

B.3 Design Considerations

General

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. Tributary 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 T-C Canal below Orland. The PFR screening provided a short list of conveyance management measures to carry forward in the NODOS Investigation. Subsequent evaluation of the short list led to the selection of the GCID Canal, the T-C Canal, and the Delevan Pipeline as the preferred conveyances for water drafted from the Sacramento River. This section, Design Considerations, of Appendix B, Engineering, describes the engineering aspects of the four alternative projects (Alternatives A, B, C, and D) currently under consideration for the proposed NODOS/Sites Reservoir Project that would use the preferred water sources and conveyances. This includes general summaries of the PFR, the preliminary feasibility reports prepared by DWR's DOE and its consultants, and the Technical Memoranda developed post-PFR. Additional information regarding the technical analyses and designs is available in the references listed in Section B.7, References, of this appendix.

Figure B.3-1 shows a map of the basic facilities that constitute the four NODOS/Sites Reservoir Project alternatives. Facility details may vary between alternatives. Table B.3-1 summarizes assumed storage reservoir sizes and assumed design flows for the conveyance facilities handling project water for each alternative. Figure B.1-1 shows the storage and conveyance facilities with flows in schematic form.

Table B.3-1. Storage and Conveyance Parameters for Project Alternatives

	Alternative A	Alternative B	Alternative C	Alternative D
Reservoir Sizes				
Sites Reservoir	1.3 MAF	1.8 MAF	1.8 MAF	1.8 MAF
Holthouse Reservoir	6,500 AF	6,500 AF	6,500 AF	6,500 AF
TRR	1,200 AF	1,200 AF	1,200 AF	1,200 AF
Canal Conveyances				
T-C Canal	2,100 cfs	2,100 cfs	2,100 cfs	2,100 cfs
GCID Canal	1,800 cfs	1,800 cfs	1,800 cfs	1,800 cfs
Delevan Pipeline Conveyance				
Diversion	2,000 cfs	No Pumping	2,000 cfs	2,000 cfs
Release	1,500 cfs	1,500 cfs	1,500 cfs	1,500 cfs
TRR Pipeline				
Diversion	1,800 cfs	1,800 cfs	1,800 cfs	1,800 cfs
Release	900 cfs	900 cfs	900 cfs	900 cfs

Key:

AF = acre-feet

cfs = cubic feet per second

GCID = Glenn-Colusa Irrigation District

MAF = million-acre-feet

T-C = Tehama-Colusa

TRR = Terminal Regulating Reservoir

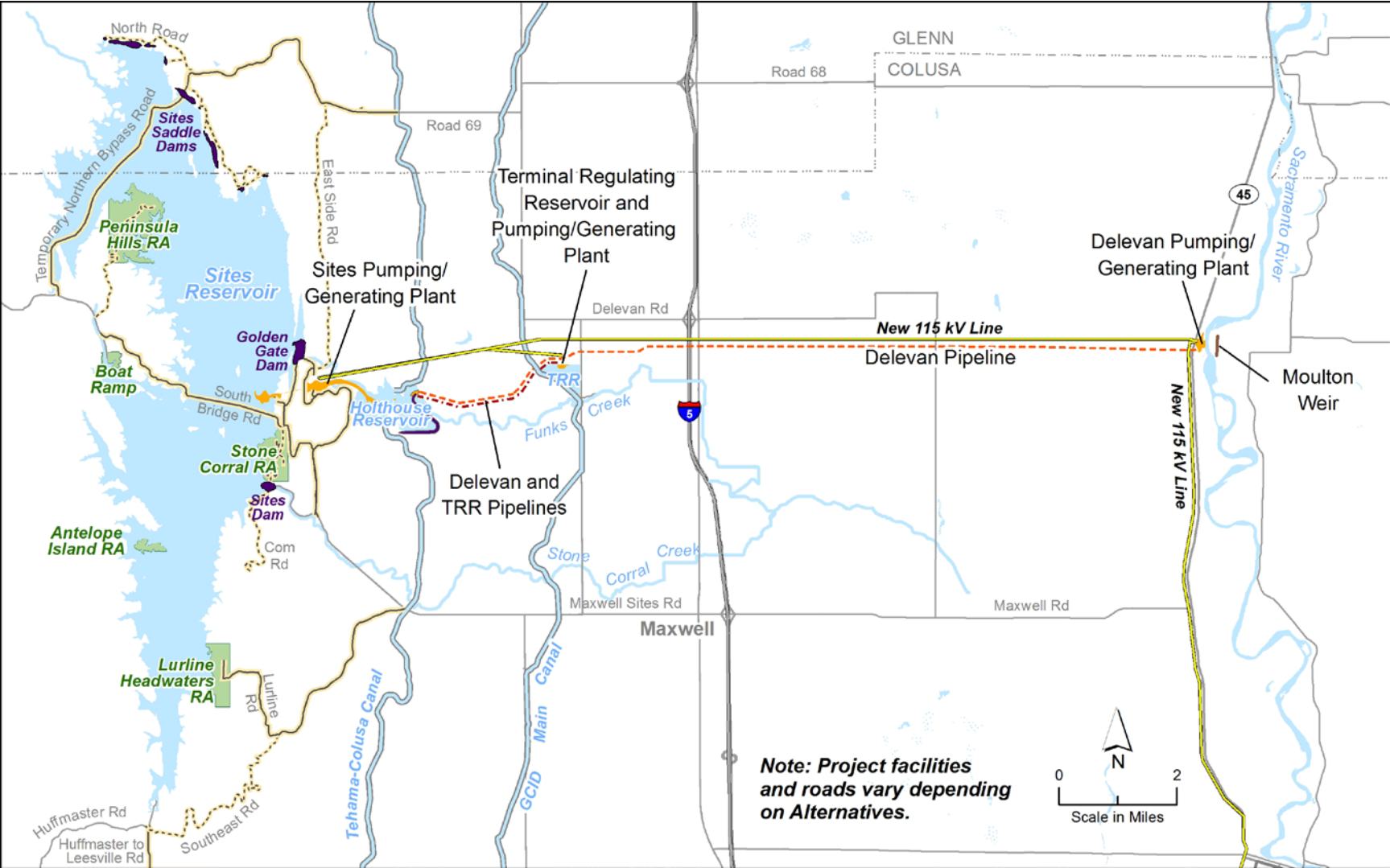


Figure B.3-1. Project Description Map

Sites Reservoir Filling

Alternatives A, C, and D would fill Sites Reservoir using water from the existing T-C Canal, GCID Canal, and from the Sacramento River via the Delevan Pipeline during the winter and spring, when water is available for diversion. The water from the Sacramento River would come to Holthouse Reservoir through the Delevan Pipeline Intake Facilities, which includes the SRPGP and associated fish screen facility. The water from the GCID Canal would come to Holthouse Reservoir through the Terminal Regulating Reservoir (TRR) Pipeline, which includes the TRR Pumping/Generating Plant. Water from the T-C Canal would flow directly into Holthouse Reservoir. From Holthouse Reservoir, water would be pumped to Sites Reservoir using the SPGP.

Alternative B would fill Sites Reservoir from Holthouse Reservoir using water conveyed from the existing T-C Canal and GCID Canal when water is available for diversion. Constructing a new pumping/generating facility on the Sacramento River is not a part of Alternative B. The Delevan Pipeline for this alternative would only be used to make releases from Sites Reservoir back to the river.

Sites Reservoir Releases

Alternatives A, C, and D would make summer irrigation and environmental releases from Sites Reservoir to Holthouse Reservoir through the SPGP. From Holthouse Reservoir, required releases would then be made to the T-C Canal, to the GCID Canal through the TRR pipeline, and to the Sacramento River through the Delevan Pipeline. The releases from Sites to Holthouse Reservoir would typically be made through the SPGP, and used to generate hydroelectric power using pump-turbine units in the plant. Likewise, the releases to the Sacramento River and to TRR from Holthouse Reservoir would be used to generate hydroelectric power using one or more dedicated turbine units in the TRR Pumping/Generating Plant and SRPGP.

Alternative B does not include constructing the SRPGP; however, as indicated above, would still include the Delevan Pipeline between Holthouse Reservoir and the Sacramento River to provide the capability to make summer releases to the river from Sites Reservoir. The releases to the river would be made through multiple energy dissipating valves contained in a valve structure constructed on the river bank, with no hydropower generation. The valve structure would be on the river, in the same cove where the pump station fish screen facility would be constructed for Alternatives A, C, and D. Alternative B, however, still includes hydroelectric power generation using release water at the SPGP and the TRR Pumping/Generating Plant.

As shown in Table B.3-1, the release rate from Holthouse Reservoir back to the GCID Canal through the TRR Pipeline would be 900 cfs for the four alternatives. This release is less than the 1,800 cfs release assumed previously for Alternatives A, B, and C. Flows in excess of 900 cfs cannot be accommodated in GCID's current system beyond TRR, based on water demands, flow restrictions at the existing Funks Creek Siphon, and reduced canal capacity downstream of TRR. The lower release rate was discussed with GCID, and confirmed. Currently, GCID has no plan to increase the size of the Funks Creek siphon or the capacity of the canal downstream. The reduced return flow also reduces the size of the turbine units in the TRR Pumping/Generating Plant, and the potential for hydroelectric power generation assumed previously.

Holthouse Reservoir

Figure B.3-1 shows the location of Holthouse Reservoir. This reservoir is a forebay/afterbay pond that works in conjunction with Sites Reservoir to collect and release NODOS/Sites Reservoir Project water, and is a common feature for all four project alternatives. In winter, the reservoir is a forebay pond for the SPGP that collects all diverted water to be pumped into Sites Reservoir. In the spring and summer, the reservoir is an afterbay pond that receives water released from Sites Reservoir through the SPGP, and distributes the water back to the canals and the Sacramento River to meet demand.

Based on a preliminary evaluation, the reservoir would have a minimum active storage of approximately 6,500 AF, located between elevation 190.0 and elevation 205.0. This storage allocation was selected based on the following operating assumptions:

- The SPGP would be operated as a pumped storage facility (particularly during the spring and summer) to help maximize hydroelectric power generation benefits. Water released through the pump-turbine units in SPGP for generation during the on-peak hours would be stored in Holthouse Reservoir. Residual water that is not needed from Holthouse Reservoir to meet downstream demands would be pumped back to Sites Reservoir during off-peak periods.
- Some additional storage would also be provided to collect inflows to the project that might still be occurring from the canals and Sacramento River concurrent with the pumped-storage operation. This could occur in some years in early spring or late fall depending on hydrologic conditions. This water would also be pumped back to Sites Reservoir during off-peak periods, along with the collected generation water.
- During peak summer periods, it may be possible that no pump-back is needed when generation releases balance demand.

The operation described above would be a daily operation and assumes that water could be released through the SPGP at maximum design flows. In the summer, generation water collected in Holthouse Reservoir would be released to the canals and the Sacramento River to meet demand. More detailed studies would be required in the future to refine project operation throughout the year, including coordinating Sites Reservoir operation with other existing Federal and State storage reservoirs.

Holthouse Reservoir would be formed by combining existing storage in Funks Reservoir with additional new storage formed behind a new dam that would be downstream (east) of the existing Funks Dam. The radial gates in the existing spillway at Funks Dam would be removed, and the existing dam would be partially breached if necessary, so that the existing pool and the new pool would function as one reservoir with common operating levels. Sediment that has accumulated in the existing Funks Reservoir would be removed to the extent necessary as part of project construction wherever it interferes with the construction of new project components. The new dam would include a new spillway sized to pass approximately 20,000 cfs, which is the emergency release flow required to meet DSOD emergency reservoir drawdown criteria for Sites Reservoir. The emergency releases would be low level releases made through four energy dissipation valves located in a valve structure adjacent to the SPGP that would flow to Holthouse Reservoir through the channel connecting the reservoir with the pump station.

Sites Reservoir

The principal storage feature of the NODOS/Sites Reservoir Project is Sites Reservoir. Figure B.3-1 shows the location of Sites Reservoir, and the various dams to be constructed to form the reservoir. Sites Reservoir would have a nominal storage capacity of 1.3 MAF for Alternative A, and 1.8 MAF for Alternatives B, C, and D.

Water in Sites Reservoir would be impounded by the Golden Gate Dam on Funks Creek, the Sites Dam on Stone Corral Creek, and by a series of saddle dams along the eastern and northern rims of the reservoir. The saddle dams close off topographic saddles in the ridge forming the reservoir. The 1.8 MAF reservoir (maximum Normal Water Surface Elevation 520.0) requires nine saddle dams. Six saddle dams would be required for the 1.3 MAF reservoir (maximum Normal Water Surface Elevation 480.0), because the maximum reservoir water level would be 40 feet lower in elevation.

The SPGP would pump water from Holthouse Reservoir to Sites Reservoir through an inlet/outlet works to the south of Golden Gate Dam. The same inlet/outlet works would be used to make releases from Sites Reservoir through the SPGP. The inlet/outlet works encompasses a large-diameter tunnel through the ridge, a vertical inlet tower in the reservoir controlling flows at the upstream tunnel portal, and a system of penstock piping connecting the downstream tunnel portal with the SPGP. Releases to or from the reservoir would be made through an array of gated outlet ports around the perimeter of the vertical outlet tower at various elevations to accommodate varying reservoir water levels, and to regulate outlet water temperature. The inlet/outlet works concept would be the same for the 1.8 MAF and 1.3 MAF reservoirs; the height of the structure and number of gated ports would be smaller for the smaller reservoir.

Sites Reservoir construction would require relocating county roads (Maxwell-Sites Road, Sites-Lodoga Road, and Huffmaster Road) and the community of Sites. Other new paved or unpaved roads would also be provided to access project facilities from existing roads, and to improve operation and maintenance access between main dam and saddle dam areas.

Although recreational use has not been considered as a primary project purpose, five potential recreational facility locations were previously evaluated. Alternatives A, B, and C would include the Stone Corral, Antelope Island, and Lurline Headwaters Recreation Areas. Alternative D would include the Stone Corral and Peninsula Hills Recreation Areas, as well as a boat-launch facility on the western side of the reservoir near the location where the existing Sites-Lodoga Road exits the reservoir. Appendix E provides additional information about the recreation evaluation.

Modifications to T-C Canal Facilities

General

All four alternatives rely on two existing canals to divert water into Sites Reservoir. The first canal is the T-C Canal, which diverts water from the Sacramento River at Red Bluff. Construction for the Tehama-Colusa Canal Authority Fish Passage Improvement Project at the Red Bluff Diversion Dam (RBDD) was completed in 2012. The project components (overlain on an aerial photograph schematic on Figure B.3-2) include a fish screen facility, a pumping plant at the Mill Site (known as the Red Bluff Pumping Plant), a canal, a siphon, a forebay, a switchyard,

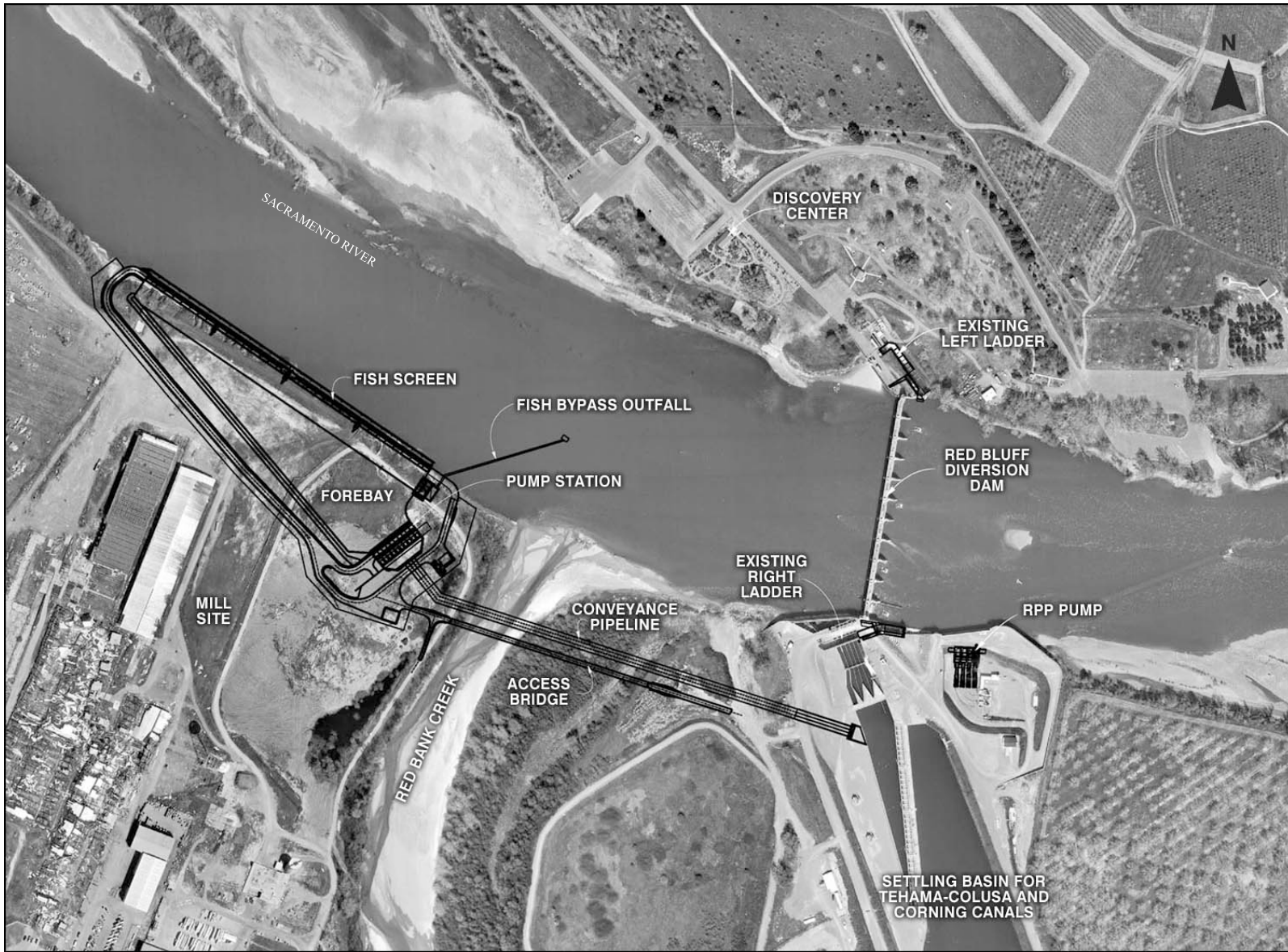


Figure B.3-2. General Layout of Fish Passage Improvement Facility

and a bridge across Red Bank Creek. The fish screen structure is designed to meet National Marine Fisheries Service and California Department of Fish and Wildlife criteria for diversion flows of 80 to 2,500 cfs.

The new pumping plan would accommodate up to 11 pumps, providing an ultimate total capacity of 2,500 cfs. Nine of the 11 vertical axial-flow pumps (seven 250 cfs and two 125 cfs) are installed in the pumping plant, providing a capacity of 2,000 cfs. Two spare pump bays are provided in the structure for capacity expansions. Two additional 250 cfs pump would be added in one of the bays for the NODOS/Sites Reservoir Project, bringing the capacity to 2,500 cfs.

New Red Bluff Pumping Plant Facilities

NODOS/Sites Reservoir Project operation would require the installation of two 250 cfs vertical axial-flow pump in one of the two spare pump bays at the Red Bluff Pumping Plant to bring the total installed capacity of the plant to 2,500 cfs.

Pump Installation, Operation, and Maintenance

Pump Installation

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

- Installation of new butterfly valves. The spare pump bays to receive the NODOS/Sites Reservoir 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 new 84-inch flap gates. Permanent supports would be required beneath the butterfly valve and flap gate.
- Installation of new pumps. 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.
- Installation of the pumping plant unit bay stoplogs, using a mobile crane, to accommodate dewatering the pump bay.
- Inspection of the pump bays and removal of all sediment. Access to the bottom floor of the pumping plant is provided at each bay via 4-foot, 6-inch by 7-foot access hatches and ladders.
- Removal of roof hatches over the pump unit bays using a mobile crane.
- Installation of the pumps 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.
- Installation of electrical conductors and a supervisory control and data acquisition (SCADA) systems.

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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. Once in the settling basin, water would flow to Check No. 1 on the T-C Canal and the Corning Pumping Plant, similar to the current operation. The additional pump at the pumping plant would allow for normal operational diversions up to 2,160 cfs for each NODOS project 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 T-C 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/Sites Reservoir Project vertical axial-flow pump and its appurtenances that would be installed as part of the 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 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 in the pumping plant forebay than prior to the NODOS project pump installation and operation.

GCID Canal Modifications

The GCID Canal is the second existing canal that would divert water into Sites Reservoir under all four alternatives. 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 the NODOS project to enhance performance and reliability:

- Upgrade the canal headworks structure and line 200 feet of the canal with concrete immediately downstream of the headgate structure (see Figure B.3-3 and Figure B.3-4)
- Replace the railroad siphon undercrossing

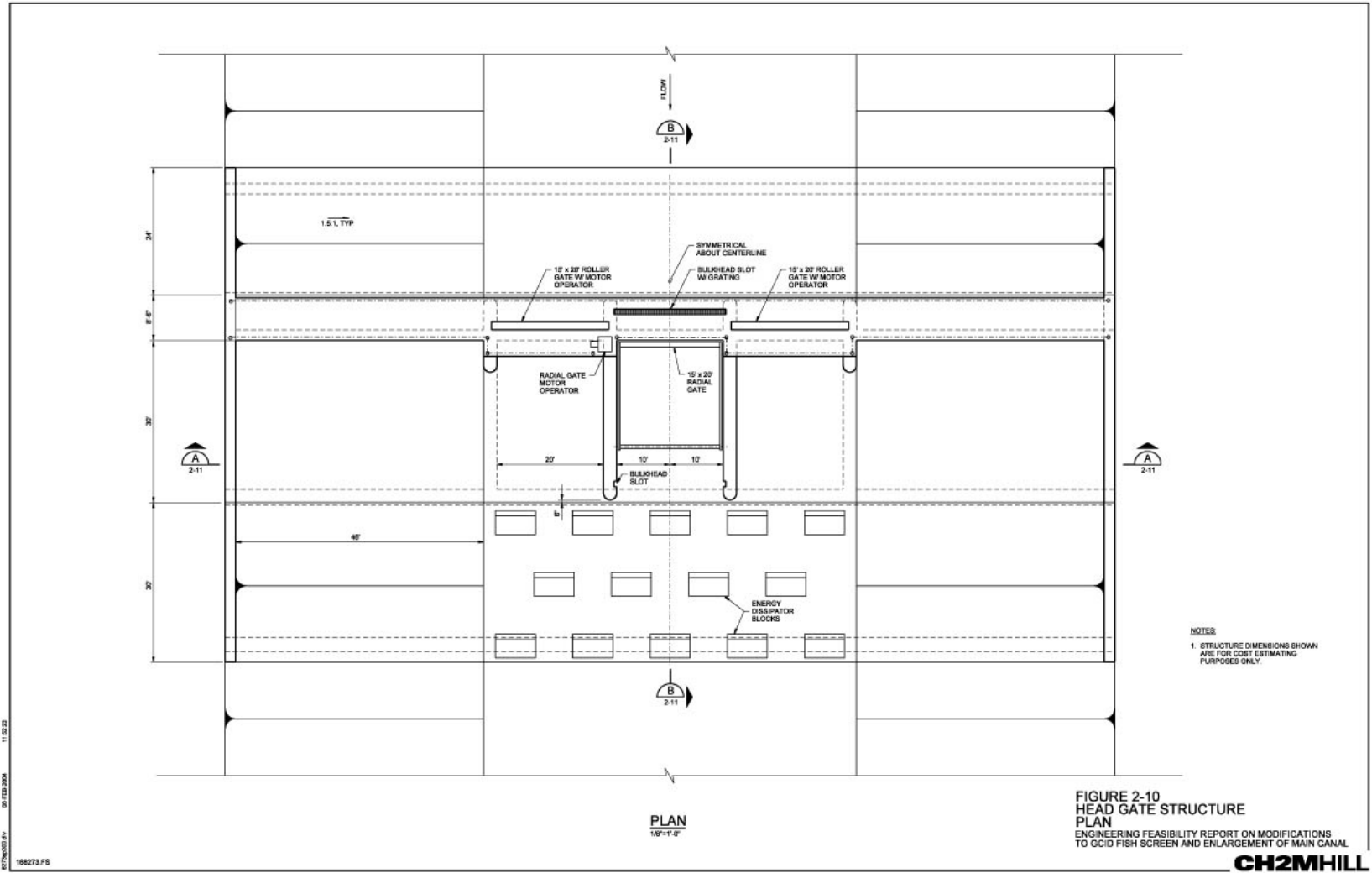


Figure B.3-3. GCID Canal Headgate Structure – Plan

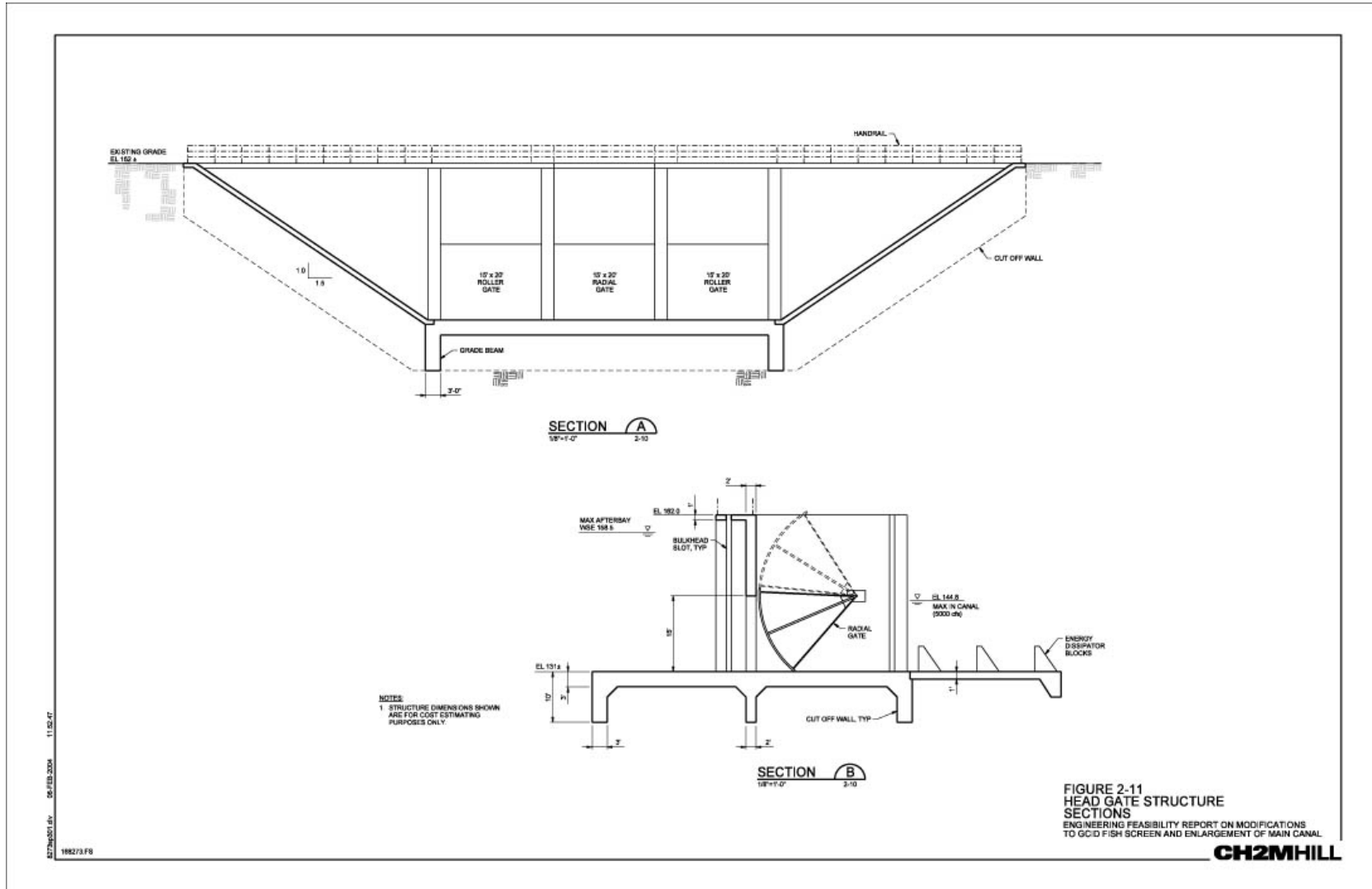


Figure B.3-4. GCID Canal Headgate Structure – Sections

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/Sites Reservoir 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 SCADA systems are being added and extended in its system, particularly in the area of the existing headworks. Incorporating SCADA systems on their canal for existing canal facilities would not be necessary. Such systems may, however, still be required for new NODOS/Sites Reservoir Project work relative to the canal. The design of new systems must be coordinated with GCID to provide for 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. Figure B.3-3 and Figure B.3-4 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 occurs during high river levels.
- 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.3-5 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.

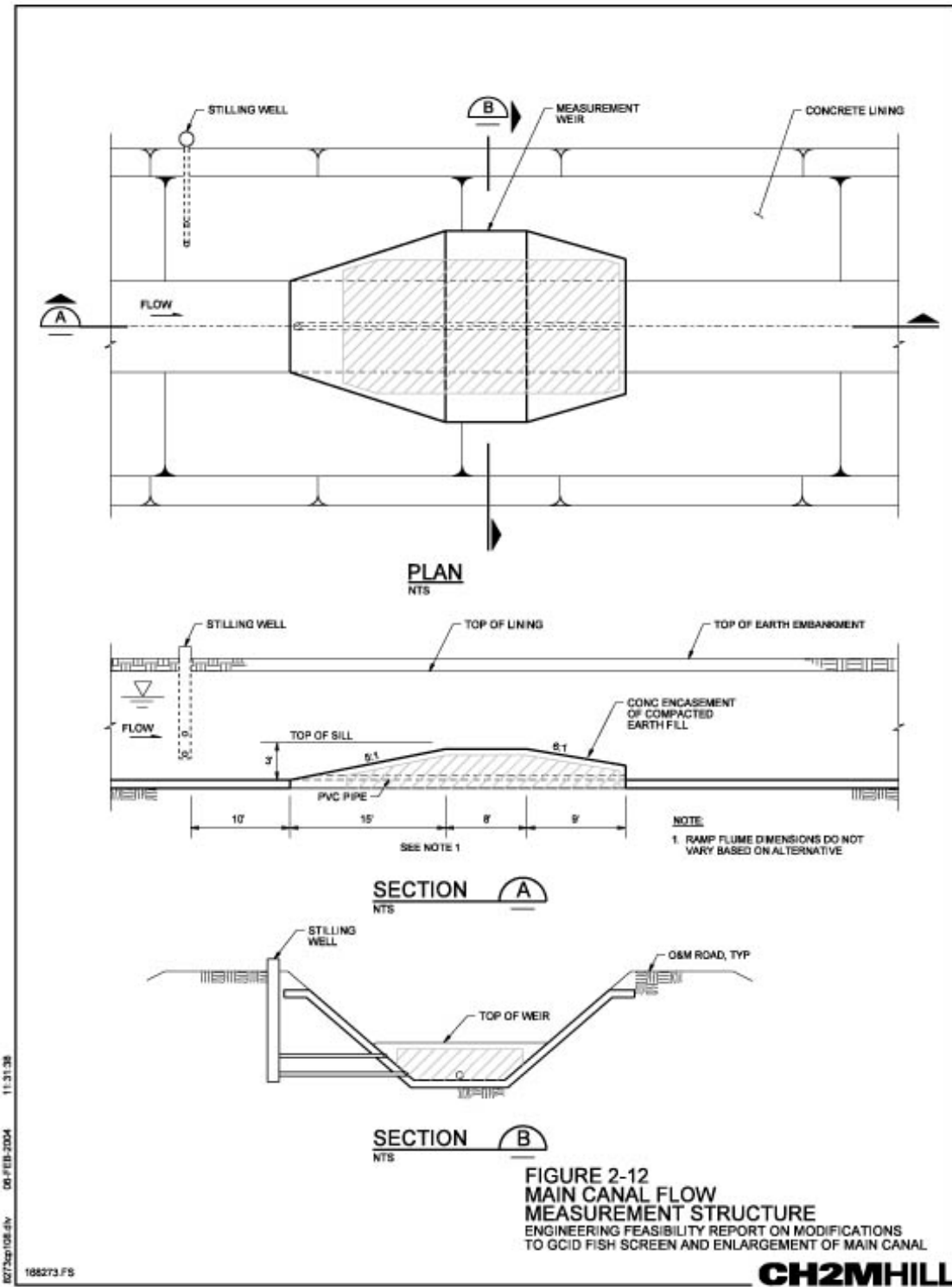


Figure B.3-5. GCID Canal Flow Measurement Structure

- 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.

Sites Reservoir

General

Figure B.3-6 presents the approximate reservoir area-capacity curve for the Sites Reservoir site. Reservoir capacities of 1.8 MAF and 1.3 MAF are currently under consideration at the site. Table B.3-2 summarizes reservoir parameters for each capacity.

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 1.8 MAF reservoir with WSE of 520 feet was based on 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 topographically low areas along the relatively steep ridges that form the eastern side of the reservoir; where seepage paths would be relatively short, 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.3-7), 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 so that the proposed size of Sites Reservoir would be technically feasible and not unduly expensive. For the larger, 1.8 MAF reservoir, rim grouting is still proposed to reduce seepage risk through narrow ridge areas, as

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described later in this section. Note that the total embankment quantities presented on Figure B.3-7 were preliminary assessments. Quantity calculations for the proposed dams described in the next section are less.

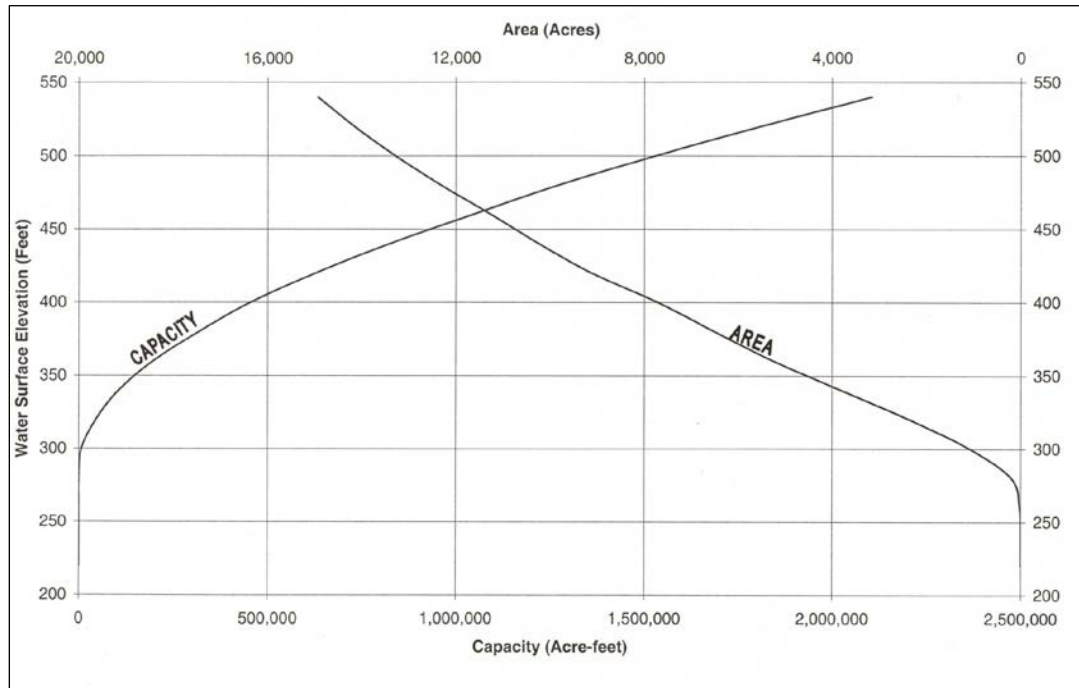


Figure B.3-6. Sites Reservoir – Area-Capacity Curve

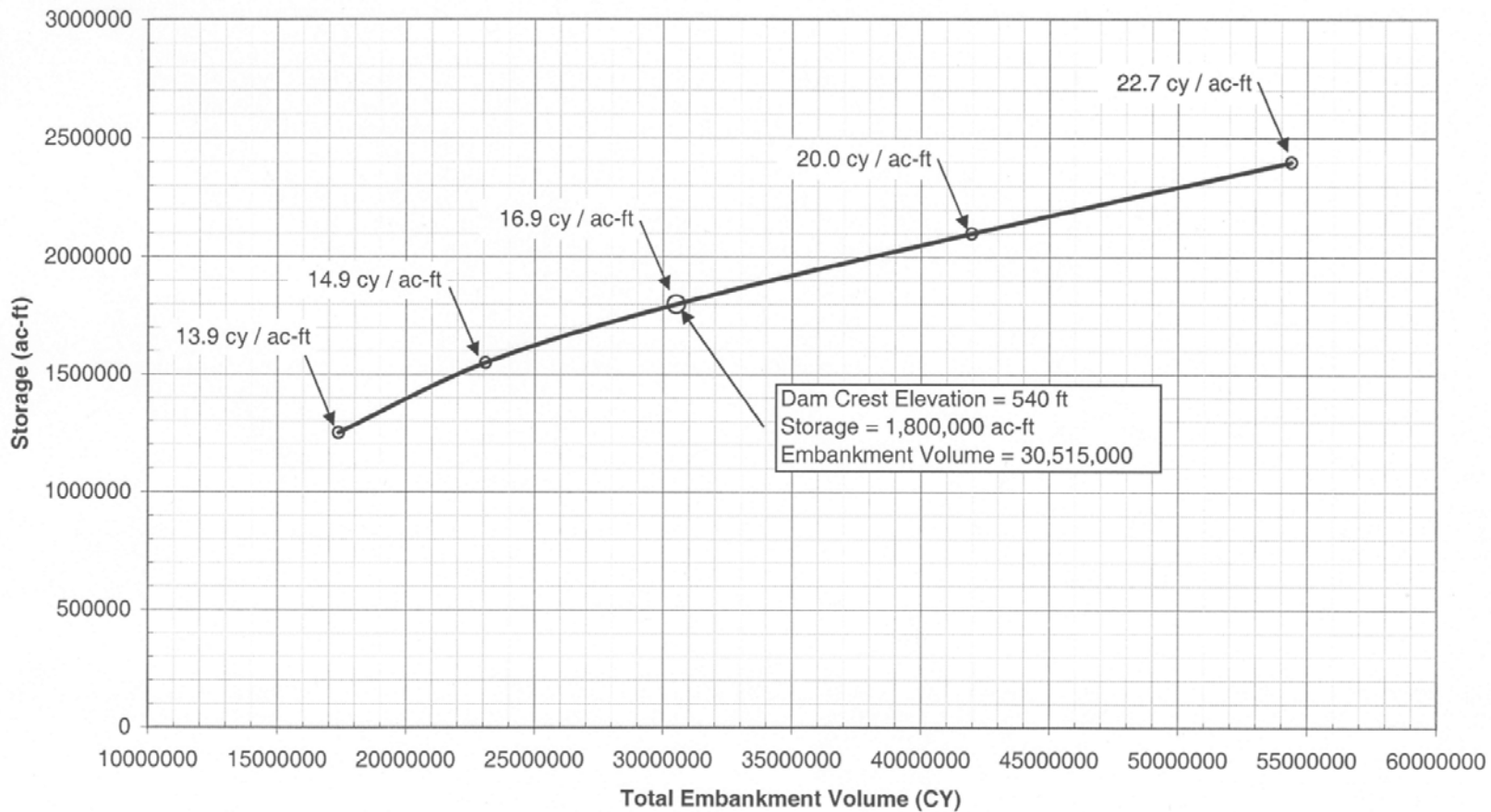
Table B.3-2. Sites Reservoir Sizes Under Consideration

	Larger Reservoir	Smaller Reservoir
Alternative	B, C, D	A
Nominal Size	1.8 MAF	1.3 MAF
Active Storage Capacity	1.60 MAF	1.08 MAF
Approximate Inundation Area	14,200 acres	12,400 acres
Dam/Saddle Dam Crest Elevation (Without Camber)	540 feet	500 feet
Maximum Operating Water Elevation	520 feet	480 feet
Minimum Operating Water Elevation	340 feet	340 feet
Top of Dead Pool	320 feet	320 feet

Key:

MAF = million acre-feet

Sites Reservoir Storage vs. Embankment Volume (Main Dams and Saddle Dams)



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.3-7. Sites Reservoir – Storage versus Embankment Volume

Sites Reservoir Dams

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 on the size of the reservoir. The main dams and saddle dams discussed in this report are all zoned earth and rockfill embankment dams.

Table B.3-3 and Table 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 1.8 MAF Sites Reservoir

Dam	Maximum Height Above Streambed (feet) ^a	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
Saddle Dam 10	Not Required for 1.8 MAF Reservoir		
Total			22,000,000

Notes:

^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.

Key:

MAF = million acre-feet

Table B.3-4. Required Dams Volumes For 1.3 MAF Sites Reservoir

Dam	Maximum Height Above Streambed (feet) ^a	Crest Length (feet)	Total Embankment Volume (cubic yards)
Golden Gate Dam (incorporating Saddle Dam 10) ^b	270	2,250	5,987,000
Sites Dam	250	850	2,853,000
Saddle Dam 1	10	490	1,400
Saddle Dam 2	40		
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 ^c	65	2,990	390,000
Saddle Dam 8b ^c	5	340	15,000
Saddle Dam 10 ^b	30	300	Included with Golden Gate Dam
Total			11,018,400

Notes:

^a Maximum height above streambed is measured from the downstream toe.^b Saddle Dam 10 is only required for 1.3 MAF Reservoir because of Golden Gate Dam relocation.^c Topography splits Saddle Dam 8 for 1.8 MAF Reservoir into two dams at same location for 1.3 MAF Reservoir.

Key:

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 and Alternatives

Available site investigations (drilling, trenching, geological mapping, and laboratory testing) and preliminary dam designs indicate that properly designed and zoned earth and rockfill dams with proper internal filter and drain systems are suitable for evaluating NODOS/Sites Reservoir Project alternatives. Future evaluations may show that other embankment types could be economically competitive. Roller Compacted Concrete and gravity type dams may not be suitable for use on the project for the main dams due to the quality of the foundation rock and potential for shears in the foundation.

Aggregate for Filters and Drains

Providing suitable aggregate for the filter and drain zones in the embankment dams is an important factor in estimating the cost of dam construction. Slightly weathered to fresh samples of the local Venado Sandstone tested for durability against concrete aggregate standards exhibit failing or marginally passing results. Given the size and importance of the dams, and recognizing that reliable filter and drain performance in the embankments over the life of the NODOS/Sites

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Reservoir Project is critical to dam safety, construction cost estimates assume that suitable aggregate would be imported from the old Stony Creek channel area, approximately 30 to 35 miles from the project site, to minimize cost estimating risk. Preliminary evaluations of potential quarry sites in the old Stony Creek channel area indicate suitable and sufficient material is available in adequate quantities for the project.

Even though the Venado Sandstone may not be suitable for use as internal filter and drain aggregate for dams or concrete aggregate for structures, it is considered suitable for use as rockfill in the shell zones of the dams, and as riprap slope protection. Fresh Venado Sandstone placed as riprap on the upstream slope of the existing Funks Dam has been performing satisfactorily since the dam was completed.

It is recommended that additional testing and evaluation of other sources for embankment filter and drain material, and for concrete aggregate, be performed during final design to determine if more economical sources can be identified closer to the project site. This should also include test quarries and additional laboratory testing of the Venado Sandstone to further evaluate its suitability as filter and drain material.

Sites Reservoir Water Diversion and Release Rates

During the winter and spring, when water is available for diversion, Sites Reservoir is supplied from the existing T-C Canal, from the existing GCID Canal through the TRR Pipeline, and from a new pump station on the Sacramento River through the Delevan Pipeline. Table B.3-5 summarizes the maximum diversion flows for each alternative.

Table B.3-5. Diversion and Release Rates

Conveyances	Diversion to Sites	Release from Sites
T-C Canal	2,100 cfs	2,100 cfs
GCID Canal (via TRR Pipeline)	1,800 cfs	900 cfs
Sacramento River (via Delevan Pipeline)	2,000 cfs	1,500 cfs

Key:

cfs = cubic feet per second

GCID = Glenn-Colusa Irrigation District

T-C = Tehama-Colusa

TRR = Terminal Regulating Reservoir

Golden Gate Dam

General

Past investigations by DWR examined alternative dam alignments for Golden Gate Dam with various maximum reservoir WSEs, and for four alternative alignments to impound Sites Reservoir using a maximum WSE of 520 feet. The DWR investigations included comparisons between the alternative alignments, based on site topography, abutment slopes and geology, required foundation excavation, availability of materials, presence of faults and shears, and constructability. Selected alignments are discussed in greater detail below.

Dam Alignment – 1.8 MAF Reservoir

Figure B.3-8 presents a plan view of the dam embankment for the 1.8 MAF reservoir for Alternatives B, C, and D (dam crest at elevation 540 feet). Golden Gate Dam would be on Funks Creek approximately 1 mile west of Funks Reservoir. Table B.3-3 provides crest length, maximum height above the streambed, and total embankment volume for the dam embankment.

Dam Alignment – 1.3 MAF Reservoir

Golden Gate Dam for the 1.3 MAF reservoir for Alternative A (dam crest elevation 500 feet) would be on Funks Creek approximately 2,000 feet upstream of the dam location for the 1.8 MAF reservoir, and on the western edge of the ridges that form the eastern reservoir rim. Table B.3-4 provides crest length, maximum height above the streambed, and total embankment volume for the dam embankment. Golden Gate Dam at this more upstream location would require a separate saddle dam (identified as Saddle Dam 10) in a low saddle area just south of the main dam. This saddle is on the reservoir side of the larger dam for the 1.8 MAF reservoir due to the location of Golden Gate Dam further downstream on the creek.

Dam Section

Figure B.3-9 provides a typical cross section for Golden Gate Dam that would be constructed for both the 1.3 MAF and 1.8 MAF reservoirs. The proposed embankment section is a zoned earth and rockfill embankment with a central impervious core and exterior rockfill shells. The dam would have a crest width of 30 feet. The minimum freeboard (without camber) would be 20 feet, measured above the maximum reservoir storage elevation. Nominal crest elevations would then be 500.0 feet for the 1.3 MAF reservoir and 540.0 feet for the 1.8 MAF reservoir.

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 types of dam embankments, the upstream and downstream slopes of the central core were selected to be 0.5H:1V, which included physical and strength properties developed from available geotechnical and laboratory test data. Safety factors were estimated for the higher dam (1.8 MAF reservoir) for full and partial pool conditions with seepage, and for a rapid drawdown condition. Predicted safety factors were found to exceed customary allowable safety factors for large dams found in Federal and State standards. Seismic performance of the dam was also evaluated using the Newmark and Makdisi-Seed Sliding Block approaches to estimate deformations for a range of possible ground accelerations generated by an M 7 event on the Great Valley fault. Deformations are estimated to be less than 2 feet. Considering the dam height and 20 feet of freeboard without camber, these displacements are considered acceptable.

Seismic induced reservoir seiches were also evaluated. Wave heights would be significantly less than the reservoir freeboard at full pool even allowing for crest deformations.

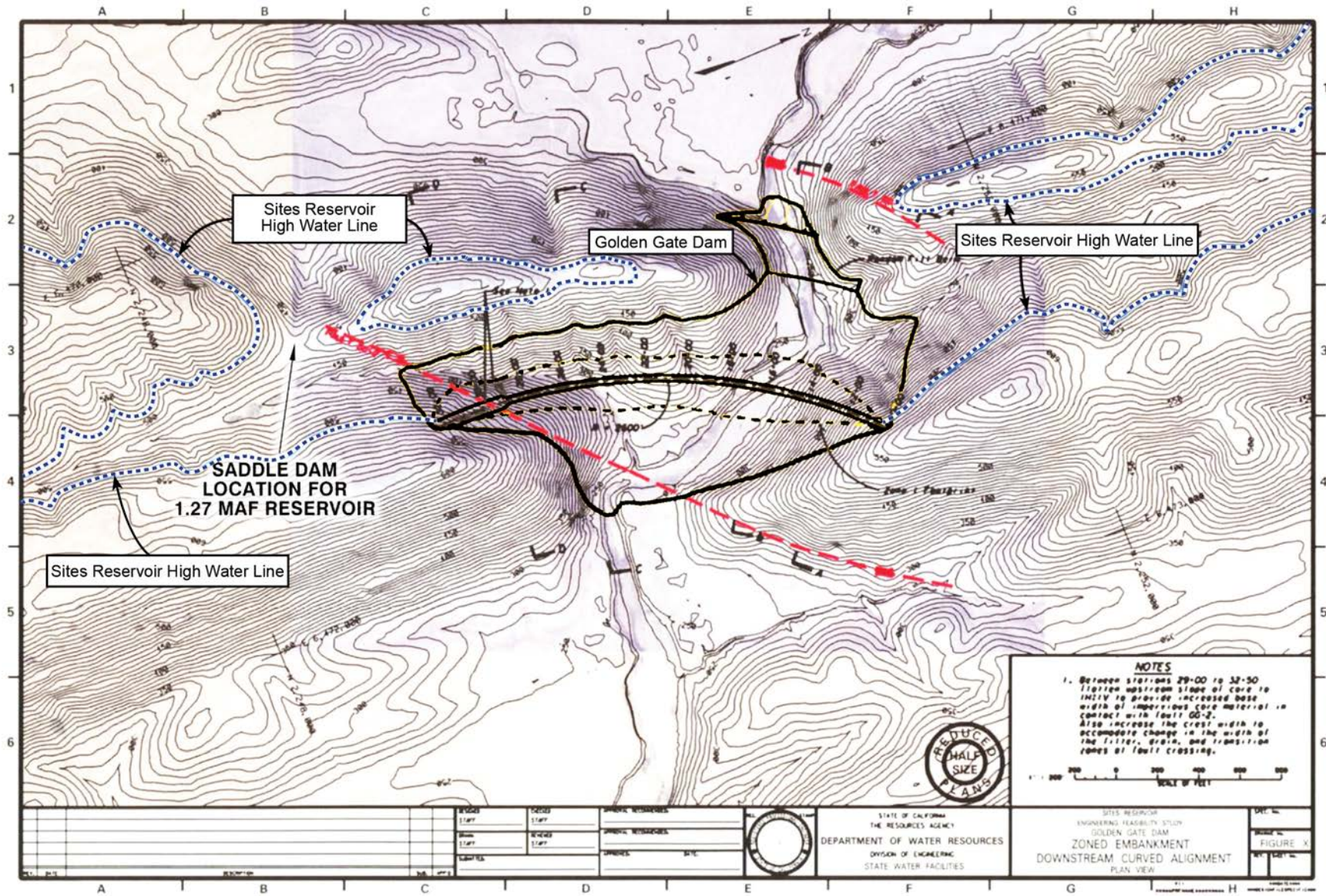


Figure B.3-8. Golden Gate Dam – Zoned Embankment – Downstream Curved Alignment – Plan View

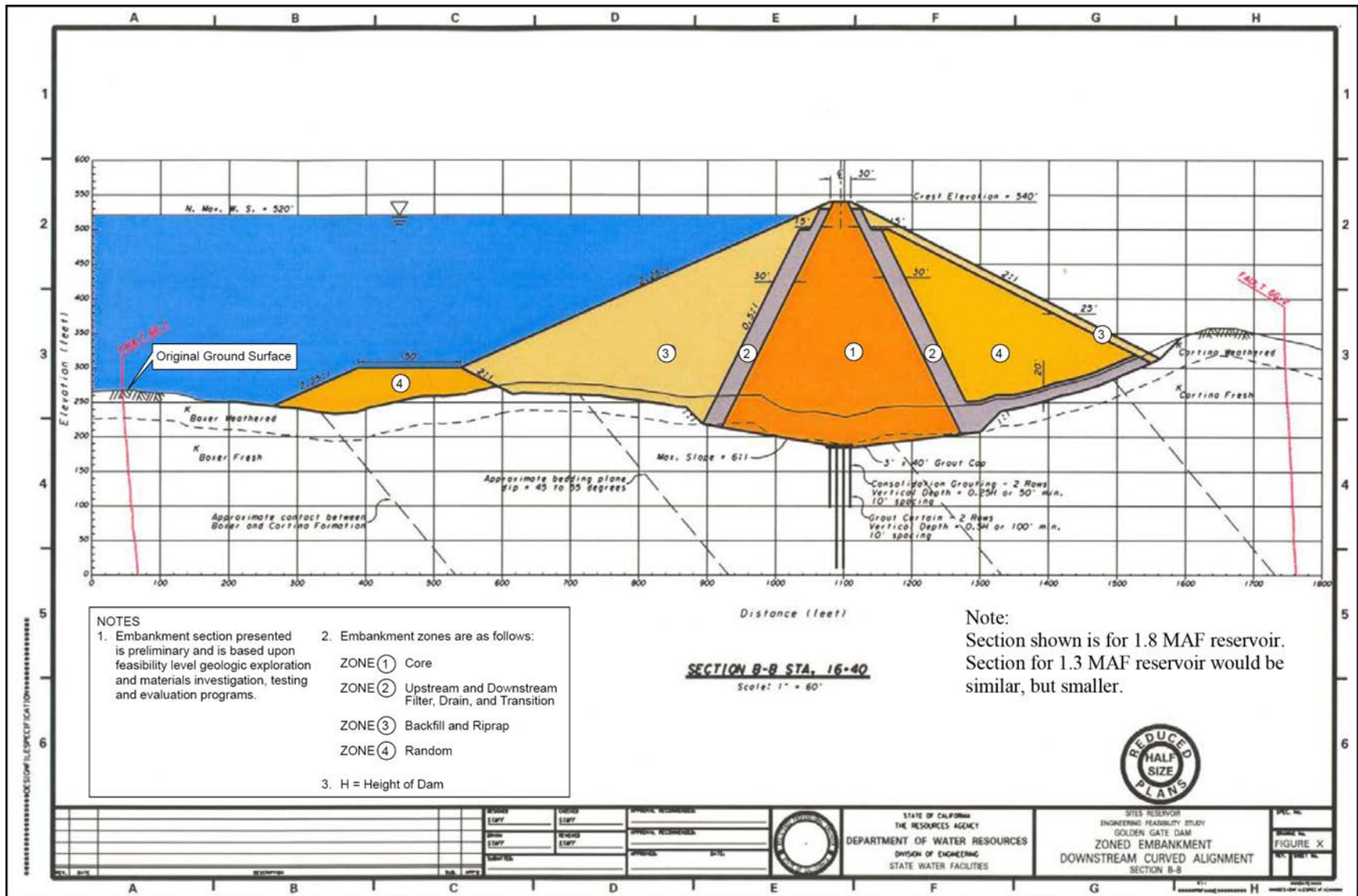


Figure B.3-9. Golden Gate Dam – Zoned Embankment – Downstream Curved Alignment – Maximum Section

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Shears are identified within the limits of the embankment footprint for both reservoir sizes (GG-1 and GG-2). The proposed zoning, including the filter and drain systems, is considered adequate to safely accommodate any movement on the shears that could occur during a seismic event in the region. As part of the final design, it is recommended that the zoning and slopes for the central core section be further evaluated to confirm the adequacy of the proposed design.

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, drain, and transition materials is included for filter compatibility between embankment materials, to provide control of embankment seepage, and to prevent piping of the core material. The 30-foot-width of filter, drain, and transition materials was selected to provide for constructability of this multi-element zone. The downstream embankment section also incorporates a 20-foot-thick blanket drain, composed 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 used 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 (Zone 4) 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.

Embankment Materials

Selection of the Golden Gate Dam embankment section was based on 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. Designations apply to the dam for both the large and small reservoir sizes.

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 on the floor of the reservoir upstream of the dam. Haul distances would be less than 1 mile. Impervious material processing beyond normal diskings and moisture conditioning in the designated borrow areas is not anticipated. Suitable materials can also come from other mandatory facility excavations (e.g., Holthouse Reservoir or Holthouse Channel) depending on schedule and economics of hauling.

Zone 2: Filter, drain, and transition materials consisting of suitable fresh rock or alluvial materials processed to various sizes to meet filter compatibility and hydraulic conductivity

requirements. To minimize estimating risk, it is assumed that Fresh Venado Sandstone of the Cortina Formation would not be suitable for use, and these embankment materials would be imported from the closest currently known off-site sand and gravel source. This source is an old, abandoned channel on Stony Creek between Orland and Willows, approximately 30 to 35 road miles from the Golden Gate Dam site.

Zone 3: Shell material consisting of processed clean rockfill with a maximum rock size of 30 inches. The shell material would be obtained from fresh Venado Sandstone of the Cortina Formation from one or more quarries developed in the eastern ridge of the reservoir near the dam site. Haul distances (one way) could be up to 1 mile. Quarry operations would require drilling and blasting with selective processing to remove mudstones, weathered sandstone, and other unsuitable materials to produce fresh Venado Sandstone with the required gradation. Suitable materials can also come from mandatory facility excavations, including the dam foundation excavation.

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 typically not be required except to remove oversize material.

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 in the anticipated grouting depth range were characterized as impermeable. However, some intervals of higher water take occurred in 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).

Because 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 were included in the dam design to reduce seepage through the dam foundation. The grout program would consist of a two-row grout curtain with one row of consolidation holes upstream and one row downstream of the curtain holes. The rows would parallel the dam centerline and be spaced

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10 feet apart. In addition, a 40-foot-wide by 3-foot-thick concrete grout cap was included to prevent surface leakage of grout during grouting of the upper stage.

Each row of consolidation and curtain grout holes would consist of mandatory primary and secondary holes spaced at 10-foot centers. In addition, it was assumed that tertiary holes (between the primary and secondary holes) would 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, 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, with a minimum depth of 100 feet. Grout injection volumes for 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 would 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 would be conducted to a depth equivalent to the curtain grout hole depth at the verification hole location.

In addition to the grouting program described above, treatment of special features such as the faults and shears encountered in the foundation excavation would also be required. Trenching, concrete backfilling and stitch grouting would be employed along significant faults and shears that cross the core, filter, and drain zones. The use of slush grouting will also be needed in areas of the core, filter, and drain zone foundations to fill small sheared zones and bedding plane joints that are open a small amount. The cost estimate for the dam includes an allowance for unlisted items that would include treatment of special features. The need for special treatments should be reexamined as part of the final design, after additional foundation investigation information becomes available.

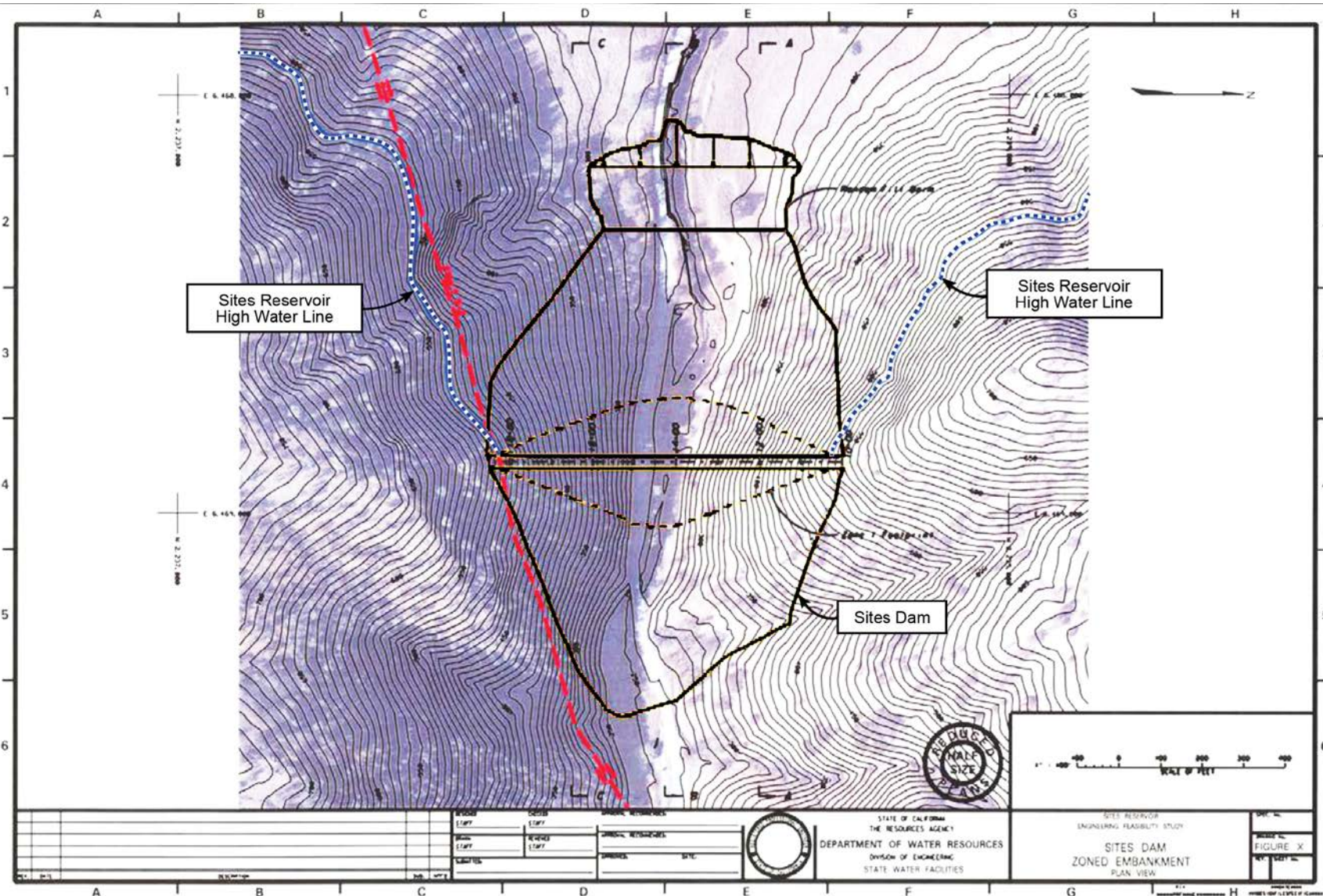
Sites Dam

General

Past investigations by DWR examined alternative dam alignments for Sites Dam to impound Sites Reservoir using a maximum WSE of 520.0 feet. The investigations included comparisons between the alternative alignments based on site topography, abutment slopes and geology, required foundation excavation, quantities of material needed and availability of materials, presence of faults and shears, and constructability. Selected alignments are discussed in greater detail below.

Dam Alignment – 1.8 MAF Reservoir

Figure B.3-10 presents a plan view of the dam embankment for the 1.8 MAF reservoir for Alternatives B, C, and D. Sites Dam would be on Stone Corral Creek approximately 0.25 mile east of the town of Sites, and 8 miles west of the town of Maxwell. Table B.3-3 provides crest length, maximum height above the streambed, and total embankment volume for the dam embankment.



Note: Plan shown is for 1.8 MAF Reservoir. Plan for 1.3 MAF Reservoir would be similar.

Figure B.3-10. Sites Dam – Zoned Embankment – Plan View

Dam Alignment – 1.3 MAF Reservoir

Sites Dam for the 1.3 MAF reservoir alternative would be at the same location selected for the 1.8 MAF reservoir, but the crest elevation would be lower. Table B.3-4 provides crest length, maximum height above the streambed, and total embankment volume for the dam embankment.

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.3-11) for both reservoir size alternatives, with the following exception:

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 used to divert Stone Corral Creek from the dam footprint. Because random materials generated from foundation excavation upstream of the dam centerline would be composed 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 within the main embankment section.

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, and 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.

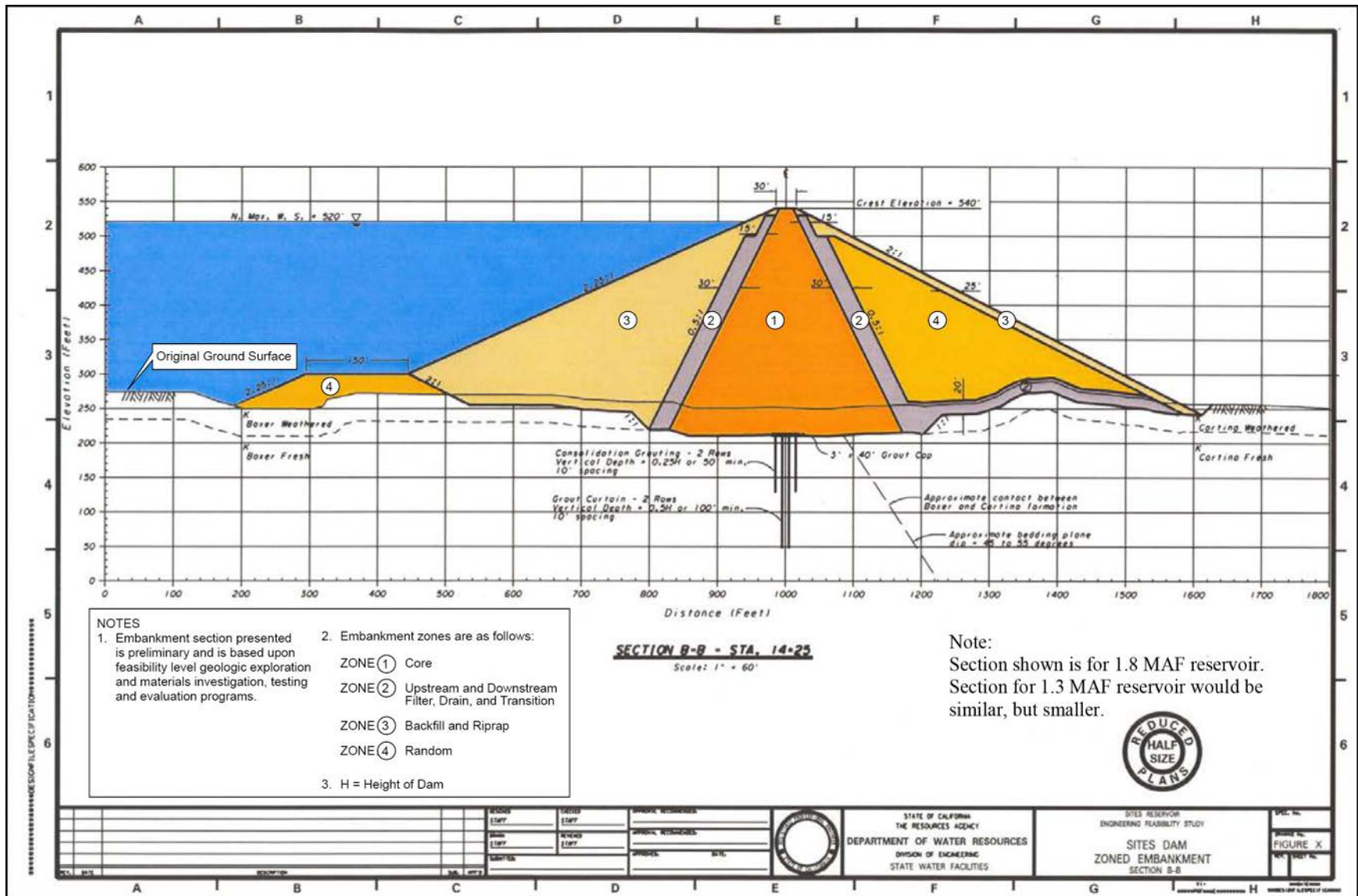


Figure B.3-11. Sites Dam – Zoned Embankment – Maximum Section

Grouting

A review of water pressure test data from Reclamation and DWR 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 table 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 indicate that some areas of higher hydraulic conductivity occur in the dam foundation, consolidation and curtain grouting were 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 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 would likely be required as described for Golden Gate Dam. The cost estimate for the dam includes an allowance for unlisted items that would include treatment of special features (shears, ledges, formation contacts and the like). The need for special treatments should be reexamined as part of final design after additional foundation investigation information becomes available.

Saddle Dams

General

This discussion is generally applicable to the saddle dams required for the 1.8 MAF reservoir for Alternatives B, C, and D, and 1.3 MAF reservoirs for Alternative A. Figure B.3-12 presents a plan view of the dams. Saddle dams are needed at topographic saddle low points along the eastern ridge of the reservoir from Funks Creek north to the northern end of the reservoir.

Table B.3-3 and Table B.3-4 provide volume for the four alternatives. For the 1.8 MAF reservoir, nine dams would be required. The dams are numbered one through nine from south to north, as shown on Figure B.3-12. Because of the lower design maximum water level for the 1.3 MAF reservoir, only six of the nine saddle dams would be required along the ridge north of Golden Gate Dam (note that Saddle Dams 8a and 8b are considered to be one dam). As mentioned previously, there is also one additional small saddle dam required for the 1.3 MAF reservoir in a saddle just south of Golden Gate Dam. This additional saddle dam is needed because Golden Gate Dam is further north on Funks Creek for the smaller reservoir, and the relocated embankment footprint does not encompass the saddle area as it does for the larger reservoir alternative.

For the 1.8 MAF reservoir for Alternatives B, C, and D, 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 large dams. Of the large dams, Saddle Dams 3, 5, and 8 are the tallest, with the largest volumes of material required.

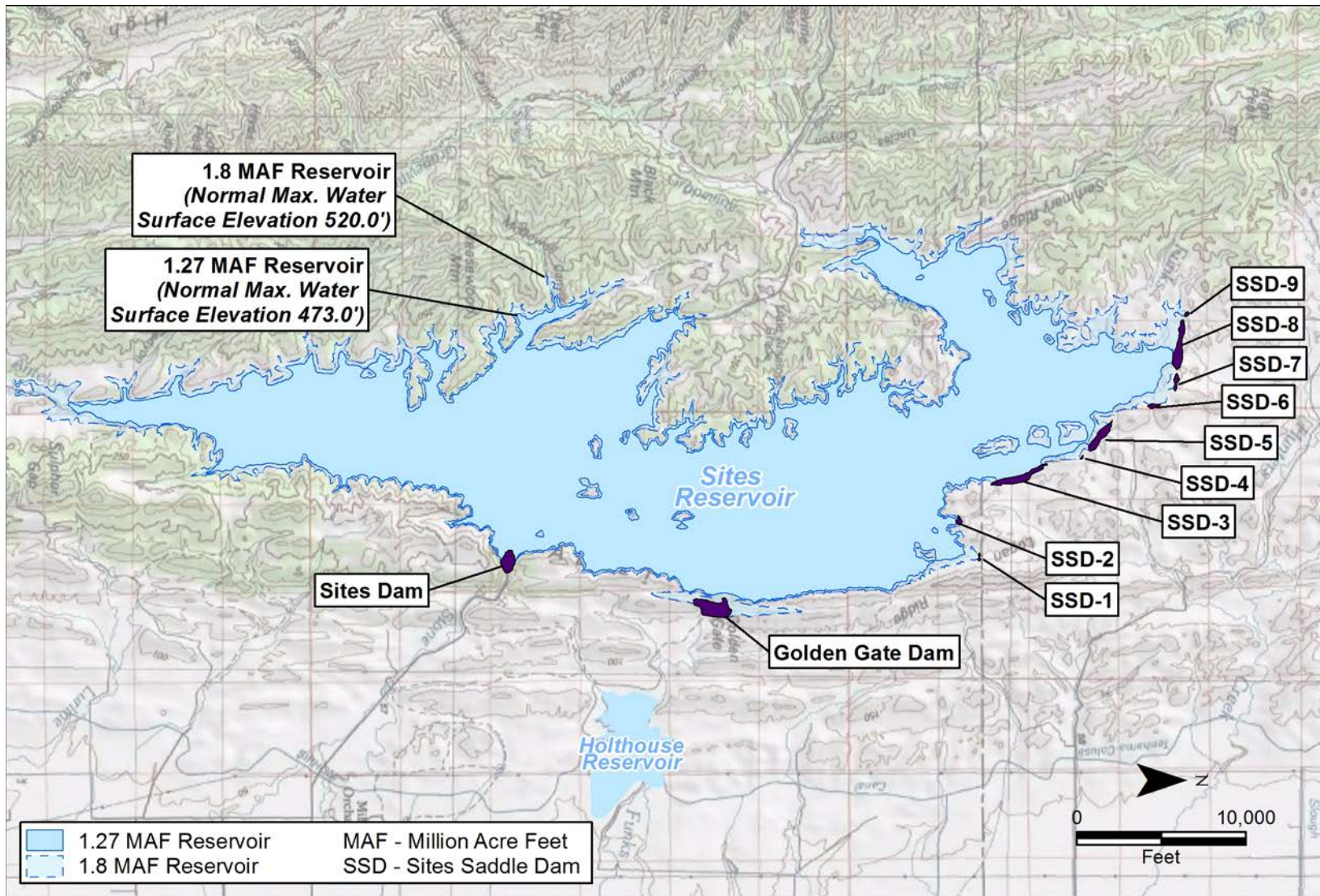


Figure B.3-12. Saddle Dam Location Map

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For the 1.3 MAF reservoir for Alternatives A, Saddle Dams 1, 2, 6, and 10 are generally characterized as small-sized dams. Saddle Dams 3, 5, and 8 (8a) are generally characterized as large dams. They are also the tallest, with the largest volumes of material required.

Dam Alignments

The topography at the northern end of the proposed Sites Reservoir is such that the preferred alignments for the saddle dams are along the relatively broad ridge between the Funks Creek and Hunters Creek watersheds. In general, saddle dam alignments were selected to cross low areas between the topographic high points to minimize length and height, and avoid creating dead storage. Saddle dam alignment selection also considered the presence of foundation defects, such as faults and shears, where data were available.

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 large-saddle dams. Note that the proposed sections would apply to the 1.3 MAF and 1.8 MAF reservoir alternatives.

Upstream and downstream slopes for all saddle dams would be 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 the required embankment stability during drawdown of the reservoir.

Figure B.3-13 shows the proposed dam embankment section for saddle dams classified as small. The dam embankment is a zoned earthfill consisting of a central impervious core with flanking upstream and downstream zones of random shell material composed predominately of mudstone. Downstream of the core, a 10-foot-wide zone of filter material is conservatively included to prevent piping of the core materials. The downstream embankment section also incorporates a 3-foot-thick blanket drain on the foundation surface to control 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. Slope protection would be provided by grass seeding. Consistent with typical designs for similar types of dam embankments, the upstream slope of the core was

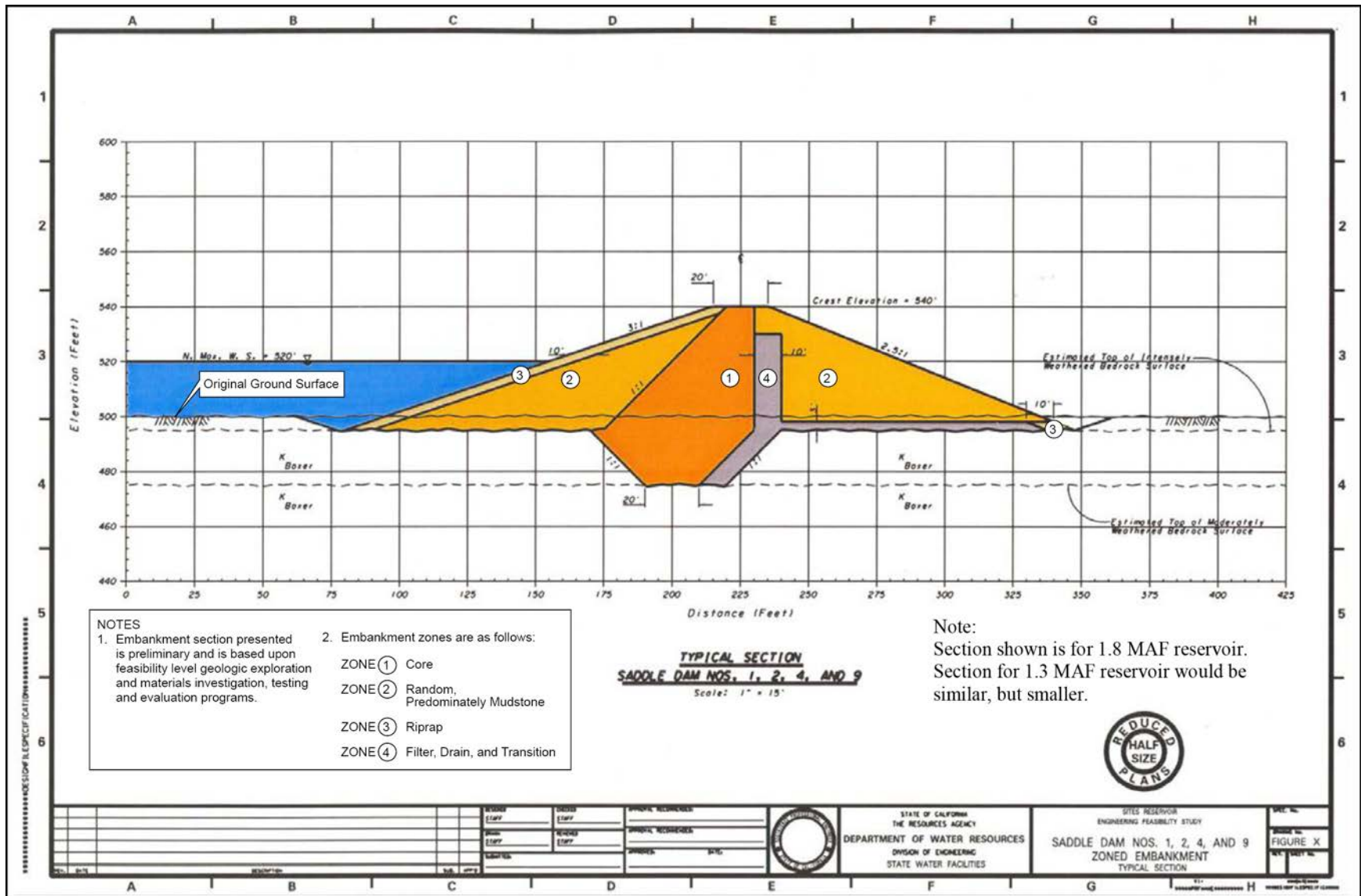


Figure B.3-13. Sites Reservoir Saddle Dams 1, 2, 4, and 9 – Zoned Embankment – Typical Section

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selected to be 1H:1V, and the downstream slope is vertical to simplify construction of the adjacent filter zone.

Figure B.3-14 shows the proposed dam embankment section for saddle dams classified as large. 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 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, composed 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 composed 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 is not anticipated.

Zone 2: Random shell material comprised predominantly 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 eastern 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 material 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

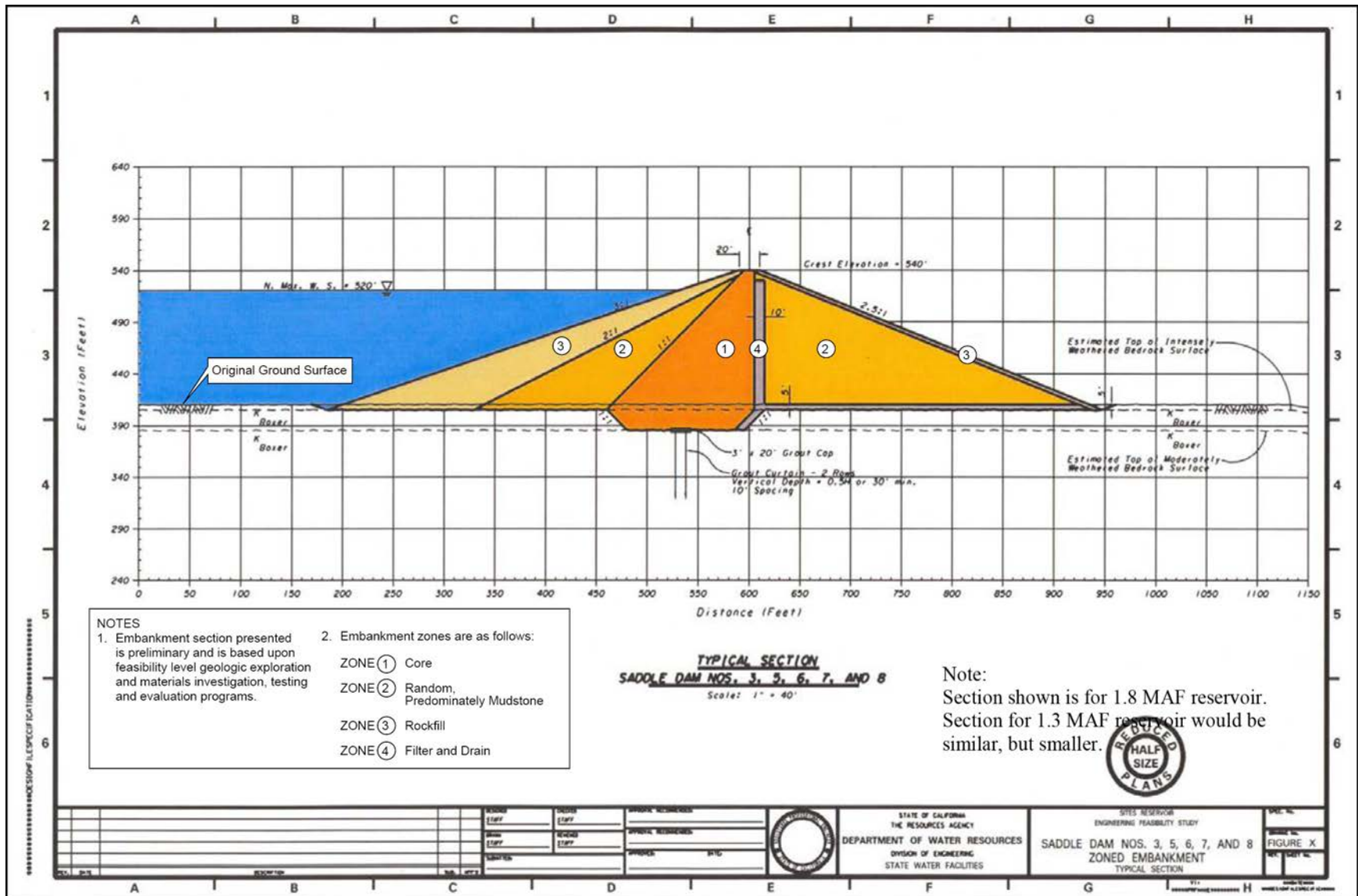


Figure B.3-14. Saddle Dams 3, 5, 6, 7, and 8 – Zoned Embankment – Typical Section

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imported from the old, abandoned channel on Stony Creek between Orland and Willows, approximately 30 to 35 road miles from the saddle dam sites.

Foundations

Bedrock underlying the saddle dam footprints is predominantly 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 on 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 provide for a competent impervious barrier was 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 some of the 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.

Sites Reservoir Emergency 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 after it has reached the maximum design pool plus PMF storage elevation.

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 3-day volume estimated at 78,420 AF.

For the 1.8 MAF reservoir, storing the PMF-estimated volume of 78,420 AF above normal maximum pool at an elevation of 520 feet would only raise the water level by approximately 5.5 feet to an elevation of 525.5 feet. Placing the invert of the emergency spillway inlet at elevation 526.0 feet 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 1.3 MAF reservoir, storing the PMF-estimated volume of 78,420 AF above normal maximum pool at an elevation of 480 feet would only raise the water level by approximately 6.25 feet to an elevation of 486.25 feet. Placing the invert of the emergency spillway inlet at elevation 487.0 feet 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 on inspection and maintenance considerations, not hydraulic requirements.

For the 1.8 MAF reservoir, a morning glory spillway with crest at elevation 526 feet would be provided on a cut bench on the left abutment of the dam, as shown on Figure B.3-15. The 7-foot-diameter outlet pipe for this spillway 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 an unnamed 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 saddle dam. The spillway pipe and energy-dissipating structure details would be coordinated with existing or provided roads in the area.

For the 1.3 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.3-16.