
MAF = million acre-feet

Design Assumptions

Preliminary feasibility level design of the Sites Reservoir dams was performed in accordance with the state of practice for dam design with conformance to current dam safety criteria. The dams were designed using the philosophy that available on-site materials should dictate selection of the dam section to ensure use of the most economical alternative. In addition, site topography, geology, seismicity, and foundation features were considered when selecting dam alignments and sections. The dam designs conform to modern economic construction practice and incorporate conservative design measures.

Dam Type

Cost estimates prepared by DWR indicate that zoned earth, rockfill embankment type dams can be constructed more economically than RCC dams at the Golden Gate and Sites Dam sites. However, the economic comparisons performed to date have assumed that RCC aggregate would be imported from the old, abandoned channel on Stony Creek because the suitability and use of locally available Venado sandstone has not been confirmed. Use of crushed and processed Venado sandstone for RCC aggregate, as well as for filter, drain, and transition materials for zoned earth rockfill embankment type dams, could result in cost savings to the project for either type of dam because the hauling distances to the dam sites would be reduced from 30 to 35 miles to 1 to 4 miles.

It is recommended that additional testing and evaluation of the Venado sandstone be performed during final design to verify its suitability for use as a filter, drain, and transition material and RCC aggregate. Once the suitability of Venado sandstone is evaluated, a study should be conducted to ensure selection of the most economical and technically feasible dam type for all of the Sites Reservoir dams. The study should include an evaluation of the advantages and disadvantages of each type of dam, as well as economics to establish that the type of dam selected for each site is most suitable to locally available construction materials, material transportation considerations, construction site constraints, and the seismicity of the project area. The study should also include consideration of foundation faulting and the ability of an RCC gravity dam to accommodate the predicted ground motions and displacements. Once a dam type is selected, the dam section should be further refined during final design to minimize costs.

B.3.3.4 Golden Gate Dam

General

Past investigations have examined alternative dam alignments for Golden Gate Dam with various maximum reservoir WSEs, four alternative alignments to impound Sites Reservoir with a maximum WSE of 520 feet. The DWR investigations included comparisons between the alternative alignments based upon:

- Conformance of the dam alignment with the site topography
- Preferable contact of the dam embankment at the abutments
- Minimizing the required foundation excavation and embankment materials
- Presence and orientation of foundation defects, such as Faults GG-1 and GG-2
- Constructability of the dam embankment based upon section development considering the complex site topography
- Economic comparisons of the more preferable alternative alignments
- Construction of the dam to provide the maximum safe storage possible

Dam Alignment – 1.81 MAF Reservoir

For the 1.81 MAF reservoir alternative, Golden Gate Dam would be located on Funks Creek approximately 1 mile west of Funks Reservoir. The proposed dam embankment has a crest elevation of 540 feet, a crest length of 2,250 feet, a maximum height of 310 feet above the streambed, and a total embankment volume of 10.6 million cubic yards. Figure B.3-5 presents a plan view of the dam embankment.

Dam Alignment – 1.27 MAF Reservoir

For the 1.27 MAF reservoir alternative, Golden Gate Dam would be located on Funks Creek approximately 2,000 feet upstream of the dam location for the 1.81 MAF reservoir, and on the western edge of the ridges that form the east reservoir rim. The proposed dam embankment has a crest elevation of 500 feet, a crest length of 2,250 feet, a maximum height of 270 feet above the streambed, and a total embankment volume of 6.0 million cubic yards. Note that Golden Gate Dam at this more upstream location would require a separate saddle dam (identified as Saddle Dam 10) to impound water in a low saddle area just south of the main dam. The downstream location for the 1.81 MAF Reservoir encompasses the saddle area and eliminates the need for a saddle dam.

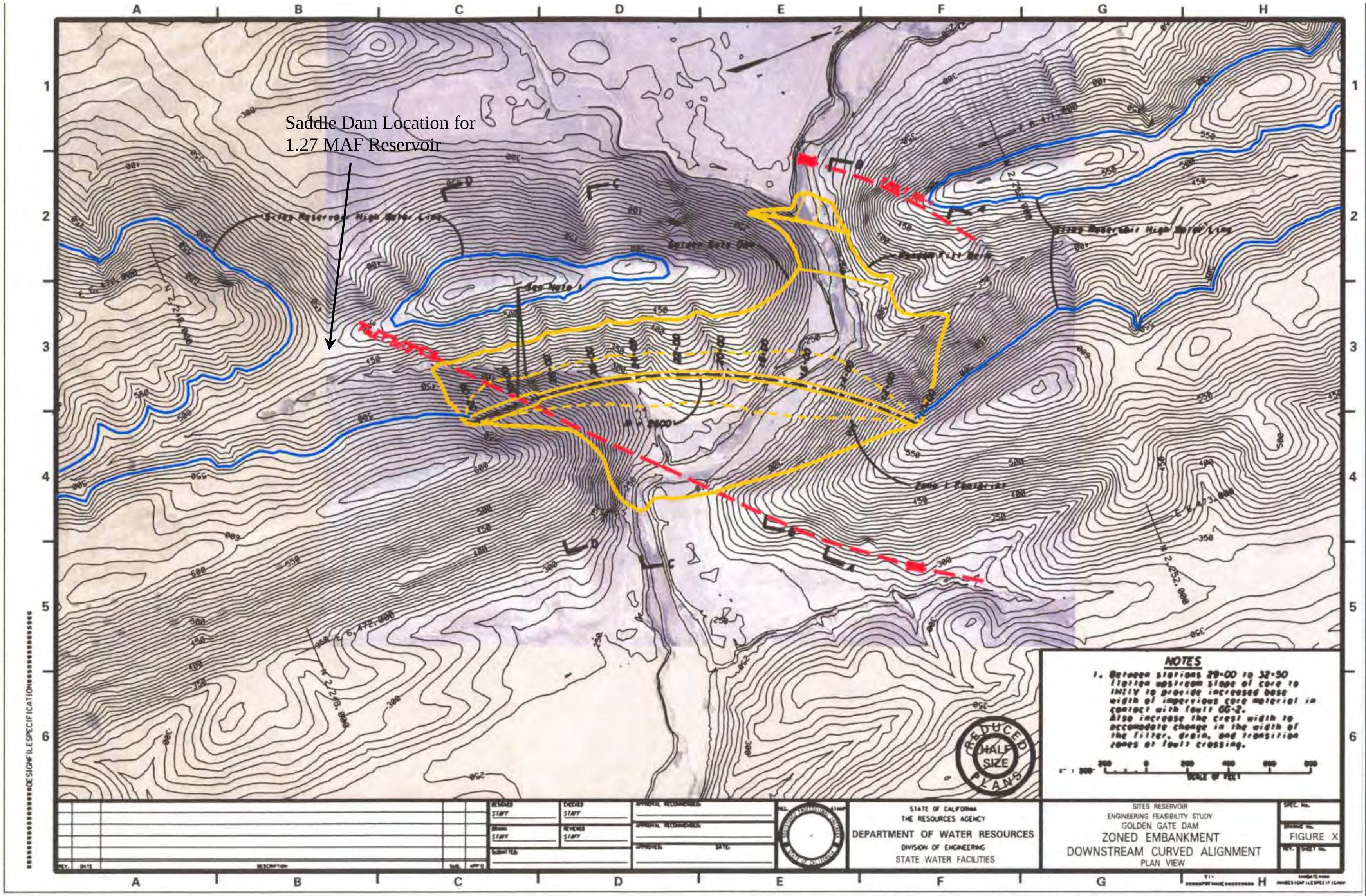


Figure B.3-5. Golden Gate Dam – Zoned Embankment – Downstream Curved Alignment – Plan View

Note:
Plan shown is for 1.81 MAF reservoir. Plan for 1.27 MAF reservoir would be similar, but located approximately 2,000 feet upstream from the location shown.

Dam Section

The proposed embankment section for the 1.27 MAF and 1.81 MAF reservoirs is a zoned earth, rockfill embankment type dam consisting of a central impervious core transitioned to exterior rockfill shells (Figure B.3-6). The dam would have a crest width of 30 feet, and 20 feet of freeboard above the maximum reservoir elevation. The upstream and downstream slopes of the dam embankment are 2.25H:1V and 2H:1V, respectively. These slopes were selected using engineering judgment and verified by performing feasibility level stability evaluations.

Consistent with typical designs for similar type dam embankments, the upstream and downstream slopes of the core were selected to be 0.5H:1V. In the vicinity of known faults, the upstream slope of the core was flattened to 1H:1V to provide an increased base width of impervious core material in contact with Fault GG-1 and GG-2. The crest width in this area was also increased to 60 feet to accommodate an increase in the width of the upstream and downstream filter, drain, and transition zones from 15 to 30 feet wide above elevation 500 feet to within 10 feet of the dam crest. This modification was included to provide an additional defensive measure against potential displacements along the fault.

Upstream of the core, a 30-foot-wide zone of filter and transition materials are included for filter compatibility between the impervious core and pervious shell material. Downstream of the core, a 30-foot-wide zone of filter and drain materials is included for filter compatibility between embankment materials, to provide control of embankment seepage, and act as a crack stopper. The 30-foot-width of filter, drain, and transition materials was selected to ensure constructability of this multi-element zone. The downstream embankment section also incorporates a 20-foot-thick blanket drain, comprised of filter and drain materials, to control foundation seepage and to provide a horizontal conduit for seepage collection at the downstream toe.

Because excavation operations for the dam foundation, rockfill quarry, and appurtenant structures would generate rock materials containing appreciable amounts of fines not meeting hydraulic conductivity requirements for specific embankment zones such as the shell, filter, and drain, random material zones were incorporated into the upstream and downstream sections of the embankment. The upstream random zone is placed at elevation 300 feet and extends 150 feet beyond the upstream toe. This random zone would function as an upstream toe berm, providing a convenient location to place materials from foundation excavation operations during the initial stages of construction, and would also be utilized to divert Funks Creek from the dam footprint during construction. This random zone would also provide increased slope stability. Because materials downstream of the chimney drain do not need to meet specific hydraulic conductivity requirements, a large downstream random zone was incorporated into the dam embankment. A 25-foot-wide zone of rockfill material is included over the random material on the downstream face of the embankment providing increased wear resistance to minimize long-term maintenance costs.

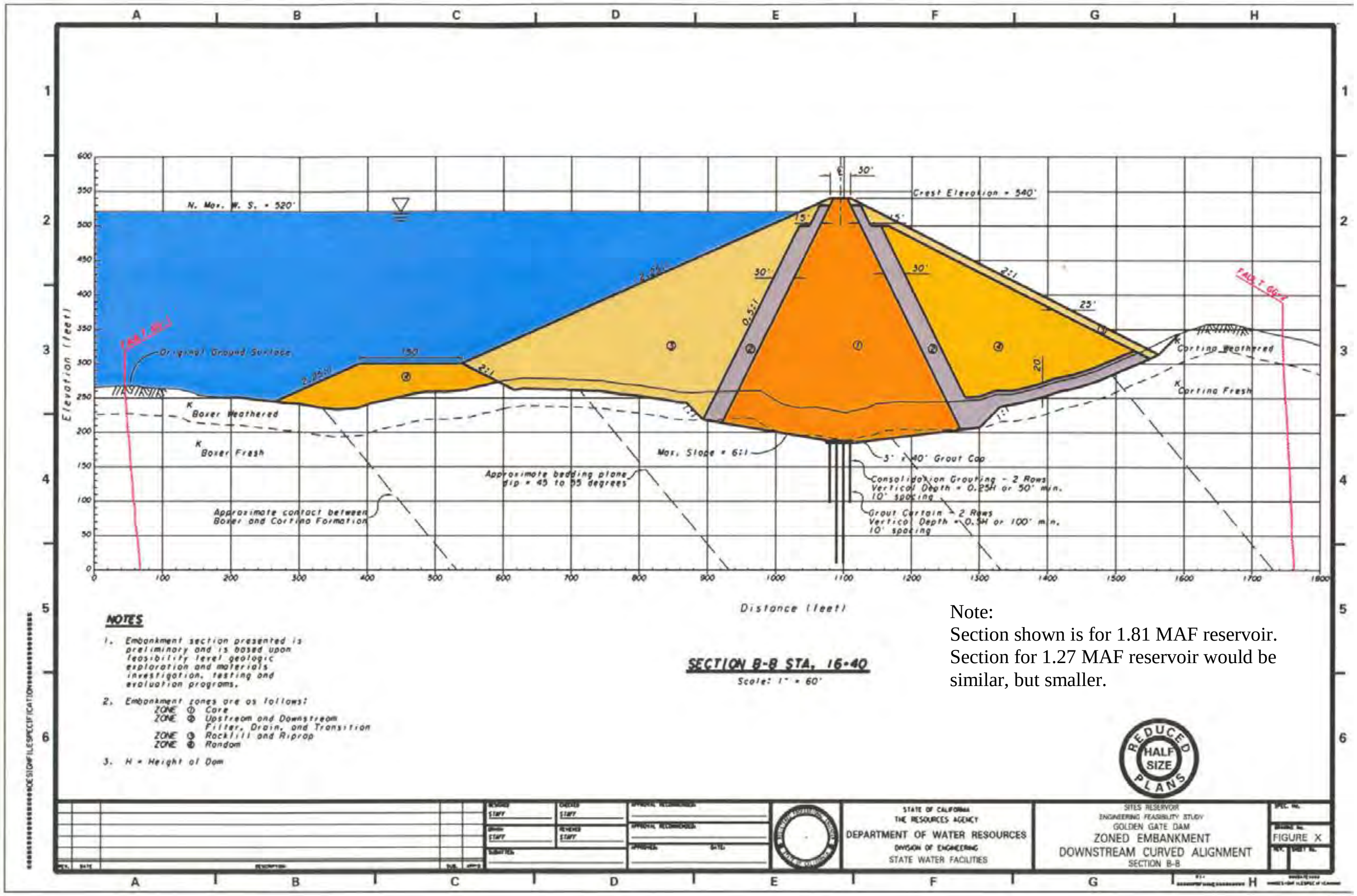


Figure B.3-6. Golden Gate Dam – Zoned Embankment – Downstream Curved Alignment – Maximum Section

Embankment Materials

Selection of the Golden Gate Dam embankment section was based upon the available on-site materials identified and evaluated as part of the materials investigation program. A summary of the materials designated for use in specific embankment zones is discussed below.

Zone 1: Impervious core material comprised of low to medium plasticity clays, with lesser amounts of high plasticity clays and clayey sands. The impervious material would be obtained from a designated borrow area upstream of the dam in the reservoir and from required excavation for the dam embankment, appurtenant structures, and Funks Reservoir enlargement. Haul distances would be less than 1 mile. Impervious material processing beyond normal disking and moisture conditioning in the designated borrow areas, would not be required.

Zone 2: Filter, drain, and transition materials consisting of fresh rock processed to various sizes to meet filter compatibility and hydraulic conductivity requirements. Although fresh Venado sandstone of the Cortina Formation may be suitable for use as these embankment materials, this material has not been confirmed at the present level of investigation due to the need for an extensive particle breakage evaluation. Therefore, it is assumed that the filter, drain, and transition materials would be imported from the closest off-site sand and gravel source to ensure a conservative estimate of material costs. The closest off-site source is an old, abandoned channel on Stony Creek between Orland and Willows, approximately 35 road miles from the Golden Gate Dam site.

Zone 3: Shell material consisting of processed clean rockfill up to 30-inch maximum particle size. The shell material would be obtained from fresh Venado sandstone of the Cortina Formation from a quarry developed in the ridge on the east side of the reservoir, near the dam site, with haul distances of less than 1 mile. Quarry operations would require drilling and blasting with selective processing to produce the required particle sizes and gradation.

Zone 4: Random material comprised of material unsuitable for use as clean rockfill. Random material would consist of weathered sandstone, mudstone, slopewash, etc. obtained from excavation of the dam foundation, appurtenant structures, and the rockfill quarry. Haul distances would be less than 1 mile and processing would not be required.

Foundation

Bedrock underlying the Golden Gate Dam footprint is predominately sandstone with interbedded mudstone of the Cortina Formation. Based on geologic characterization and visual observation of limited amounts of drill core, moderately weathered bedrock is considered to be a suitable foundation surface for the shell, transition, filter, and drain. In addition, slightly weathered to fresh bedrock is considered a suitable foundation surface for the central impervious core. To meet the foundation objectives, recent and older alluvium, decomposed, and intensely weathered bedrock would be excavated from the entire footprint of Golden Gate Dam to obtain a moderately weathered bedrock surface. In addition, moderately weathered bedrock would be excavated from the impervious core footprint down to the top of slightly weathered and/or fresh bedrock surface.

Grouting

A review of water pressure test data from DWR drill holes in the Golden Gate Dam foundation indicates that the slightly weathered to fresh bedrock, is, overall, fairly impermeable. Approximately 80 percent of the intervals tested within the anticipated grouting depth range were characterized as impermeable. However, some intervals of higher water take occurred within the upper portions of some of the drill holes to depths of up to 80 feet below the estimated excavated foundation surface (slightly weathered to fresh bedrock).

Since water pressure test data indicated that some areas of higher hydraulic conductivity occur in the upper portion of the dam foundation, consolidation and curtain grouting was included in the dam design to reduce seepage through the dam foundation. The grout program will consist of a two-row grout curtain with one row of consolidation holes upstream and one row downstream of the curtain holes. The rows will parallel the dam centerline and be spaced 10 feet apart. In addition, a 40-foot-wide by 3-foot-thick grout cap was included to prevent surface leakage of grout during grouting of the upper stage.

Each row of consolidation and curtain grout holes will consist of mandatory primary and secondary holes spaced at 10-foot centers. In addition, it was assumed that tertiary holes, split spaced between the primary and secondary holes, will be required over half the length of the dam to meet grout closure criteria. Consistent with dam foundation grouting practices, the drilling depth of consolidation holes was estimated to be one quarter the height of the dam, together with a minimum depth of 50 feet. In addition, the drilling depth of curtain holes was estimated to be one half the height of the dam, together with a minimum depth of 100 feet. Grout takes in the consolidation and curtain holes were estimated at 0.5 and 1.0 sacks of cement per linear foot of grout hole, respectively, as recommended by Reclamation.

The grouting quantity estimates used for cost estimating also include verification testing as part of the grouting program. Verification holes will be drilled between the curtain grout rows, along the dam centerline. These holes were assumed to have an average hole spacing of 75 feet commensurate with typical spacing used for other dam foundation grouting programs. Verification testing will be conducted to a depth equivalent to the curtain grout holes.

In addition to the grouting program described here, additional grouting and/or treatment of special features such as the GG-2 fault will likely be required. Stitch grouting may be used to treat this type of special feature as well as other special features, faults, and sheared areas of bedrock that cross the core, filter and drain zones. The use of slush grouting may also be desirable in areas of the core, filter and drain zones foundations to fill small sheared zones and bedding plane joints that are open a small amount. The need for additional grouting and/or treatment not discussed here will need to be examined further during final design once additional geologic information is available.

B.3.3.5 Sites Dam

General

The proposed alignment for Sites Dam was selected to:

- Conform to the site topography
- Provide favorable abutment contacts
- Minimize the amount of embankment over Fault S-2 on the right abutment
- Minimize the amount of excavation within the core footprint to provide an approximately level foundation transverse to the dam centerline
- Minimize the amount of embankment materials

Alternative alignments were also examined both upstream and downstream of the selected location, but were eliminated from consideration because these alignments:

- Did not conform to site topography or provide as favorable abutment contacts as selected alignment
- Increased the amount of embankment over Fault S-2
- Increased the amount of embankment materials and the quantity of excavation needed to provide a level foundation for impervious core material

Dam Alignment – 1.81 MAF Reservoir

Sites Dam for the 1.81 MAF reservoir alternative would be located on Stone Corral Creek approximately 0.25 mile east of the town of Sites and 8 miles west of the town of Maxwell. The proposed dam embankment has a crest elevation of 540 feet, a crest length of 850 feet, a maximum height of 290 feet above the streambed, and a total embankment volume of 3.8 million cubic yards. Figure B.3-7 presents a plan view of the dam embankment.

Dam Alignment – 1.27 MAF Reservoir

Sites Dam for the 1.27 MAF reservoir alternative would be at the same location selected for the 1.81 MAF reservoir, but the crest elevation would be lower. The proposed dam embankment has a crest elevation of 500 feet, a crest length of 850 feet, a maximum height of 250 feet above the streambed, and a total embankment volume of 2.85 million cubic yards.

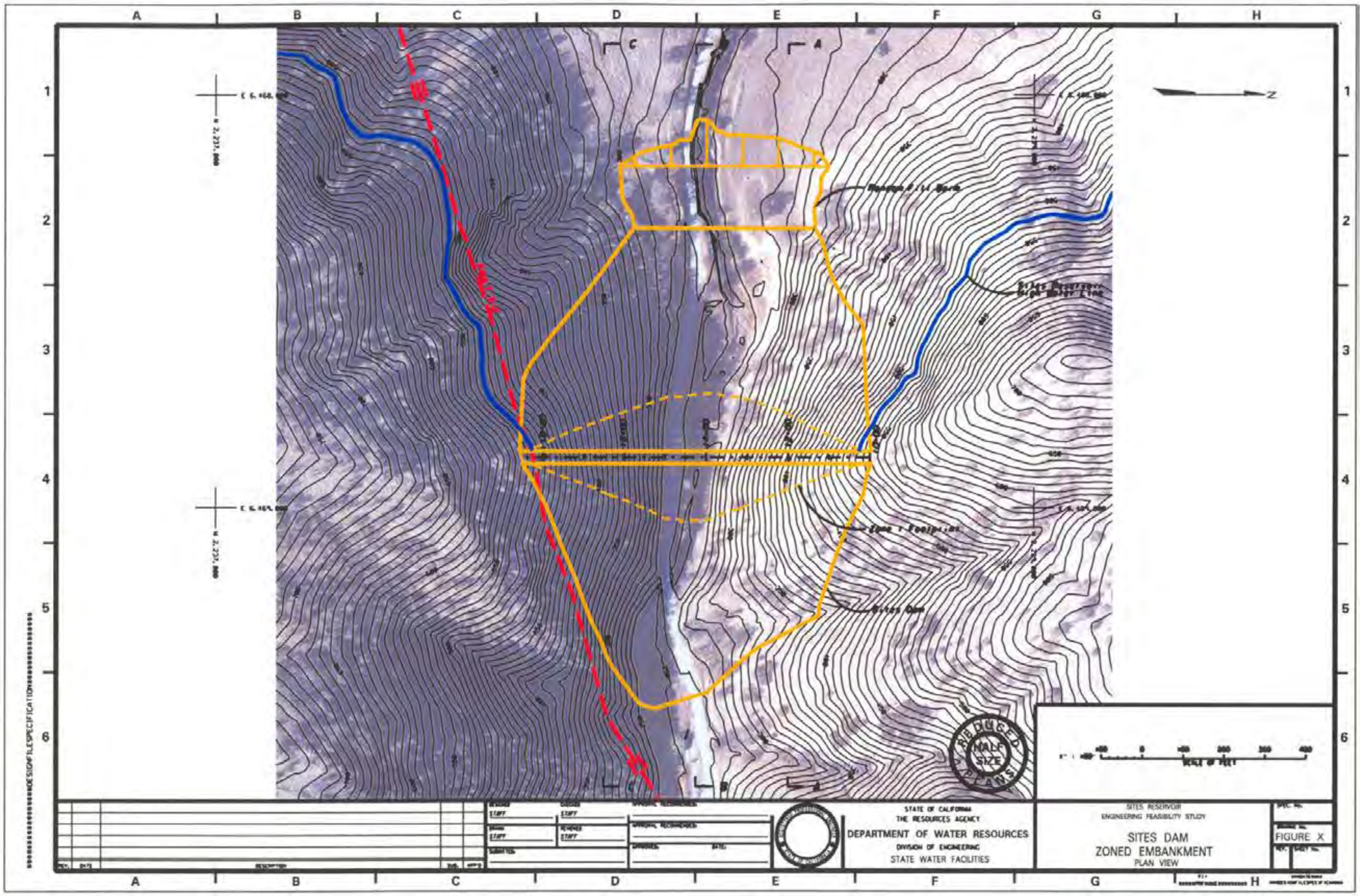


Figure B.3-7. Sites Dam – Zoned Embankment – Plan View

Note:
Plan shown is for 1.81 MAF reservoir. Plan for 1.27 MAF reservoir would be similar.

Dam Section

Because the height, foundation conditions, and available on-site material sources for constructing Sites Dam are very similar to Golden Gate Dam, the proposed dam section adopted for Golden Gate Dam was also adopted for Sites Dam (Figure B.3-8) for both reservoir size alternatives with the following exceptions:

- The upstream slope of the core was not flattened from 0.5H:1V at any location along the dam alignment as was done at Golden Gate Dam. Fault S-2 crosses the Sites Dam embankment on the upper right abutment near the dam crest. Therefore, flattening the impervious core slope was not considered necessary.

- Similar to Golden Gate Dam, an upstream random zone would function as an upstream toe berm, provide a convenient place for waste materials from foundation excavation operations during the initial stages of construction, and also would be utilized to divert Stone Corral Creek from the dam footprint. Because random materials generated from foundation excavation upstream of the dam centerline would be comprised of Boxer Formation and would have lower shear strength than random materials generated from the Cortina Formation, these materials would be incorporated into the upstream toe berm and not the main embankment section.

- The impervious core material (Zone 1) would be obtained from a designated borrow area upstream of the dam, in the reservoir, and from required excavation for the dam embankment. Impervious material obtained from required excavation for Funks Reservoir enlargement would not be incorporated into Sites Dam.

Foundation

Bedrock underlying the Sites Dam footprint consists of both Boxer and Cortina Formations. The upstream footprint of the dam would be predominately founded on Boxer Formation and the downstream footprint of the dam would be founded on Cortina Formation. At the Sites Dam site, the Boxer Formation is generally characterized as mudstone with sandstone interbeds while the Cortina Formation is generally characterized as sandstone with interbedded mudstone. Although the dam footprint would be founded on two different bedrock formations, this is not considered to present a problem with construction of an embankment dam at this site. Similar to Golden Gate Dam, moderately weathered bedrock is considered a suitable foundation surface for the shell, transition, filter, and drain. In addition, slightly weathered to fresh bedrock is considered a suitable foundation surface for the central impervious core.

To meet the foundation objectives, recent and older alluvium, decomposed, and intensely weathered bedrock would be excavated from the entire footprint of Sites Dam to obtain a moderately weathered bedrock surface. In addition, moderately weathered bedrock would be excavated from the impervious core footprint down to the top of slightly weathered and/or fresh bedrock surface.

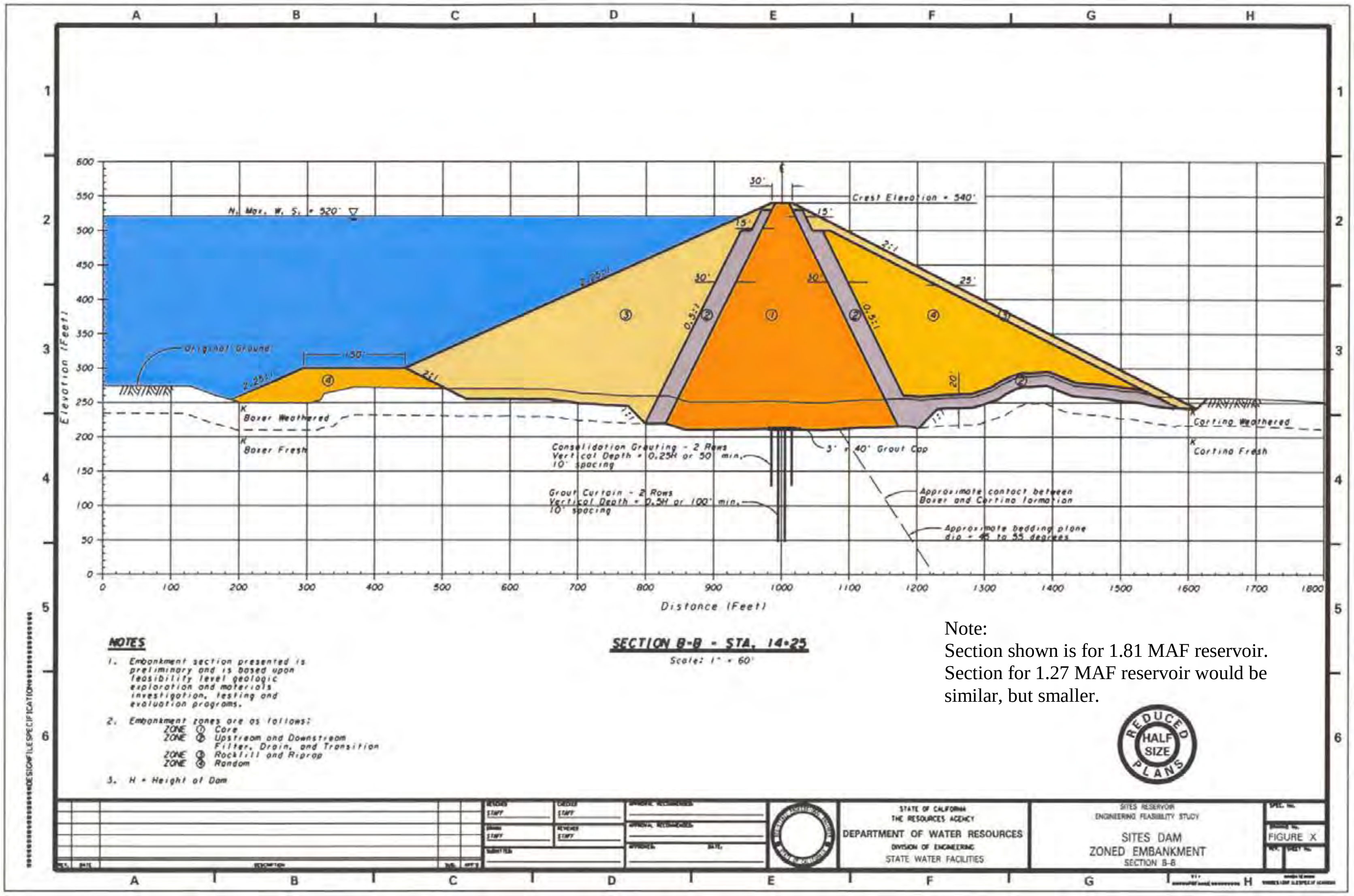


Figure B.3-8. Sites Dam – Zoned Embankment – Maximum Section

Grouting

A review of water pressure test data from DWR and Reclamation drill holes in the Sites Dam foundation indicates that the slightly weathered to fresh bedrock is generally impermeable at depth, but has an interval of higher hydraulic conductivity closer to the surface. The higher water takes generally occurred to depths of 40 to 60 feet below the estimated excavated foundation surface. Below this depth range the rock mass was generally impermeable. Because water pressure test data indicates that some areas of higher hydraulic conductivity occur in the dam foundation, consolidation and curtain grouting was included in the dam design to reduce seepage through the dam foundation. The grouting program was assumed to be the same as presented for Golden Gate Dam, which represents a fairly typical design for a dam of this type and size. In addition to the grouting program described herein, additional grouting and/or treatment of special features such as Fault S-2 would likely be required. The need for additional grouting and/or treatment would be examined further once additional geologic information is available.

B.3.3.6 Saddle Dams

General

This discussion is generally applicable to the saddle dams required for the 1.81 MAF and 1.27 MAF Reservoirs. Saddle dams would be located at the same sites regardless of reservoir size. Fewer saddle dams would be required for the smaller reservoir because the maximum normal water level is approximately 40 feet lower.

A series of saddle dams would be located at the northern end of Sites Reservoir between the Funks Creek and Hunters Creek watersheds roughly along the Glenn-Colusa County line. The number of dams required depends upon the reservoir size. For the 1.81 MAF reservoir, nine dams are required. For the 1.27 MAF reservoir, six dams are required. The dams are numbered from south to north and the same numbers are used for both reservoir alternatives.

As mentioned above, there is one additional saddle dam (Saddle Dam 10) required just south of Golden Gate Dam for the 1.27 MAF reservoir. The quantities for this small saddle dam are included in the quantities for Golden Gate Dam and Saddle Dam 10 is not included in the saddle dam descriptions presented below.

Saddle Dams 1, 4, and 9 are generally characterized as small-sized dams. Saddle Dams 2, 3, 5, 6, 7, and 8 are generally characterized as medium-sized dams. Saddle Dams 3, 5, and 8 are the tallest and largest of the saddle dams. Tables B.3-3 and B.3-4 present saddle dam characteristics, including height and embankment volume. Figure B.3-9 presents a plan view of the saddle dams.

Dam Alignments

The topography at the north end of the proposed Sites Reservoir is such that the preferred alignments for the saddle dams are located along the relatively broad ridge

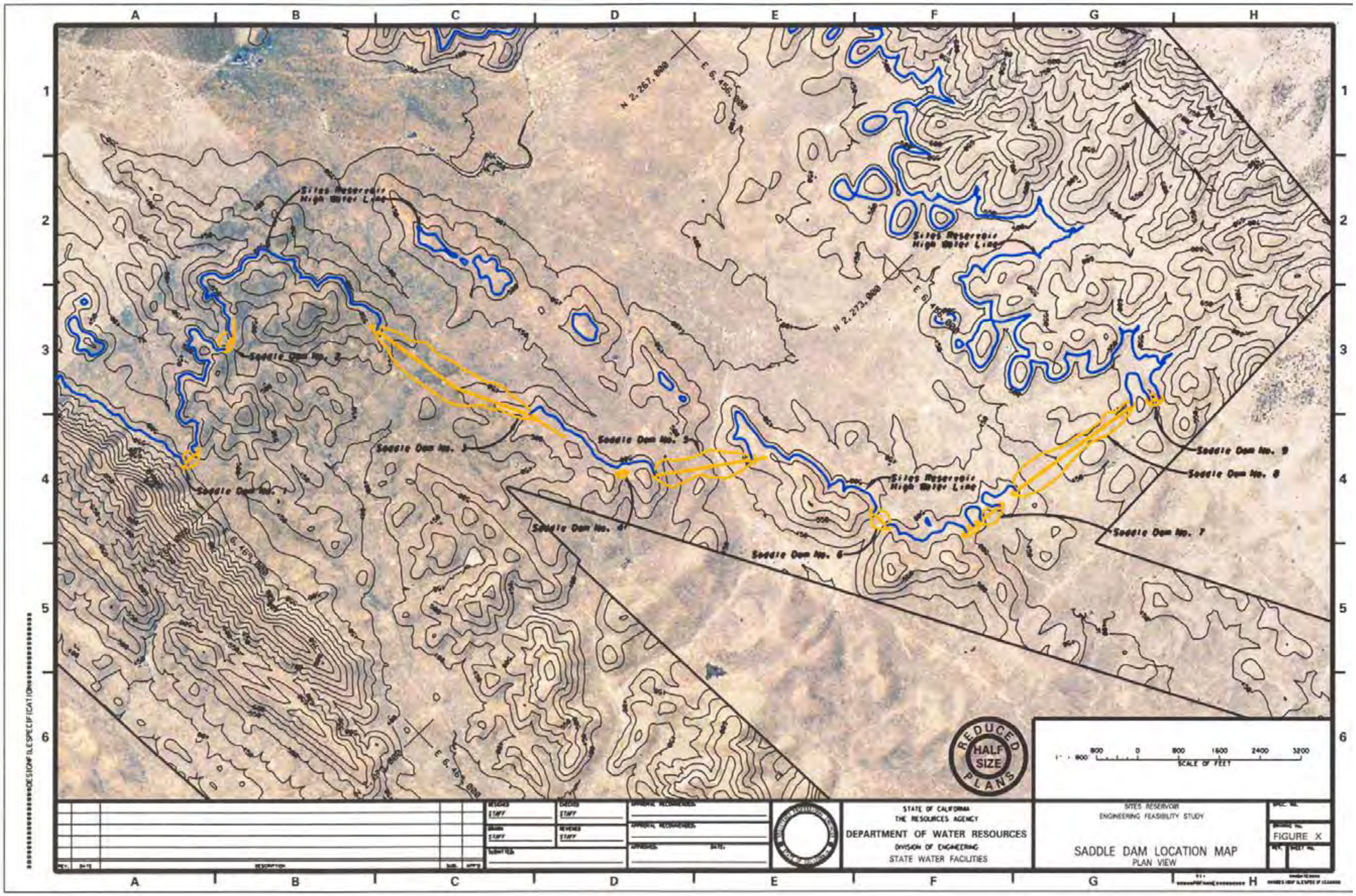


Figure B.3-9. Saddle Dam Location Map

between the Funks Creek and Hunters Creek watersheds. In general, saddle dam locations were selected to coincide with the northern ridge, as much as practicable, to minimize the creation of dead storage. Saddle dam alignments were selected using general design considerations listed below:

- Conform to site topography
- Provide favorable abutment contacts
- Minimize the amount of excavation within the core footprint to provide an approximately level foundation transverse to the dam centerline
- Minimize the amount of embankment materials
- Presence and orientation of foundation defects, such as faults

Dam Sections

Because the topography of the sites and available construction materials are similar for all of the saddle dams, two typical dam sections were developed based on the height of the maximum WSE relative to the ground surface elevation at the downstream toe of the saddle dam. The proposed sections are identified in this report as small- and medium-sized saddle dams. Both the small and medium saddle dam sections would have a crest elevation of 540 feet, a crest width of 20 feet, 20 feet of freeboard above the maximum normal reservoir elevation, and upstream and downstream slopes of 3H:1V and 2.5H:1V, respectively. These slopes were selected using engineering judgment and verified by performing preliminary feasibility level stability evaluations.

The primary difference between small and large saddle dam sections is that large saddle dams include a larger zone of rockfill material on the upstream slope to provide improved embankment stability during drawdown of the reservoir. Figure B.3-10 shows the proposed dam embankment section for the small saddle dams (Saddle Dams 1, 4, and 9). The dam embankment is a zoned earthfill consisting of a central impervious core with flanking upstream and downstream zones of random shell material comprised predominately of mudstone. Downstream of the core, a 10-foot-wide zone of filter material is conservatively included to act primarily as a crack stopper. The downstream embankment section also incorporates a 3-foot-thick blanket drain on the foundation surface as a conservative measure against potential seepage through defects in the foundation.

A 10-foot-wide zone of riprap is included for upstream slope protection. Placement of riprap slope protection on the downstream slope was not considered necessary because the random shell materials are anticipated to be fairly plastic and resistant to surface erosion from rainfall runoff. Consistent with typical designs for similar type dam embankments, the upstream slope of the core was selected to be 1H:1V, and the downstream slope is vertical to simplify construction of the adjacent filter zone.

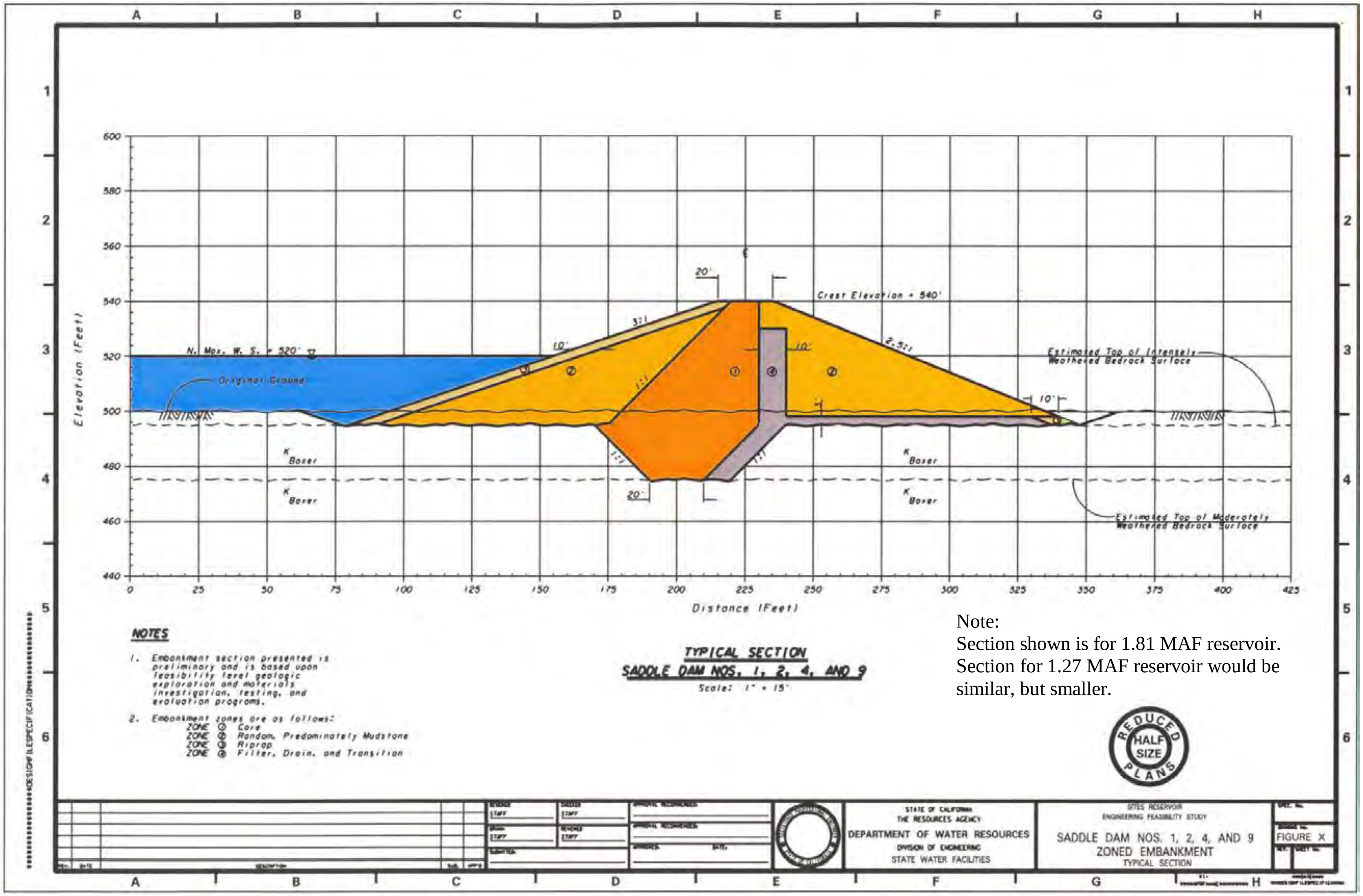


Figure B.3-10. Sites Reservoir Saddle Dams 1, 2, 4, and 9 – Zoned Embankment – Typical Section

Figure B.3-11 shows the proposed dam embankment section for the medium saddle dams (Saddle Dams 3, 5, 6, 7, and 8). The dam embankment is zoned earthfill consisting of a central impervious core with upstream and downstream zones of random shell material and an upstream rockfill zone included for improved stability and slope protection. Downstream of the core a 10-foot-wide zone of filter material is included to control embankment seepage. The downstream embankment section also incorporates a 5-foot-thick blanket drain, comprised of filter and drain materials, to control foundation seepage and provide a horizontal conduit for seepage collection at the downstream toe. A 10-foot-wide layer of riprap has been included to provide downstream slope protection. Similar to the small saddle dam section, the upstream slope of the core was selected to be 1H:1V, and the downstream slope is vertical to simplify construction of the chimney filter.

Embankment Materials

Selection of embankment sections for the saddle dams was based on the availability of on-site materials identified and evaluated as part of the materials investigation program. A summary of materials designated for use in specific embankment zones is discussed below:

Zone 1: Impervious core material comprised of low to medium plasticity clays, with lesser amounts of high plasticity clays, and clayey sands. The impervious material would be obtained from designated borrow areas upstream of the saddle dams in the reservoir with haul distances of less than 1 mile. Impervious material processing beyond normal disking and moisture conditioning in the designated borrow areas would not be required.

Zone 2: Random shell material comprised predominately of mudstone from the Boxer Formation. Random material would be obtained from designated borrow areas upstream of the saddle dams in the reservoir and from required foundation excavation for the dam embankments with haul distances of less than 1 mile. Processing of the random shell materials would not be required.

Zone 3: Rockfill and riprap consisting of processed clean rock up to 30-inch maximum particle size. The rockfill and riprap would be obtained from fresh Venado sandstone of the Cortina Formation from a quarry developed in the ridge on the east side of the reservoir approximately 3 to 4 miles from the saddle dam sites. Quarry operations would require drilling and blasting with selective processing to produce the required particle sizes and gradation.

Zone 4: Filter and drain materials consisting of sand and gravel processed to various sizes to meet filter compatibility and hydraulic conductivity requirements. Similar to Golden Gate and Sites Dams, it is assumed that this material would be imported from the old, abandoned channel on Stony Creek between Orland and Willows, approximately 30 road miles from the saddle dam sites.

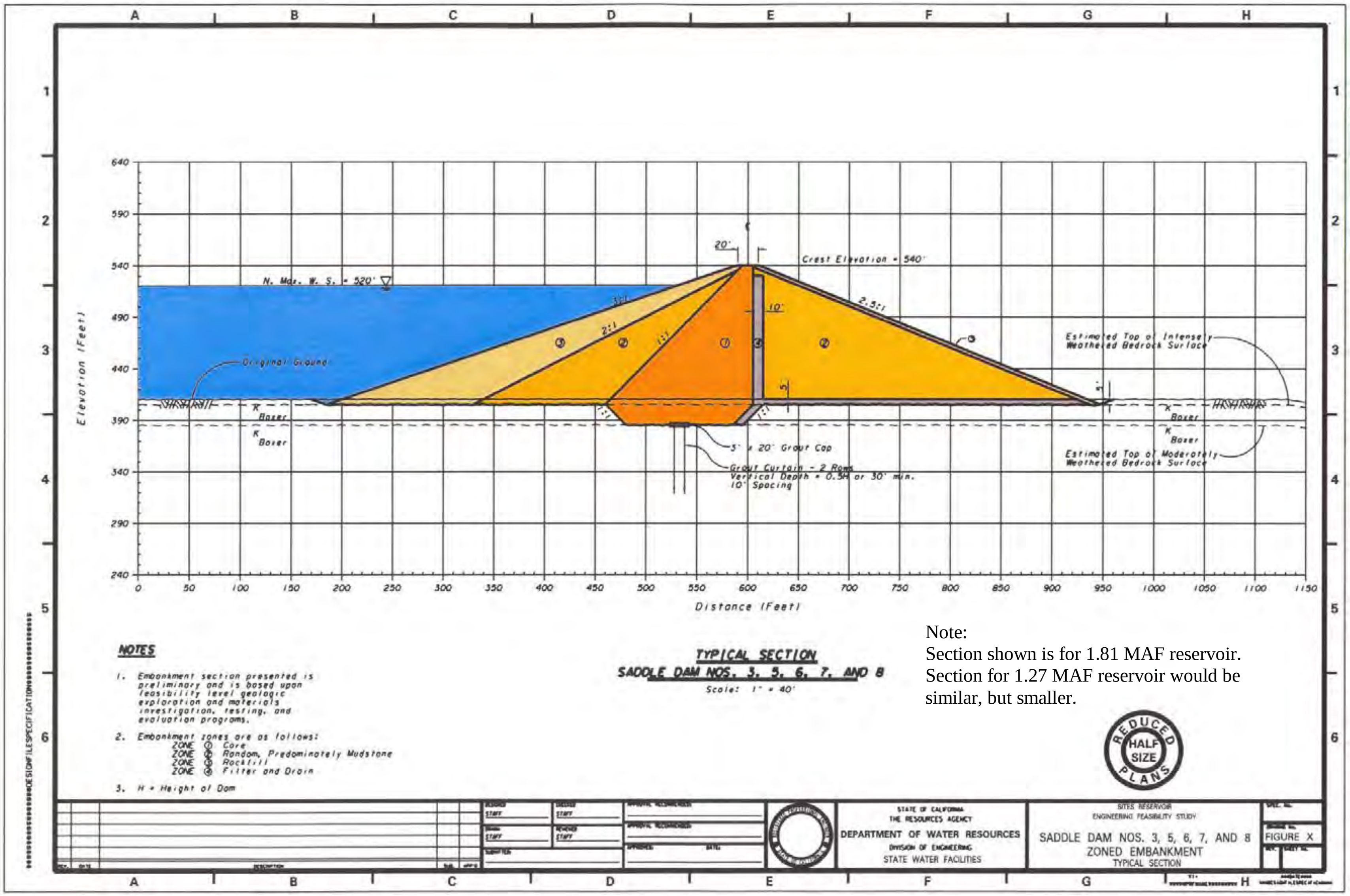


Figure B.3-11. Saddle Dams 3, 5, 6, 7, and 8 – Zoned Embankment – Typical Section

Foundations

Bedrock underlying the saddle dam footprints is predominately mudstone with siltstone, sandstone, and a conglomerate of the Boxer Formation. Because the saddle dams are founded on the same geologic unit and geologic information is not available at all at the saddle dam sites, preliminary foundation design for the saddle dams was performed by reviewing available geologic information and adopting uniform foundation objectives, excavation, and treatment for all of the saddle dams. Based upon geologic characterization and visual observation of limited amounts of drill core, intensely weathered bedrock is considered a suitable foundation surface for the shell, random, filter, and drain. In addition, moderately weathered bedrock is considered a suitable foundation surface for the central impervious core. To meet the foundation objectives, colluvium and decomposed bedrock would be excavated from the entire footprint of the saddle dams to obtain an intensely weathered bedrock surface. In addition, intensely weathered bedrock would be excavated from the impervious core footprint to obtain a moderately weathered bedrock surface. To ensure that a competent impervious barrier is obtained at the contact with the moderately weathered bedrock surface under the core footprint, a minimum bottom trench width of 20 feet was incorporated into the saddle dam foundation design.

Grouting

A review of DWR water pressure test data from drill holes in the saddle dam foundations indicates that the bedrock varies from impermeable to having a relatively high hydraulic conductivity. Within the anticipated grouting depth range approximately 50 percent of the intervals tested were characterized as fairly impermeable and approximately 30 percent of the intervals were characterized as having a relatively high hydraulic conductivity.

Because water pressure test data indicate that some areas of higher hydraulic conductivity occur in the dam foundations, curtain grouting was included in the design of the medium saddle dams to reduce seepage through the dam foundations. Grouting was not included in the foundation design of the small saddle dams because a relatively large portion of these dams is freeboard and due to the relatively low head and long flow path below the core trench. Foundation grouting would consist of a two-row vertical grout curtain spaced 10 feet apart parallel to the dam centerline. Each row of curtain grout holes would consist of mandatory, primary, and secondary holes spaced at 10-foot centers and tertiary holes split-spaced between the primary and secondary holes. Consistent with dam foundation grouting practices, the drilling depth of curtain holes was estimated to be one-half the dam height, or a minimum depth of 30 feet. In addition to the grouting program described here, additional grouting and/or treatment of special features, such as the Salt Lake Fault, would likely be required. This additional grouting and/or treatment would be examined further once additional geologic information is available.

B.3.3.7 Sites Reservoir Signal Spillway

General

Normally, an emergency spillway is required by DWR’s DSOD to evacuate the design flood inflow. In the case of an offstream reservoir that can accommodate the design flood inflow within available freeboard, such as Sites Reservoir, the emergency spillway is primarily required for the very improbable case where the SRPGP would continue pumping into the reservoir despite being at maximum pool.

Design Assumptions

Preliminary design of the Sites Reservoir emergency spillway was performed in accordance with the state of practice for dam appurtenant structure design with conformance to current dam safety criteria. The PMF peak is estimated at 8,500 cfs with a probable three-day volume estimated at 78,420 AF.

For the 1.81 MAF reservoir, the PMF-estimated volume of 78,420 AF would be retained within approximately 5.5 feet above normal maximum pool at an elevation of 525.5 feet. Placing the invert of the emergency spillway inlet at this elevation would store a PMF event without spillway flow for beneficial use even when the reservoir is at maximum normal pool elevation at the start of the storm.

For the 1.27 MAF reservoir, the PMF-estimated volume of 78,420 AF would be retained within approximately 6.25 feet above normal maximum pool at an elevation of 486.25 feet. Placing the invert of the emergency spillway inlet at this elevation would store a PMF event without spillway flow for beneficial use even when the reservoir is at maximum normal pool elevation at the start of the storm.

Design Details

The emergency spillway selected for the preliminary studies would consist of one 7-foot-diameter concrete pipe, buried in the abutment or the bottom of Saddle Dam 6. The size is selected based upon inspection and maintenance considerations, not hydraulic requirements.

For the 1.81 MAF reservoir , a morning glory spillway would be provided on a cut bench on the left abutment of the dam, as shown on Figure B.3-12. The outlet pipe would run under the dam on a cut bench on the dam abutment foundation. On the downstream side of the dam, the pipe would run down slope to the creek. An energy dissipating structure would be located at the end of the pipeline to control the discharge of water to the creek. Even though no outflows of significance are expected, the energy dissipating structure would be sized for a flow of approximately 700 cfs, which is the maximum expected outflow if the reservoir water level should approach the crest of the dam.

For the 1.27 MAF reservoir, there would be only a minimal dam at the Saddle Dam 6 site because the ground level of the saddle is approximately at elevation 500 feet, 20 feet above the reservoir maximum operating level. However, it is anticipated that a core trench backfilled with clay would be required across the saddle to control through seepage when the water is at or above the maximum operating level. The spillway at this location would include an excavated entry channel, a pipe through the saddle (and core trench), and an energy dissipating structure at the downstream end of the pipeline, as shown on Figure B.3-13.

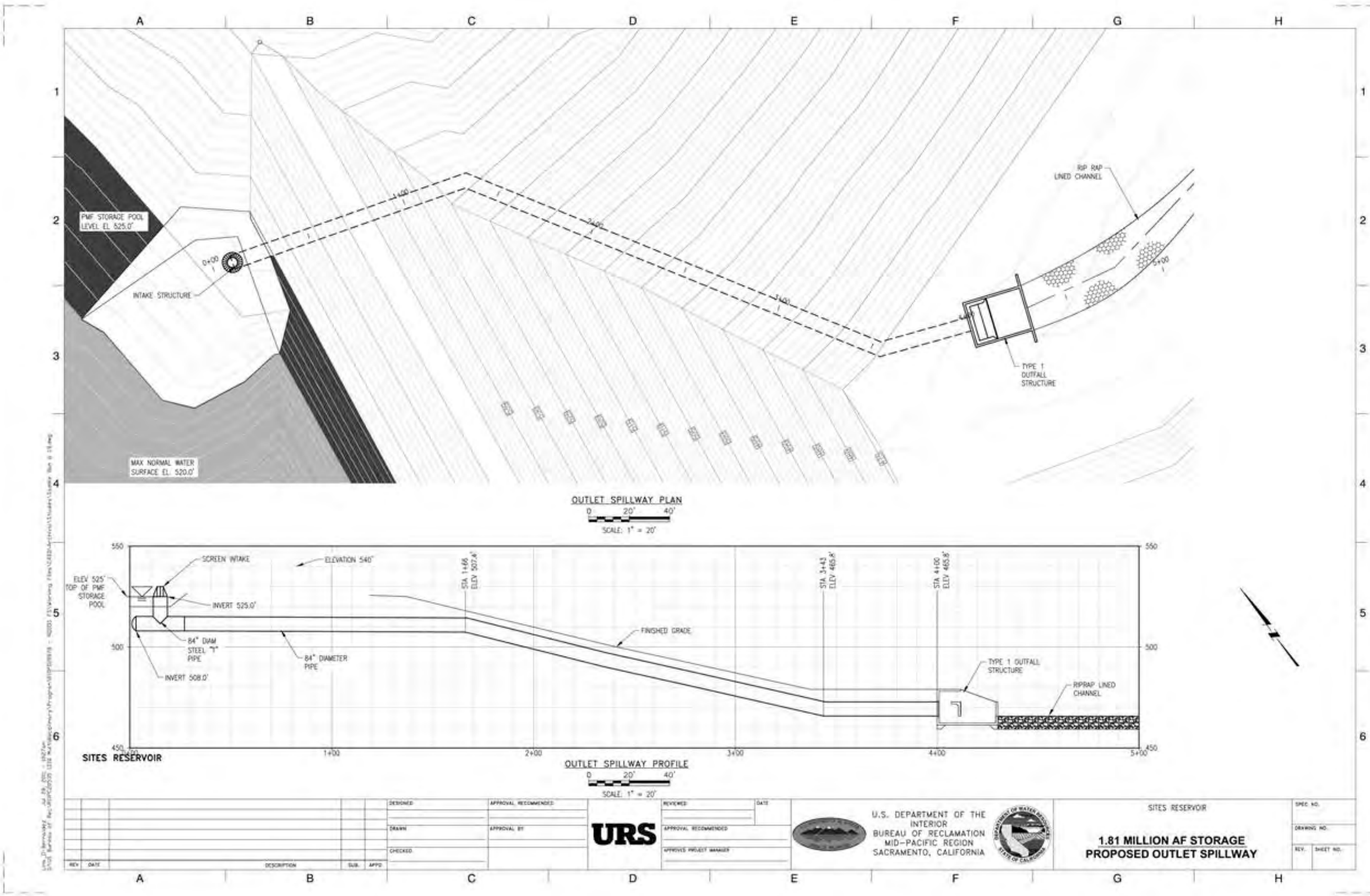


Figure B.3-12. Signal Spillway – Saddle Dam 6 – 1.81 MAF Reservoir – Plan and Elevation

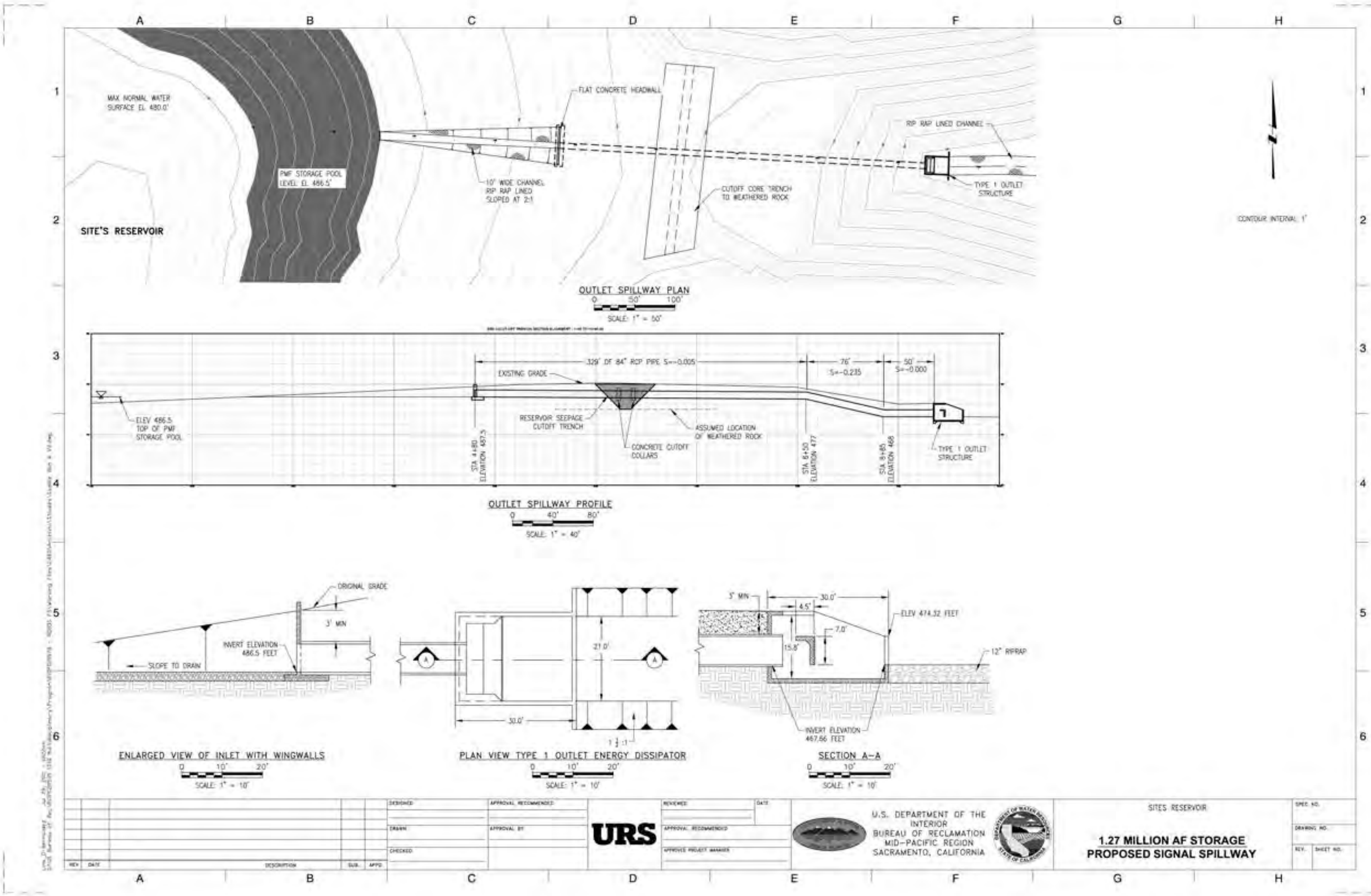


Figure B.3-13. Signal Spillway – Saddle Dam 6 – 1.27 MAF Reservoir – Plan, Elevation, Sections

Even though no outflows of significance are expected, the energy dissipating structure would be sized for a flow of approximately 600 cfs, which is the maximum expected outflow if the reservoir water level should approach the crest of the dam.

B.3.3.8 Sites Reservoir Inlet/Outlet Works

Options Evaluated, Selected Option

In the detailed technical report covering preliminary design of the Sites Reservoir appurtenant facilities, two options were evaluated for the inlet/outlet works: a low-level inlet/outlet with gate shaft, and a multi-level inlet/outlet. The two options provide different benefits, such as variability of the release water's temperature and dissolved oxygen content of the outflow. A general description of the two options is summarized below:

- Option 1 – Low-level inlet/outlet structure with a gate shaft for an emergency fixed-wheel gate.

- Option 2 – Low-level inlet/outlet structure for emergency drawdown plus a multi-level valved inlet tower and shaft with an emergency fixed-wheel gate.

Estimates developed for Options 1 and 2 indicate comparable costs for each.

Releases made from the low-level outlet included in Option 1 would come from the bottom of the reservoir pool, resulting in cold water outflows. Whereas, the multi-level tower included in Option 2 would provide flexibility in temperature control and dissolved oxygen content of the reservoir releases, which may provide benefits to downstream agricultural and environmental water users. Because the costs of Options 1 and 2 are similar and Option 2 provides increased flexibility, Option 2 is the preferred alternative and the only one carried forward for further evaluation and costing. Option 2 details are described below.

General

The reservoir inlet/outlet works would consist of a multi-level, valved inlet/outlet tower and gate shaft, a 30-foot-diameter pressure tunnel approximately 4,000 feet long, with bifurcated piping at the SPGP to the emergency drawdown bypass piping and energy dissipation chambers, as well as to the pumping-generating units. Under normal operating conditions, the reservoir outflows will pass through the plant units to generate electricity. In the event of an emergency requiring a rapid drawdown of the reservoir pool, the reservoir outlet works would be operated by bypassing the plant and directing the outflow to the energy dissipation valves. The proposed size of the inlet/outlet works is controlled by the flow capacity required to make emergency reservoir drawdown releases, discussed in further detail in subsequent sections. Figure B.3-14 presents a plan view of the inlet/outlet works. Figures B.3-15 and B.3-16 show profiles for the inlet/outlet works and tunnel for the 1.27 MAF and 1.81 MAF reservoirs.

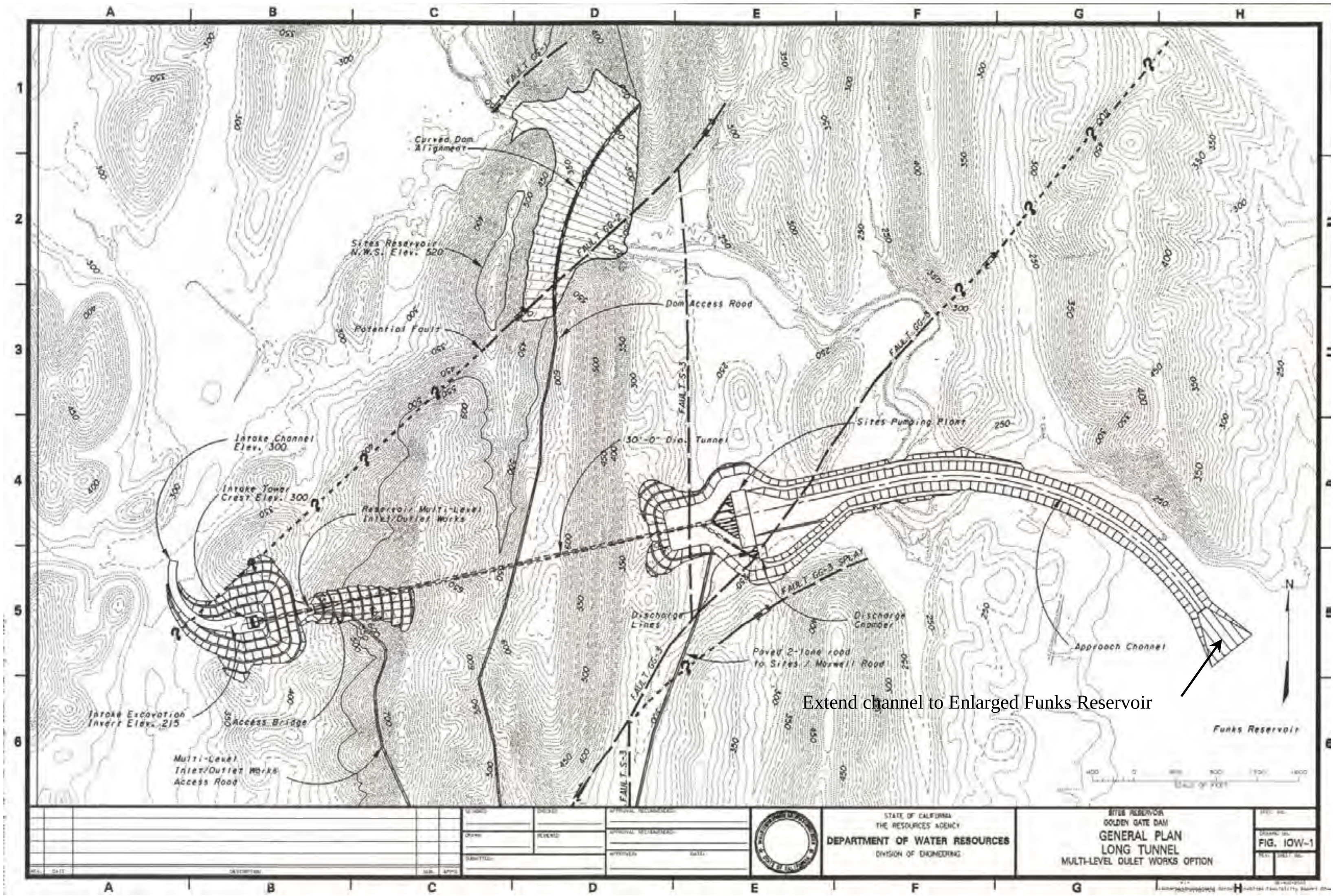


Figure B.3-14. Multi-Level Outlet Works – General Plan – Long Tunnel

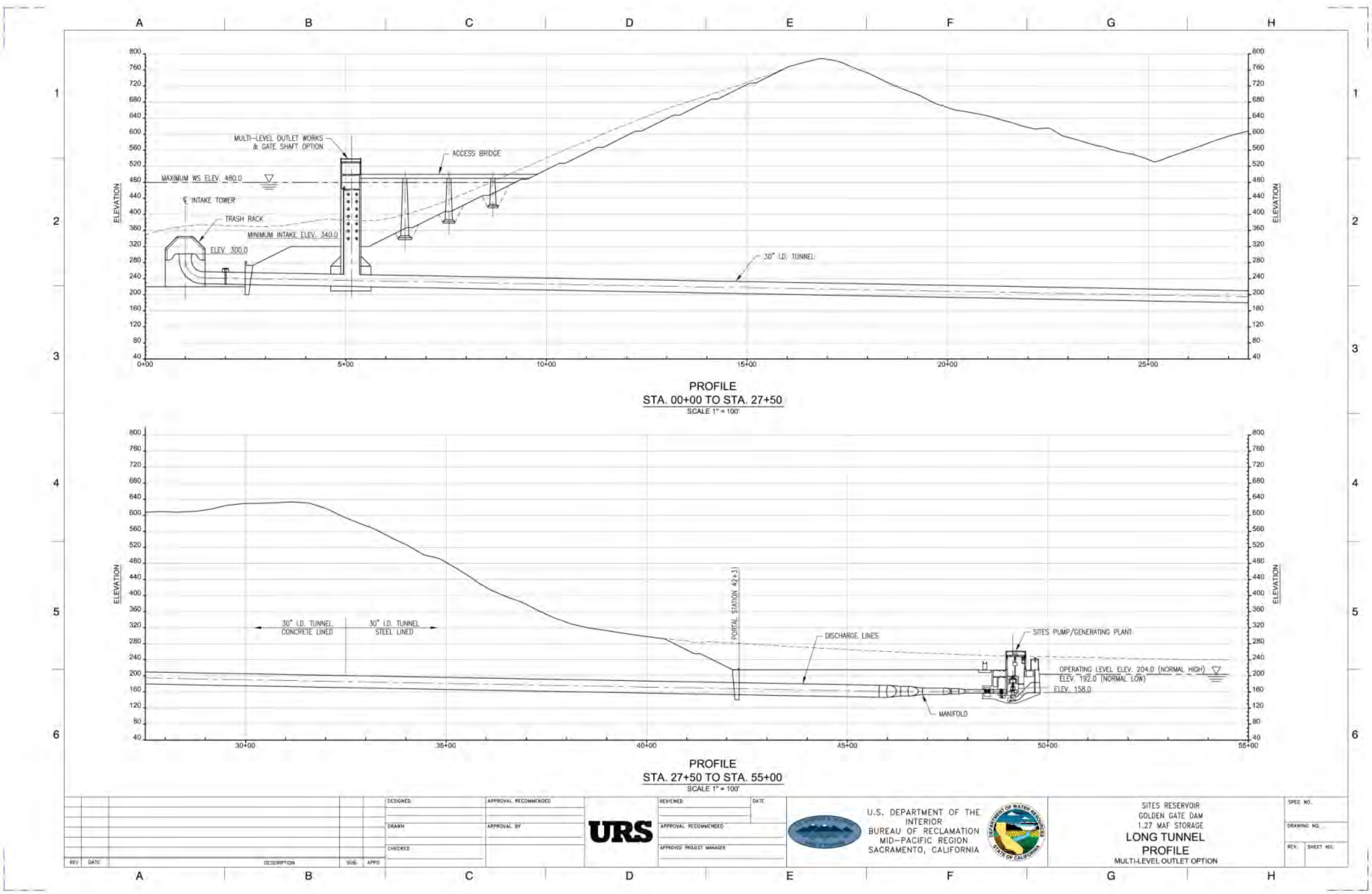


Figure B.3-15. Outlet Works – Profile – 1.27 MAF Reservoir

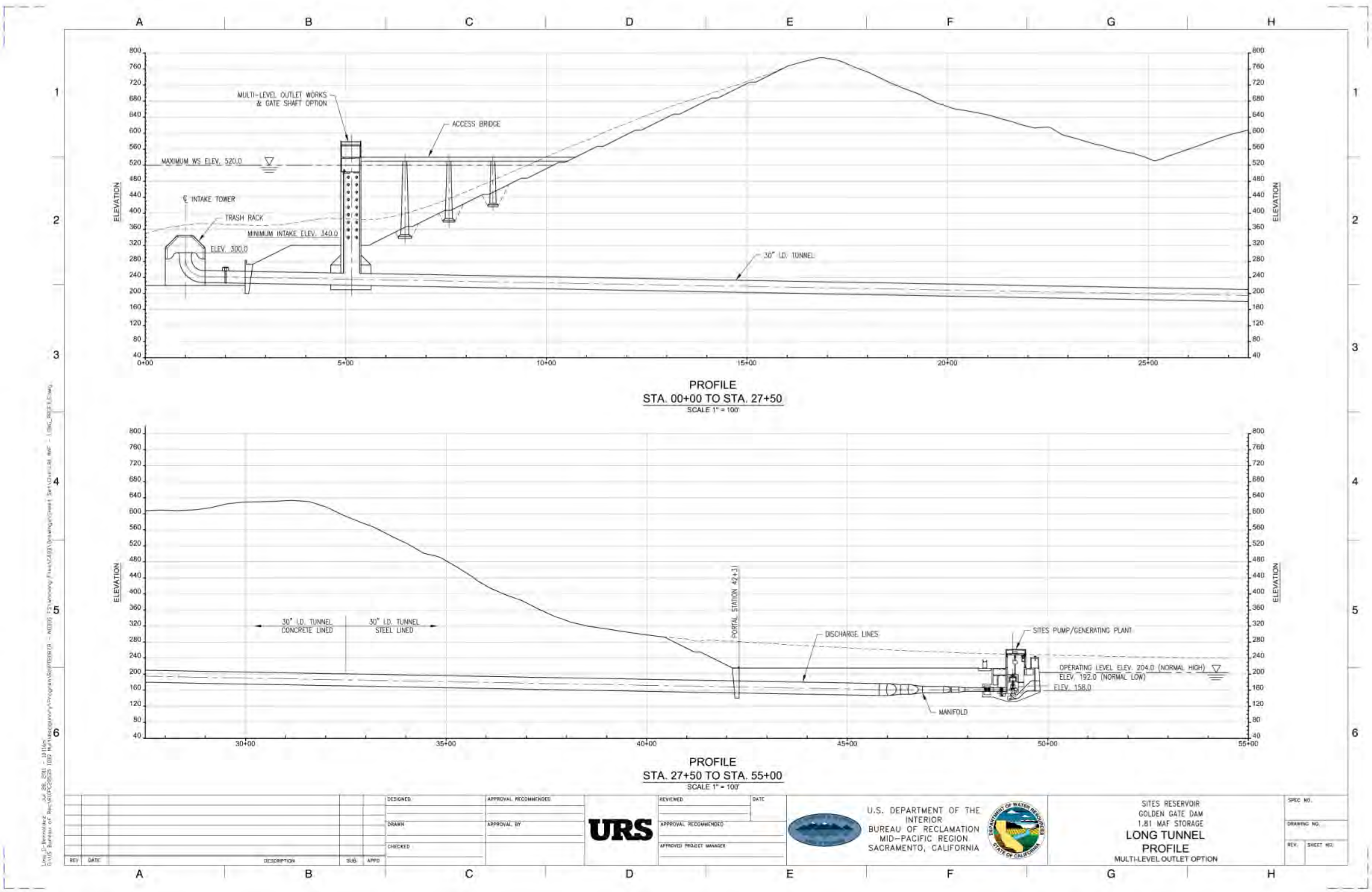


Figure B.3-16. Outlet Works – Profile – 1.81 MAF Reservoir

Design Assumptions

Preliminary design of the reservoir inlet/outlet works was performed in accordance with the state of practice for dam appurtenant structure design with conformance to current dam safety criteria.

Inlet/Outlet Tower

The multi-level inlet/outlet tower was modeled after DWR's Castaic Dam Outlet Works and has multiple inlet ports with the capability of drawing water at different levels into the reservoir. This tower would provide the flexibility to control the dissolved oxygen content and temperature of reservoir releases. The tower details would be similar for both alternative reservoir sizes currently being considered, but the tower elevations and number of inlet ports would be different. Table B.3-5 provides a comparison of tower details for the two reservoir sizes under consideration.

Table B.3-5. Sites Reservoir Inlet/Outlet Tower Consideration

	1.81 MAF Reservoir	1.27 MAF Reservoir
Top Elevation	580.0 feet	540.0 feet
Bottom Elevation (Top of Bench)	320.0 feet	320.0 feet
Inside Diameter	30 feet	30 feet
Outside Diameter	39 feet	39 feet
Number of Ports	36 (4 each at 9 levels)	28 (4 each at 7 levels)
Functional Reservoir Release Elevations	520 feet to 340 feet	480 feet to 340 feet

The multi-level inlet/outlet tower contains trashracks with outlet port controlled by butterfly valves bolted onto thimbles embedded in the tower in tiers with four valves spaced around each tier. The tower would contain movable fish screens in two tiers for operational purposes. Valves on any tier can be operated independently or all valves can be operated together. The tiers are spaced approximately 20 feet apart down the tower beginning approximately 30 feet below the maximum reservoir water level. In addition to the valved outlets, the inlet/outlet tower/shaft would also contain two 9-foot by 35-foot fixed-wheel gates that can be extended down into the outlet tunnel to isolate the tunnel, tower shaft, and discharge piping for inspection and maintenance. Figure B.3-17 presents section views of the proposed tower. A bridge provides access to the multi-level tower from the nearby access road. The bridge length varies depending on reservoir size and would have a superstructure consisting of simple spans of welded-plate girders acting compositely with a lightweight concrete deck. The girders are supported by the multi-level inlet/outlet tower, reinforced-concrete piers which are excavated into the rock foundation, and a reinforced-concrete abutment. Roadway clearance would be 16 feet between barrier railings.

The foundations of the low-level, inlet/outlet gate shaft, and multi-level inlet/outlet tower structures are in the Boxer Formation.

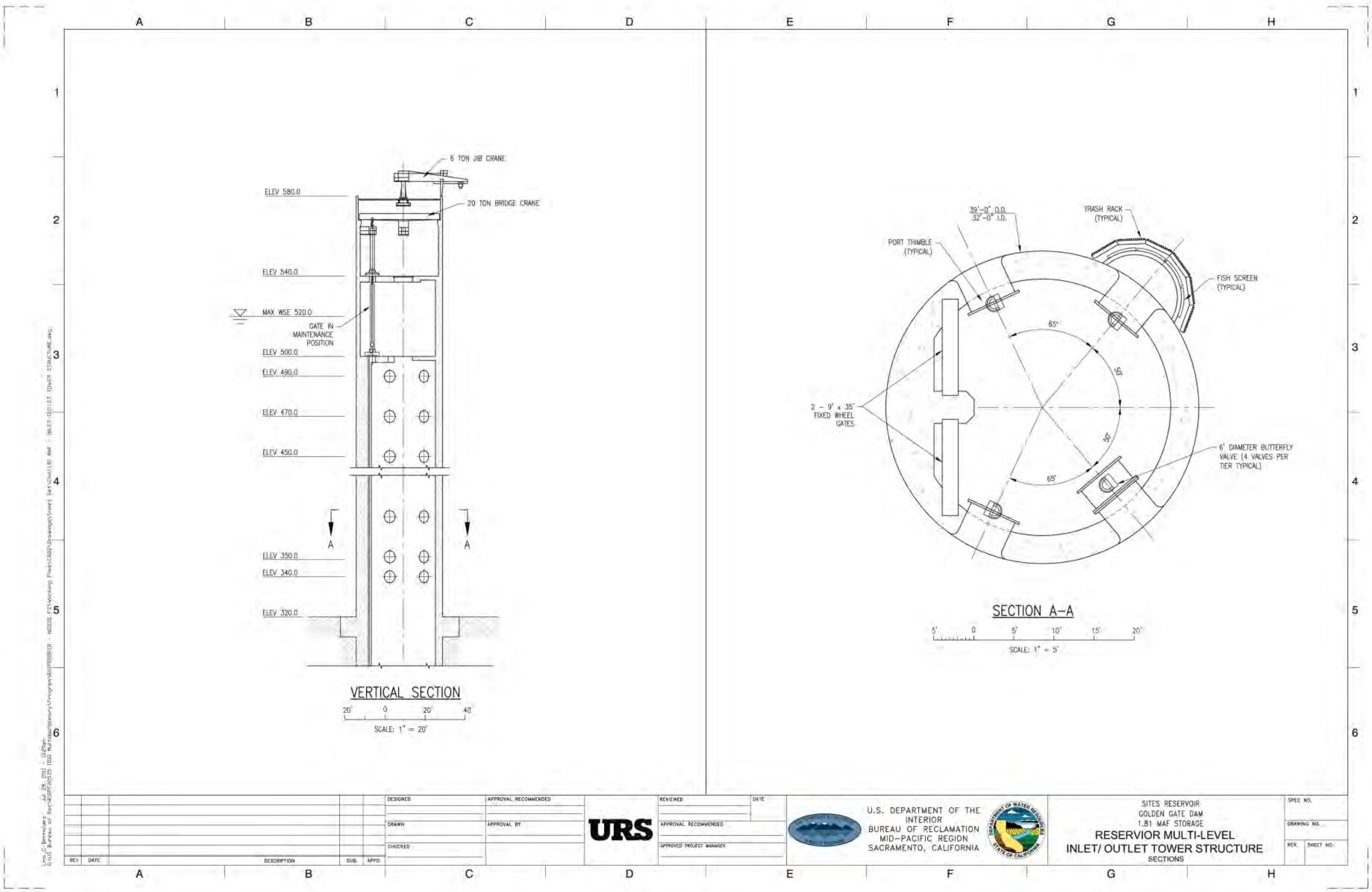


Figure B.3-17. Golden Gate Dam – Reservoir Multi-Level – Inlet/Outlet Tower Structure – Sections

During future design investigations, it is recommended that an inclined tower also be considered as an alternative to the vertical tower configuration evaluated as part of the preliminary feasibility level investigation. Final selection of the tower type should be based on an evaluation of the alternatives including economic, operational, and performance considerations.

Pressure Tunnel

Tunnel details and details of the inlet control structure to the tunnel on the bottom of the reservoir would be the same for the 1.81 MAF and 1.27 MAF reservoirs. The 30-foot-diameter tunnel, modeled after DWR's Angeles Tunnel, would convey flows through the right abutment of Golden Gate Dam. Two tunnel alignments were investigated in the detailed technical report covering preliminary feasibility design of the Sites Reservoir appurtenant facilities, but due to faulting issues, the one recommended for future study and discussed further in this report is the long tunnel alignment, approximately 4,000 feet in length. The geology of the tunnel alignment consists of the Boxer Formation for the western 1,500 feet of the tunnel, and the Cortina Formation for the eastern 2,500 feet.

The proposed tunnel size is controlled by emergency drawdown releases, which would occur through a pipe bypass to a discharge valve dissipating chamber. Maximum discharge releases of 23,000 cfs are possible with a corresponding tunnel velocity of 32.5 fps. Pumping velocities through the tunnel would be approximately 8.35 fps for the 5,900 cfs pumping plant included as part of Alternative A and Alternative C, and 5.51 fps for the 3,900 cfs plant included as part of Alternative B.

The tunnel from the upstream portal would be concrete-lined to prevent rock fallout and to ensure a smooth interior surface, thus reducing head loss and minimizing seepage into the surrounding rock. The 30-foot-diameter concrete-lined tunnel would extend from the inlet to a vertical gate shaft with a fixed-wheel gate. Downstream of the gate shaft, the 30-foot-diameter concrete-lined tunnel would continue until the depth-of-rock cover dictates use of a steel liner. Figures B.3-15 and B.3-16 present a profile view of the proposed tunnel for the two reservoir sizes under consideration.

Gate Shaft

The gate shaft would be an excavated shaft extending down from the base of the inlet/outlet tower (elevation 320.0 feet) to the invert of the outlet tunnel. The gate shaft is modeled after DWR's Angeles Tunnel gate shaft. The gate shaft would have the same inside diameter (30 feet) as the inlet/outlet tower. The shaft would intersect the tunnel in a thickened tunnel section, and the two 9-foot by 35-foot fixed-wheel gates would raise and lower within the shaft, permitting complete dewatering and inspection of the inlet/outlet works and tunnel downstream of the gate shaft. These gates also serve as an emergency shutoff device in case of a downstream tunnel collapse. Inspection of the tunnel upstream of the gate shaft would require closing the reservoir inlet/outlet low-level structure with bulkhead gates lowered into slots using a barge-mounted crane on the lake.

Penstock and Manifold

Flow in and out of Sites Reservoir from the SPGP would occur through buried steel penstocks and manifolds beginning at the downstream tunnel portal. The initial penstock diameter at the tunnel portal is 30 feet, and remains that size until the first bifurcation of the discharge line. A series of bifurcations transition the penstock to diameters of 26, 18, 13, 9, and 6.5 feet before entering the pumping-generating plant. The penstock lines were not sized by an economic analysis, but rather for pumping flow velocities of 10 fps or less.

Figure B.3-18 presents a plan view of the penstock and manifold for a 5,900 cfs SPGP.

All buried penstocks would be concrete-encased, with concrete anchor blocks to resist the thrust forces on bends, reduction bifurcations, branches, etc. A maximum 45-degree angle between the main penstock and the bifurcations should be used for optimum efficiency. The penstocks have been designed for a pressure equivalent to the full maximum static head plus 10 percent of the maximum operational load. A minimum load of 75 pounds per square inch (psi) was assumed based on the pressure head at low pool. Steel, rather than prestressed concrete penstocks, were used because a large amount of steel tunnel liner is already required and, economically, the short lengths of differing diameter penstock favor steel.

Emergency Release

Emergency release facilities were sized to meet general emergency drawdown guidelines required by DSOD. This general guideline is that large reservoir facilities should have the capability to release 10 percent of the reservoir head within 10 days. In the case of Sites Reservoir, this amount correlates to approximately 30 feet of drawdown (400,000 AF) at 20,200 cfs average outflow (maximum release capacity of tunnel and outlet works is 23,000 cfs).

The option considered would be a bypass pipeline from the 30-foot-diameter inlet/outlet works tunnel. A 26-foot-diameter pipeline would bifurcate to two 18-foot-diameter pipelines and again bifurcate to four 13-foot-diameter pipelines containing butterfly back-up valves. The pipelines would then reduce to 8-foot-diameter with fixed-cone dispersion (Howell Bunger) valves located in reinforced-concrete energy dissipation chambers. Figures B.3-18 and B.3-19 present plan and section views of the bypass piping and dissipation chambers.

It should be noted that the flow through the fixed-cone dispersion valves would be released downstream to Holthouse Reservoir and would need to be routed to prevent downstream flooding. Investigations to date have not addressed the issue of passing the emergency release past Holthouse Reservoir, but the new dam has an RCC spillway section sized to pass the emergency flow. In addition, it is recommended that the emergency drawdown

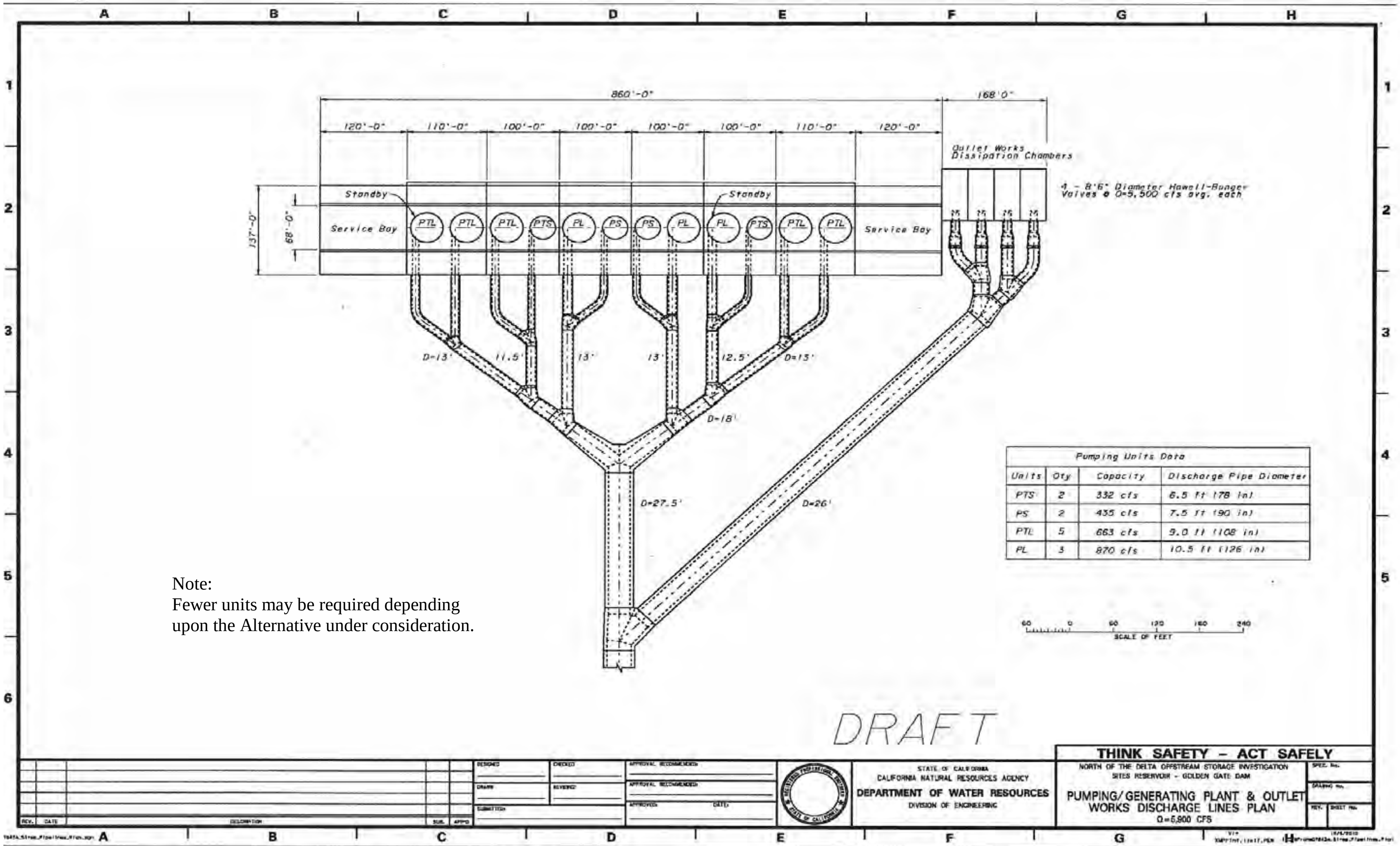


Figure B.3-18. Sites Pumping-Generating Plant – Discharge Lines Plan – Q=5,900 cfs

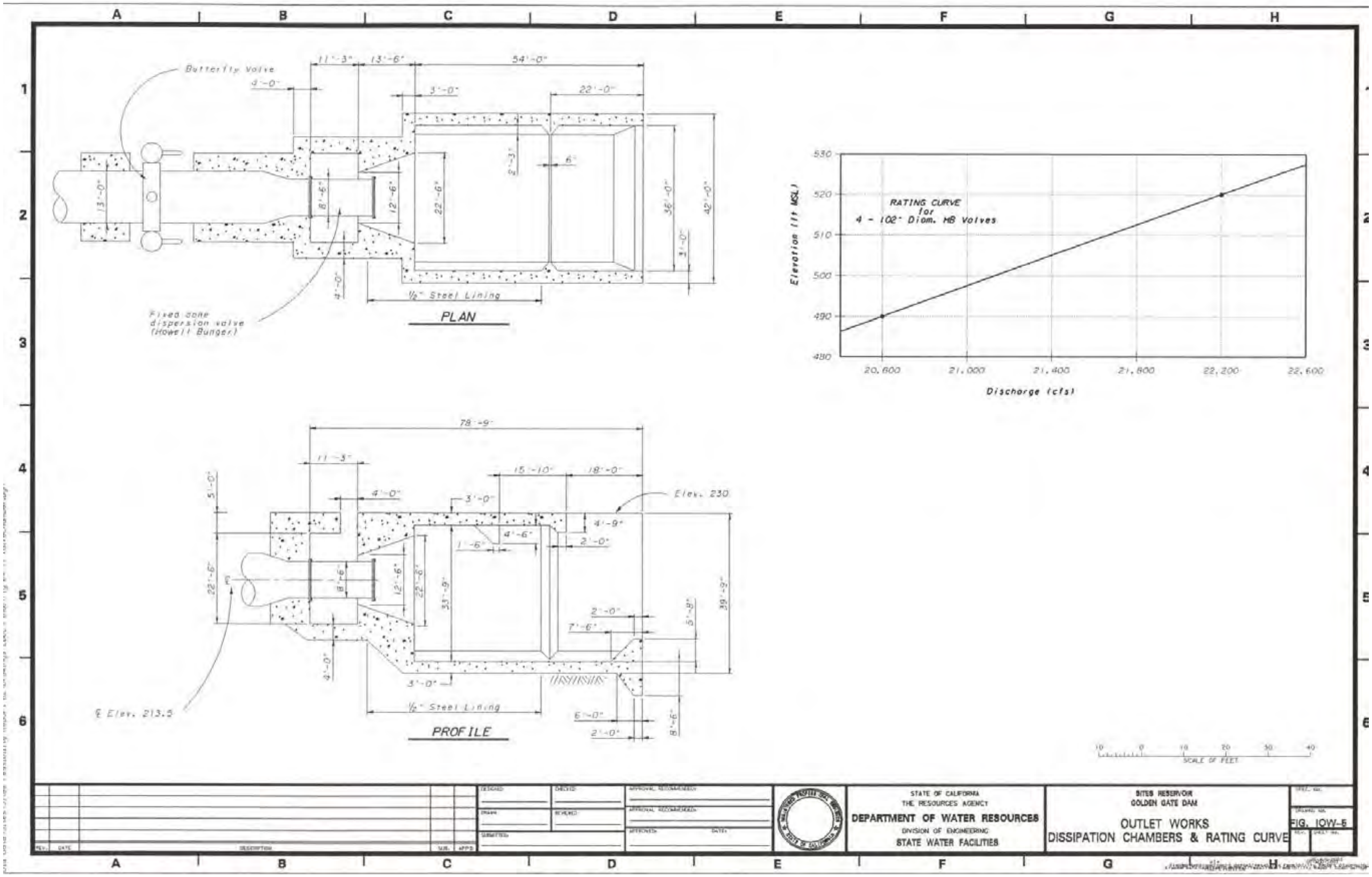


Figure B.3-19. Sites Pumping-Generating Plant – Dissipation Chambers and Rating Curve

requirements be revisited during future design investigations as the DSOD generally examines evacuation requirements on a site-specific basis for large reservoirs.

B.3.3.9 Sites Reservoir Clearing

General

Clearing within the reservoir for either reservoir size currently under consideration would require the removal of selected larger vegetation and demolition of a small number of structures used for residential dwellings and ranching operations.

Vegetation Removal

Ninety two percent of the reservoir inundation area footprint is composed of annual grasslands. As a result, clearing and grubbing would not be needed in this area. The remaining eight percent consists of blue oak woodland, agricultural crops, and other vegetation, which will be cleared. To ensure unobstructed flow through the reservoir inlet/outlet works, selected larger vegetation would need to be removed from the inundation area prior to first filling. Reservoir clearing would consist predominately of the removal of blue oak woodland for a total cleared area of roughly 1,000 acres.

Structure Demolition

The reservoir inundation area includes the small community of Sites, which has about 19 residential dwellings as well as approximately 15 scattered ranch compounds. These structures would have to be demolished and removed from the inundation area prior to first filling.

B.3.3.10 East Ridge Stability

General

Much of the reservoir's eastern rim is impounded by relatively high, steep ridges trending north-south. The stability of this natural ridge was evaluated under predicted reservoir loading. The focus of the preliminary feasibility level ridge analysis was to determine the most critical ridge section and to perform a static stability analysis to ensure the proposed size of Sites Reservoir is technically feasible.

The critical section was found by examining USGS quadrangles of the proposed reservoir complex. The steepest and thinnest ridge sections were found to be immediately south of the Golden Gate Dam site, the most critical of which was modeled. This section has slopes approximating 30 degrees with a crest at an elevation of approximately 555 feet, 35 feet above the reservoir's maximum WSE.

East Ridge Geology

Generally, the ridge on the east side of the reservoir is comprised of upper cretaceous marine sedimentary rocks of the Cortina and Boxer Formations. The rock characterized by these formations is interbedded sandstone and mudstone that strike generally north and dip approximately 50 degrees to the east. Exploratory drilling found generally good quality rock at depth with light to moderate weathering and fracturing.

East Ridge Rock Strength Estimates

Conservative strength values used in the ridge stability analysis for sandstone and mudstone were developed from unconfined compressive strength testing on samples recovered from the initial phases of the preliminary geologic exploration program, in conjunction with published data. The unconfined compressive strength results were classified by rock type, condition (dry or saturated), and level of weathering. In the case of the mudstone, some friable materials were recovered in the exploratory drilling so the assumption used in the model was that the critical failure surfaces would tend to occur along the mudstone bedding planes. In addition, some of the mudstone samples slaked (mostly parallel to the bedding plane) and could not be tested. Therefore, zero cohesion and a low-friction angle were used to model the mudstone bedding planes. Table B.3-6 provides a summary of the strengths used in the stability model.

Table B.3-6. Rock Strengths Used in East Ridge Stability Analysis

Rock Type	Cohesion	Friction Angle (degrees)
Sandstone	850 psi (122,000 psf)	35
Mudstone	132 psi (19,000 psf)	15
Mudstone (Bedding Planes)	0 psi (0 psf)	10

psi = pounds per square inch
psf = pounds per square foot

- Full Reservoir – Water surface at elevation 520 feet, phreatic surface linear from upstream water surface to downstream toe of slope.

- Partial Pool – Water surface at elevation 395 feet, phreatic surface linear from the upstream water surface to the downstream toe of slope.

- Rapid Drawdown of Reservoir – Empty reservoir with phreatic surface coincident with the upstream toe of slope to the upstream high-water mark (elevation 520 feet), then linear to downstream toe of slope.

The downstream slope was evaluated on the presumption that downstream failures would initiate along the zero cohesion, weak mudstone bedding planes. The upstream slope was modeled for two cases: failure surfaces encompassing the weak mudstone bedding planes, and failure surfaces that crossed the weak mudstone bedding planes. Figure B.3-20 illustrates a typical sliding surface evaluated as part of the stability analysis.

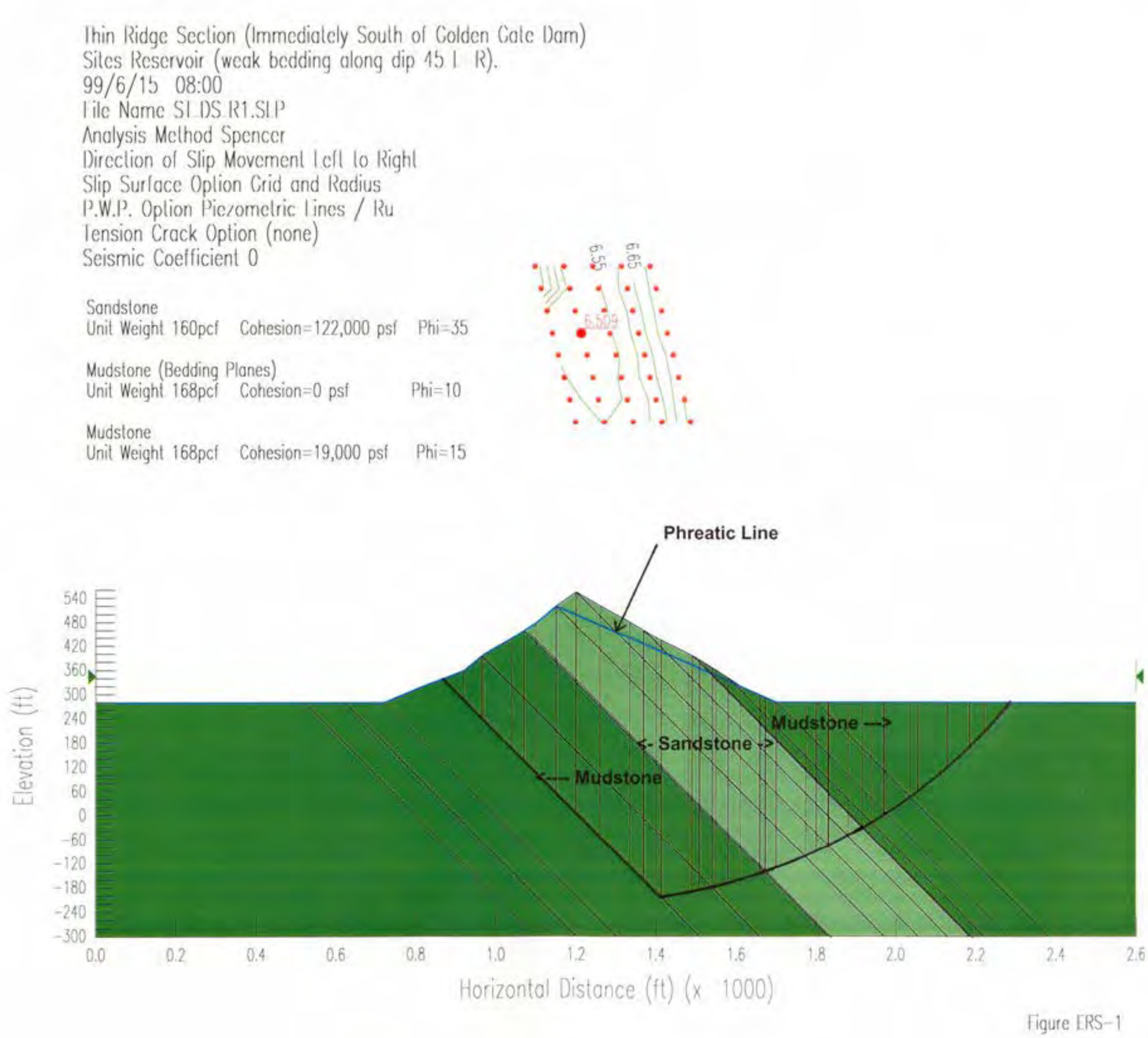


Figure B.3-20. East Ridge Section (immediately south of Golden Gate Dam)

Stability Model Results

The minimum factor of safety of the critical sliding surface was found to be greater than six for the cases examined indicating adequate stability of the east ridge under reservoir loading. Future design investigations should include an additional evaluation of the critical areas of the east ridge with particular emphasis placed on incorporating refined geologic information (as it becomes available) into the stability models in conjunction with a detailed evaluation of the effects of reservoir seepage through the ridge.

3.4 Sites Pumping-Generating Plant

B3.4.1 Introduction

The proposed SPGP would be located approximately 3,300 feet southeast (downstream) of Golden Gate Dam (Figure B.3-14). The location and layout, including the plant/control building and conveyances, were determined on the basis of hydraulic and plant equipment requirements, foundation conditions, and the orientation of local faults. The final plant location should be determined by establishing a point of economic balance between the cost of the required excavation tunnel length and discharge lines, versus the cost of long-term pumping.

The SPGP was modeled after DWR's Chrisman Pumping Plant with modifications to reflect the pumping height difference, the additional pumping units, and miscellaneous equipment needed for each alternative. The Chrisman Pumping Plant has flow capacities and head requirements similar to the SPGP design criteria (June 2003).

B.3.4.2 Design Considerations

Preliminary design of the pumping-generating plant was performed in accordance with the state of practice for pumping plants with conformance to current dam safety criteria where applicable. The SPGP was designed as a pumping-generation plant because all water stored in Sites Reservoir will have to be released back through the plant units. Making reservoir releases through the plant units provides an effective method of dissipating energy from the outflow. The configuration of the conceptual design relies upon using the generating units for ordinary releases and scheduled releases for up to six hours per day on-peak, and a plant bypass with cone valves for emergency releases.

B.3.4.3 Plant Design

The SPGP would be located on a relatively low, flat bench at an approximate elevation of 215 feet to minimize excavation volume. The materials excavated for the pumping plant foundation will consist of colluvium, underlain by weathered and fresh sandstone of the Cortina Formation. Excavation for the approach channel is expected to consist of alluvium, underlain by weathered and fresh sandstone. Both excavations are likely to encounter groundwater and require dewatering during excavation as the maximum excavation depth would be at an elevation of 144 feet, and groundwater surface elevation would be 10 to 20 feet below the original ground level.

The proposed excavation would have 2H:1V slopes, terraced with 15-foot-wide benches at 40-foot vertical intervals. In the next design phase, studies should be performed to evaluate the stability of steeper excavation slopes.

The SPGP would lift water from Holthouse (Enlarged Funks) Reservoir into Sites Reservoir. The SPGP would be connected to Holthouse Reservoir by a long, excavated approach channel. Currently, Funks Reservoir operates in coordination with the TC Canal between elevations of 203 and 205 feet. However, with the Holthouse Reservoir design, the SPGP would operate with tailwater elevations down to an elevation of 192 feet during pumping to take advantage of the full 6,500 AF active capacity of the enlarged reservoir. On the inlet side of the pumping plant connecting it to Sites Reservoir would be the 30-foot-diameter tunnel.

Because Holthouse Reservoir operates down to a lower elevation (192 feet) compared to the existing Funks Reservoir, the pumping-generating units in the SPGP need to be lowered approximately 12 to 15 feet below earlier designs to maintain the same relative submergence at the minimum design water level.

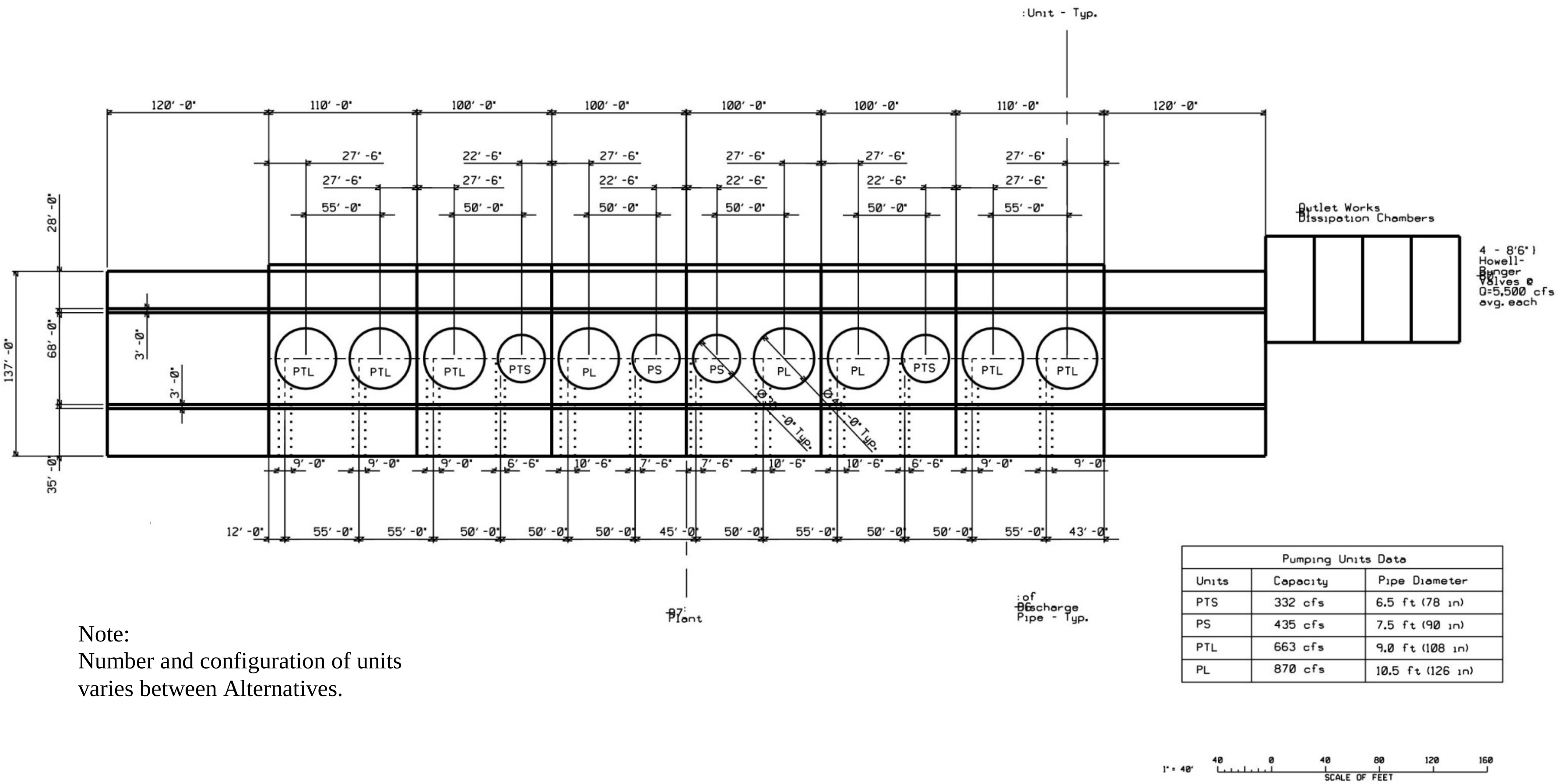
Figures B.3-21, B.3-22, and B.3-23 present conceptual details for the SPGP. Note that details will differ between project alternatives because the number of units and unit sizes may differ.

Tables B.3-7, B.3-8, and B.3-9 present a summary of the pumping and generating equipment to be provided for each of the three project alternatives. Pumping and generating power varies between alternative due to reservoir elevation and flow differences. Alternative B requires less pumping units than the other alternatives because the pumping requirement has been reduced by 2,000 cfs without a Sacramento River Pumping Plant.

Water from Holthouse Reservoir would be drawn into the pumping plant by the various pumping and pumping-generating units. The number of units operating would be selected to approximately provide the pumping capacity needed to deliver all water stored in the reservoir on a daily basis during the off-peak pumping period. The pumps would be connected to a complex intake/outflow manifold. When water is drawn out of Holthouse Reservoir and pumped up to Sites Reservoir, the pumped water would flow through successive pipe connections until all eleven pipes coming from the pump units are combined into a single 26-foot-diameter pipe. This pipe then would join the 26-foot-diameter pipe coming from the emergency bypass outlet, and the two pipes would connect to the 30-foot-diameter tunnel discussed earlier.

The pumping plant would be a conventional, indoor-type pumping-generating plant with an in-line arrangement of vertical pumping units. The SPGP would have a reinforced concrete substructure and a steel superstructure with the draft tube invert at an elevation of 170 feet. The base of the service bay foundation at the dewatering sump would be at an elevation of 160 feet, and pumping unit bays would be founded at an elevation of 144 feet. The five primary floor levels in the substructure would be:

- Ground Level Elevation 215.0 feet
- Electrical Gallery Elevation 200.0 feet
- Mechanical Elevation 200.0 feet
- Motor Floor Elevation 178.0 feet
- Suction Elbow Elevation 141.0 feet



Pumping Units Data		
Units	Capacity	Pipe Diameter
PTS	332 cfs	6.5 ft (78 in)
PS	435 cfs	7.5 ft (90 in)
PTL	663 cfs	9.0 ft (108 in)
PL	870 cfs	10.5 ft (126 in)

Note:
Number and configuration of units varies between Alternatives.

Figure B.3-21. Sites Pumping-Generating Plant – General Arrangement – Service Floor – Q=5,900 cfs

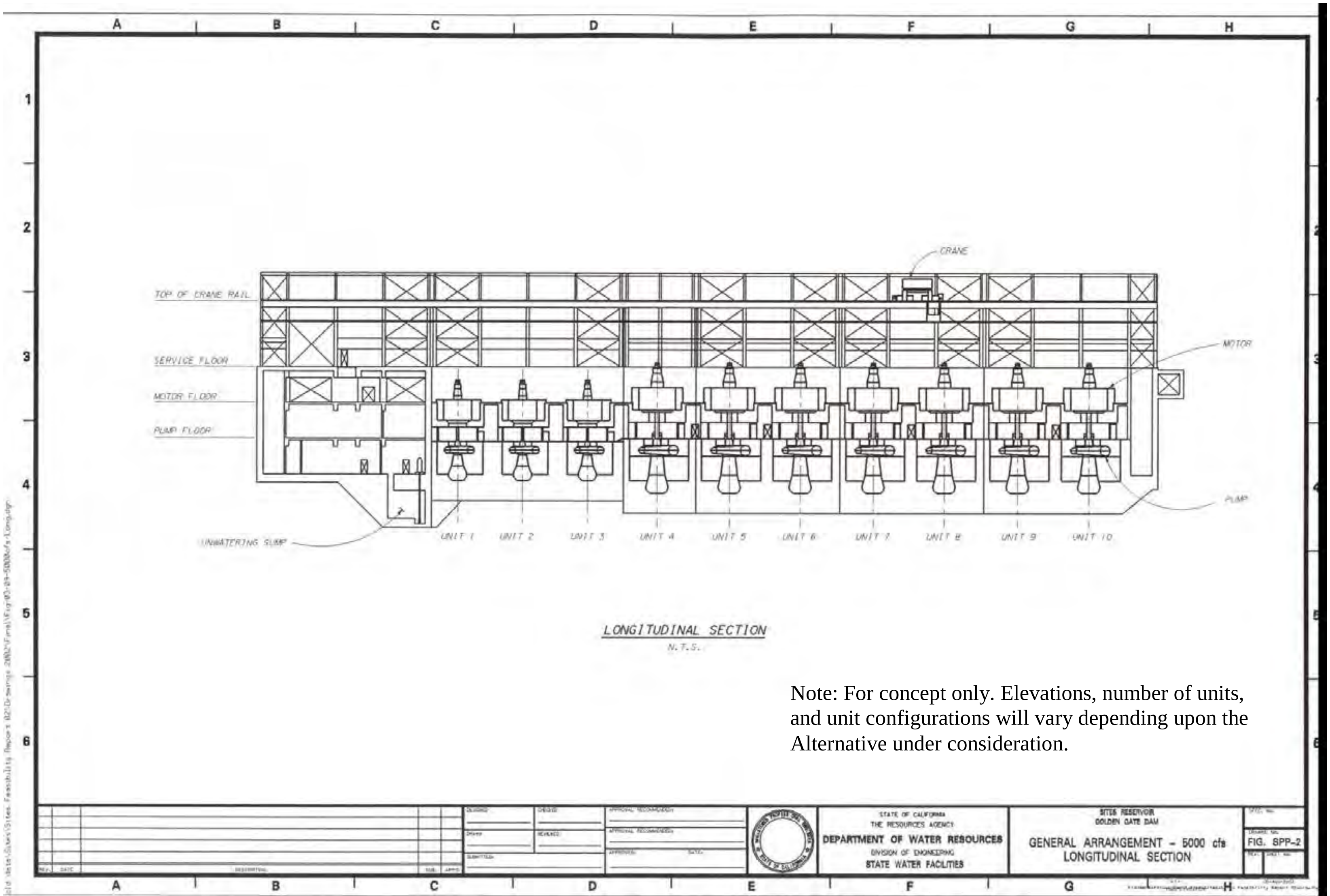
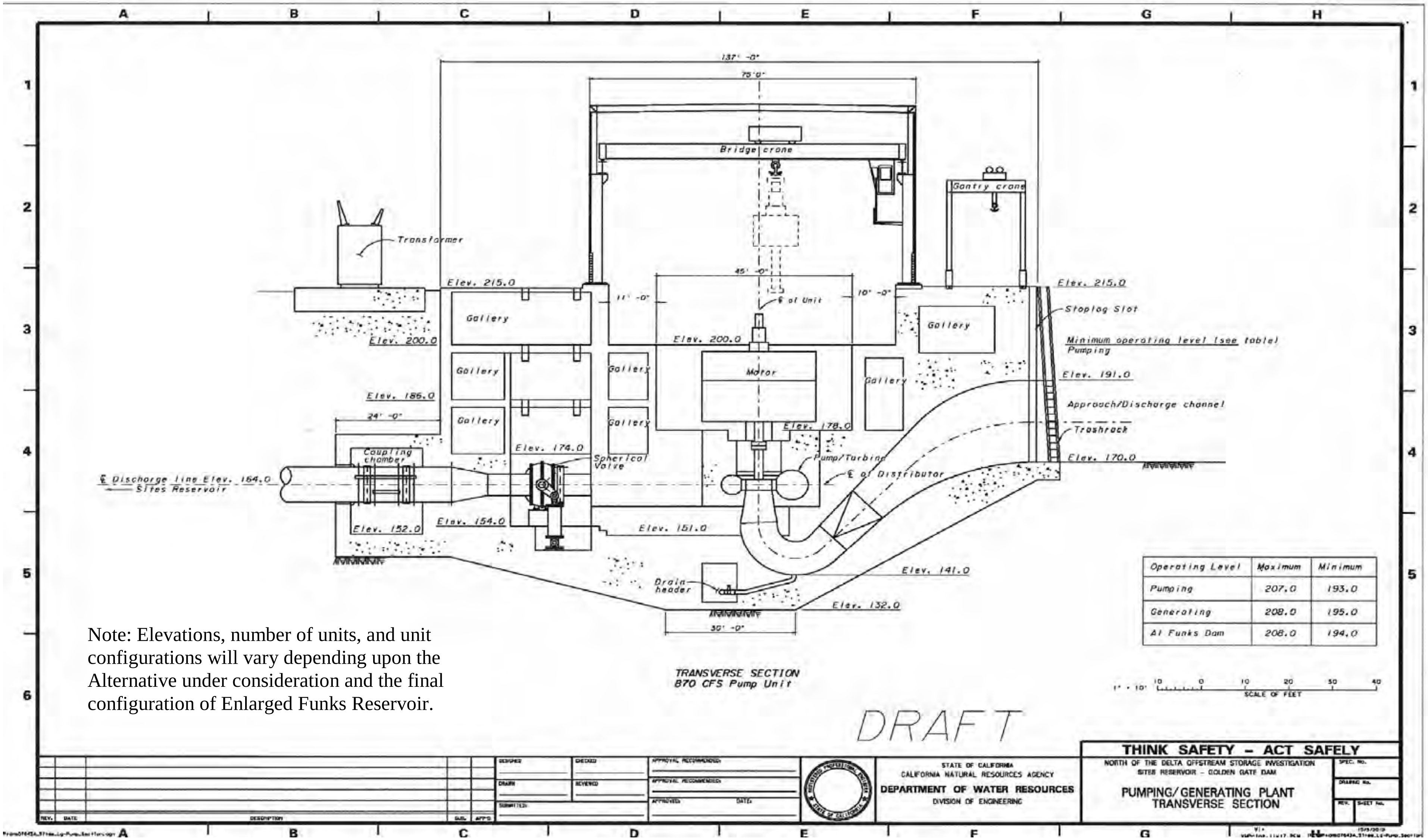


Figure B.3-22. Sites Pumping-Generating Plant – Q=5,900 cfs – Longitudinal Section



Operating Level	Maximum	Minimum
Pumping	207.0	193.0
Generating	208.0	195.0
At Funks Dam	208.0	194.0

Note: Elevations, number of units, and unit configurations will vary depending upon the Alternative under consideration and the final configuration of Enlarged Funks Reservoir.

Figure B.3-23. Sites Pumping-Generating Plant – Transverse Section

Table B.3-7. Sites Pumping-Generating Equipment for Alternative A

Unit Type	Number of Units	Net Head (feet)	Pumping Capacity Per Unit (cfs)	Generating Capacity Per unit (cfs)	Motor Power Total (MW)	Generating Power Per Unit (MW)	Total Plant Pumping Capacity (cfs)	Total Plant Generating Capacity (cfs)
Pump – Francis Vane Dual-Speed	2 (+1 Standby)	290	870	-	32.0	-	5,926	5,100
		162	870	-	17.9	-		
Pump – Francis Vane Dual-Speed	2	290	435	-	16.0	-		
		162	435	-	9.0	-		
Pump/Turbine Reversible Francis, Dual-Speed	4 (+1 Standby)	290/270	663	1,020	48.8	77.0		
		162/142	663	1,020	27.3	41.3		
Pump/Turbine Reversible Francis, Dual-Speed	2	290/270	332	510	12.2	19.3		
		162/142	332	510	6.8	10.3		

cfs = cubic feet per second
MW = megawatt

Table B.3-8. Sites Pumping-Generating Equipment for Alternative B

Unit Type	Number of Units	Net Head (feet)	Pumping Capacity Per Unit (cfs)	Generating Capacity Per unit (cfs)	Motor Power Total (MW)	Generating Power Per Unit (MW)	Total Plant Pumping Capacity (cfs)	Total Plant Generating Capacity (cfs)
Pump – Francis Vane Dual-Speed	2 (+1 Standby)	323	300	-	12.3	-	3,916	5,100
		195	300	-	7.4	-		
Pump/Turbine Reversible Francis, Dual-Speed	4 (+1 Standby)	323/310	663	1,020	54.3	87.7		
		195/182	663	1,020	32.8	51.5		
Pump/Turbine Reversible Francis, Dual-Speed	2	323/310	332	510	13.6	22.0		
		195/182	332	510	8.2	12.9		

cfs = cubic feet per second
MW = megawatt

Table B.3-9. Sites Pumping-Generating Equipment for Alternative C

Unit Type	Number of Units	Net Head (feet)	Pumping Capacity Per Unit (cfs)	Generating Capacity Per unit (cfs)	Motor Power Total (MW)	Generating Power Total (MW)	Total Plant Pumping Capacity (cfs)	Total Plant Generating Capacity (cfs)
Pump – Francis Vane Dual-Speed	2 (+1 Standby)	330	870	-	36.4	-	5,926	5,100
		202	870	-	22.3	-		
Pump – Francis Vane Dual-Speed	2	330	435	-	18.2	-		
		202	435	-	11.2	-		
Pump/Turbine Reversible Francis, Dual-Speed	4 (+1 Standby)	330/310	663	1,020	55.5	87.7		
		202/182	663	1,020	34.0	51.5		
Pump/Turbine Reversible Francis, Dual-Speed	2	330/310	332	510	13.9	22.0		
		202/182	332	510	8.5	12.9		

cfs = cubic feet per second
MW = megawatt

Each bay would be a structurally independent monolith, separated from adjacent bays by expansion joints. These joints would be keyed together to prevent transverse and vertical differential movement between bays while allowing unrestricted movement in the longitudinal direction. Shear keys between all bays would also help prevent differential movements during earthquakes.

The SPGP would be equipped with cranes to facilitate operation and maintenance of the plant. There would be a 100-ton capacity indoor bridge crane for assembly and maintenance of pumping-generating units and associated equipment. A 50-ton capacity outdoor traveling gantry crane would be installed for assembly and maintenance of butterfly valves. In addition, a 10-ton capacity, outdoor, traveling gantry crane would be installed to aid in the installation and removal of inlet gates and trashracks.

B.3.4.4 Reverse Flow to Sacramento River

For all three project alternatives under consideration, water stored in the Sites Reservoir will be released back to the Sacramento River from the Holthouse Reservoir using the Delevan Pipeline. This conveyance was selected based upon previous evaluations of three alternatives. Because of the available head, releases through the Delevan Pipeline can be made to the river by gravity without the need for pumping.

The Delevan Pipeline is buried between Holthouse Reservoir and the Sacramento River and is composed of two 12-foot-diameter reinforced concrete pipes with a return flow capacity of 1,500 cfs at a velocity of approximately 6.6 fps. For Alternative A and Alternative C, release flows would pass through the SRPGP and the excess head in the system would be used to generate hydroelectric power. For Alternative B, there would be no pumping-generating plant at the river and the excess head would be dissipated through energy dissipating valves before being released to the river.

B.3.5 Holthouse (Enlarged Funks) Reservoir

B.3.5.1 Existing Funks Reservoir

Location

The existing Funks Reservoir is located on Funks Creek approximately 7 miles northwest of Maxwell. This reservoir, constructed in 1975 by Reclamation, had an approximate active storage capacity of 2,250 AF and covered a surface area of 232 acres measured at an elevation of 205 feet. An earthfill dam with a crest elevation of 214 feet impounds the reservoir on the east. Water enters the existing reservoir from the canal on the north side and is releases to the downstream extension of the TC Canal by gravity through an outlet control structure on the south side. The reservoir is used as a regulating reservoir for the TC Canal and, in accordance with information received from TCCA, the preferred operating water level range in the reservoir is between 200 and 205 feet. However, releases still could be made to the downstream canal at water levels down to an elevation of 198.0 feet.

Flood Flows

Flood flows in Funks Creek must pass through Funks Reservoir in the winter. Funks Reservoir includes a gated, reinforced concrete spillway with three 25-foot by 20-foot radial gates to pass these flows. The gate sill (bottom) is located at an elevation of 186.0 feet. TCCA operates the spillway gates. The spillway discharge capacity is approximately 23,000 cfs with all gates fully open at the maximum design water surface at an elevation of 206.5 feet (based upon spillway rating curve on the design drawings).

Sediment Accumulation

Because it is an on-stream reservoir, a significant portion of the reservoir active storage has been lost to sediment accumulation from Funks Creek. While topographic data is available for the reservoir from the original construction drawings, there is no current bathymetric data to support an estimate of the amount of sedimentation that has actually accumulated. However, it is believed that the current active capacity could be as low as 1,500 AF. This decreased capacity would mean that approximately 750 AF, or 1.2 million loose cubic yards of sediment has accumulated. A bathymetric survey of the exiting reservoir should be performed as part of future design phases of the project to establish the volume and physical characteristics of the sediment so the material can be properly managed during design and construction.

A large portion of the accumulated sediment may have to be removed and relocated to construct the new Holthouse Reservoir, in particular the low-level flow channel connecting the reservoir with the SRPGP. Once a diversion system is installed to route Funks Creek flows around the Holthouse Reservoir work site, the sediment can be dewatered over a period of time by ditching and sumping. Once dry enough to be excavated and moved, the material can be disposed of in the lower elevations of the new Holthouse Reservoir in a dead storage area or in backwater areas around the perimeter of the existing reservoir. The construction schedule for the project should allow adequate time to dewater and remove the material without affecting the new dam construction (which is outside the limits of sediment accumulation)

B.3.5.2 Holthouse (Enlarged Funks) Reservoir

Need

Holthouse Reservoir is required for the NODOS project to facilitate balancing and regulating Sites Reservoir inflows and outflows through the SPGP and to provide sufficient supplemental storage to allow simultaneous pump back power generation on demand for up to six hours per day. During fall and winter months, inflows from the conveyance system and water for power generation would be stored during on-peak power periods. The stored water plus ongoing off-peak inflows from the conveyance systems would then be pumped to Sites Reservoir during the partial-peak/off-peak power period on a daily basis. During the spring and summer months when releases are being made from Sites Reservoir, released water would be receiving and distributing project flows. This section discusses the preliminary feasibility design of modifications necessary to enlarge Funks Reservoir to provide increased storage capacity for operation of the conveyance system and regulation of flows for the proposed SRPGP.

Design Considerations

Figures B.3-24, B.3-25, and B.3-26 show preliminary feasibility level design details for Holthouse Reservoir. Details include the following:

- Constructing a new dam east of the existing Funks Dam and breaching the existing dam so that a larger composite reservoir is formed with an active storage capacity of approximately 6,500 AF. The combined surface area for the new reservoir will be approximately 530 acres measured at the maximum water storage level.

- Constructing an inlet/outlet works as part of the new dam so that all water entering or leaving the project passes through the new reservoir.

- Constructing a flow channel from the new reservoir back to the SPGP so that pumping and generating can be performed within the full limits of the active storage within the reservoir.

Holthouse Dam - General

The new dam would include two sections, a combination concrete and RCC dam section near the center that accommodates the inlet/outlet facilities for the Delevan and TRR Pipelines and earth embankments on either side of the concrete dam to close off the valley and form the reservoir. The total length of the dam would be approximately 7,800 feet. The RCC Dam component would be approximately 400 feet long. Maximum dam heights would be approximately 45 feet. The crest elevation of the dam would be at an elevation of 214 feet to match the crest of the existing Funks Reservoir Dam and the surrounding topography.

A grout curtain will be installed under both dam sections to control underseepage.

RCC Dam and Spillway Section, Spillway Design

The RCC Dam section is provided near the center of the dam to serve as the support structure for the inlet/outlet pipes for the Delevan and TRR Pipelines, and to provide an emergency spillway. Figure B.3-25 shows a typical cross-section through the dam. By constructing Golden Gate Dam, the flows in Funks Creek will be significantly reduced from pre-project conditions. However, a spillway with a capacity of 23,000 cfs is required in the dam to pass the emergency Sites Reservoir drawdown flows required by DSOD, which is discussed above relative to the inlet/outlet works. The stair-step RCC spillway with confining walls on both sides provides the required spill capability. The spillway crest length would be approximately 375 feet and the crest would be set at an elevation of 206 feet, which corresponds to the current normal maximum operating level in Funks Reservoir. When passing the maximum design flow, the water depth over the spillway would be approximately 7 feet. A spillway bridge would provide access across the dam.

The dam would extend from 15 to 20 feet below existing grade to be founded on weathered rock based upon information on the design drawings for the existing Funks Dam.

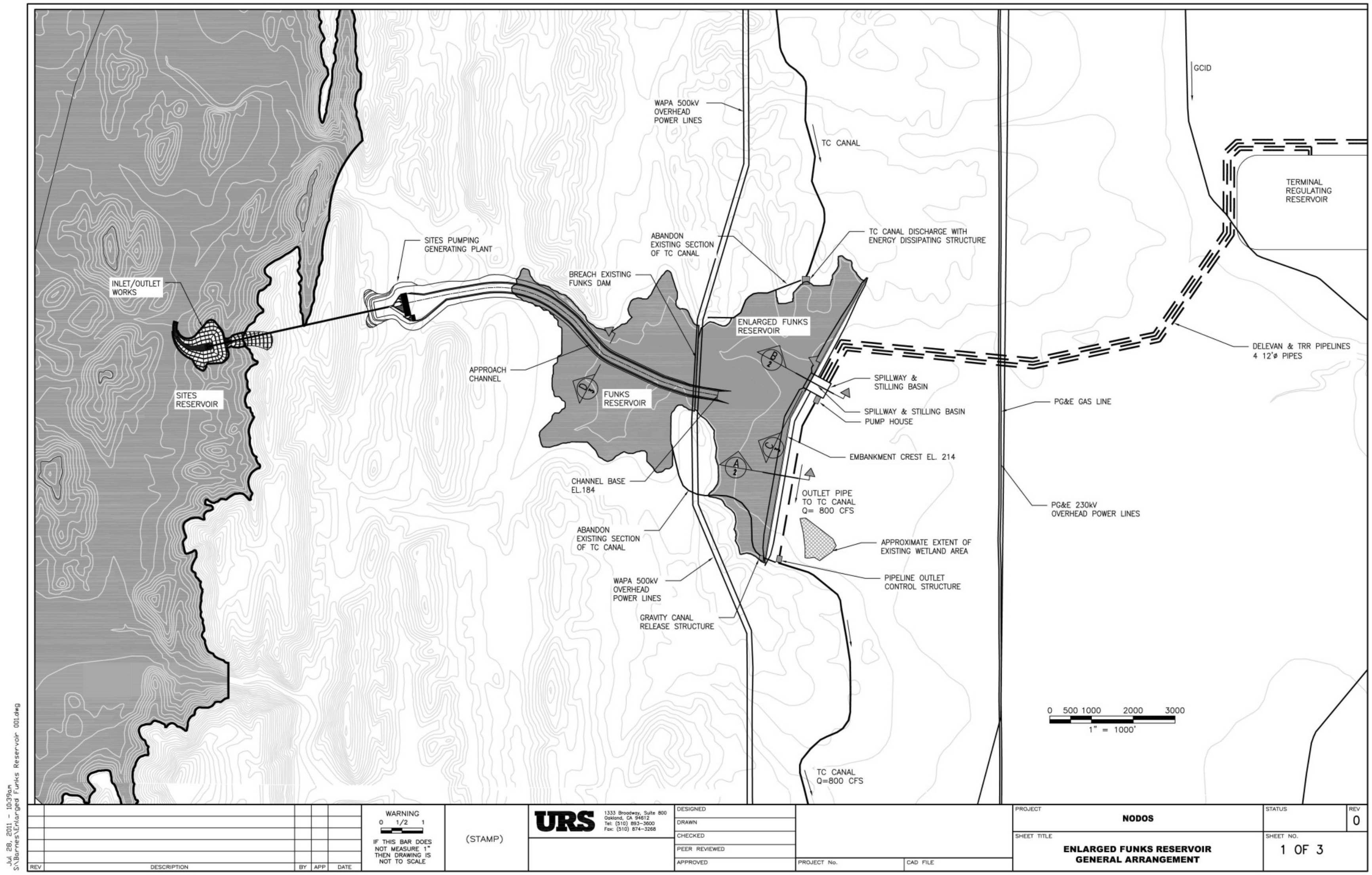


Figure B.3-24. Funks Reservoir – Modification – Plan

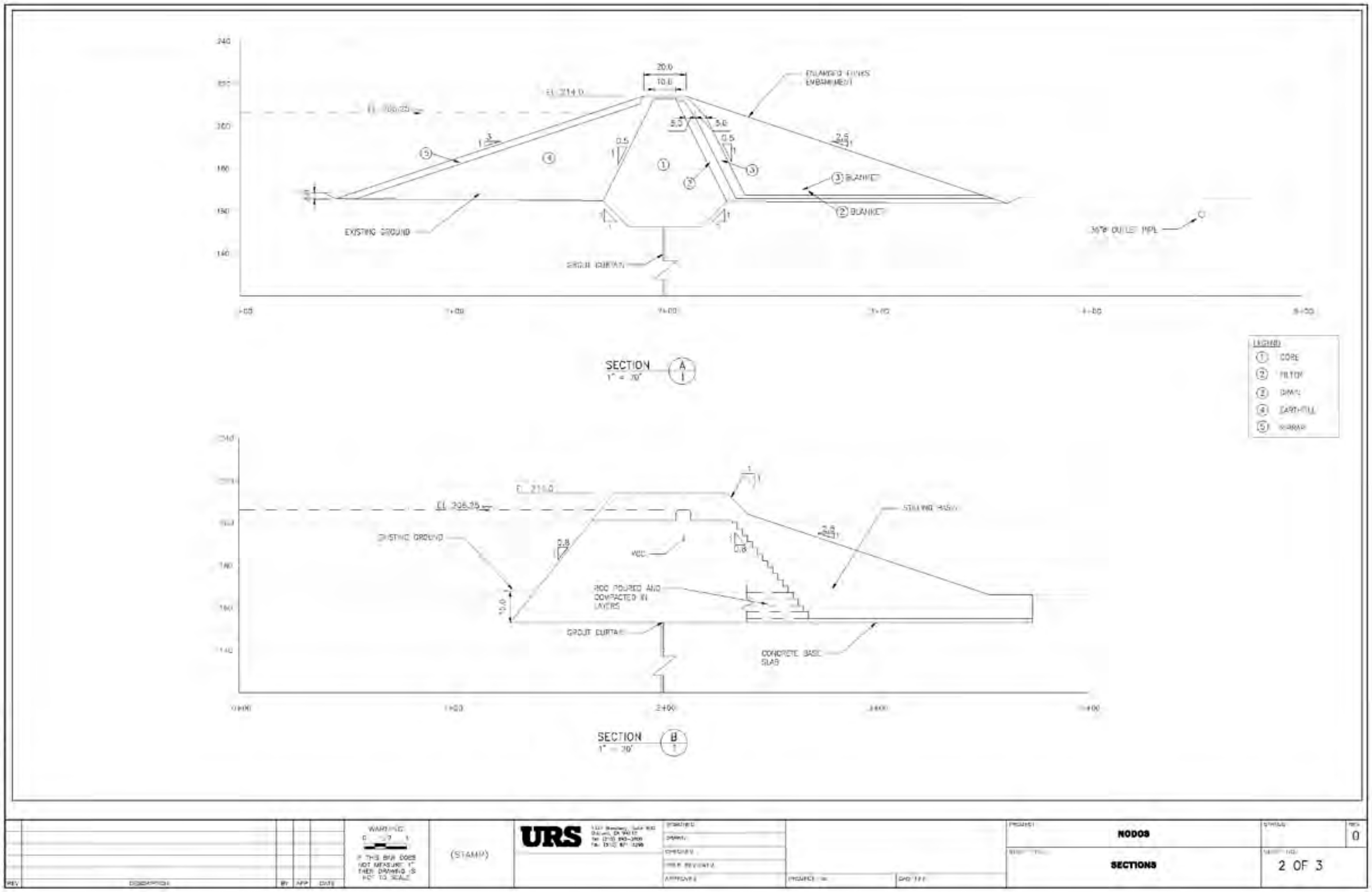


Figure B.3-25. Funks Reservoir – Modification – Sheet 1

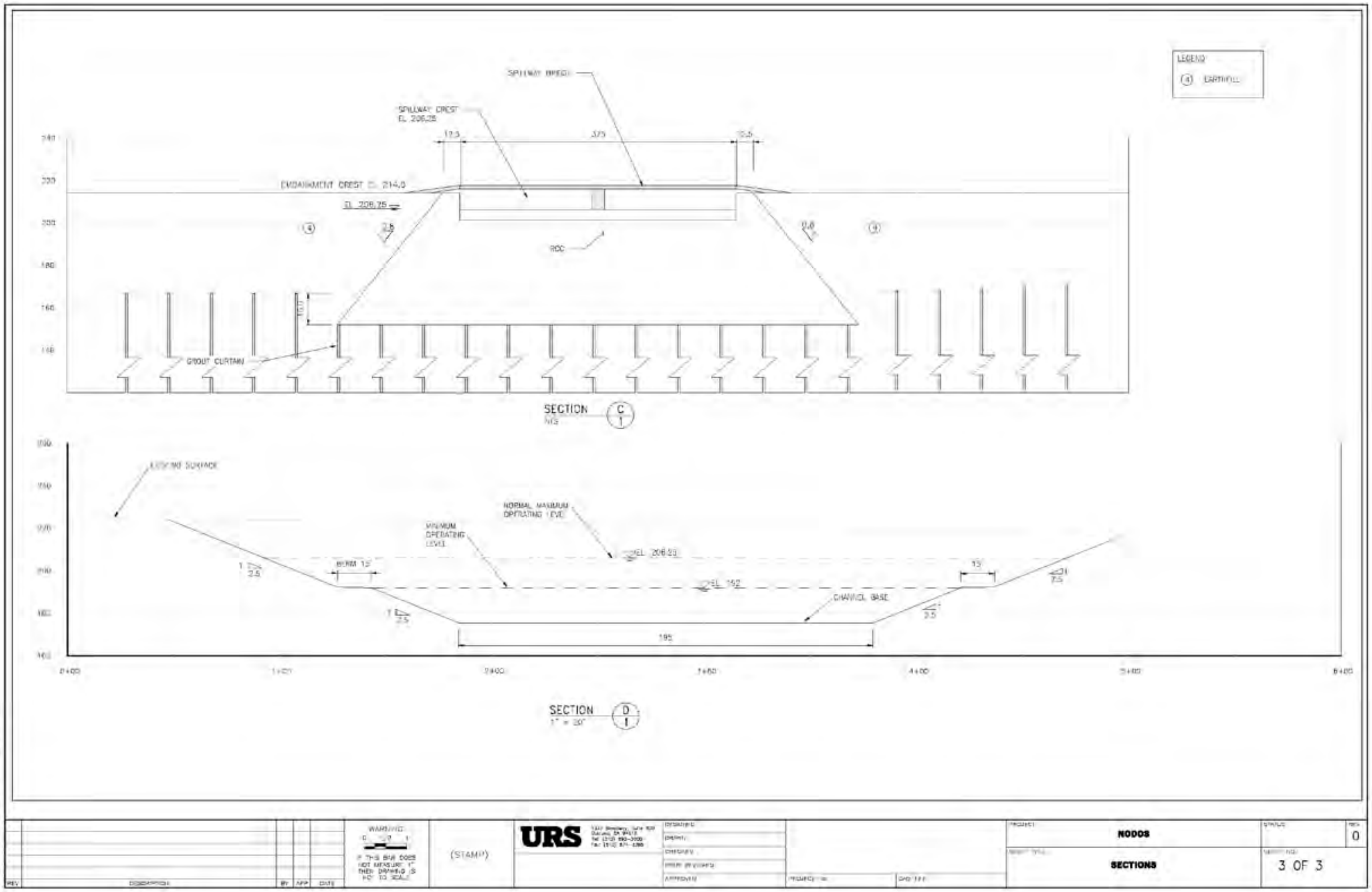


Figure B.3-26. Funks Reservoir – Modification – Sections Sheet

Earth Dam

The earth dam component would be a zoned earthen embankment with clay core section and earth and rockfill shell zones upstream and downstream. Figure B.3-25 shows a typical cross-section through the dam. This cross-section is similar to the one used for the existing Funks Dam. The core section of the dam would extend from 15 to 20 feet below existing grade to be founded on weathered rock based upon information on the design drawings for the existing Funks Dam.

Construction Materials

Construction materials for the earth dam would come from required excavations for SPGP and the channel connecting the enlarged reservoir with SPGP. The material in the existing Funks Dam also can be reused to construct the new dam. Approximately 680,000 cubic yards of core material and 2,200,000 cubic yards of earth and rockfill would be required to construct the dam.

Construction materials for the RCC dam would include imported sands and gravel. Suitable processed rock from project excavations could also be used if the material is found to be suitable for such a use. On-site material sources should be explored in future investigations for the NODOS project. Approximately 150,000 cubic yards of RCC material would be required.

Reservoir Operating Levels

Operating levels within the Holthouse Reservoir can vary between elevation 206.0 feet and 192.0 feet, which provides the required active storage capacity of approximately 6,500 AF. Elevation 206.0 feet corresponds to current normal operating level in existing Funks Reservoir. Note that there is approximately 1,000 AF of dead storage in the Holthouse Reservoir due to topographic elevations near the new dam. This space can be allocated to sediment accumulation, disposal of excess excavated material from the project, or for disposal of existing sediment.

Pipeline Inlet/outlet Works

The RCC dam section would also function as the inlet/outlet work through which all project water would pass. The Delevan and TRR Pipelines would pass through the RCC section and daylight on the upstream side in the reservoir. Trash racks would be located over the pipe entrances and slide gates would also be provided to isolate each pipeline if necessary. Immediately downstream of the RC section, a valve vault will be provides to accommodate butterfly valves for each pipeline than can be used to regulate or isolate flows if necessary. Primary flow control is assumed to be at the downstream end of each pipeline at TRR, SRPGP, or Sacramento River release structure depending upon the project alternative.

TC-Canal Pumping Plant

The operating levels within the Holthouse Reservoir, as mentioned previously, can drop down below elevation 198.0 feet, which is the minimum level that can supply water to the downstream TC Canal by gravity. To supply water to the canal when reservoir elevations are below 198.0 feet, a low-head pumping plant would be provided with a capacity of 800 cfs. Two pumps plus one spare would be provided adjacent to the RCC spillway section. Pumps, valves, header piping, and other required support equipment would be located in a concrete masonry unit block building. The pumps would draw water from the inlet/outlet piping. The water would be pumped up to the canal in a buried pipeline running just outside the downstream toe of the dam. Pipe diameter would be a single, 10-foot line or two 7-foot-diameter pipes. An energy dissipating structure would be provided at the discharge point to the canal.

WAPA Transmission Line Relocation

Currently, a tower-supported Western Area Power Administration (WAPA) transmission line passes through the planned Holthouse Reservoir area. Based upon preliminary contacts with WAPA, the current preferred relocation alternative is to move a segment of the line to the west and cross at a narrow spot in the existing Funks Reservoir. Figure B.3-24 shows this proposed relocation. The span is approximately 1,000 feet.

SPGP Canal

An excavated canal is required to connect the Holthouse Reservoir with the SPGP inlet/outlet. Figure B.3-24 shows the location of the canal; Figure B.3-26 shows a typical cross-section. The channel width and bottom elevation would be set to limits the flow velocity to 2 fps or less for a pumping capacity of 5,900 cfs and a minimum water level in the reservoir at an elevation of 192.0 feet. No canal lining is assumed to be required.

The channel will be excavated in soil and rock. The channel length is approximately 2.5 miles and the excavated volume is approximately 6.2 million yards. Some of the material may be suitable for the Holthouse Dam and Golden Gate Dam construction, and the remainder will require disposal. Suitability of the excavated material requires further investigation and evaluation in future phases of project design.

Existing TC Canal Connections

The TC Canal will be modified to enter Holthouse Reservoir at a point just behind the new dam. A baffle block spillway would be constructed at that location to convey water down into the reservoir regardless of reservoir level. Approximately 0.5 miles of the existing canal beyond the new tie-in point up to the current connection point to Funks Reservoir would be abandoned.

A portion of the downstream canal within the new reservoir limits would also be abandoned. Because it would be possible to supply water to the downstream canal by gravity at times when the new reservoir is high, a new gate-controlled outlet would be provided from the reservoir near the abutment of the new dam.

B.3.5.3 TC Canal Construction Bypass

Installation of a bypass is required to divert TC Canal flow before starting modifications to the existing Funks Reservoir. The bypass would be maintained as a permanent feature following enlargement of Funks Reservoir as requested by TCCA.

Typical summer releases from Funks Reservoir range from 500 cfs to 1,000 cfs. Total flows of 50 cfs to 200 cfs for off-peak limited agricultural releases are needed between November and February, possibly stretching to March, depending on the weather. The proposed bypass alternative consists of a 12-foot-diameter pipeline starting approximately 2,600 feet upstream of the TC Canal Inlet into Funks Reservoir. The bypass would route the required flows around Funks Reservoir during reservoir modification construction. Figure B.3-27 illustrates a typical profile and section views of a proposed bypass.

The bypass would consist of installing two cofferdams on the upstream portions of the TC Canal to isolate the area of embankment cut and pipe installation. The reservoir would be dewatered and the existing check structure would be dismantled and reconstructed approximately 3,000 feet upstream. The check structure consists of two 18-foot by 15-foot, 6-inch gates, electrical control, hoists, and concrete supports and reinforcement. The facility would be relocated slightly downstream of the bypass. The bypass would need to be gated or valve controlled to regulate releases downstream as required by the TCCA.

B.3.6 Modifications Required to Existing Canals to Supply Sites Reservoir

Alternatives for the NODOS project involve diverting water from the Sacramento River to storage in Sites Reservoir through the TC Canal and GCID Canal as well as through the Delevan Pipeline. Preliminary feasibility engineering analyses have been conducted on alternative capacities for the TC Canal and modifications to the GCID Canal conveyance to enhance performance and reliability. Both canals would supply water to the Holthouse Reservoir which would serve as a forebay/afterbay reservoir for the SRPGP.

B.3.6.1 Existing TC Canal

Evaluation of TC Canal capacity alternatives resulted in the recommendation that the TC Canal remain at its current capacity of 2,100 cfs. Because this is the current capacity, no modifications are required or recommended to the TC Canal from the inlet on the Sacramento River to the project. However, local modifications are required in the vicinity of Holthouse Reservoir. These modifications include a new energy dissipating spillway structure to release water to the reservoir near the new Holthouse Dam and the abandonment of the segment of canal along the north shore of Holthouse Reservoir from the new inlet west to the existing point of entry into existing Funks Reservoir.

B.3.6.2 GCID Canal

The evaluation of the existing GCID Canal resulted in the recommendation that the canal remain at its current 1,800 cfs capacity, but that the following modifications be considered as part of NODOS to enhance performance and reliability:

- **Upgrade the canal headworks structure**

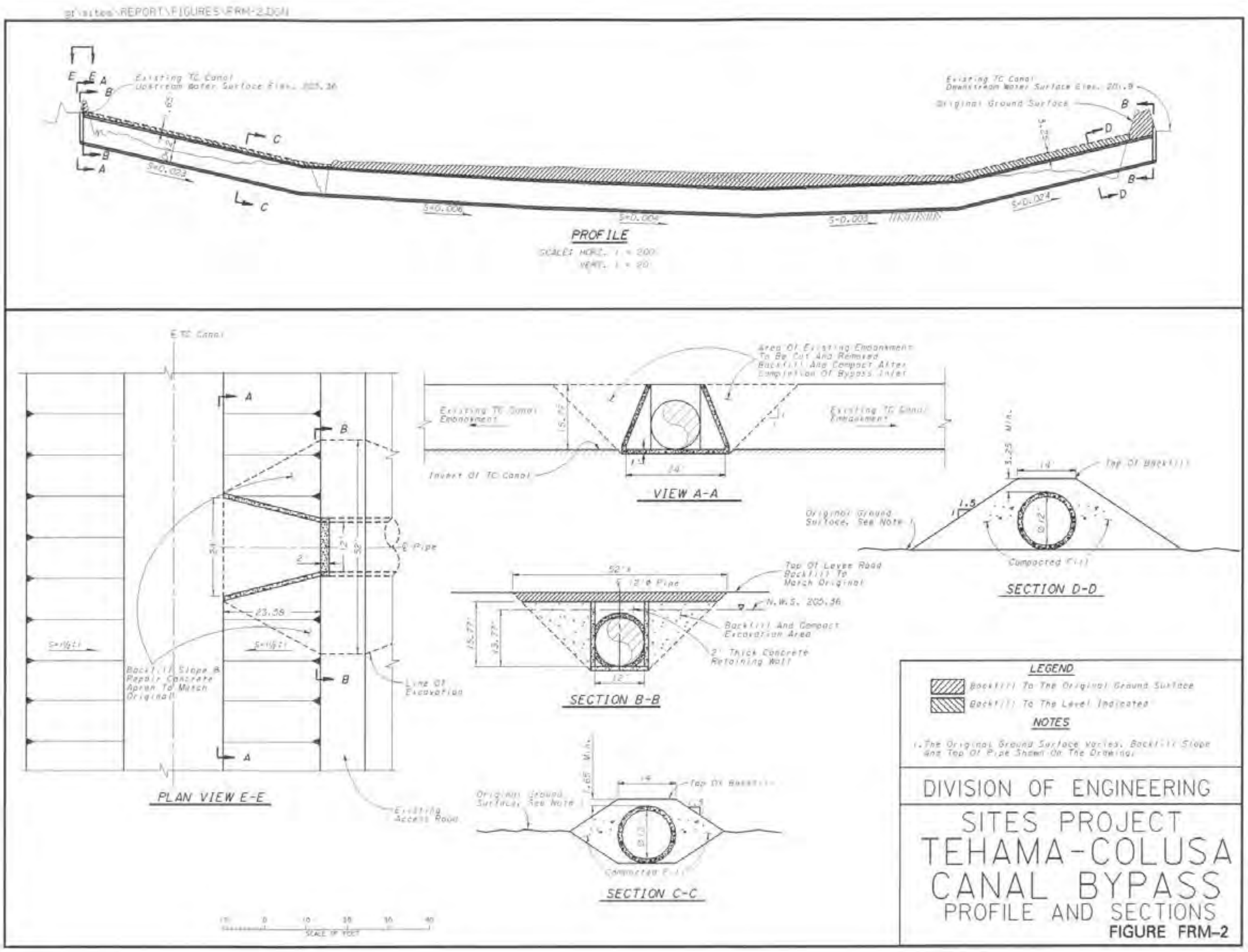


Figure B.3-27. Tehama-Colusa Canal Bypass – Profile and Sections

- Replace the railroad siphon undercrossing
- Replace the Delevan Road Bridge

In accordance with conversations in July 2011 between DWR and GCID, several other recommendations made previously in reports have been removed from the list of proposed canal modifications. Recommendations removed include:

- Perform minor reshaping along the lower 13 miles upstream from the TRR to ensure the required design capacity. (This effort would be part of ongoing maintenance.)
- Replace the Tuttle Creek check structure. (Replacement likely to be completed by GCID prior to NODOS implementation.)
- Modify the existing fish screen structure at the canal entrance. (Overtopping can occur during occasional high river flow conditions, but water quality at such times may preclude extracting project water into the canal.)

Seasonal Canal Maintenance

GCID indicates that the canal typically is out of service for maintenance each year between January 7 and February 20. Any NODOS project work required in the canal or to tie new facilities to the canal should be scheduled during this period whenever possible. If this outage schedule cannot be accommodated, then a canal diversion must be provided around the work area.

SCADA Systems

GCID indicates that supervisory control and data acquisition (SCADA) systems are being added and extended within its system, particularly in the area of the existing headworks. Incorporating SCADA systems on their canal for existing canal facilities will not be necessary. Such systems may, however, still be required for new NODOS project work relative to the canal. The design of new systems must be coordinated with GCID to insure proper integration.

Headworks Modifications

Modification of the headworks structure at the canal inlet was also recommended. The existing headgate structure would be left in place to continue as the bridge for County Road 203. A new headworks structure would be constructed downstream of the existing structure. Figures B.3-28 and B.3-29 show the replacement headgate structure in plan and section, respectively. The new headgate structure would provide the following three main operations:

- Isolate the GCID Canal, as needed, for repairs or other purposes, such as the canal reach between the Main Pump Station and Stony Creek to prevent local flooding during high river levels.
- Control flow when the headworks are under gravity inflow conditions and the pumping plant is shut down, which occur during high river levels.

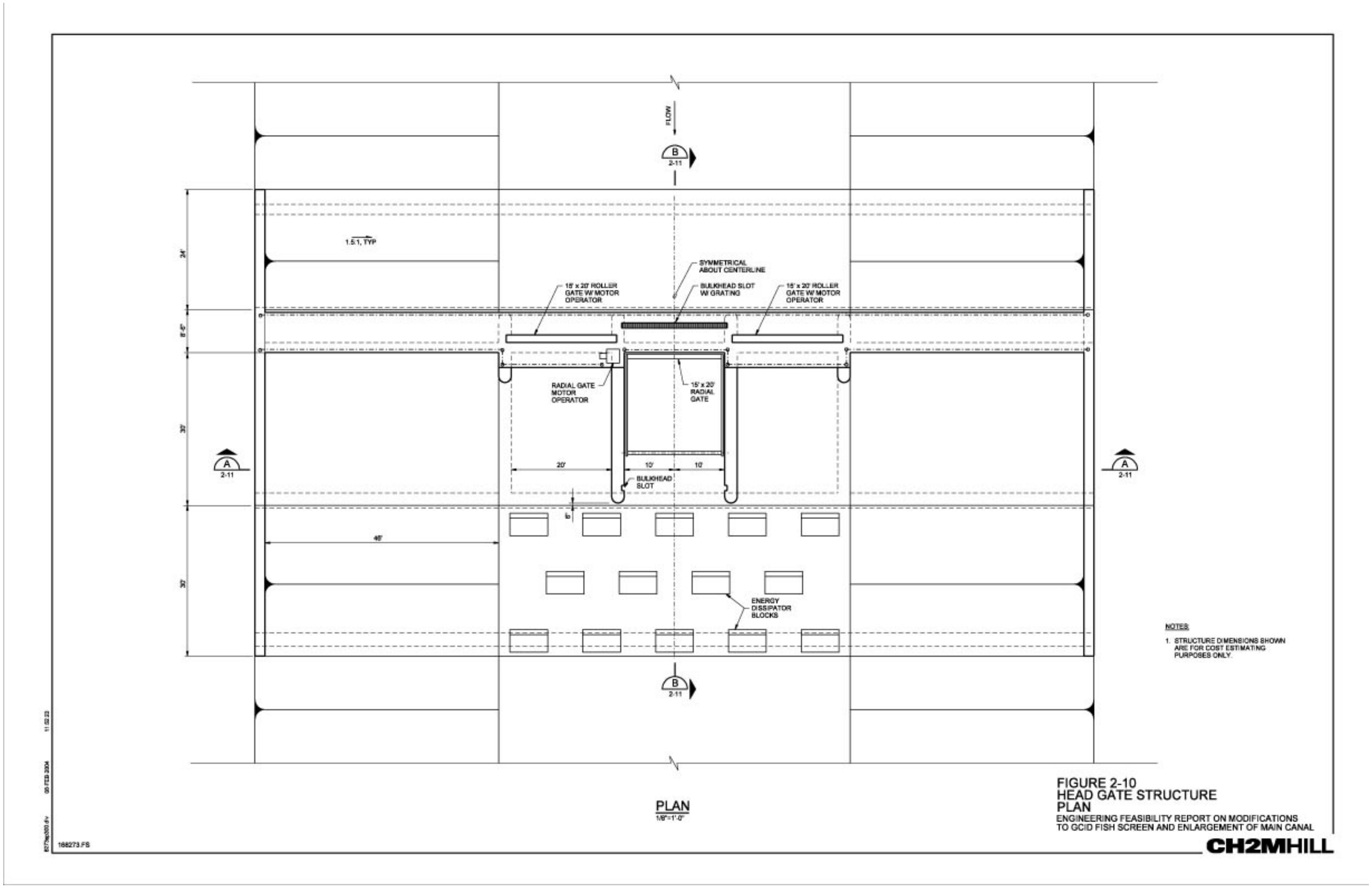


Figure B.3-28. GCID Canal Headgate Structure – Plan

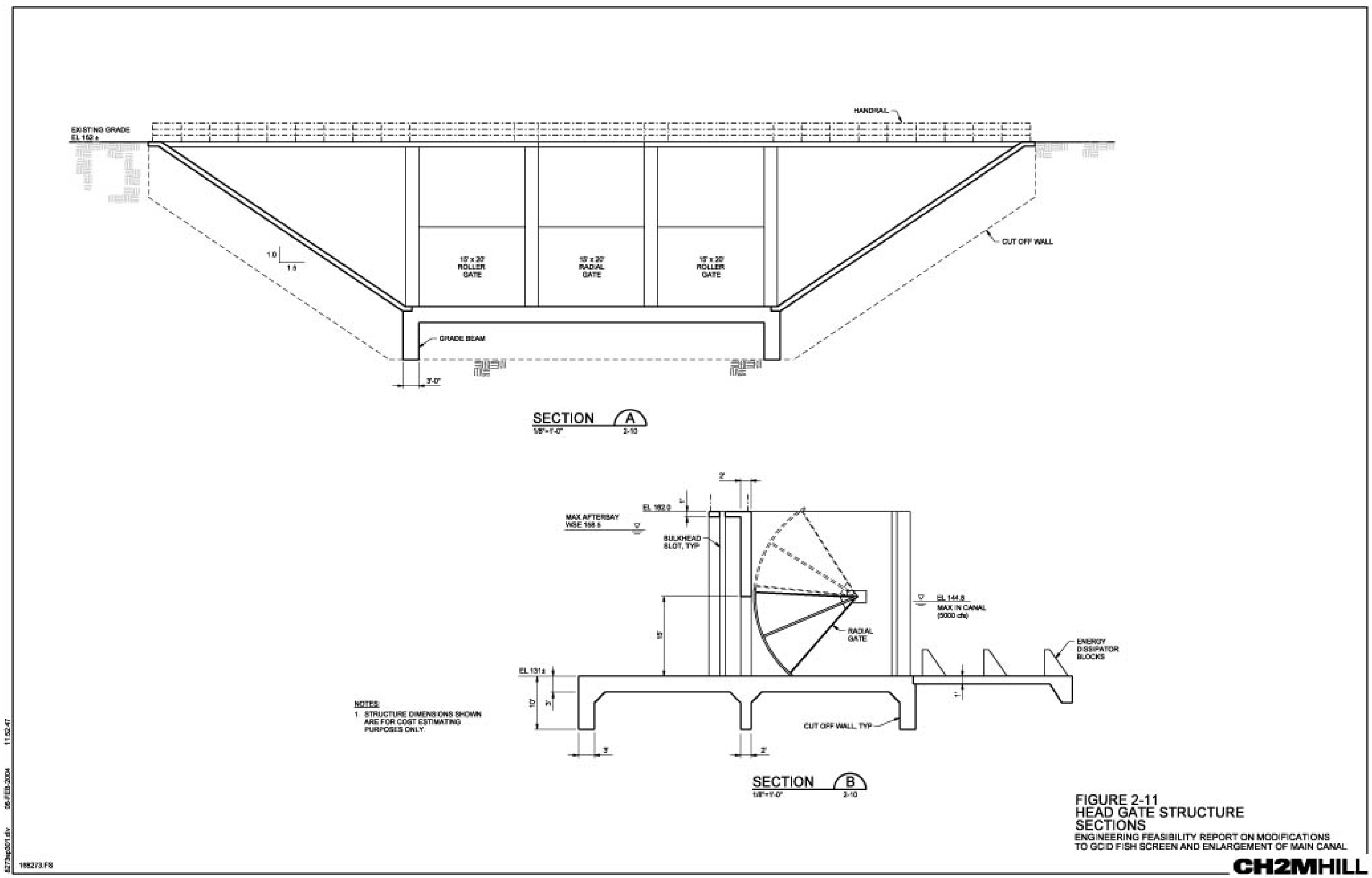


Figure B.3-29. GCID Canal Headgate Structure – Sections

- Control water elevations downstream of the existing headworks, as necessary to extend their operating range under higher river levels.

Flow measurements at the head of the GCID Canal could be provided using a range of methods such as pump curves, canal stage gages and rating curves, in-line measurement flumes, and local flow meters at nearby control points such as siphon barrels. Figure B.3-30 shows the Main Canal flow measurement structure. Existing flow meters installed in the Stony Creek Siphon would provide flow measurement near the head of the canal. SCADA links would provide operational input to adjust both pump and gravity flow rates at the Main Pump Station, as necessary.

Design considerations for the new headgate include:

- The structure's invert and crest would be based on matching the existing canal invert and top of bank elevations, respectively.

- The relatively deep channel section in this reach of the canal would result in a structure that is more than 30 feet high.

- The design condition for this structure, for sliding and over-turning stability, would occur with maximum water levels on the upstream side (during high river levels), and a drained canal on the downstream side (for emergency shutdown).

The resulting hydrostatic forces require a substantial concrete structure with cut-off walls keyed into the canal invert and side slopes. Canal lining would extend approximately 200 feet downstream of the structure.

Two vertical roller gates and one radial gate would provide a wide range of water level and flow control. The roller gates would be set to meet the approximate upstream water level requirements, and the radial gate would be used for finer adjustments, flow control, and water level control. All three gates would be motor-operated, and tied into a SCADA system to the Main Pump Station controls building.

The water level and flow control functions would involve operating conditions that would result in water surface drops across the headgate of between 3 and 15 feet, which would require a set of energy dissipater blocks immediately below the gates to slow down and stabilize the water discharging under each gate.

The connection from the GCID Canal to the TRR would have an energy dissipation bay with check structure, as well as the TRR inlet channel and inlet control structure. The inlet channel would connect the GCID Canal to the TRR. The inflow control structure is similar to a standard GCID Canal check structure, with three large radial gates to control flow into the reservoir.

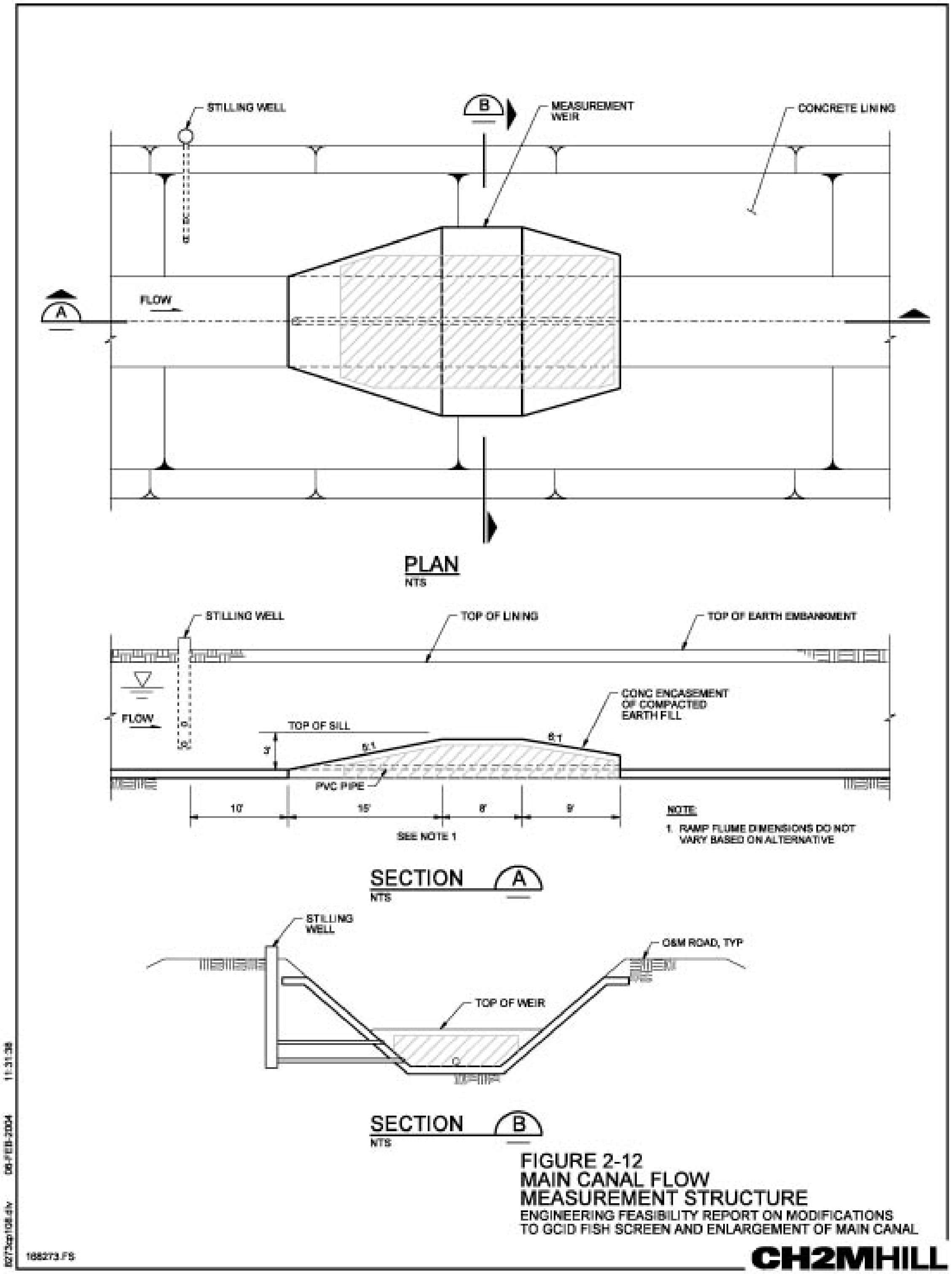


Figure B.3-30. GCID Canal Flow Measurement Structure

B.3.7 Terminal Regulating Reservoir and TRR Pumping Plant

Water conveyed down the GCID Canal would be conveyed into a future TRR. A new pump station, the TRR-to-Holthouse Pumping Plant, would then convey the water from the TRR to Holthouse Reservoir via the new TRR Pipeline when water is being diverted from the canal for ultimate delivery to Sites Reservoir. Water can also be released back to the TRR from Holthouse Reservoir via the TRR Pipeline to meet irrigation needs in the GCID Canal. The returning water can be used to generate hydroelectric power using two turbine units located in the TRR Pumping Plant. The TRR would provide routine operational storage for the Pumping Plant and an afterbay for the turbine units. It would also provide regulatory storage to balance out normal and emergency flow variations in the 40 miles of GCID Canal between the headworks pump station on the Sacramento River and TRR.

B.3.7.1 Terminal Regulating Reservoir Design

The TRR would be constructed on the valley floor close to the GCID Canal by excavating below grade and constructing a low perimeter embankment around the excavated area. The general location and arrangement of the TRR would be selected to facilitate gravity flow to and from the GCID Canal.

The reservoir would have a maximum storage capacity of 2,000 AF and a footprint covering approximately 200 acres. The bottom dimensions would be roughly 2,900 feet by 2,900 feet. The pond location or embankment alignment would be adjusted locally to avoid impacting several existing structures in the pond vicinity. The depth would be approximately 15 feet, comprised of two feet of dead storage, 5 feet of operational storage, 5 feet for emergency storage, and 3 feet of freeboard. The embankment height above existing grade will vary around the pond perimeter, at approximately 6 feet. The pond should be non-jurisdictional with respect to Division of Safety of Dams. The TRR normal operational storage capacity would be based on the need to provide normal transient operating storage for the TRR Pump Station. The emergency storage (approximately 1,400 AF) would permit continued TRR Pump Station operation for up to 8 hours without inflow from GCID Canal. Major appurtenance features would include a GCID Canal transition bay, a connecting channel from the GCID Canal to the TRR, and a flow control inlet structure.

The TRR reservoir complex would be comprised of four hydraulic structural systems. These systems include a GCID Canal energy dissipation bay with check structure, a reservoir inlet channel and control structure, a reservoir, and pump station. The GCID canal energy dissipation bay with check structure will function to reduce the flow into a stable pool, just before the turnout to the TRR connecting channel. The TRR inlet channel and control structure connects the GCID canal to the TRR and controls flows into the reservoir. The reservoir would function to provide operational and emergency storage as well as a forebay for the TRR Pump Station. In this latter capacity, it would provide operational storage to smooth out the normal transient mismatches in flow rates between the inflow rate of the existing GCID, and the outflow rate of the TRR Pump Station. The final size of the TRR will be determined by the requirements of the dead, normal operational, and emergency storages.

B.3.7.2 TRR Pumping-Generating Plant

The structural building for the TRR Pumping-Generating Plant would be similar to that described for the Sacramento River Pumping Plant. The basic TRR Pumping-Generating Plant elevations at the plant inlet are listed below:

- Maximum Water Elevation 123.00 feet
- Minimum Water Elevation 104.00 feet
- Intake Elevation 94.00 feet
- Pump Station Finished Floor Elevation 130.00 feet

The number of pump and turbine units is listed below:

- Three large pumping units each rated at 620 cfs (two operational and one standby), with 7.5-foot-diameter inlet valves.
- Two small pumping units each rated at 325 cfs, with 5-foot-diameter inlet valves
- Two turbine units each rated at 750 cfs, with 8.5-foot-diameter inlet valves

One 32-foot-diameter spherical air chamber would be required on each pump station discharge line to reduce the surge pressures in the pipeline.

Access to construction site would be from Interstate 5 and approximately 2.5 miles west on Delevan Road, then approximately 0.5 mile south on Noel Evan Road. All temporary construction utilities will be provided by contractor.

Power Supply

Although power will be generated during project operation, a comprehensive power study is needed to assess power supply versus demand, and to assess the costs for connecting the new site facilities to existing power circuits. An operational study will also be essential in determining power needs.

Excavation

Excavation will be conducted using temporary slopes of 1.5H:1V for the 25-foot-deep trench along the proposed pipelines, and a temporary slope of 2H:1V for the 40-foot-deep foundation of the pump stations. The pump station foundations will be excavated in in situ materials and no major improvements to the foundations will be required. The proposed cut slopes are based on drill hole information and on-site inspection conducted by DWR geologists. Preliminary stability analysis has been performed for this study. During construction, the topsoil material will be excavated, stockpiled separately, and replaced to ensure native grasses and plants will grow.

Groundwater was encountered in most of the auger holes (based on drill hole logs provided by DWR) and dewatering would be required during construction.

Staging areas for the project will be located at each pump station site and at designated locations along the pipeline alignment. Excess suitable material from conveyance excavation can be hauled to either of the proposed dam sites, located west of Funks Reservoir. Excavation depths for the project will range between 25 and 40 feet.

Refilling Pump Units

The discharge lines periodically may need to be dewatered for inspection and maintenance. These lines need to be filled at a slow rate to allow the release of air through air and vacuum valves. To accomplish this, one or two 100 cfs pump units, depending on design capacity, would be installed at the Sacramento River and/or TRR Pump Stations.

B.3.7.3 Operation

Monitoring and Control

As noted above, GCID indicates that SCADA system upgrades are being made to their system prior to the proposed NODOS project implementation and that the scope and functionality of the new SCADA systems for the proposed NODOS project must be coordinated with GCID's system.

Extensive remote monitoring and combinations of local and remote controls would be required to operate the modified GCID Canal using an integrated SCADA system. The SCADA system would need to provide information on operating parameters such as Main Pump Station flows, canal water levels, check structure gate positions, major lateral turnout flows, canal spills, and TRR inflows and water levels. Communication and coordination between the GCID system and other components of the Sites Reservoir supply system, such as the TRR pump station, would require integration of the local GCID SCADA with the regional NODOS SCADA system.

Canal Capacity Allocation

The canal would have a total capacity of 1,800 cfs. The total conveyance capacity would have to be used to meet both GCID's internal water supply needs and NODOS conveyance requirements. Therefore, the balance of available capacity between these two uses would vary by month based on factors such as weather, irrigation demands within GCID, Sacramento River flows, and overall NODOS system operating conditions. Figure B.3-31 shows the approximate monthly conveyance capacity allocation for the 1,800 cfs GCID Canal, based on average monthly GCID service area demands.

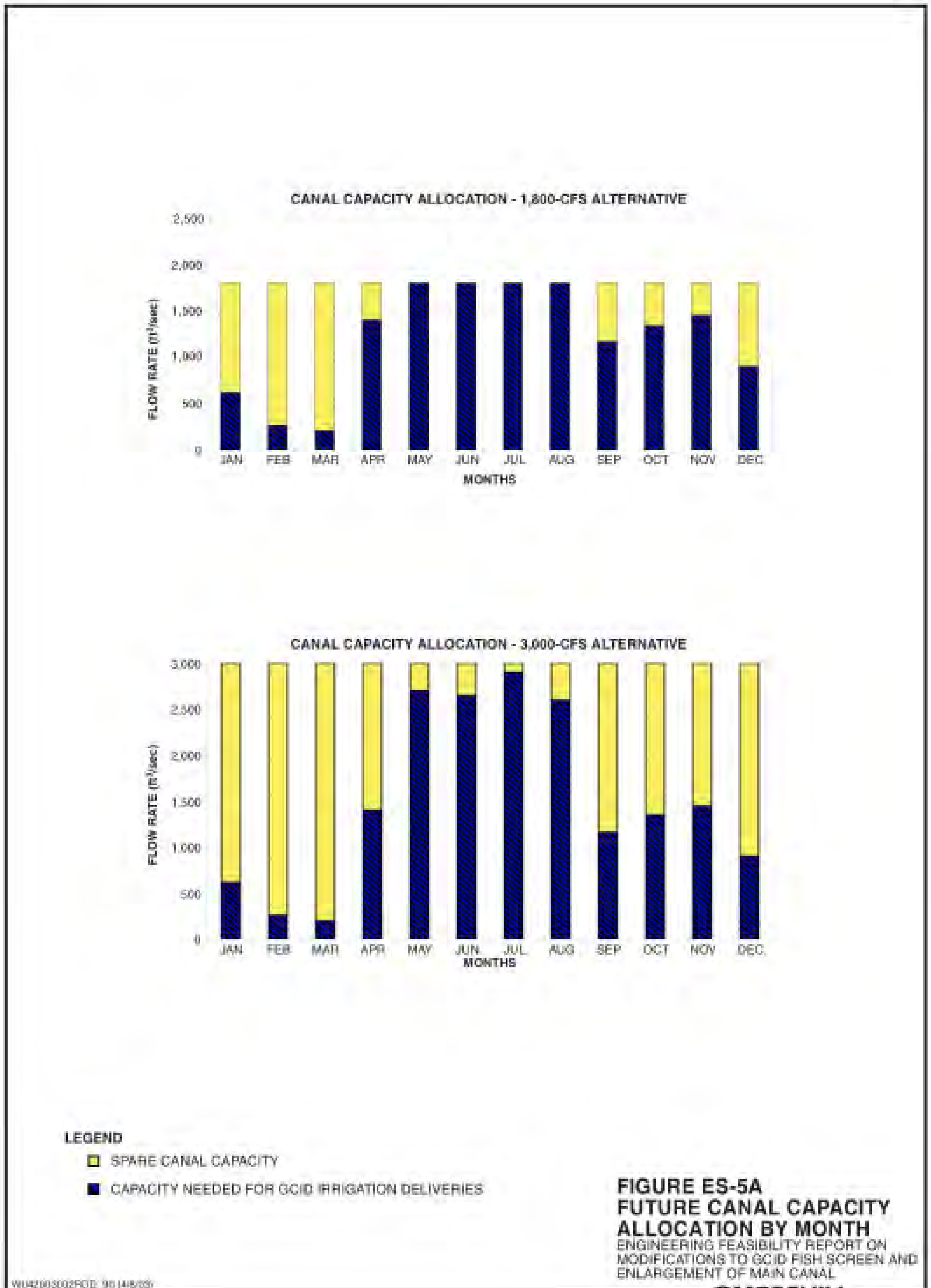


Figure B.3-31. Monthly Conveyance Capacity Allocation for GCID Canal

B.3.8 New Sacramento River Pumping-Generating Plan and Conveyance for Alternative A and Alternative C

This section describes Alternative A and Alternative C, which include pumping-generating capability on the Sacramento River. Alternative B does not include pumping or generating capability on the Sacramento River, but still includes the conveyance pipeline from Holthouse Reservoir to convey water from Sites Reservoir back to the Sacramento River. Alternative C is discussed in a following section.

For Alternative A and Alternative C, the SRPGP would be constructed with a pumping capacity of 2,000 cfs, an intake and fish screen structure at the Sacramento River, and the Delevan Pipeline from the SRPGP to the Holthouse Reservoir. The conveyance system would also serve to return discharges capacity of 1,500 cfs from Holthouse Reservoir to the Sacramento River. Total length of the conveyance is approximately 13.5 miles with a difference in elevation of approximately 150 feet. The return flow can be used to generate hydroelectric energy with the residual head in the conveyance system. Figures B.3-32 and B.3-33 show the new Sacramento River Conveyance configuration.

B.3.8.1 River Diversion and Pumping-Generating Plant

The proposed pumping-generating plant project would involve the construction of (1) a pumping-generating plant with manifolds, (2) an afterbay with sheet piling wall, (3) two air chambers, (4) two 12-foot-diameter pipes (Delevan Pipelines), (5) a retaining wall, (6) a control building, (7) an electrical switchyard, (8) on/off ramps from/to Highway 45, and (9) a roadway.

Preliminary design of the proposed new facilities was performed considering the following criteria:

- Divert up to 2,000 cfs from the Sacramento River during high flows and convey it to Holthouse Reservoir
- Sacramento River diversion would be located below RM 158.3, with a fish screen facility and pumping-generating plant to raise water to Holthouse Reservoir
- 100-year flood elevation at the Sacramento River with an elevation of 82.0 feet
- Minimize the amount of Sacramento River bank cuts
- Deter fish from entering intake bays, and return fish with a fish-friendly lift bypass with outfall downstream
- Conveyance to maximize ground usage after installation and minimize environmental impacts

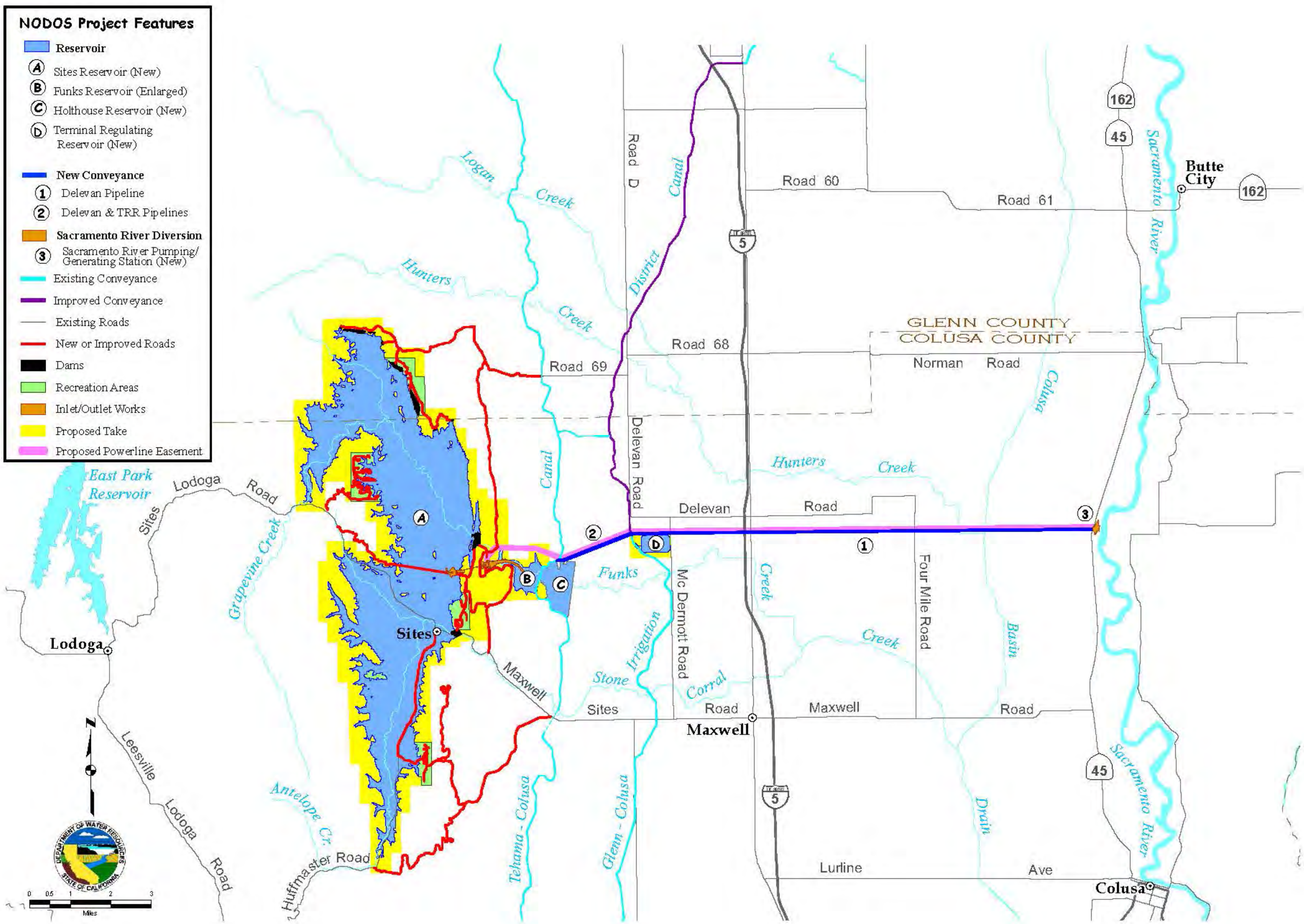


Figure B.3-32. New Sacramento River Conveyance Layout

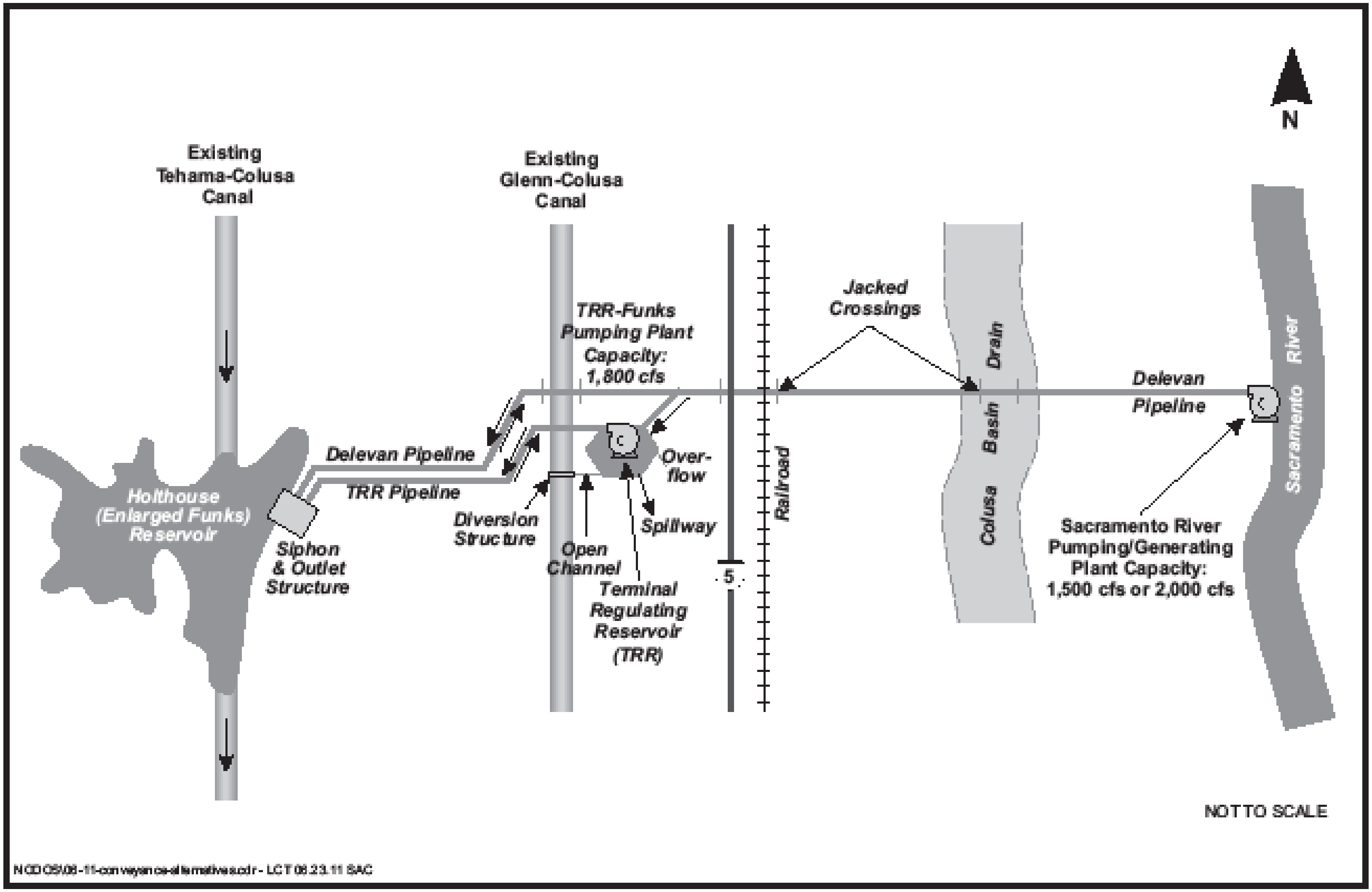


Figure B.3-33. Schematic Layout of New Sacramento River Conveyance Delevan Pipeline

- Use on-bank design parallel to river for fish screen
- Variable conveyance flow requirements
- Suitable excess materials from the conveyance excavation could be transported to Golden Gate Dam
- Construction of SRPGP

The SRPGP and fish screen facilities (Figure B.3-34) are located on the west side of the Sacramento River, slightly downstream of RM 158.5 and east side of Highway 45. The plant and afterbay are protected against a 100-year flood by the West Levee road. Based on the fish screen design and construction studies of June 2008, the proposed location of the plant is considered the best for hydraulics for fish screen facilities.

For each 12-foot-diameter pipeline, the system has the capacity to transport 1,000 cfs of water to Holthouse Reservoir and 750 cfs for reverse flow and power generation. The pumping/generating plant would consist of four pumping units each with a design pumping capacity 600 cfs, one spare pumping unit with a design pumping capacity 600 cfs, two generating units each with a generating capacity of 750 cfs, pipelines, mechanical and electrical equipment, aboveground control and O&M buildings, and related equipment.

The overall dimension of the plant building is approximately 300 feet long by 80 feet wide with multiple story structure to provide spaces for mechanical and electrical equipment. A gantry crane with crane rails approximately 60 feet apart would be installed on the finish floor of the plant for moving pumps, motors, pumping-generating units, valves and electrical/mechanical equipment. The maximum height of the crane would be approximately 25 feet with a capacity of 100 tons.
Figures B.3-35 and B.3-36 show a floor plan of the motor floor and pump floor, respectively. Figures B.3-37 and B.3-38 show traverse and longitudinal sections of the plant, respectively. Plans and sections will vary between project alternatives due to differences in number of units and unit sizes. These figures are intended to present general concepts only.

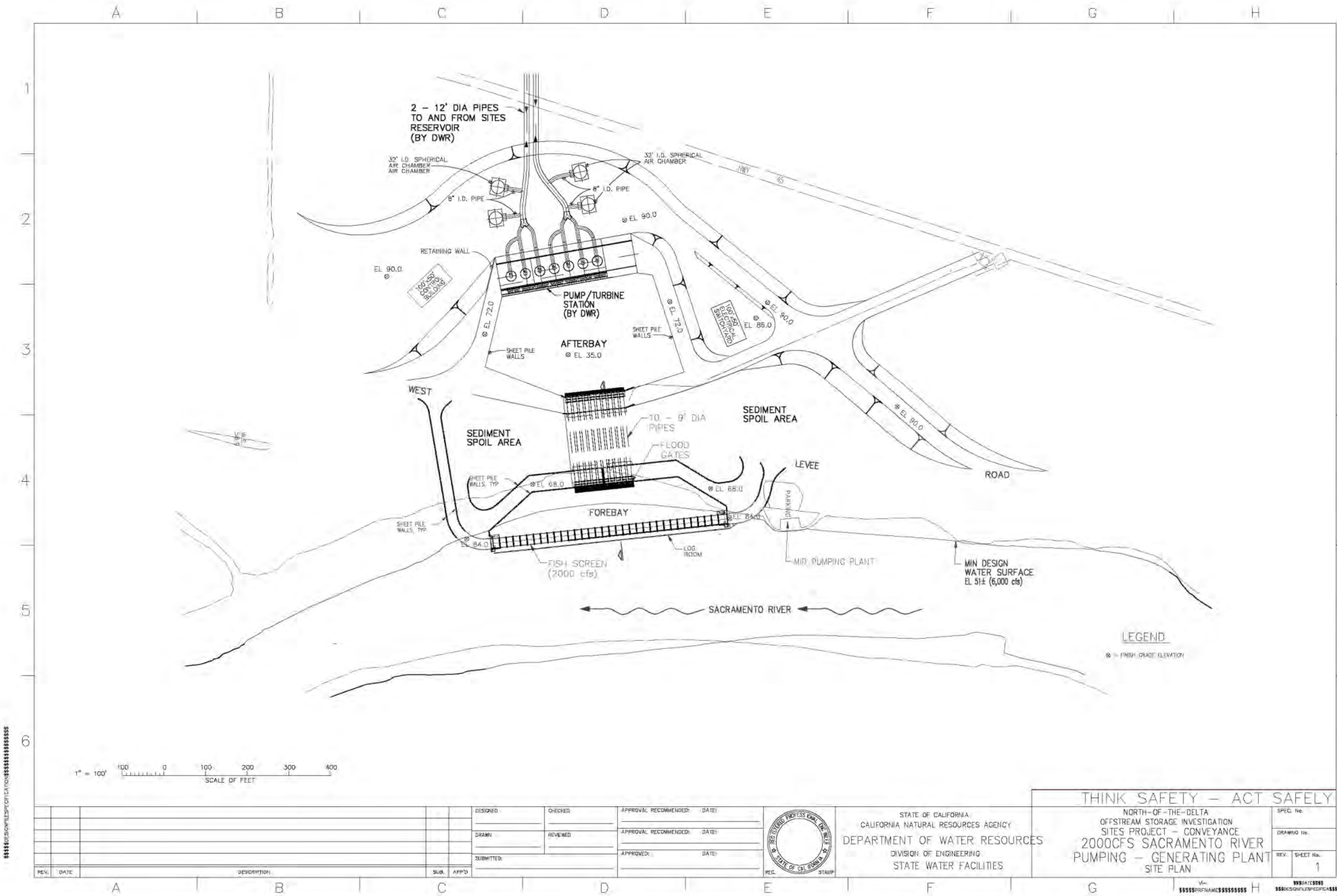


Figure B.3-34. Sacramento River Pumping-Generating Plant Site Layout

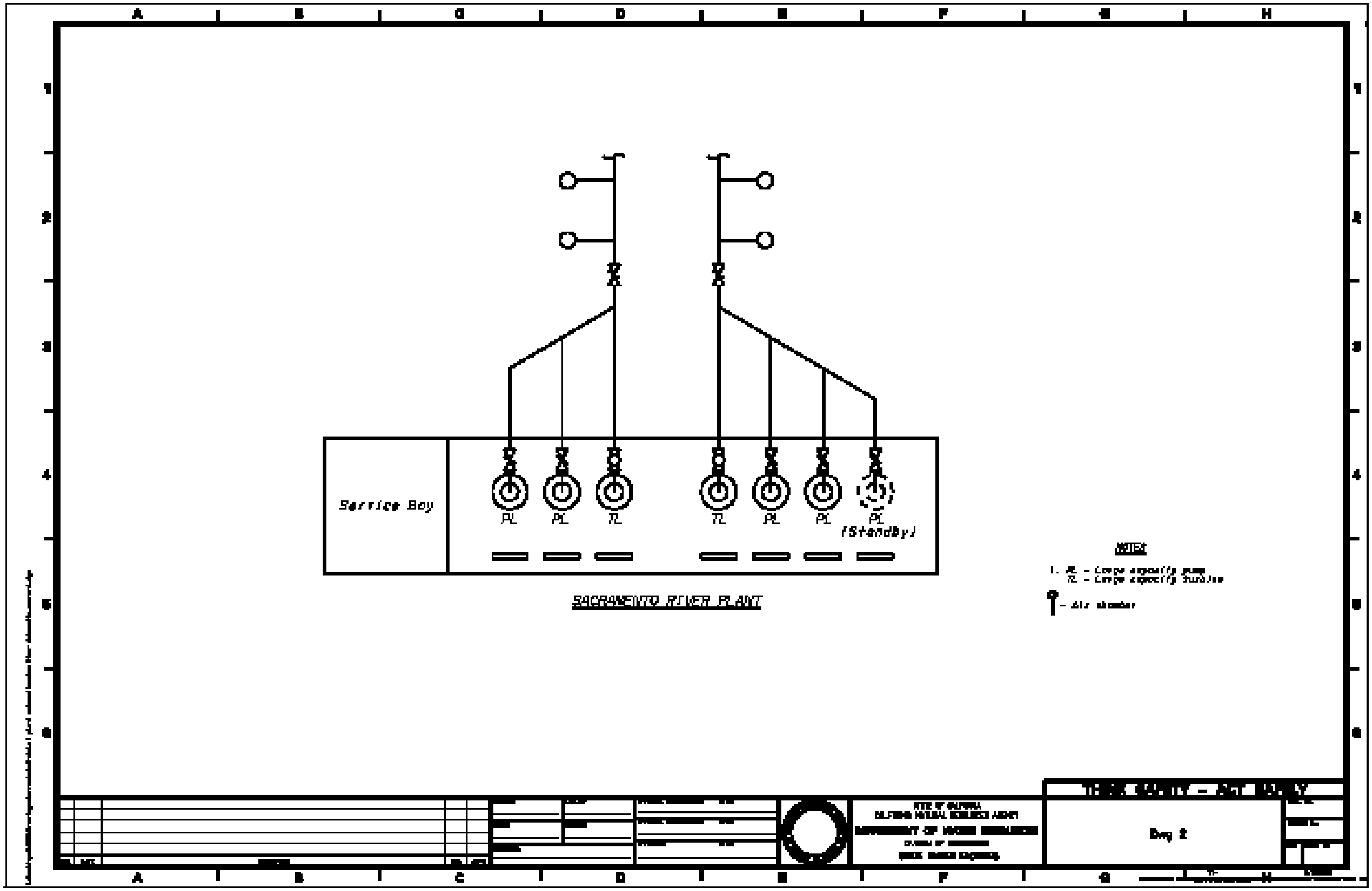


Figure B.3-35. Sacramento River Pumping-Generating Plant Motor Floor Plan

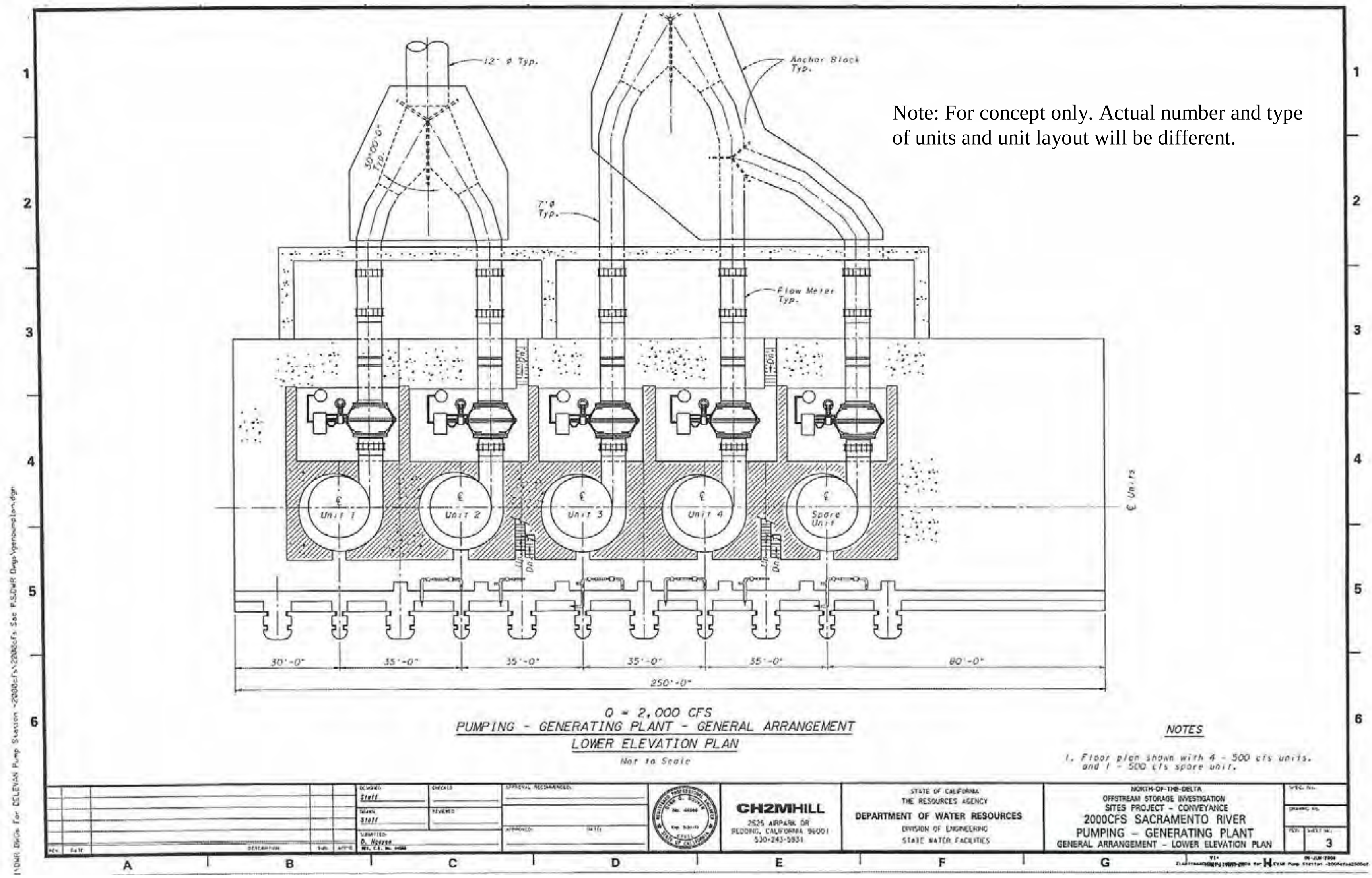


Figure B.3-36. Sacramento River Pumping-Generating Plant Lower Elevation Plan

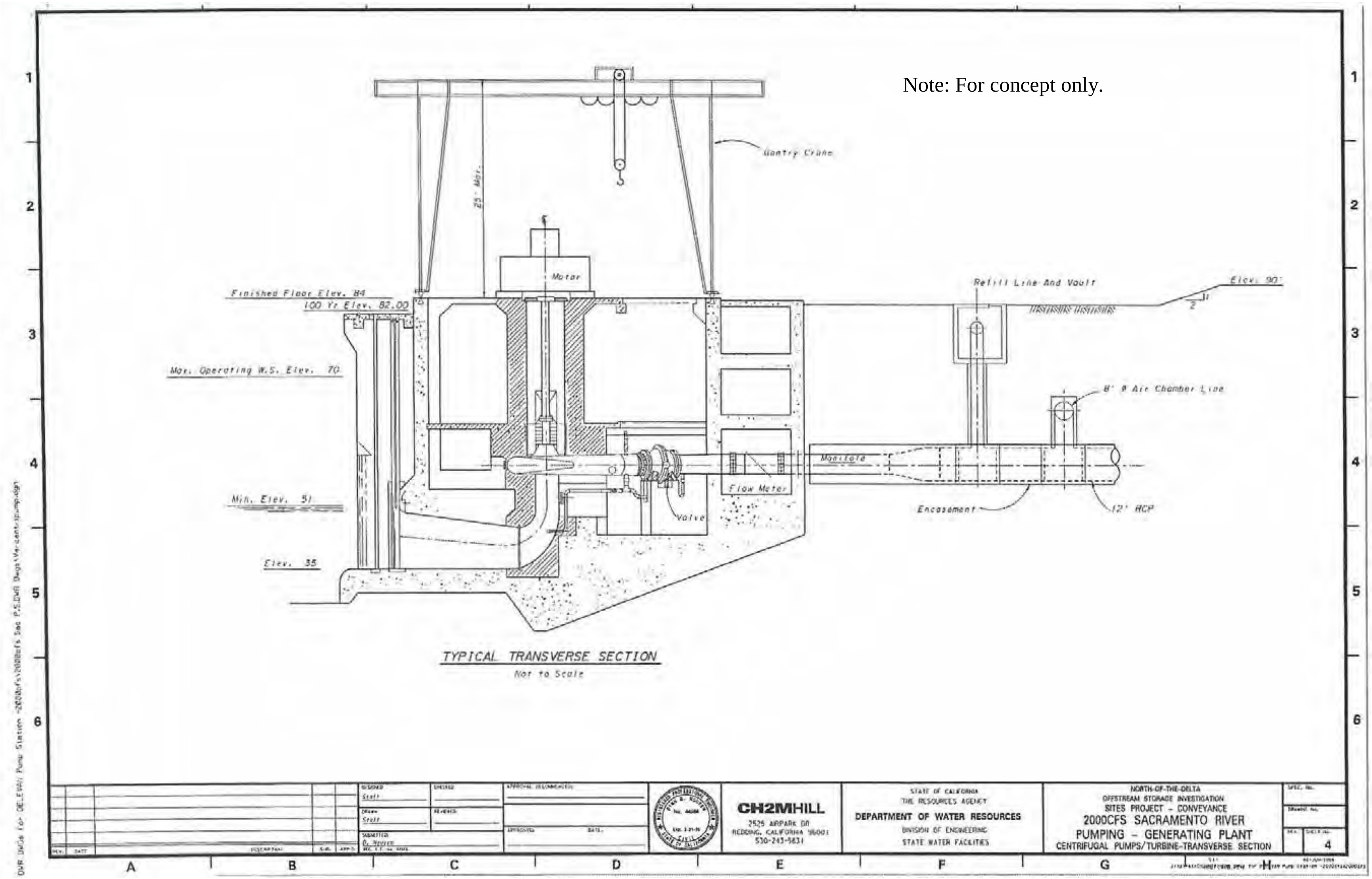


Figure B.3-37. Sacramento River Pumping-Generating Plant Transverse Section

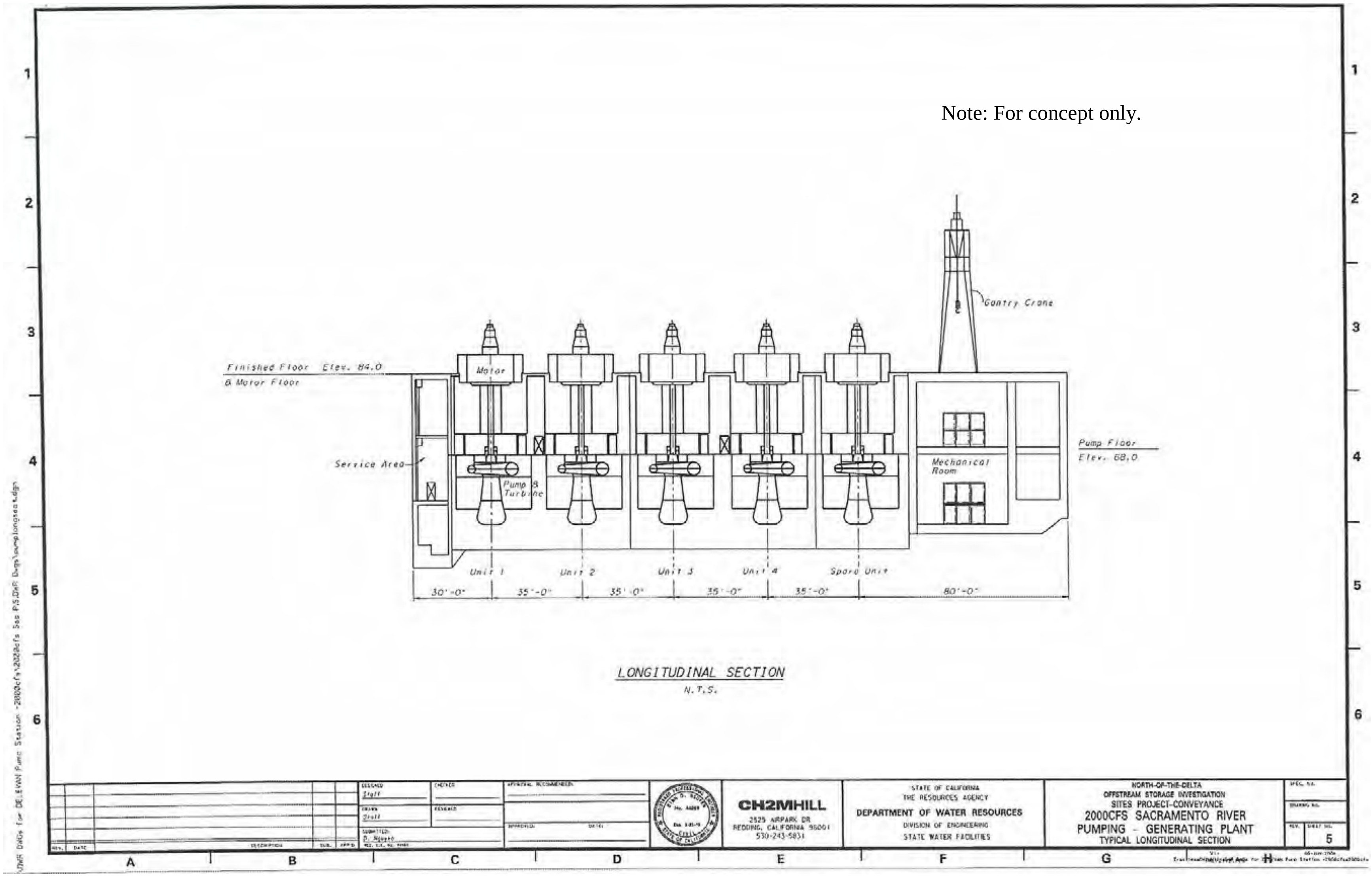


Figure B.3-38. Sacramento River Pumping-Generating Plant Longitudinal Section

The pumping-generating plant would allow for dewatering of the pipelines and also reverse flow into the Sacramento River through the units. No additional bypass system for dewatering is required at the plant, provided there is at least one unit that is operational.

Some basic elevations are listed below:

Maximum operating water elevation:	76.00 feet
Minimum water elevation:	55.00 feet
Intake elevation:	45.00 feet
Plant finished floor elevation:	84.00 feet

Table B.3-10 shows the amount of power required and the power generated by the SRPGP.

Table B.3-10. Sacramento River Pumping-Generating Plant

	Q = 2,000 cfs
Number of Installed Pumping Units	4 + 1 spare
Unit Pump Capacity (cfs)	600
Number of Installed Generating Units	2
Unit Generating Capacity (cfs)	750
Dynamic Head (feet)	256
Static Head (feet)	150
Unit Pump Power (hp)[a]	22,000
Unit Turbine Power Produced (MW)[a]	5.4

Afterbay

Approximately 100,000 square feet of the afterbay with bottom elevation of 35.0 feet also will be constructed. The fish screen facilities are located on the east of the west levee, and connected to the afterbay by ten 9-foot-diameter pipes and the flood gates. A maintenance and access road is proposed around the afterbay at elevation 72.0 feet. The maximum and minimum water elevations in the afterbay are 70.0 feet and 51.0 feet, respectively. Water also would flow back through turbines at the plant, generating electricity and reducing the discharge head, then discharge into the afterbay back through the levee, the fish screen, and into the river.

A sediment spoil area, for the afterbay sediment removal, is provided on the north-eastern end of the afterbay. To remove sediment, a long-reach excavator is required in combination with a suction dredge or a clamshell. The suction dredge or clamshell would be used to remove the additional sediment in the area where the excavator cannot reach. The sediment ultimately will be hauled off site.

Air Chamber

An air chamber (Figure B.3-39) will be required on each discharge line to reduce the surge pressures in the pipeline. The air chamber is sized so that the upsurge pressure is limited to 125 percent of the rated pumping head (256 feet).

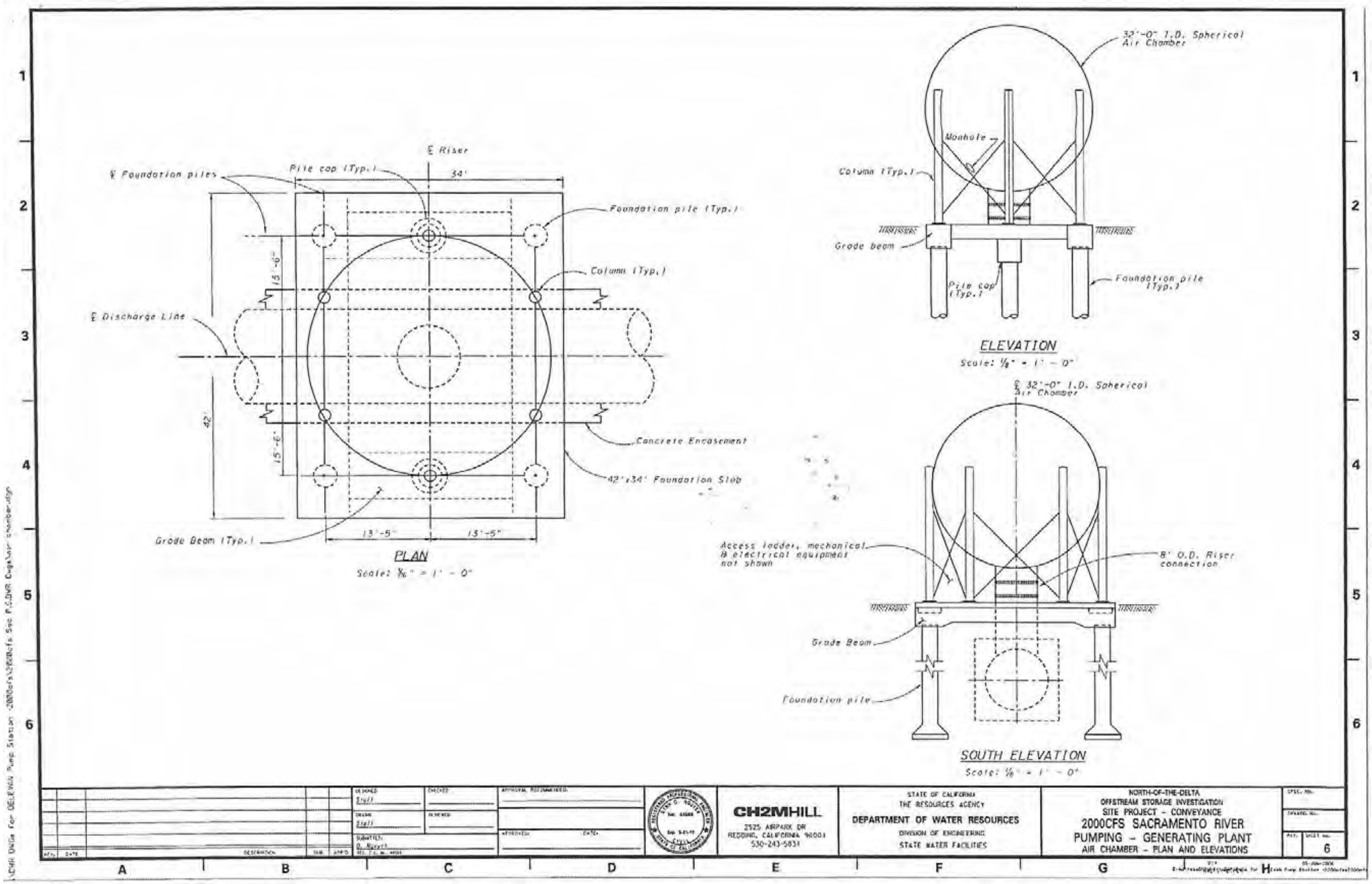


Figure B.3-39. Sacramento River Pumping-Generating Plant Air Chamber Plan and Elevations

Two 32-foot inside-diameter spherical air chambers rated to 150 psi with 8-foot-diameter outlets connected to the Delevan Pipelines will be fabricated and installed near the pumping-generating plant building to relieve water hammer pressures.

Access Road

To have easy and safe access to the plant, a new on/off ramp from/to Highway 45 would be constructed. The width of the access road would be approximately 40 feet. The road would lead to both the plant building and flood gates.

Mechanical Features

Mechanical features of the SRPGP would include:

- Pumping and Generating Units
- 84-inch online spherical valve on each discharge line
- Air chambers and butterfly valves with hydraulic power units
- Compressors
- Generators
- Gantry crane – 100 tons
- Service air and water systems
- Acoustical flowmeters

Electrical Features

Electrical features of the SRPGP would include:

- Switchyard
- Governors
- Transformers
- Control system
- Switchgears
- Grounding grids
- Control cabinets

B.3.8.2 Facility Location Selection

Site Geomorphology

The *June 2008 Sacramento River Fish Screen Facility Feasibility Study* findings are summarized below.

The proposed fish screen facility location, on the west bank of the Sacramento River at RM 158.5, was chosen on the basis of favorable hydraulic conditions (reduced bedload movement and aligned flow lines), geologic stability of the bank, and minimization of riparian disruption (Figure B.3-40).

The following historical river meander information is taken from the DWR's 2007 pre-design report for this site:

> "The proposed Sacramento River Pumping Facility location is in a section of river that is generally considered active. Within this section of river the main river channel has meandered significantly throughout the monitored history of the river (1896 to present)."

Observation of the over 100 years of river meander data at Moulton Weir shows that the west bank of the river was extremely stable until 1960, when it moved 138 meters east. Maxwell Irrigation District, which has a screened intake very close to the proposed pumping facility location, has had to move its facility as the river has meandered to the east. Since this time the river has slowly moved westerly (Figure B.3-41). The existing data indicate that the present location (2004) is less than 40 meters east of the most westerly extent of the 100-year meander belt.

Figure B.3-41 also shows that the present configuration of the river channel is quite narrow by comparison with historic conditions. A wider river channel could significantly change the sweeping flow velocities and sediment transport characteristics in the area, and change the operational capacity of fish screens. The 2004 configuration shows an approximate approximate river channel width, at the proposed pumping facility location, of 90 meters. The 100-year average channel width at this location has been approximately 268 meters and has exceeded 600 meters in width at times. Since the completion of Shasta Dam in 1946, the river has averaged 225 meters in width. River meander data over the last several decades suggest that the point bar to the north of the proposed pumping facility location has been quite active, moving generally south (Figure B.3-41). If this trend continues, the point bar could eventually envelop the pumping facility location, cutting it off from the river channel. The point bar is currently approximately 460 meters north of the proposed pumping facility location.

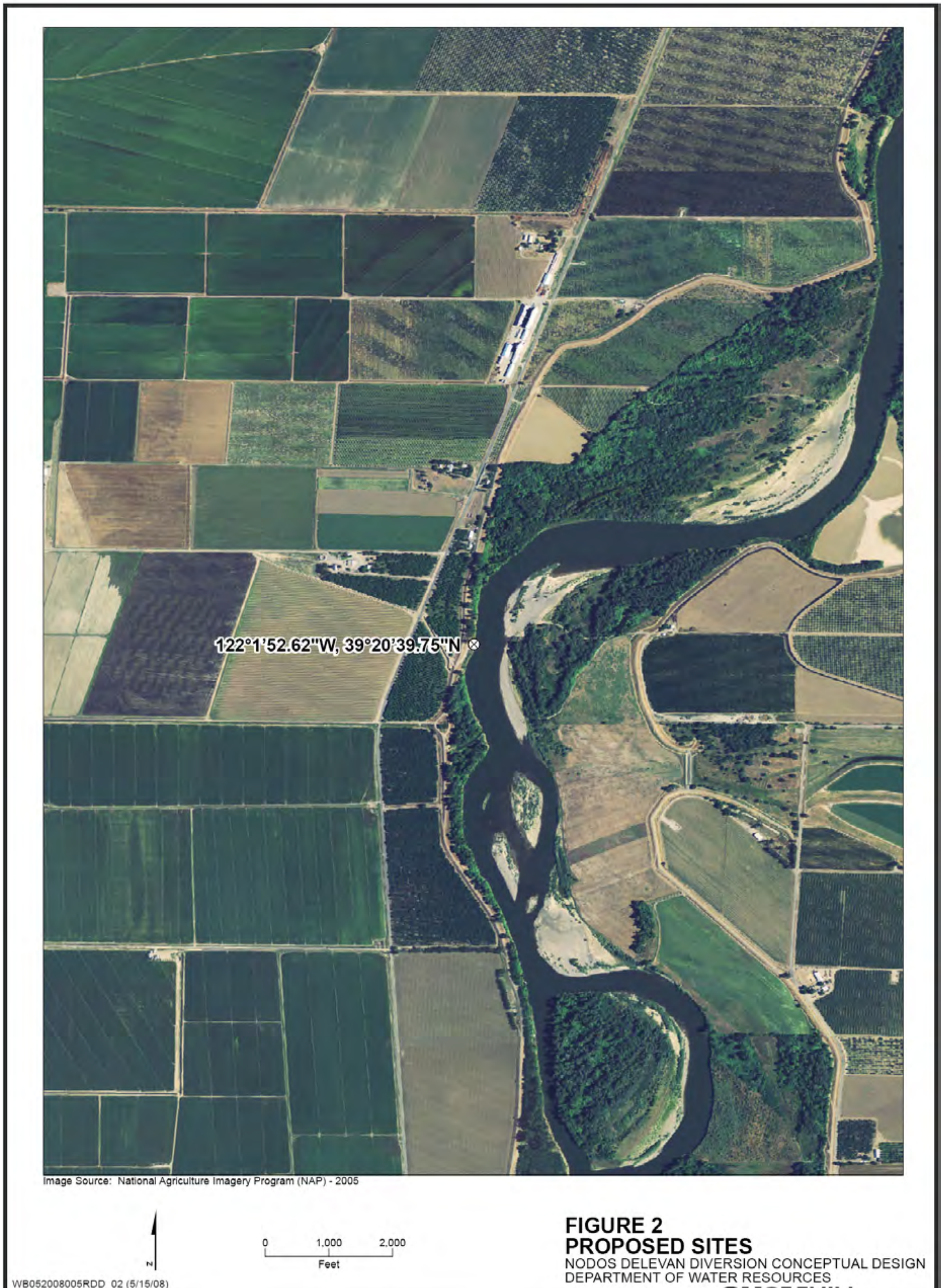


Figure B.3-40. Proposed Site of Sacramento River Pumping-Generating Plant

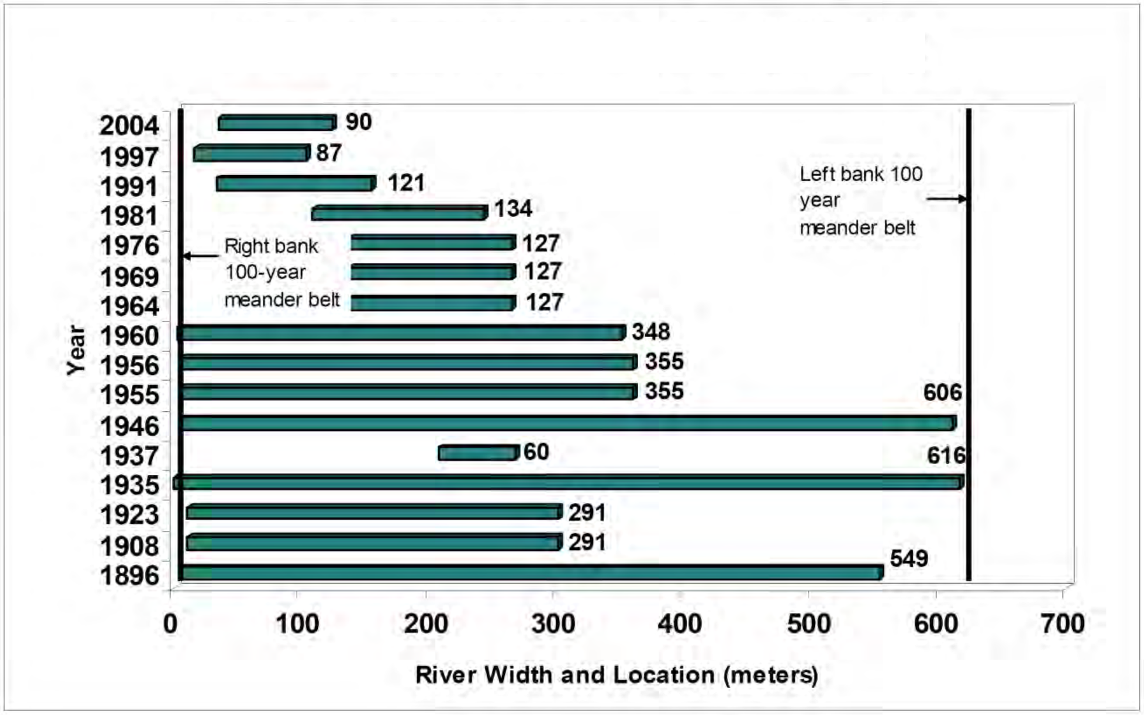


Figure B.3-41. Width and Location of Sacramento River at Moulton Weir (1896 to 2004)

River Hydraulics

The Sacramento River is a regulated river that is largely controlled by Shasta Dam. Several unregulated side streams between Redding and the project site can contribute significant runoff in the winter. The river has several overflow weirs, but they are downstream of the project site. The flood flows in this reach of the Sacramento River can exceed 150,000 cfs with some regularity.

Velocity Profiles

Previous work by DWR in 2007, and related DWR experience along the Sacramento River both upstream and downstream of this site indicate that sweeping velocities will not be problematic for this project. The location of this site on a stable, outside river bend are favorable for sweeping velocities, fish passage, and prevention of sediment deposition in front of the fish screen structure. The following passage is taken from the DWR's 2007 pre-design report for this site.

> "Using bathymetric data at 2-foot contour intervals, cross-sections were drawn at 25 foot intervals along the horizontal center line of the river channel. Three cross-sections [shown in Figure B.3-42] were selected and analyzed to determine the stage-velocity-flow relationship. The locations were chosen to develop the proposed Sacramento River Pump Station intake location. Using the cross-sections, the area and wetted parameter of the channel were calculated for various stage heights. Manning's Equation was then used to calculate the theoretical average velocity of the river at these locations for the various stage heights.

The analysis shows that even at very low stages with theoretical flows of 500 to 2,000 cfs, the river velocity stays above 2 feet per second. CALSIM-II operations runs for NODOS project Alternative WS1B indicate that diversions from the proposed Sacramento River Pumping Plant would take place at minimum river flows greater than 4000 cfs. Measured velocity profiles of the area will be required for final design of the fish screen facility; however, these data indicate that sweeping velocity should not be a limiting factor in the design and operation of the facility. An assumption of sweeping velocity greater than 0.66 feet per second is reasonable for planning level design."

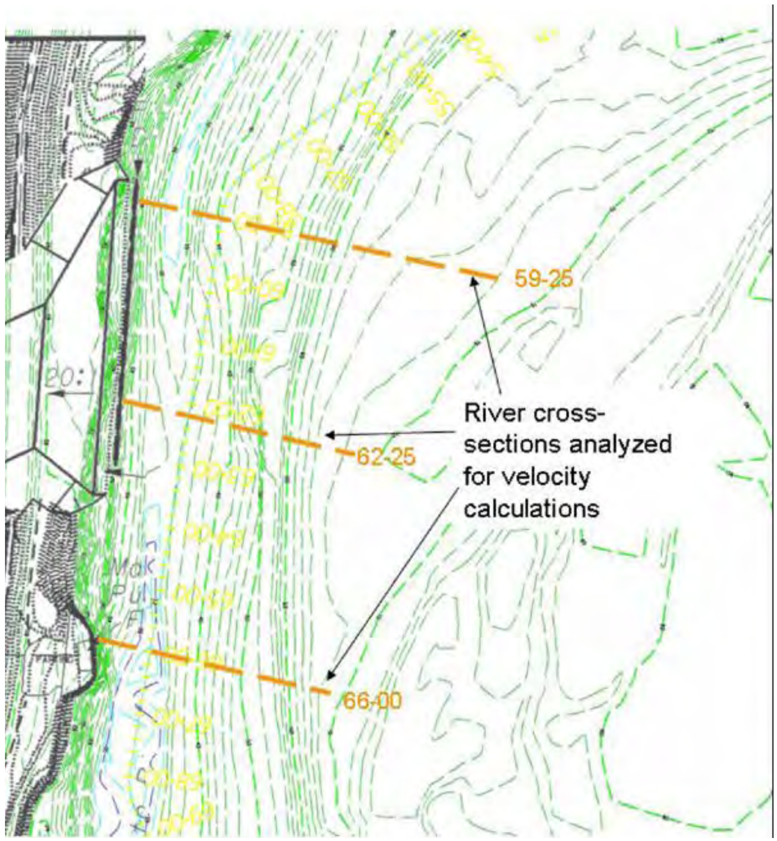


Figure B.3-42. Plan View of Project Location with Cross-Sections

Development of the Rating Curve and Monthly Flow Duration

A rating curve (Figure B.3-43) for the fish screen facility was created using a mathematical method and four measured points. Flow-duration information was developed on a monthly scale to look at seasonal influences and flows in the river.

Flow-duration curves were developed from a single gauging station. Using the daily flow data at each station, flow data at the Butte City gauge, considered to be more applicable to the fish screen facility, flow-duration curves were developed from the flow-duration table with daily flow data compiled from 1970 to 1995.

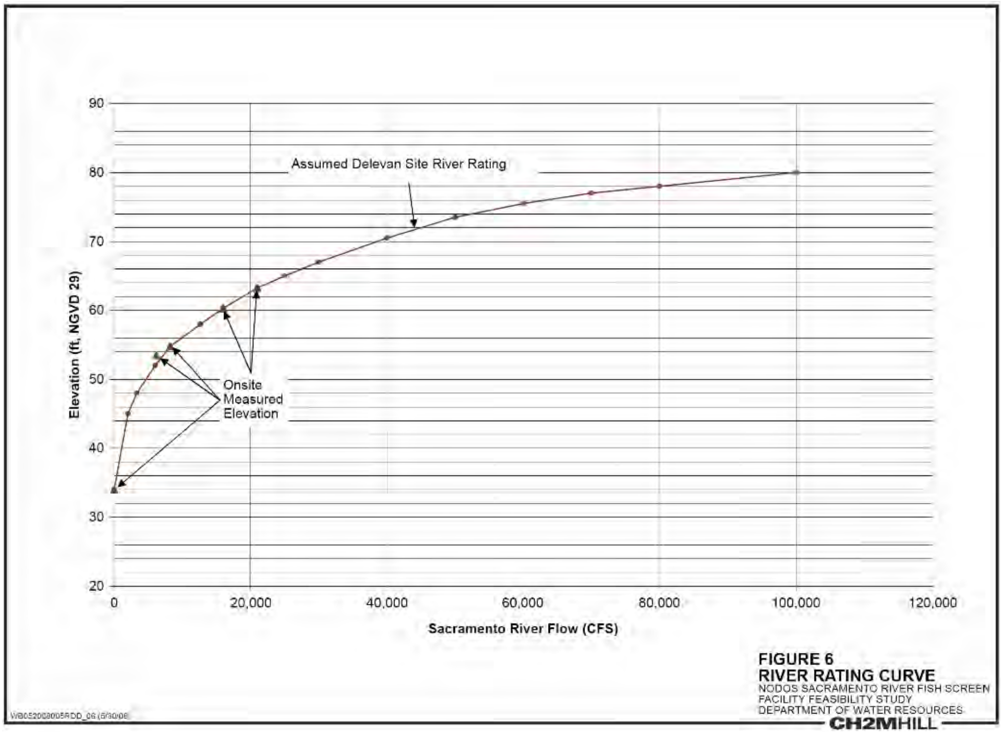


Figure B.3-43. River Rating Curve

Assuming normal-depth flow, rating curves at Butte City and Colusa, and using the following measured and assumed points shown in Table B.3-11, a rating curve (Figure B.3-43) developed for the fish screen facility.

Table B.3-11. On-Site Measured Points, North-of-Delta Offstream Storage, Sacramento River Fish Screen Facility Feasibility Study

Date of Point Taken	Maxwell Irrigation District Deck Elevation (feet, NGVD)	Depth to Water Surface (feet)	Calculated Sacramento Water Surface Elevation (feet)	Flow Rate (cfs)
2/5/2008	71	10.67	60.33	16,000
2/27/2008	71	7.75	63.25	21,000
3/10/2008	71	16.20	54.80	8,230
4/4/2008	71	17.50	53.50	6,212

cfs = cubic feet per second
NGVD = National Geodetic Vertical Datum

B.3.8.3 Sacramento River Intake Fish Screens

Two intake alternatives that were considered are described as follows:

- Alternative 1 – Flat Plate Fish Screen – This alternative consists of one large structure with minimal moving parts drawing water through 13-foot by 15-foot flat plate screens of stainless-steel wedge wire. Multiple fish screen bays would convey water through the structure, and blowout bays would allow for equalization of WSE in an emergency situation. No separate discharge facilities would be required with this alternative. Water would be discharged back to the river through the screening facility.

- Alternative 2 – T-screens – This alternative consists of one large structure with significant moving parts drawing water through 14 units with a total of 28 fish screen barrels. A separate discharge facility would be required for this alternative.

Alternative 1 was selected as the preferred alternative for the intake fish screens. Aspects of this alternative are described below.

Design Criteria and Assumptions

The development of the conceptual fish screening and discharge alternatives considered as part of this project is based on certain design criteria and assumptions, as follows:

Regulatory criteria as presented and discussed with the Anadromous Fish Screen Program Technical Team that is composed of representatives from the following federal and state resources agencies: USFWS, Reclamation, National Oceanic and Atmospheric Administration Fisheries, DWR, and California Department of Fish and Game (DFG).

- Mechanical and structural criteria based on engineering principles, and design and construction experience of similar structures along the Sacramento River and in the Pacific Northwest.

- Operational and maintenance criteria based on discussions with DWR and experience with diversions along the Sacramento River.

- River hydraulics.

Regulatory Criteria

The following regulatory criteria were considered for fish screen intakes:

- Average approach velocity (water velocity perpendicular to the screen, 3 inches from the face of the screen), less than or equal to 0.33 fps

- Minimum sweeping velocity (water velocity parallel to the screen, 3 inches from the face of the screen) of two times the approach velocity

- Uniform distribution of the approach velocity across individual fish screen panels

- In-river construction window of April 1 to November 1 – a waiver is required

- Screen slot opening size of 1.75 millimeters

Mechanical and Structural Criteria

The following mechanical and structural criteria were considered:

- The structure will be constructed in the dry using a cofferdam.
- The structure will need to divert water at high river levels and high river flows.
- The structure will need to divert water within a varying degree of river elevations.
- The invert elevation of the structure will be set at 38 feet.
- Facilities will be sited and designed such that electrical equipment will be sealed or placed at an elevation above the 100-year flood stage plus 2 feet of freeboard (elevation 84 feet).
- A log boom will keep large, floating debris from hitting and potentially damaging the screen.
- Panels will be 13 feet tall by 15 feet wide with an effective screen area of 95 percent.
- Panels will be made of stainless-steel vertical-wedge wire.
- Panels will be positioned with minimum protrusion into the river channel parallel to river flow.
- The fish screen structure will contain the following:
  - Two blowout panels in the event that water levels need to be rapidly equalized to maintain the integrity of the structure (i.e., prevent a significant overturning moment).
  - A louver system to manipulate the inflow uniformity across the screens.
  - A continuous catwalk below the fish screen structure deck with an access hatch to provide adequate access for screen and louver maintenance.
  - A sediment removal system that keeps river sediment from accumulating in the fish screen structure (sedimentation affects the uniformity of the fish screen approach velocity).
  - A screen cleaning system that keeps small debris from plugging the screen within the parameters set by the resource agencies.
- Pier walls separating each bay will be 1.5 feet wide.

- Screen panels will be submerged to the degree necessary to allow diversions without violating fish protection criteria.

Operational and Maintenance Criteria

The following operational and maintenance criteria (discussed in greater detail in following sections) were considered:

- Diversion operations will be such that a minimum of 4,000 cfs will remain in the river channel immediately downstream of the diversion point.

- Water will be diverted within a minimum of 5,000 to 6,000 cfs in the river channel immediately upstream of the diversion point. The assumed associated minimum WSE at this design condition is 51 feet.

- A sediment removal system will be installed within the fish screen bays, moving sediment back into the river channel or into the forebay. Sediment that has settled out into the forebay will be removed mechanically to maintain optimal operational hydraulics. This effort likely will be an annual operation.

Fish Screen Structure Details

The fish screen structure, as shown on Figure B.3-44 for a 2,000-cfs diversion, consists of thirty-two 13-foot by 15-foot flat plate screens, two blowout bays, two fish screen cleaners, a sediment removal system, and tuning baffles. Each item is necessary for the proper function of the proposed fish screen.

The flat plate screens provide the screening mechanism to prevent fish from entering the pumps. The blowout bays prevent damage to the fish screen structure by providing a mechanical safeguard to prevent differential levels in WSE between the forebay and the Sacramento River from exceeding 4 feet. The fish screen cleaner is a mechanism that moves along the river side of the screen and removes debris buildup by means of a large brush. The sediment removal system is a system that pumps water into the sediment removal piping and forces the water out at high pressure through nozzles in each bay. This jetting action suspends and moves the sediment out of the fish screen structure. The tuning baffles behind the screen panels provide a means to distribute the velocities of water across the fish screen to reduce uneven distribution of flow across the structure.

The fish screen's size, as shown on Figure B.3-44, allows a maximum pumped flow of 2,000 cfs through the structure while not exceeding an intake velocity of 0.33 fps across the screens. Using the river's flow/stage durations and a design river flow of 6,000 cfs, the minimum river stage at which the facility will operate is 51 feet. The invert of the screen structure is set at 38 feet, 4 feet above the invert of the river; this position will reduce sediment deposition in front of the screen panels. The river's stage and the screen's invert at the designed flow results in a screen height of 13 feet. For this design, the screens are 15 feet wide, and an estimated 32 screen bays are required to match the desired 0.33-fps velocities across the screen. The structure would be 559.5 feet long, the distance created by 32 screen bays, the additional two blowout bays, and room for screen cleaning equipment.

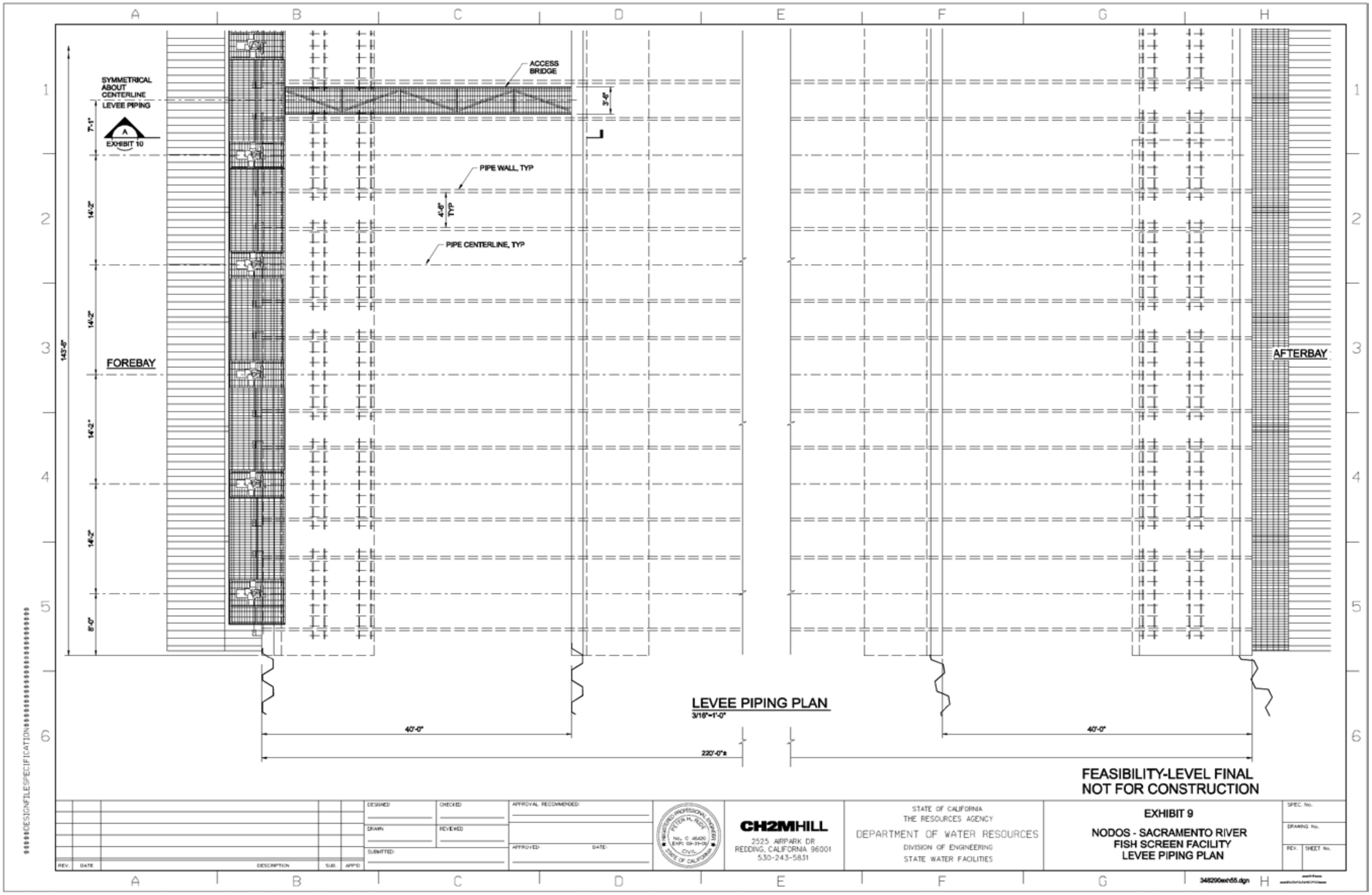


Figure B.3-44. Sacramento River Fish Screen Levee Piping Plan

The levee tubes, as seen on Figure B.3-45, are a means to transfer water from the river through the levee to the pumping-generating plant afterbay. As water enters the tubes, it will pass through a trash rack and sluice gate. It will then travel through ten 9-foot-diameter pipes and exit through a trash rack into the afterbay. The tubes will handle a maximum flow of 2,000 cfs and maintain a velocity no greater than 3 fps during operation. The velocity through the pipes will allow for sediment to pass through without settling out, and it will also minimize turbulence. The levee tubes' design will maintain a maximum velocity of 3 fps at 2,000 cfs during pumping intake and 2.3 fps at 1,500 cfs during discharge.

The sluice gates, as seen on Figure B.3-45, provide flow control on the pipes. The trash screens prevent debris, people, or animals from entering the pipes.

Operation

Operation and maintenance of the facility includes the fish screen facility, the screen cleaner system, the blowout panels, the sediment removal system, the SCADA system, the fish screen structure, the sluice gates, lighting, and maintenance schedules.

The fish screen facility will operate in three different operational modes: the intake mode, the discharge mode, and the emergency mode.

Intake Mode

Intake mode occurs when the diversion is going to be pumping water from the Sacramento River to Sites Reservoir. The intake mode will be when the facility sees the largest flows and velocities. Flow of water will move through the fish screen into the forebay, through the levee tubes, into the afterbay, and finally be pumped to Sites Reservoir. During this operation, the screen cleaning mechanism will be working continuously to prevent buildup on the screen panels, the sediment removal system will be operating, the pumps will be operating, and the SCADA system will be monitoring water levels and pressures across the screen.

Discharge Mode

Discharge mode occurs when water from Sites Reservoir will flow back through the pumping plant to generate electricity, and the water will be discharged into the afterbay, through the levee tubes, into the forebay, and then through the fish screen and into the Sacramento River. During this operation, the fish screen cleaning mechanism will not need to be operating because it is located on the river side of the fish screen, the sediment removal system will remain in operation, and the SCADA system will be monitoring water levels and pressures across the screen.

Emergency Mode

When the diversion is operating in intake or discharge mode and a pressure differential greater than 1.5 feet across the fish screen occurs, emergency mode is activated. The pumps will stop operation, and the sluice gates will close to allow the

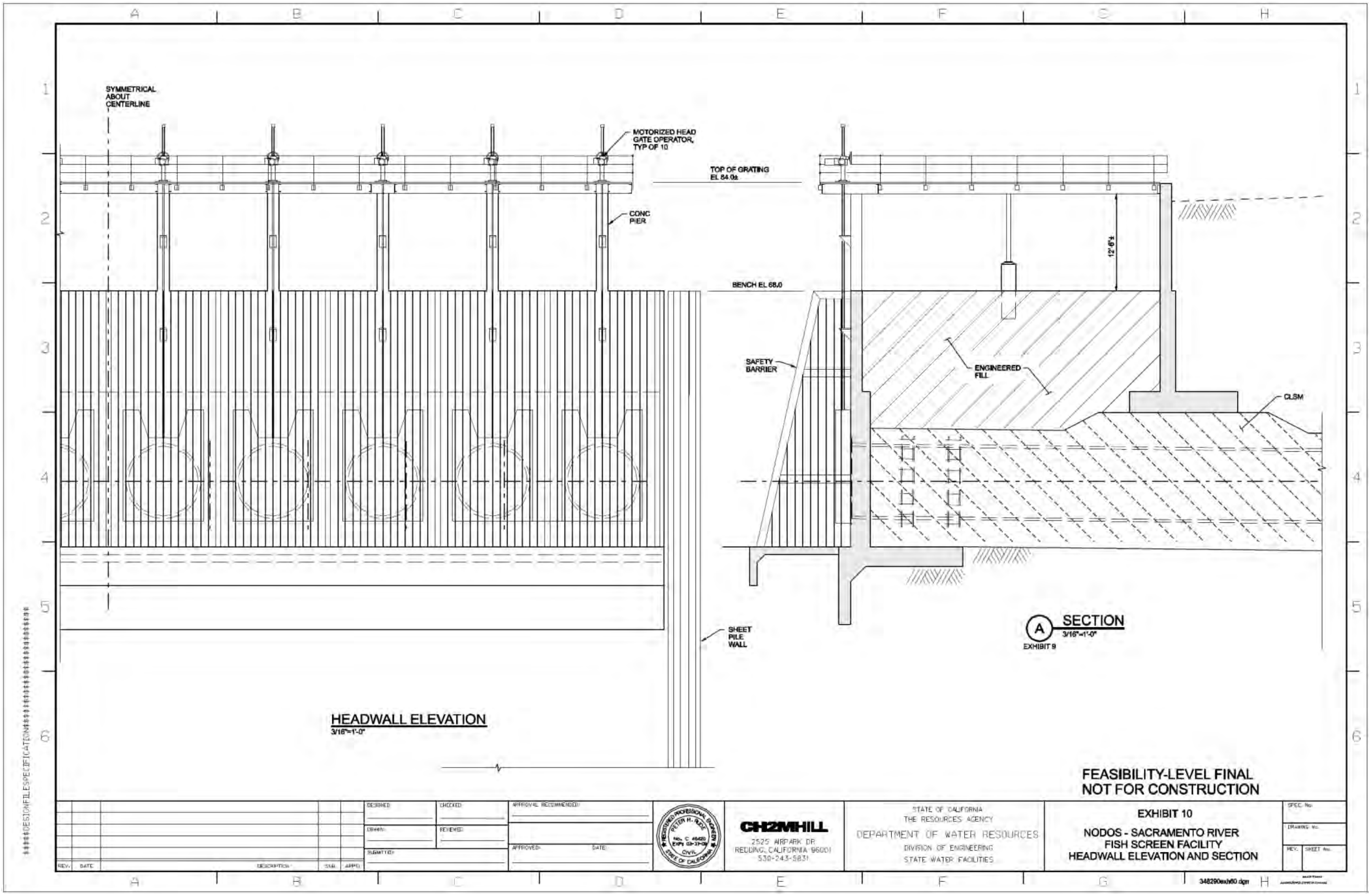


Figure B.3-45. Sacramento River Fish Screen Facility Headwall Elevation and Section

forebay to fill up to match the water surface of the Sacramento River. If the pressure differential grows to above 3 feet, then the two blowout panels will trigger and release to allow an inflow of river water to allow the water levels to equalize. In addition, in the case where the river is at a very high water level and the pumps are to turn off, the sluice gates should close at the same rate the pumps power down to prevent flooding of the afterbay.

A SCADA system will control all of the different operational modes. The system will be located on site and will broadcast status information to a manned remote location. The SCADA systems provide a means to control the diversion without staffing the onsite facility.

Sediment Removal Fish Screen Bay Sediment Removal System

The fish screen bay sediment removal system consists of piping and jetting nozzles that remove sediment buildup from the bottom of the fish screen structure. Over time, sediment will build up on the floor of the fish screen structure, and the sediment removal system prevents that buildup. The sediment removal system will operate for approximately 10 minutes per bay three times per week, and will suspend the built-up sediment, which is then removed with the flows through the structure.

The forebay sediment removal system requires special equipment to remove built-up sediment between the fish screen and the levee piping. Depending on sediment load in the river, the forebay will need to be cleaned approximately once per year, removing approximately 3 to 5 feet of sediment (3,800 to 6,300 cubic yards) during each cleaning. To remove the sediment, a long-reach excavator is required in combination with a suction dredge. The suction dredge will remove sediment from approximately 40 percent of the forebay that the long-reach excavator cannot reach. The suction dredge will have a pump that is in the range of 50 to 60 horsepower and will pump the sediment at a rate of approximately 600 to 700 gallons per minute through approximately 500 feet of 6- to 8-inch-diameter discharge line to the sediment disposal area located southwest of the forebay, as shown on Figure B.3-34.

B.3.8.4 Delevan Pipeline Discharge Facility, Alternative B

Because Alternative B does not include the SRPGP, the Delevan Pipeline Discharge Facility would be required at the Sacramento River to make releases to the river in a controlled fashion. Figures B.3-46 and B.3-47 show details for the proposed discharge structure. The Delevan Pipeline would be reduced in stages from 12 foot diameter, to 8 foot, then to 4 foot before reaching the energy dissipating valve house. The valve house would be located just above the design Sacramento River flood level at the site, which is at an approximate elevation of 82 feet.

The energy dissipating valves would be 48-inch-diameter fixed-cone valves located in confining vaults to control excessive spray and help dissipate the energy. From the valve structure, release water would flow down a short channel section before reaching a baffle block spillway leading down to the river. The system is design for a maximum release flow of 1,500 cfs. The baffle block spillway is selected because it can convey the water to the river in a controlled fashion regardless of the river level.

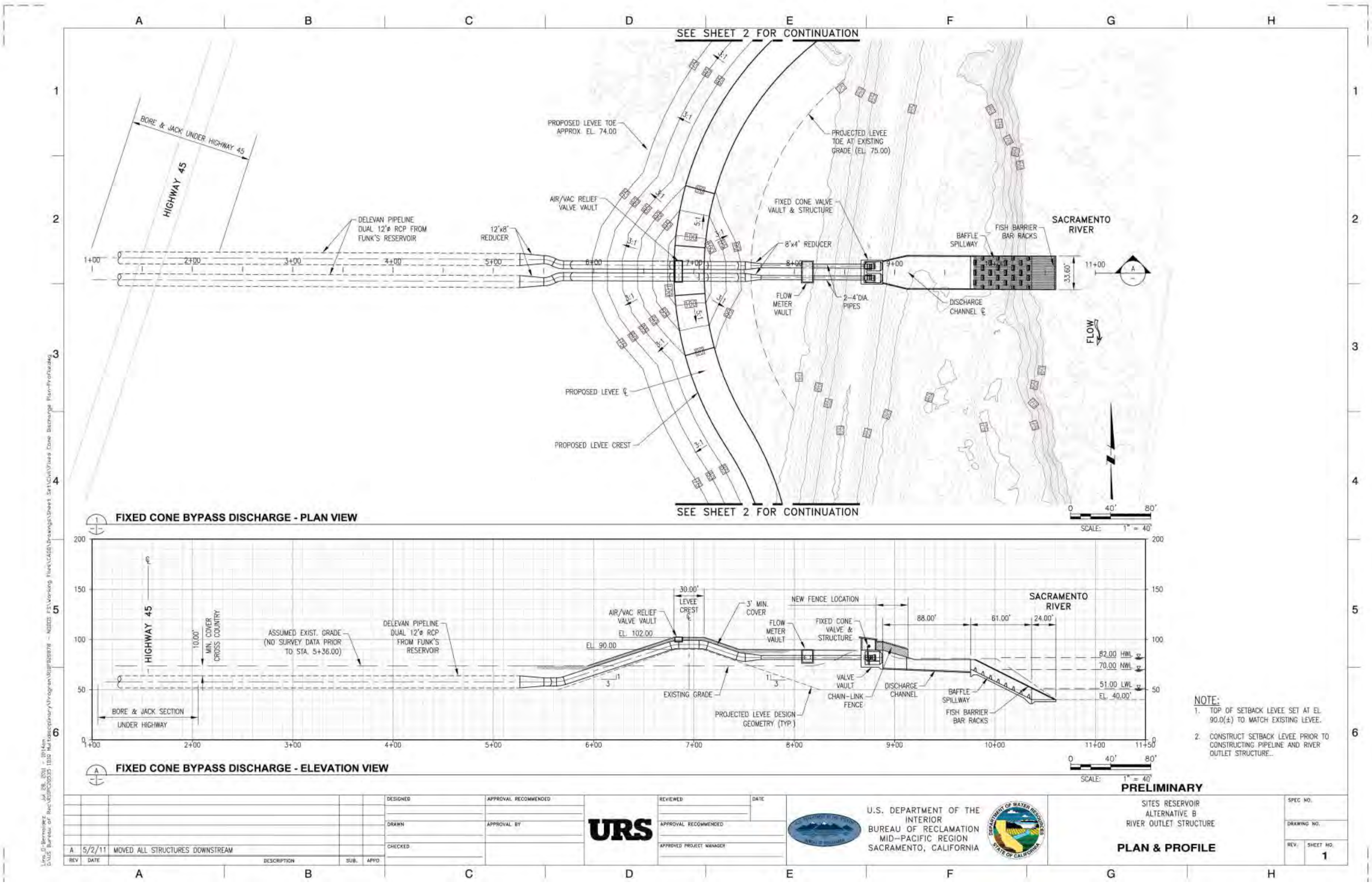


Figure B.3-46. Sacramento River Release Structure Plan

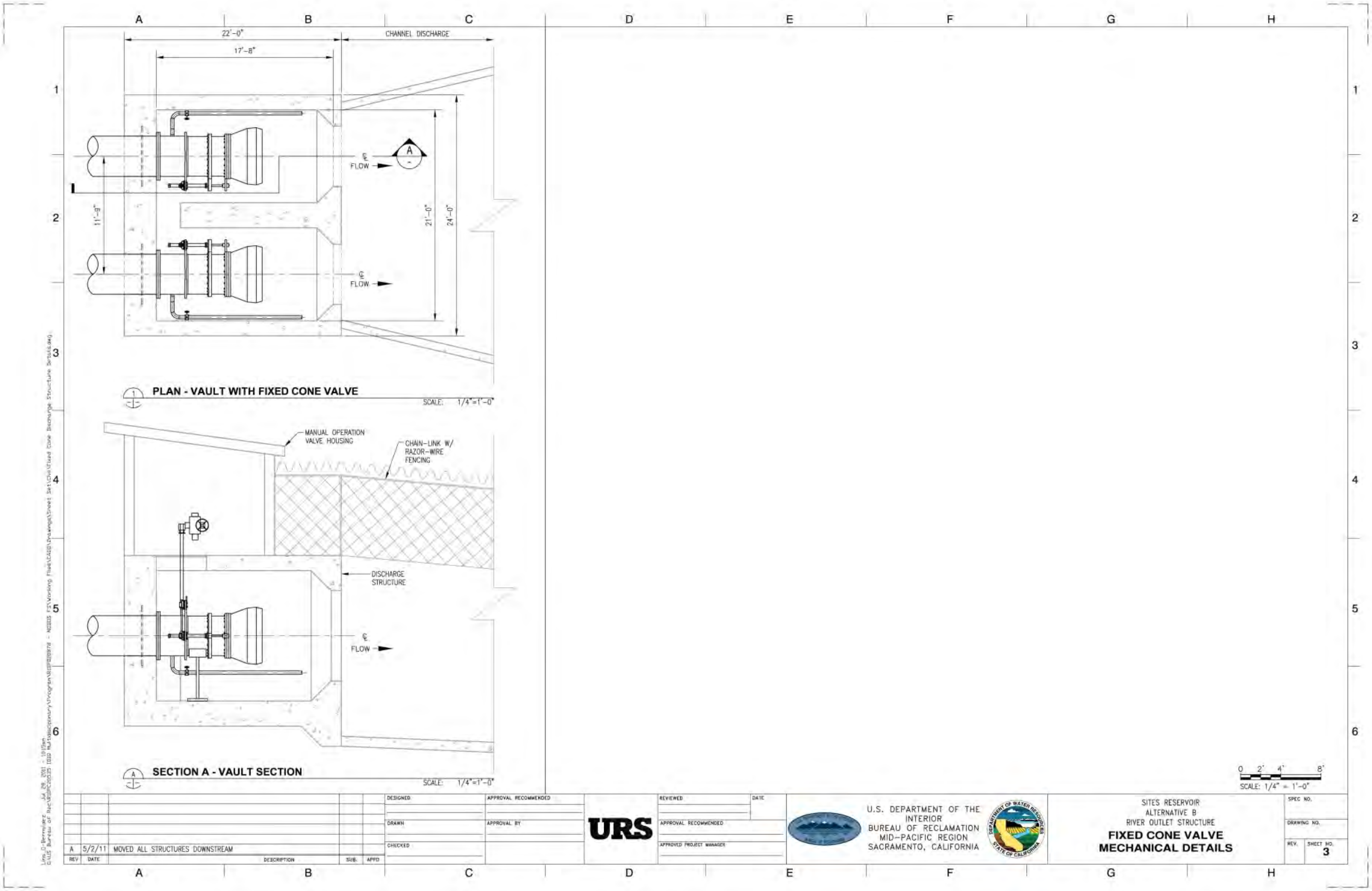


Figure B.3-47. Sacramento River Release Structure Control Valve House

The valve house, channel, and spillway would be located within the current river overbank area so that the facilities do not encroach within the flow area when the river is at its maximum design level. The downstream side of the spillway exposed to the river would be fitted with fish barrier racks to prevent migrating adults fish from entering the spillway chute. The clear spacing of the bars in the rack would be 1.5 inches.

At the maximum design flow, the width of the spillway structure would be designed to maintain the release velocity from the structure at or below 3 fps at the minimum river design level at an elevation of 51 feet.

The piping leading up to the valve house would include air/vacuum relief valves in a vault and a flow metering vault, as shown on Figure B.3-46. Air/vacuum valve vault details would be similar to those shown on Figure B.3-48.

The release structure is located in a reach of the Sacramento River that is protected by a federal project levee system administered by USACE and the Central Valley Flood Protection Board. The top of the levee in the project area is at an elevation of 90 feet. Before the levee is breached and construction begins on the valve house and spillway, a setback levee must be constructed around the site as shown on Figure B.3-48. The setback levee will tie into the exiting levee on the north and south sides of the site so that there is no interruption of flood protection during construction. In addition, current levee regulations do not permit piping to pass through or under project levees. For this reason, the piping is elevated where it crosses the setback levee so that it is above elevation 90.0.

Construction of the release facility would include the following:

- Constructing the setback levee along with any slurry walls that might be required to control through-seepage and underseepage.

- Constructing a cofferdam along the shore of the river to permit spillway construction in dry conditions.

- Excavation of the bank and backfilling of the area to construct the spillway, channel and valve house. Approximately 6,000 cubic yards of excavation would be required.

- Backfilling around the structures and on the waterside of the setback levee as required.

- Placing riprap rock slope protection for a minimum of 100 feet upstream and downstream of the spillway to control erosion.

- Installing revegetation measures.

Construction of the Delevan Pipeline from the setback levee to Holthouse Reservoir would be the same as described above for conveyance.

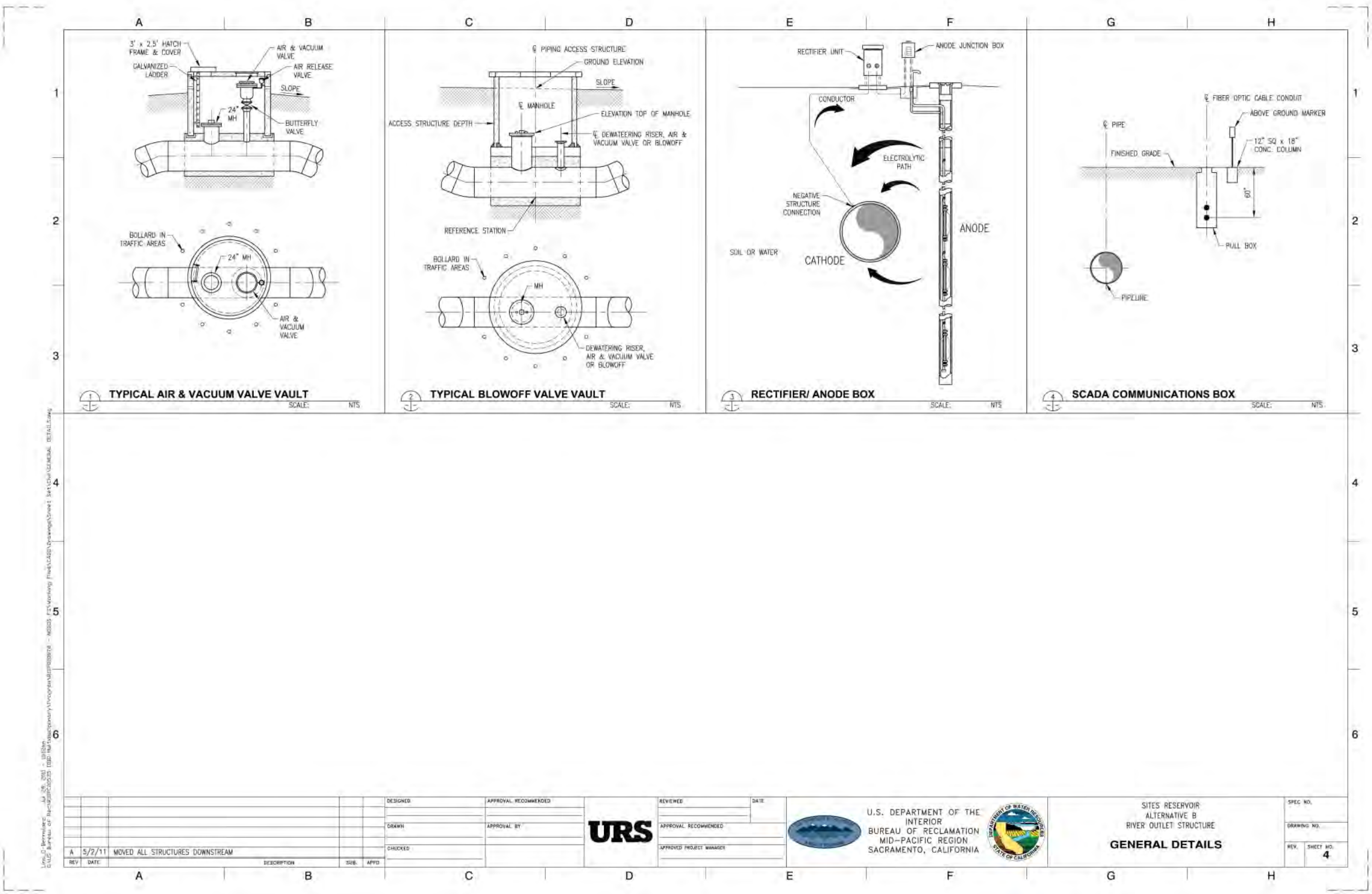


Figure B.3-48. Typical Pipeline Air Valve and Blowoff Details

B.3.8.5 Delevan and Terminal Regulating Reservoir Pipelines

Two new pipelines would be constructed to convey water between project facilities. The approximately 13.5-mile-long Delevan Pipeline would convey water from the Sacramento River to Funks Reservoir to fill Sites Reservoir, and would also convey water from Funks Reservoir to the Sacramento River for releases. The 3.5-mile-long TRR Pipeline would convey water from the TRR to Funks Reservoir. Located to the west of the TRR, the TRR Pipeline would parallel the Delevan Pipeline and would share a common trench and outlet structure into Funks Reservoir.

Both pipelines would be bi-directional, allowing water to be pumped from the Sacramento River (Delevan Pipeline) or the TRR (TRR Pipeline) to Holthouse Reservoir for storage, and allowing water to flow by gravity from Holthouse Reservoir for release to the Sacramento River or the TRR/GCID Canal. As water released from Enlarged Funks Reservoir flows through the pump stations at the end of the pipelines, it would pass through turbines to generate electricity.

The Delevan Pipeline would convey 2,000 cfs from the SRPGS to Holthouse Reservoir. The pipeline would convey 1,500 cfs of water from Holthouse Reservoir to the Sacramento River. The pipeline would consist of two 12-foot-diameter reinforced concrete pipes.

The TRR Pipeline would have a 1,800 cfs capacity to convey water from the TRR to Holthouse Reservoir. The capacity of the pipeline to convey water from Holthouse Reservoir to the TRR would be 1,500 cfs. The pipeline would consist of two 12-foot-diameter reinforced concrete pipes. Both pipelines would be buried a minimum of 10 feet (to top of pipe) below the ground surface.

Although both pipelines have the same diameter, they will convey different quantities in diversion mode because their pump stations have different capacities. A 12-foot-diameter pipe provides a range of diversion capacities. In reverse flow/ generating mode, the capacities of the pipelines are approximately 25 percent lower than in diversion mode because in reverse flow mode the water flows only by the force of gravity. In diversion mode, the water flows because of the force applied by the pumps.

The Delevan Pipeline would begin at the SRPGS near the Sacramento River. The pipeline would be aligned due west until reaching the GCID Canal. At the GCID Canal, the Delevan Pipeline would turn southwesterly and would parallel the TRR Pipeline in a shared trench until it reaches the Holthouse Reservoir.

The proposed pipeline would be underground and would have two types of aboveground features. All other appurtenant structures would be below ground. Table B.3-12 lists the proposed aboveground features and their physical characteristics.

Table B.3-12. Aboveground Features Associated with the Proposed Delevan Pipeline

Aboveground Feature	Height Above Ground	Color	Appearance	Feature Locations
Manhole and Air Valve	4 feet maximum, 108 inches diameter	Gray	Concrete Box	At high points and at a minimum of every 2,500 feet along the pipeline
Manhole and Blowoff Valve	1 foot maximum, 108 inches diameter	Gray	Concrete Box	At low points and at the Sacramento River and the GCID Canal Crossing

GCID = Glenn-Colusa Irrigation District

Facilities associated with the Delevan and TRR Pipelines are:

- Blowoff structures
- Air valve structures
- Crossings
- Siphon outlet and energy dissipater structure
- Relocation of existing transmission power lines, natural gas pipelines, water lines and other utilities

These facilities are described below.

Blowoff Structures

Blowoff structures would be provided to clean low points in the pipeline and allow dewatering at the Sacramento River and at the GCID Canal, at a minimum. Structures would be 108 inches in diameter and would have a 30-inch manhole and 16-inch blowoff valve. Blowoff valves release water from the pipeline. These valves are located at major water conveyances so that water can be drained directly into the river or canal and carried downstream. Figure B.3-48 shows typical blowoff details.

Air Valve Structures

Air and vacuum valves are required to evacuate air within the pipeline during filling, and supply air during normal dewatering, as well as to release accumulated air. Air and vacuum valves would be located at high points in the pipelines. The distance between valves would not exceed the manufacturer’s recommended spacing of 2,500 feet. The structures are 108 inches in diameter and would house a 30-inch manhole and a 14-inch nozzle for the air and vacuum valve assembly. The manhole would be used to access the pipeline for future maintenance or inspections. Figure B.3-48 shows typical air valve structure details.

Crossings of Existing Utilities

The proposed alignment of the pipelines would require three crossings of major existing infrastructure. At these crossings, the pipelines would be tunneled through the ground below the existing infrastructure so that services would not be interrupted. The three crossings are: Interstate 5, Union Pacific Railroad, and the GCID Canal. At these locations, the bore and jack construction method jack would likely be used. Bore and jack construction entails excavating a large pit on each side of the existing infrastructure (highway, railroad, or canal in this case) and then tunneling horizontally under the structure without disturbing it. All additional work required for bore and jack construction would be conducted within the construction easement and would not require the disturbance of additional land.

The proposed pipeline routes would also require crossing the easements of two high-voltage electrical transmission lines: the WAPA 230-kilovolt (kV) and the Pacific Gas and Electric Company (PG&E) 500kV lines. No permanent aboveground structures, other than a gravel maintenance road, would be constructed where the electric utility easements and the pipeline easements would intersect. In addition, the Delevan Pipeline would also cross the CBD. This is an agricultural drain that conveys agricultural tailwater and runoff from approximately one million acres of agricultural land to the Sacramento River. The crossing location would be at the northern end of the drain. Construction of this crossing likely would take place during late fall, after the irrigation season and before winter rains begin. The CBD likely would be drained so that the pipeline trench could be excavated and the pipeline could be installed. After installation, the CBD would be returned to service and reconstructed to pre-project conditions.

Other existing infrastructure that the pipelines likely would cross include: gas lines, water lines, sewer lines, communications lines, and other infrastructure. The proposed pipelines likely would be installed beneath most existing utilities. Disruptions to these utilities would be minimized to the extent possible, and the ground surface would be restored to pre-construction conditions after pipeline installation. Construction activities for the proposed pipelines and modifications to existing utilities would occur within the identified construction easement and would require only slightly more excavation than that required for the pipeline.

Project Construction

Pipeline construction likely would be done in three independent and concurrent sections. Two of the sections likely would begin from the same point and move in opposite directions. As pipelines are installed and tested, the trench would be backfilled to minimize the amount of open trenching.

The construction easement for the pipeline would be a linear area 13.5 miles long and approximately 300 feet wide from the Sacramento River to the TRR (10 miles), and approximately 335 feet wide from the TRR to Holthouse Reservoir (3.5 miles). The additional width of the easement is needed to accommodate the additional pipelines from the TRR to Funks Reservoir. An additional temporary easement of 20 acres would be required for a concrete batch plant. Easement boundaries would be marked with tape, flagging, or fencing. The easement would pass through multiple areas close to residences, and would intersect several roads. The entire pipeline construction area would not be fenced. In high-visibility areas or where the construction site requires a higher degree of protection for security or safety, a 6-foot-high chain link fence would be installed around the work site.

Trenching/Excavation of Pipeline Route

Approximately 6.3 million cubic yards of material would be excavated for the pipeline trench. Topsoil would be stockpiled separately from other excavated materials. Trench excavation would be approximately 23 feet deep. For the Delevan only section (Sacramento River to TRR), the trench would be approximately 120 feet wide. Trench excavation for the 3.5 miles from the TRR to Funks Reservoir would be approximately 165 feet wide to accommodate both the Delevan and TRR Pipelines. A total of four 12-foot-diameter pipes would be installed. Trench side slopes would be approximately 1H:1.5V. No shoring would be installed under normal excavation conditions. Special conditions at some locations (unknown at this time) may require additional depth or width, or steeper or flatter side slopes, or shoring to accommodate localized soil conditions.

The pipeline trench would be excavated using trenchers and tracked and/or wheeled excavators and backhoes, or pushed up using bulldozers. The type of soils encountered would determine the type of equipment used for trenching. Harder soils, such as caliche, would require larger trenchers. In specific areas, vacuum excavation, “pot-holing” with a backhoe or hand digging may be necessary to locate buried utilities.

Excavation activities similar to the pipeline excavation would also be done for electrical transmission pole footings that would be installed within the pipeline right-of-way (ROW). These activities would occur simultaneously with the pipeline excavation.

Dewatering

Dewatering of the trench would be necessary in many locations and could be permitted to discharge into local irrigation ditches and drainage canals and/or the CBD after settling of silts. Silts would be disposed of with excavated material. Dewatering would be in accordance with Central Valley Regional Water Quality Control Board requirements and California Storm Water Quality Association Best Management Practices for dewatering.

Bedding Preparation

One foot of bedding material would be installed in the trench before installation of the pipeline. Bedding material would likely be sand, consolidated backfill, or cemented controlled density fill. The bedding material would be poured into the trench by dump truck, and spread along the bottom of the trench by a small grader or similar type of equipment.

On-Site Fabrication of Pipe

All pipe would be fabricated on-site at the concrete batch plant. A fabrication and curing area for the pipe would be located within the 20-acre batch plant footprint. Pipe would be fabricated on site from straight lengths of reinforcing steel. This activity would require the use of a hydraulic reinforcing steel bender. Formed reinforcing steel pieces would be tied/welded together on site to complete the reinforcing steel structures. The structures would then be installed in pipe forms and then the forms would be filled with poured concrete.

Each section of pipe would be allowed to cure (dry and harden) for a minimum of 28 days before being moved into place along the pipeline. Movement of pipe sections within the concrete batch plant site would be done using a 50-ton capacity fork lift or wheeled crane. Fabrication of pipe would proceed year-round, and installation of pipe through the area that is considered giant garter snake (GGS) habitat would occur from May 1st through October 1st of each year of construction. In the western 3.5 miles of the pipeline that is not considered GGS habitat, this construction timing restriction would not apply. During spring and early summer, there would be pipe sections stockpiled on the site.

Installation of Pipe and Valves

The finished sections of pipe would be transported from the concrete batch plant to the installation location primarily along the pipeline route on flatbed trucks traveling along the construction access roadway (within the construction easements). These trucks would cross public roadways. Pipe sections would be offloaded from flatbed trucks and placed in the excavated pipeline trench by a 50-ton capacity crane. Once in place, the metal joining plates cast into the end of each pipe would be welded together and the joint would be covered with a cement-based sealing compound. At valve locations, pre-fabricated valves would be delivered to the site on flatbed trucks and installed into previously constructed structures within the trench using the same crane.

Backfill of Trench

Approximately 5 million cubic yards of material would be needed to backfill the trench after the pipes are installed. Excavated material would be reused to backfill the trench or moved to other project locations for use, to the extent possible, after placement of pipes. Excess spoils from the excavation (estimated at 1.3 to 6.3 million cubic yards) would be spread on adjacent agricultural lands of willing landowners within the 800-yard-wide corridor along the pipeline, used as backfill at the SRPGP, or placed in the Sites Reservoir footprint. Excess spoils may also be used to reinforce existing levees in the area under a separate program. Reuse of excavated material may be limited by water content of excavated material and soil compaction requirements.

B.3.9 Road Relocations and Access Roads

B.3.9.1 Introduction

This section evaluates the relocation of roads owned by individuals, counties, or other agencies that would be inundated or cut off by the construction of Sites Reservoir. Sites Reservoir would inundate portions of Maxwell Sites Road and Sites-Lodoga Road, blocking travel between Maxwell and Lodoga. These roads are owned by Colusa County. Approximately 6 miles of the gravel Huffmaster Road would also be inundated. This is a private road and provides access to properties mostly within the Sites Reservoir area. In addition, as the project would include five new recreation areas, road access to these sites would also be needed.

B.3.9.2 Road Relocations

General

Road relocations include the paved route from Maxwell to Lodoga along with gravel roads for construction, recreation, facility access, and landowner access not provided by the paved road.

Roadway Element Standards

Preliminary feasibility studies of road relocations conformed to the following roadway element standards:

- Both paved and unpaved roads are 52 feet wide.
- Maximum grade of 6 percent (State of California Highway Standard).
- Minimum horizontal curvature of 100 feet radius.
- Paved roads have 40 feet of asphalt.
- Cut-and-fill side slopes are 2H:1V for depths less than 20 feet, and 1H:1V with 10-foot benches for depths greater than 20 feet.
- Stream crossings require bridges or culverts. For preliminary studies, all culverts and bridges were considered to be the same size. Bridges are 40 feet wide and 80 feet long. Culverts are 6 feet in diameter and 100 feet in length. The Southern Route has the most stream crossings: 8 bridges and 29 culverts.

Alternative Alignments

Four alternative road alignments were evaluated. Two road alignments (North Road and South Road) were evaluated, as well as two road alignments that include bridge segments (North Bridge Road and South Bridge Road), as shown on Figure B.3-49. Road relocation would depend on final alternative selected, and on other associated features, such as in proposed recreation area locations. Bridge routes would provide more direct access with reduced travel times than would road routes around the north or south ends of the reservoir. The two alternative bridge alignments were selected by attempting to optimize the combined cost for a bridge and new connecting roadways. The alignments of the approaches to the bridges were selected along the ridges rather than along the shoreline of the reservoir due to the greater environmental sensitivity of the lower elevation areas. The southern bridge alternative would utilize more of the existing county road between Lodoga and Sites and would have the shortest travel time, but would result in a longer bridge crossing (8,500 feet). The north bridge alternative would have a much shorter bridge crossing (4,800 feet), but it would involve more new roadway construction than the southern alternative.

Existing topography was the major controlling factor in choosing the new road alignments. The total lengths of the road and bridge alignment alternatives between Sites and Lodoga, including their corresponding approaches, are summarized below:

- North Road alternative: 39 miles
- South Road alternative: 34 miles
- South Bridge alternative: 25 miles
- North Bridge alternative: 36 miles

Conceptual Bridge Designs

Figures B.3-50 and B3-51 show the preliminary feasibility bridge designs prepared that define bridge type and span configuration**.**

Bridge Type

A segmentally constructed concrete box girder bridge using the cantilevered construction method was chosen as the appropriate bridge type for preliminary

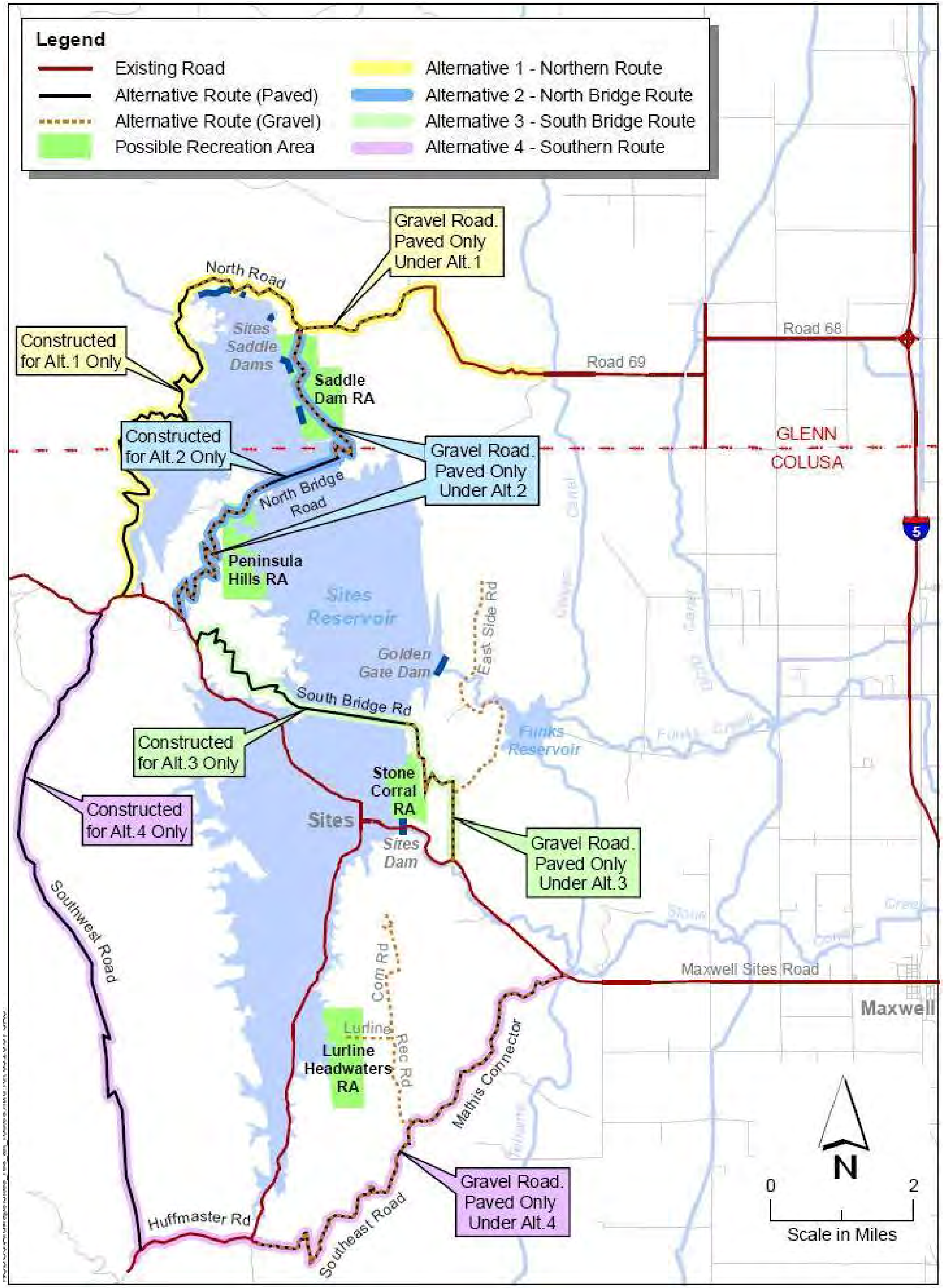


Figure B.3-49. Sites Reservoir Road Relocation Route Alternatives

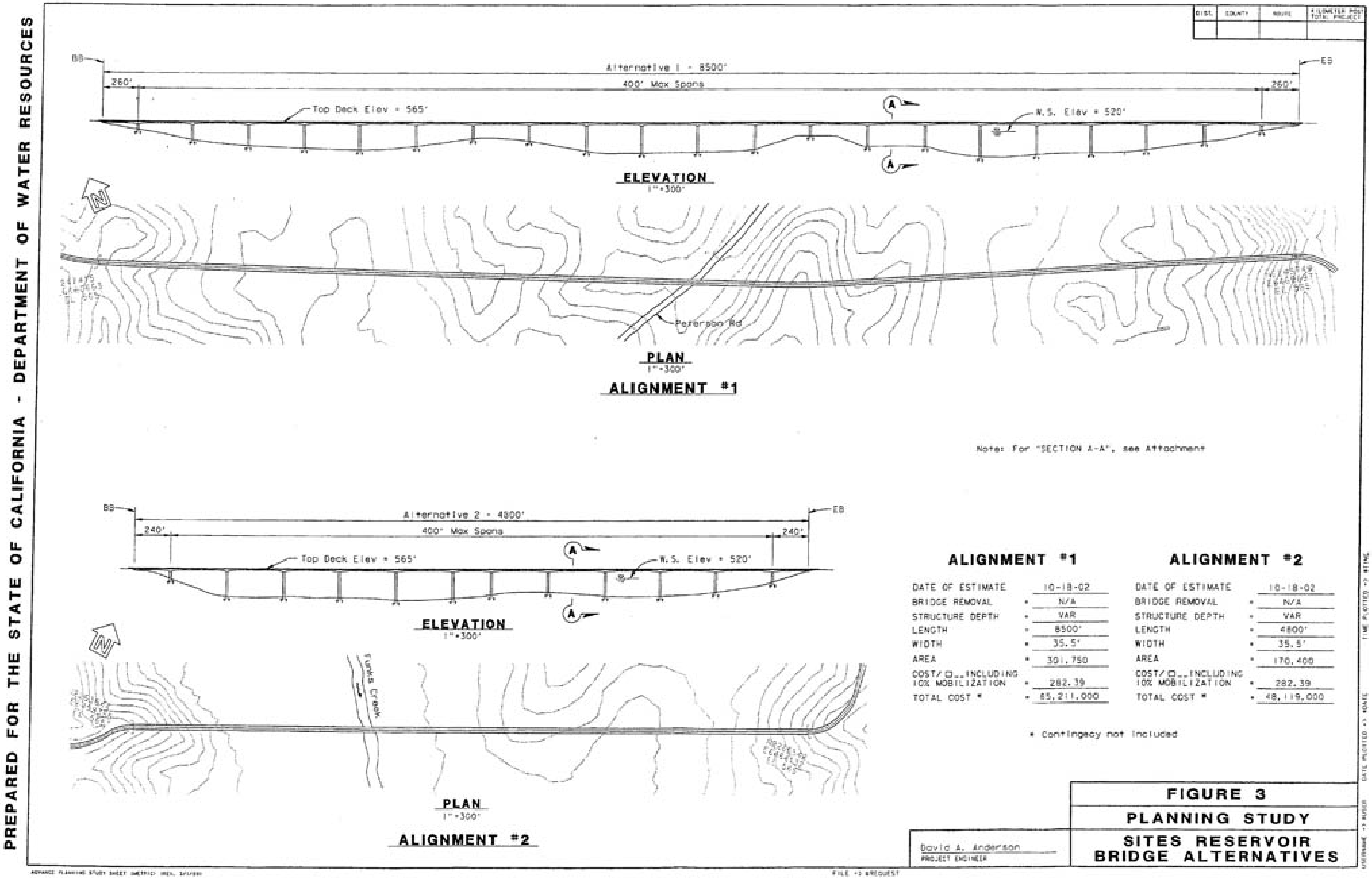


Figure B.3-50. Sites Reservoir Road Relocation – Conceptual Bridge Designs

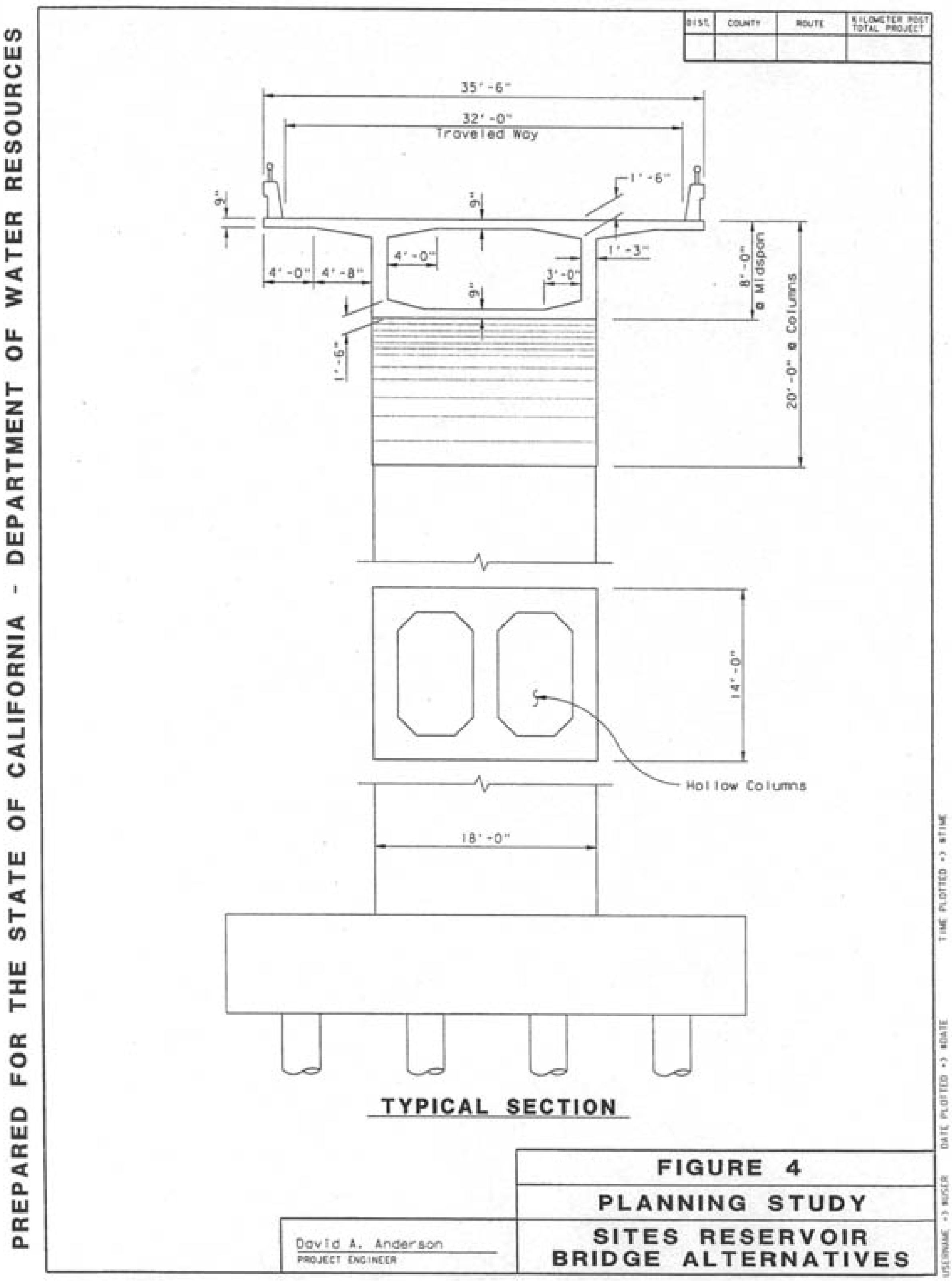


Figure B.3-51. Sites Reservoir Road Relocation – Conceptual Bridge Designs

feasibility design. The maximum span lengths are 400 feet utilizing a haunched depth section ranging from approximately 20 feet deep at the columns and 8 feet deep at mid-span. The weight of haunched sections would probably require the cast-in-place method to be utilized. The columns were assumed to be cast-in-place concrete with a hollow cellular shape supported on 36-inch-diameter drilled shaft piers.

Bridge Width

A 32-foot clear bridge width was chosen as it pertains to Rural Collector Roads. The width for the contiguous rural collector road assumes two 12-foot lanes and 4-foot shoulders, based on a relatively high hourly traffic volume that might occur during summer months with recreational traffic and given the potential for use by bicycles and motorist emergency pull-outs. A concrete barrier with bicycle height railing was considered.

Evaluation of Road Travel Times

Road travel times were estimated using the alternative bridge alignments between Maxwell and Lodoga and included both existing and new road segments. Both road grade profile and horizontal curvature were considered to estimate average driving speeds. The maximum assumed speed used for this evaluation was 55 mph for straight, flat (grade less than 2 percent) roadways. A speed reduction of 10 percent was taken for road grades between 2 and 4 percent and a reduction of 20 percent was taken for grades greater than 4 percent. The speed reductions for horizontal curvature were 30 percent for moderate road curvature and 60 percent for high curvature. Based on these criteria, the total road distances (between Maxwell and Lodoga), average driving speeds, and travel times for the four road alternatives are summarized in Table B.3-13.

Table B.3-13. Road Travel Time Summary

Road	Distance (miles)	Driving Speed (mph)	Driving Time (minutes)	Cost (million $)
Existing Road	24	50	29	N/A
North Road Alternative	39	45	58	105
South Road Alternative	34	45	49	129
North Bridge Alternative	36	50	47	149
South Bridge Alternative	25	50	33	195

mph = miles per hour
N/A = not applicable

Comparison of Alternative Alignments

Table B.3-13 shows that the South Bridge Route would have the shortest travel time between Maxwell and Lodoga, estimated at approximately 33 minutes. However, this route would have the highest cost of the four alternatives. To identify the preferred route, all variables would need to be evaluated, including not only construction costs, O&M costs, and travel times, but also environmental issues and identifying who would be the most frequent road users. Users would include weekend recreational traffic and daily traffic (e.g., travel to and from school).

It is noted that the North Road Route is partly along the shoreline of Sites Reservoir and has nearly10 miles of roadway with high curvature. To reduce the length of high-curvature roadway, the roadway could be straightened somewhat by constructing causeways with culverts across small inlets/bays.

Construction Sequencing

Work at the Sites Dam site would cut off the road between Maxwell and Lodoga. It would be preferable to have the new road alignment completed by the time work starts at the Sites Dam site. The existing Maxwell-Lodoga Road, however, could be used until the embankment dam fill reaches the existing roadway level, at which time traffic would be diverted to the new road alignment. This approach would require safety and traffic control implementations in the construction zone.

B.3.9.3 Access Roads

General

At a later stage of project development, additional roads would be included in the road alignment alternatives to provide access to potential recreation areas and project facilities. Access roads would be needed at four locations around the proposed reservoir.

Two roads would be included on the southeast side of the reservoir to allow access to a potential recreation area as well as to an existing communication center. These roads are shown as Lurline Rec. Road and Com Road on Figure B.3-49.

- A road would be included on the east side of the reservoir to allow access to Golden Gate Dam, the pumping plant at Funks Reservoir, and the potential Stone Corral Recreation Area. The road would also provide access to a landowner. This road is shown as East Side Road on Figure B.3-49.

- Two roads would be included on the northeast side of the reservoir to allow access to the nine saddle dams and a potential recreation area. One road is shown as an extension of North Road, providing access to Saddle Dams 6 through 9. The other road is unnamed and provides access to Saddle Dams 1 through 5 and the potential Saddle Dam Recreation Area. Both roads are shown on Figure B.3-49.

- A road would be included on the west side of the reservoir to allow access to the potential Peninsula Hills Recreation Area. Additional access roads may be added as plans for recreation areas are further developed.

Design Assumptions

The access roads would be unpaved and classified as rural local roads. Roadway element standards would be as described previously. Because access roads would be included no matter which alternative is chosen, access road costs were incorporated into road relocation cost estimates.

B.3.9.4 Electric Utilities and Utility Relocations

Project Electrical Utility Requirements

Transmission Lines

Sites Reservoir Pumping-Generating Plant

The 230kV system starts in the switchyard where a 230kV transmission line supply power comes in for the pumping-generating plant. The pumping-generating plant consists of seven (including one future unit), 3-phase, 13.2kV motor-generator and five motors (including one future unit). All 12 units are divided into four groups as follow:

- Three motor-generator units
- One motor-generator and two motors
- Three motors
- Three motor-generator units.

The 230kV transmission line in the switchyard is connected to four line breakers that are connected in parallel. Line breakers are connected to following transformers

- Line breaker # 1 is connected to 85MVA, 230kV-13.2kV
- Line breaker # 2 is connected to 75MVA, 230kV-13.2kV
- Line breaker # 3 is connected to 95MVA, 230kV-13.2kV
- Line breaker # 4 is connected to 75MVA, 230kV-13.2kV.

Two normally opened, motor-operated, 3-pole disconnect switches are used to tie 85MVA and 75MVA transformer together and 95 MVA and 75MVA together. The tie switches are normally open during normal operation and are not used until one of the two line breakers is disconnected for maintenance. When this occurs, the tie switch will be closed to provide power for two groups of motors out of four groups through two step-down transformers from either of the operating line breakers.

TRR Pumping-Generating Plant

The 230kV system starts in the switchyard where a 230kV transmission line supply power comes in for the plant. The pumping-generating plant consists of five (including one future unit), 3-phase, 13.2 kV pumping motors and two generators. The five pumping motors are split into two groups. One group consists of two motors and other group consists of three motors.

The 230kV transmission line in the switchyard is connected to two line breakers that are connected in parallel. One line breaker is connected to a 14MVA, 230kV – 13.2kV transformer to provide operating power to first group of motors. The second line breaker is connected to a 24MVA transformer to provide operating power to other group of motors.

A normally opened, motor-operated, 3-pole disconnect switch is used to tie two 230kV – 13.2kV transformers together. The tie switch is normally open during normal operation and is not used until one of the two line breakers, is disconnected for maintenance. When this occurs, the tie switch will be closed to provide power for both groups of motors through two step down transformers from either of the operating line breakers.

Each side of the 230kV line breaker bushings is connected to a manually operated disconnect switch. These disconnect switches will be opened when the line is not loaded and the breaker is under maintenance.

Sacramento River Pumping-Generating Plant

The 230kV system starts in the switchyard where a 230kV transmission line supply power comes in for the pumping-generating plant. The pumping-generating plant consists of five (including one future unit), 3-phase, 13.2 kV pumping motors and two generators. The five pumping motors are split into two groups. One group consists of two pumps and other group consists of three pumps.

The 230kV transmission line in the switchyard is connected to two line breakers that are connected in parallel. One line breaker is connected to a 46MVA, 230kV – 13.2kV transformer to provide operating power to first group of motors. The second line breaker is connected to a 69MVA transformer to provide operating power to other group of motors.

A normally opened, motor-operated, 3-pole disconnect switch is used to tie two 230kV – 13.2kV transformers together. The tie switch is normally open during normal operation and is not used until one of the two line breakers, is disconnected for maintenance. When this occurs, the tie switch will be closed to provide power for both groups of motors through two step down transformers from either of the operating line breakers.

Utility Relocations

The project would require the relocation of utilities including gas pipelines, power lines, telephone lines, and fiber optic cable. The service lines to a microwave station adjacent to the Sites Reservoir site would also require relocation.

B.3.9.5 Right-of-Way

Sites Reservoir

The Sites Reservoir inundation area would encompass most of the Antelope Valley including the small community of Sites. The land is mainly non-irrigated pasture with improvements being mostly farm structures and a few residences. ROW would need to be acquired for the entire area inundated by the reservoir plus a 100-foot buffer zone from the high-water mark of the reservoir at elevation 520 feet. The buffer zone would allow for easier access for future maintenance.

Funks Reservoir Enlargement

Funks Reservoir is owned by Reclamation. However, it is assumed that there would be an additional need to acquire further ROW for the new Holthouse Reservoir to be located downstream of exiting Funks Reservoir.

Conveyance

New Sacramento River Conveyance

ROW would need to be acquired for the SRPGP and the Delevan Pipeline that would convey water from the Sacramento River to Funks Reservoir. The maximum capacity considered is 2,000 cfs. The Delevan Pipeline would cross through an area primarily consisting of rice fields. Permanent acquisitions would need to be made for the conveyance facilities and temporary acquisitions would need to be made for construction purposes.

Tehama-Colusa Canal Conveyance

No ROW acquisition is required for the TC Canal modifications. Reclamation currently owns the lands occupied by and adjacent to RBDD, the TC Canal, and the Corning Canals. Additional land would be needed for staging and for disposal/storage of sediment from future dredging operations.

Glenn-Colusa Irrigation District Canal

No additional ROW would be required to for modifications to the GCID Canal. GCID has a combination of permanent easements (ROW) and fee ownership for lands occupied by and adjacent to the HCPS, GCID Canal, and related structures and facilities. Additional easements or land acquisition may be required for the proposed modifications. Additional temporary easements and ROW agreements would be needed for the TRR earthwork borrow and disposal sites, and contractor staging areas. Acreage for these items was not considered. Approximately 200 acres of new ROW would specifically be needed for the TRR-Funks Pipeline.

B.3.10 Recreation

The proposed Sites Reservoir has been identified by DWR and CALFED as an important proposed facility under consideration in California, and its recreational component has opportunities to serve the growing Sacramento and Red Bluff regions for generations to come. A study of recreational opportunities for the Sites Reservoir has identified the Stone Corral Recreational Area as the preferred option because of its suitability and flexibility to accommodate the two reservoir sizes being considered.

Appendix E contains a detailed evaluation of potential recreational areas considered for the project.

B.4 PRELIMINARY FEASIBILITY COST ESTIMATE

B4.1 Introduction

This section presents preliminary feasibility-level cost estimates for NODOS project Alternatives A, B, and C described in Section B.3. Table B.4-1 provides a list of the major project components for each alternative that are included in the cost estimates.

The feasibility-level construction cost estimates presented in this section include contract costs, contingencies, and non-contract costs for all facilities shown on Table B.4-1, and other allowances for items such as environmental mitigation, ecosystem enhancements, and temporary and permanent easement acquisition. Key estimating assumptions are also discussed in this section.

The estimates of project cost provided in this section are based upon a compilation of existing estimates prepared by DWR or its consultants. These estimates reflect the current conceptual level of project design. The only changes made to existing estimates include adding new project features (such as the Holthouse Reservoir) or modify the original estimates to reflect design updates (such as changing pumping units at the Sites Pumping Plant to pumping/generating units). Estimates also have been updated from their base year (preparation date) to July 2010 price levels.

Complete cost estimates are provided for three project alternatives. It is assumed that this cost information will be combined with other analyses and evaluations to identify a preferred project and that additional design and cost estimating will be performed to confirm the feasibility of the project. To support that work, additional field investigations and construction material evaluations should be performed. The actual project construction cost ultimately will depend upon the final design details of the preferred project alternative, and the labor and material costs, market conditions, and other variable factors existing at the time of bid. Accordingly, the final project cost will vary from the preliminary feasibility estimates presented in this section. The cost estimates presented in this section are suitable for estimating benefit-cost ratios and comparing project alternatives. However, these estimates should not be used for budgeting or financial planning of any sort for the project.

B.4.2 Level and Classification of Cost Estimates

The cost estimates presented in this section are considered to be Class 4 estimates as defined by the Association for Advancement of Cost Engineering. Class 4 estimates are generally prepared based on limited information and can have wide accuracy ranges. Typically, engineering is anywhere from 1 to 5 percent complete. Class 4 estimates are generally used for determining project pre-feasibility, market studies, project screening, and preliminary budget approval. Expected accuracy ranges for Class 4 efforts are from 15 to 30 percent on the low side, to 20 to 50 percent on the high side, depending on the complexity of the project, quality of reference information, and the inclusion of appropriate contingency allocations.

Table B.4-1. Alternative Project Components

Facility	Project Alternative		
	A	B	C
Sites Reservoir Clearing and Demolition	•	•	•
Sites Dam	•	•	•
Golden Gate Dam	•	•	•
Saddle Dams			
Saddle Dam 1	•	•	•
Saddle Dam 2		•	•
Saddle Dam 3	•	•	•
Saddle Dam 4		•	•
Saddle Dam 5	•	•	•
Saddle Dam 6	•	•	•
Saddle Dam 7		•	•
Saddle Dam 8	•	•	•
Saddle Dam 9		•	•
Signal Spillway at Saddle Dam 6	•	•	•
Sites Reservoir Inlet/Outlet			
Tunnel	•	•	•
Inlet/Outlet Structure	•	•	•
Access Road	•	•	•
Sites Pumping-Generating Plant			
Pumping-Generating Plant	•	•	•
Emergency Drawdown Bypass	•	•	•
Plant Access Road	•	•	•
Temporary Bypass TC Canal	•	•	•
Holthouse (Enlarged Funks) Reservoir	•	•	•
TRR Reservoir	•	•	•
TRR Pumping-Generating Plant	•	•	•
TRR Pipeline	•	•	•
Delevan Pipeline	•	•	•
Sacramento River Release-Only Structure		•	
Sacramento River Pumping-Generating Plant			
Pumping-Generating Plant	•		•
Fish Screen Facility	•		•
Project Access Roads and Bridges			
Access Roads (Public and Private)	•	•	•
South Bridge	•	•	•
Electrical Transmission (230-kV Option)	•	•	•
GCID Canal Modifications	•	•	•
Recreation	•	•	•

GCID = Glenn Colusa Irrigation District
kV = kilovolts
TC = Tehama Colusa
TRR = Terminal Regulating Reservoir

B.4.3 Cost Estimate Considerations

B.4.3.1 Estimating Terminology

Contract Costs: Contract costs include detailed quantity and unit price estimates, plus allowances for mobilization/demobilization and unlisted items.

Contingency: Contingency is estimated as a percentage of the contract cost.

Field Cost: The field cost is the sum of the contract cost and contingency.

Non-Contract Costs: Non-contract costs include engineering, administration, legal services, and permitting costs.

Construction Cost: The construction cost is the sum of the field cost and non-contract costs.

B.4.3.2 Construction Cost Components

Contract Cost

The contract cost estimates presented in this section were compiled using individual estimate worksheets for project components prepared by DWR. These estimates were initially prepared in 2002 and some have been updated to 2005 and 2007 price levels. Where DWR estimate worksheets were not available for project features (such as roads and bridges, and Sacramento fish screen facilities), estimates found in feasibility study documents prepared by DWR or its consultants were used and the base year for the estimate was assumed to be the year that the study was prepared. For new features recently added (such as the Holthouse Reservoir), quantities were estimated based upon available concept drawings and cost estimates were developed. A constructability review of the preliminary feasibility design for each project alternative was not performed.

In accordance with Reclamation estimating guidelines, an additional allowance of 10 percent of the contract cost for the facility is included in the facility cost to cover unlisted items.

Escalation

The contract costs for all previous estimates were escalated from the base year (the year and month the estimate was prepared) to July 2010 price levels. As described above, some of the previous DWR estimates were escalated to 2005 or 2007 price levels. To do this, DWR applied used California Department of Transportation (Caltrans) Price Index data. However, for the estimates presented in this section, it is assumed that the current Reclamation Construction Cost Trend (CCT) index factors would be more appropriate as they better represent the actual facilities to be constructed for the project. The CCT factors were applied to the original estimates before any Caltrans Price Index was applied so that all estimates were escalated to July 2010 price levels from the original base using the same escalation method.

Contingency

Contingency is a percentage allowance added to the escalated contract cost. Contingencies are funds for use after construction starts to compensate the contractor for unforeseen or changed site conditions, owner-directed orders for change, quantity overruns, etc. Contingency allowances are generally higher for appraisal-level estimates than for feasibility-level estimates. For the estimates presented in this section, an allowance of 20 percent of the July 2010 contract cost has been used for contingency.

B.4.3.3 Non-Contract Costs

Non-contract costs include engineering and design, construction management, project close-out, administration, legal services, permitting, etc. For the estimates presented in this section, the non-contract costs were estimated to be 25 percent of the total field costs (contract cost plus contingency). Actual non-contract costs will vary from facility to facility; however, 25 percent is assumed to represent the average value.

B.4.3.4 Other Cost Allowances

Environmental Mitigation

Many environmental laws affect the state's major water supply programs and environmental concerns play a major role in water policy and planning. Costs related to environmental permitting and mitigation have not been fully developed as yet. They will be determined in later stages of the planning process for the selected project alternative as design and construction details are further developed. To prepare the cost estimates presented in this section, an allowance of 10 percent of the contract cost was assumed to allow for any mitigation for environmental and cultural resource impacts in accordance with National Environmental Pollution Act (NEPA) and California Environmental Quality Act (CEQA) requirements.

Right-of-Way

ROW costs represent the estimated fair market value of the real estate required for the NODOS project and do not include staff costs for appraisals or acquisition, damages to the remaining land caused by the acquisition or construction of the project, or utility relocations that may be necessary. ROW cost estimates were based on a study conducted by DWR in 2004 to estimate the cost of land acquisition for Sites Reservoir, conveyance options, and road relocations. These estimates were updated by DWR to April 2007 prices using an escalation factor. They were further updated to July 2010 price levels using the Reclamation CCT index.

Ecosystem Enhancement

Ecosystem enhancement (gravel restoration, etc.) is a part of the project. The actual cost for these enhancements is not fully defined at this time. To allow for ecosystem enhancement activities, DWR has made an allowance of 30 million dollars for each project alternative for the estimates presented in this section.

B.4.3.5 Assumptions

Major assumptions made to prepare the preliminary feasibility cost estimates include:

- Competitive market conditions would prevail at the time of bid tender.
- The construction schedule is the current schedule showing construction completion in 2023.
- Environmental mitigation and ecosystem enhancement measures would continue to similar to those currently used in practice.
- Builder’s Risk Insurance would be available to the contractor.
- Materials such as sand, gravel, cement, etc., would remain available within the haul distances used to prepare the estimates.

B.4.3.6 Exclusions

Major exclusions from the cost estimates include:

- Escalation to midpoint of construction
- Off-site power supply
- Existing infrastructure relocations (except roads)
- Temporary construction ROW and easements
- Temporary haul road and overpass (Interstate 5, railroads and canals)

B.4.4 Cost Estimate Summaries

Table B.4-2 presents the estimated cost for each of the three project alternatives.

B.4.5 Future Estimate Consideration

As described above, the estimates discussed in this section are considered to be Class 4 estimates. These are considered suitable for evaluating alternative projects at this time. However, after a preferred alternative is selected, a more detailed Class 3 estimate should be prepared to verify project feasibility. That estimate should be based upon enhanced design information and updated quantity calculations. Factoring existing estimates as was done for the estimates presented in this section should not be considered.

Table B.4-2. Estimate Summary

Estimate Summary - July 2010[1]			
Facility	Project Alternative		
	A	B	C
Sites Reservoir Clearing and Demolition	1,523,800	1,686,300	1,686,300
Sites Dam	75,802,900	93,248,600	93,248,600
Golden Gate Dam	142,459,700	217,924,400	217,924,400
Saddle Dams	-	-	-
Saddle Dam 1	564,500	1,301,600	1,301,600
Saddle Dam 2	Not Required	1,735,500	1,735,500
Saddle Dam 3	29,077,400	64,719,500	64,719,500
Saddle Dam 4	Not Required	291,000	291,000
Saddle Dam 5	10,192,900	31,064,400	31,064,400
Saddle Dam 6	372,000	3,568,400	3,568,400
Saddle Dam 7	Not Required	6,581,200	6,581,200
Saddle Dam 8	Not Required	38,448,600	38,448,600
Saddle Dam 8a[2]	7,809,900	Not Required	Not Required
Saddle Dam 8b	231,400	Not Required	Not Required
Saddle Dam 9	Not Required	759,200	759,200
Signal spillway at Saddle Dam 6	1,188,000	3,564,000	3,564,000
Sites Reservoir Tunnel and Inlet/Outlet Structures	-	-	-
Tunnel	96,653,700	96,653,700	96,653,700
Inlet/Outlet Structure	25,141,300	28,132,100	28,132,100
Access Road	1,745,700	1,745,700	1,745,700
Sites Pumping/Generating Plant	-	-	-
Pump/Generating Plant	428,164,100	384,723,500	428,164,100
Emergency Drawdown Bypass	35,223,200	35,223,200	35,223,200
Plant Access Road	1,538,500	1,538,500	1,538,500
Temporary Bypass TC Canal	15,460,700	15,460,700	15,460,700
Holthouse (Enlarged Funks)	120,114,100	120,114,100	120,114,100
TRR Reservoir	40,019,900	40,019,900	40,019,900
TRR Pump/Generating Plant	213,456,400	213,456,400	213,456,400
TRR Pipeline	62,809,100	62,809,100	62,809,100
Delevan Pipeline	342,190,000	342,190,000	342,190,000
Sacramento River Release Only Facility	Not Required	13,516,300	Not Required
Sacramento River Pumping/Generating Plant	-	-	-
Pump Generating Plant	237,068,600	Not Required	237,068,600
Fish Screen Facility	48,914,800	Not Required	48,914,800
Project Access Roads and Bridges	-	-	-
Access Roads (Public and Private)	120,276,900	120,276,900	120,276,900
South Bridge	125,601,000	125,601,000	125,601,000
Electrical Transmission (230kV Option)	23,009,400	14,935,900	23,009,400
GCID Canal Modifications	18,578,500	18,578,500	18,578,500
Recreation	6,978,300	6,978,300	6,978,300
Contract Cost[3]	2,232,166,700	2,106,846,500	2,430,827,700
Contingency[4]	446,433,300	421,369,300	486,165,500
Field Cost	2,678,600,000	2,528,215,800	2,916,993,200
Non-Contract Costs[5]	558,041,700	526,711,600	607,706,900
Construction Cost	3,236,641,700	3,054,927,400	3,524,700,100
Environmental Mitigation[6]	223,216,700	210,684,700	243,082,800
Ecosystem Enhancement Fund	30,000,000	30,000,000	30,000,000
Land Acquisition	89,608,700	88,237,100	89,608,700
Estimated Total Project Alternative Cost	**3,579,467,100**	**3,383,849,200**	**3,887,391,600**

Note: 1. Escalated cost determined using USBR Construction Cost Trend Factors
2. Saddle Dams 8a and 8b for Alternative A replace Saddle Dam 8 for Alternative B and C at same locati
3. Contract cost includes 10% allowance for unlisted Items
4. Contingency allowance is 20% of contract cost
5. Non-contract cost allowance is 25% of contract cost
6. Environmental mitigation allowance is 10% of contract cost

B.5 DESIGN AND CONSTRUCTION SCHEDULE

B.5.1 Introduction

Figure B.5-1 presents a preliminary, feasibility-level design and construction schedule for the NODOS project. The schedule is based upon constructing Alternative C, which involves the largest facility sizes for dams, reservoirs, and pumping-generating plants. The schedule for Alternative A and Alternative B likely would have a similar duration because these alternatives still include many of the same critical path features in common with Alternative A that govern the schedule duration.

The schedule shown on Figure B.5-1 was originally developed by DWR and has been updated to reflect features added to or removed from the project since the schedule was originally prepared. This update includes the addition of the Holthouse Reservoir and associated facilities, and the various modifications made to the pumping-generating plants.

Future schedule updates should include further evaluations to confirm the lead time required to procure and deliver major mechanical and electrical equipment items, and the lead time required to fabricate large diameter concrete pipe on site for the Delevan Pipeline.

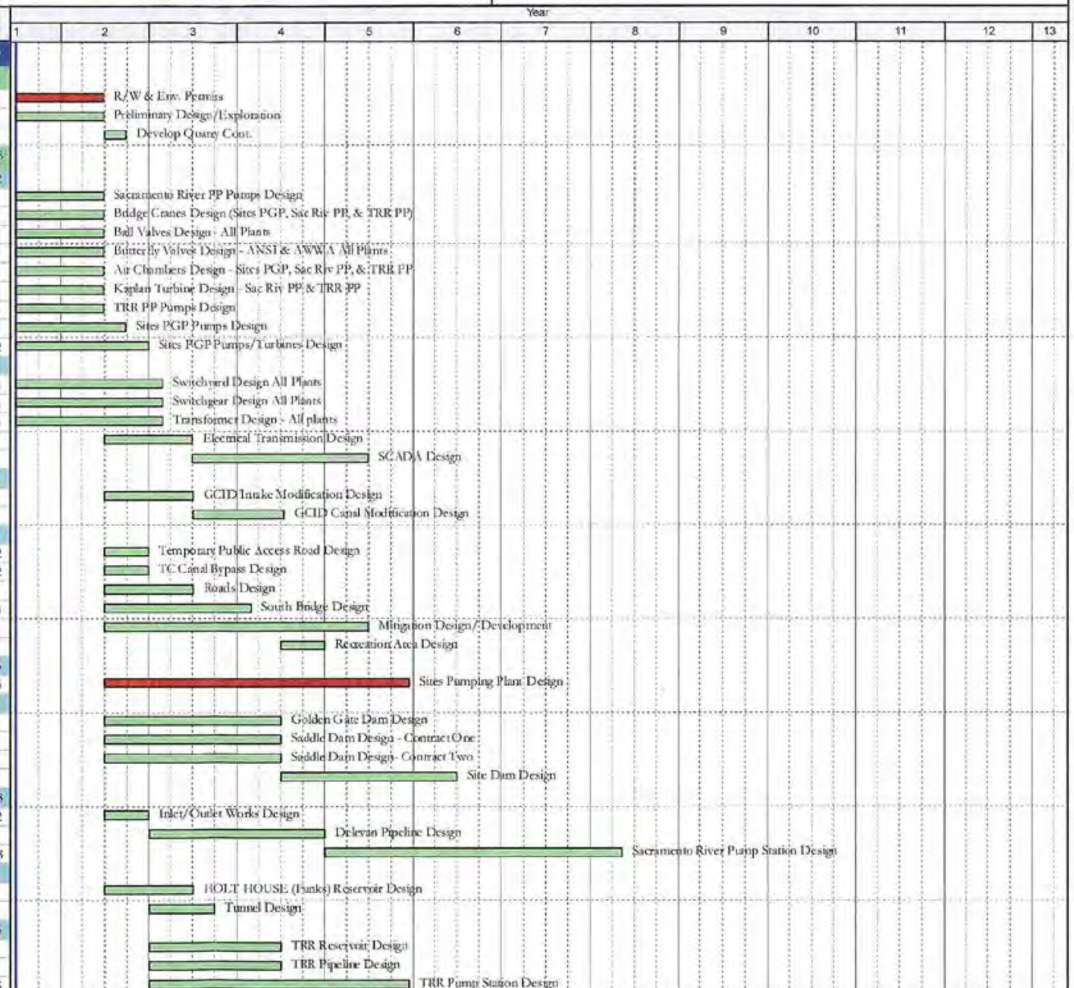
CONCEPT SCHEDULE

NODOS - Sites Reservoir - 7-12-10 Update

Job No. NODOS

13-Jul-11 11:51

Activity ID	Activity Name	Duration	Start	Finish
NODOS - Sites Reservoir - 7-12-10 Update		2937	01-Jul-11	16-Feb-23
Planning		314	01-Jul-11	28-Sep-12
A1010	R/W & Env. Permits	251	01-Jul-11	29-Jun-12
A1020	Preliminary Design/Exploration	251	01-Jul-11	29-Jun-12
A1030	Develop Quarry Cont.	63	02-Jul-12	28-Sep-12
Design		1754	01-Jul-11	14-May-18
Pumps, Turbine & Mechanical Equipment Design		376	01-Jul-11	31-Dec-12
A2310	Sacramento River PP Pumps Design	250	01-Jul-11	28-Jun-12
A2240	Bridge Cranes Design (Sites PGP, Sac Riv PP, & TRR PP)	251	01-Jul-11	29-Jun-12
A2250	Ball Valves Design - All Plants	251	01-Jul-11	29-Jun-12
A2260	Butterfly Valves Design - ANSI & AWWA All Plants	251	01-Jul-11	29-Jun-12
A2270	Air Chambers Design - Sites PGP, Sac Riv PP, & TRR PP	251	01-Jul-11	29-Jun-12
A2300	Kaplan Turbine Design - Sac Riv PP & TRR PP	251	01-Jul-11	29-Jun-12
A2320	TRR PP Pumps Design	251	01-Jul-11	29-Jun-12
A2290	Sites PGP Pumps Design	314	01-Jul-11	28-Sep-12
A2280	Sites PGP Pumps/Turbines Design	376	01-Jul-11	31-Dec-12
Electrical Equipment Design		1006	01-Jul-11	01-Jul-15
A2340	Switchyard Design All Plants	416	01-Jul-11	28-Feb-13
A2350	Switchgear Design All Plants	416	01-Jul-11	28-Feb-13
A2360	Transformer Design - All plants	416	01-Jul-11	28-Feb-13
A2200	Electrical Transmission Design	249	02-Jul-12	28-Jun-13
A2230	SCADA Design	506	01-Jul-13	01-Jul-15
GCID Intake & Canal Modification		508	02-Jul-12	14-Jul-14
A2090	GCID Intake Modification Design	251	02-Jul-12	02-Jul-13
A2100	GCID Canal Modification Design	257	03-Jul-13	14-Jul-14
General		755	02-Jul-12	01-Jul-15
A2010	Temporary Public Access Road Design	125	02-Jul-12	31-Dec-12
A2330	TC Canal Bypass Design	125	02-Jul-12	31-Dec-12
A2180	Roads Design	251	02-Jul-12	02-Jul-13
A2190	South Bridge Design	416	02-Jul-12	03-Mar-14
A2220	Mitigation Design / Development	755	02-Jul-12	01-Jul-15
A2210	Recreation Area Design	126	02-Jul-14	01-Jan-15
Sites Pump Station		877	02-Jul-12	18-Dec-15
A2170	Sites Pumping Plant Design	877	02-Jul-12	18-Dec-15
Golden Gate, Sites, Saddle Dams		1017	02-Jul-12	01-Jul-16
A2020	Golden Gate Dam Design	500	02-Jul-12	01-Jul-14
A2040	Saddle Dam Design - Contract One	500	02-Jul-12	01-Jul-14
A2050	Saddle Dam Design - Contract Two	500	02-Jul-12	01-Jul-14
A2030	Site Dam Design	517	02-Jul-14	01-Jul-16
Sacramento River Pump Station		1503	02-Jul-12	14-May-18
A2060	Inlet/Outlet Works Design	125	02-Jul-12	31-Dec-12
A2110	Delevan Pipeline Design	501	02-Jan-13	01-Jan-15
A2120	Sacramento River Pump Station Design	877	02-Jan-15	14-May-18
Funks		313	02-Jul-12	30-Sep-13
A2160	HOLT HOUSE (Funks) Reservoir Design	251	02-Jul-12	02-Jul-13
A2070	Tunnel Design	188	02-Jan-13	30-Sep-13
TRR Pump Station, Reservoir & Pipeline Design		751	02-Jan-13	17-Dec-15
A2130	TRR Reservoir Design	375	02-Jan-13	01-Jul-14
A2150	TRR Pipeline Design	375	02-Jan-13	01-Jul-14
A2140	TRR Pump Station Design	751	02-Jan-13	17-Dec-15



Remaining Level of Effort | Actual Work | Near Critical
Actual Level of Effort | Remaining Work | Critical Remaining Work

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TASK filters: In Progress, Not Started.

Figure B.5-1. Project Design and Construction Schedule (Sheet 1 of 3)

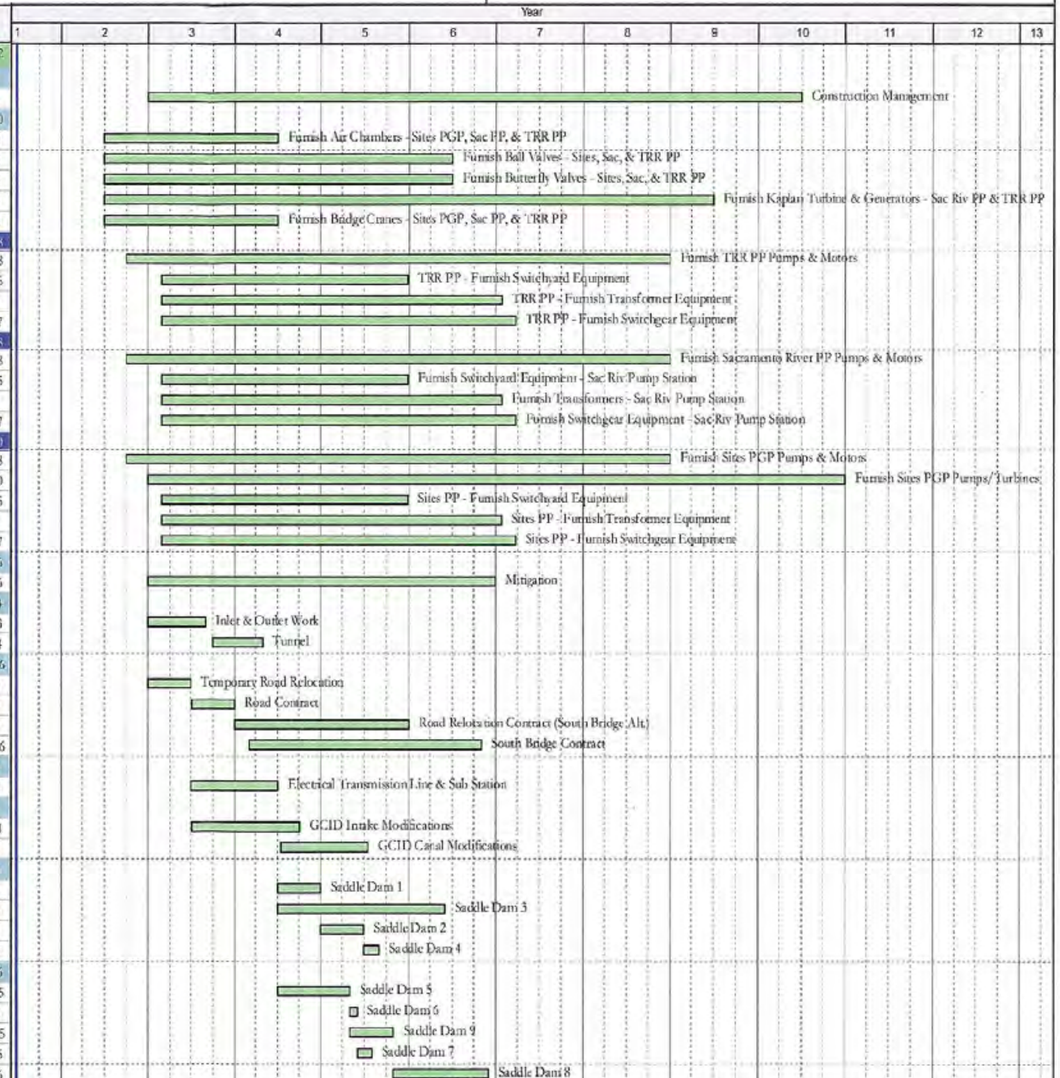
CONCEPT SCHEDULE

NODOS - Sites Reservoir - 7-12-10 Update

Job No. NODOS

13-Jul-11 11:51

Activity ID	Activity Name	Duration	Start	Finish
Construction (ALL WORK BELOW IN CALENDAR DAYS)		2641	30-Jun-12	18-Dec-22
Construction Management		2741	01-Jan-13	03-Jul-20
A3520	Construction Management	2741	01-Jan-13	03-Jul-20
Material & Equipment Procurement		3107	30-Jun-12	31-Dec-20
A3040	Furnish Air Chambers - Sites PGP, Sac PP, & TRR PP	731	30-Jun-12	30-Jun-14
A3020	Furnish Ball Valves - Sites, Sac, & TRR PP	1462	30-Jun-12	30-Jun-16
A3030	Furnish Butterfly Valves - Sites, Sac, & TRR PP	1462	30-Jun-12	30-Jun-16
A3070	Furnish Kaplan Turbine & Generators - Sac Riv PP & TRR	2556	30-Jun-12	29-Jun-19
A3010	Furnish Bridge Cranes - Sites PGP, Sac PP, & TRR PP	729	02-Jul-12*	30-Jun-14
TRR		2283	01-Oct-12	31-Dec-18
A3090	Furnish TRR PP Pumps & Motors	2283	01-Oct-12*	31-Dec-18
A3350	TRR PP - Furnish Switchyard Equipment	1035	01-Mar-13	30-Dec-15
A3365	TRR PP - Furnish Transformer Equipment	1425	01-Mar-13	23-Jan-17
A3360	TRR PP - Furnish Switchgear Equipment	1485	01-Mar-13	24-Mar-17
Sacramento River Pump Station		2283	01-Oct-12	31-Dec-18
A3080	Furnish Sacramento River PP Pumps & Motors	2283	01-Oct-12*	31-Dec-18
A3306	Furnish Switchyard Equipment - Sac Riv Pump Station	1035	01-Mar-13	30-Dec-15
A3309	Furnish Transformers - Sac Riv Pump Station	1425	01-Mar-13	23-Jan-17
A3308	Furnish Switchgear Equipment - Sac Riv Pump Station	1485	01-Mar-13	24-Mar-17
Sites		3014	01-Oct-12	31-Dec-20
A3060	Furnish Sites PGP Pumps & Motors	2283	01-Oct-12*	31-Dec-18
A3050	Furnish Sites PGP Pumps/Turbines	2922	01-Jan-13	31-Dec-20
A3420	Sites PP - Furnish Switchyard Equipment	1035	01-Mar-13	30-Dec-15
A3430	Sites PP - Furnish Transformer Equipment	1425	01-Mar-13	23-Jan-17
A3425	Sites PP - Furnish Switchgear Equipment	1485	01-Mar-13	24-Mar-17
Mitigation		1461	01-Jan-13	31-Dec-16
A3490	Mitigation	1461	01-Jan-13	31-Dec-16
Inlet Modifications & Tunnel		485	01-Jan-13	30-Apr-14
A3240	Inlet & Outlet Work	243	01-Jan-13	31-Aug-13
A3250	Tunnel	212	01-Oct-13	30-Apr-14
Roads & Bridges		1403	01-Jan-13	03-Nov-16
A3100	Temporary Road Relocation	181	01-Jan-13	30-Jun-13
A3440	Road Contract	184	03-Jul-13	02-Jan-14
A3450	Road Relocation Contract (South Bridge Alt.)	730	03-Jan-14	02-Jan-16
A3460	South Bridge Contract	976	04-Mar-14	03-Nov-16
Electrical Transmission Line & Sub Station		367	29-Jun-13	30-Jun-14
A3470	Electrical Transmission Line & Sub Station	367	29-Jun-13	30-Jun-14
GCID		743	03-Jul-13	15-Jul-15
A3280	GCID Intake Modifications	457	03-Jul-13	02-Oct-14
A3290	GCID Canal Modifications	366	15-Jul-14	15-Jul-15
Saddle Dam Contract One		701	02-Jul-14	01-Jun-16
A3120	Saddle Dam 1	184	02-Jul-14	01-Jan-15
A3140	Saddle Dam 3	701	02-Jul-14	01-Jun-16
A3130	Saddle Dam 2	181	02-Jan-15	01-Jul-15
A3150	Saddle Dam 4	62	02-Jul-15	01-Sep-15
Saddle Dam Contract Two		885	02-Jul-14	02-Dec-16
A3170	Saddle Dam 5	304	02-Jul-14	01-May-15
A3180	Saddle Dam 6	31	02-May-15	01-Jun-15
A3210	Saddle Dam 9	184	02-May-15	01-Nov-15
A3190	Saddle Dam 7	61	02-Jun-15	01-Aug-15
A3200	Saddle Dam 8	397	02-Nov-15	02-Dec-16



Remaining Level of Effort | Actual Work | Near Critical
Actual Level of Effort | Remaining Work | Critical Remaining Work

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TASK filters: In Progress, Not Started.

Figure B.5-1. Project Design and Construction Schedule (Sheet 2 of 3)

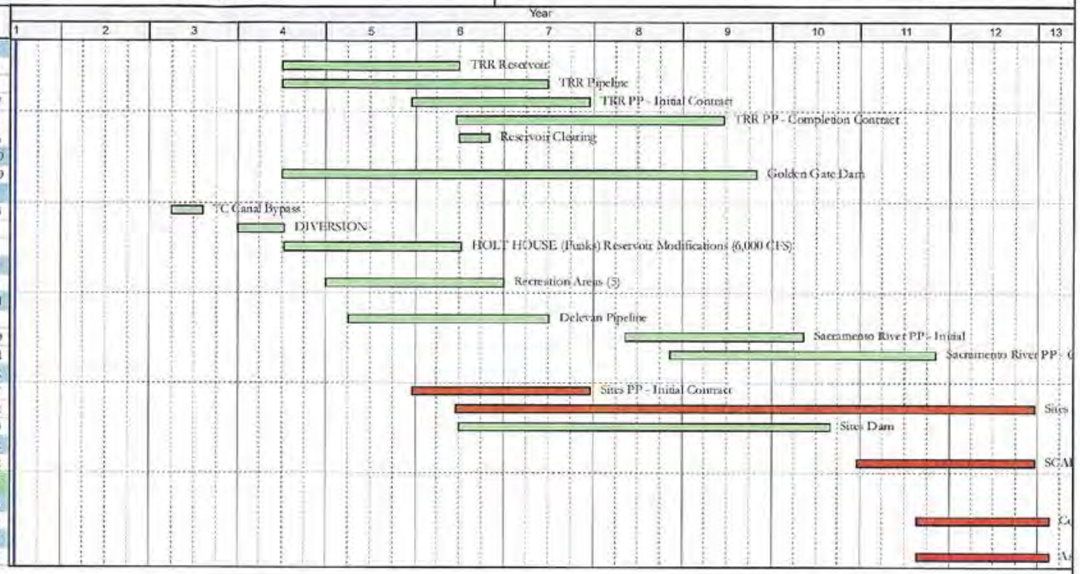
CONCEPT SCHEDULE

NODOS - Sites Reservoir - 7-12-10 Update

Job No. NODOS

13-Jul-11 11:51

Activity ID	Activity Name	Duration	Start	Finish
TRR Reservoir		1251	02-Jul-14	19-Jun-19
A3310	TRR Reservoir	365	02-Jul-14	30-Jun-16
A3380	TRR Pipeline	549	02-Jul-14	01-Jul-17
A3330	TRR PP - Initial Contract	731	18-Dec-15	17-Dec-17
A3340	TRR PP - Completion Contract	1094	21-Jun-16	19-Jun-19
A3510	Reservoir Clearing	122	01-Jul-16	30-Oct-16
Golden Gate Dam		1949	02-Jul-14	01-Nov-19
A3220	Golden Gate Dam	1949	02-Jul-14	01-Nov-19
Funks reservoir Modifications (6,000 CFS)		826	01-Apr-13	09-Jul-16
A3260	TC Canal Bypass	129	01-Apr-13	07-Aug-13
ADD1000	DIVERSION	130	03-Jan-14	10-Jul-14
A3390	HOLT HOUSE (Funks) Reservoir Modifications (6,000 CF	730	11-Jul-14	09-Jul-16
Recreation Areas (5)		731	02-Jan-15	01-Jan-17
A3480	Recreation Areas (5)	731	02-Jan-15	01-Jan-17
Sac River Pump Station (2,000 CFS)		1667	01-Apr-15	08-Nov-21
A3375	Delevan Pipeline	459	01-Apr-15	02-Jul-17
A3302	Sacramento River PP - Initial	728	15-May-18	11-May-20
A3304	Sacramento River PP - Completion	1094	11-Nov-18	08-Nov-21
Sites Pump Station (6,000 CFS)		1769	19-Dec-15	18-Dec-22
A3410	Sites PP - Initial Contract	731	19-Dec-15	18-Dec-17
A3415	Sites PP - Completion Contract	2373	20-Jun-16	18-Dec-22
A3230	Sites Dam	792	02-Jul-16	30-Aug-20
SCADA Connections		731	18-Dec-20	18-Dec-22
A3500	SCADA Connections	731	18-Dec-20	18-Dec-22
Close-Out		391	18-Aug-21	16-Feb-23
Contract Close-Out		391	18-Aug-21	16-Feb-23
A3530	Contract Close-Out	391	18-Aug-21	16-Feb-23
As-Built		390	20-Aug-21	16-Feb-23
A3540	As-Built	390	20-Aug-21	16-Feb-23



Remaining Level of Effort | Actual Work | Near Critical
Actual Level of Effort | Remaining Work | Critical Remaining Work

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TASK filters: In Progress, Not Started.

Figure B.5-1. Project Design and Construction Schedule (Sheet 3 of 3)

B.6 CONSTRUCTION EQUIPMENT UTILIZATION AND WORKFORCE

B.6.1 Introduction

To support environmental impact assessments, a preliminary estimate was prepared for construction equipment utilization and construction workforce. The following pages summarize equipment and staffing for each of the major project features.

Equipment numbers and utilization hours are based upon the earthwork and material quantities developed for the project features to support cost estimating. Conservative hourly equipment production rates were used to estimate the number of pieces of construction equipment required and the number of operating hours required to complete major tasks within schedule limits. Support equipment was also included in the evaluation. For example, this evaluation would include excavators and loaders to fill haul vehicles and water trucks, bulldozers, and compactors to spread and compact fills. Estimates were also included for the support equipment (cranes, hoists, and the like) needed for pipelines, buildings and other facilities.

Haul equipment was also included where appropriate for each facility for filter and drain material, concrete aggregates, aggregate base and the like. The equipment numbers are based upon the need to import such materials from an off-site quarry approximately 35 miles from the project.

A reasonable estimate was also included to haul major equipment components to the facilities, including pumps, motors, generators, mechanical equipment, embedded items, and the like. Haul equipment for construction material was also estimated (reinforcing steel, structural steel, ready-mix concrete, on-site fabricated pipe, and the like).

Typically, equipment numbers and hours were based upon 10-hour, single-shift work, six days per week. The only exception to this scheduling would be the tunneling work at Sites Dam; that effort was assumed to be double-shift work (22 hours per day, six days per week).

Staff numbers shown for each facility include equipment operators and other construction personnel needed to complete construction. Staff estimates do not include owner or engineer staff that would be supervising construction.

The equipment utilization and staff estimated represent construction of Alternative C, which would have the largest impact.

Tunnel - Inlet and Outlet Including Sites Pumping-Generating Plant

	Tunnel			Lake Outlet			Inlet area			Total
Equipment	Count	Days	Equip days	Count	Days	Equip days	Count	Days	Equip days	Equip days
Bulldozer			-	5	86	430	5	266	1,330	1,760
Compactor			-			-			-	-
Concrete pump	1	194	194	1	29	29	1	83	83	306
Concrete Trucks	3	194	582	4	29	116	4	83	332	1,030
Crane			-	1	200	200	1	150	150	350
Dump truck Tunnel	3	200	600			-			-	600
Excavator			-			-			-	-
Fork Lift	1	248	248	1	72	72	1	190	190	510
Fuel Truck	1	200	200	1	86	86	1	266	266	552
Generator	1	200	200			-			-	200
Grader			-	1	40	40	2	266	532	572
Highway Trucks	5	54	270	3	34	102	10	80	800	1,172
Loader - tunnel	2	200	400			-			-	400
Material trucks- concrete	8	194	1,552	8	38	304	8	110	880	2,736
Off-Road Truck										
Roller										
Scissor lift	1	100	100			-			-	100
Scraper	-	-	-	5	86	430	10	266	2,660	3,090
Tunnel boring Machine	1	200	200			-			-	200
Water Trucks			-	1	86	86	1	266	266	352
Welding truck	1	194	194	1	50	50	1	50	50	294
			-			-			-	-
Staff	42		-	46		-	59		-	-

All equipment other than tunnels assumed 10 hr. day. Tunnels assumed 22 hours/day.

Pipelines - Delevan and TRR

	Excavation			Backfill			Pipe & Manhole Installation			Total
Equipment	Count	Days	Equip days	Count	Days	Equip days	Count	Days	Equip days	Equip days
Backfill Loader				2	467	934			-	934
Bulldozer	4	538	2,152	2	467	934			-	3,086
Compactor	-		-	2	467	934			-	934
Conc. Material trucks							8	28	224	224
Concrete Trucks			-	-	-	-	1	83	83	83
Crane Fabrication							1	1,000	1,000	1,000
Crane Installation							1	500	500	500
Dump truck 20% Exc.	10	400	4,000	-		-			-	4,000
Dump truck Comp. BF	-			10	467	4,670			-	4,670
Excavation Loader	1	400	400	-		-			-	400
Excavator	1	400	400	-		-			-	400
Fork Lift - Installation							1	500	500	500
Fork Lift - Yard							1	1,000	1,000	1,000
Fuel Truck	1	500	500	1	467	467			-	967
Generator	1	500	500	1	83	83			-	583
Grader			-	1	467	467			-	467
Highway Trucks	10	20	200			-	10	499	4,990	5,190
Jacking Equipment	-	-	-	-	-	-	1	600	600	600
Pipe Fabrication Equip.							1	1000	1000	1000
Pipe transport truck							2	550	1,100	1,100
Scraper 80% EXC.	22	485	10,670	-		-			-	10,670
Scraper Common BF	-			8	383	3,064			-	3,064
Water Trucks	1	500	500	1	467	467			-	967
Staff	61			43			37			

Dams & Sites Inundation

	Excavation & Fill			Drilling & Grouting			Concrete			Total
Equipment	Count	Days	Equip days	Count	Days	Equip days	Count	Days	Equip days	Equip days
Bulldozer	10	1,774	17,740							17,740
Compactor	10	1,535	15,350							15,350
Conc. Material Trucks							8	215	1,720	1,720
Concrete pump			-				1	81		-
Concrete Trucks							8	81	648	648
Crane			-							-
Drilling machine				4	488	1,952				1,952
Dump truck	10	83	830							830
Filter Material Loader	3	1,160	3,480							3,480
Excavator			-							-
Fork Lift - Yard			-	1	89	89				89
Fuel Truck	2	1,774	3,548							3,548
Generator			-				1	81	81	81
Grader	5	1,535	7,675							7,675
Grout Pump				4	488	1,952				1,952
Highway Trucks	120	375	45,000	2	164	328				45,328
Off-Road Trucks	24	1,160	27,840							27,840
Scraper - Excavation	20	340	6,800							6,800
Waste/ Tree loader	1	83	83							83
Water Trucks	4	1,774	7,096							7,096
			-							-
Staff	219		-	12			10			-

Gravel Roads

	Grading			Agg Base			Paving			Total
Equipment	Count	Days	Equip days	Count	Days	Equip days	Count	Days	Equip days	Equip days
Bulldozer	3	64	192	3	29	87			-	279
Compactor	3	23	69	3	29	87			-	156
Concrete Trucks			-			-			-	-
Crane			-			-			-	-
Drilling machine			-			-			-	-
Dump truck			-	3	36	108	3	5	15	123
Excavator			-			-			-	-
Fork Lift - Yard			-			-			-	-
Fuel Truck	1	64	64	1	29	29			-	93
Generator			-			-			-	-
Grader	2	23	46	2	29	58			-	104
Grout Pump			-			-			-	-
Highway Trucks			-			-			-	-
Loader			-	1	36	36	1	5	5	41
Off-Road Trucks			-			-			-	-
Paving Machine			-			-	1	5	5	5
Roller			-			-	2	5	10	10
Scraper - Excavation	3	49	147			-			-	147
Water Trucks	2	64	128	2	29	58	1	5	5	191
Staff Max	24		-	25		-	18		-	-
			-			-			-	-

TRR Pumping-Generating Plant

	Excavation & Backfill			Concrete & Rock			Other Activities			Total
Equipment	Count	Days	Equip days	Count	Days	Equip days	Count	Days	Equip days	Equip days
Batch plant equipment			-	1	104	104			-	104
Bulldozer	5	233	1,165			-			-	1,165
Compactor	2	100	200			-			-	200
Concrete Pump			-	1	104	104			-	104
Concrete Trucks			-	4	104	416			-	416
Crane	-		-	-		-	1	200	200	200
Drilling machine			-			-			-	-
Dump truck	10	125	1,250			-			-	1,250
Excavator			-			-			-	-
Fork Lift			-			-	1	400	400	400
Fuel Truck	1	233	233			-	1	100	100	333
Generator			-	1	104	104			-	104
Grader	2	100	200			-			-	200
Highway Trucks			-	8	147	1,176	8	73	584	1,760
Loader	1	125	125			-			-	125
Off-Road Trucks			-			-			-	-
Paving Machine			-			-			-	-
Roller			-			-			-	-
Scraper - Excavation & BF	5	233	1,165			-			-	1,165
Water Trucks	2	233	466			-			-	466
Staff Max	38		-	25		-	101		-	-

Peak staff for other activities is set at 80 for building construction. Duration for buildings should be about two years.

TRR Reservoir

	Clear & Excavation			Backfill			AC Lining & Structures			Total
Equipment	Count	Days	Equip days	Count	Days	Equip days	Count	Days	Equip days	Equip days
Backfill Loader			-			-			-	-
Bulldozer	4	180	720	4	33	132			-	852
Compactor			-	2	33	66			-	66
Concrete Trucks			-			-			-	-
Crane			-			-			-	-
Drilling machine			-			-			-	-
Dump truck			-			-			-	-
Excavation Loader	1	152	152			-			-	152
Excavator			-			-			-	-
Fork Lift - Yard			-			-	1	140	140	140
Fuel Truck	1	152	152	1	33	33			-	185
Generator			-			-			-	-
Grader			-	1	33	33			-	33
Grout Pump			-			-			-	-
Highway Trucks			-			-	5	140	700	700
Off-Road Trucks	10	152	1,520			-			-	1,520
Paving Machine			-			-	1	20	20	20
Scraper - Excavation	4	130	520	4	33	132			-	652
Water Trucks			-			-			-	-
			-			-			-	-
Staff	30			22			57			-

Staff allowed for structures is 40 for a period of 16 months

Holthouse (Enlarged Funks) Reservoir

	Excavation & Fill			Drilling & Grouting			Concrete			Total
Equipment	Count	Days	Equip days	Count	Days	Equip days	Count	Days	Equip days	Equip days
Bulldozer - Dam	10	285	2,850			-			-	2,850
Dozers for Clear/Grub	5	40	200			-			-	200
Dozers for Channel	20	530	10,600			-			-	10,600
Compactor	4	199	796			-			-	796
Conc. Material Trucks			-			-	8	59	472	472
Concrete pump			-			-	1	22	22	22
Concrete Trucks			-			-	8	22	176	176
Crane			-			-			-	-
Drilling machine			-	1	85	85			-	85
Dump truck	2	4	8			-			-	8
Backfill Loader	1	149	149			-			-	149
Excavator			-			-			-	-
Fork Lift - Yard			-			-	1	59	59	59
Fuel Truck	1	570	570			-			-	570
Generator			-			-	1	22	22	22
Grader	2	199	398			-			-	398
Grout Pump			-	2	85	170			-	170
Highway Trucks	80	50	4,000	2	18	36			-	4,036
Off-Road Trucks	10	149	1,490			-			-	1,490
Scraper - for Dam	10	86	860			-			-	860
Scraper - for Dam	20	530	10,600			-			-	10,600
Waste/ Tree loader	1	4	4			-			-	4
Water Trucks	4	570	2,280			-			-	2,280
			-			-			-	-
Staff	180		-	11		-	51		-	-

Sacramento River Pumping-Generating Plant (Including Fish Screening facilities)

	Excavation & Backfill			Concrete & Rock			Other Activities			Total
Equipment	Count	Days	Equip days	Count	Days	Equip days	Count	Days	Equip days	Equip days
Batch plant equipment			-	1	104	104			-	104
Bulldozer	5	233	1,165			-			-	1,165
Compactor	2	100	200			-			-	200
Concrete Pump			-	1	104	104			-	104
Concrete Trucks			-	4	104	416			-	416
Crane	-		-	-		-	1	200	200	200
Drilling machine			-			-			-	-
Dump truck	10	125	1,250			-			-	1,250
Excavator			-			-			-	-
Fork Lift			-			-	1	400	400	400
Fuel Truck	1	233	233			-	1	100	100	333
Generator			-	1	104	104			-	104
Grader	2	100	200			-			-	200
Highway Trucks			-	8	147	1,176	8	73	584	1,760
Loader	1	125	125			-			-	125
Off-Road Trucks			-			-			-	-
Paving Machine			-			-			-	-
Roller			-			-			-	-
Scraper - Excavation & BF	5	233	1,165			-			-	1,165
Water Trucks	2	233	466			-			-	466
Staff Max	38		-	25		-	101		-	-

Peak staff for other activities is set at 80 for building construction. Duration for buildings should be about two years.

Paved Roads & Bridges

	Grading			Agg Base & Paving			Bridges			Total
Equipment	Count	Days	Equip days	Count	Days	Equip days	Count	Days	Equip days	Equip days
Backhoe/drill				6	245	1,470	0	0	-	
Caisson Drill Rig							1	105	105	
Bulldozer	12	811	9,732			-	2	19	38	9,770
Compactor			-			-			-	-
Concrete Trucks			-	4	211	846	10	140	1,400	2,246
Concrete material trucks							12	310	3720	
Concrete Pump							2	140	280	
Crane			-			-	2	500	1,000	1,000
Drilling machine			-			-			-	-
Dump truck	10	30	300	20	315	6,300	5	35	175	6,775
Excavator			-			-	2	13	26	26
Fork Lift - Yard			-			-			-	-
Fuel Truck	1	811	811	1	315	315			-	1,126
Generator			-			-	1	500	500	500
Grader	4	211	844	4	315	1,260			-	2,104
Grout Pump			-			-			-	-
Highway Trucks			-	1	235	235	12	88	1,056	1,291
Loader	2	285	570	2	315	630	1	35	35	1,235
Off-Road Trucks	20	228	4,560			-			-	4,560
Paving Machine			-	2	40	80			-	80
Roller	4	211	844	2	40	81			-	925
Scraper	16	546	8,736			-			-	8,736
Water Trucks	2	811	1,622	2	315	630			-	2,252
Staff Max	81		-	48		-	59		-	-
			-			-			-	-

GCID Canal and Headworks Modifications

	Grading & Paving			Concrete Work			Miscellaneous			Total
Equipment	Count	Days	Equip days	Count	Days	Equip days	Count	Days	Equip days	Equip days
Bulldozer	1	32	32			-			-	32
Compactor	1	159	159			-			-	159
Concrete Trucks			-	6	26	156			-	156
Crane			-			-			-	-
Drilling machine			-			-			-	-
Dump truck	4	192	768			-			-	768
Excavator			-			-			-	-
Fork Lift - Yard			-			-			-	-
Fuel Truck	1	205	205			-	1	130	130	335
Generator			-	1	26	26	1	130	130	156
Grader			-			-			-	-
Grout Pump			-			-			-	-
Highway Trucks			-	12	35	420	2	130	260	680
Loader	1	192	192			-			-	192
Off-Road Trucks			-			-			-	-
Paving Machine	1	33	33			-			-	33
Roller	2	33	66			-			-	66
Scraper - Excavation	3	46	138			-			-	138
Water Trucks	1	205	205			-			-	205
Staff Max	25		-	29		-	14		-	-
			-			-			-	-

Recreation Facilities

	Grading & Paving			Concrete Work			Miscellaneous			Total
Equipment	Count	Days	Equip days	Count	Days	Equip days	Count	Days	Equip days	Equip days
Bobcat	2	158								
Bulldozer	1	116	116			-			-	116
Compactor			-			-			-	-
Concrete Trucks			-	2	33	66			-	66
Concrete Material trucks				4	44					
Crane			-			-			-	-
Drilling machine			-			-			-	-
Dump truck	3	158	474			-			-	474
Excavator			-			-			-	-
Fork Lift - Yard			-			-	1	121	121	121
Fuel Truck	1	158	158	1	33	33	1	121	121	312
Generator			-	1	33	33			-	33
Grader	1	28	28			-			-	28
Grout Pump			-			-			-	-
Highway Trucks	4	10	40			-	2	121	242	282
Loader	1	158	158			-			-	158
Off-Road Trucks			-			-			-	-
Paving Machine	1	22	22			-			-	22
Roller	1	50	50			-			-	50
Scraper - Excavation			-			-			-	-
Water Trucks	1	144	144			-			-	144
Staff Max	24		-	18		-	14		-	-
			-			-			-	-

B.7 REFERENCES

Buer, Koll (DWR), 1994. *Use of Alternative Gravel Sources for Fishery Restoration and Riparian Habitat Enhancement in Shasta and Tehama Counties, California*. Internal Memorandum. August.

CALFED Bay-Delta Program, 1997. *Ecosystem Restoration Program Plan*. Review Draft. Vols. I and II.

California Department of Conservation. Map of Abandoned and Active Gravel Mines along Upper Sacramento River, provided via e-mail by Alicia Johnson (August 19, 2008).

Davis-Dolwig Act (Enacted in 1961). Chapter 867, Statutes of 1961 (AB 261, Davis).

DWR, 1965. *The Recreation Potentials of the Tentative Water Projects of the Upper Sacramento River Basin Investigation*. Bulletin 150, Appendix A.

DWR, 2000. *North of Delta Offstream Storage Investigation Draft Progress Report, Appendix J: Recreation Requirements and Opportunities: Sites Reservoir Alternative*. April.

DWR, 2001. *Comparative Inventory of Recreation Facilities at California's Largest Reservoirs, 2000.* December.

DWR, 2003. *North of Delta Offstream Storage Investigation – Sites Reservoir Engineering Feasibility Study Pumping Plants and Appurtenant Facilities*. June.

DWR, Division of Engineering (DOE), 1999a. *Large Sites Reservoir, Progress Report on Feasibility Studies for Golden Gate and Sites Dams*. Draft. June.

DWR, DOE, 1999b. *Large Sites Reservoir, Progress Report on Feasibility Studies for Ridge Analysis*. Draft. June.

DWR, DOE, 2000. *Engineering Progress Report on Feasibility Studies for Sites Reservoir*. Draft. August.

DWR, DOE, 2002a. *Sites Reservoir Addendum to Embankment Volume and Reservoir Storage Assessment*. May.

DWR, DOE, 2002b. *Sites Reservoir Embankment Volume and Reservoir Storage Assessment.* May.

DWR, DOE, 2002c. *Sites Reservoir Feasibility Study – Materials Investigation, Testing, and Evaluation Program*. June.

DWR, DOE, 2003a. *Sites Reservoir Engineering Feasibility Study* - *Golden Gate, Sites, and Saddle Dams*. February.

DWR, DOE, 2003b. *Geologic Feasibility Report, Sites Reservoir Project, Project Geology Report No. 94-30-02*. July.

DWR, DOE, 2003c. *Sites Project, Feasibility Study for Conveyance, Colusa Basin Drain, and Funks Reservoir Modification*. August.

DWR, DOE, 2003d. *Sites Reservoir Engineering Feasibility Study - Pumping Plants and Appurtenant Facilities*. June.

DWR, DOE, 2008. *An Addendum Amending the August 2003 Feasibility Study Report for Sites P-G Plant*. July.

CH2M HILL, 2001. *Fish Passage Improvement Project Preliminary Design Report, Phase II, Volume II.*

CH2M HILL, 2003a. *Stony Creek Alternative for Conveyance to Sites Reservoir North-of-the-Delta Offstream Storage Evaluations*. May.

CH2M HILL, 2003b. *Engineering Feasibility Report on Modifications to GCID Fish Screen and Enlargement of Main Canal*. April.

CH2M HILL, 2008. *Feasibility Study for DWR for the Fish Screen Facility and the Discharge of Water Back into the Sacramento River*. June.

Colusa Basin Flood Flow Frequency Analysis, 1987. March 10.

Haas, G., R. Aukerman, V. Lovejoy, and D. Welch, 2004. *Water Recreation Opportunity Spectrum Users' Guidebook*. United States Department of the Interior, Bureau of Reclamation, Office of Program and Policy Services, Denver Federal Center, Lakewood, Colorado. July.

Minear, J.T., and G.M. Kondolf et al., 2006. Gravel Augmentation for Salmon Habitat below Dams in Northern California. PowerPoint presentation available at <http://rrnw.org/skamania2006/presentations/Minear.RRNW.020206.pdf> (accessed August 19, 2008).

National Park Service (U. S. Department of Interior – National Park Service), 1965. *Project Report on the Recreation Potential of the Proposed West Sacramento Canal Unit Central Valley Project California*.

Bureau of Reclamation (Reclamation), Haas, G., R. Aukerman, V. Lovejoy, and D. Welch, 2004. *Water Recreation Opportunity Spectrum Users' Guidebook*. Office of Program and Policy Services, Denver Federal Center, Lakewood, Colorado. July.

Reclamation (United States Department of the Interior [USDOI]), 2006. *Draft Shasta Lake Water Resources Investigation Plan Formulation Report.* December.

Reclamation, 2007. *A Conceptual Framework for Modeling of Physical River Processes and Riparian Habitat on the Sacramento River, California*. NODOS Project Report. April.

Reclamation, 2008a. *Draft North of Delta Offstream Storage Investigation Plan Formulation Report.* September.

Reclamation, 2008b. *Record of Decision on Final Environmental Impact Statement for Fish Passage Improvement Project at the Red Bluff Diversion Dam*.

Sites Reservoir PMF Analysis, 2004. August.

The Nature Conservancy, G. Mathias Kondolf, and Stillwater Sciences, 2007a. *Sacramento River Ecological Flows Study: Off-Channel Habitat Study Final Report.* Prepared for the CALFED Ecosystem Restoration Program and the Resources Legacy Fund Foundation.

The Nature Conservancy, Stillwater Sciences, 2007b. *Sacramento River Ecological Flows Study: Gravel Study Final Report.* Prepared for the CALFED Ecosystem Restoration Program. December 21.

The Nature Conservancy, Stillwater Sciences, Essa Technologies, 2008. *Sacramento River Ecological Flows Study Final Report.* Prepared for the CALFED Ecosystem Restoration Program.

URS Corporation (URS) and CH2M HILL, 2003. *Tehama-Colusa Canal Alternatives for North-of-the-Delta Offstream Storage*. May.

URS, 2008a. *Pump-Back Technical Memorandum*. August.

URS, 2008b. *Environmental Enhancement for Fisheries Technical Memorandum*. October.

URS, 2009. *Recreation Technical Memorandum*. March.

Water Project Recreation Act, 1965. United States Code Citation: 16 U.S.C. 460l-12 -460l-22, 662, [16 U.S.C 460(L)(12)- 460(L)(21); P.L.89-72; July 9, 1965; 79 Stat. 213] as amended by: P.L. 93-251; March 7, 1974; 88 Stat. 33 and P.L. 94-576; October 21, 1976; 90 Stat. 2728.

William Lettis and Associates, Inc., 2002. *North of the Delta Offstream Storage Investigations, Phase II Fault and Seismic Hazards Investigation*. September.