CHAPTER 3 DESIGN CONSIDERATIONS

3.1 General

This chapter describes the engineering aspects of the alternative features currently under consideration for the proposed NODOS Project. This section includes a general summary of the Plan Formulation Report (PFR), and the preliminary engineering reports prepared by DWR and its consultants, and the Technical Memoranda (TM) developed post-PFR. The three project alternatives being considered are presented in Table 3-1.

Table 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			
T-C Canal	2,100 cfs	2,100 cfs	2,100 cfs
GCID Canal	1,800 cfs	1,800 cfs	1,800 cfs
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 = Glenn-Colusa Irrigation District
MAF = million-acre-feet
T-C = Tehama-Colusa

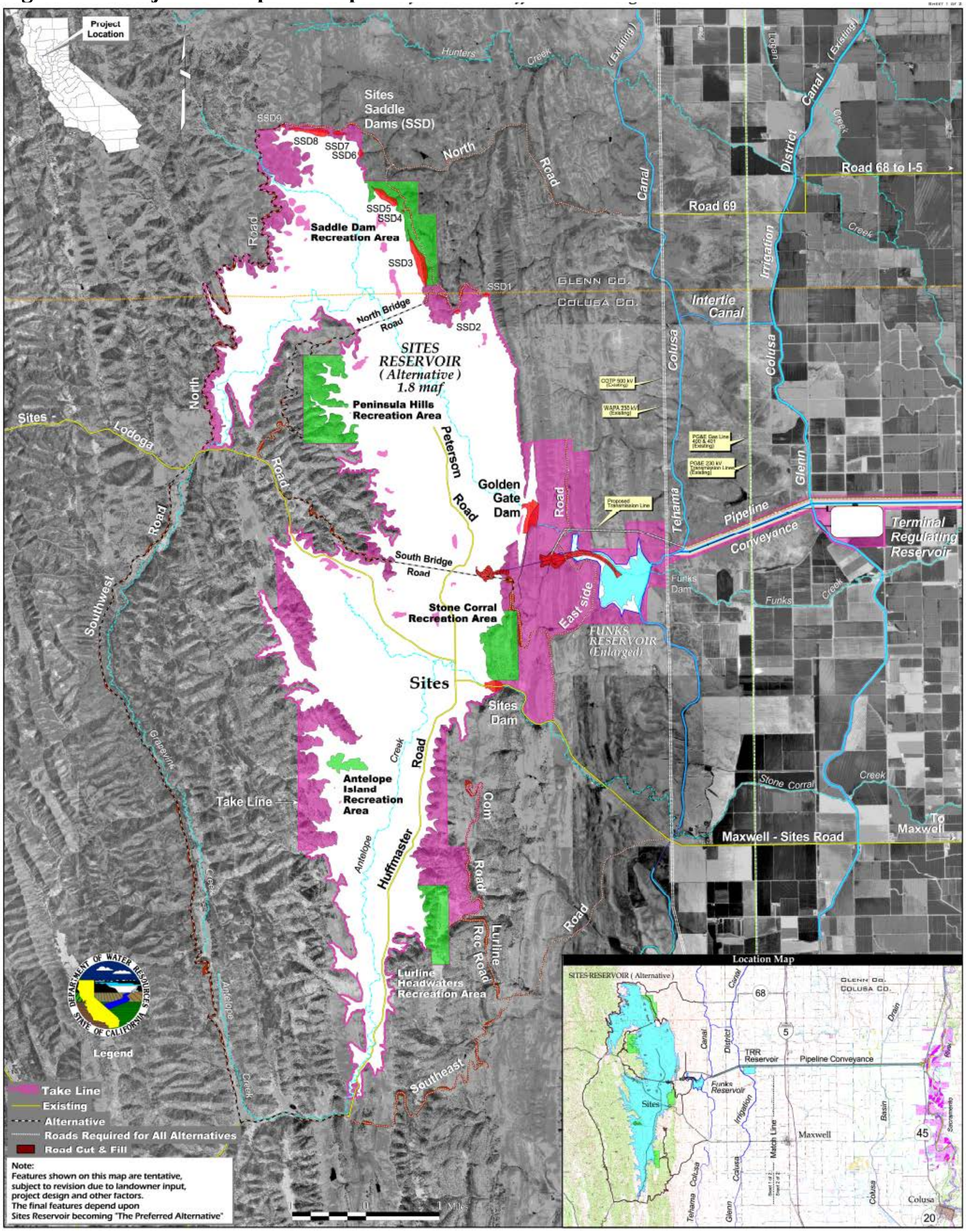
Figure 1-1 in Chapter 1 shows a schematic diagram of the potential NODOS Project components and direction of flow. Figure 3-1 is a map of the NODOS Project facilities.

Alternative A and Alternative C would fill Sites Reservoir with water from the Sacramento River through the existing T-C Canal, GCID Canal, and a new pipeline (Delevan Pipeline) during the winter and spring when water is available for diversion. The Delevan Pipeline Intake Facilities include the Sacramento River Pumping and Generating Plant (SRPGP) and associated fish screen facility. Releases from Sites Reservoir to the T-C 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 diversion through the new Delevan Pipeline. In this alternative, Sites Reservoir would be filled using the existing T-C 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 Delevan Pipeline Discharge Facility (Includes outlet energy dissipating structure) for Alternative B.

Figure 3-1. Project Description Map



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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 SRPGP. 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 would be a common feature for all project alternatives that serves as a forebay and afterbay pond for the SRPGP. 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 would 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 SRPGP 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 pumped 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 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 SRPGP 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. The inlet/outlet works encompasses everything between the inlet tower and the back side of the sites Pumping/Generating Plant, including the tunnel and penstock piping. Releases could 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 county roads (Maxwell Sites Road, Sites 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 T-C Canal, and the Delevan Pipeline as the preferred conveyances for water diverted 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 T-C Canal below Orland. Neither was recommended for further consideration, based on their apparent inefficiency.

3.2 Red Bluff Diversion Dam Fish Passage Improvement Project

3.2.1 General

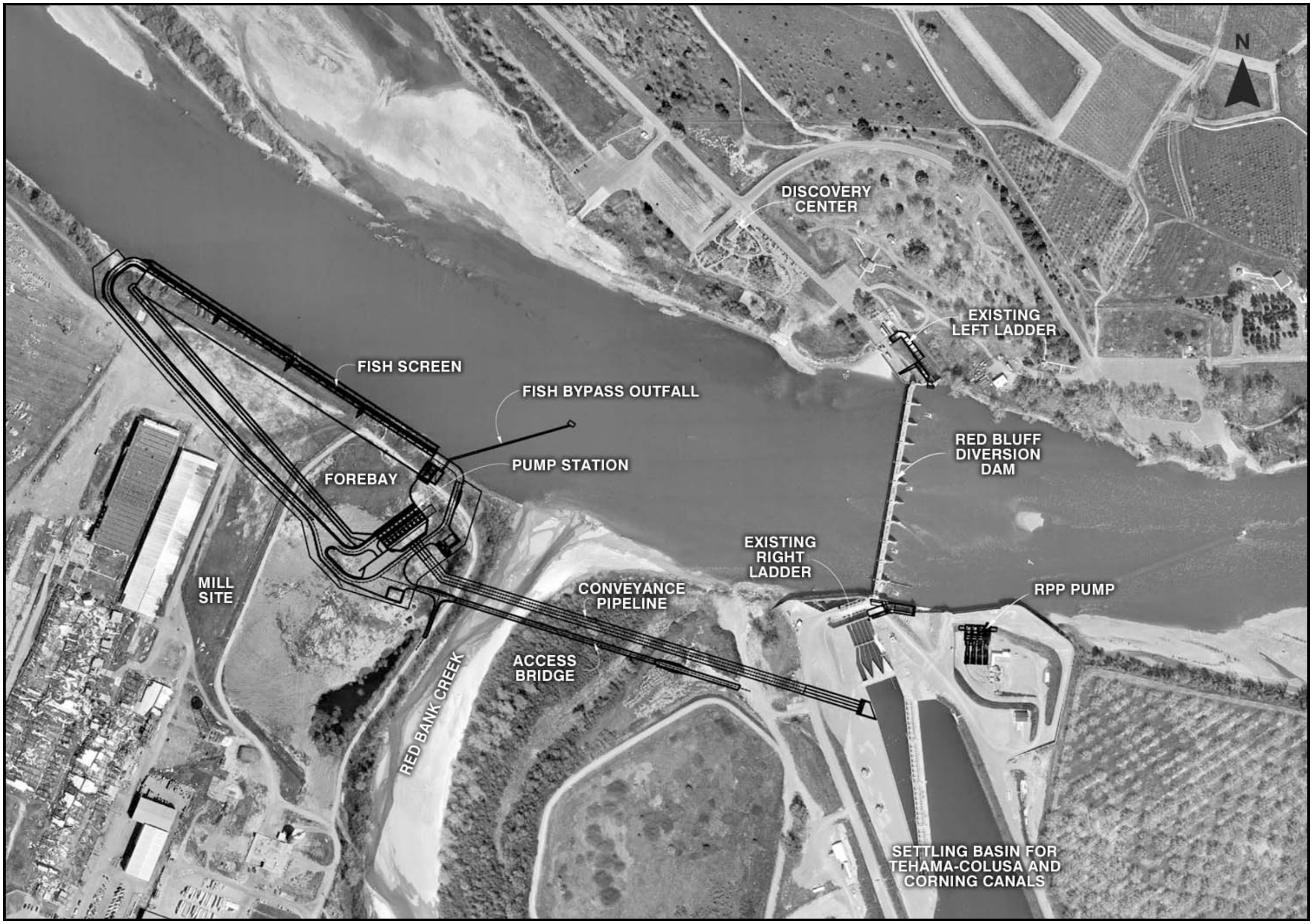
Construction for the TCCA Fish Passage Improvement Project at the Red Bluff Diversion dam (RBDD) was completed in 2012. The project (shown in schematic form on Figure 3-2) includes a fish screen facility, a pumping plant at the Mill Site (known as the Red Bluff Pumping Plant), canal, siphon, a forebay, switchyard, and a bridge across Red Bank Creek. The new pumping plan accommodates up to 11 pumps providing a total capacity of 2,500 cfs.

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

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 vertical axial-flow pump in one of the spare pump bays at the Red Bluff Pumping Plant to bring the total installed capacity of the plant to 2,250 cfs. The NODOS Project scope is limited to the additional pump's installation and operation impacts and maintenance responsibilities.

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



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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:

- Installation of a new butterfly valve. 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.

- Installation of a new pump. 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 bay and removal 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.

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

- Installation of 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.

- Installation of electrical conductors and a 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 T-C 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 T-C Canal. The difference between the installed capacity (2,250 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.

3.3 Sites Reservoir

3.3.1 General

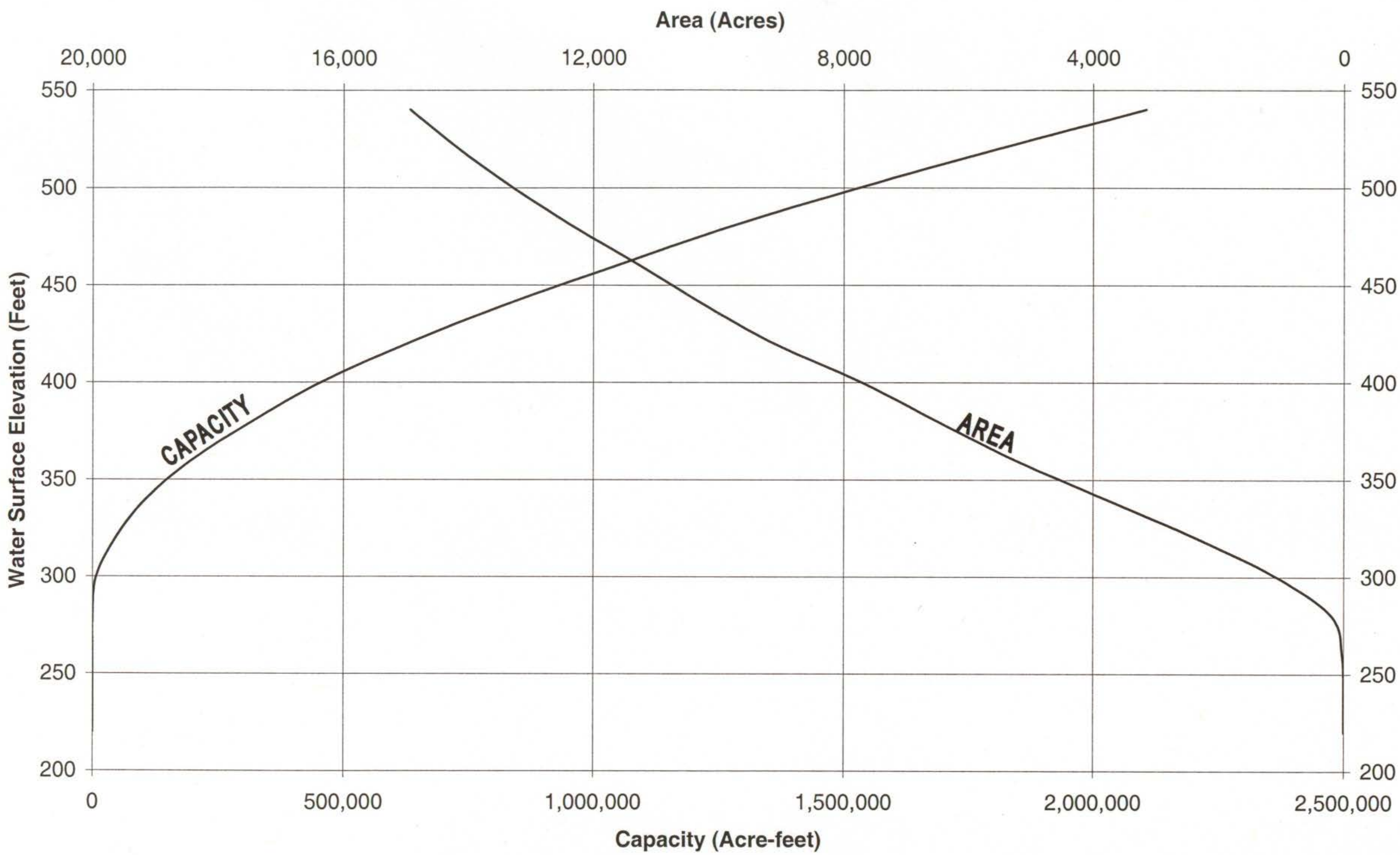
Figure 3-3 presents the reservoir area-capacity curve for the Sites Reservoir site. Reservoir capacities of 1.81 MAF and 1.27 MAF are currently under consideration at the site. Table 3-2 summarizes reservoir parameters for each capacity.

Table 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
Approximate 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 3-3. Sites Reservoir – Area-Capacity Curve



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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 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.

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 be 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 3-3 and 3-4 present a summary of dam characteristics required to impound Sites Reservoir for the two reservoir sizes currently under consideration.

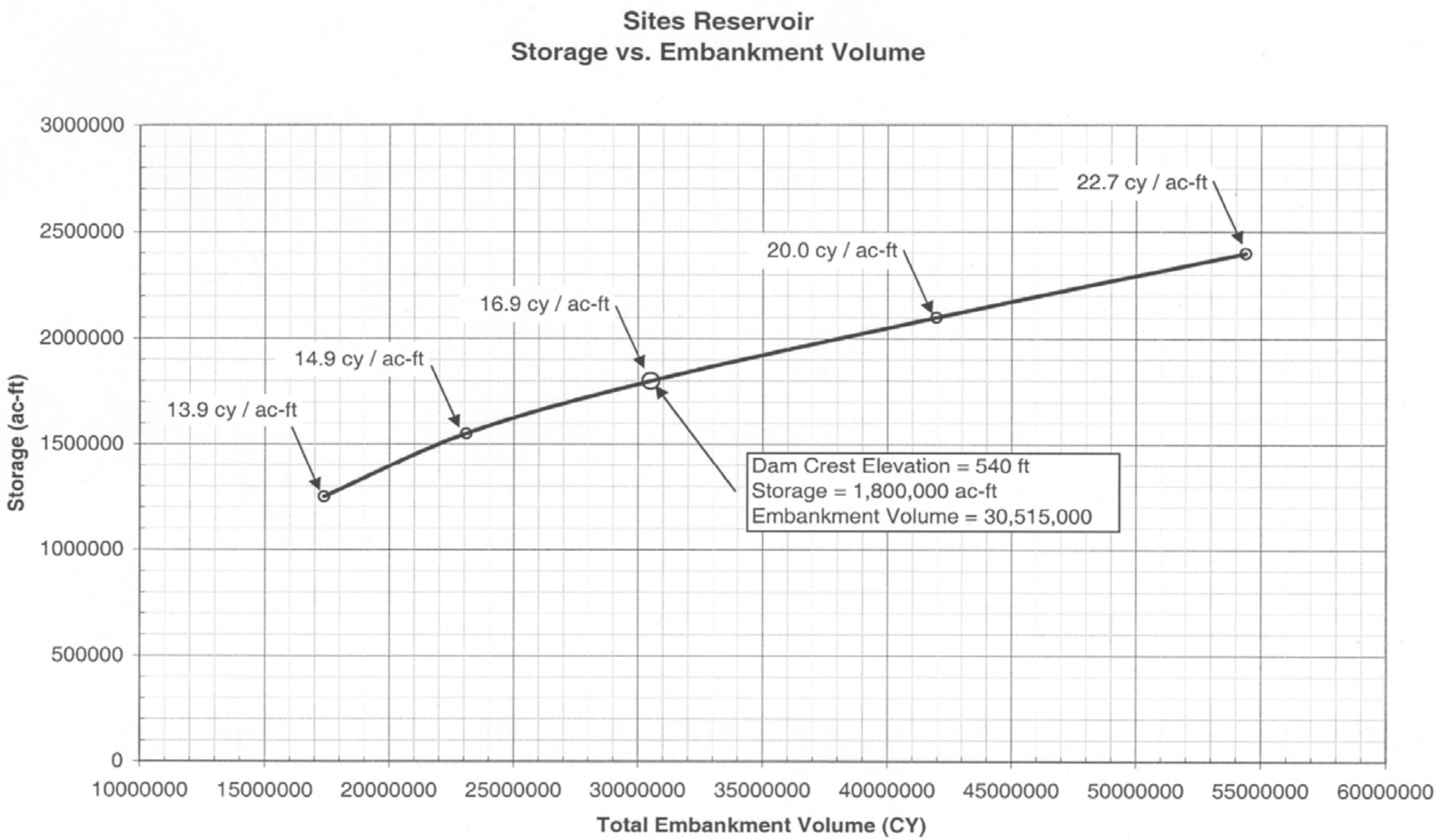
Table 3-3. Required Dams Volumes For 1.81 MAF Sites Reservoir

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

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



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.

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Table 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 (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 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.
[b] Saddle Dam 10 is required for 1.27 MAF Reservoir only.

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.

3.3.4 Golden Gate Dam

General

Past investigations have examined alternative dam alignments for Golden Gate Dam with various maximum reservoir WSEs, and 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

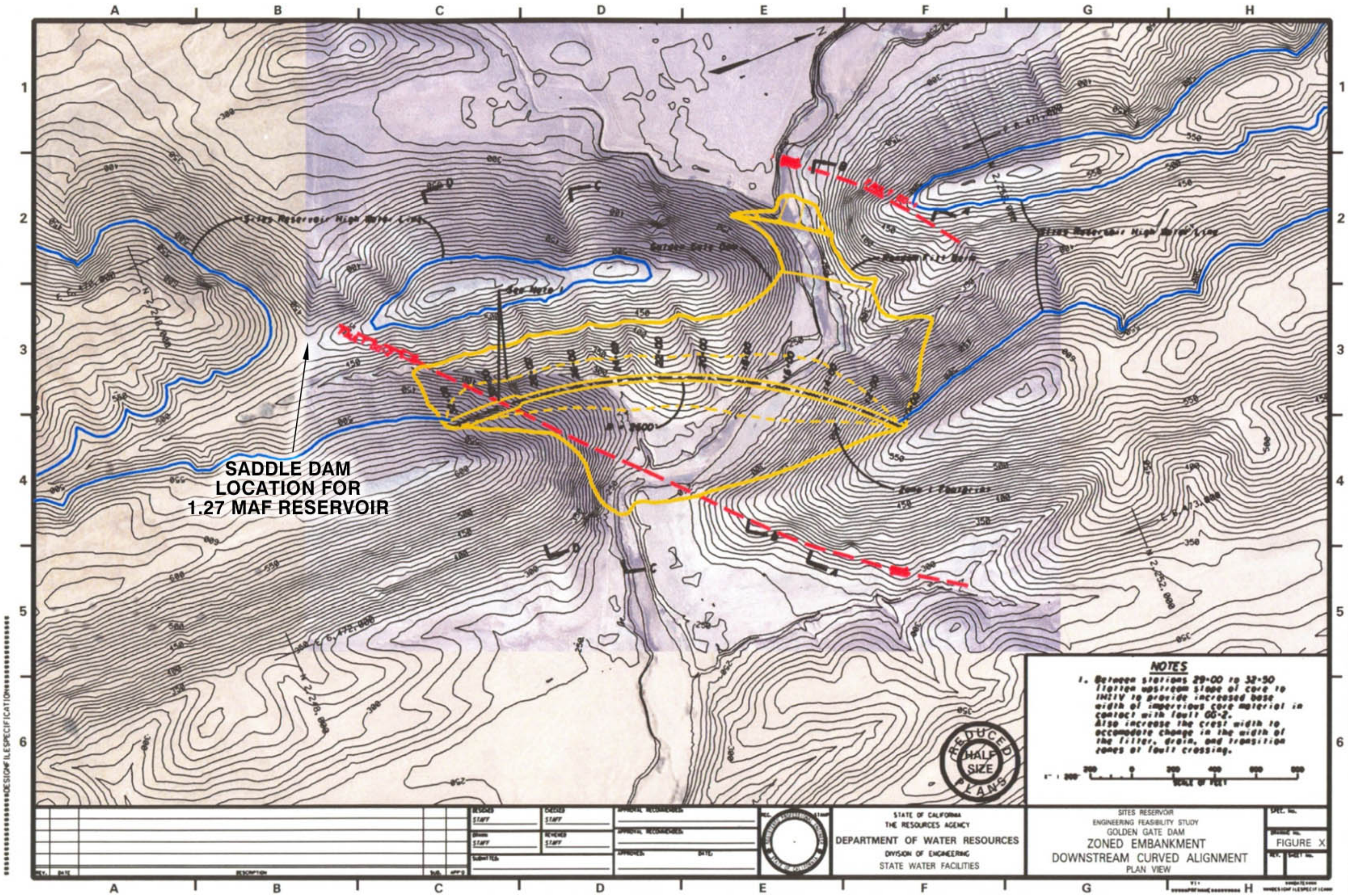
Dam Alignment – 1.81 MAF Reservoir

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 would have 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 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 would have 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 Golden Gate Saddle Dam) 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 3-5. Golden Gate Dam – Zoned Embankment – Downstream Curved Alignment – Plan View



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PRELIMINARY- SUBJECT TO CHANGE

Dam Section

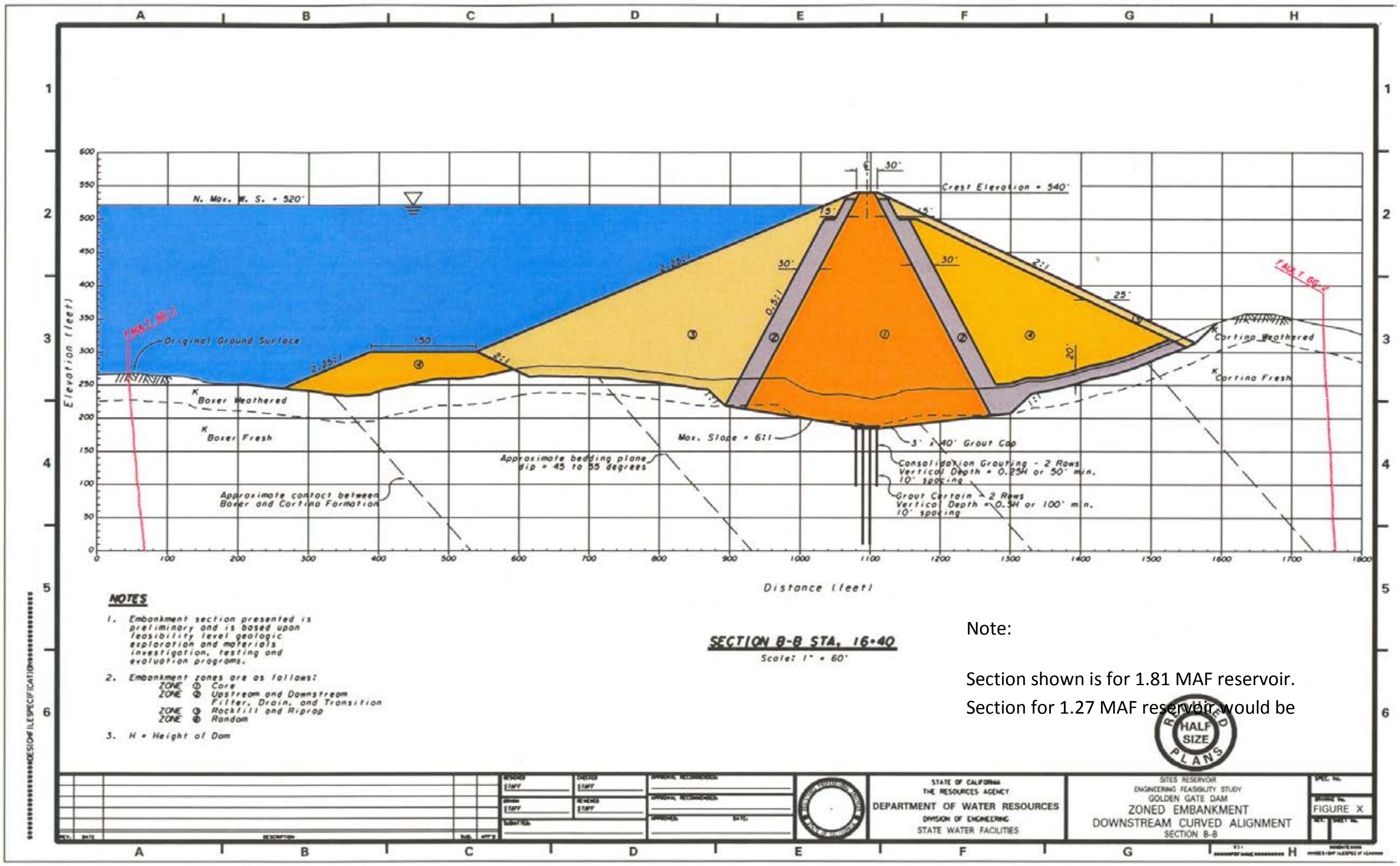
The proposed embankment section for the 1.27 MAF and 1.81 MAF reservoirs is a zoned earth, rockfill embankment type dam consisting of a central impervious core transitioned to exterior rockfill shells (Figure 3-6). The dam would have a crest width of 30 feet, and 20 feet of freeboard above the maximum 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 3-6. Golden Gate Dam – Zoned Embankment – Downstream Curved Alignment – Maximum Section



PRELIMINARY- SUBJECT TO CHANGE

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 Holthouse Reservoir. 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 (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 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 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.

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

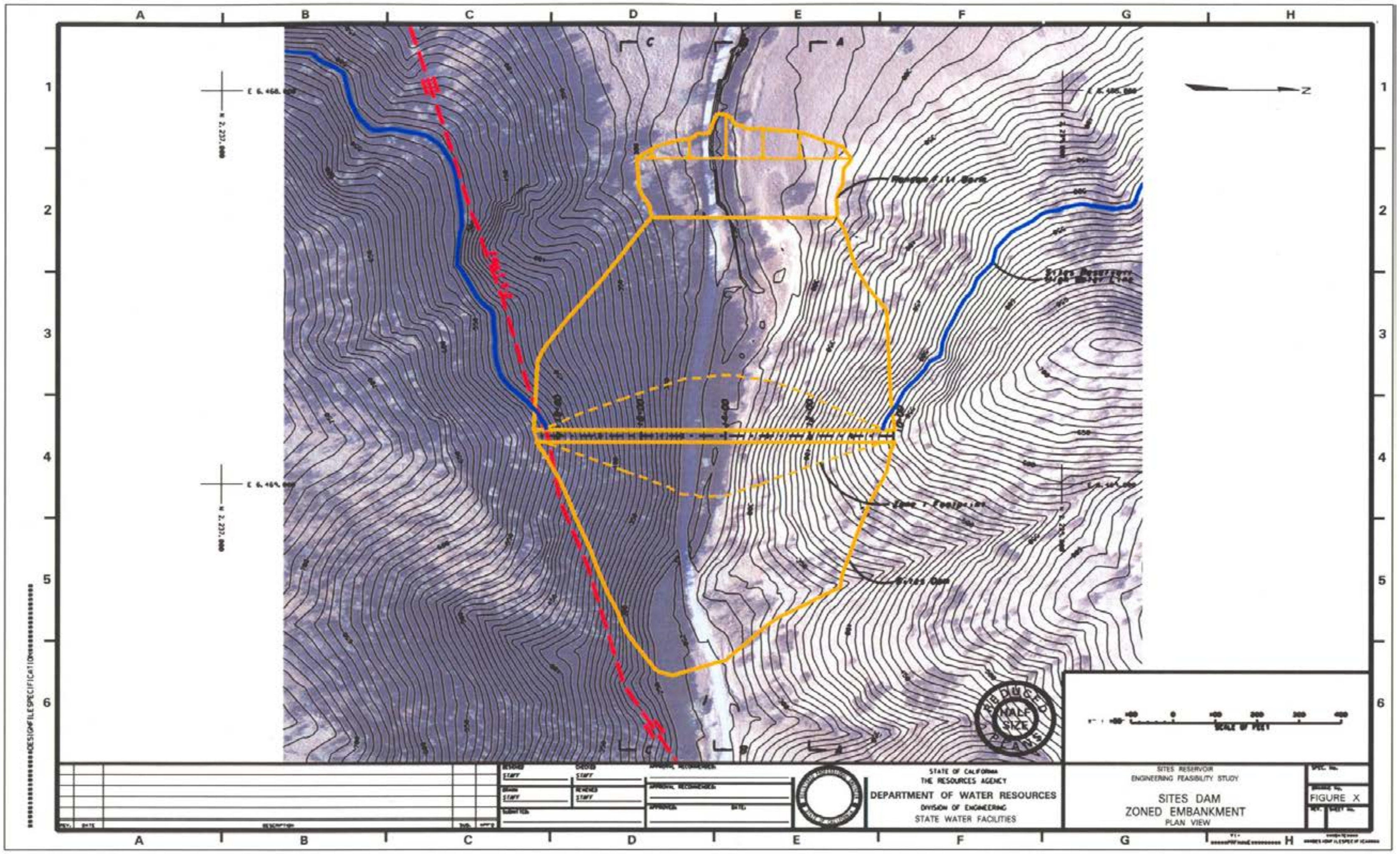
Dam Alignment – 1.81 MAF Reservoir

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



PRELIMINARY- SUBJECT TO CHANGE

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 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.

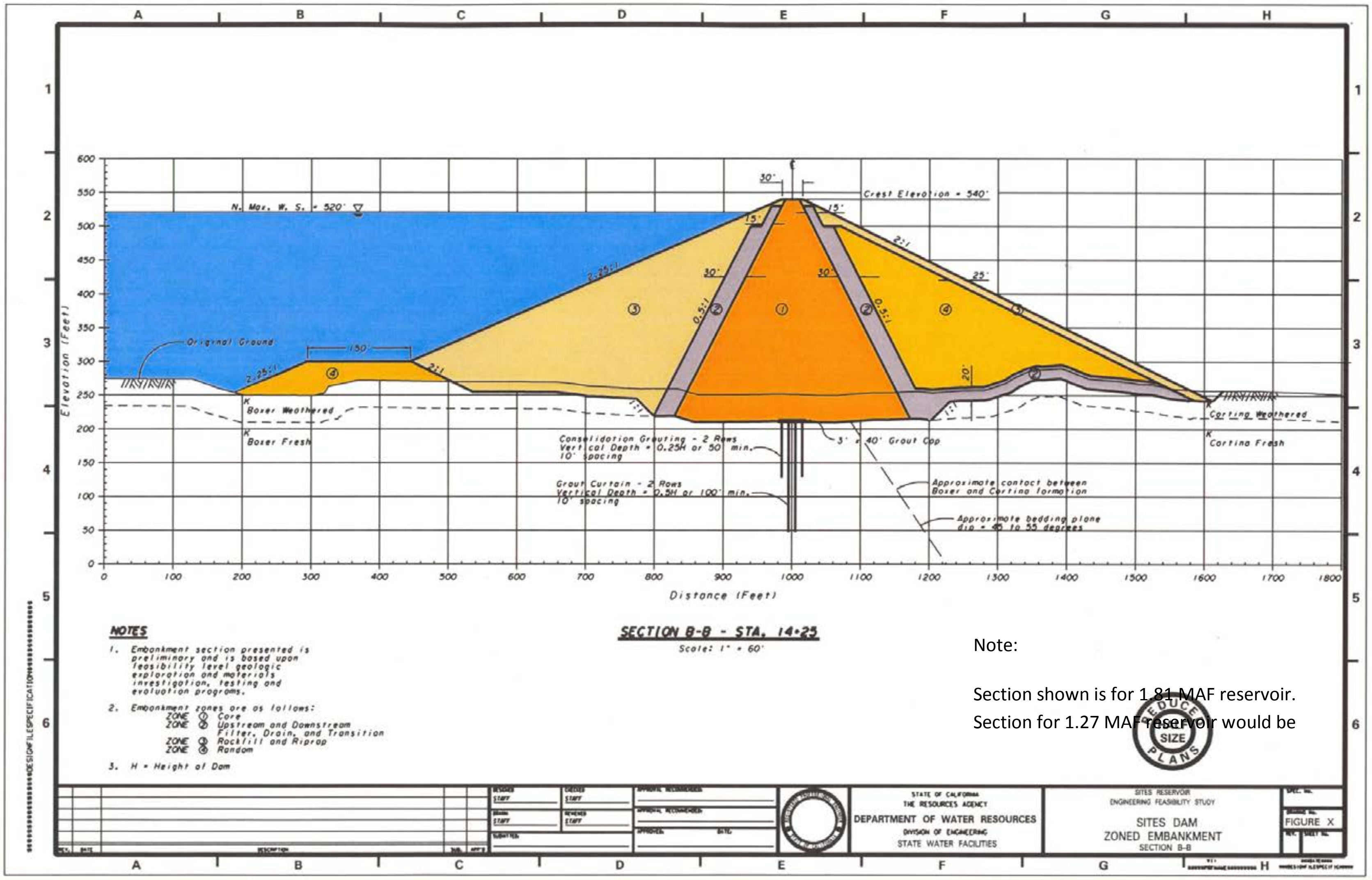
Foundation

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



PRELIMINARY- SUBJECT TO CHANGE

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 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.

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 (Golden Gate Saddle Dam)) 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 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 3-3 and 3-4 present saddle dam characteristics, including height and embankment volume. Figure 3-9 presents a plan view of the saddle dams.

Dam Alignments

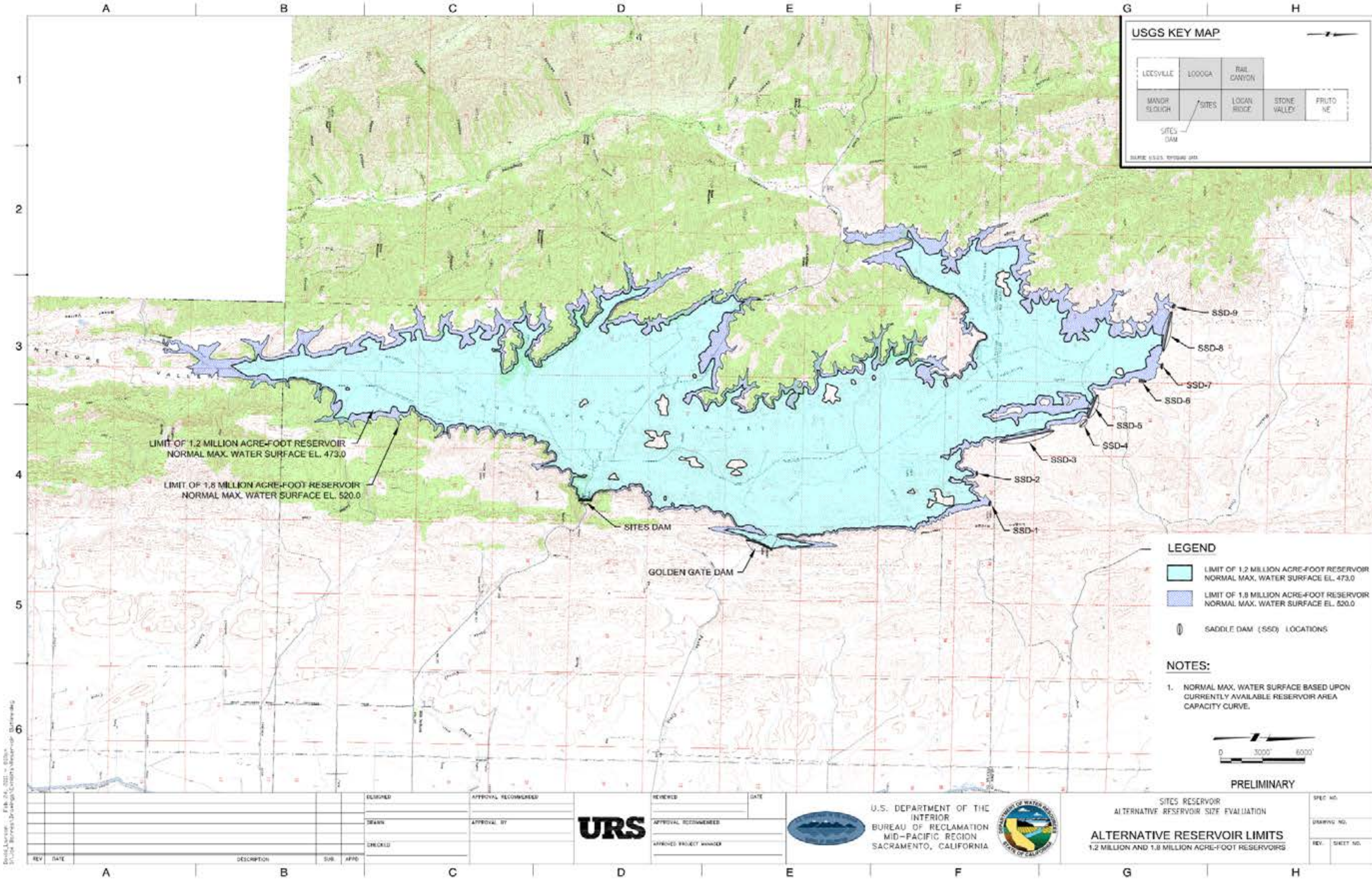
The topography at the north end of the proposed Sites Reservoir is such that the preferred alignments for the saddle dams are located along the relatively broad ridge between the Funks Creek and Hunters Creek watersheds. In general, saddle dam locations were selected to coincide with the northern ridge, as much as practicable, to minimize the creation of dead storage. Saddle dam alignments were selected to:

- 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
- Provide the most appropriate location and orientation with respect to 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.

Figure 3-9. Saddle Dam Location Map

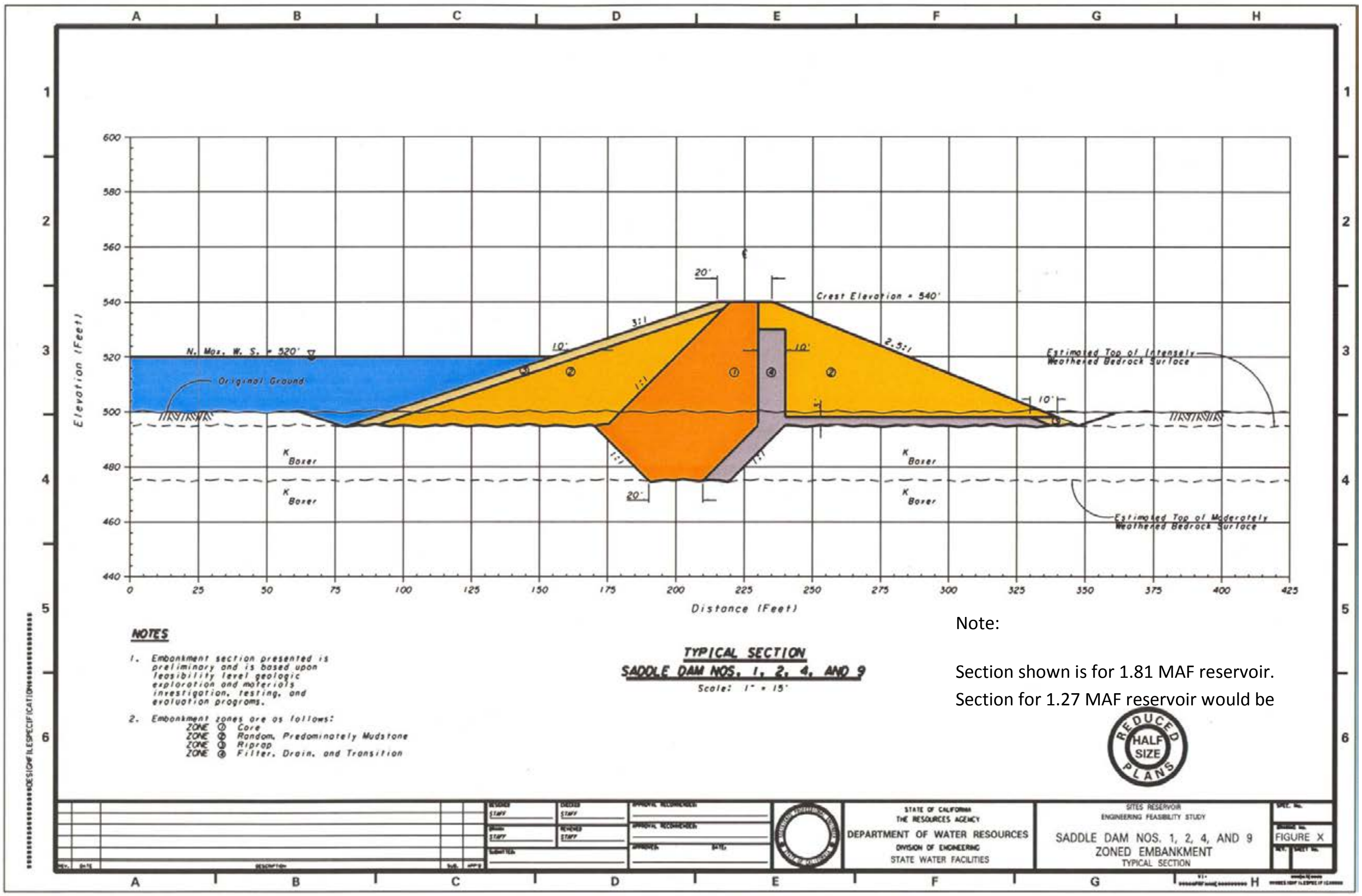


PRELIMINARY- SUBJECT TO CHANGE

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



PRELIMINARY- SUBJECT TO CHANGE

Figure 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:

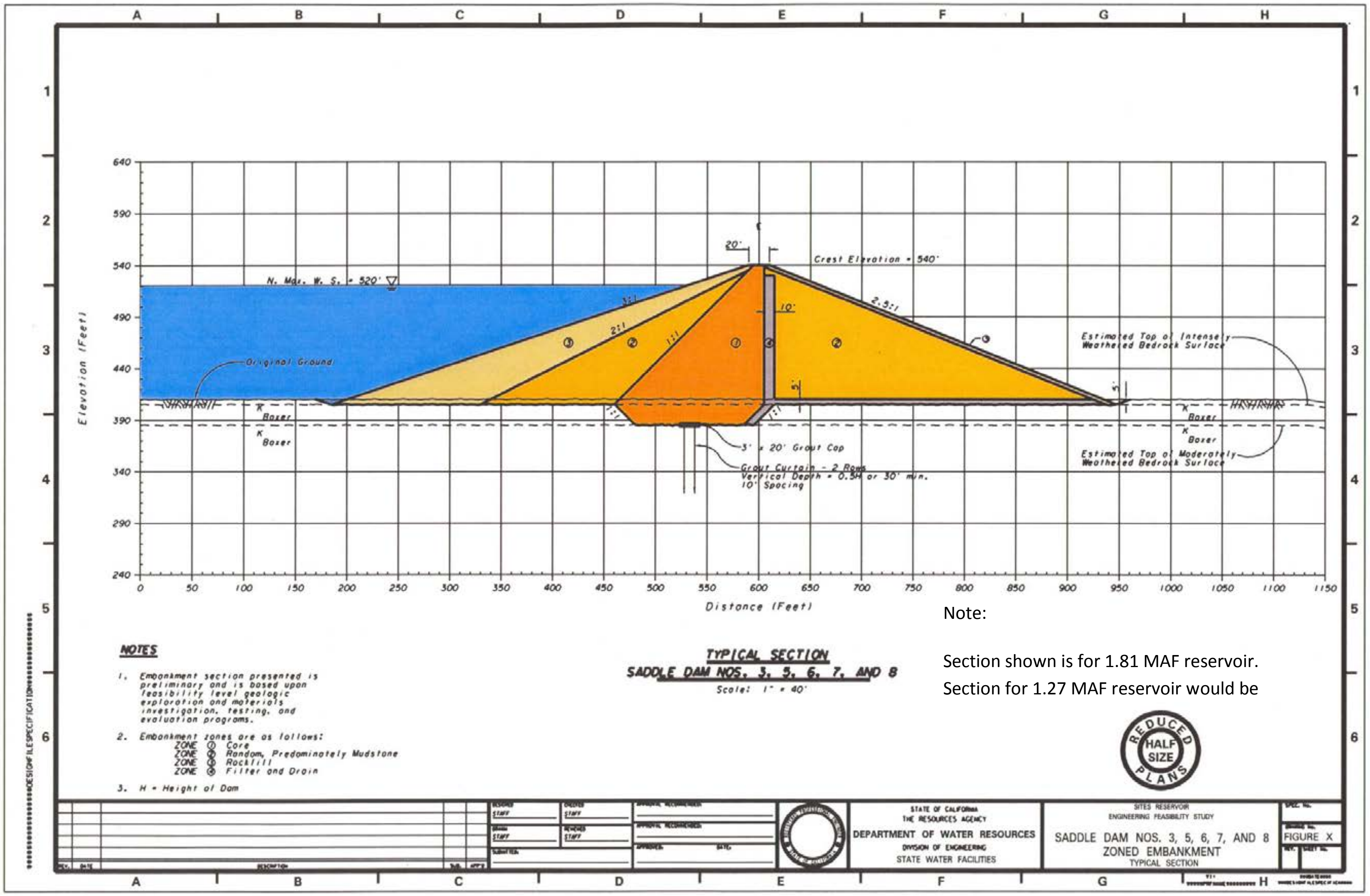
Zone 1: Impervious core material comprised of low to medium plasticity clays, with lesser amounts of high plasticity clays, and clayey sands. The impervious material would be obtained from designated borrow areas upstream of the saddle dams in the reservoir with haul distances of less than 1 mile. Impervious 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 3-11. Saddle Dams 3, 5, 6, 7, and 8 – Zoned Embankment – Typical Section



PRELIMINARY- SUBJECT TO CHANGE

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.

3.3.7 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 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 3-12. The 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 1.27 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 3-13.

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.

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

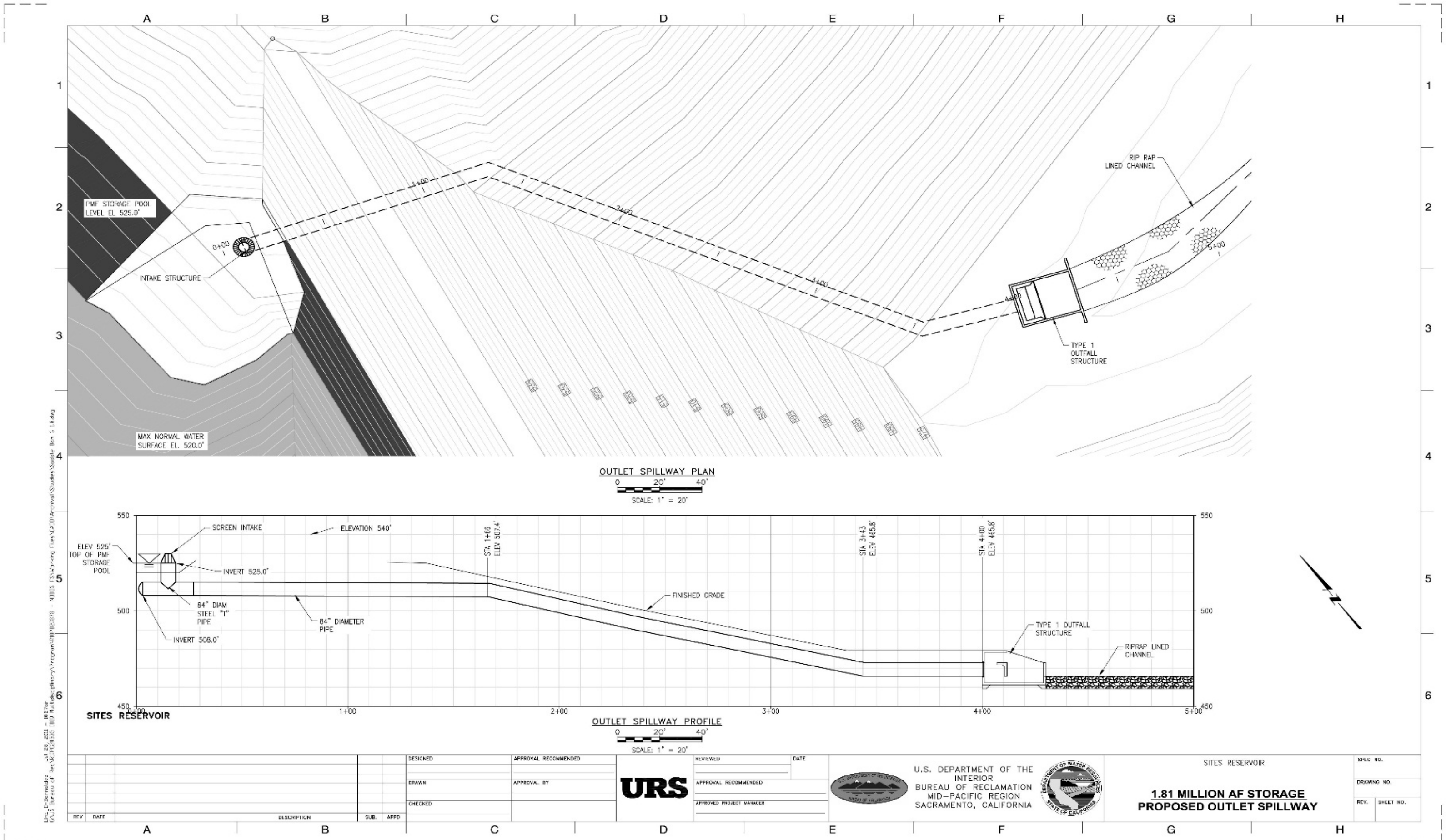


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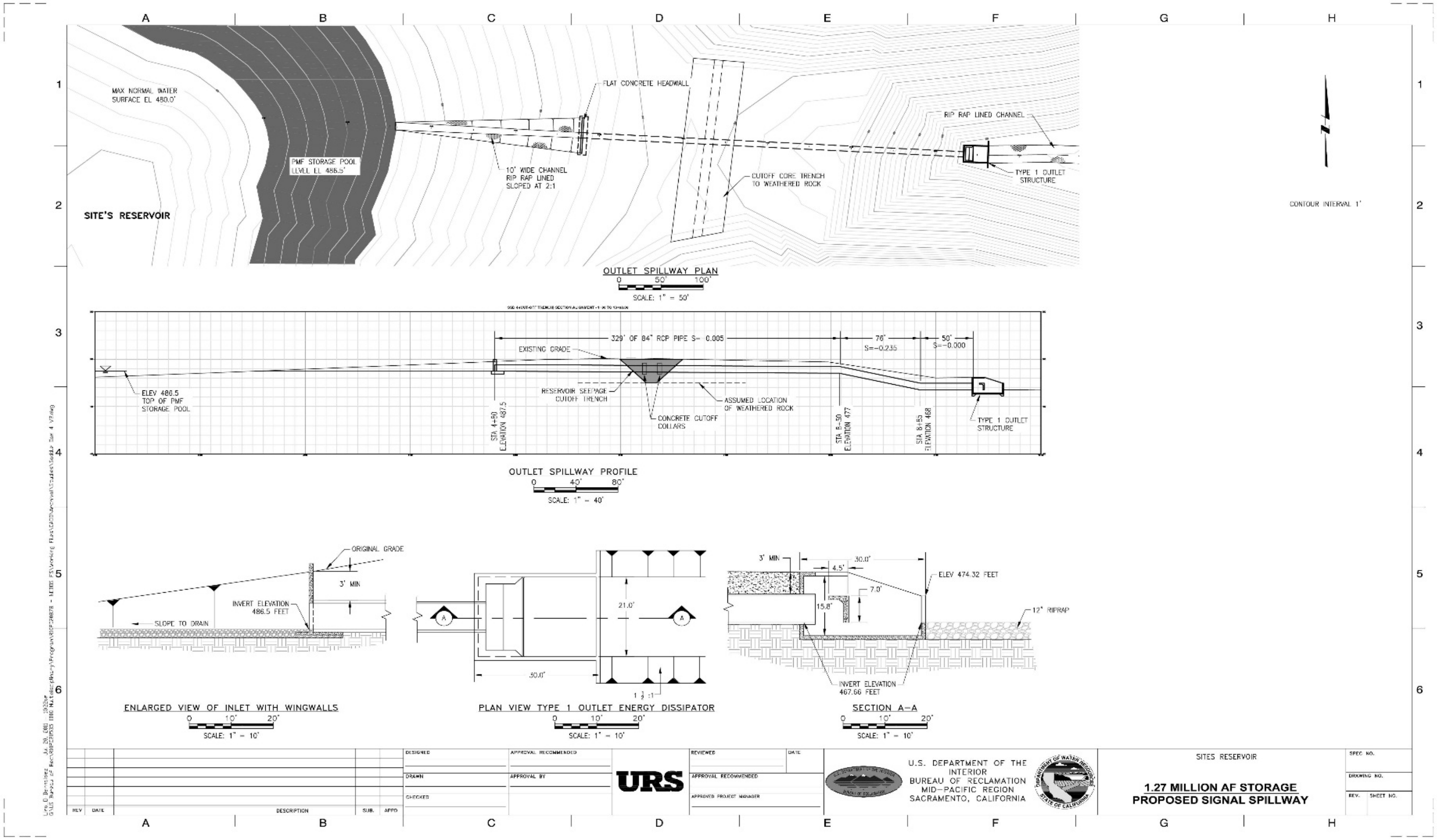
Figure 3-13. Signal Spillway – Saddle Dam 6 – 1.27 MAF Reservoir – Plan, Elevation, Sections



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3.3.8 Sites Reservoir Inlet/Outlet

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.

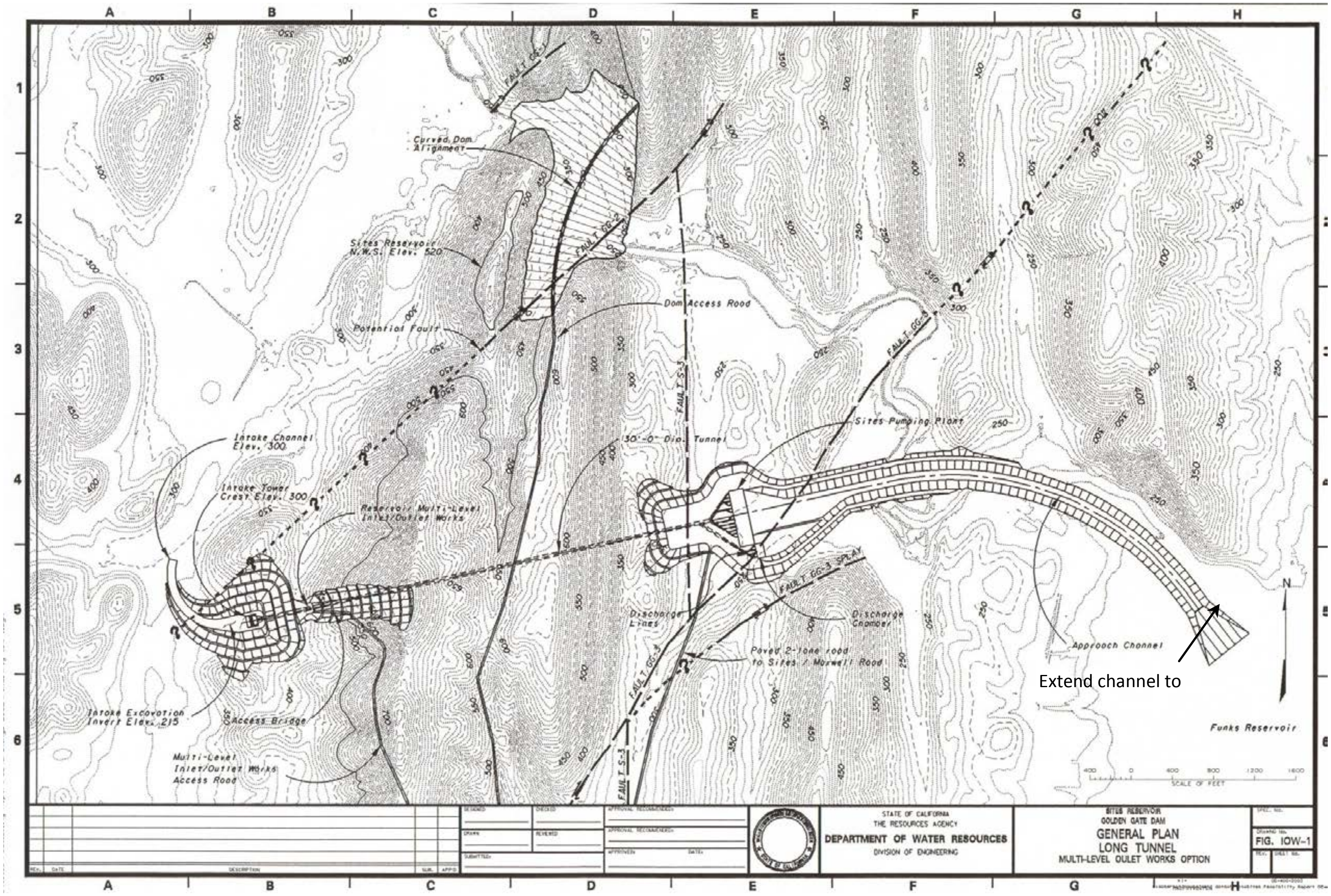
Estimates developed for Options 1 and 2 indicate comparable costs for each.

Releases made from the low-level outlet included in Option 1 would come from the bottom of the reservoir pool, resulting in coldwater 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

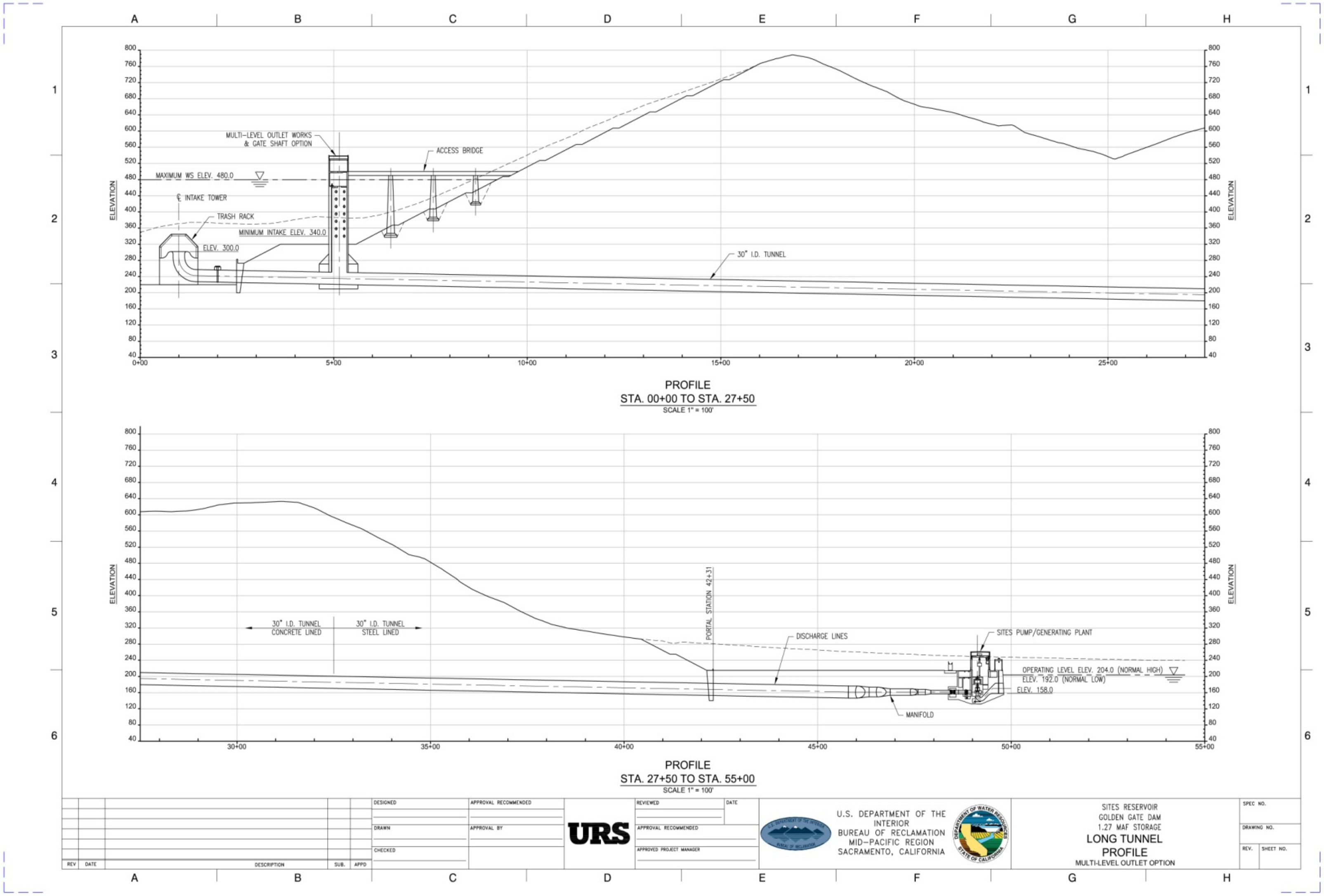
The reservoir inlet/outlet would consist of a multi-level, valved inlet/outlet tower and gate shaft, a 30-foot-diameter pressure tunnel approximately 4,000 feet long, and buried piping downstream from the tunnel connecting to the SPGP and energy dissipation valve structure. Under normal operating conditions, 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 valve structure. The proposed size of the inlet/outlet is controlled by the flow capacity required to make emergency reservoir drawdown releases, discussed in further detail in subsequent sections. Figure 3-14 presents a plan view of the inlet/outlet. Figures 3-15 and 3-16 show profiles for the inlet/outlet tower and tunnel for the 1.27 MAF and 1.81 MAF reservoirs, respectively.

Figure 3-14. Inlet/Outlet – General Plan



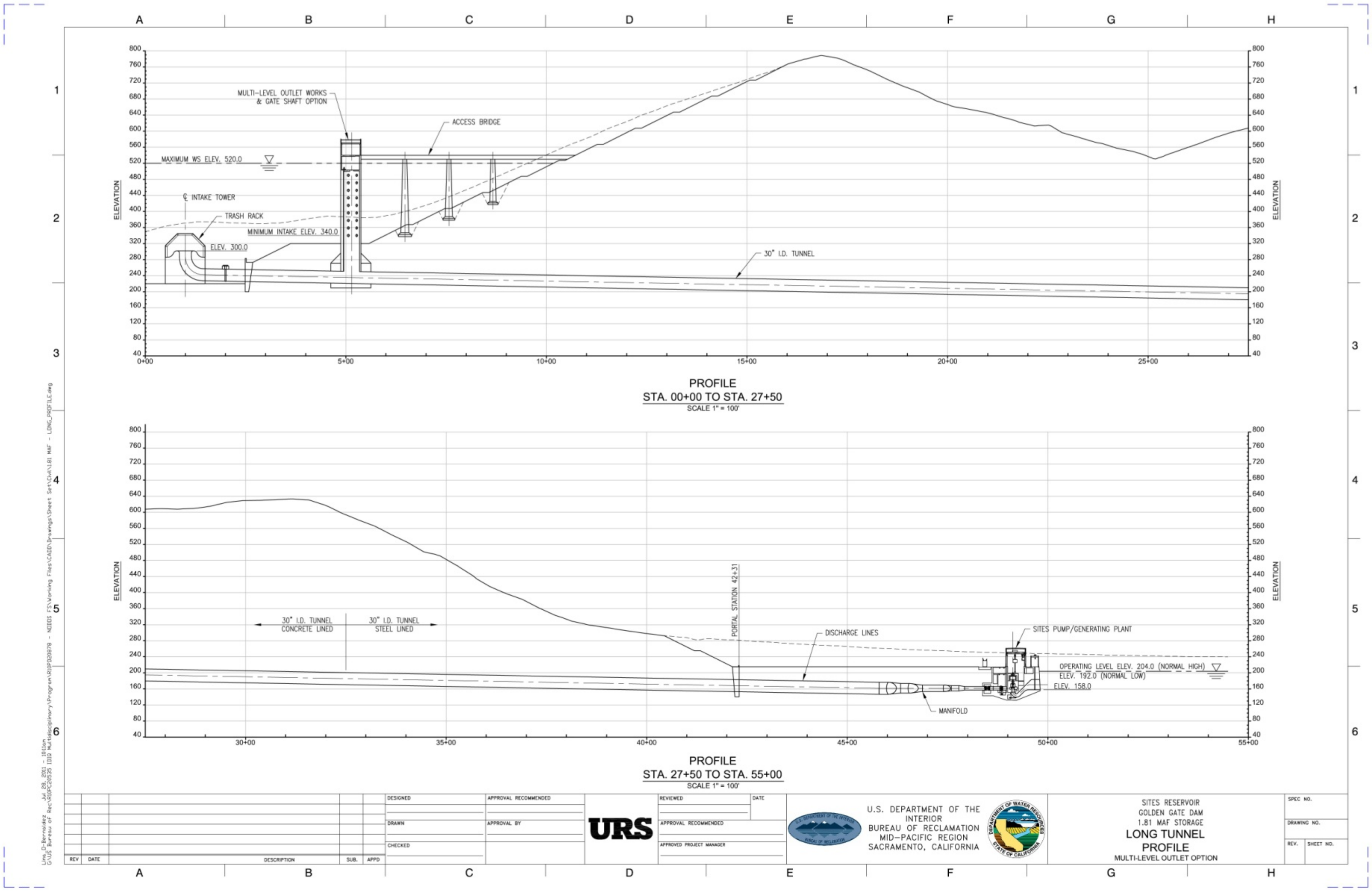
PRELIMINARY- SUBJECT TO CHANGE

Figure 3-15. Inlet/Outlet – Profile – 1.27 MAF Reservoir



PRELIMINARY- SUBJECT TO CHANGE

Figure 3-16. Inlet/Outlet – Profile – 1.81 MAF Reservoir



PRELIMINARY- SUBJECT TO CHANGE

Design Assumptions

Preliminary design of the reservoir inlet/outlet was performed in accordance with the state of practice for dam appurtenant structure design with conformance to 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 3-5 provides a comparison of tower details for the two reservoir sizes under consideration.

Table 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 ports controlled by butterfly valves. The valves are 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 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.

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.

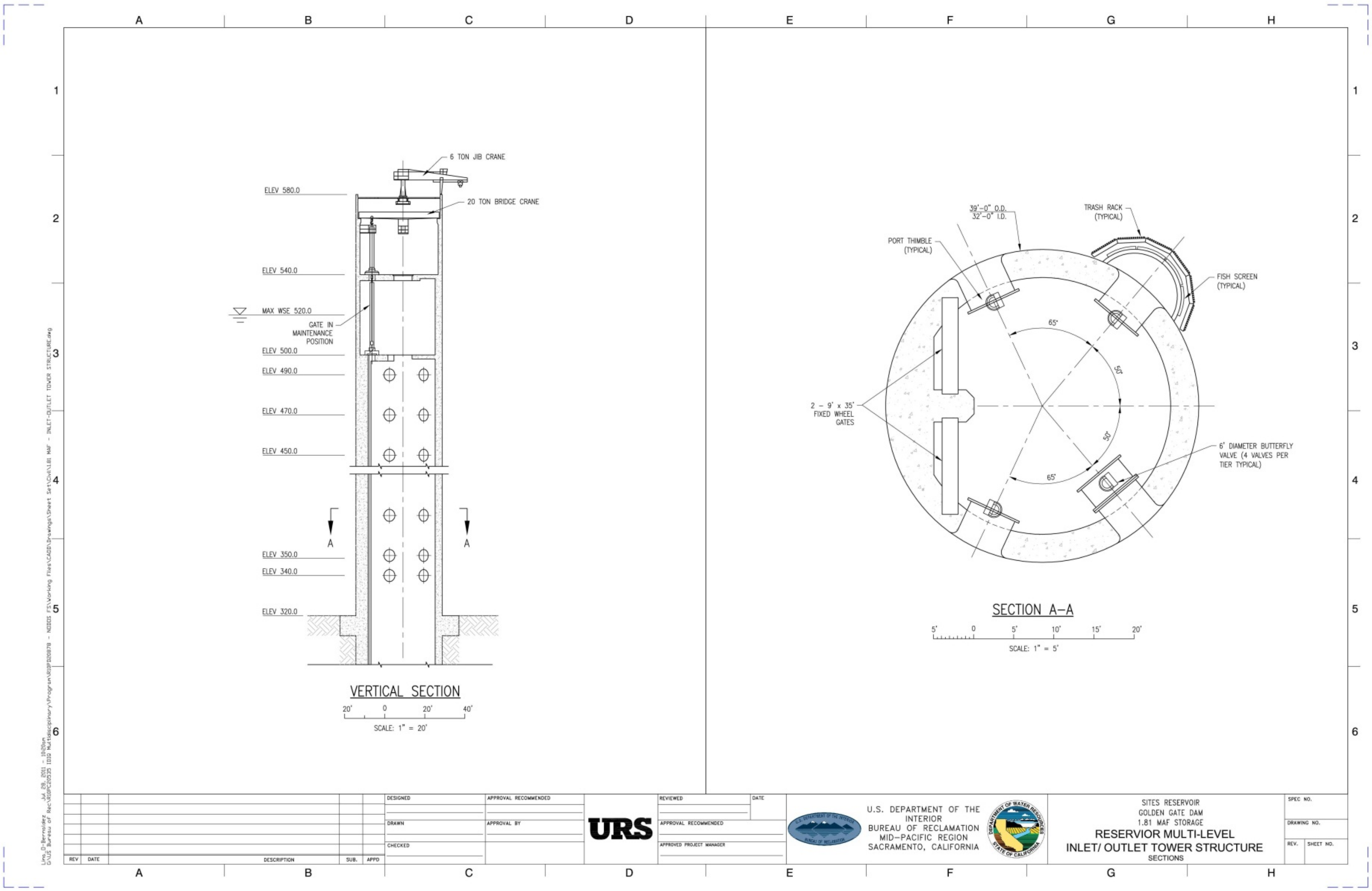
Pressure Tunnel

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 3-15 and 3-16 present a profile view of the proposed tunnel for the two reservoir sizes under consideration.

Figure 3-17. Inlet/Outlet Tower Structure – Typical Sections



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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 for the energy dissipation valve structure. Subsequent bifurcations and size reductions create a manifold of piping that connects to the various piping and pumping/generating units in the SPGP. Figure 3-18 presents a plan view of the penstock and manifolds for a 5,900 cfs SPGP. The manifold lines were not sized by an economic analysis, but rather for pumping flow velocities of 10 fps or less.

All buried penstocks and manifolds 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 manifold piping favor steel.

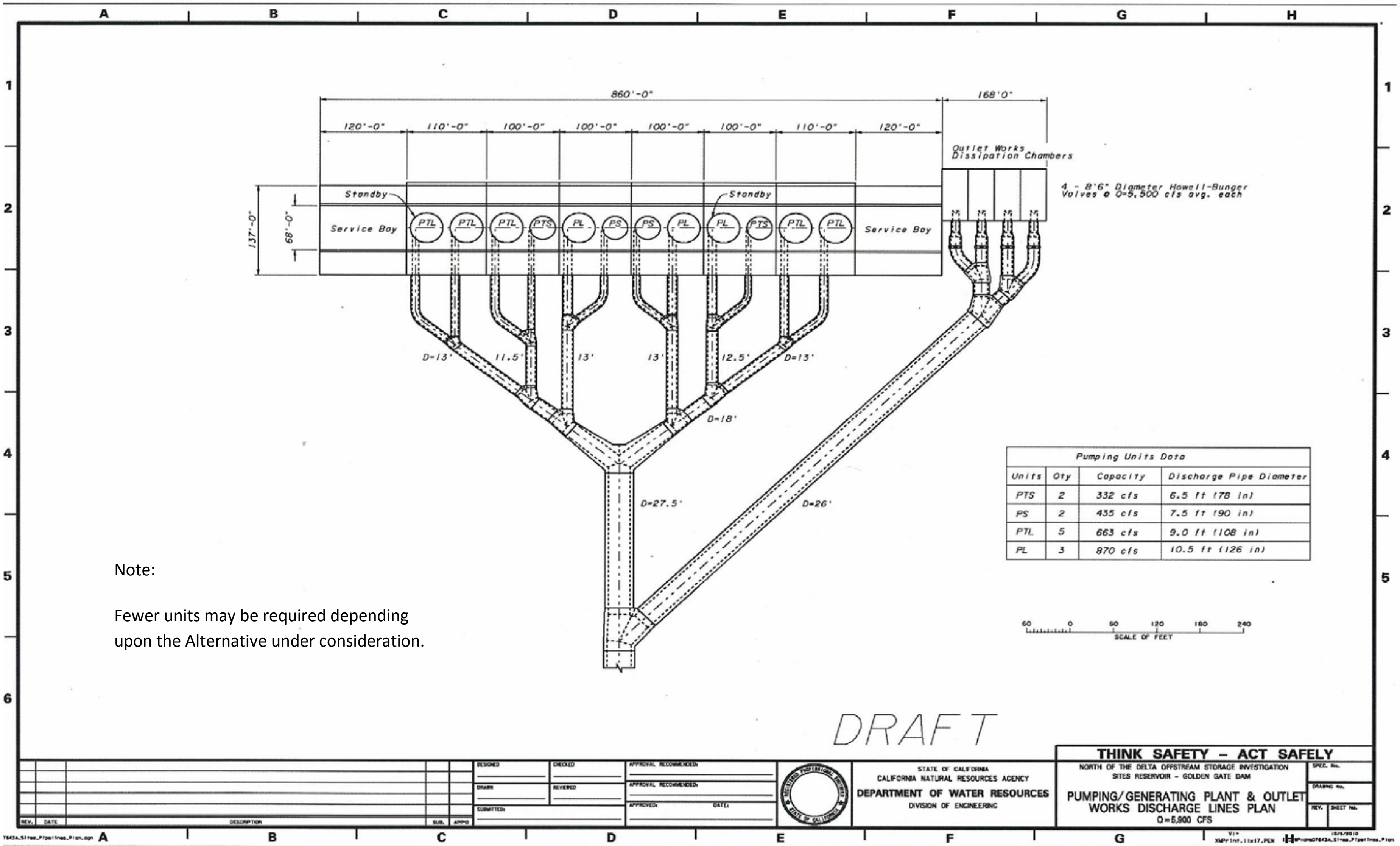
Emergency Release

Emergency release facilities were sized to meet general emergency drawdown guidelines required by DSOD. This general guideline is that large reservoir outlet facilities should have a flow capacity capable that could lower the maximum reservoir storage depth by 10 percent within 10 days. In the case of Sites Reservoir, this correlates to approximately 30 feet of drawdown (400,000 AF) and an average outflow of approximately 20,000 cfs would be required.

The option considered would be a 26-foot diameter bypass pipeline branching off of the 30-foot-diameter buried penstock and leading to four energy dissipation valves. The 26-foot-diameter pipeline would bifurcate and reduce in size several times to join to four 8´-6″ fixed-cone dispersion (Howell Bunger) valves located in reinforced-concrete energy dissipation chambers. Figures 3-18 and 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. Further evaluation will be needed tin detailed design regarding 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 requirements be revisited during future design investigations as the DSOD generally examines evacuation requirements on a site-specific basis for large reservoirs.

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



Pumping Units Data			
Units	Qty	Capacity	Discharge Pipe Diameter
PTS	2	332 cfs	6.5 ft (78 in)
PS	2	435 cfs	7.5 ft (90 in)
PTL	5	663 cfs	9.0 ft (108 in)
PL	3	870 cfs	10.5 ft (126 in)

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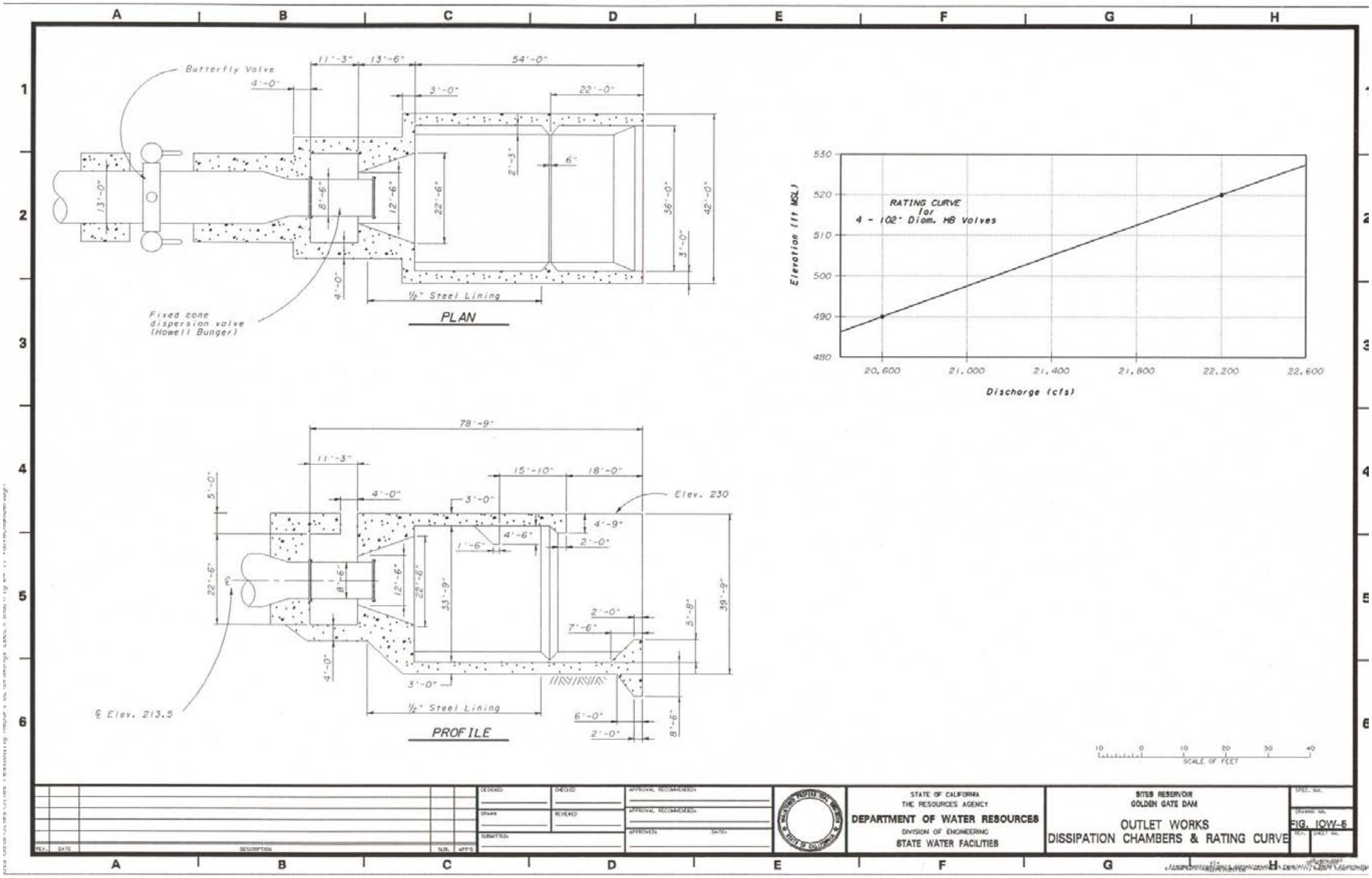
Figure 3-19. Sites Pumping/Generating Plant – Dissipation Chambers and Rating Curve



PRELIMINARY- SUBJECT TO CHANGE

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

Approximately 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. Existing fencing would need to be removed.

Relocations

It would be necessary to relocate two existing cemeteries.

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 3-6 provides a summary of the strengths used in the stability model.

Table 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

Assumed phreatic surfaces used in stability analyses include the following:

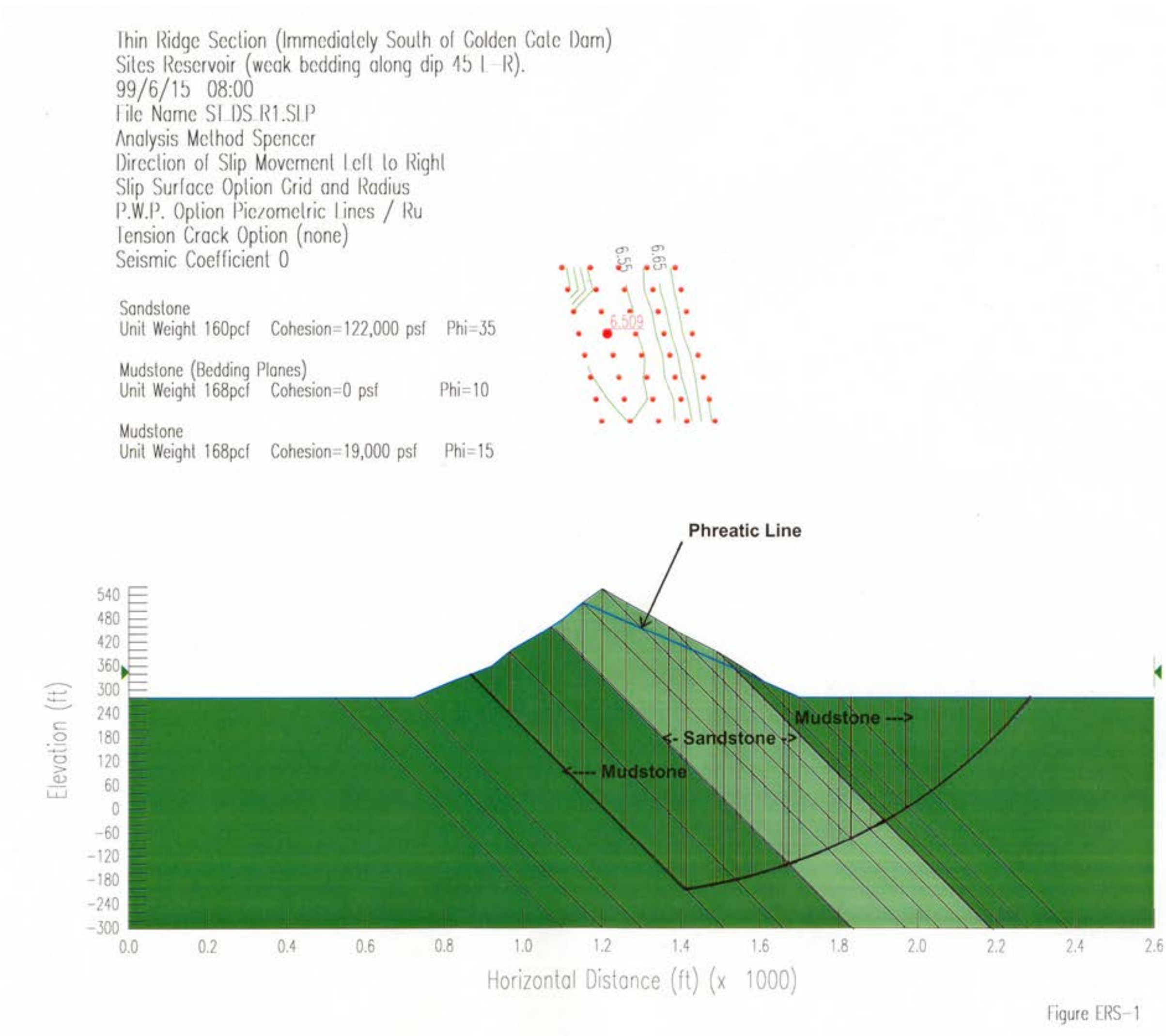
- Full Reservoir – Water surface at elevation 520 feet, phreatic surface linear from upstream water surface to downstream toe of slope.

- Partial Pool – Water surface at elevation 395 feet, phreatic surface (pore water pressure at atmospheric conditions) 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 3-20 illustrates a typical sliding surface evaluated as part of the stability analysis.

Figure 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

3.4.1 Introduction

The proposed SPGP would be located approximately 3,300 feet southeast (downstream) of Golden Gate Dam (Figure 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).

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.

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 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 T-C 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 3-21, 3-22, and 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 3-7, 3-8, and 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.

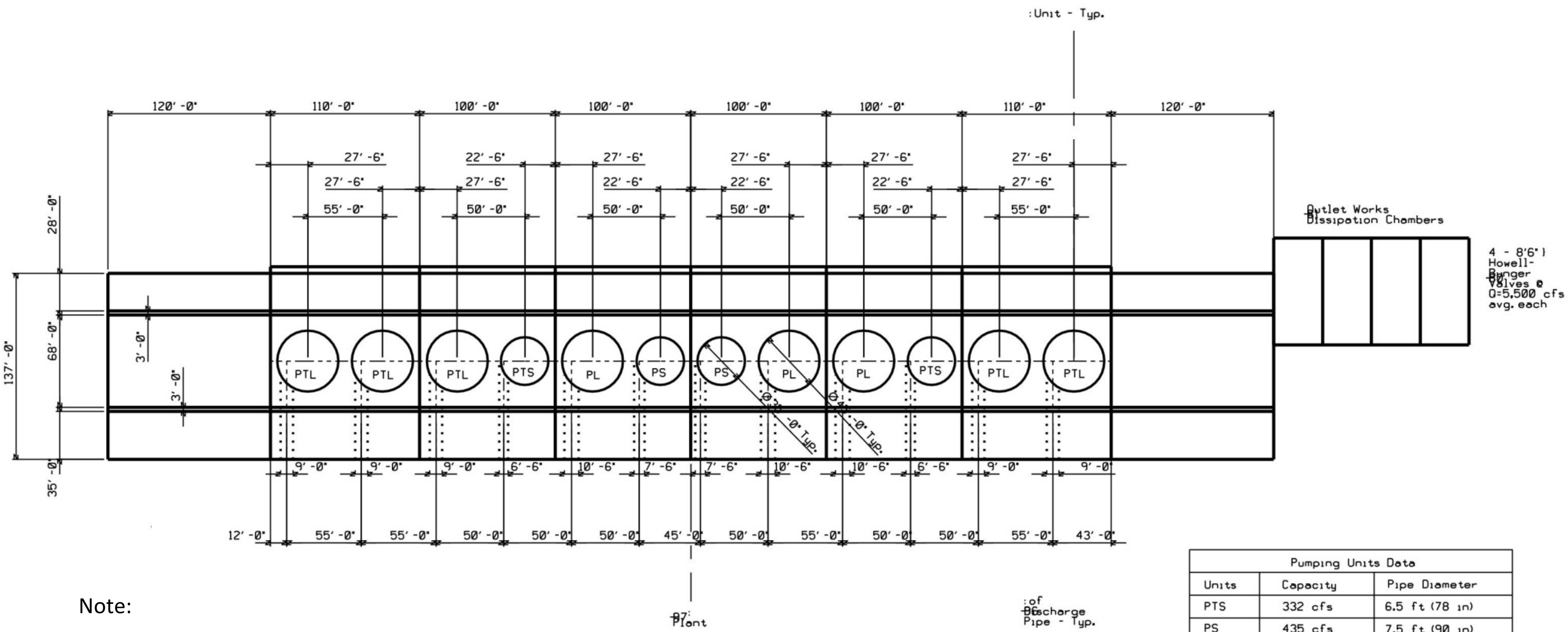
The pumping plant would be a conventional, indoor-type pumping/generating plant with an in-line arrangement of vertical pumping units. The SPGP would have a reinforced concrete substructure and a steel superstructure with the draft tube invert at an elevation of 170 feet. The base of the service bay 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

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.

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



Note:

Number and configuration of units varies between Alternatives.

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)

1" = 40' 40 0 40 80 120 160 SCALE OF FEET

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

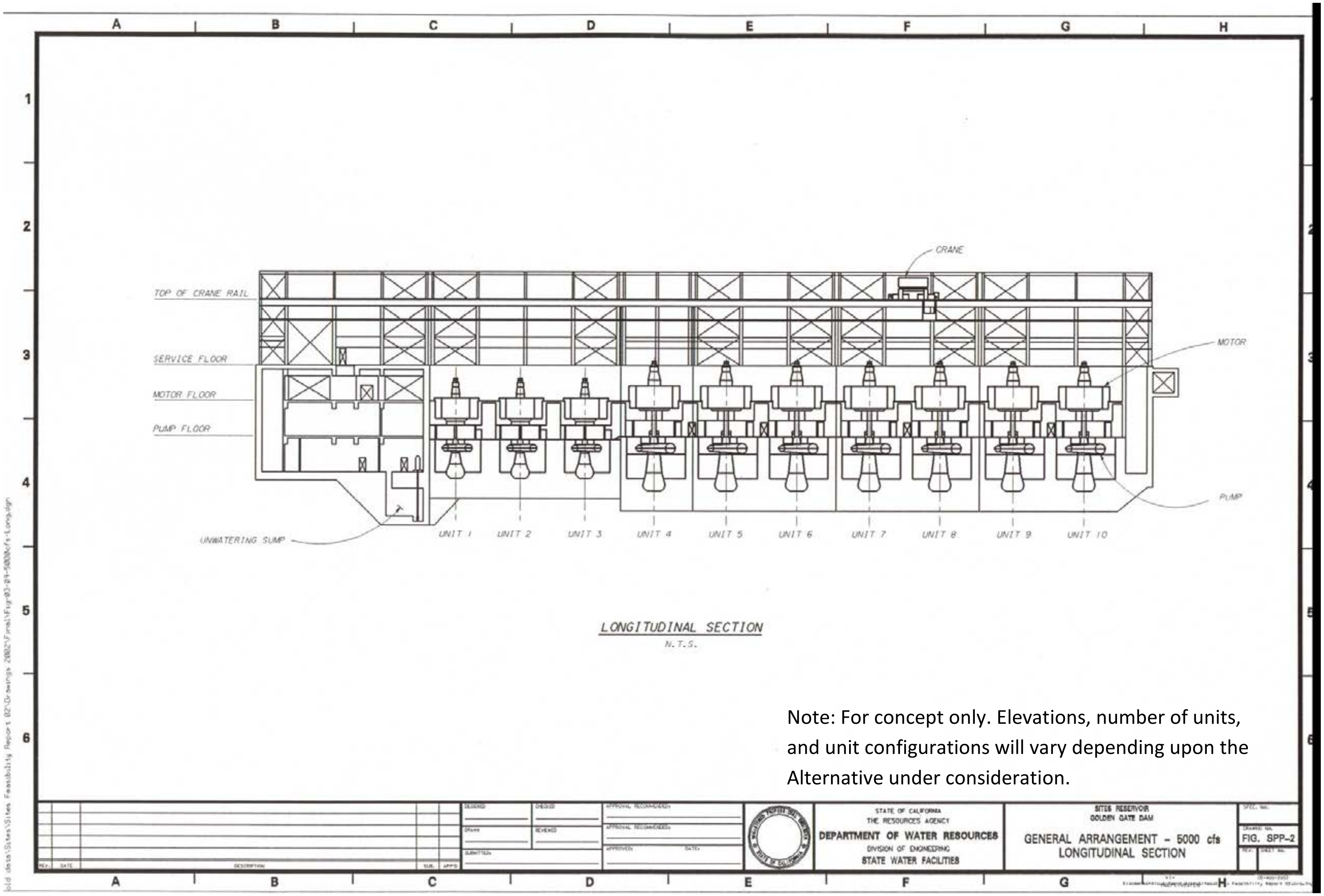

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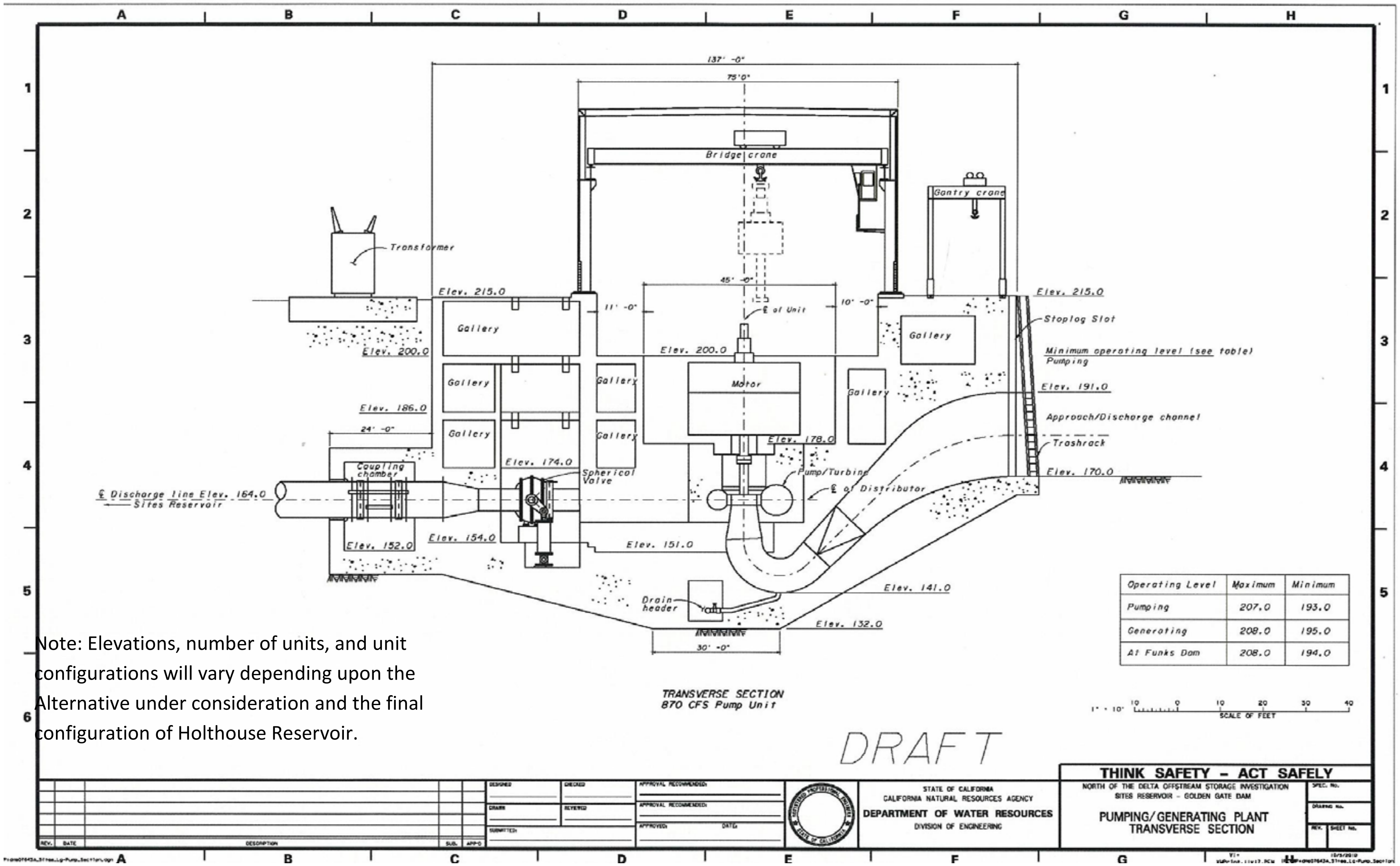
Figure 3-23. Sites Pumping/Generating Plant – Transverse Section **Figure 3-23. Sites Pumping/Generating Plant – Transverse Section**



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

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Table 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 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 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

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.

3.5 Holthouse Reservoir

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 released to the downstream extension of the T-C Canal by gravity through an outlet control structure on the south side. The reservoir is used as a regulating reservoir for the T-C 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)

3.5.2 Holthouse 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 3-24, 3-25, and 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 would 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 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 3-25. Holthouse Reservoir – Modification – Sheet 1

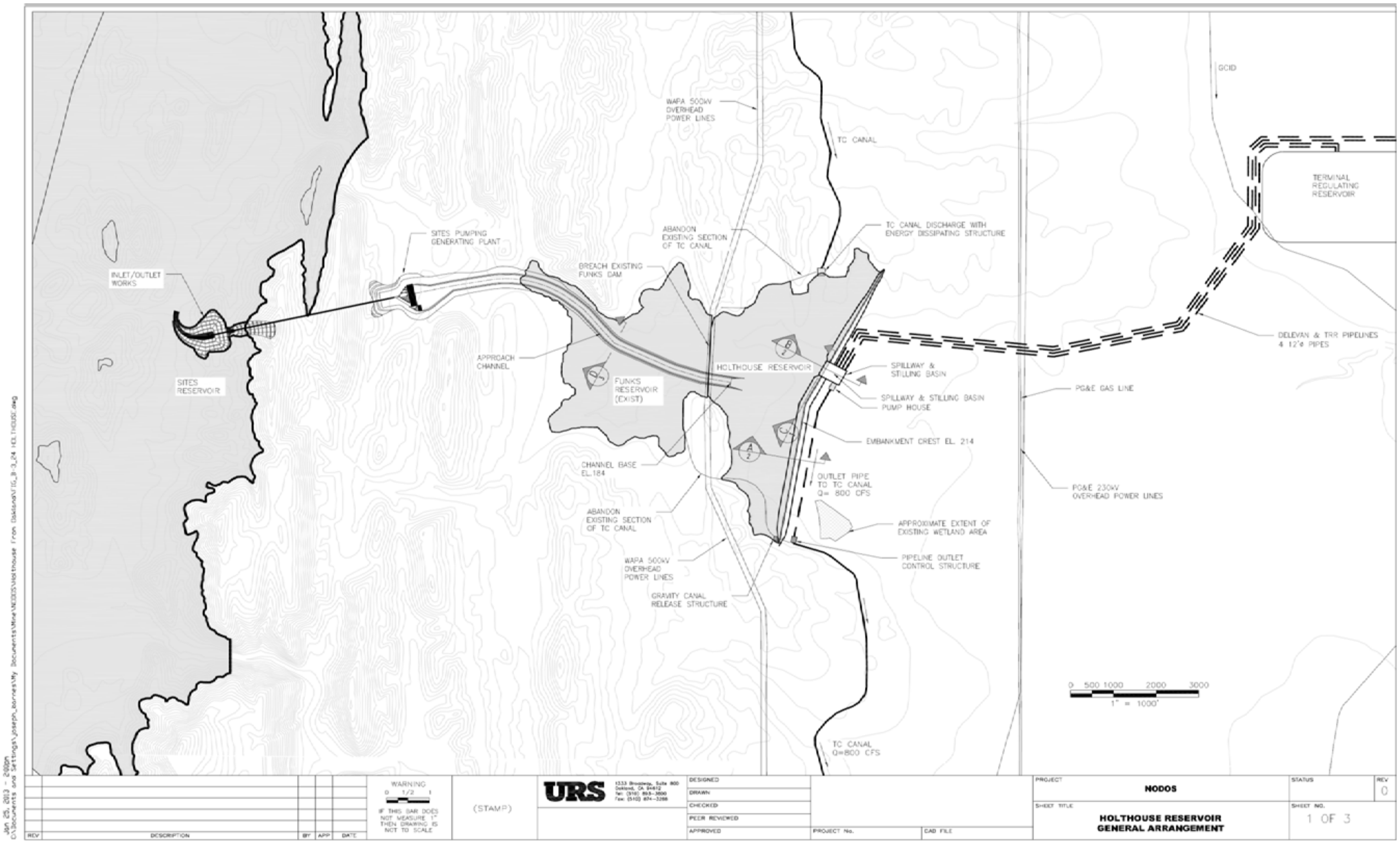

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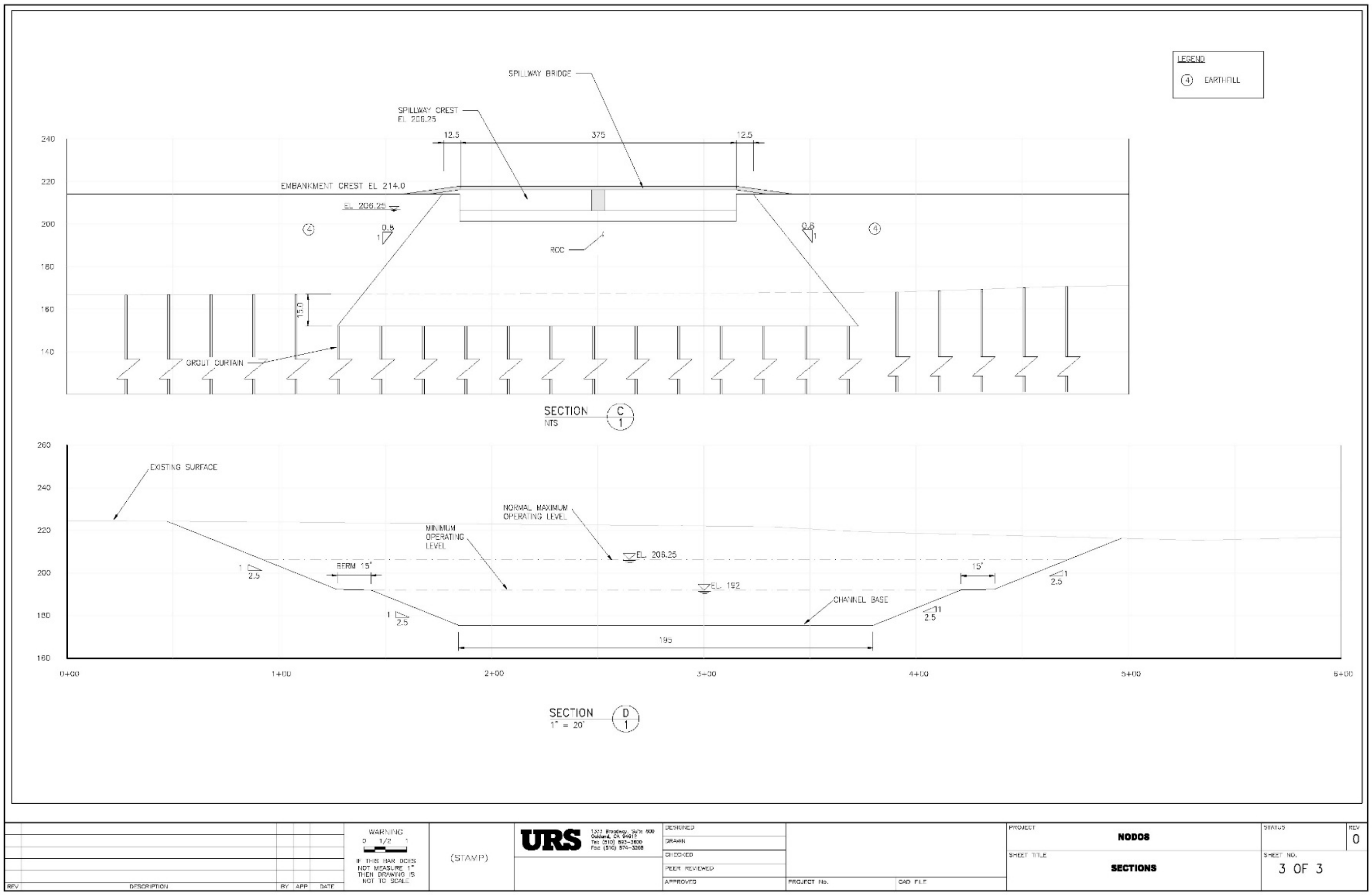
Figure 3-26. Holthouse – Modification – Sections Sheet



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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 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.

T-C-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 T-C 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 proposed 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 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 3-24 shows the location of the canal; Figure 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 8,300 feet 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 T-C Canal Connections

The T-C 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.

3.5.3 T-C Canal Construction Bypass

Installation of a bypass is required to divert T-C 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 T-C Canal Inlet into Funks Reservoir. The bypass would route the required flows around Funks Reservoir during reservoir modification construction. Figure 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 T-C 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.

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 T-C Canal and GCID Canal as well as through the Delevan Pipeline. Preliminary feasibility engineering analyses have been conducted on alternative capacities for the T-C 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.

3.6.1 Existing T-C Canal

Evaluation of T-C Canal capacity alternatives resulted in the recommendation that the T-C Canal remain at its current capacity of 2,100 cfs. Because this is the current capacity, no modifications are required or recommended to the T-C 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.

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 and line 200 feet of the canal immediately downstream of the headgate structure with concrete
- Replace the railroad siphon undercrossing

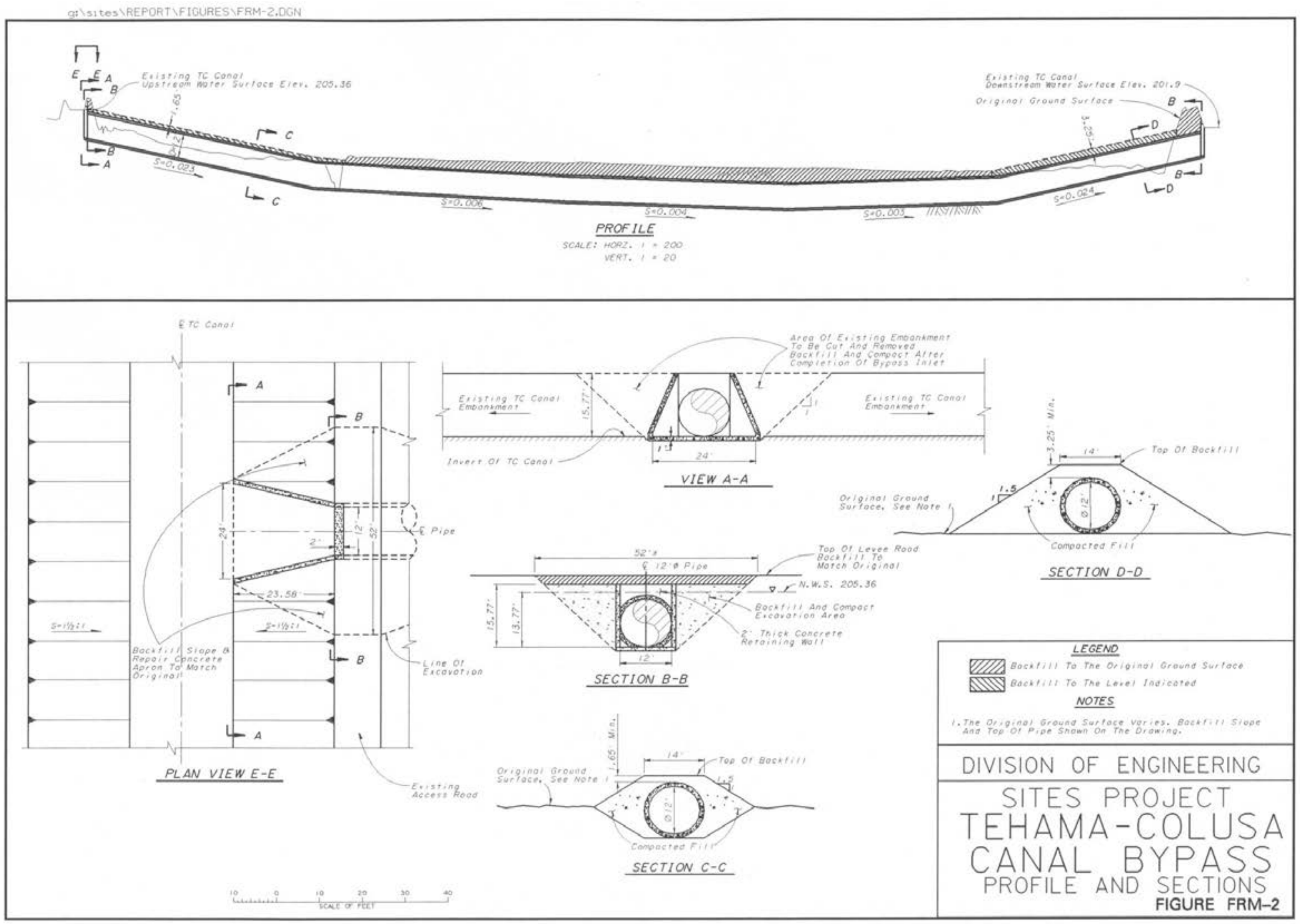
Seasonal Canal Maintenance

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.

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



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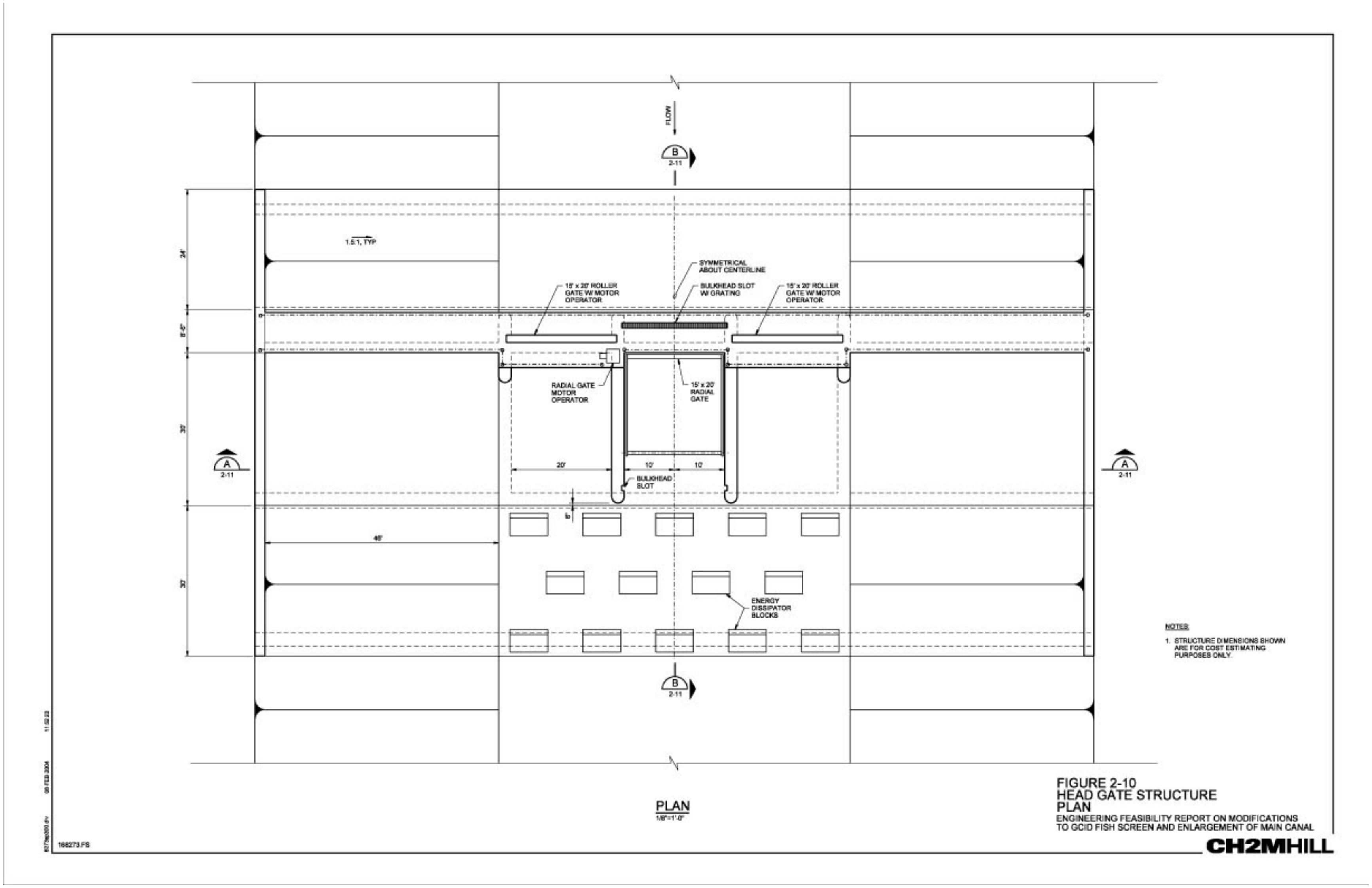
Headworks Modifications

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 3-28 and 3-29 show the replacement headgate structure in plan and section, respectively. The new headgate structure would provide the following three main operations:

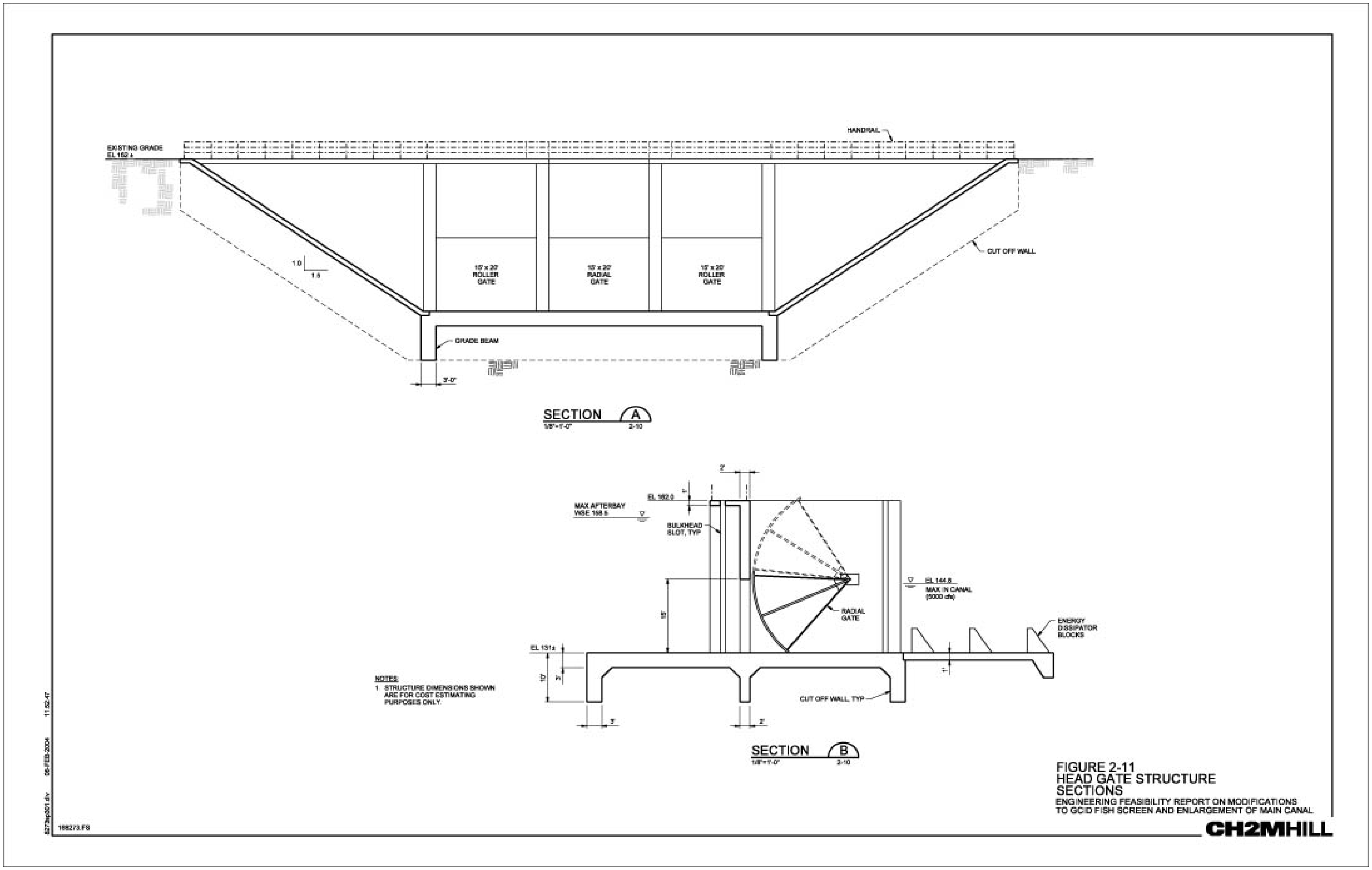
- Isolate the GCID Canal, as needed, for repairs or other purposes, such as the canal reach between the Main Pump Station and Stony Creek to prevent local flooding during high river levels.
- Control flow when the headworks are under gravity inflow conditions and the pumping plant is shut down, which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

Figure 3-28. GCID Canal Headgate Structure – Plan



PRELIMINARY- SUBJECT TO CHANGE

Figure 3-29. GCID Canal Headgate Structure – Sections



PRELIMINARY- SUBJECT TO CHANGE

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 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).

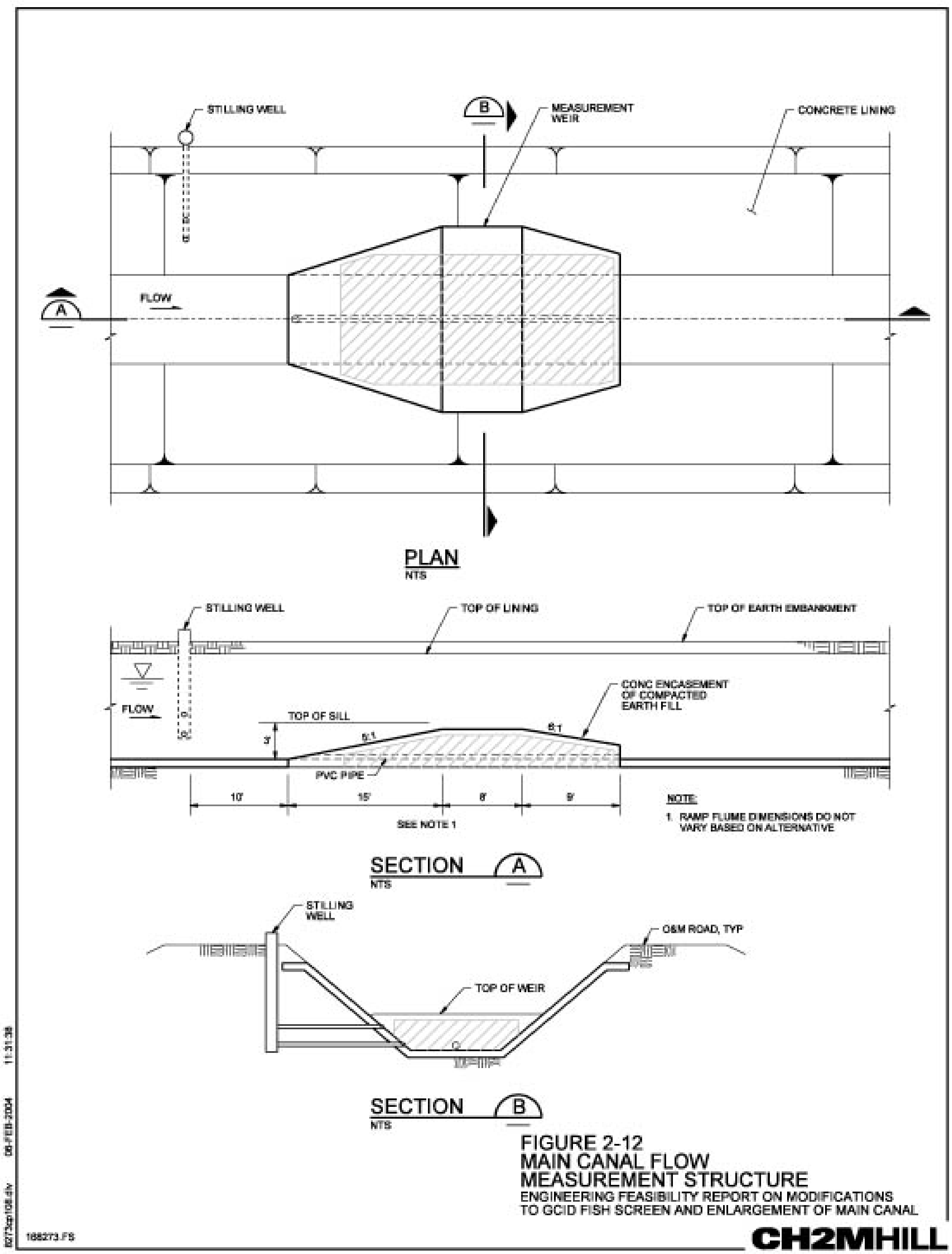
The resulting hydrostatic forces require a substantial concrete structure with cut-off walls keyed into the canal invert and side slopes. Canal lining would extend approximately 200 feet downstream of the structure.

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 3-30. GCID Canal Flow Measurement Structure



PRELIMINARY- SUBJECT TO CHANGE

3.7 Terminal Regulating Reservoir and TRR Pumping/Generating Plant

Water conveyed down the GCID Canal would be conveyed into a proposed TRR. A new pump station, the TRR Pumping/Generating 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 generating units located in the TRR Pumping/Generating Plant. The TRR would provide routine operational storage for the pumping plant and an afterbay for the turbine generating 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. A pipeline will also be provided for the TRR to Funks Creek to facilitate reservoir drainage.

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 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.

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 generating 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 generating 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

Power will be generated during project operation. A comprehensive power study is being performed to assess power supply versus demand, and to assess the costs for connecting the new site facilities to existing power circuits. Completion of the ongoing 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.

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.

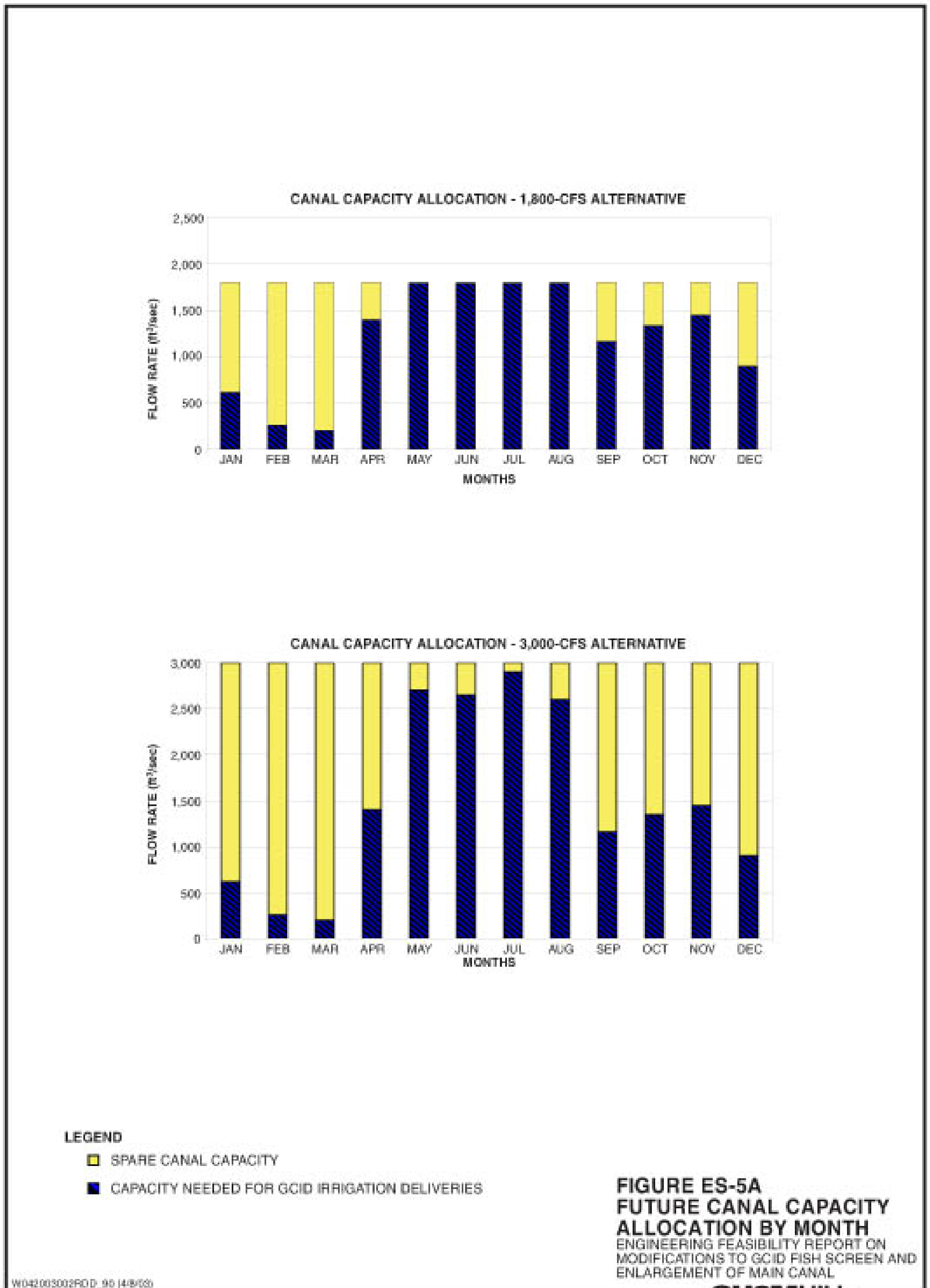
Extensive remote monitoring and combinations of local and remote controls would be required to operate the modified GCID Canal using an integrated SCADA system. The SCADA system would need to provide information on operating parameters such as Main Pump Station flows, canal water levels, check structure 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 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 3-31. Monthly Conveyance Capacity Allocation for GCID Canal



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3.8 New Sacramento River Pumping/Generating Plant 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 be capable of releasing up to 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 3-32 and 3-33 show the new Sacramento River Conveyance configuration.

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 158.5, 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
- 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 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 of 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 3-35 and 3-36 show a floor plan of the motor floor and pump floor, respectively. Figures 3-37 and 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 3-32. New Sacramento River Conveyance Layout

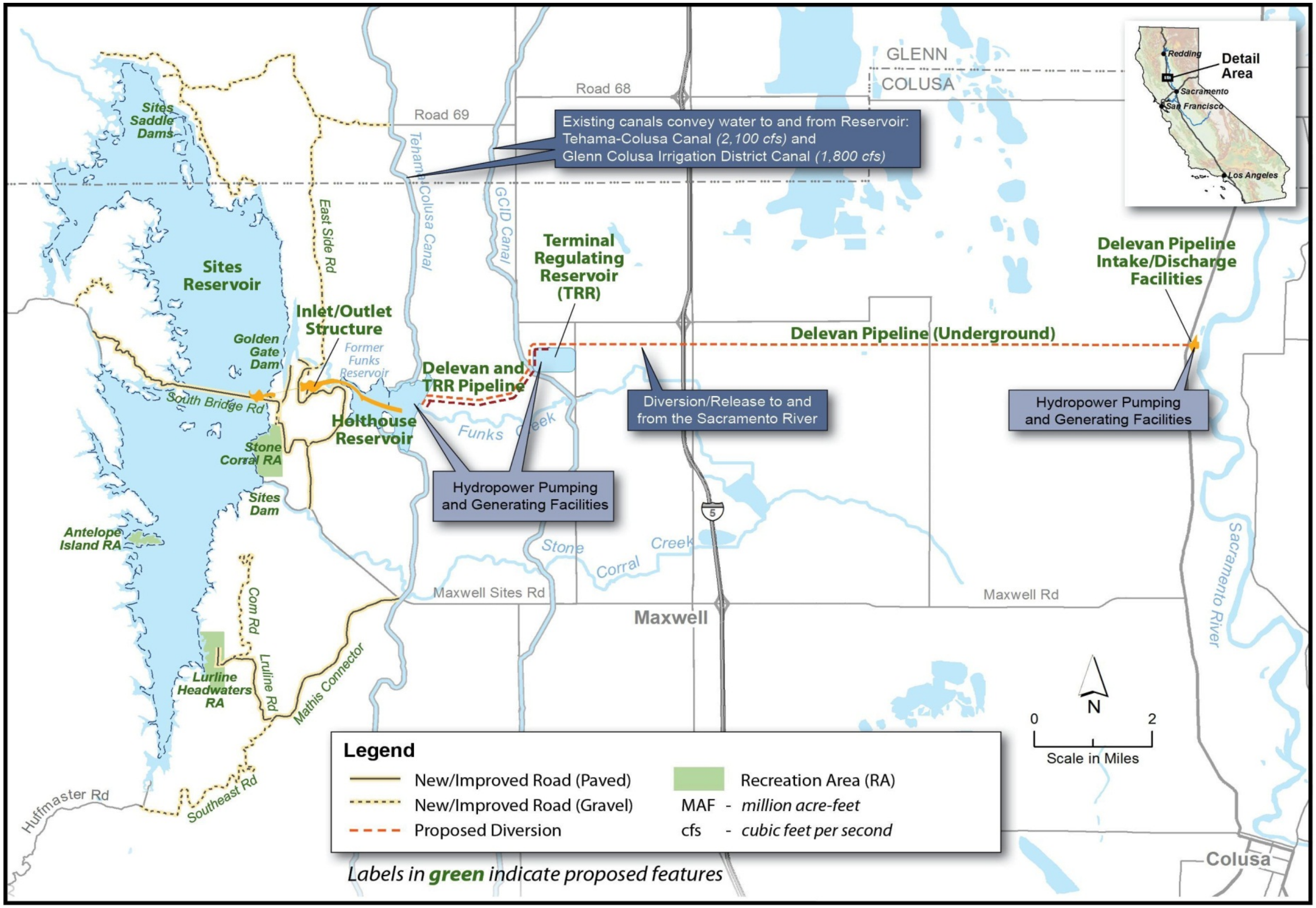

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Figure 3-33. Schematic Layout of New Sacramento River Conveyance Delevan Pipeline

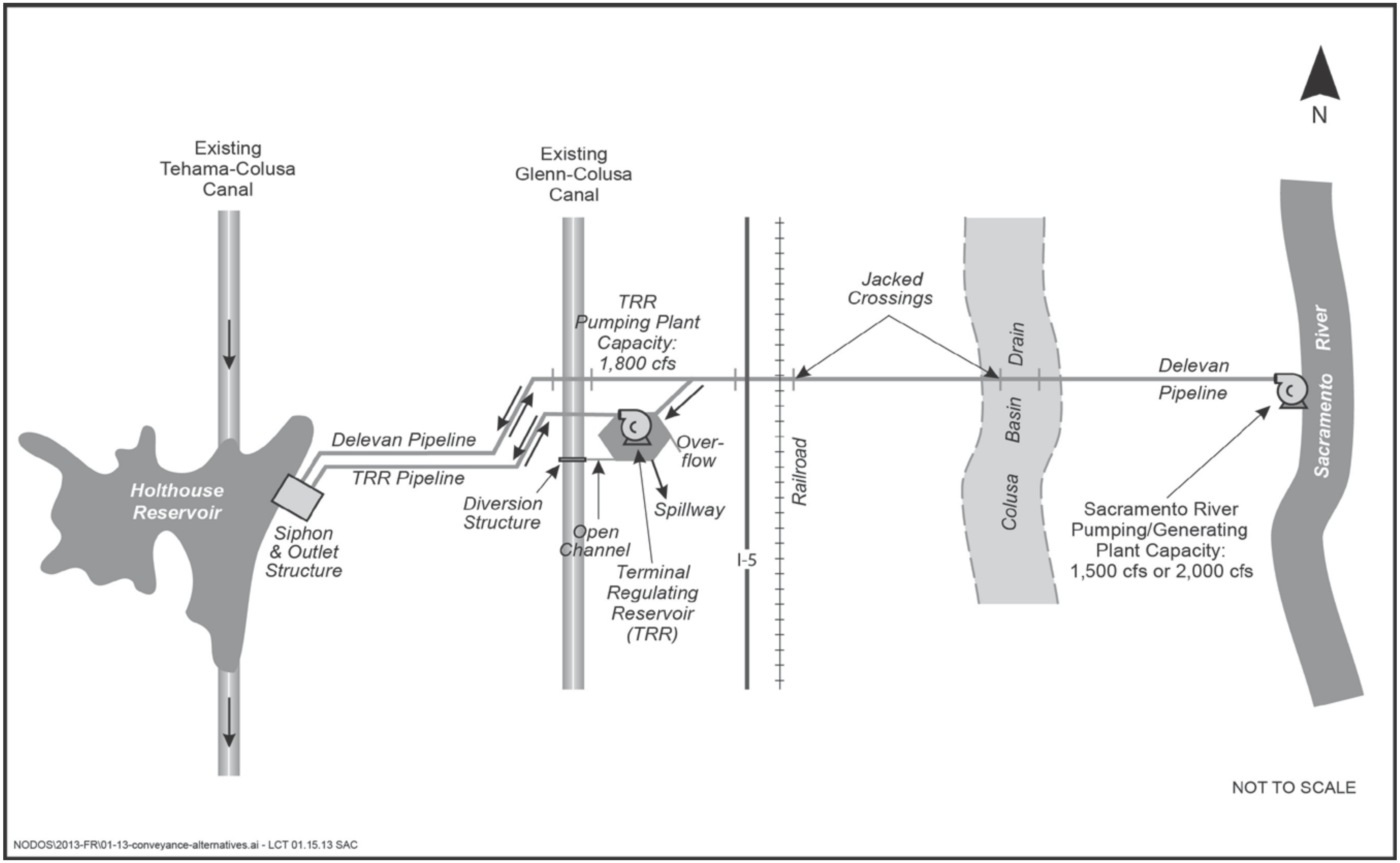

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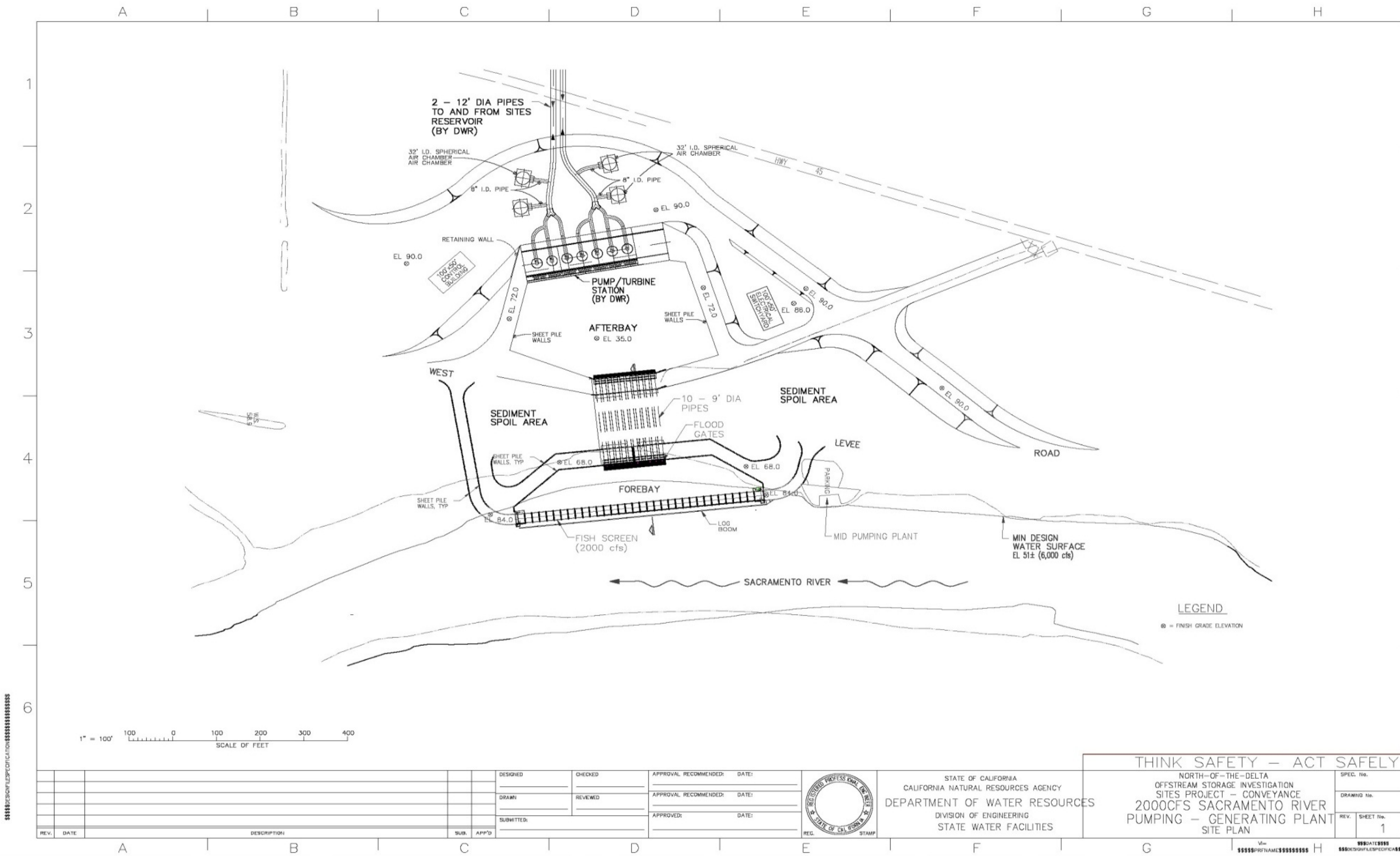
Figure 3-34. Sacramento River Pumping/Generating Plant Site Layout



PRELIMINARY- SUBJECT TO CHANGE

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

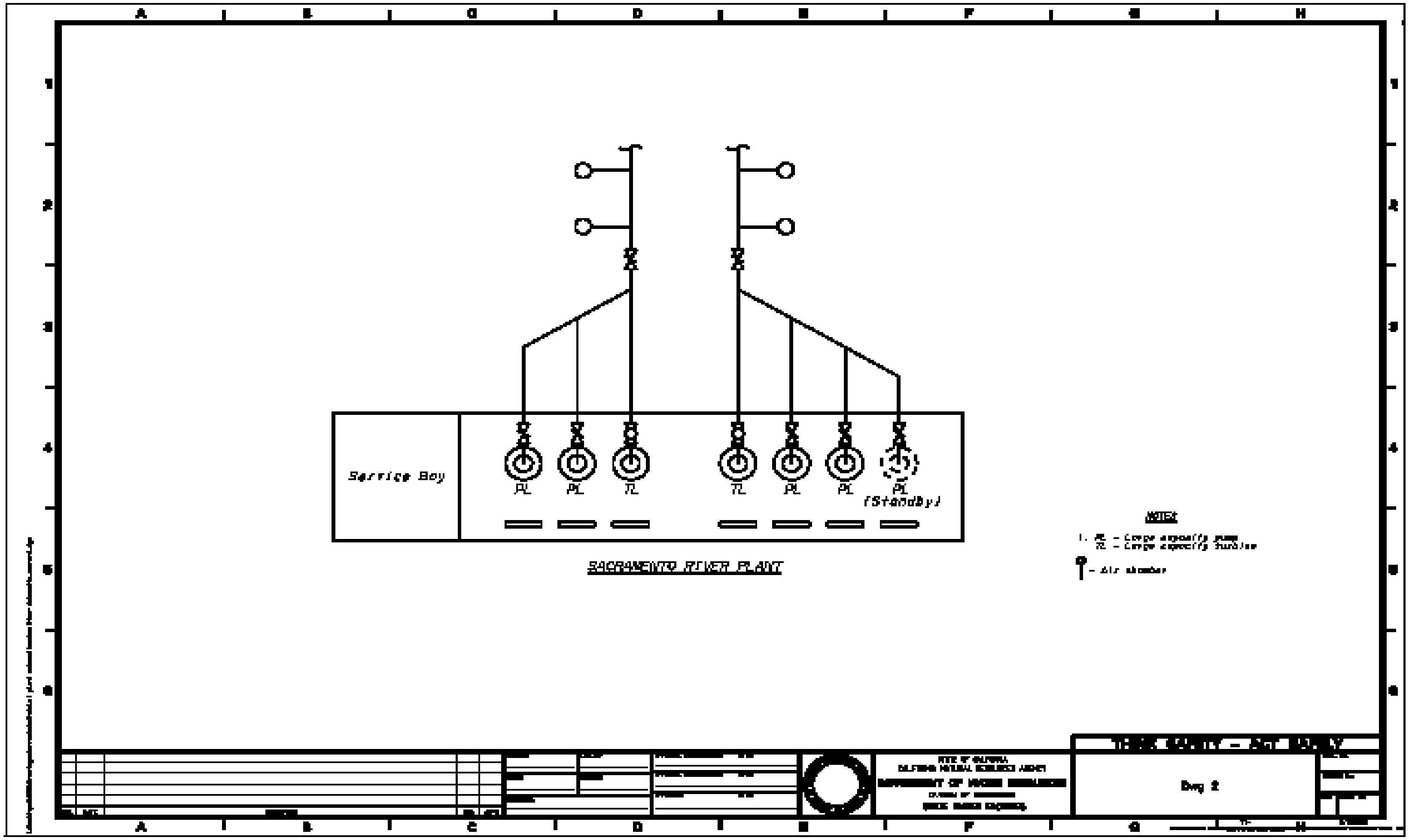


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Figure 3-36. Sacramento River Pumping/Generating Plant Lower Elevation Plan

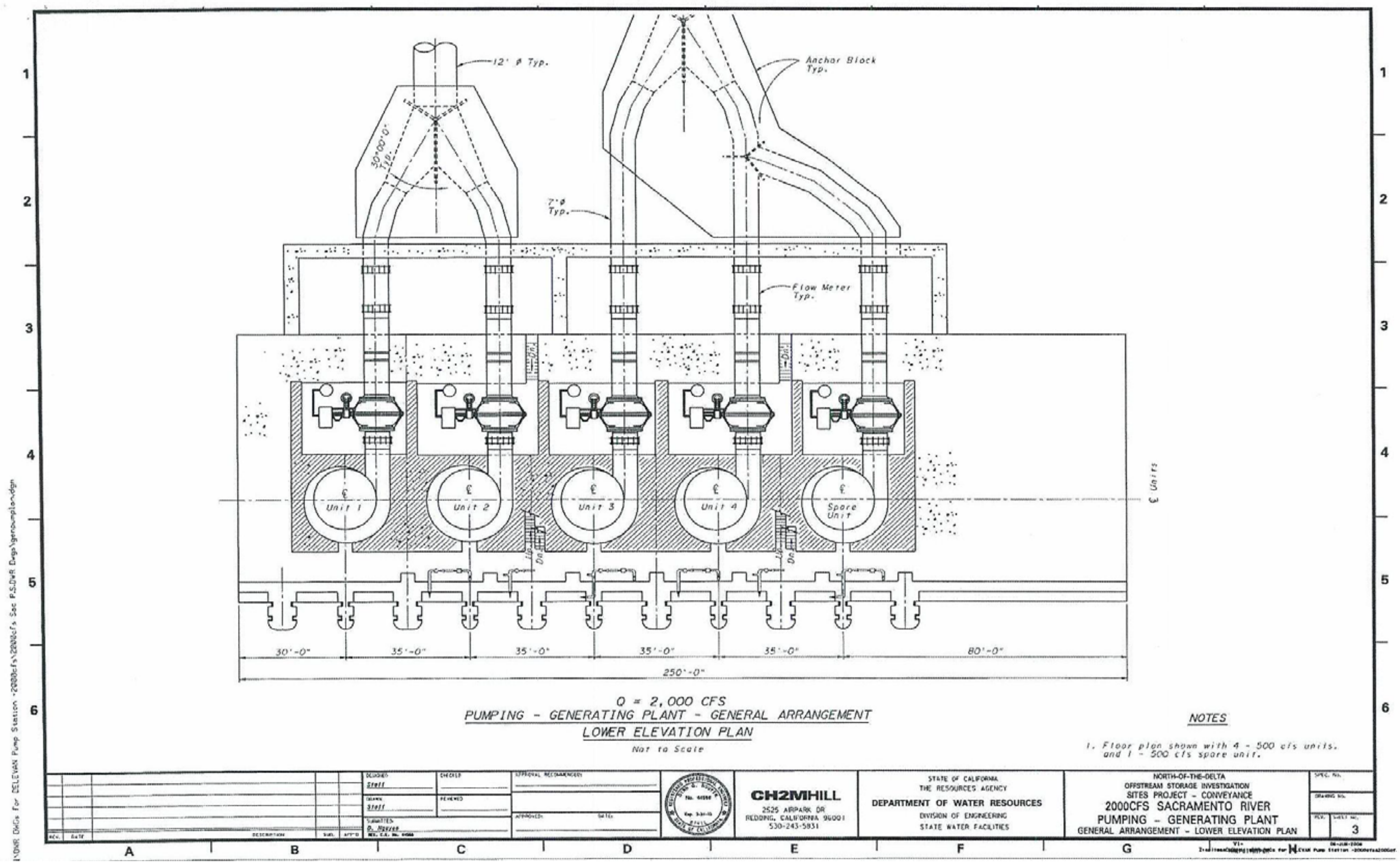

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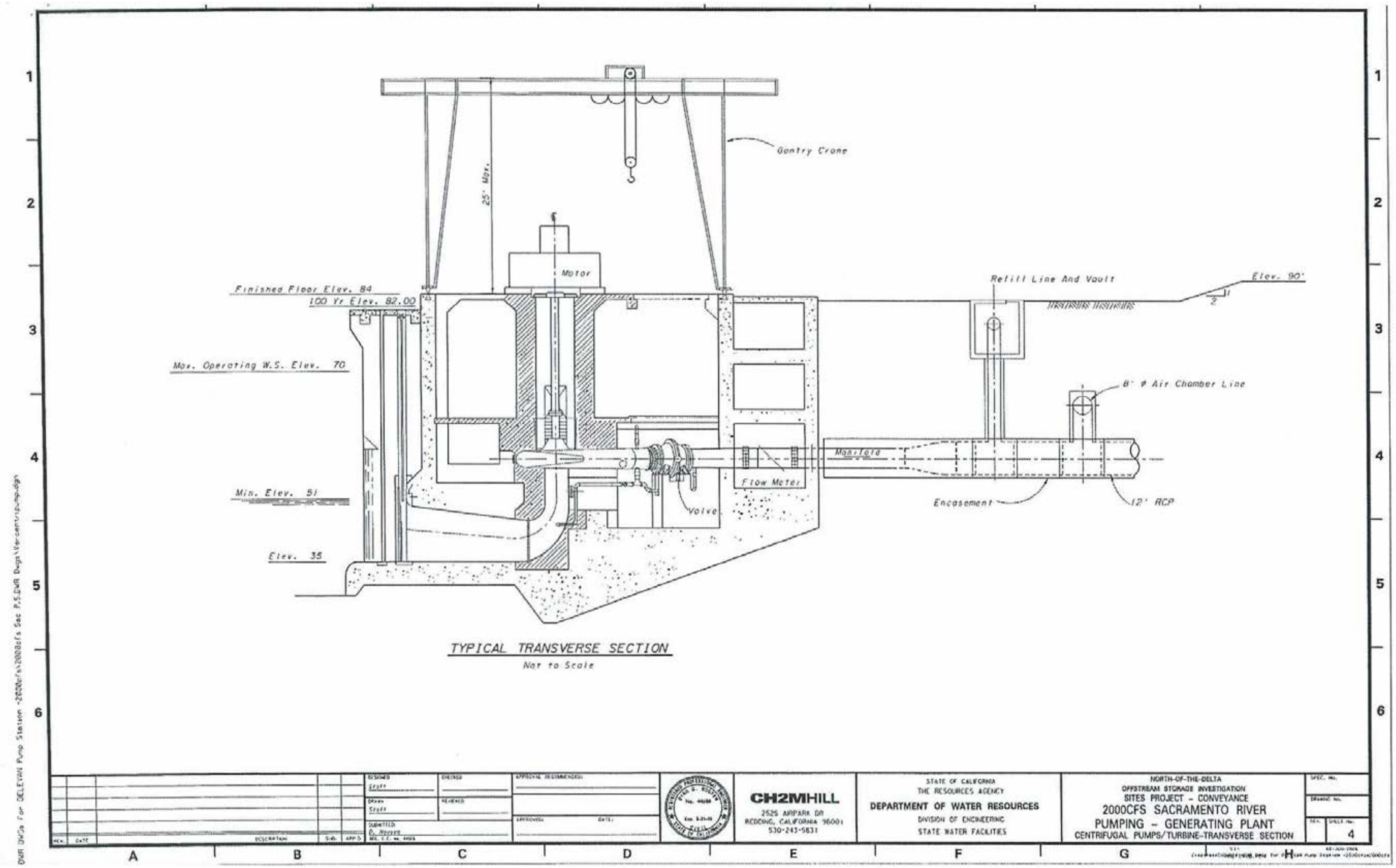
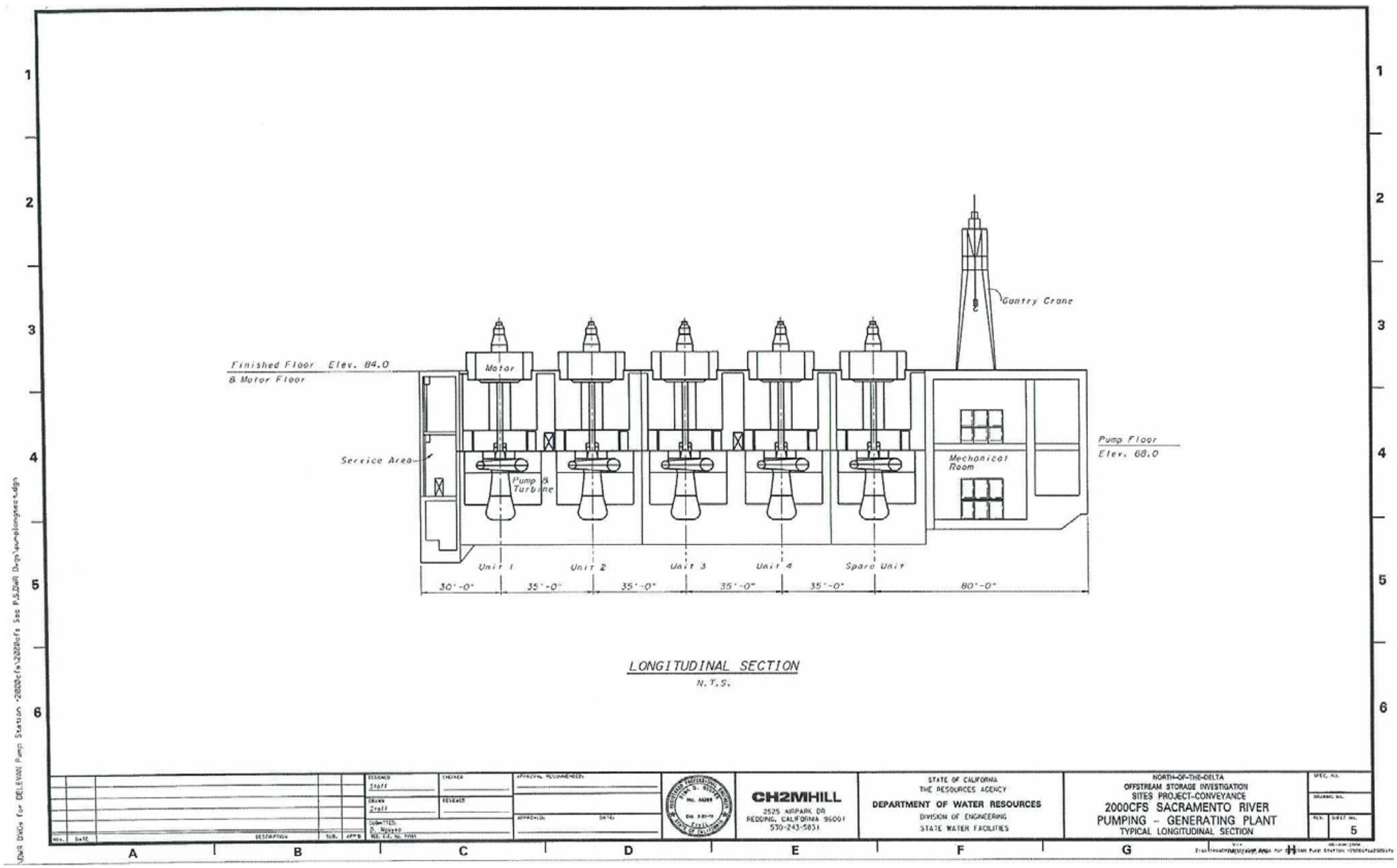
Figure 3-37. Sacramento River Pumping/Generating Plant Transverse Section



PRELIMINARY- SUBJECT TO CHANGE

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



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

Table 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

An afterbay (approximately 100,000 square feet) with bottom elevation of 35.0 feet also would be constructed. The fish screen facilities would be 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

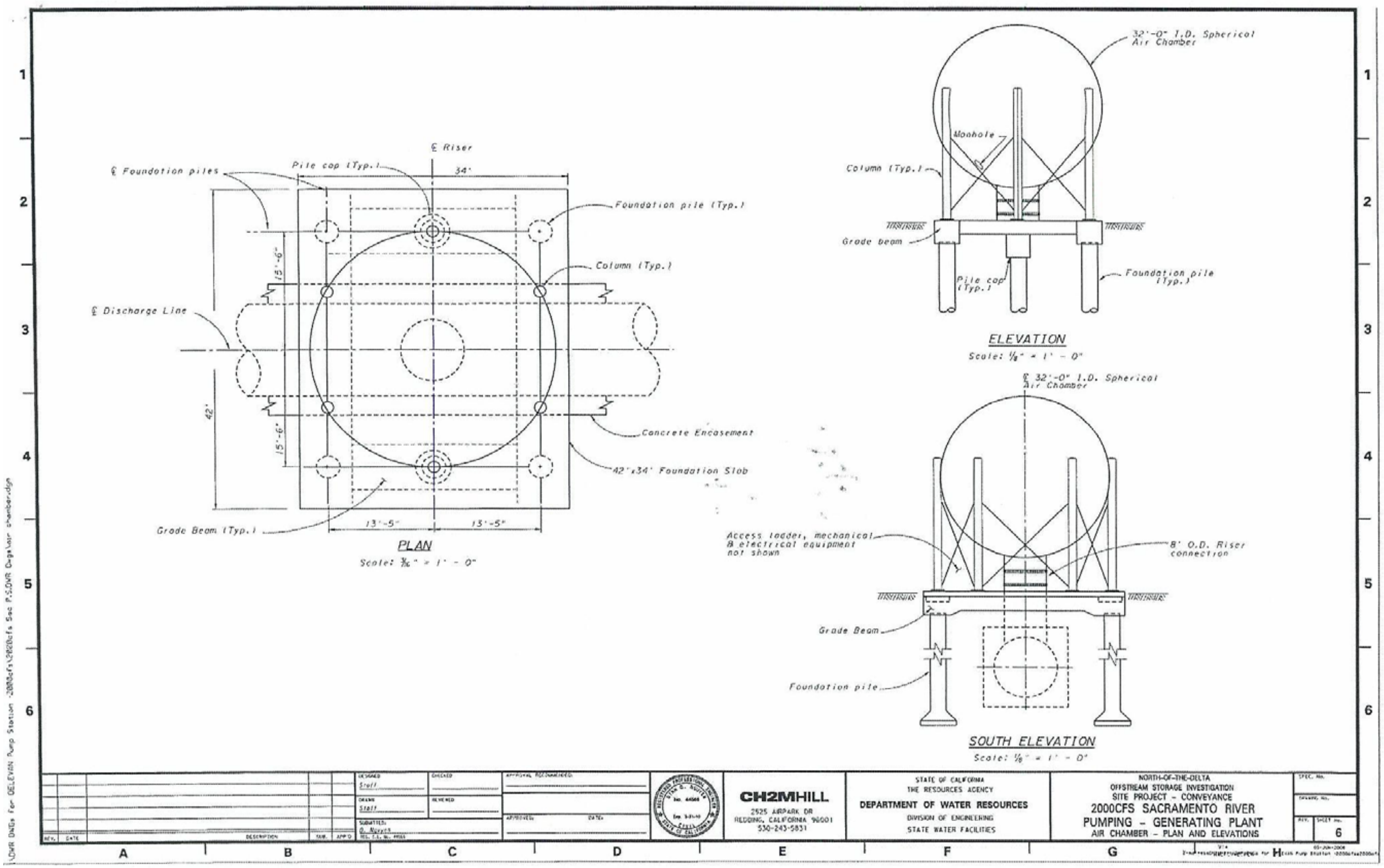
An air chamber (Figure 3-39) will be required on each discharge line to reduce the surge pressures in the pipeline. The air chamber is sized so that the upsurge pressure is limited to 125 percent of the rated pumping head (256 feet).

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 avoid water hammer.

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.

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



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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

3.8.2 Facility Location Selection

Site Geomorphology

This section summarizes the *June 2008 Sacramento River Fish Screen Facility Feasibility Study* findings.

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 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 immediately upstream of 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 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 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 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 3-41). If this trend continues, the point bar could eventually envelop the pumping facility location, cutting it off from the river channel.

Subsequent study by DWR indicates that as of 2012, the point bar is located approximately 430 meters north of the proposed intake facility location. A second point bar in the vicinity of the proposed intake facility is located directly across the river on the east bank.

The upstream point bar moved downstream approximately 120 meters between 1958 and 1976, or the equivalent of 7 meters per year. In 1981, the U.S. Army Corps of Engineers installed bank protection along the bank across from the point bar, and then extended the bank protection both upstream and downstream in 1987. Between 1976 and 1990, the point bar moved an additional 45 meters, but has not moved in the last 22 years since then.

This is a typical point bar reaction to the placement of bank protection, and it is expected that the point bar will not move in the future if the bank protection continues to function.

The downstream point bar, across from the diversion point on the east bank, also moved downstream for a distance of approximately 300 meters between 1958 and 1990, or the equivalent of 10 meters per year. During this time, the river was meandering westward at the same rate as the point bar. Around 1990, the river encountered geologic control (older, more erosion resistant geologic deposits) along the west bank, essentially stopping the migration. Since then, the point bar has not moved.

The river meander and point bar analyses conducted for this Project indicate the following:

- The upstream point bar will not continue to move downstream as long as the bank protection installed in 1981 and 1987 remains intact.

The downstream point bar across from the diversion point has not moved since about 1990. It may migrate downstream, although unlikely, but this movement would not affect the Delevan Pipeline Intake Facilities.

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

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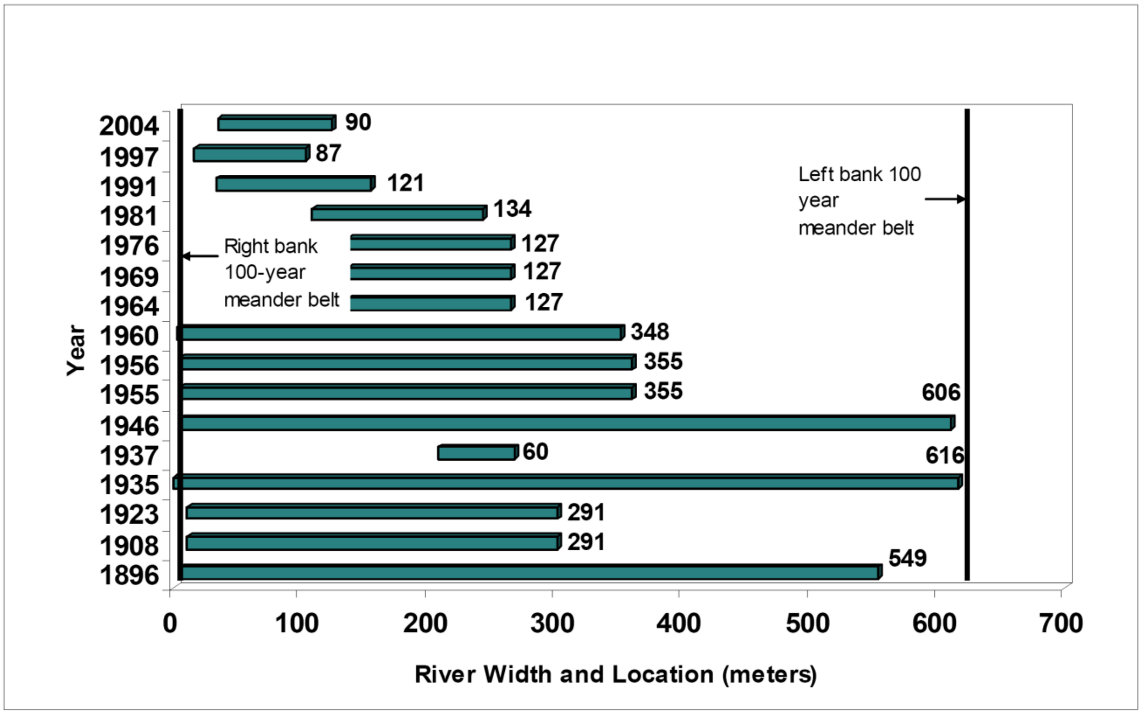
Figure 3-41. Width and Location of Sacramento River at Moulton Weir (1896 to 2004)



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It is however possible that the intake facilities may become a locus of deposition. A possibility exists that the point bar across from the intake facilities may develop a cutoff channel along the eastern boundary, essentially turning the existing point bar into an island. This is not an uncommon occurrence on the Sacramento River once a bend has bank protection installed or encounters geologic control (geologic control and the fish screens will have the same effect as bank protection). Normally, this type of cutoff forms during large flood events.

There is evidence of an incipient high flow channel, and periodic maintenance dredging occurs at this site to facilitate flow into Moulton Weir during floods. If a cutoff should occur along the high flow channel alignment, and this channel becomes the main channel, then the Delevan diversion point would be in a backwater area. Deposition may then occur along the length of the fish screens, and sweeping flow velocities along the screens may not be sufficient to meet established criteria (this is similar to the conditions at the GCID diversion near Hamilton City where the diversion channel requires periodic dredging to maintain proper function). The probability is low that a cutoff will occur. Should a cutoff occur in the future, periodic dredging could be required to maintain connectivity with the river and ensure adequate sweeping velocities for proper screen function.

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

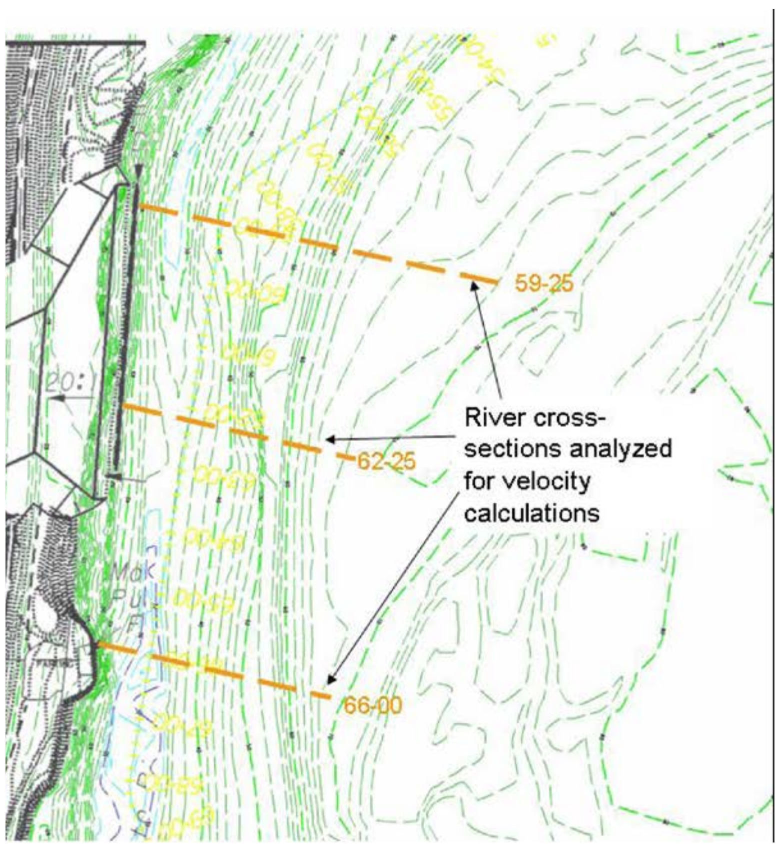
Previous work by DWR in 2007 and related DWR experience along the Sacramento River both upstream and downstream of this site indicate that sweeping velocities will not be problematic for this project. The location of this site on a stable, outside river bend is favorable for sweeping velocities, fish 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 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 Model 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 3-42. Plan View of Project Location with Cross-Sections

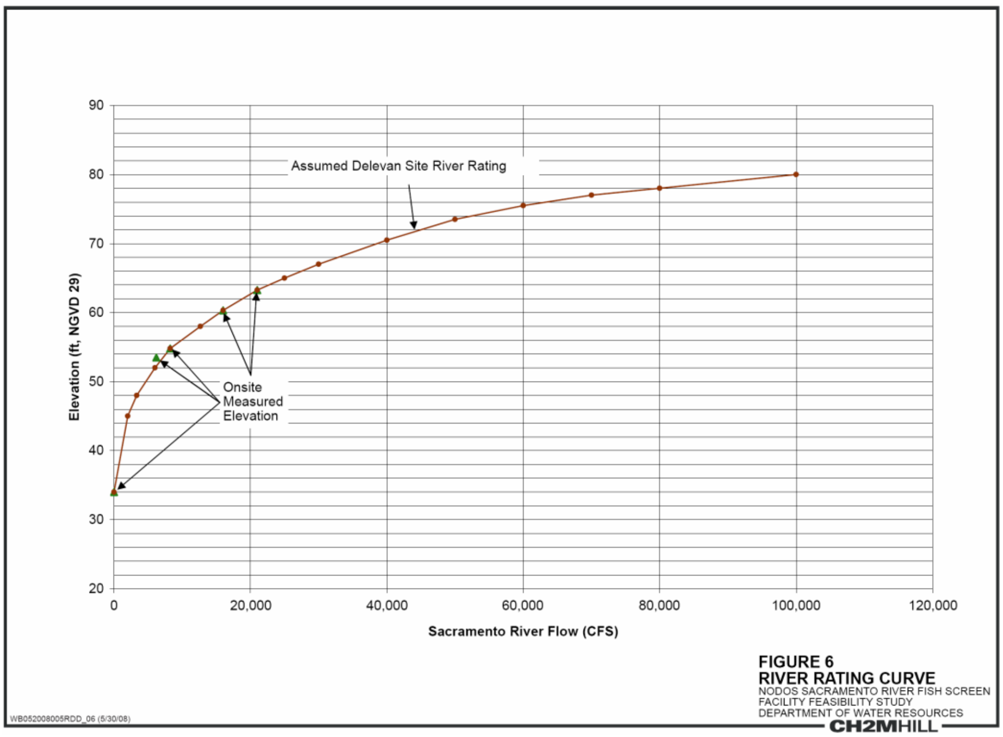


Development of the Rating Curve and Monthly Flow Duration

A rating curve (Figure 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 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 3-11, a rating curve (Figure 3-43) developed for the fish screen facility.

Table 3-11. On-Site Measured Points, North-of-the-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

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. Alternative 1 was deemed the best alternative for addressing predation issues, providing operational flexibility, handling large debris, handling sediment, minimizing the number of moving parts, reducing operating costs, and avoiding additional discharge facilities. There is also more available performance data for the flat plate fish screen. This reduces the risk and uncertainty (CH2M HILL, North-of-the Delta Offstream Storage – Sacramento River Fish Screen Facility Feasibility Study, June 2008). 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 (NOAA) Fisheries, DWR, and California Department of Fish and Wildlife (CDFW).

- 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.

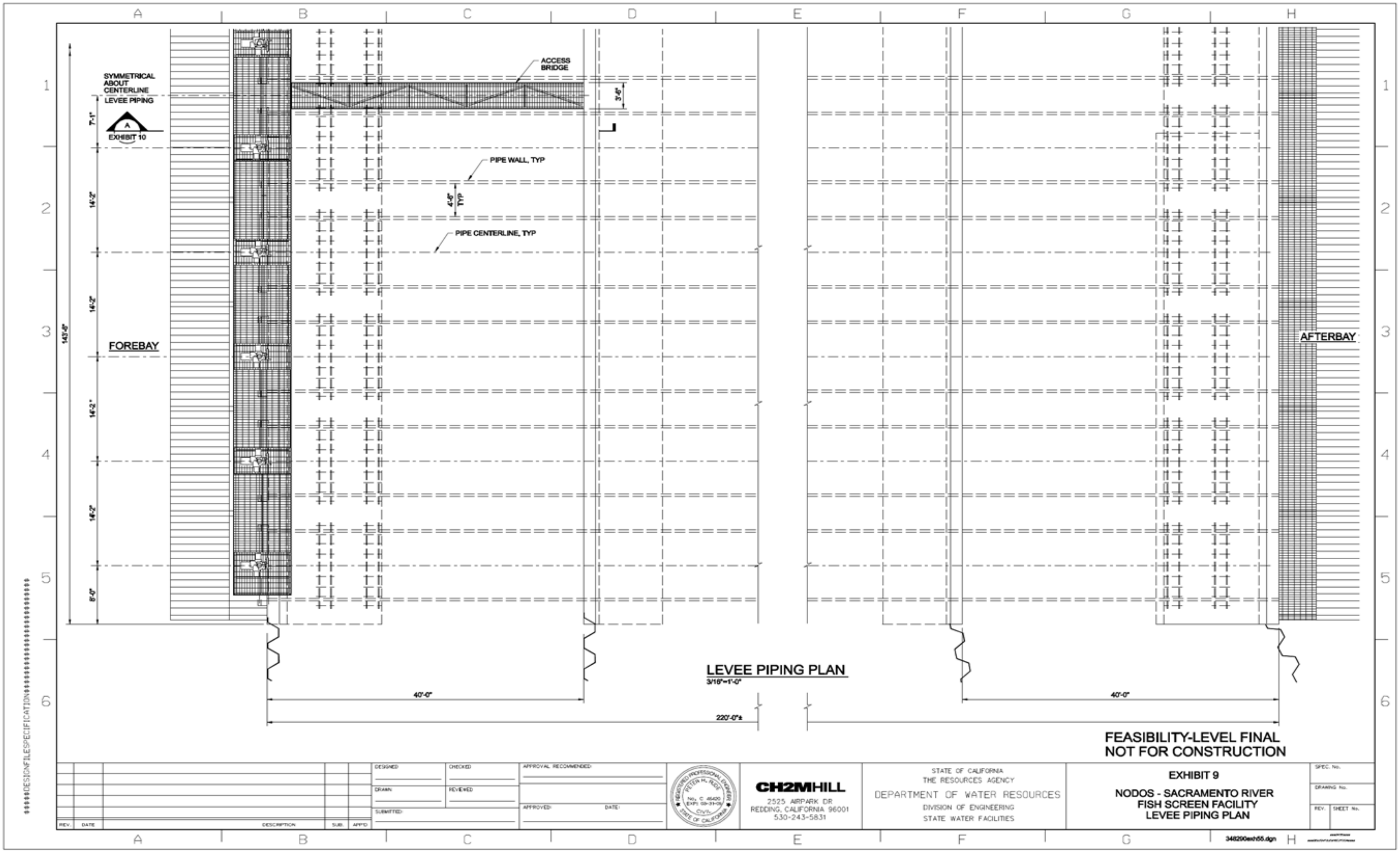
Fish Screen Structure Details

The fish screen structure, as shown on Figure 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 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 3-44. Sacramento River Fish Screen Levee Piping Plan



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The levee tubes, as seen on Figure 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 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.

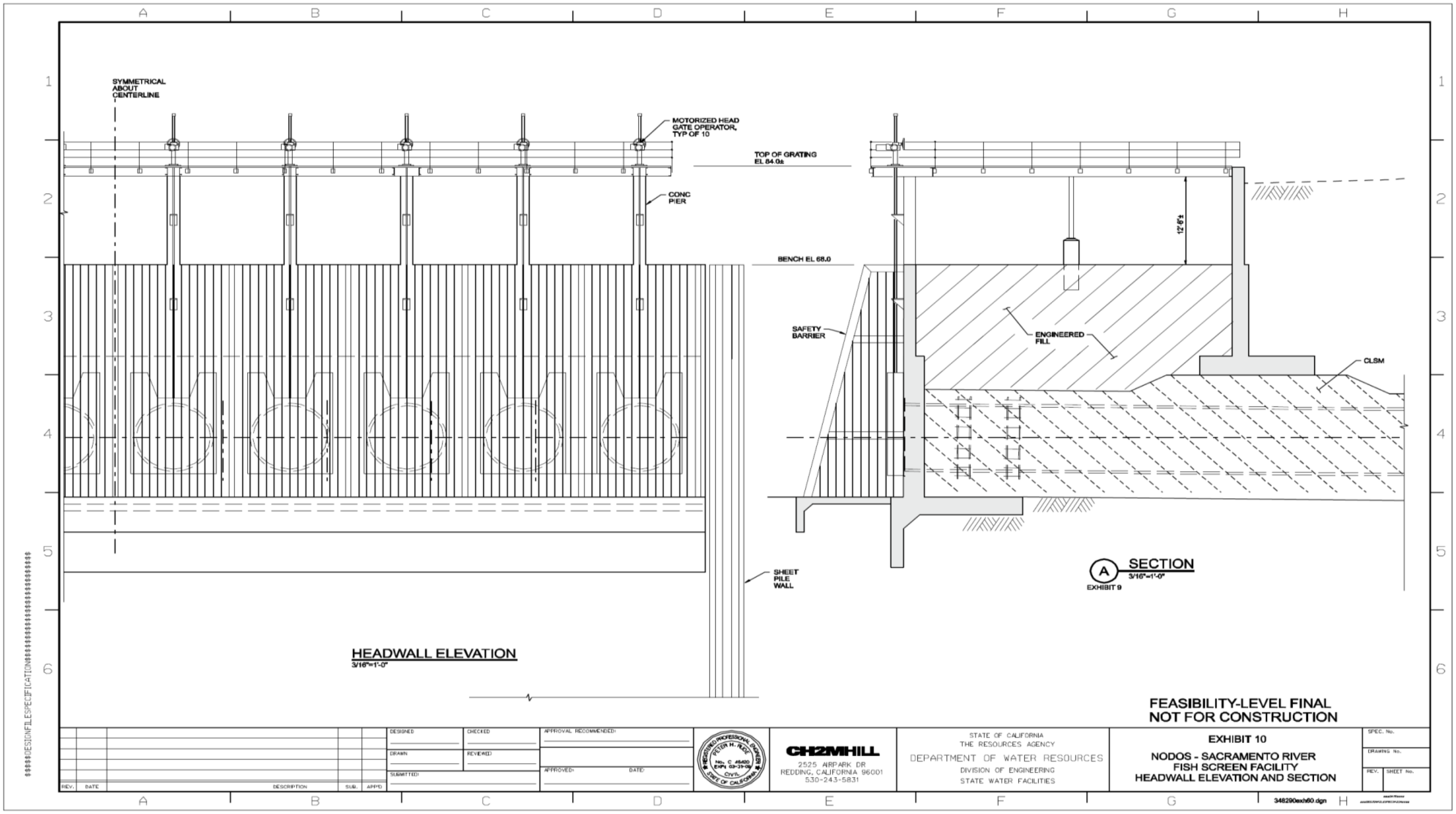
Intake Mode

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.

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



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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 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.

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 would build up on the floor of the fish screen structure, but 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 3-34.

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 3-46 and 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 designed 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.

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 adult 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 3-46. Air/vacuum valve vault details would be similar to those shown on Figure 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 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 feet.

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.90.0.

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

Figure 3-46. Sacramento River Release Structure Plan

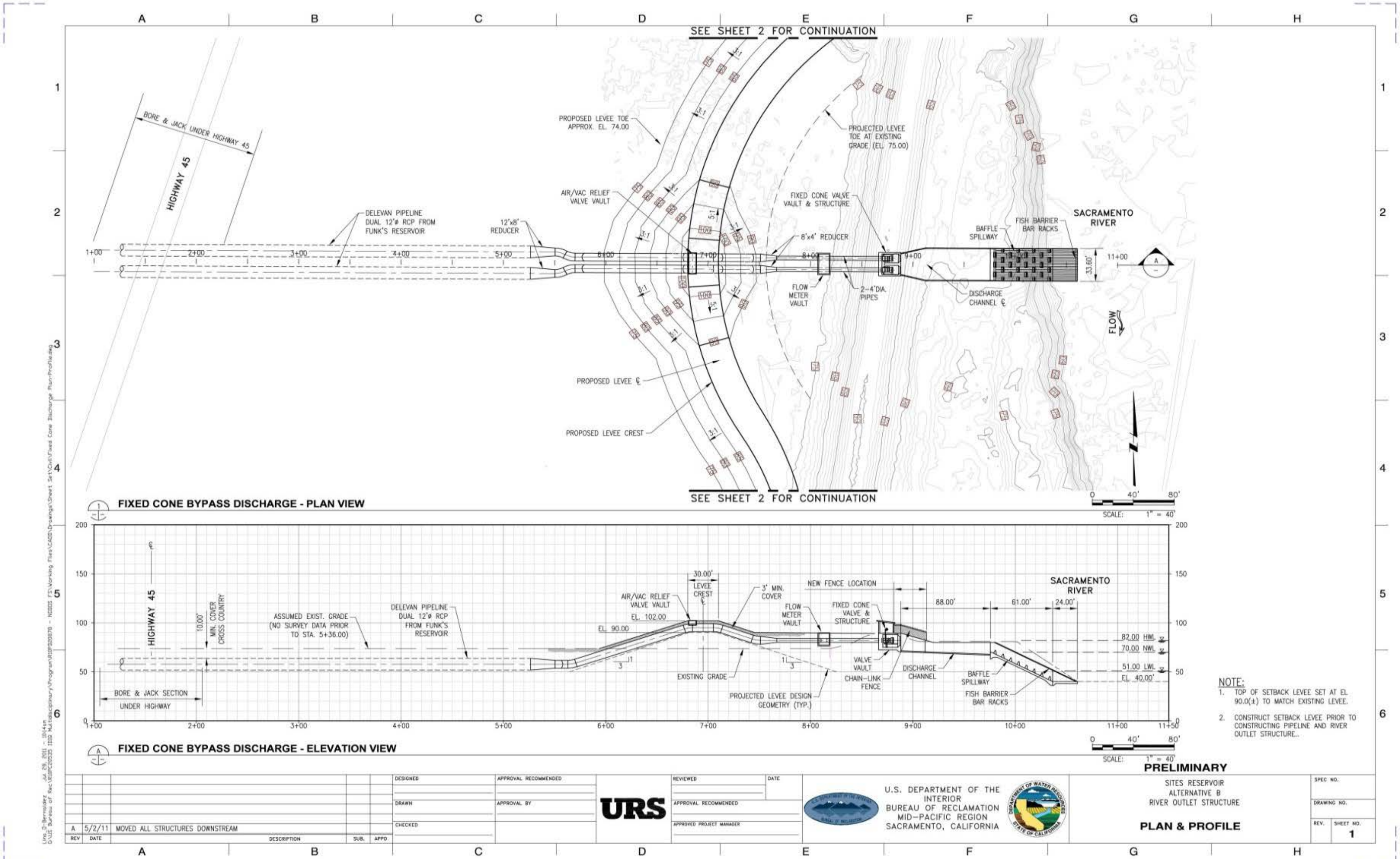

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Figure 3-47. Sacramento River Release Structure Control Valve House

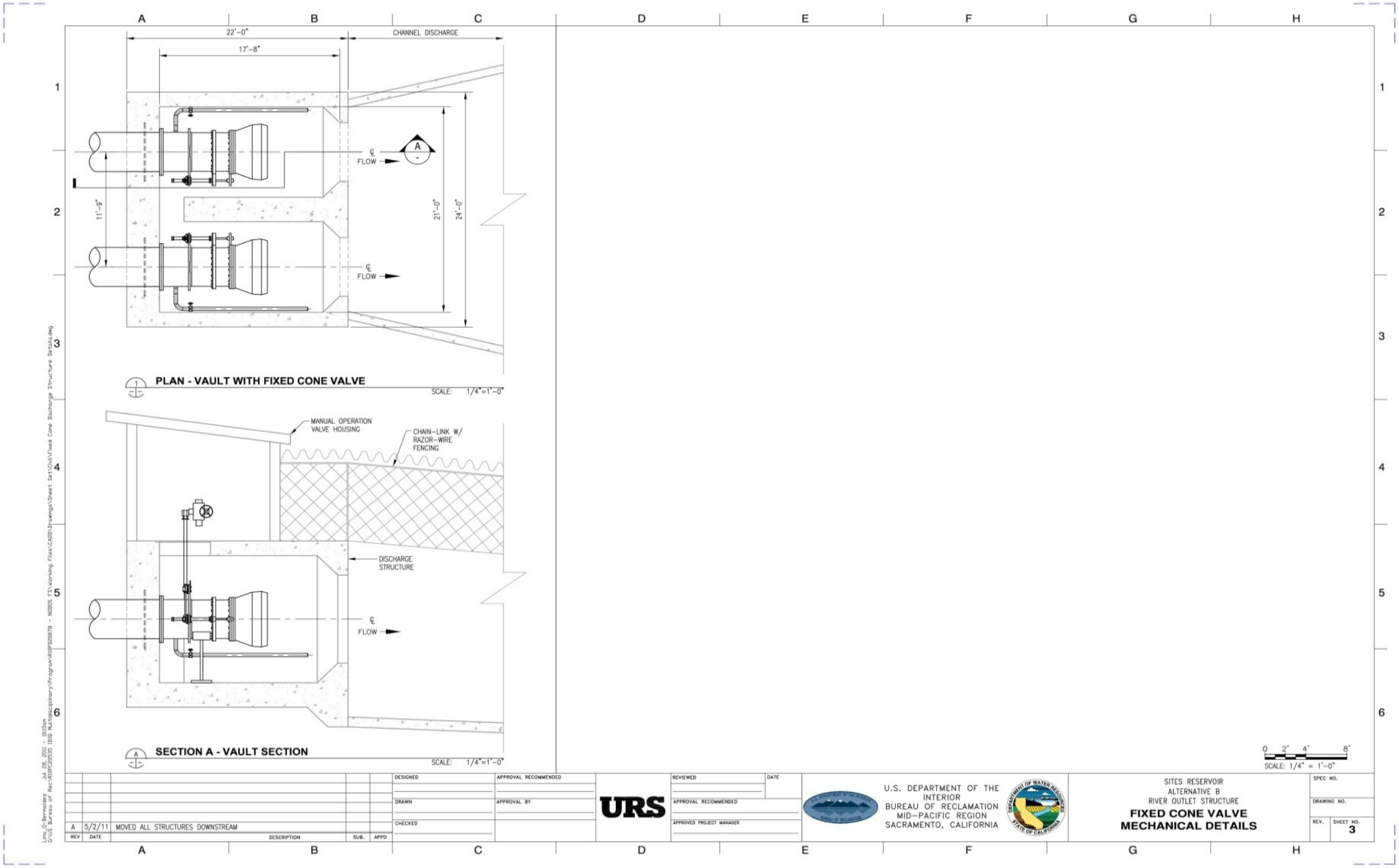

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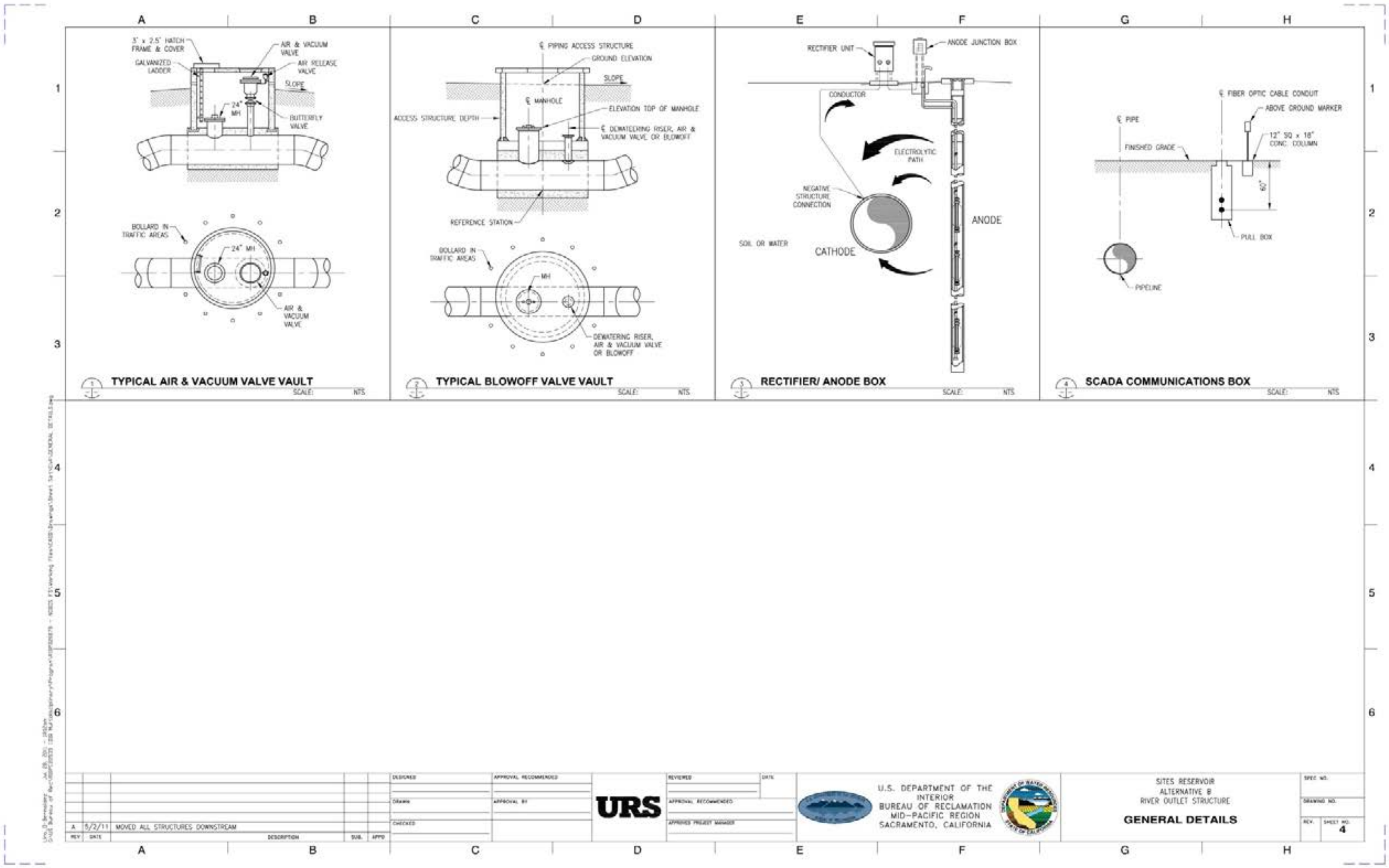
Figure 3-48. Typical Pipeline Air Valve and Blowoff Details



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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 Holthouse Reservoir to fill Sites Reservoir, and would also convey water from Holthouse Reservoir to the Sacramento River for releases. The 3.5-mile-long TRR Pipeline would convey water from the TRR to Holthouse 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 Holthouse 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 Holthouse 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 SRPGP 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 SRPGP 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 3-12 lists the proposed aboveground features and their physical characteristics.

Table 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/or 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 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 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 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 distribution areas and would not require the disturbance of additional land.

The proposed pipeline routes would also require crossing the easements of a high-voltage electrical transmission line: the Pacific Gas and Electric Company (PG&E) 230 kilovolt (kV) line. 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. Approximately 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 pipes would be fabricated on-site at the concrete batch plant. A fabrication and curing area for the pipes would be located within the 20-acre batch plant footprint. Pipes 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 the pipes would proceed year-round, and installation of pipes 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 pipes 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.

3.9 Road Relocations and Access Roads

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 up to five new recreation areas, road access to these sites would also be needed.

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.

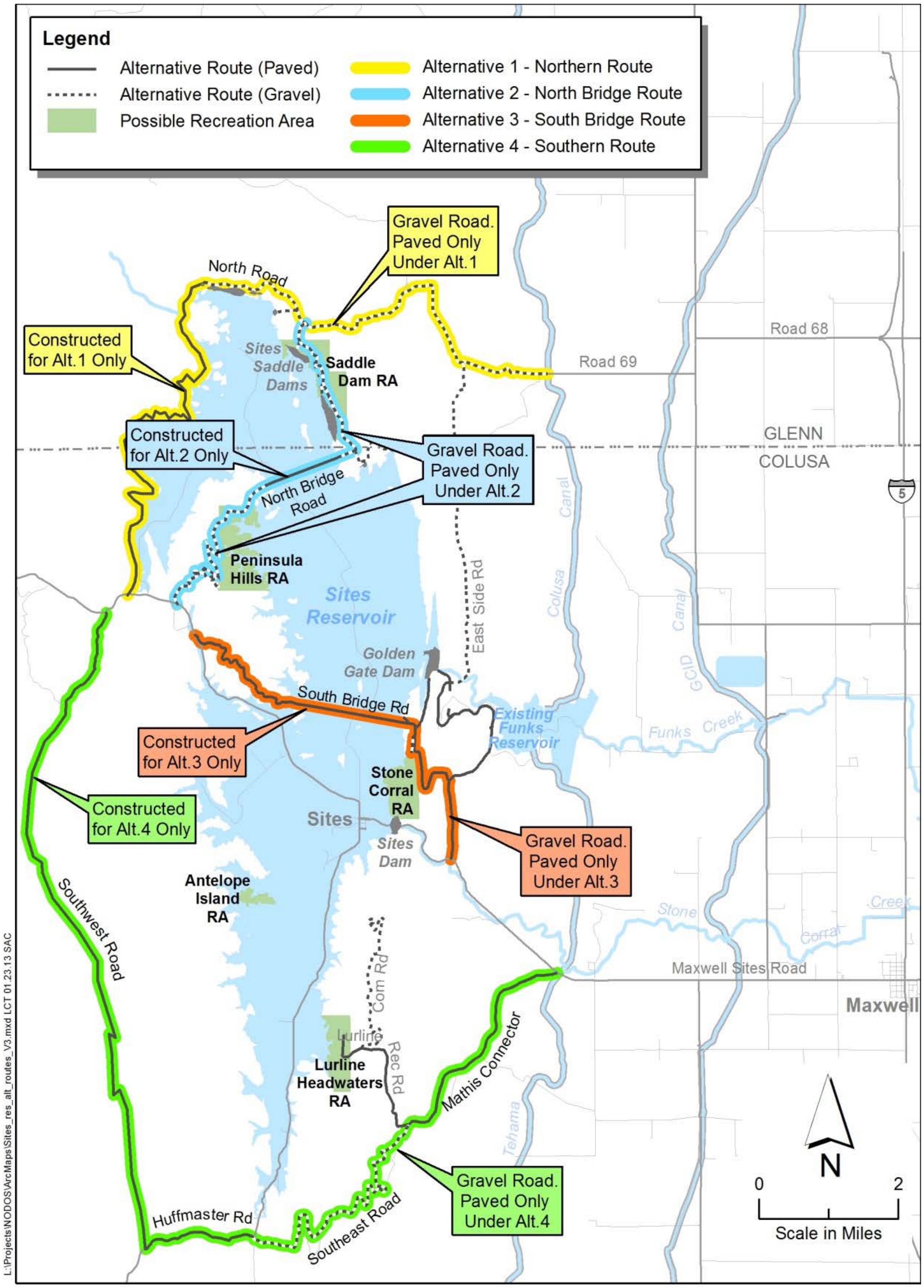
Alternative Alignments

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 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

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



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Conceptual Bridge Designs

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

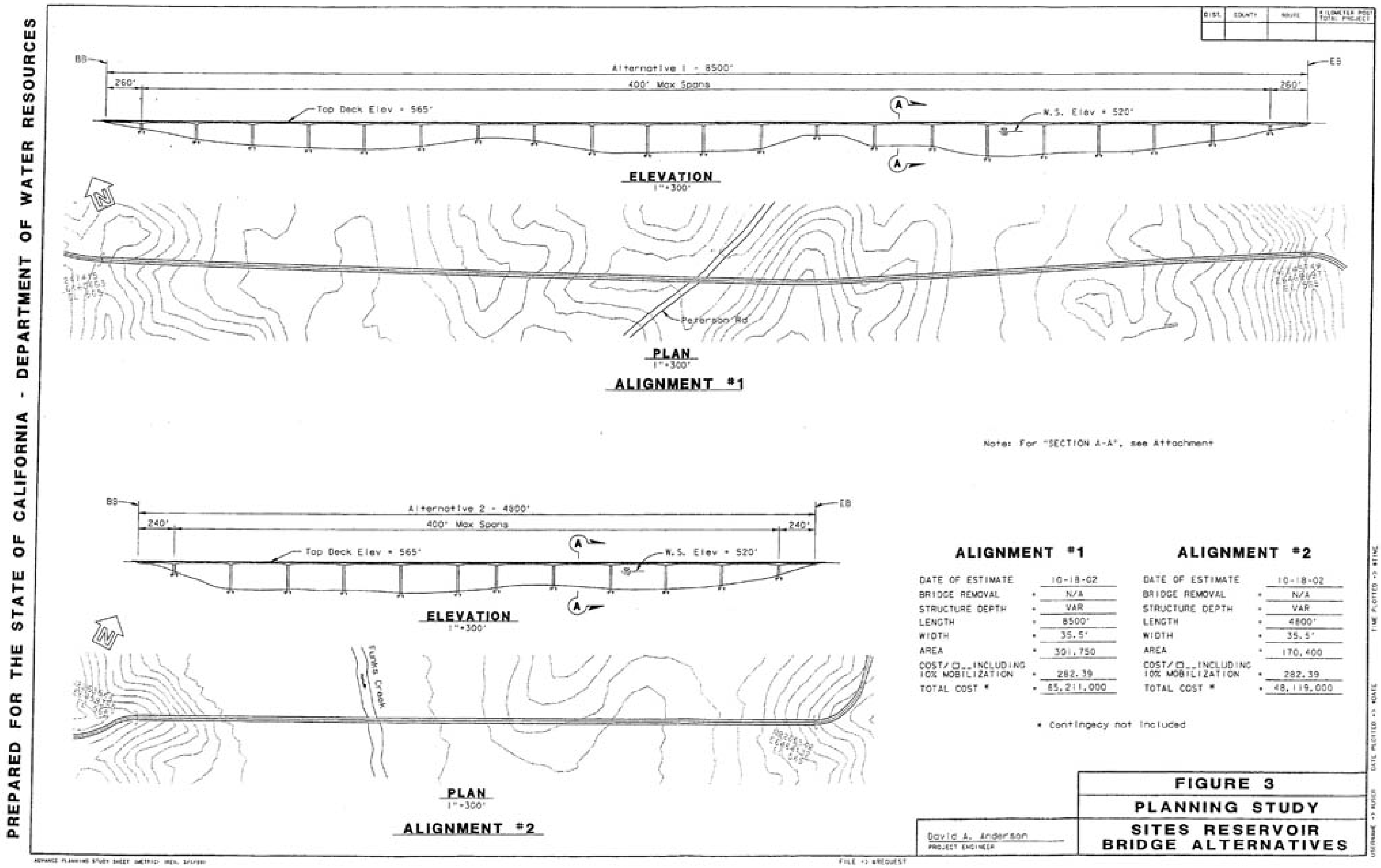
Bridge Type

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

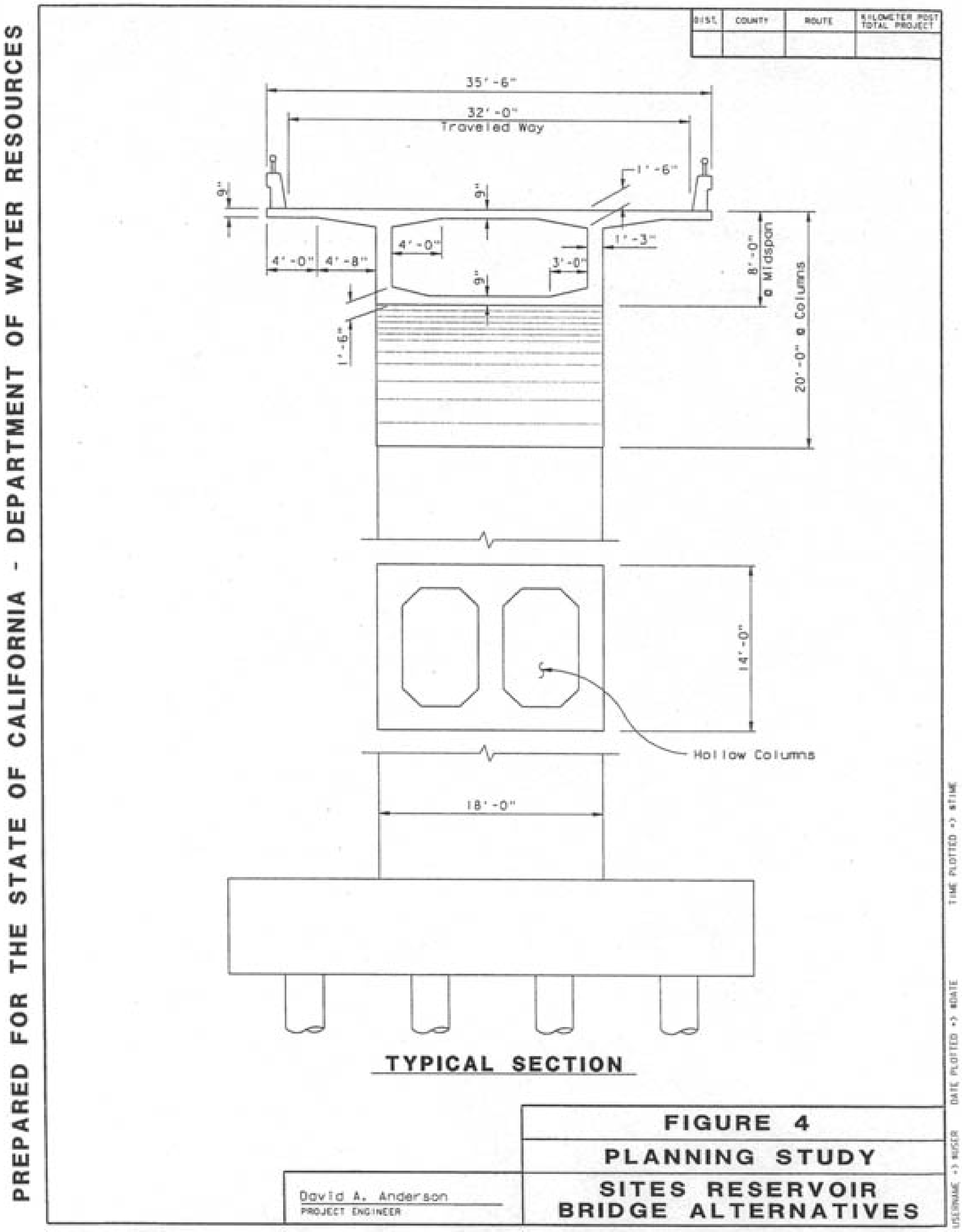
A 32-foot clear bridge width was chosen as it pertains to Rural Collector Roads. The width for the contiguous rural collector road assumes two 12-foot lanes and 4-foot shoulders, based on a relatively high hourly traffic volume that might occur during summer months with recreational traffic and given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

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



PRELIMINARY- SUBJECT TO CHANGE

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



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 3-13.

Table 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 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.

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 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 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 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.

3.9.4 Electric Utilities and Utility Relocations

Project Electrical Utility Requirements

Transmission Lines

Sites Reservoir Pumping/Generating Plant

The 230 kV system starts in the Sites Electrical Switchyard where a 230 kV 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.

<u>Sacramento River Pumping/Generating Plant</u>

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.

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. A Proposed Take Line has been established to provide buffer and access.

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 T-C Canal modifications. Reclamation currently owns the lands occupied by and adjacent to RBDD, the T-C Canal, and the Corning Canal. 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 GCID Canal, and related structures and facilities. Additional temporary easements and ROW agreements would be needed for the TRR earthwork borrow and disposal sites, and contractor staging areas. Approximately 200 acres of new ROW would specifically be needed for the TRR-Funks Pipeline.

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.