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California State Water Project

Volume III
Storage Facilities

Bulletin Number 200

November 1974

State of California
The Resources Agency
Department of Water Resources

STATE OF CALIFORNIA
The Resources Agency

Department of Water Resources

BULLETIN No. 200

CALIFORNIA STATE WATER PROJECT

Volume III

Storage Facilities

NOVEMBER 1974

NORMAN B. LIVERMORE, JR.
Secretary for Resources
The Resources Agency

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FOREWORD

This is the third of six volumes which record aspects of the planning, financing, design, construction, and operation of the California State Water Project. The subjects of the other volumes are: Volume I, History, Planning, and Early Progress; Volume II, Conveyance Facilities; Volume IV, Power and Pumping Facilities; Volume V, Control Facilities; and Volume VI, Project Supplements.

The State Water Project conserves and distributes water to much of California's population and irrigated agriculture. It also provides electric power generation, flood control, water quality control, new recreational opportunities, and enhancement of sports fisheries and wildlife habitat.

Construction of the first phase of the State Water Project was completed in 1973. The $2.3 billion reimbursable cost is being repaid by the water users and other beneficiaries. It is expected that another $0.7 billion will be spent during the next decade to construct authorized facilities for full operation.

This volume discusses storage facilities of the State Water Project and how the various dams and reservoirs function in the Project as a whole. The individual dams and reservoirs are described in detail, the extent of coverage varying with the size, importance, and uniqueness of the individual facility. Geologic conditions at each site are discussed and construction highlights are presented.

John R. Teerink

John R. Teerink, *Director*
Department of Water Resources
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ABSTRACT

The storage facilities of the State Water Project are discussed in this volume. Twenty reservoirs and their associated dams are now in operation. They are located throughout the Project over a distance of about 650 miles. Three additional dams will be constructed in the future to complete all authorized storage facilities; however, these are not included in the discussion of the individual storage facilities presented in this volume.

Five of the existing dams were designed and constructed by other agencies: four by the U. S. Bureau of Reclamation as part of the Federal-State Joint-Use Facilities and one by the City of Los Angeles Department of Water and Power. All of these facilities constructed by others were partially funded by the Department of Water Resources, and the Department also is their operator, except for the small forebay constructed by Los Angeles Department of Water and Power.

The more interesting and unique aspects of the design and construction details of each dam are discussed under the appropriate headings. Included are descriptions of site geology, seismicity, embankments, outlet works, spillways, and equipment.

The volume is written in the language of engineers and engineering geologists engaged in design and construction acitivities throughout project development. Highly technical discussions and extensive details are avoided in an attempt to interest the largest cross section of readers. Design analyses and alternatives studied generally are included whenever they are related to major decisions and unusual physical features. Difficulties which arose during construction or after start of operations also are discussed. These difficulties probably were no greater or less than encountered by others involved in similar major projects.

Consulting firms and boards were selected and retained by the Department to provide broad experience and expertise in several areas of project work. Extensive model-testing programs designed to ensure appropriate and economic design were utilized and supervised by the Department.

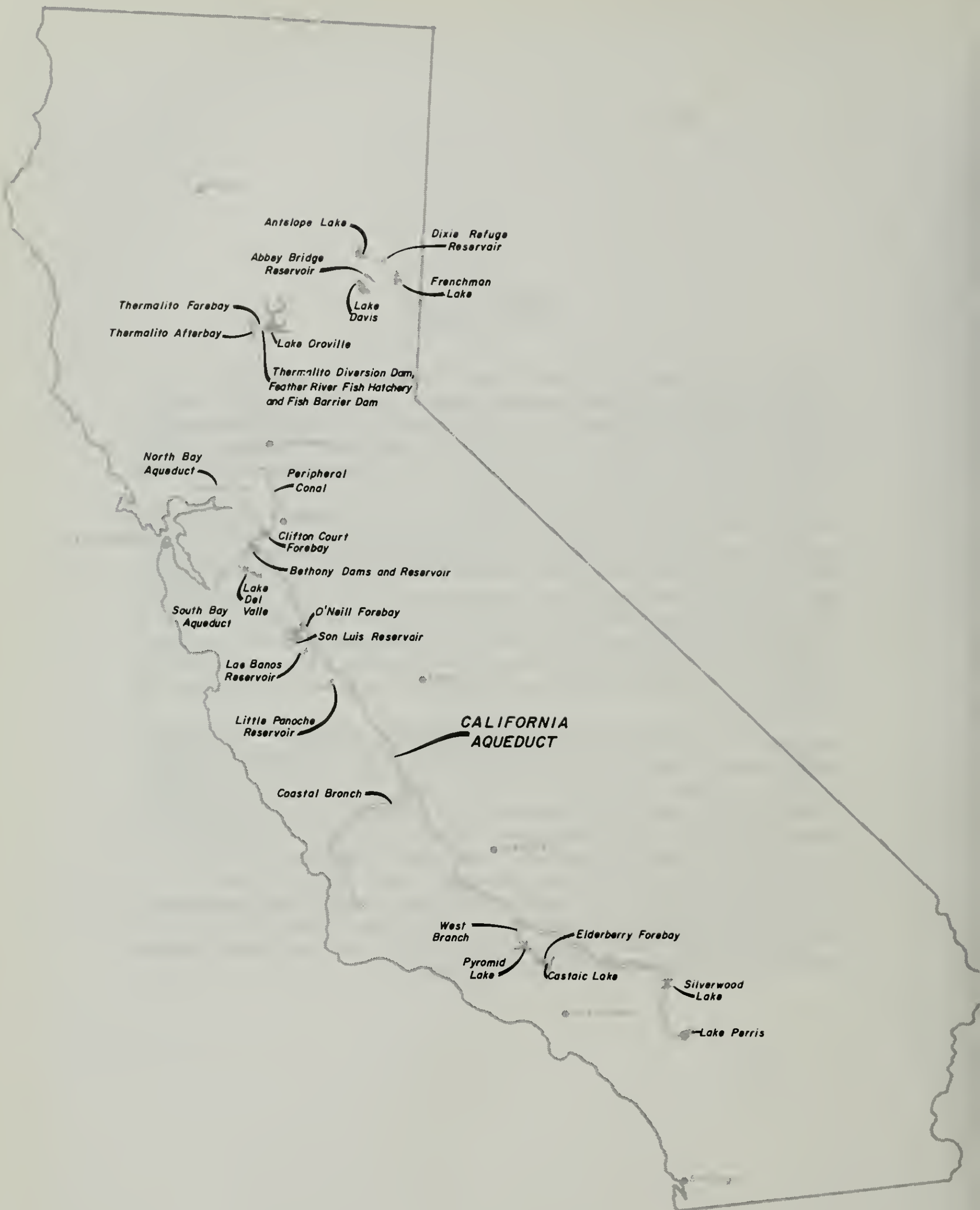

Figure 1. Location Map—State Water Project Reservoirs

CHAPTER I. GENERAL

Overview

There are 20 completed reservoirs in the State Water Project, the locations of which are shown on Figure 1. Fifteen were built and are operated by the Department of Water Resources, and four (San Luis Joint-Use Facilities) were built by the U.S. Bureau of Reclamation and now are operated by the Department. The last of the 20 completed reservoirs, Elderberry Forebay, was built and is operated by the City of Los Angeles Department of Water and Power as part of the Pyramid-Castaic power development. Table 1 presents a brief statistical summary of the 20 completed reservoirs and their dams, all of which are discussed in this volume.

The first project dams to be constructed were Frenchman in the Upper Feather River watershed and Bethany Forebay at the head of the South Bay Aqueduct. Both prime contracts were let in 1959. Oroville Dam, the largest dam of the Project, was started in 1962, and reservoir storage began in November 1967. Other work on the Project proceeded southward during the 1960s and early 1970s with Pyramid, the last dam in the initial phase of the Project, completed in early 1974.

Virtually all of the project yield of 4,230,000 acre-feet annually comes from two sources: the Feather River watershed above Oroville Dam, and surplus winter and spring runoff from other watersheds tributary to the Sacramento-San Joaquin Delta. Surplus water in the Delta is pumped into San Luis Reservoir for summer and fall releases. Thus, Oroville Dam and San Luis Dam are the two key conservation features of the State Water Project.

Nine of the existing 20 reservoirs are involved in the generation of power, with all but one utilizing pumped storage. In the Oroville Division, two pumping-generating plants, Edward Hyatt Powerplant underground in the left abutment of Oroville Dam and Thermalito Powerplant, involve four reservoirs—Lake Oroville, Thermalito Diversion Pool, Thermalito Forebay, and Thermalito Afterbay. Pumping-generating plants also are located between San Luis Reservoir and O'Neill Forebay and between Pyramid Lake and Elderberry Forebay. The ninth reservoir, Silverwood Lake, is the forebay for Devil Canyon Powerplant.

Pyramid, Castaic, and Silverwood Lakes and Lake Perris are large reservoirs located near the metropolitan areas of Southern California, where water supplies primarily are imported. The three aqueducts

TABLE 1. Statistical Summary of 20 Completed Reservoirs and Their Dams

Name of Reservoir	Reservoirs			Dams			
	Gross Capacity[1] (acre-feet)	Surface Area (acres)	Shoreline (miles)	Structural Height (feet)	Crest Elevation[2] (feet)	Crest Length (feet)	Volume (cubic yards)
Frenchman Lake	55,477	1,580	21	139	5,607	720	537,000
Antelope Lake	22,566	931	15	120	5,025	1,320	380,000
Lake Davis	84,371	4,026	32	132	5,785	800	253,000
Lake Oroville	3,537,577	15,805	167	770	922	6,920	80,000,000
Thermalito Diversion Pool	13,328	323	10	143	233	1,300	154,000
Fish Barrier Pool	580	52	1	91	181	600	10,500
Thermalito Forebay	11,768	630	10	91	231	15,900	1,840,000
Thermalito Afterbay	57,041	4,302	26	39	142	42,000	5,020,000
Clifton Court Forebay	28,653	2,109	8	30	14	36,500	2,440,000
Bethany	4,804	161	6	121	250	3,940	1,400,000
Lake Del Valle	77,106	1,060	16	235	773	880	4,180,000
San Luis	2,038,771	12,700	65	385	554	18,600	77,645,000
O'Neill Forebay	56,426	2,700	12	88	233	14,350	3,000,000
Los Banos	34,562	623	12	167	384	1,370	2,100,000
Little Panoche	13,236	354	10	152	676	1,440	1,210,000
Silverwood Lake	74,970	976	13	249	3,378	2,230	7,600,000
Lake Perris	131,452	2,318	10	128	1,600	11,600	20,000,000
Pyramid Lake	171,196	1,297	21	400	2,606	1,090	6,860,000
Elderberry Forebay	28,231	460	7	200	1,550	1,990	6,000,000
Castaic Lake	323,702	2,235	29	425	1,535	4,900	46,000,000
Totals	6,765,817	54,642	491			168,450	266,599,500

[1] At maximum normal operating level.
[2] Above sea level.

which supply water to Southern California (California, Owens Valley, and Colorado River Aqueducts) cross the San Andreas fault system and likely could be disrupted, at least temporarily, by fault movement. In view of this, these four reservoirs were constructed as large as practicable to provide a reserve water supply should such an event occur.

Two reservoirs, Oroville and Del Valle, are drawn down prior to the flood season to create an adequate storage capacity to control downstream floods. Costs allocated to flood control were borne by the Federal Government. Although incidental flood control benefits accrue to many of the other reservoirs in the system, no federal flood control payments were made.

Substantial recreation benefits are derived from reservoirs throughout the Project, with the Upper Feather River reservoirs built primarily for this purpose. Recreation use, in general, has greatly exceeded predictions, and onshore recreation developments have lagged the demand.

Additional reservoirs, Abbey Bridge and Dixie Refuge in the Upper Feather River watershed and Buttes in the Mojave Division, are planned for future construction. The first two will be used primarily for recreation and the third to regulate water deliveries.

The following sections briefly describe the dams and reservoirs and relate them to the remainder of the State Water Project.

Upper Feather River Division

Beginning with the northernmost features of the Project, three of five authorized reservoirs (Figure 2)

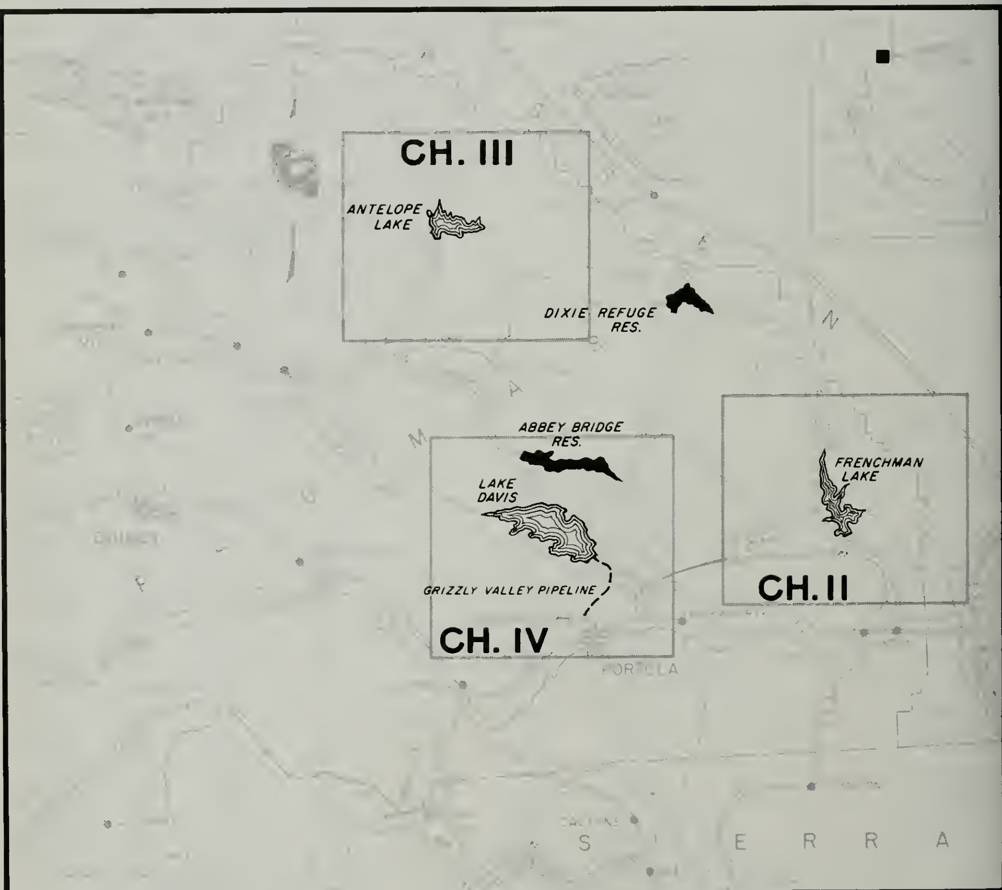

Figure 2. Upper Feather River Division

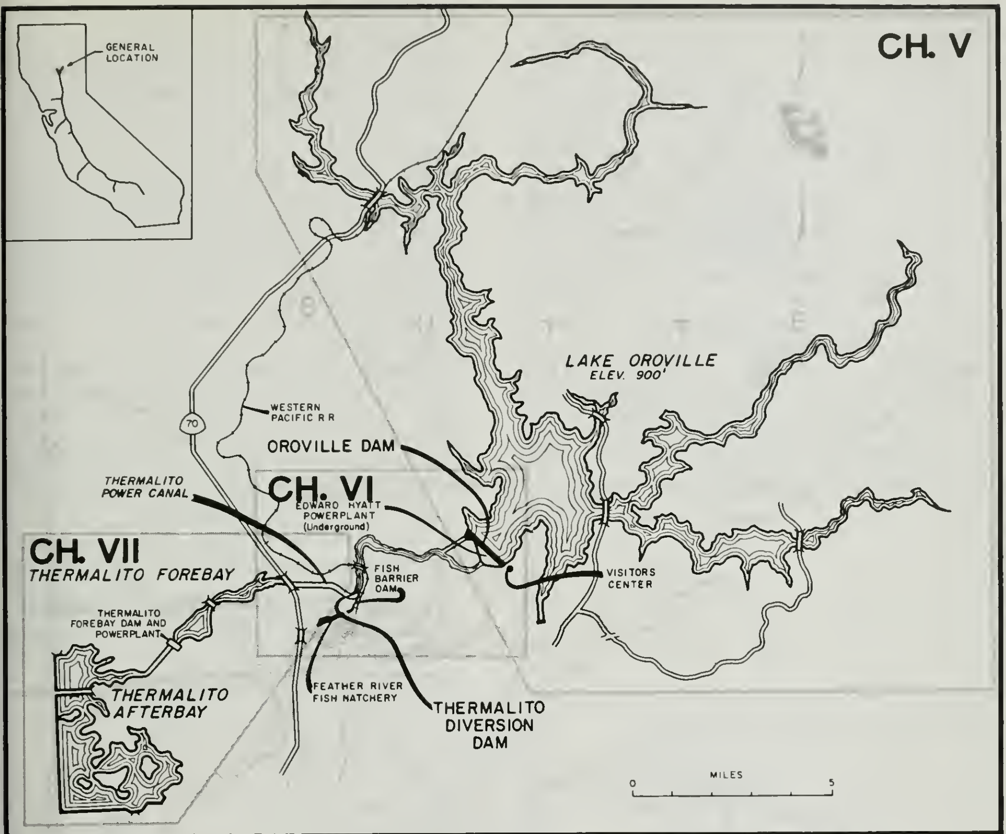

Figure 3. Oroville Division

have been completed: Frenchman Lake, Antelope Lake, and Lake Davis. All five reservoirs are, or will be, located on the upper tributaries to the Feather River—a river system with a drainage area in excess of 3,600 square miles above Oroville Dam. The three completed reservoirs have a combined storage capacity of 162,414 acre-feet and provide for local irrigation, recreation, and incidental flood control. All of the dams are of earthfill construction and vary in height from 120 to 139 feet. Frenchman Dam, the largest of the dams, is a 139-foot-high earth embankment containing 537,000 cubic yards of material. The largest reservoir in the group is Lake Davis, which has a gross capacity of 84,371 acre-feet.

Oroville Division

About 90 miles downstream on the Feather River at Oroville and about 100 miles upstream from the Sacramento-San Joaquin Delta is the Oroville Division (Figure 3). Included in this division are Oroville Dam and the Oroville-Thermalito power complex. Oroville Dam, one of the two principal conservation features of the Project, impounds 3,537,577 acre-feet of water. Included in this storage capacity is provision for flood control. The reservoir has a surface area of 15,805 acres and a shoreline of 167 miles.

This 80,000,000-cubic-yard embankment dam, which is the highest earthfill dam in the United States at the present time (1974), rises 770 feet above streambed excavation and has a crest length of 6,920 feet.

Power at Oroville Dam is produced by the Edward Hyatt Powerplant and the Thermalito power facilities, which in turn encompass a diversion dam, a power canal, a forebay, and an afterbay. Edward Hyatt

Powerplant is located underground in the left abutment of Oroville Dam. It contains three conventional generators and three motor-generators coupled to Francis-type reversible pump-turbines. The latter units provide for off-peak pumped-storage operations.

Releases from Edward Hyatt Powerplant are diverted from the Feather River by the 143-foot-high Thermalito Diversion Dam, a concrete gravity overpour structure with a 560-foot-long radial gate crest section. These releases pass at a maximum rate of 16,900 cubic feet per second (cfs) through the 10,000-foot-long Thermalito Power Canal and Thermalito Forebay to Thermalito Powerplant. The Thermalito Diversion Pool, Power Canal, and Forebay have a common water surface to accommodate flow reversals for the pumped-storage operation.

Thermalito Forebay Dam is a 15,900-foot-long embankment with a maximum height of 91 feet. The powerplant intake structure is an integral part of the Dam. This plant is equipped with one Kaplan turbine and three pump-turbines and operates under a static head of 100 feet.

Thermalito Afterbay has a gross capacity of 57,041 acre-feet and stores plant discharges for the pumped-storage operation as well as reregulates flows for return to the Feather River. The afterbay dam is a 42,000-foot-long earth structure with a maximum height of 39 feet.

Migrating salmon and steelhead blocked by the development are diverted from the River into the Feather River Fish Hatchery by the Fish Barrier Dam, located ½ mile downstream of Thermalito Diversion Dam. This 91-foot-high, concrete, overpour structure is discussed in Volume VI of this bulletin.

North San Joaquin Division and South Bay Aqueduct

The initial and northernmost reach of the California Aqueduct, designated the North San Joaquin Division, is 68 miles long (Figure 4). Principal features consist of Clifton Court Forebay in the Delta, the Delta Fish Protective Facility, 3 miles of unlined in-

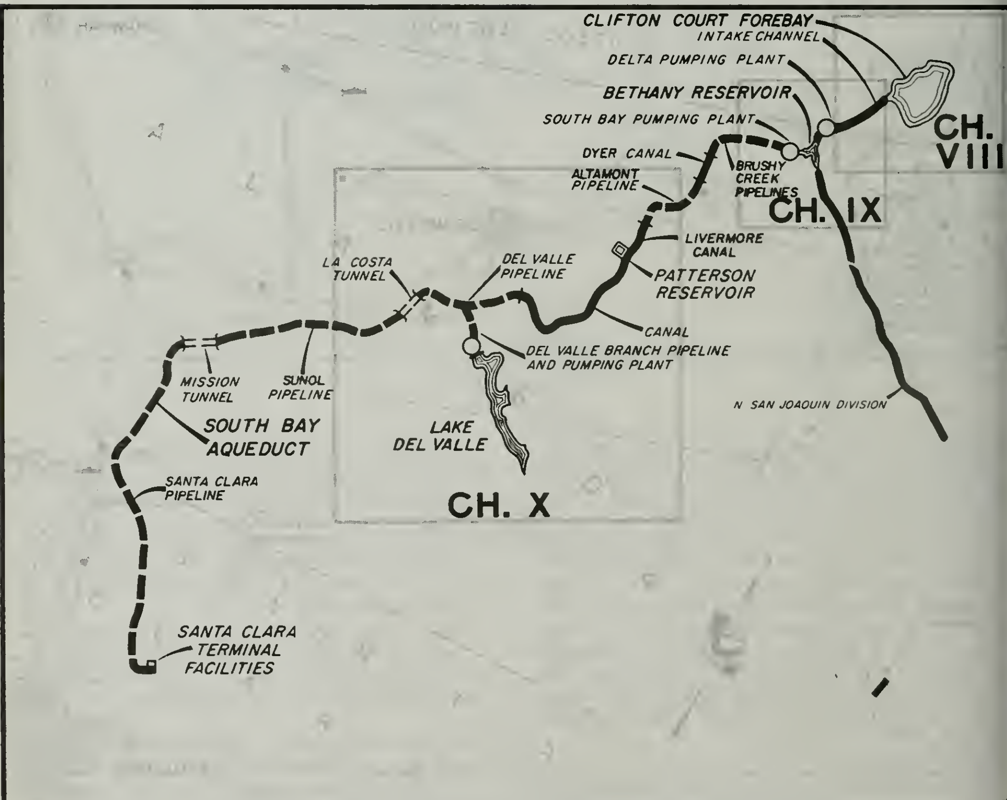

Figure 4. South Bay Aqueduct and Part of North San Joaquin Division

take channel, Delta Pumping Plant, Bethany Reservoir, and 64 miles of concrete-lined canal and appurtenant structures. The Division terminates at O'Neill Forebay. The design capacity of this aqueduct reach decreases from 10,300 cfs at its head to 10,000 cfs at its terminus.

Water released from Oroville Dam flows down the Feather and Sacramento Rivers, and the surplus waters in the Sacramento-San Joaquin Delta then are diverted into the California Aqueduct at Clifton Court Forebay. This forebay provides storage for off-peak pumping at Delta Pumping Plant and minimizes any adverse effects on existing Delta channels by diverting and storing large amounts of water at high tide. It has a surface area of 2,109 acres. The water is diverted from the Delta into the Forebay from adjacent Delta waterways, namely Old River and West Canal, and is regulated by an intake structure with five automatically controlled radial gates.

The water eventually will be conveyed to Clifton Court Forebay through the planned 43-mile-long Peripheral Canal. This unlined canal will start at the Sacramento River 18 miles south of the City of Sacramento and will skirt the eastern perimeter of the Delta. It will be hydraulically isolated from the Delta channels and will connect to the east side of the Forebay.

Bethany Forebay Dam was included in the initial construction of the South Bay facilities. The 890-acre-foot forebay, formed by the 119-foot-high 300,000-cubic-yard embankment, was used to provide operational flexibility as well as conveyance capability. Initially, water was supplied to the South Bay Aqueduct from the federal Delta-Mendota Canal through a small unlined canal. An interim pumping plant at the toe of the Dam lifted the water from the unlined canal into the forebay. South Bay Pumping Plant, located on the forebay, lifts the water 545 feet to the head of the 42-mile-long South Bay Aqueduct. The plant has since been expanded to the present 330-cfs capacity. During construction of the California Aqueduct and Delta Pumping Plant in the North San Joaquin Division, the forebay was expanded into the present Bethany Reservoir. Four embankments similar to the Forebay Dam were constructed, a channel was excavated to connect two sections of the Reservoir, and the California Aqueduct was cut into the north and south ends of the Reservoir. The Reservoir functions as a 1½-mile reach of canal and provides operational flexibility for Delta Pumping Plant, located 2 miles to the north.

Regulatory storage for South Bay Aqueduct now is provided in the 100-acre-foot Patterson Reservoir and the 77,106-acre-foot Lake Del Valle (Patterson Reservoir is discussed in Volume II of this bulletin). Lake Del Valle is located on Arroyo Del Valle near the midpoint of the South Bay Aqueduct. Project water is pumped into Lake Del Valle and released from it through a 120-cfs branch pipeline and Del Valle Pumping Plant. The Lake, formed by 235-foot-high Del Valle Dam, also provides flood control for Livermore Valley and conserves local runoff. This embankment structure contains 4,150,000 cubic yards of earth materials. The conservation function was established by agreement with a local agency and is incidental to project operation. Local runoff is stored when project

Figure 5. San Luis Division

operations permit and is released into the stream channel as requested by the local agency.

San Luis Division

The 106-mile-long reach, designated the San Luis Division, constitutes the Federal-State Joint-Use Facilities (Figure 5). It includes, among other features, San Luis Dam and Reservoir and appurtenant pumping-generating facilities. This entire division was designed and built by the U.S. Bureau of Reclamation and is operated by the Department of Water Resources on a cost-sharing basis, which is discussed in Chapter XI of this volume.

San Luis Reservoir, located at the head of this reach, provides 2,038,771 acre-feet of off-line storage, of which 1,067,908 acre-feet is the State's share. The main dam is an earthfill structure 385 feet high with a crest length of 18,600 feet. A total of 77,645,000 cubic yards of material was used in its construction. Water delivered to O'Neill Forebay through the California Aqueduct and Delta-Mendota Canal is pumped during off-peak periods into San Luis Reservoir. On-peak power is generated from releases made through the eight reversible units in the San Luis Pumping-Generating Plant, located at the toe of the main dam.

O'Neill Forebay, with a gross capacity of 56,426 acre-feet, serves as a regulation pool for the San Luis Pumping-Generating Plant and also as a gravity diversion pool for flows continuing south in the California Aqueduct. The reservoir has a surface area of 2,700 acres. The forebay dam required 3,000,000 cubic yards of material and has a maximum height of 88 feet and a crest length of 14,350 feet. The distance across the Forebay in the direction of the flow of the California Aqueduct is about 3 miles.

Los Banos and Little Panoche Detention Dams located south of San Luis Reservoir protect the Aqueduct, the Delta-Mendota Canal, and other improvements from floodflows. Los Banos Detention Dam also provides a 470-acre recreation pool. The reservoirs have a capacity of 34,562 and 13,236 acre-feet, respectively. The Dams are 167 and 152 feet high and contain 2,100,000 and 1,210,000 cubic yards of earth materials, respectively.

Because of en route deliveries, the flow capacity of the California Aqueduct through the San Luis Division decreases from 13,100 to 8,350 cfs, 7,050 cfs of which is for the State Water Project. Dos Amigos Pumping Plant, located 16 miles south of O'Neill Forebay, provides a 113-foot lift in the Aqueduct.

South San Joaquin and Tehachapi Divisions

The last reach of the California Aqueduct in the Central Valley, designated the South San Joaquin Division, is 121 miles long and primarily is canal. Three pumping plants (Buena Vista, Wheeler Ridge, and Wind Gap) with a total lift of 956 feet are located within this reach. The aqueduct capacity decreases from 8,100 to 4,400 cfs reflecting en route deliveries to water users. Volume I of this bulletin contains a dis-

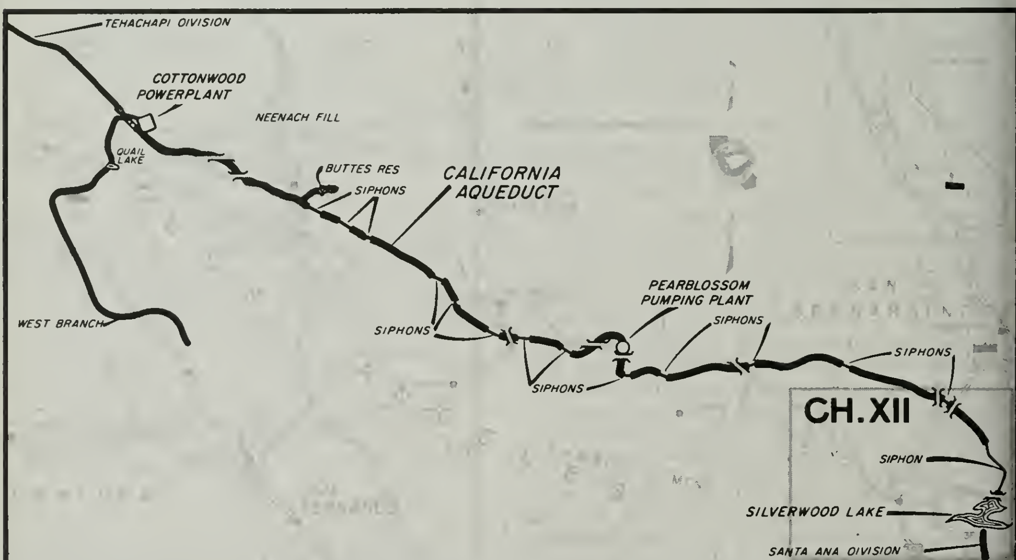

Figure 6. Mojave Division

ussion of the events which brought about the disparty in flows between this division and the San Luis)ivision to the north.

The next reach of aqueduct, designated the Tehachpi Division, crosses the Tehachapi Mountains. A. D. :dmonston Pumping Plant, located at the northern ›ase of the Tehachapi Mountains, has the capability to ift 4,410 cfs nearly 2,000 feet in a single lift through wo 12½-foot-diameter, underground, discharge lines. 'hese discharge lines enlarge to a 14-foot diameter bout halfway up the slope. At the top of the lift, these ines are joined by a manifold.

Three 23½-foot and one 20-foot-diameter tunnels, otaling 7.9 miles in length, are joined by siphons to arry the water through the summit region of the Tehachapis. The longest tunnel, 4¾ miles, is the 20-oot-diameter Carley V. Porter Tunnel. The longest iphon, the 2,452-foot-long Pastoria Siphon, conveys he water across Pastoria Creek between Tehachapi Tunnels Nos. 2 and 3. It consists of a single, elevated, 92-inch-diameter, steel pipeline designed for 2,680 fs. A similar parallel pipeline is required to bring this each up to ultimate planned capacity.

Tehachapi Afterbay, at the southern end of this di-ision, provides minor regulatory storage to accomnodate flow mismatch between A. D. Edmonston Pumping Plant and the downstream pumping plants. t consists of a concrete-lined, trapezoidal, canal secion, 24 feet deep, over most of its 0.6-mile length. The Vest Branch bifurcates from the California Aqueduct t the southern end of the Afterbay. The Afterbay is iscusssed in Volume II of this bulletin.

Iojave Division

Extending southeasterly from the Tehachapi After-›ay is the 102-mile-long reach of Aqueduct designated he Mojave Division (Figure 6). The Aqueduct conists of 93.4 miles of concrete-lined canal and a total of .9 miles of pipeline. At the head of this reach, two hutes accommodate a 132-foot total drop in the vertial alignment. Midway along the Division, the Pear-lossom Pumping Plant, with a capacity of 1,380 cfs, ifts the water 540 feet, the high point along the entire California Aqueduct alignment.

Silverwood Lake, a 74,970-acre-foot, in-line, storage eservoir, is located at the end of this division. The Lake, formed by Cedar Springs Dam, regulates deliv-:ries in the system, provides emergency storage of vater for use during aqueduct outages, and furnishes vater-oriented recreation. Cedar Springs Dam is a 7.6-nillion-cubic-yard earth and rockfill dam 249 feet iigh with a crest length of 2,230 feet.

›anta Ana Division

The remaining reach along the California Aqueluct, designated the Santa Ana Division, is 34 miles ong and terminates at Lake Perris, a 131,452-acre-foot eservoir (Figure 7). Flows are released from Silvervood Lake into the 3.8-mile-long 12¾-foot-diameter San Bernardino Tunnel. This high-head pressure tun-

Figure 7. Santa Ana Division

nel, with a capacity of 2,020 cfs, is directly connected to the Devil Canyon Powerplant penstock. The plant, situated at the mouth of Devil Canyon at the southern base of the San Bernardino Mountains, is a 119,700-kilowatt installation which operates under a normal static head of 1,418 feet. The penstock, which varies in diameter from 9½ feet to 8 feet, is an elevated steel pipeline. The Powerplant houses two impulse turbines rated at 81,000 horsepower each and two generators rated at 63,000 kVA each.

The Santa Ana Valley Pipeline, which comprises about 28 miles of buried high-pressure pipe 9 to 10 feet in diameter, conveys a flow of 444 cfs to the Project's terminal reservoir in Riverside County. This design flow capacity has been recalculated recently at 469 cfs due to a lowering of the pipeline outlet to Lake Perris.

Lake Perris, the terminus of the "main line" California Aqueduct, regulates deliveries and provides emergency storage and recreation. Perris Dam, of zoned earthfill construction, is 128 feet high and has a crest length of over 2 miles. The embankment required 20 million cubic yards of fill.

West Branch Division

The West Branch originates at Tehachapi Afterbay and extends southerly about 32 miles toward Los Angeles (Figure 8). Water flows by gravity southward from the Afterbay about 1½ miles to Oso Pumping Plant. This plant has a capacity of 3,128 cfs against a maximum static head of 231 feet. It is an indoor-type installation housing eight electric motor-driven pumps which require a total of 93,800 horsepower. The eight pumping units are manifolded to five 9-foot-diameter discharge lines.

From the end of the Oso Pumping Plant discharge lines, a concrete-lined canal extends 2.7 miles in a southwesterly direction to Quail Lake, a small regulation pool discussed in Volume II of this bulletin. From Quail Lake, the canal continues 2.3 miles to a transition to the temporary Gorman Creek Improvement Facilities. The Improvement Facilities, primarily an 8-foot-base-width concrete-lined channel, extend 6 miles to Pyramid Lake, a 171,196-acre-foot, in-line, storage reservoir. To utilize the 740-foot drop upstream from this reservoir, a pipeline will replace the Gorman Creek Improvement and a power plant will be installed with initial operations planned for 1982. Pyramid Dam is a 400-foot-high zoned earth and rockfill embankment, with a crest length of 1,090 feet.

From Pyramid Lake, which also functions as a power pool, water is diverted through the 30-foot-diameter 7.15-mile-long Angeles Tunnel to Castaic Powerplant. Maximum flow through the concrete-lined tunnel during periods of peak demand will be 18,400 cfs. A surge tank is located near the downstream portal of the Tunnel. The chamber is 120 feet in diameter and 383 feet high, with 158 feet of its height aboveground.

Castaic Powerplant will have a generating capacity of 1,200 megawatts. Six pump-turbines with motor-generator units are being installed. Companion Unit 7 Powerplant contains a 50-megawatt generator which can be used in the pump-starting process.

The plants discharge into the 28,231-acre-foot Elderberry Forebay, from which the water is either pumped back to Pyramid Lake or released into Castaic Lake immediately downstream of the forebay dam. Elderberry Forebay Dam is a 200-foot-high, 6,000,000-cubic-yard, zoned embankment.

The power facilities between Pyramid Lake and Castaic Lake are joint-use facilities of the City of Los Angeles and the Department of Water Resources. The City constructed and operates Castaic Powerplant, Unit 7 Powerplant, and Elderberry Forebay.

Castaic Dam is a 425-foot-high earthfill embankment with a crest length of 4,900 feet creating a reservoir with a capacity of 323,702 acre-feet. Approximately 46 million cubic yards of material was used in its construction. Castaic Lake provides operational and emergency storage and water-oriented recreation.

Design

The Department's staff located in Sacramento designed all the storage facilities except those previously credited to the U. S. Bureau of Reclamation or the Los Angeles Department of Water and Power. The same engineers designed or reviewed all significant changes made during the construction process.

Appendix A of this volume mentions the consultants and others outside the Department who contributed to the design and construction of the storage facilities.

The latest available design techniques were employed, hydraulic model studies were run on all major structures, and extensive, sometimes innovative, exploration and soils-testing programs were conducted. This work is covered in the chapters on the various dams. For example, Chapter V of this volume includes discussions of the early application of the finite element technique to the design of the Oroville Dam grout gallery and of the embankment seismic analysis that led to the finite element analysis currently being used on dams.

Construction

Construction contracts for storage facilities of the State Water Project were awarded and administered in accordance with provisions of the State Contract Act, Sections 14250 to 14424, Government Code, Statutes of the State of California. The State Contract Act requires that bids be solicited in writing and that the contract be awarded to the lowest responsible bidder. To comply with the Act, the following procedures were employed:

1. Prequalification of prospective contractors—A two-phase prequalification procedure was used to es

Figure 8. West Branch Division

Figure 9. Location of Construction Project Offices

tablish qualified bidder lists of those contractors desiring to bid. First, if the required financial statement indicated that the contractor had the necessary resources, the request for prequalification was processed further. Second, the contractor's ability was assessed based on the firm's overall experience and other uniform factors.

2. Advertisement and award of contracts—Public notice of a project was given once a week for at least two consecutive weeks in a newspaper published in the county in which the project was located and in a trade paper of general circulation in either San Francisco or Los Angeles, as appropriate. A "Notice to Contractors", in each case, was sent to all contractors on the list of qualified bidders. This document generally described the requirements and extent of the work and indicated the time and place for receiving the bids.

Contracts were awarded to the lowest responsible bidder. Responsible bids were those meeting all the conditions of bidding stated in the bidding requirements and determined to be reasonable in cost when compared with the engineer's estimate.

The Department's organization for supervision of construction activities consisted of project offices at selected locations throughout the State and a headquarters construction office located in Sacramento (Figure 9). Each project office was responsible for all project construction work within a particular geographical area and was staffed with construction engineers, inspectors, engineering geologists, and laboratory and other technicians. The headquarters construction office provided administrative and liaison services to the project offices. Factory inspection of materials and equipment to be incorporated in the work was performed by an equipment and materials section of the headquarters construction office.

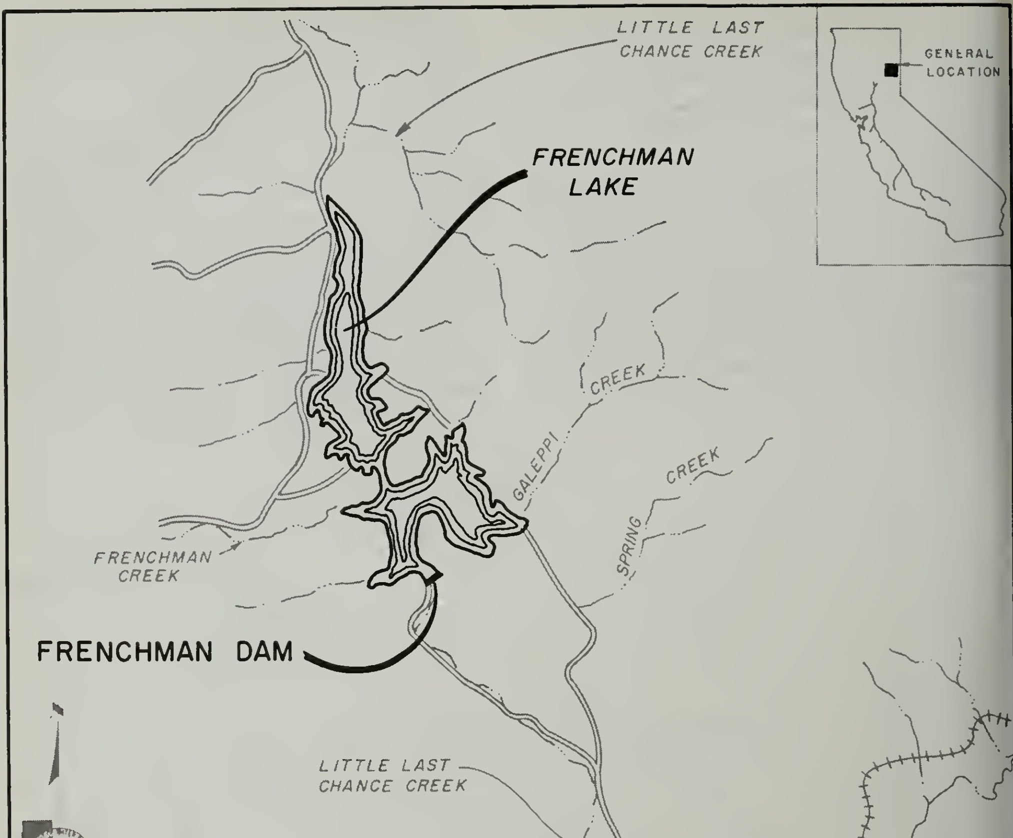

Figure 10. Location Map—Frenchman Dam and Lake

CHAPTER II. FRENCHMAN DAM AND LAKE

General

Description and Location

Frenchman Dam is a 139-foot-high, homogeneous, earthfill structure with internal blanket and chimney drains. The spillway is located on the right abutment. It has an unlined approach channel; a 50-foot-wide, ungated, ogee crest; a 30-foot-wide rectangular chute; and a flip-bucket terminal structure. The outlet works is located along the base of the left abutment.

Frenchman Lake has a capacity of 55,477 acre-feet at spillway crest, a water surface area of 1,580 acres, and a 21-mile shoreline.

Frenchman Dam and Lake are located entirely within the Plumas National Forest on Little Last Chance Creek, a tributary of the Middle Fork Feather River. The site is about 15 miles northeast of Portola and about 30 miles northwest of Reno, Nevada. The nearest major roads are State Highways 70 and 49 and U. S. 395 (Figures 10 and 11).

Figure 11. Aerial View—Frenchman Dam and Lake

A statistical summary of Frenchman Dam and Lake is shown in Table 2, and the area-capacity curves are shown on Figure 12.

Purpose

The principal purposes of Frenchman Lake are recreation and irrigation water supply. Flood control is an incidental benefit but was not considered to be a purpose. Operation studies indicate that the reservoir is capable of supplying an average of 10,000 acre-feet annually for irrigation through controlled releases without an adverse effect on the lake storage for recreation.

Chronology

Investigations of water and recreation development in the Upper Feather River Basin resulted in published reports in 1955 and 1957 (see Bibliography). In 1957, the Legislature authorized construction of five dams in this development. One of these was Frenchman Dam.

Preliminary design work included economic comparisons of rockfill and homogeneous earthfill dam sections as well as economic comparison of four sites on Little Last Chance Creek. From this work, the type of dam and final location were chosen. Final design work was initiated in 1959. Construction began in September 1959, and the Dam was completed in 1961.

Regional Geology and Seismicity

The Dam and reservoir are in the northern Sierra Nevada on the southeast corner of the tilted Diamond Mountain fault block. Pre-Cretaceous granitic rock was covered by Tertiary volcanic and pyroclastic rocks. Subsequent erosion of the volcanic series of rocks carved deep canyons and, in places, completely uncovered the older granitic rocks. The Dam and reservoir are mainly on the volcanic series of rocks. Frenchman Dam is near a seismically active area along the California-Nevada border.

Design

Dam

Description. The 139-foot-high embankment was designed as a homogeneous, rolled, earthfill structure with internal sloping and horizontal drains located downstream from the axis. Embankment general plan is shown on Figure 13, and sections are shown on Figure 14.

Stability Analysis. Stability of the Dam was determined by the Swedish Slip Circle method of analysis. Cases analyzed included loading of full reservoir and critical lower reservoir levels along with earthquake loading. Earthquake loading involved a foundation horizontal acceleration of 0.1g in the direction of instability of the mass being analyzed.

Settlement. Since the Dam was to be founded on rock, no foundation settlement was anticipated; therefore, settlement analyses were conducted on embankment material only. Tests indicated a maximum

TABLE 2. Statistical Summary of Frenchman Dam and Lake

FRENCHMAN DAM	
Type: Homogeneous earthfill	
Crest elevation	5,607 feet
Crest width	30 feet
Crest length	720 feet
Streambed elevation at dam axis	5,478 feet
Lowest foundation elevation	5,468 feet
Structural height above foundation	139 feet
Embankment volume	537,000 cubic yards
Freeboard above spillway crest	19 feet
Freeboard, maximum operating surface	19 feet
Freeboard, maximum probable flood	0 feet

FRENCHMAN LAKE	
Maximum operating storage	55,477 acre-feet
Minimum operating storage	2,335 acre-feet
Dead pool storage	1,840 acre-feet
Maximum operating surface elevation	5,588 feet
Minimum operating surface elevation	5,520 feet
Dead pool surface elevation	5,517 feet
Shoreline, maximum operating elevation	21 miles
Surface area, maximum operating elevation	1,580 acres
Surface area, minimum operating elevation	171 acres
Drainage area	82 square miles
Average annual runoff	27,000 acre-feet

SPILLWAY	
Type: Ungated ogee crest with lined chute and flip bucket	
Crest elevation	5,588 feet
Crest length	50 feet
Maximum probable flood inflow	32,000 cubic feet per second
Peak routed outflow	15,000 cubic feet per second
Maximum surface elevation	5,607 feet
Standard project flood inflow	14,200 cubic feet per second
Peak routed outflow	7,950 cubic feet per second
Maximum surface elevation	5,600.5 feet

OUTLET WORKS

Type: Reinforced-concrete conduit beneath dam at base of left abutment, valve chamber at midpoint—discharge into impact dissipator

Diameter: Upstream of valve chamber, 36-inch concrete pressure conduit—downstream, 30-inch steel conduit in a 6-foot - 6-inch concrete horseshoe conduit

Intake structure: Uncontrolled low-level tower with concrete plug emergency bulkhead

Control: Downstream control structure housing 24-inch fixed-cone dispersion valve and 8-inch globe valve—30-inch butterfly guard valve in valve chamber

Capacity	165 cubic feet per second

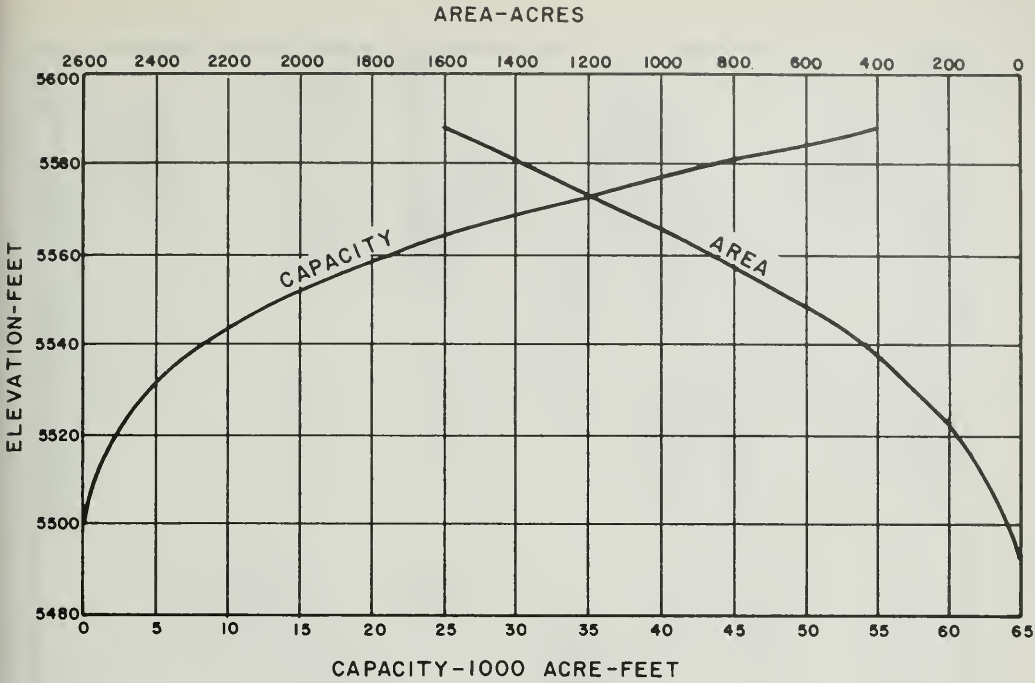

Figure 12. Area-Capacity Curves

consolidation of 4% for 8 tons per square foot. Most of the settlement was expected to occur during construction. A nominal crest camber of 12 inches was provided.

Construction Materials. An alluvial terrace 2 miles upstream of the Dam site was selected for borrow on the basis of surface examination, auger holes, and testing of materials sampled.

Natural moisture ranged from 12 to 51% and specific gravity from 2.66 to 2.80. Shear tests showed an angle of internal friction of 30 degrees and a cohesion of 500 pounds per square foot (direct shear, consolidated-quick; and triaxial shear, consolidated-undrained, and consolidated-drained tests were run). Permeability range was determined to be 0.0007 to 0.0010 of a foot per day. Riprap and bedding were found at the Dam site, but filter and drain materials had to be imported.

Foundation. At the Dam site three members, or layers, of the Tertiary volcanic series of rocks dip about 40 degrees southwest into the right abutment. One of these members, an olivine basalt flow, forms most of the left abutment down to the stream channel. Pyroclastic rocks, mainly an andesite tuff breccia, underlay the channel section and lower half of the right abutment. The pyroclastic rock in the channel was covered by alluvium which was removed during foundation excavation. Capping the right abutment is a layer of hornblende andesite.

A normal fault, dipping 30 degrees to the southwest and striking nearly perpendicular to the dam axis, passes through the intake tower foundation and along the left side of the channel section. The fault zone, from 2 to 4 feet wide, contains brecciated to clayey sheared rock and seams of fat clay. Rock several feet on either side of this zone is strongly fractured and platy. The fault forms the contact between the tuff breccia and altered basalt in the channel section downstream from the cutoff.

A grout curtain approximately 50 feet in depth was placed across the entire length of the dam foundation. A second grout curtain approximately 100 feet deep was placed upstream of this curtain and was intended to grout any cavities not reached by the 50-foot curtain.

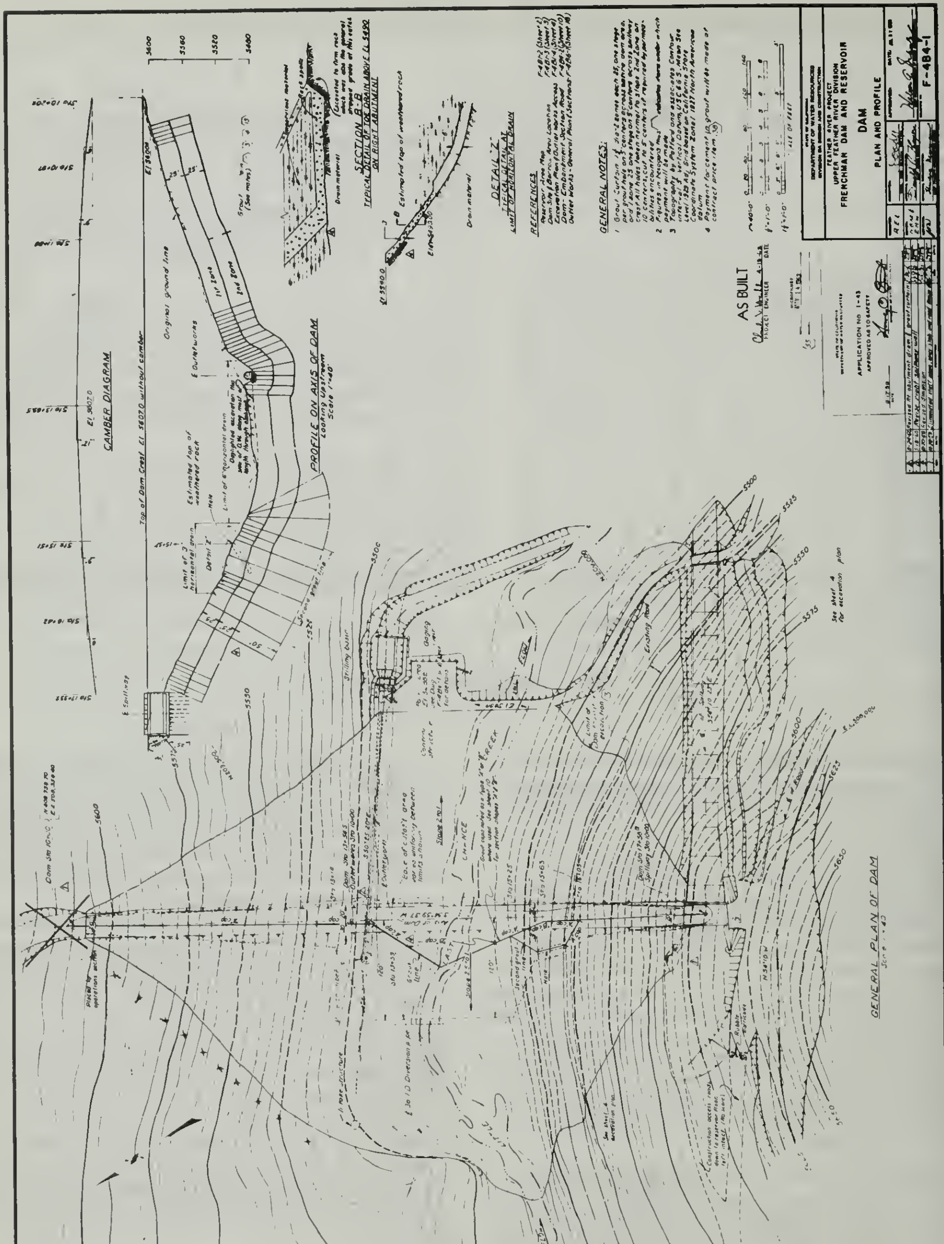

Figure 13. General Plan and Profile of Dam

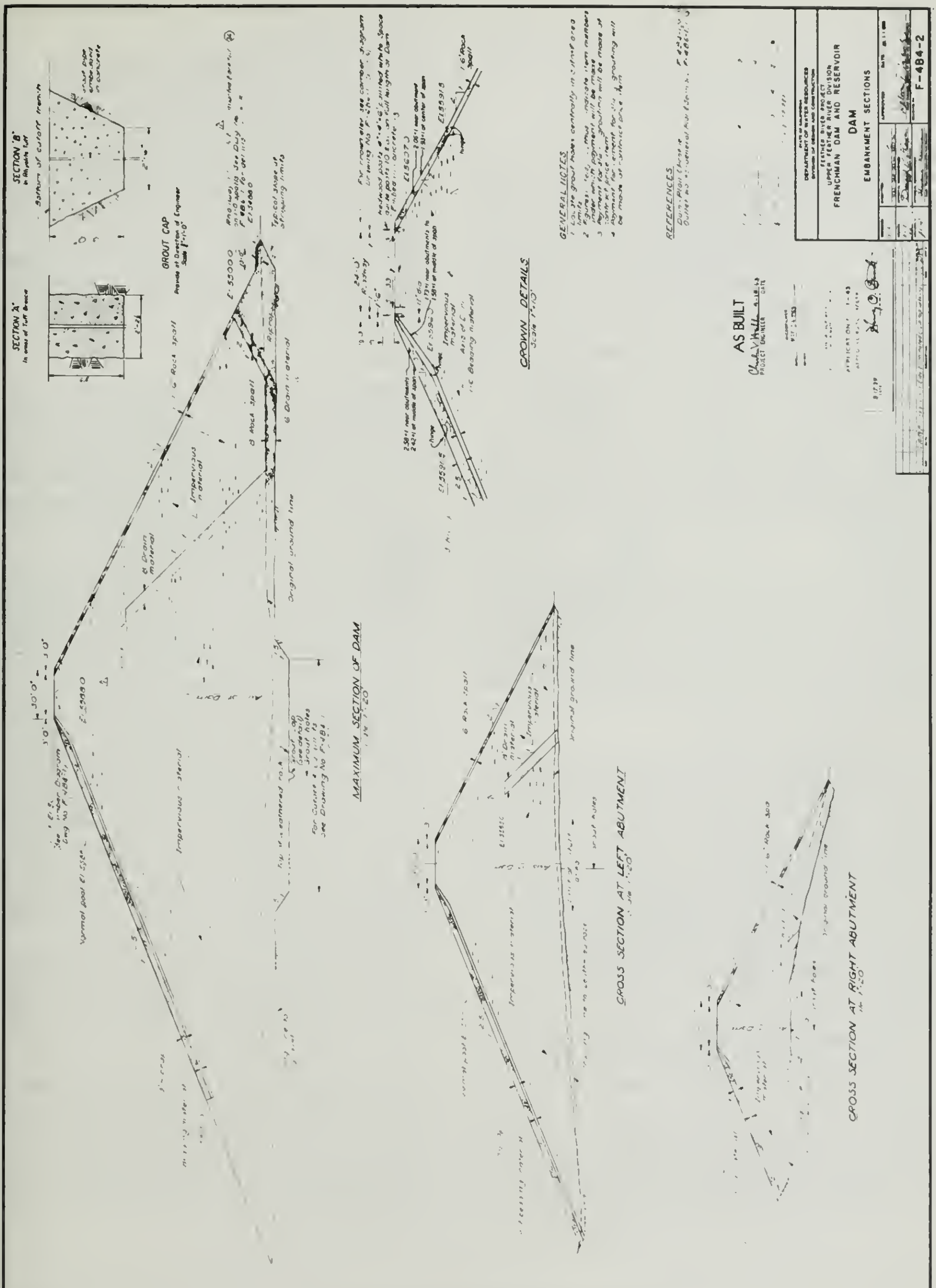

Figure 14. Embankment Sections

Instrumentation. Instrumentation at Frenchman Dam included (1) embankment settlement monuments, (2) piezometers, (3) electric pressure cells, (4) a cross-arm settlement unit, (5) observation wells, and (6) a seepage measurement weir (Figure 15).

Outlet Works

Description. The outlet works is located along the base of the left abutment. It consists of (1) a low-level intake tower with a concrete bulkhead that can be lowered to seal the opening; (2) a 36-inch, reinforced-concrete, pressure conduit from intake to valve chamber; (3) a valve chamber containing a 30-inch butterfly valve located just upstream of the dam axis; (4) a 30-inch, steel, outlet pipe installed in a 78-inch horseshoe conduit from the valve chamber to the downstream terminus; and (5) a control structure with discharge valves and stilling basin at the toe of the Dam (Figure 16). Access to the valve chamber is through the horseshoe conduit. A bulkheaded, 18-inch, penstock wye was installed on the 30-inch outlet pipe 11 feet upstream from the control structure to provide for possible future power development. A gauging weir is located downstream from the stilling basin to provide measurement of flow through the outlet works.

Because of the wide range of flows to be controlled, two discharge valves were installed: a 24-inch fixed-cone dispersion valve to control high flows for irrigation (up to 88 cubic feet per second with reservoir at elevation 5,520 feet) and an 8-inch globe valve to control low flow for maintaining fish life in the stream (as low as 1 cubic foot per second). Rating curves are shown on Figures 17 and 18.

Structural Design. The intake structure is a tower approximately 34 feet high with outside dimensions of 5 feet by 5 feet and an inside diameter of 36 inches. It was designed as a vertical cantilever, fixed at the base. Loading cases included: (1) construction condition, no embankment in place, and a horizontal wind pressure of 15 pounds per square foot; and (2) at-rest earth pressure of the finished embankment and reservoir pressure. Trashracks were designed for a differential hydrostatic head of 40 feet. Yield point stresses were allowed in the trashbars while normal working stresses were allowed in the supporting concrete members.

Upstream and downstream conduits are composed of monoliths approximately 25 feet long with waterstops at each joint upstream of the embankment chimney drain.

The 30-inch outlet pipe was fabricated from ¼-inch steel plate with a yield point of 30,000 pounds per square inch. It has a ½-inch mortar lining. The pipe is set on saddles inside the horsehoe conduit and is designed with allowable stress of 16,000 pounds per square inch in tension and 12,000 pounds per square inch in compression.

Mechanical and Electrical Installations. A 120-volt, 3,000-watt, gasoline-operated, generating set is provided at the control structure to supply power for lighting and ventilating. All valves were intended to be operated manually; however, because of the time required to develop sufficient hydraulic pressure by hand to operate the 30-inch butterfly valve, a motor-driven linkage powered by the generator set was installed in February 1963.

A ventilating system operated by a ⅛-horsepower motor with an output of 406 cubic feet per minute was installed in the control house to ventilate the valve chamber and conduit.

Spillway

Description. The spillway is located on the right abutment. It consists of an unlined approach channel; a 50-foot-long, ungated, ogee crest; a transition chute section; a 30-foot-wide rectangular chute; and a flip-bucket terminal structure (Figure 19). Two bridges cross the spillway: one at the crest for connecting U.S. Forest Service roads, and one at the lower end to provide access to the control structure.

Hydraulics. Reservoir storage above the spillway reduces the standard project flood from a peak inflow of 14,250 cubic feet per second (cfs) to an outflow of 7,950 cfs with 6½ feet of freeboard and the maximum probable flood from a peak inflow of 32,000 cfs to an outflow of 15,000 cfs without any freeboard.

Structural Design. Crest walls were constructed monolithically with the ogee crest and were designed for (1) normal earth loads plus live loads on the bridge, and (2) normal earth loads plus seismic loading. The crest is anchored with No. 10 reinforcing bars embedded 6 feet into the rock foundaton on approximately 5½-foot centers. The upstream end is provided with a 6-foot-deep cutoff and a 25-foot-deep grout curtain. The crest was designed for loads transferred from the walls and for uplift conditions during maximum spillage.

Walls and floors of the transition and chute sections were constructed monolithically. Floors are anchored with No. 9 reinforcing bars embedded 6 feet into the rock on approximately 5-foot centers in both directions.

The flip bucket was designed with a 50-foot-radius vertical curve with a 20-degree upward deflection. The floor upstream of the cutoff is anchored with three rows of No. 9 bars embedded 6 feet into the rock foundation at 6-foot centers in both directions. A 4-inch pipe surrounded by filter material is provided beneath the concrete floor to drain the bucket foundation.

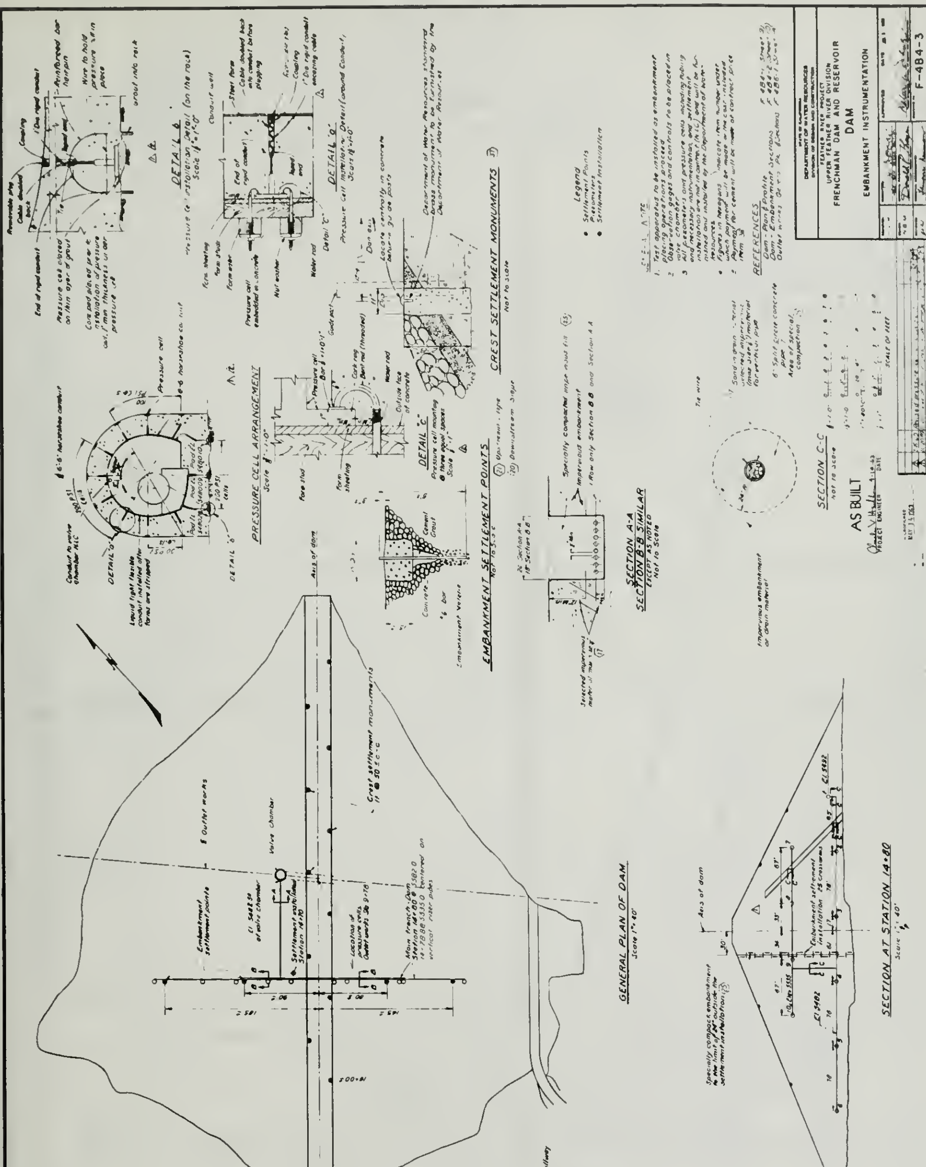

Figure 15. Location of Embankment Instrumentation

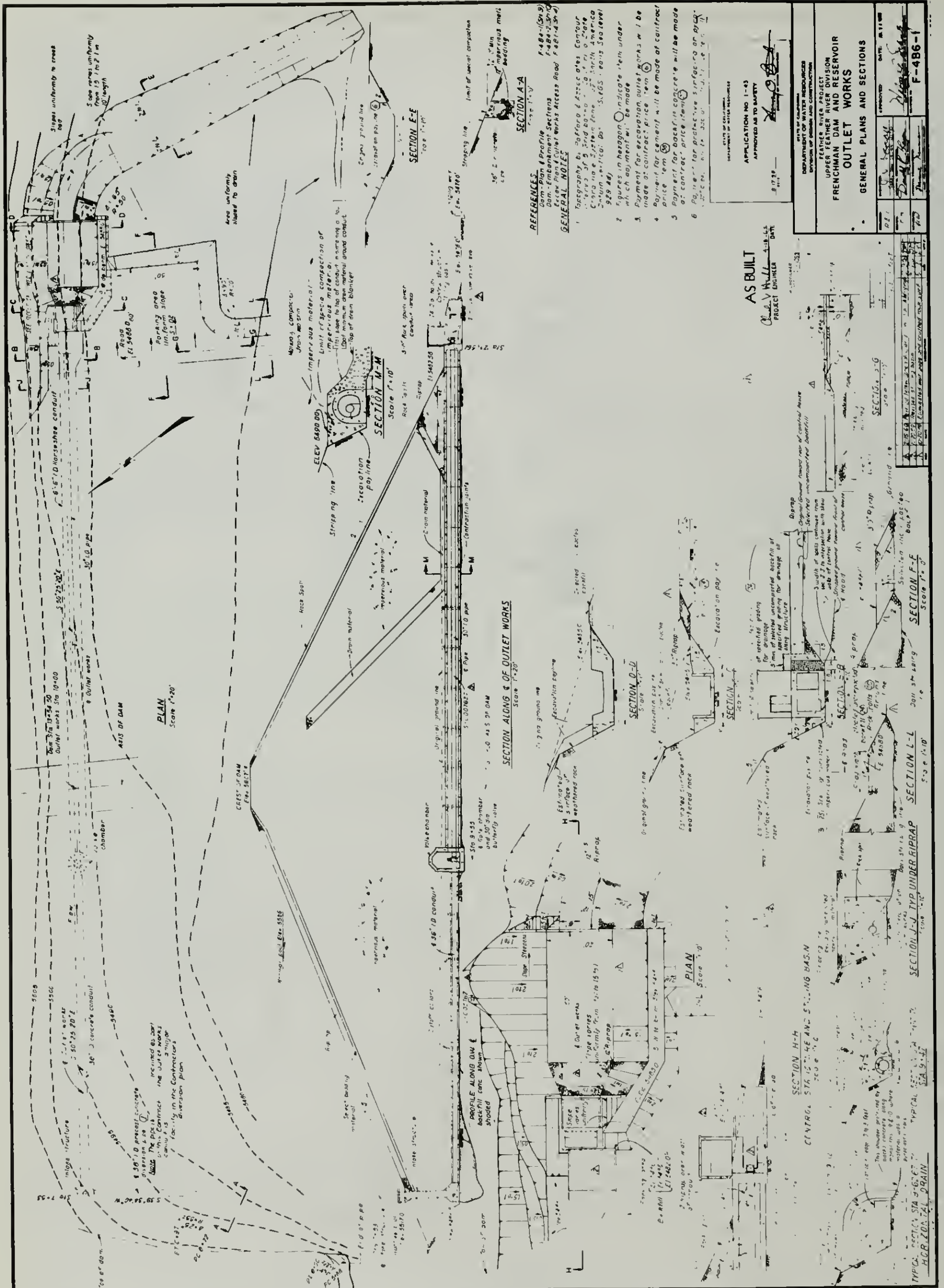

Figure 16. General Plan and Profile of Outlet Works

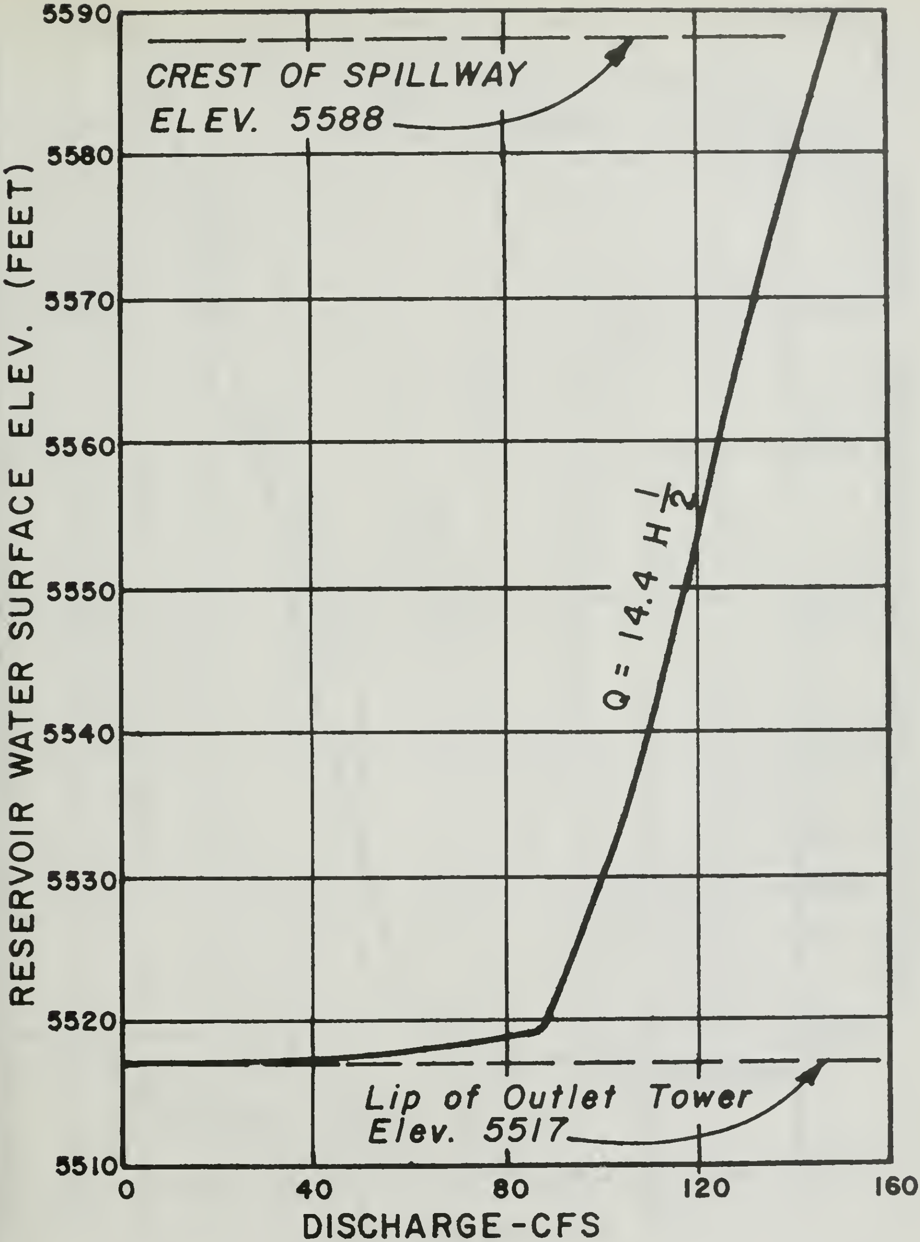

Figure 17. Outlet Works Rating Curve (24-Inch Hollow-Cone Valve)

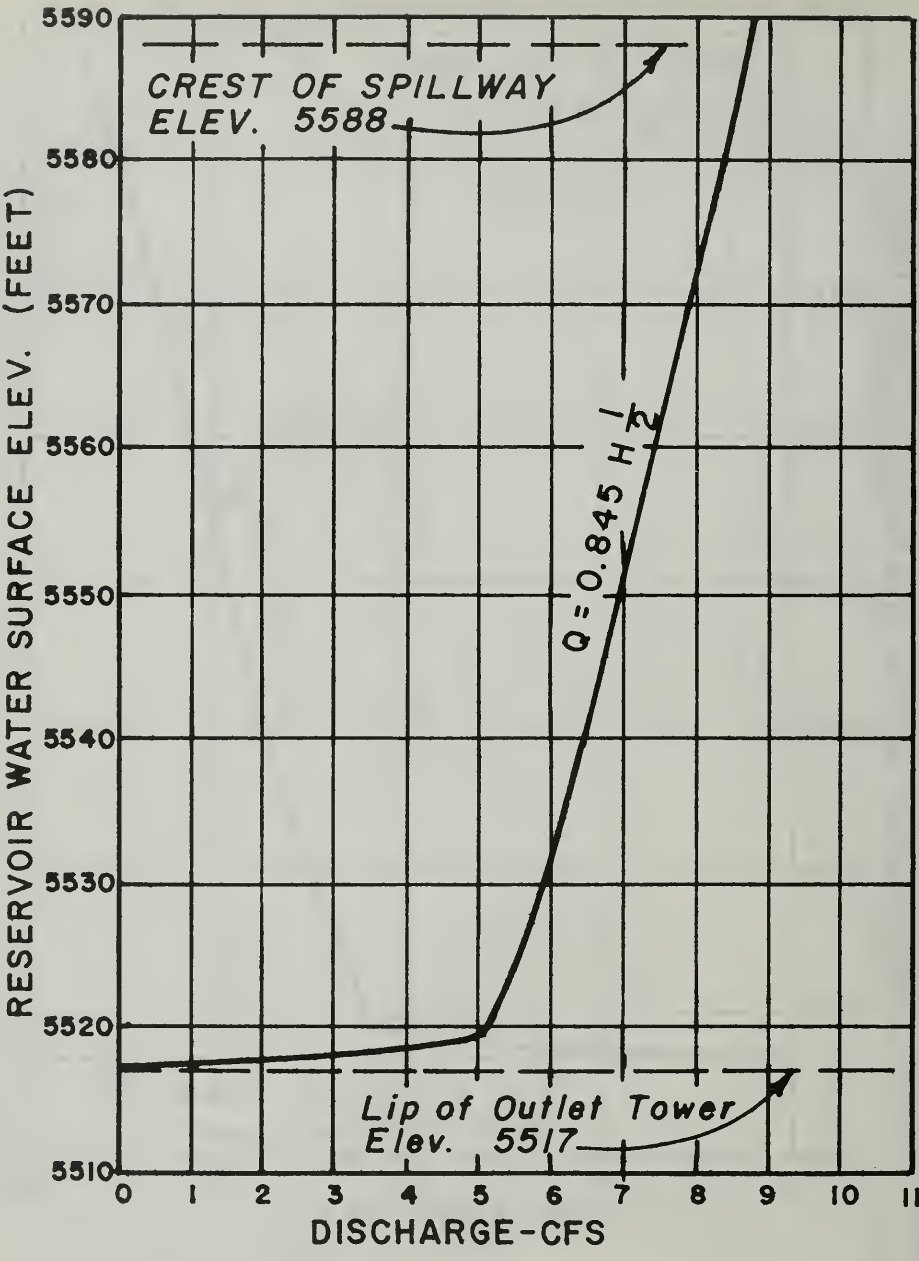

Figure 18. Outlet Works Rating Curve (8-Inch Globe Valve)

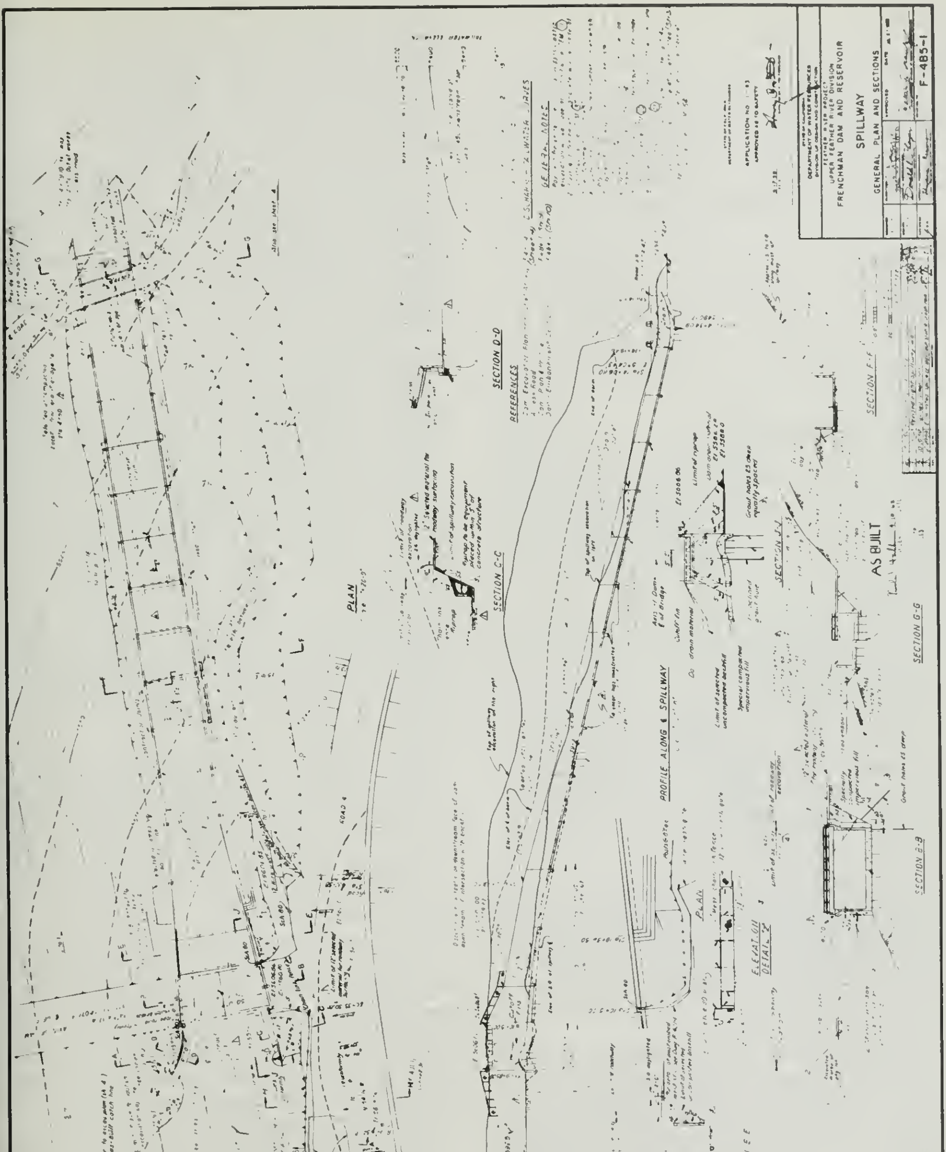

Figure 19. General Plan and Profile of Spillway

Construction

Contract Administration

General information for the construction contract for Frenchman Dam is shown in Table 3. The principal features included in the contract, designated as Specification No. 59-19, were an earthfill dam, spillway, outlet works, and relocation of portions of U. S. Forest Service roads.

TABLE 3. Major Contract—Frenchman Dam

Specification	59-19
Low bid amount	$1,809,110
Final contract cost	$1,708,093
Total cost-change orders	$22,485
Starting date	9/15/59
Completion date	10/18/61
Prime contractor	Isbell Construction Company

Diversion and Care of Stream

Diversion of Little Last Chance Creek during construction was the responsibility of the contractor. The outlet works was utilized. Between the time the foundation excavation was begun and the outlet works completed, streamflow was pumped around the job site through a 6-inch aluminum pipe extended along the left abutment.

A cofferdam upstream of the toe of the Dam was placed and the impounded water transported to the outlet works at the base of the intake tower by a 36-inch reinforced-concrete pipe. This pipe was left in place and plugged with concrete at its junction with the base of the intake structure after its use for diversion was discontinued. Downstream water requirements during the remainder of the construction period were fulfilled by means of a 6-inch bypass between the diversion pipe upstream of the concrete plug and the outlet conduit.

Foundation

Dewatering. A 15-foot-wide cutoff trench was excavated to bedrock between the cofferdam and the upstream toe of the Dam. A 24-inch, perforated, corrugated-metal pipe with a 12-inch riser was installed and the trench was backfilled with drain rock. Water accumulating in the pipe was removed by pumping from the riser.

Excavation. A total of 50,953 cubic yards of unsuitable foundation material was removed from the streambed and abutments. Cavities in the right abutment were filled with concrete. The largest was mined to a depth of nearly 50 feet. It contained a preserved redwood tree trunk, 2½ feet in diameter and approximately 20 feet long, estimated to be 10 million years old. Portions of this trunk were preserved for display. An auxiliary grout curtain 100 feet in depth was placed around the upstream limits of the cavity area.

To reduce the possibility of seepage through a fault zone in the stream channel, all sheared and strongly fractured rock was removed to a depth of several feet below the cutoff surface. A 5-foot-deep trench that varied from 10 to 20 feet in width was excavated along the fault in the cutoff area and then backfilled with embankment material.

Grouting. A total of 6,567 cubic feet of grout was injected into the foundation. Where surface leakage occurred, the most effective treatment was intermittent low-pressure pumping of thick grout. The grout then was permitted to harden in the hole, which was redrilled and regrouted in the succeeding stage. Uplift due to grouting was negligible (maximum recorded was ½ inch).

Handling of Borrow Materials

Impervious. Impervious borrow material was obtained from lenticular to thickly bedded terrace deposits 2 miles upstream from the Dam site (Figure 20). Mixing of these deposits was accomplished by power-shovel excavation from a vertical face approximately 30 feet high. In-place samples indicated that field moisture was from 1 to 5% above optimum. It was, therefore, necessary to blend freshly excavated material with that which had been spread and dried. A total of 515,632 cubic yards of excavation yielded 487,330 cubic yards of compacted impervious fill.

Drain. Due to the fact that the contractor was unable to find suitable sand in the area for the drain, he imported 11,000 cubic yards of sand from Reno, Nevada (30 miles) and blended it with 5,000 cubic yards of local crushed andesite.

Slope Protection. Approximately 22,000 cubic yards of riprap (1 cubic yard maximum size) was obtained from the andesite quarry located on the right abutment downstream from the Dam. Between 10,000 and 15,000 cubic yards of smaller rock for crushing into rock spalls and riprap bedding also was obtained from this source.

Embankment Construction

Impervious. The impervious fill was placed in designated lanes parallel to the axis of the Dam by six bottom-dump trucks and two scrapers. It then was spread into 6-inch lifts by bulldozers and mixed by disking. Compaction was accomplished by six passes of two double-drum sheepsfoot rollers in tandem, pulled by a tractor. Areas inaccessible to or missed by the larger unit were compacted by 12 passes of a single sheepsfoot roller pulled by a tractor.

Special compaction near abutments and structures was accomplished by gasoline engine-operated hand compactors. Where this equipment was used, it was necessary to scarify and wet the top of each lift before placing the next one to avoid laminations.

Design recommendations for embankment compac-

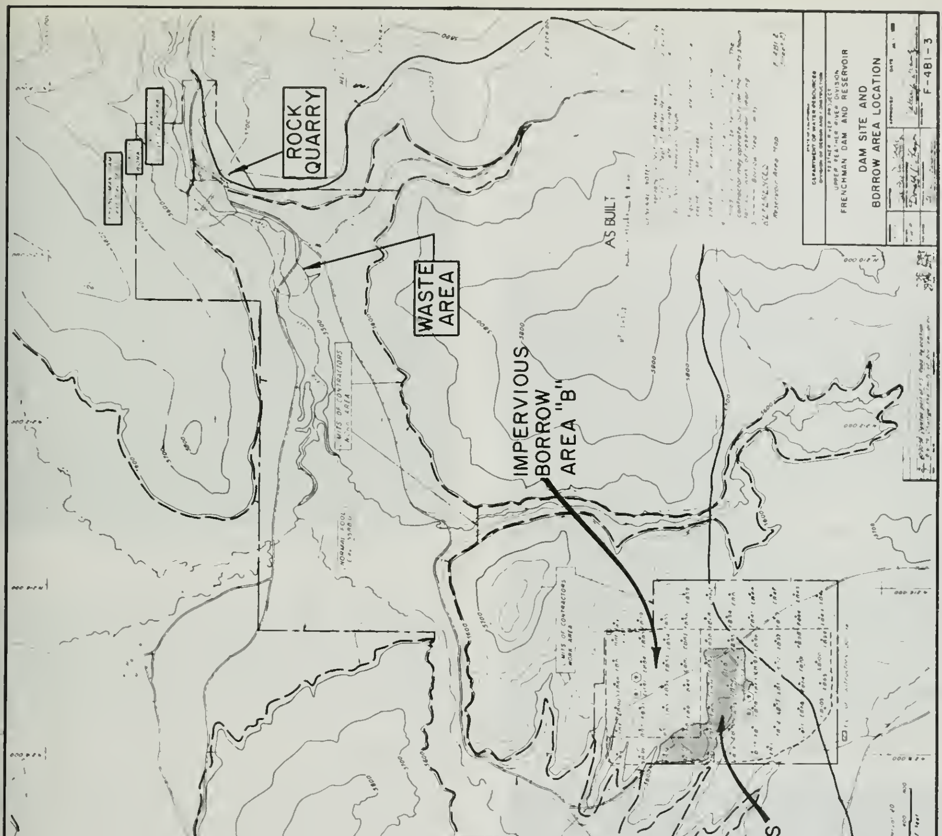

Figure 20. Location of Borrow Areas and Frenchman Dam Site

tion were: (1) rejection of material with a maximum dry density of less than 104 pounds per cubic foot, (2) a desirable relative compaction of 98%, and (3) rejection of material below 95% relative compaction. Average dry density for all rolled fill tested was 107.0 pounds per cubic foot (pcf) ranging from 90.7 to 115.2 pcf, with an average relative compaction of 97.7% ranging from 91.4 to 103.8%. Average dry density for specially compacted fill (hand-held compactors) was 107.8 pcf, ranging from 100.9 to 117.2 pcf, with an average relative compaction of 98.8% ranging from 94.5 to 102.9%. Only borrow materials with maximum dry density exceeding 104 pcf were drawn from the borrow areas. Areas within the borrow where light materials were known to exist in substantial quantities were avoided. Fill with a relative compaction of less than 95% was either rejected or reworked and retested. The embankment under construction is shown on Figure 21 and the completed Dam on Figure 22.

Drain. The horizontal portion of the drain was compacted by four passes of a tractor, while the sloping portion was compacted with two passes of a vibratory roller. The decision to change methods of compaction was made after tests showed that the vibratory roller method of compaction resulted in higher and more uniform compaction than the specified method. To ensure satisfactory compaction upstream of the drain, the impervious material was lapped 1 to 2 feet over the drain during compaction, then trimmed back to a neat line.

Slope Protection. Bedding was end-dumped onto

Figure 22. Completed Embankment

the upstream dam slope which had been trimmed to firm material. Riprap then was end-dumped onto the bedding and selectively placed by a clamshell bucket and hand labor. Rock spalls were end-dumped on the downstream face of the Dam, shaped, then compacted by backing a vibratory roller over the slope with a tractor.

Outlet Works

Rough excavation was done with a ripper-equipped tractor and a 2½-cubic-yard shovel. Light to moderate blasting was necessary in areas of basalt. The fault zone parallel to the stream through the Dam site was encountered for about the first 100 feet upstream of the outlet structure. Loose material was excavated to a depth of 3 to 13 feet below grade and backfilled to foundation grade with concrete. Where pyroclastics were encountered, initial excavation was made to within 0.3 to 0.5 feet above final grade elevation, and final grading was done the day before concrete was placed. This prevented rapid deterioration of the foundation surface and eliminated the need for spraying with protective asphalt covering. All areas of overexcavation were backfilled to foundation grade with concrete. Inasmuch as this concrete was placed during the summer, there was no need for cold-weather protection. Placement generally was routine and was accomplished with a truck crane and a bottom-dump bucket (Figure 23).

Spillway

Excavation. A total of 34,016 cubic yards was excavated for the spillway. Blasting was required only in the approach channel at the crest and at the upper part

Figure 21. Embankment Construction

Figure 23. Outlet Works Concrete Placement

of the chute. Overexcavation was required at and below the crest where blocks of andesite were loosened by the blasting or taken from the foundation during ripping operations. Before the spillway slab was placed, cavities and areas that had been overexcavated were backfilled with concrete to foundation grade. Because the pyroclastic and sedimentary rocks in the foundation of the spillway deteriorated rapidly when dried or rewetted, it was necessary to spray them with a protective coat of asphalt to prevent slaking.

Anchorage. Anchor bars were replaced by shear keys for about 100 feet in the middle of the chute when it became apparent that the foundation rock in this area would not provide sufficient anchorage for the bars.

Concrete Placement. The floor of the spillway was placed by means of a steel slip form that was drawn uphill with a winch to strike off the concrete. Forms for the crest and wall were constructed of plywood backed by studs and walers.

Placement of the spillway concrete was started in the fall of 1960 and completed in the spring of 1961. The sequence of construction was the floor slab first, then the crest, flip bucket, outlet works bridge and, finally, the walls and crest bridge. A total of 1,678 cubic yards of concrete was placed in the spillway and 83 cubic yards in the bridges crossing the spillway (Figures 24 and 25).

Concrete Curing. If mean daily temperatures fell below 40 degrees Fahrenheit, the contractor was required to maintain concrete temperatures above 50 degrees Fahrenheit for the first three days after placement and above 32 degrees Fahrenheit for the next three days. To accomplish this, a plastic tent was placed over the walls of the spillway chute and steam was piped in from a steam generator for six days. The tent then was removed and a curing compound applied.

The high walls of the spillway crest were protected by $2\frac{1}{2}$-inch-thick batting with an aluminum reflective surface. Single layers were placed between the vertical

Figure 24. Spillway Chute

Figure 25. Spillway Flip Bucket

studs and a double layer over the open concrete at the top. Six days later, the forms were stripped and a curing compound applied.

Portable heaters were placed under the deck of the crest bridge to supplement the steam in the plastic tent, and blowers were used to circulate the air. The contractor suspended operations after December 6, 1960, and unstripped forms were left in place until the following spring.

Backfill. Uncompacted backfill behind the spillway walls was obtained from waste material rejected during the processing of other rock.

Concrete Production

Concrete was produced at a semiautomatic batching plant located about one-half mile upstream from the Dam. Coarse aggregate was obtained from a pit near Reno and sand from Washoe Lake, approximately 17 miles south of Reno. Mixing water was obtained directly from Little Last Chance Creek but, when the stream became dirty from eroded materials, it was necessary to truck water in from a different location. Concrete was mixed in 7-cubic-yard truck mixers for a minimum of 80 revolutions at the rate of 8 to 10 revolutions per minute. The normal load did not exceed 6 cubic yards.

An air-entraining agent was used in the concrete to provide resistance to deterioration from freezing and thawing cycles which occur in the project area.

Reservoir Clearing

The reservoir area below elevation 5,588 feet was cleared of sagebrush, trees, down timber, rubbish, and farm buildings. Trees were cut off 1 foot above the ground. Fencing in the reservoir area was left in place as long as practical to fulfill existing grazing requirements and was later removed by department personnel.

Closure

Storage in Frenchman Lake was begun in 1961 by closing the 6-inch bypass valve on the inlet tower. Early in March 1962, it became apparent that the reservoir level would not be high enough by the start of the irrigation season (March 15) to release water-right entitlements (up to 94.4 cubic feet per second) through the outlet works. The need to pump water to meet downstream requirements during the closure was eliminated by executing individual agreements with downstream water-right holders. The maximum storage in 1962 was 13,811 acre-feet; therefore, only fish and stream maintenance and water entitlement releases were made in that year.

In February 1963, the reservoir rose above the minimum recreation pool of 21,425 acre-feet. It exceeded 43,000 acre-feet in May 1963 and dropped to a low of 31,383 acre-feet in October 1964. The first spill was in April 1965, and the reservoir has spilled in about half of the succeeding years. It has remained above a pool of 36,000 acre-feet since October 1964.

BIBLIOGRAPHY

California Department of Water Resources, "A Plan for the Development and Operation of Recreation Facilities at Frenchman Reservoir, Upper Feather River Basin", March 1961.

———, Bulletin No. 59, "Investigation of Upper Feather River Basin Development: Interim Report on Engineering, Economic and Financial Feasibility of Initial Units", February 1957.

———, Bulletin No. 59-2, "Investigation of Upper Feather River Basin Development", October 1960.

———, "Design Report for Frenchman Dam", December 1966.

California Department of Public Works, "Northeastern Counties Investigation: Report on Upper Feather River Service Area", April 1955.

———, "Program for Financing and Constructing the Feather River Project as the Initial Unit of the California Water Plan", February 1955.

Diller, J. S., "Geology of the Taylorsville Region", U. S. Geologic Survey Bulletin 353, 1908.

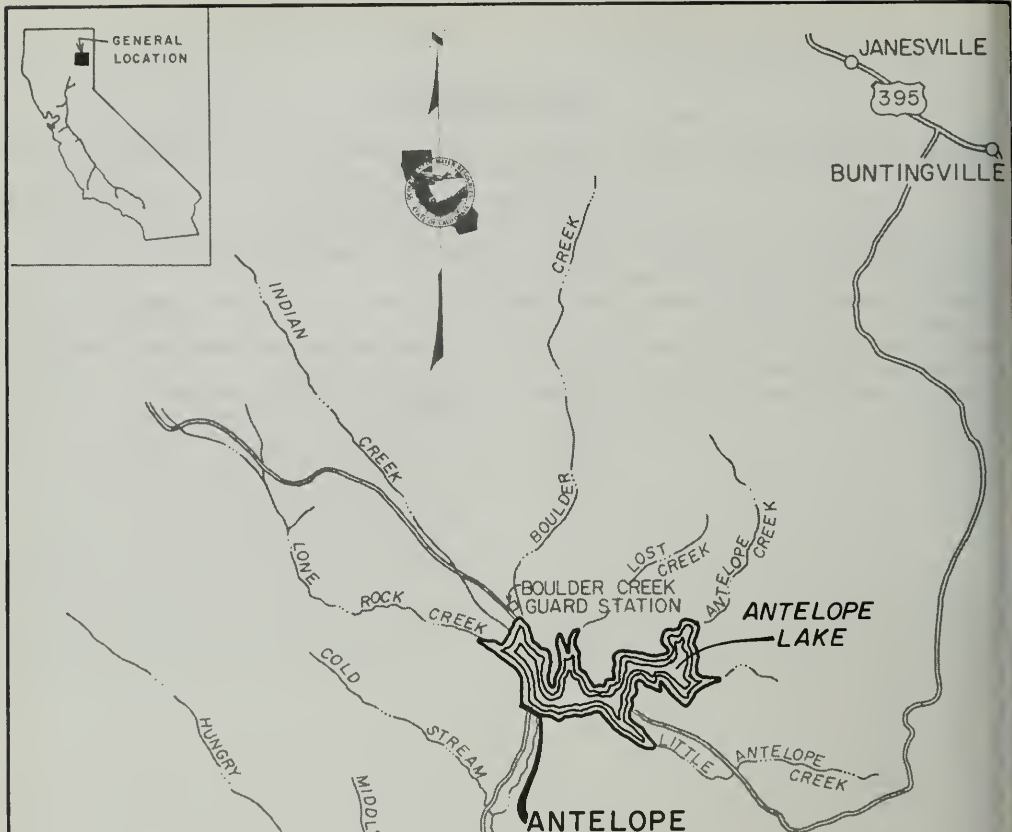

Figure 26. Location Map--Antelope Dam and Lake

CHAPTER III. ANTELOPE DAM AND LAKE

General

Description and Location

Antelope Dam consists of two zoned earth embankments: a 120-foot-high main dam and a 60-foot-high auxiliary dam. The spillway is located on a ridge between the embankments. It is an open channel structure with a 60-foot-long ungated weir. An outlet works, consisting of a low-level intake tower, a 36-inch steel-lined pressure conduit, and a control valve structure, is located along the base of the right abutment of the main dam.

Antelope Lake has a capacity of 22,566 acre-feet, a water surface area of 931 acres, and a 15-mile shoreline.

Antelope Dam and Lake are located entirely within the Plumas National Forest on Upper Indian Creek, a tributary of the North Fork Feather River, 43 miles by road northeast of Quincy. The nearest major roads are State Highways 70 and 89 (Figures 26 and 27).

Figure 27. Aerial View—Antelope Dam and Lake

A statistical summary of Antelope Dam and Lake is shown in Table 4, and the area-capacity curves are shown on Figure 28.

Purpose

The purposes of Antelope Lake are streamflow maintenance and recreation. Flood control is an incidental benefit. Operation studies indicate that the reservoir is capable of releasing 20 cubic feet per second from April 1 through June 30 and 10 cubic feet per second during the remainder of the year without an adverse effect on the recreation water level of the Lake. These flow quantities were determined with the guidance of the Department of Fish and Game to be sufficient to provide fishery enhancement in the stream below the Dam. They also will meet downstream water rights. Surface drawdown will vary from 2 to 12 feet, averaging less than 6 feet.

Chronology

Investigations of water and recreation development in the Upper Feather River Basin resulted in published reports in 1955 and 1957 (see Bibliography). In 1957, the Legislature authorized construction of five dams in this development. One of these was Antelope Dam.

Preliminary design involved studies of three embankment axes, and the final alignment selected was based on minimum embankment quantity. Final design work was initiated in 1961. Construction began in August 1962, and the Dam was completed in 1964.

Regional Geology and Seismicity

Antelope Dam is near the northern extremity of the Sierra Nevadas on the westward-tilted Diamond Mountain fault block. The site is in a relatively small granitic area surrounded by Tertiary and older volcanic and metavolcanic rocks. Pliocene and Pleistocene volcanic rocks of the southern Cascade Range and Modoc Plateau lie as close as 20 miles north of the site. Volcanically active Mount Lassen lies approximately 50 miles to the northwest. Honey Lake Valley, which consists chiefly of Quaternary Lake deposits and other alluvium, is about 10 miles to the northeast and belongs to the Basin and Range provinces. Relatively small deposits of Tertiary auriferous gravels, deposited by the ancient Jura River, occur within the Indian Creek drainage basin and partially have been reworked to form a portion of the Quaternary alluvium and terrace deposits found adjacent to the Dam site.

Studies of seimicity from 1769 to 1960 indicated the Dam site is in a seismically active area. Most of the earthquakes affecting the site originated in Genessee Valley, Honey Lake Valley, and Mount Lassen National Park. Maximum intensity experienced at the site probably was 7 on the Modified Mercalli scale.

Design

Dam

Description. Both the 120-foot-high main dam and the 60-foot-high auxiliary dam were designed as zoned

TABLE 4. Statistical Summary of Antelope Dam and Lake

ANTELOPE DAM	
Type: Zoned earthfill	
Crest elevation	5,025 feet
Crest width	30 feet
Crest length	1,320 feet
Streambed elevation at dam axis	4,918 feet
Lowest foundation elevation	4,905 feet
Structural height above foundation	120 feet
Embankment volume	380,000 cubic yards
Freeboard above spillway crest	23 feet
Freeboard, maximum operating surface	23 feet
Freeboard, maximum probable flood	0 feet
ANTELOPE LAKE	
Maximum operating storage	22,566 acre-feet
Minimum operating storage	500 acre-feet
Dead pool storage	500 acre-feet
Maximum operating surface elevation	5,002 feet
Minimum operating surface elevation	4,950 feet
Dead pool surface elevation	4,950 feet
Shoreline, maximum operating elevation	15 miles
Surface area, maximum operating elevation	931 acres
Surface area, minimum operating elevation	57 acres
Drainage area	71 square miles
Average annual runoff	20,900 acre-feet

SPILLWAY	
Type: Ungated ogee crest with lined chute and flip bucket	
Crest elevation	5,002 feet
Crest length	60 feet
Maximum probable flood inflow	32,500 cubic feet per second
Peak routed outflow	23,400 cubic feet per second
Maximum surface elevation	5,025 feet
Standard project flood inflow	18,300 cubic feet per second
Peak routed outflow	12,900 cubic feet per second
Maximum surface elevation	5,017 feet
OUTLET WORKS	
Type: Steel-lined reinforced-concrete conduit beneath dam at base of right abutment—discharge into impact dissipator	
Diameter: 36 inches	
Intake structure: Uncontrolled low-level tower	
Control: Downstream control structure housing 10- and 24-inch butterfly valves—guard valve, 36-inch slide gate on intake tower	
Capacity	136 cubic feet per second

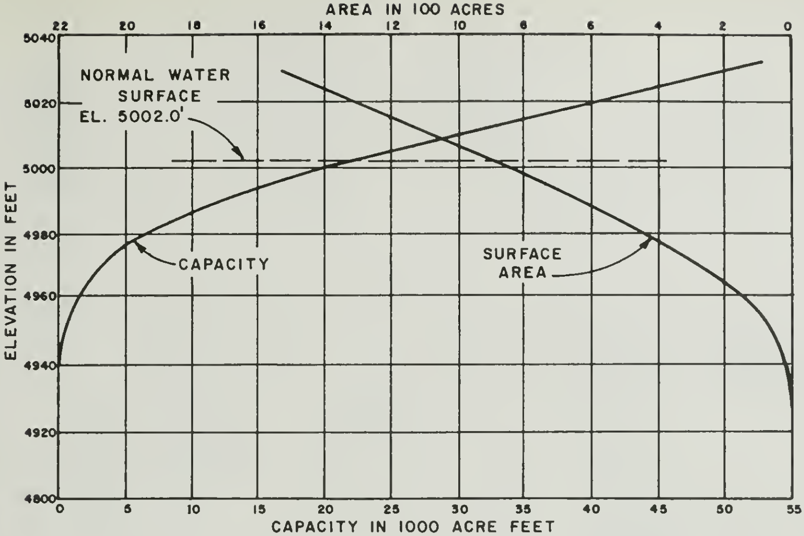

Figure 28. Area-Capacity Curves

earthfill structures consisting of upstream impervious zones of decomposed granite and downstream pervious zones of streambed sands and gravels. The embankment plan is shown on Figure 29, and the sections are shown on Figure 30. No instrumentation of the embankment was planned during design.

Stability Analysis. Embankment slope stability was analyzed by the Swedish Slip Circle method. Satisfactory safety factors were established under all anticipated loading cases. These cases included full reservoir and other critical reservoir levels along with earthquake loads. Earthquake loading was assumed to be an acceleration of the foundation (0.1g) in the direction of instability of the mass being analyzed. Design strength for the impervious material was determined by soil testing while that for the gravels was based on published information for similar material.

Settlement. No rigorous settlement analysis was made. Consolidation tests indicated that practically all settlement would occur during construction. A camber of 1% of the fill height was provided.

Construction Materials. On the basis of surface examination, boring logs, and soil test data, decomposed granite at a site in Antelope Creek about two-thirds of a mile upstream of the Dam was selected for impervious borrow. In-place moisture ranged from 4.4 to 13.8%, averaging 6.9%. Specific gravity values ranged from 2.67 to 2.80, averaging 2.74. In-place dry densities ranged from 92.2 pounds per cubic foot (pcf) to 100.8 pcf. A maximum dry density of 122.6 pcf at an optimum moisture content of 11.5% was obtained from material that had a field value of 100.8 pcf indicating a shrinkage factor of approximately 18%. Direct shear tests on impervious fill showed a strength of 33 degrees for effective stress analyses, and a strength of 31 degrees and cohesion of 200 pounds per square foot for total stress analyses. Permeability of the compacted material ranged from 0.0002 to 0.09 of a foot per day, averaging about 0.01 of a foot per day.

Streambed sands and gravels from within the reservoir area were selected for the pervious embankment. Areas of free-draining material were delineated based on exploration with backhoe trenches. Sampling was accomplished in the trenches. Average compacted dry density was found to be 137 pcf. Permeability for compacted samples in this density range was from 1 to 7 feet per day. Testing showed that permeability decreased rapidly as density increased to 140 pcf. Strength of the material, based on testing of similar materials, was assumed to be 38 degrees.

Foundation. Rock occurring at the Dam site is biotite granodiorite, which has a shallow cover of alluvium along the stream channel.

The channel section consisted of jointed fresh rock overlain by varying depths of silty sandy alluvium which was removed during construction. Outcrops of vertical and overhanging rock were shaped to a ½:1 maximum slope against which the embankment material could be compacted. A cutoff trench of variable width and a single grout curtain consisting of two 25-foot grouted zones (total depth 50 feet) were provided for the main dam.

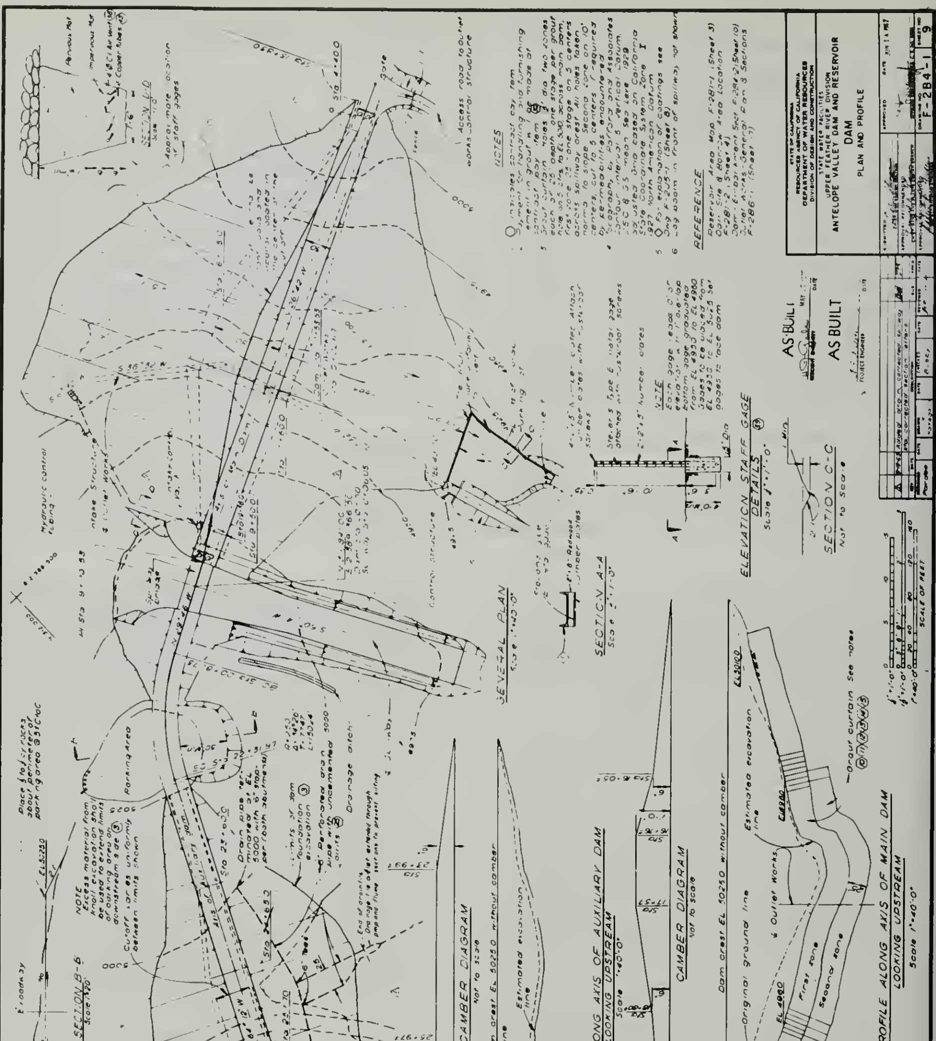

Figure 29. General Plan and Profile of Dam

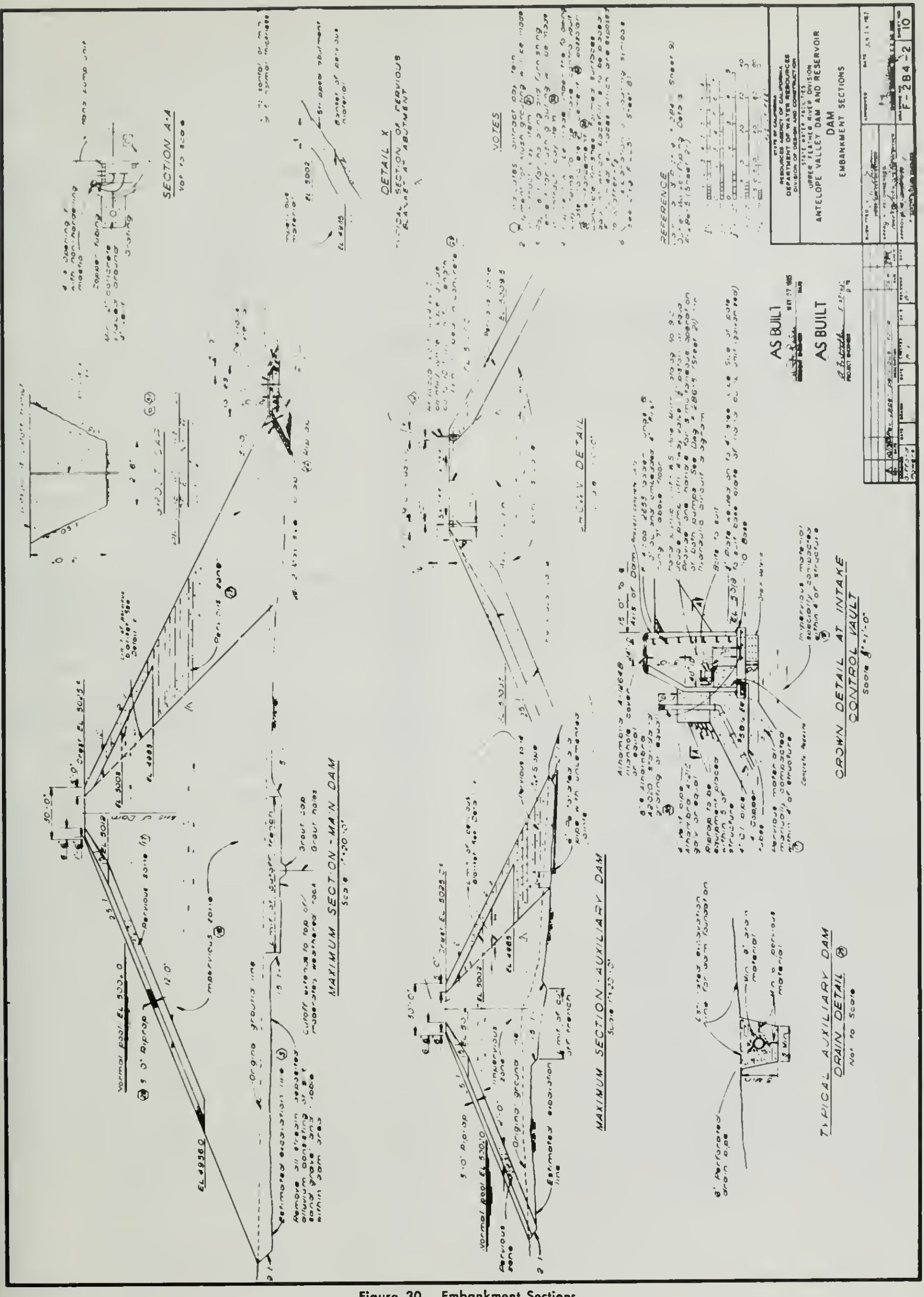

Figure 30. Embankment Sections

The auxiliary dam foundation consisted of decomposed granodiorite up to 35 feet in depth. All organic material and loose decomposed rock were removed from the embankment area. A cutoff trench to firm decomposed rock was provided near the middle of the impervious zone. No grout curtain was planned under the auxiliary dam.

No evidence was found of major faults, although the presence of slickensided fractures and a narrow shear zone in the left abutment of the auxiliary dam indicated minor displacements.

Outlet Works

Description. The outlet works is located along the base of the right abutment of the main dam. It consists of an intake tower with a hydraulically operated, 36-inch, slide gate and metal trashrack; a 36-inch, steel-lined, pressure conduit; and two discharge valves (10- and 24-inch butterfly valves) in a control structure at the toe of the Dam. Plan and sections of the outlet works are shown on Figure 31.

The discharge valves are located in an impact-type energy dissipator. Only the 10-inch valve was intended for throttling service; the 24-inch valve was intended to operate fully open when needed. Rating curves are shown on Figure 32. Operators are located immediately above the valves in the valve house. The reinforced-concrete dissipator structure has a width of 11 feet and a length of 17 feet, which discharges into a channel, returning the flow to the creek.

Structural Design. The intake structure is a tower approximately 30 feet high with an inside diameter of 36 inches and minimum wall thickness of 1 foot. It was designed as a free-standing tower capable of withstanding a wind pressure of 15 pounds per square foot as well as normal operating loads when inundated by the reservoir. Trashracks were designed for a differential hydrostatic head of 40 feet with yield stresses allowed. The racks are welded to a cover plate which has lifting hooks to enable removal of the entire assembly.

The slide gate and its hydraulic cylinder are enclosed in a concrete box which sits on top of the vertical inlet. The gate was designed for a 56-foot head acting uniformly over the entire gate disc in the direction of seating; a minimum factor of safety of 5 was required. Four hydraulic lines and the 4-inch, cast-iron, air pipe were placed in a riprap-protected trench that extends from the intake structure to the operating vault at the crest of the Dam (Figures 29 and 30).

A 12-inch bypass valve was installed at the base of the intake tower for stream diversion during construction.

The conduit is a 36-inch-inside-diameter, steel-lined, reinforced-concrete, cut-and-cover section. The conduit was designed so that the ¼-inch steel liner would carry all of the internal pressure, and external loads would be carried by the concrete (minimum thickness 10 inches). There are six 1½-foot-thick seepage-cutoff collars which extend 2½ feet from the top and sides of the conduit at 25-foot spacing.

Mechanical Installation. The 36-inch slide gate originally was intended to operate by means of a hand-operated hydraulic pump in the vault at the crest of the Dam. Due to the time required to operate the valve, a linkage was installed that is driven by an electric motor powered by a 12-volt battery. All other valves are operated directly by hand.

Spillway

Description. The spillway is located on the ridge between the main dam and the auxiliary dam. It consists of an unlined approach channel; a 60-foot-long, curved, ungated, ogee crest; a transition section; a 40-foot-wide rectangular chute; and a flip-bucket terminal structure (Figure 33). A bridge, open to the public, crosses the spillway at the crest and connects U.S. Forest Service roads in the area.

Hydraulics. Reservoir storage above the spillway reduces the standard project flood from a peak inflow of 18,300 cubic feet per second (cfs) to a peak outflow of 12,900 cfs with 8 feet of freeboard, and maximum probable flood from a peak inflow of 32,500 cfs to a peak outflow of 23,400 cfs with no freeboard.

Structural Design. Crest walls are cantilevered from the crest and were designed for two situations: backfill loads plus live loads on the bridge and backfill loads plus seismic loading.

The crest is anchored with No. 10 reinforcing bars embedded 6 feet into the rock foundation on approximately 5½-foot centers in both directions. The upstream end is provided with a 6-foot-deep cutoff that acts as a shear key. A 25-foot-deep grout curtain was provided in the cutoff invert. The crest was designed for loads transferred from the walls and for uplift conditions with water above the crest. The influence of anchor bars was discounted in these analyses.

Walls and floors of the transition and chute sections were constructed monolithically. Floors are anchored with No. 9 reinforcing bars embedded 6 feet into the rock on approximately 5-foot centers, in both directions. The flip bucket has a 50-foot radius and a 20-degree upward deflection. A shear key is placed near the center of the bucket, and the floor upstream of the key is anchored with No. 9 bars embedded 6 feet into the rock foundation at 5-foot centers. A 4-inch pipe drain is provided beneath the bucket.

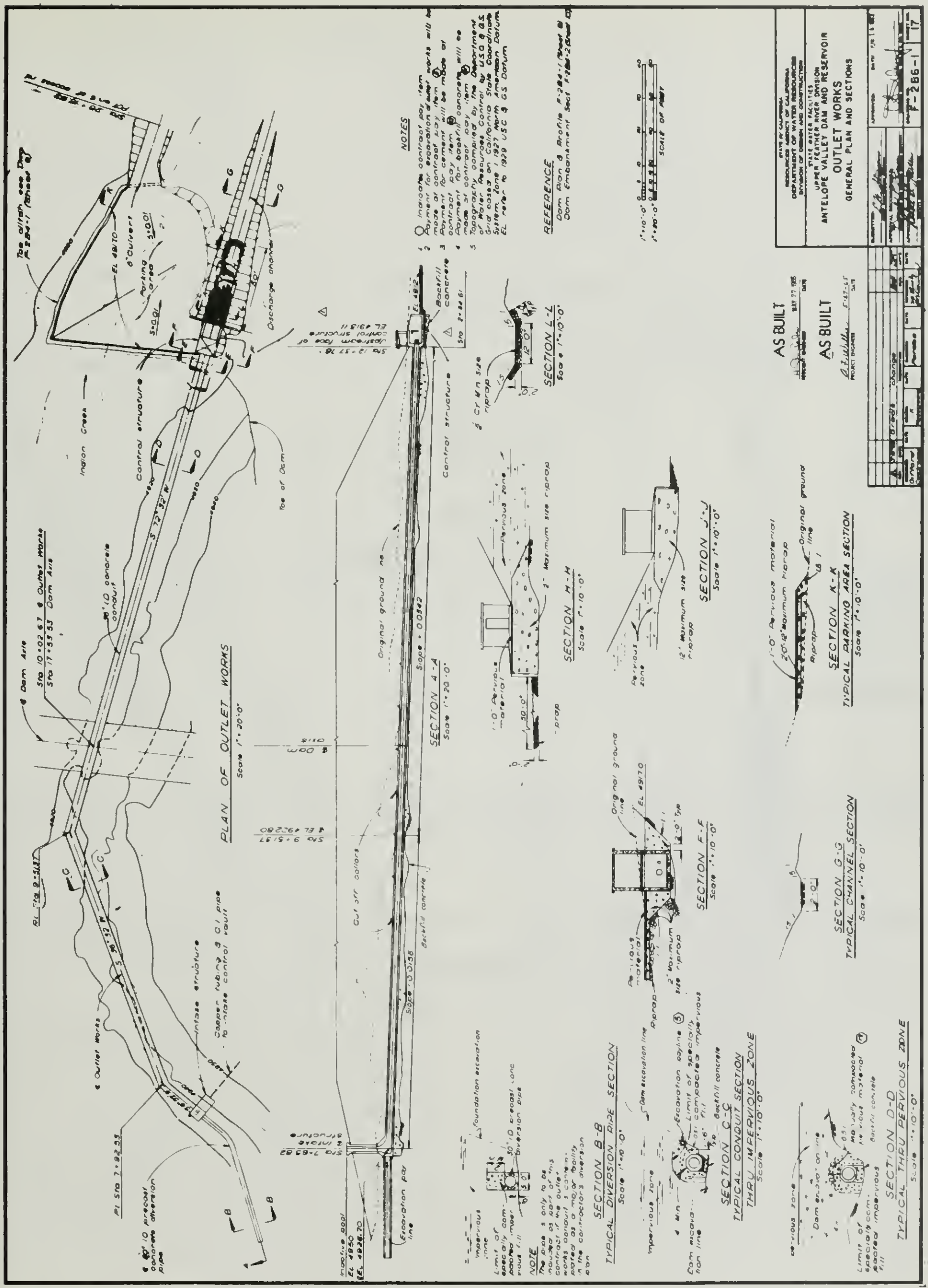

Figure 31. General Plan and Sections of Outlet Works

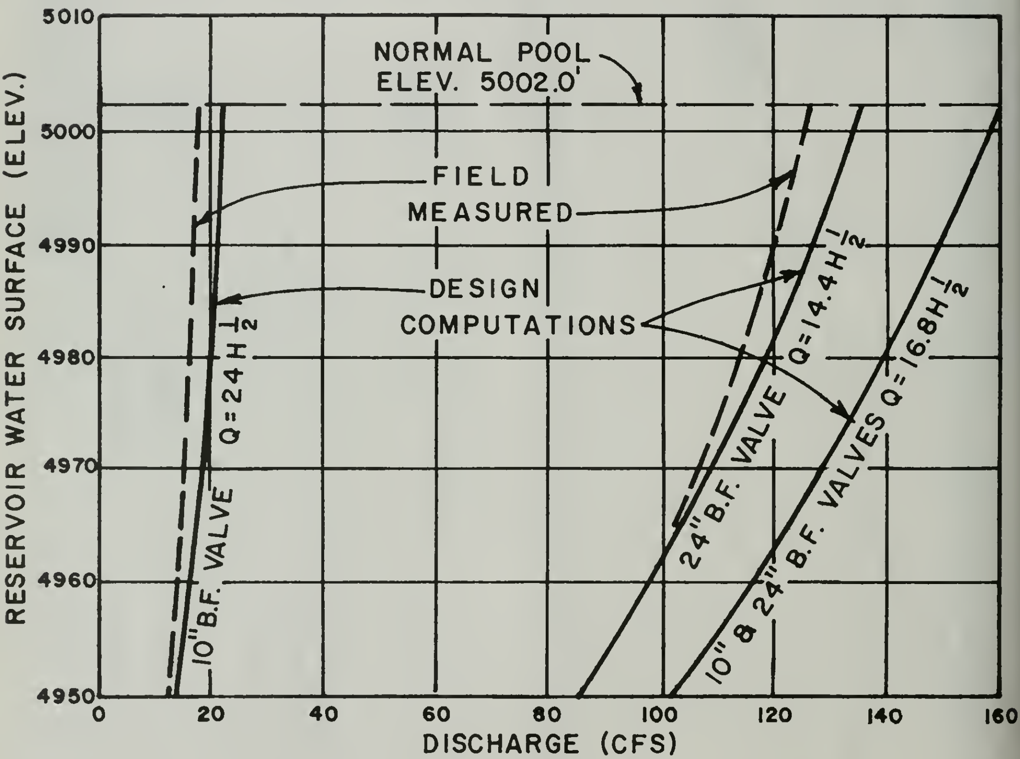

Figure 32. Outlet Works Rating Curves

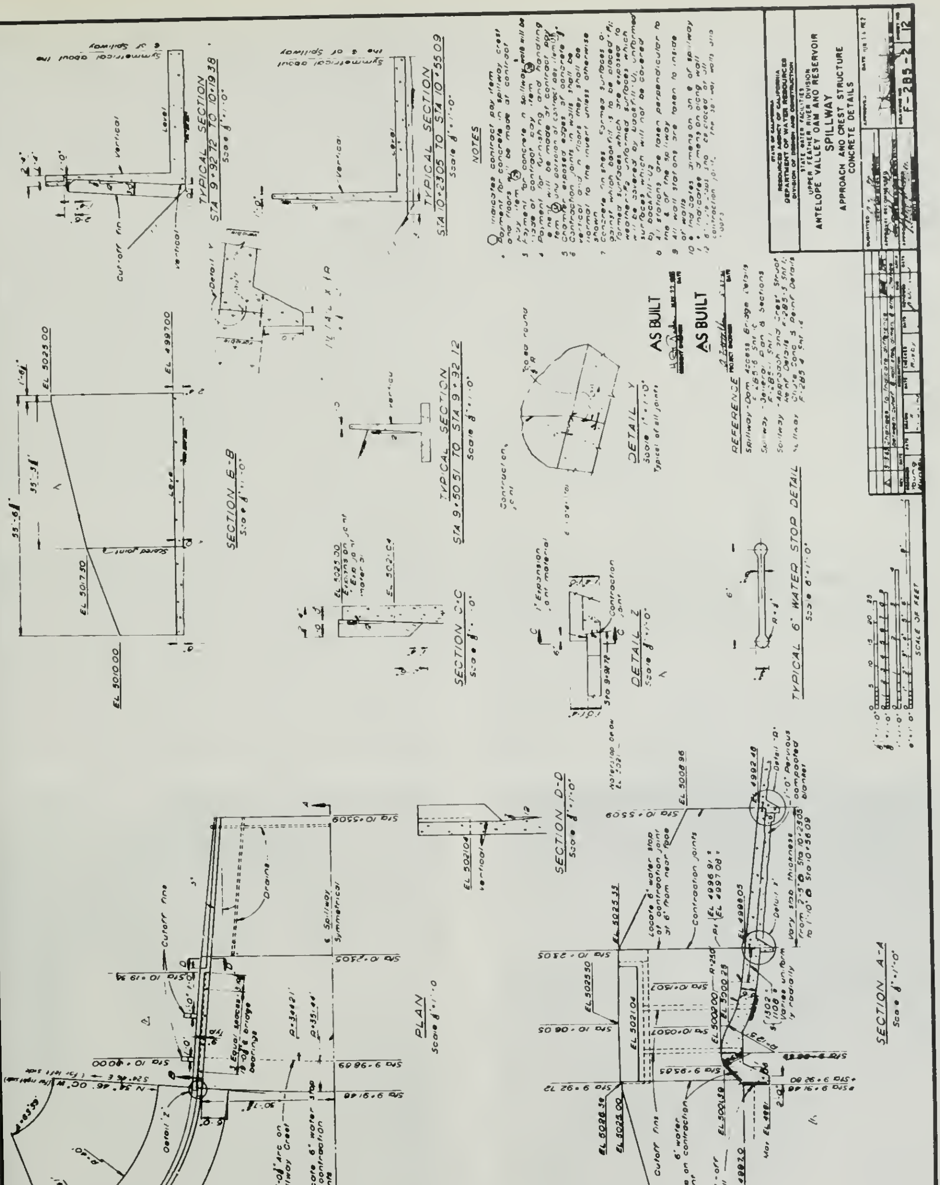

Figure 33. General Plan and Sections of Spillway

Construction

Contract Administration

General information for the construction contract for Antelope Dam is shown in Table 5. Principal features involved the construction of an earthfill embankment and spillway and the furnishing and installation of materials and equipment for a gated intake tower, a steel-lined conduit, and two butterfly discharge valves. Miscellaneous road work also was included in the contract. The contract was designated Specification No. 62-20.

TABLE 5. Major Contract—Antelope Dam

Specification	62-20
Low bid amount	$2,909,774
Final contract cost	$3,254,049
Total cost-change orders	$78,735
Starting date	8/29/62
Completion date	7/7/63
Prime contractor	Norman I. Fadel, Inc. and Granite Construction Company

Foundation

Dewatering. Dewatering of the foundation posed no problems other than the capping of several small springs encountered in the main dam foundation. Stream diversion was accomplished through a temporary pipeline laid through the foundation area (Figure 34).

Excavation. A total of 71,652 cubic yards of material unsuitable for foundation was removed from the streambed, abutments, and auxiliary dam saddle area. Excavation was accomplished with dozers, a backhoe, and hand labor to finish cleaning the abutments. Blasting was employed to shape foundation rock outcrops to proper slope. Where foundation material was subject to ripping, as on the right abutment and for the full length of the auxiliary dam, a well-defined cutoff trench was excavated. The main dam channel and left abutment were massive rock, and no cutoff trench was excavated.

Grouting. Grout caps were necessary at only three locations: at the bottom of the left abutment of the auxiliary dam (Figure 35), at each side of the spillway crest, and on the upper left abutment of the main dam. A single grout curtain 50 feet in depth was placed in two 25-foot zones for the full length of the main dam. Several 100-foot holes were grouted, and a secondary grout curtain across the outlet works foundation was added. Although no grouting of the auxiliary dam was assumed necessary during design, a short reach of the foundation was grouted because of a shear found in that area. A total of 724 cubic feet of grout was injected into the foundation.

Handling of Borrow Materials

Impervious. Impervious material was obtained from an area two-thirds of a mile upstream of the Dam on Antelope Creek (Figure 36). The borrow pit was moisture-conditioned by a sprinkler system prior to excavation and by a water truck during excavation. A total of 318,981 cubic yards of excavation yielded 271,353 cubic yards of compacted impervious fill.

Pervious. Pervious material was obtained from an area 1 mile upstream of the Dam on Indian Creek (Figure 36). A design modification during construc-

Figure 34. Temporary Diversion Pipe and Outlet Works Conduit

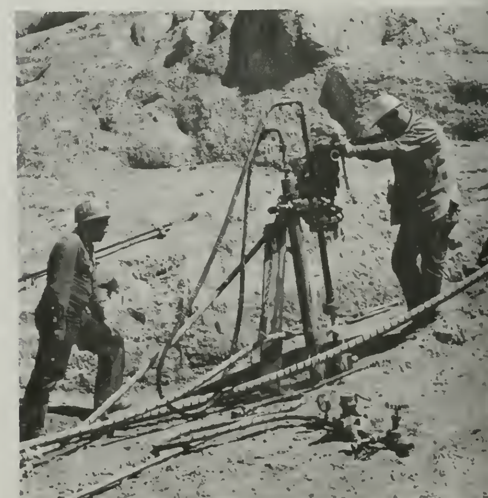

Figure 35. Drilling Grout Hole—Left Abutment Auxiliary Dam

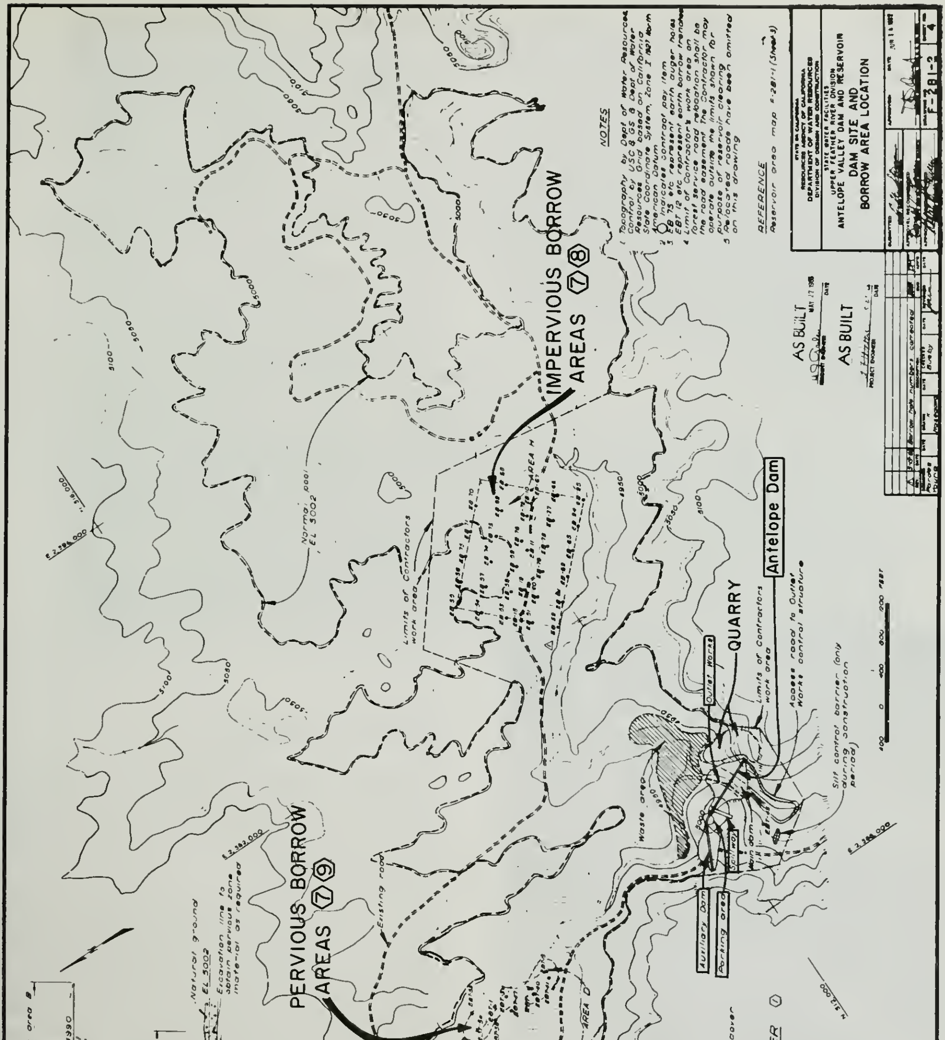

Figure 36. Location of Borrow Areas and Antelope Dam Site

tion utilized more impervious material, which was in abundant supply, and reduced the required yardage of pervious material which was in limited supply. A total of 113,097 cubic yards of excavation yielded 95,305 cubic yards of in-place pervious fill.

Excavation of pervious material presented a problem because a high percentage of the total quantity was below the water table. Best results were obtained by step excavation. One portion of the pit was lowered 2 to 3 feet below the general area; then borrow was taken from the higher area of the pit. This procedure allowed drainage of the gravels and escape of water to the stream by gravity.

Slope Protection. Riprap was quarried from several locations near the Dam site. Riprap particle size varied from a maximum of one cubic yard down to one-half of a cubic foot. A total of 13,526 cubic yards of riprap was placed on the upstream face of the Dam. Pervious fill was employed as bedding for riprap.

Embankment Construction

Impervious. Impervious fill was placed parallel to the axis in 6-inch lifts and compacted by 12 passes of a sheepsfoot roller as specified. Design recommendations for compaction were as follows: (1) minimum field dry density of 111 pcf, (2) field densities for 80% of the samples to be greater than 97% of laboratory dry density, and (3) an absolute minimum dry density of 95% of laboratory maximum. Material that did not meet these criteria was either rejected or reworked.

Figure 37. Outlet Works Control Structure

Pervious. Pervious material was spread and compacted by four passes of a crawler tractor. On numerous occasions, excessive moisture caused hauling and compacting equipment to bog down in the placed material. Special effort was required to haul and place the driest pit material.

Slope Protection. Riprap was end-dumped on the slope and placed with a rock rake to the 3-foot thickness specified.

Outlet Works

Excavation. The entire length of the outlet works conduit is founded in rock (Figure 34). When rock was not encountered at grade, overexcavation was directed to sound rock and backfilled to subgrade with concrete. Blasting was required to shape the trench and to obtain foundation grade. A backhoe was used to remove material, and all foundation overbreak was backfilled with concrete.

Concrete. Placement generally was routine and was accomplished with a truck crane and a bottom-dump bucket (Figure 37).

Spillway

Excavation. The major portion of the excavation was made with a shovel which loaded trucks for haul to the waste area. Some blasting was necessary to excavate near the flip-bucket end of the spillway. After rough excavation of the shear key trenches, hand labor was used for final shaping.

Concrete. Placement generally was routine and was accomplished with a truck crane and a bottom-dump bucket. The completed spillway is shown on Figure 38.

Concrete Production

Concrete was produced using a portable batch plant stationed near the Dam site and transported with three transit mix trucks. Coarse aggregate was obtained from dredger tailings near Oroville. Sand was supplied from the Pentz pit on Dry Creek. The contractor attempted to produce coarse aggregate from Indian Creek gravel but discontinued the operation due to high cost and low quality. Mixing water was obtained directly from Indian Creek.

As the concrete was placed during the summer, there was no need for cold-weather protection during construction.

An air-entraining agent was used in the concrete to provide resistance to deterioration from ensuing freezing and thawing cycles.

Reservoir Clearing

Clearing of the Dam site and reservoir area was started in September 1962, immediately following the

Figure 38. Spillway

removal of merchantable timber. Merchantable timber previously had been flagged by the U. S. Forest Service and removed under another contract. In general, the contractor's clearing method consisted of using a bulldozer to push over standing timber and move it into piles for burning. This initial rough clearing was followed by brush-raking most of the reservoir area. Raking proved to be insufficient since many logs, trees, and limbs were wholly or partly covered and were not gathered into the burning piles.

The contractor stopped this operation after burning permits were terminated in 1963. He started again late in the fall of that year but, before he could take care of the buried material, the ground froze making removal of the debris virtually impossible. Final cleanup was made in the spring of 1964 after the reservoir was partially filled. Floating debris was gathered by hand at the edge of the reservoir and placed on open rafts which were towed by motor boats to selected locations where the debris was burned at a later time.

Closure

Storage commenced in Antelope Lake during January 1964 even though some minor work remained to be done. During the spring, only stream maintenance releases were made, and storage rose to a maximum of 12,565 acre-feet by June. In October and November, all but 3,103 acre-feet of storage was released, in accordance with prior rights, to the downstream facilities of Pacific Gas and Electric Company. In January 1965, the Lake was filled and the first spill occurred.

Instrumentation

Embankment instrumentation was not included in the construction contract. After the reservoir filled, however, seepage appeared on the downstream face. Seven observation well piezometers were installed from the crest, seven from the downstream face of the Dam, and one from the downstream face of the auxiliary dam in the fall of 1965 (Figure 39). In addition, a 4½- to 5-foot-deep trench was excavated in the downstream face of the main dam at Station 17+10 from elevation 5,000 to 4,917 feet. The trench was partially filled with 3 feet of crushed aggregate, then brought to original grade with material excavated from the trench. The observation wells and the trench apparently punctured impervious layers accidentally created during placement of the downstream shell. The wells and trench now provide vertical drainage paths, and no further seepage has appeared on the surface.

Embankment stability was reviewed using the data from these installations, and it was determined that seepage, as observed, had not significantly affected the stability of the Dam.

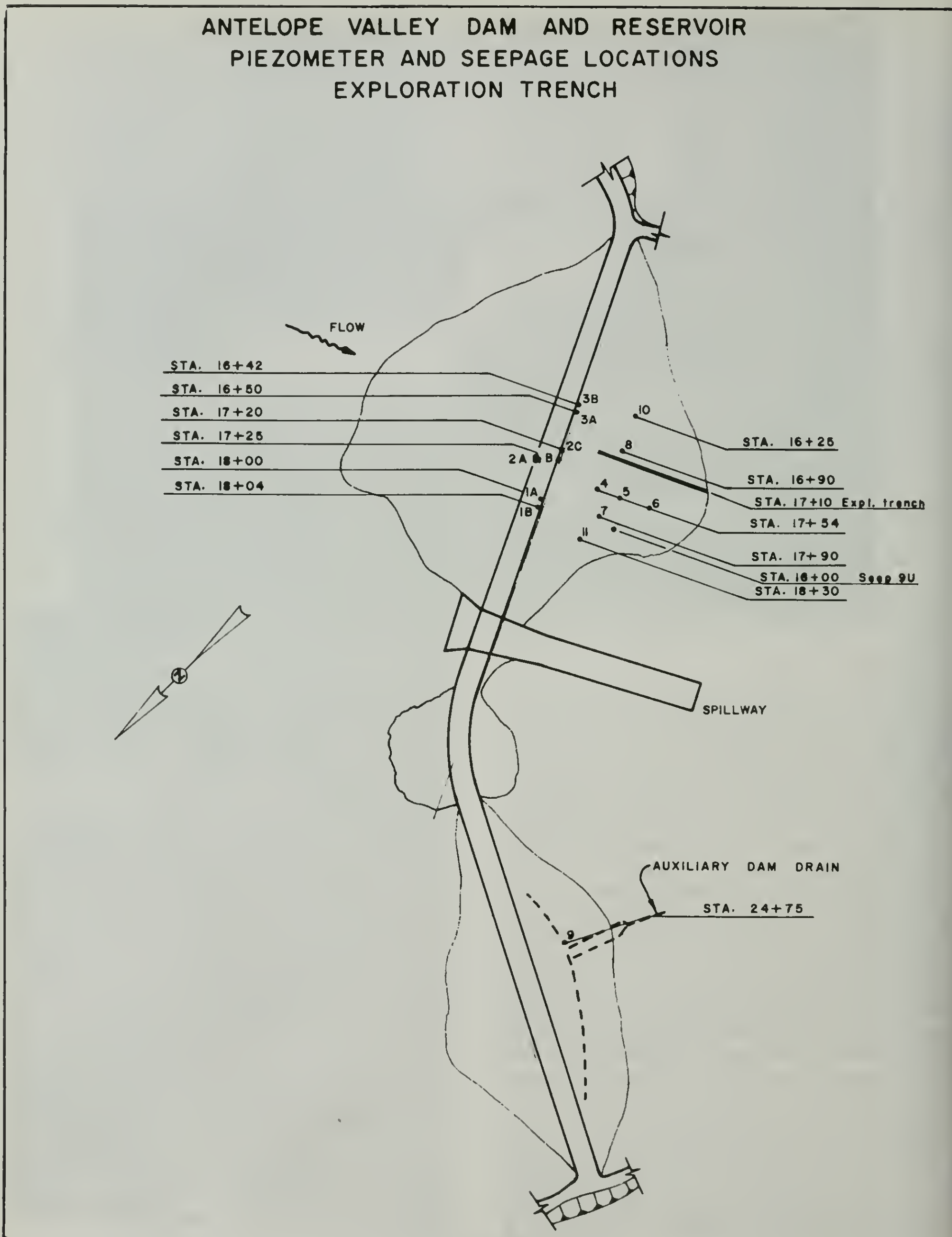

Figure 39. Location of Embankment Instrumentation

BIBLIOGRAPHY

California Department of Water Resources, Bulletin No. 59, "Investigation of Upper Feather River Basin Development: Interim Report on Engineering, Economic and Financial Feasibility of Initial Units", February 1957.

________, Bulletin No. 59-2, "Investigation of Upper Feather River Basin Development", October 1960.

________, "A Plan for the Recreation Development of Antelope Valley Reservoir", March 1962.

California Department of Public Works, "Northeastern Counties Investigation: Report on Upper Feather River Service Area", April 1955.

________, "Program for Financing and Constructing the Feather River Project as the Initial Unit of the California Water Plan", February 1955.

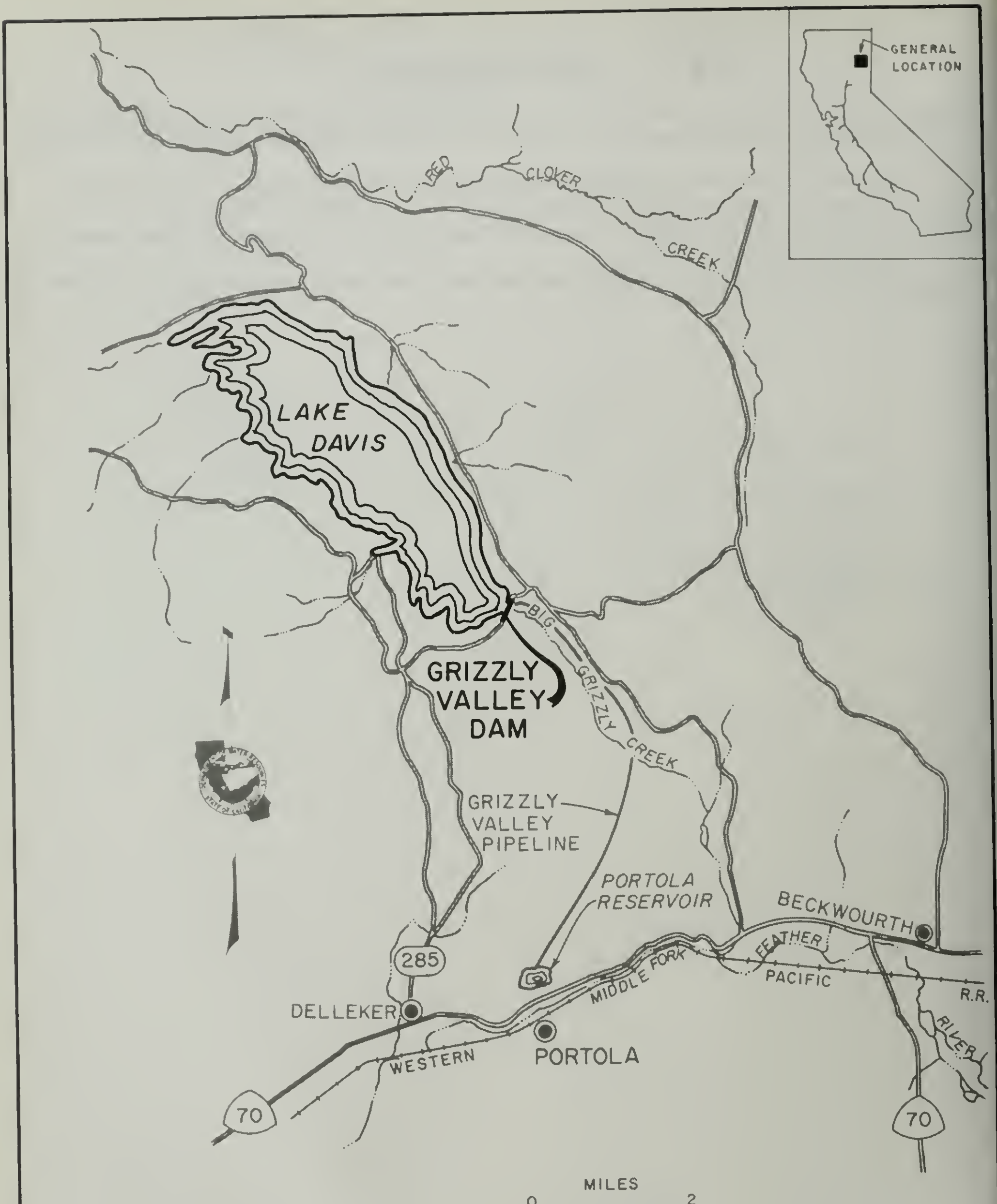

Figure 40. Location Map—Grizzly Valley Dam and Lake Davis

CHAPTER IV. GRIZZLY VALLEY DAM AND LAKE DAVIS

General

Description and Location

Grizzly Valley Dam is an earth and rockfill structure about 800 feet long at the crest with a structural height of 132 feet. An ungated, broad-crested, open-chute spillway is located on the right abutment. The outlet works consists of an inclined intake structure on the left abutment, a low-level intake, a 36-inch outlet conduit along the base of the left abutment, and a downstream control house. The reservoir, which is now named Lake Davis, has a capacity of 84,371 acre-feet of water and a water surface area of 4,026 acres.

The Dam is located 8 miles north of Portola within the Plumas National Forest on Big Grizzly Creek, a tributary of the Middle Fork Feather River. The nearest major road is State Highway 70 (Figures 40 and 41).

Figure 41. Aerial View—Grizzly Valley Dam and Lake Davis

A statistical summary of Grizzly Valley Dam and Lake Davis is shown in Table 6, and the area-capacity curves are shown on Figure 42.

Purpose

The primary purposes of Lake Davis are recreation, fish and wildlife enhancement, and domestic water supply. During the planning stage, integration of the yield from Lake Davis with that from Frenchman Lake by a canal to Sierra Valley was considered. Transportation charge for delivering Lake Davis water to the Frenchman service area was beyond the ability of the agricultural interests to pay; thus, this portion of the State Water Project was abandoned as financially infeasible. Agricultural water now released from Lake Davis is for the fulfillment of prior rights, about 900 acre-feet per year. Incidental flood protection is afforded by the reservoir but was not considered as a project purpose.

Chronology

Investigations of water and recreational development in the Upper Feather River Basin resulted in published reports in 1955 and 1957 (see Bibliography). In 1957, the Legislature authorized construction of five dams in this development. One of these was Grizzly Valley Dam.

After an economic comparison of four sites and four dam heights, the axis of the dam was chosen and the normal pool level was established at elevation 5,775 feet. Detailed design work was initiated in 1963. Construction began in October 1964 and was completed in 1967.

Regional Geology and Seismicity

Grizzly Valley is one of several fault block valleys found in the Sierra Nevadas and parallels the northwest structural trend of the northern Sierra Nevada. The former lake that periodically occupied Grizzly Valley in the geologic past was finally drained when Big Grizzly Creek cut its present gorge at the valley outlet. Granitic rocks outcrop near the valley floor. Andesitic lava flows and pyroclastics cap the adjacent higher ridges. Andesitic dikes cut the granitic rocks and are believed to be feeder dikes to the Tertiary andesitic volcanics. The crustal forces which caused the formation of the Valley also produced strong jointing, fracturing, and minor faulting in the bedrock at the Dam site.

Seismicity is believed to be only moderate, and no active faults were observed in the area. The site is near the seismically active area along the California-Nevada border.

Design

Dam

Description. The 132-foot-high dam was designed as a zoned earthfill structure. Plan, profile, and sec-

TABLE 6. Statistical Summary of Grizzly Valley Dam and Lake Davis

GRIZZLY VALLEY DAM

Type: Zoned earth and rockfill

Crest elevation	5,785 feet
Crest width	30 feet
Crest length	800 feet
Streambed elevation at dam axis	5,670 feet
Lowest foundation elevation	5,653 feet
Structural height above foundation	132 feet
Embankment volume	253,000 cubic yards
Freeboard above spillway crest	10 feet
Freeboard, maximum operating surface	10 feet
Freeboard, maximum probable flood	0.6 feet

LAKE DAVIS

Maximum operating storage	84,371 acre-feet
Minimum operating storage	90 acre-feet
Dead pool storage	90 acre-feet
Maximum operating surface elevation	5,775 feet
Minimum operating surface elevation	5,700 feet
Dead pool surface elevation	5,700 feet
Shoreline, maximum operating elevation	32 miles
Surface area, maximum operating elevation	4,026 acres
Surface area, minimum operating elevation	25 acres
Drainage area	44 square miles
Average annual runoff	25,000 acre-feet

SPILLWAY

Type: Ungated broad crest with lined chute and flip bucket

Crest elevation	5,775 feet
Crest length	30 feet
Maximum probable flood inflow	17,500 cubic feet per second
Peak routed outflow	2,420 cubic feet per second
Maximum surface elevation	5,784.4 feet
Standard project flood inflow	9,220 cubic feet per second
Peak routed outflow	1,190 cubic feet per second
Maximum surface elevation	5,780.8 feet

OUTLET WORKS

Type: Steel-lined reinforced-concrete conduit beneath dam at base of left abutment—discharge into impact dissipator

Diameter: 36 inches

Intake structure: Two-level inclined structure with 30-inch butterfly shutoff valves—low-level intake tower with concrete plug emergency bulkhead

Control: Downstream control structure housing 10- and 30-inch butterfly valves for stream release and a 16-inch butterfly valve at the beginning of Grizzly Valley Pipeline—24-inch butterfly guard valve in low-level intake conduit at junction with inclined structure

Capacity, stream maintenance	222 cubic feet per second
Design delivery to pipeline	8.25 cubic feet per second

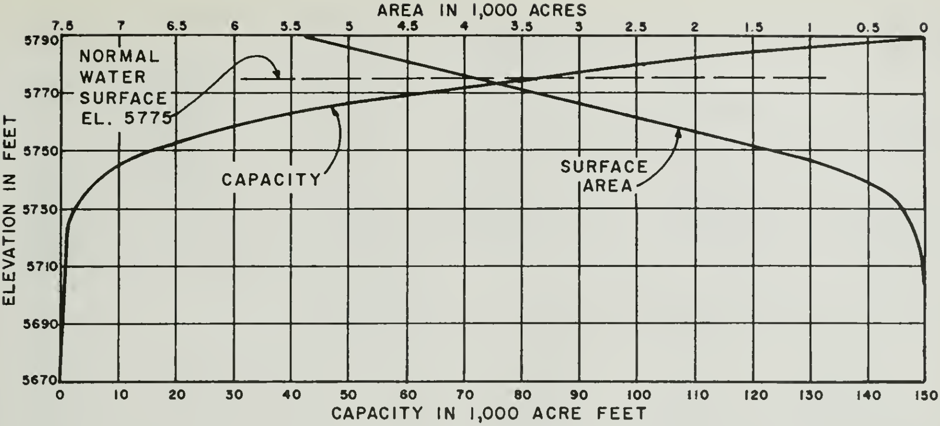

Figure 42. Area-Capacity Curves

tions of the Dam are shown on Figure 43. Zone 1 is compacted clay, Zone 2 is an upstream section of decomposed granite, Zone 3 is rolled rockfill, and Zone 4 is transition material consisting of a graded mixture of crushed rock and fine sand.

Stability Analysis. Embankment stability was determined by the Swedish Slip Circle method of analysis. Adequate safety factors existed for all loading conditions. Loading conditions analyzed included full reservoir and other critical reservoir levels coupled with earthquake loads. Earthquake loading assumed a horizontal acceleration of the foundation in the direction of instability of the soil mass being analyzed. The acceleration used was 0.1g. Zone 1 and 2 material parameters were based on soil testing. Material parameters for the other zones were selected after a review of available information on the properties of similar materials.

Settlement. Settlement analyses based on laboratory tests indicated the core could settle 4 to 5 inches at the maximum section after saturation. A camber of 12 inches at the maximum section, more than twice the long-time postconstruction settlement, was provided.

Construction Materials. Sediments from the former lakebed about one-half mile upstream of the Dam site were selected for the impervious borrow on the basis of surface examination and auger holes. Natural moisture ranged from 20 to 30% and specific gravity from 2.72 to 2.79. Shear tests showed a strength, effective stress basis, of 30 degrees with zero cohesion and a strength, total stress basis, of 11.3 degrees with 2,000 pounds per square foot (psf) cohesion. Permeability was determined to be 0.0002 of a foot per day.

Decomposed granite for the upstream transition was located adjacent to the lake sediments. Natural moisture was as low as 8%, and specific gravity was in the same range as the clayey sediments. Shear tests showed a strength of 35 degrees with zero cohesion, effective stress basis; and a strength of 20 degrees with 2,000 psf cohesion, total stress basis. Permeability was determined to be 0.02 of a foot per day.

The rockfill and transition material was assigned a strength of 38 degrees, zero cohesion, based on published data for similar materials. The permeability rate was greater than 2 feet per day. Streambed sands and gravels were explored by backhoe trenching. Holes were drilled at the proposed quarry site for determination of rock quality and thickness of weathered layer.

Foundation. The Dam site is in a deep gorge. Bedrock consists of a biotite-hornblende granodiorite, which is strongly joined and irregularly weathered. There are numerous outcrops of fresh rock on the abutments. A 50-foot-wide andesite dike cuts the granodiorite along the right abutment parallel to the stream channel. The dike rock is more resistant to weathering and is less jointed than the granodiorite. Channel fill consisted of gravel, sand, and angular blocks of andesite and granodiorite from 1 to 5 feet across. Depth of overburden at the Dam site averaged about 8 feet.

Foundation preparation consisted of removing alluvium, weathered granodiorite, and hard rock excavation to eliminate overhangs and steep slopes. A deeper cutoff trench was provided beneath the core to obtain foundation rock with less jointing and thus more impermeability. The grout curtain consists of 25- to 50-foot-deep holes across the foundations for the entire

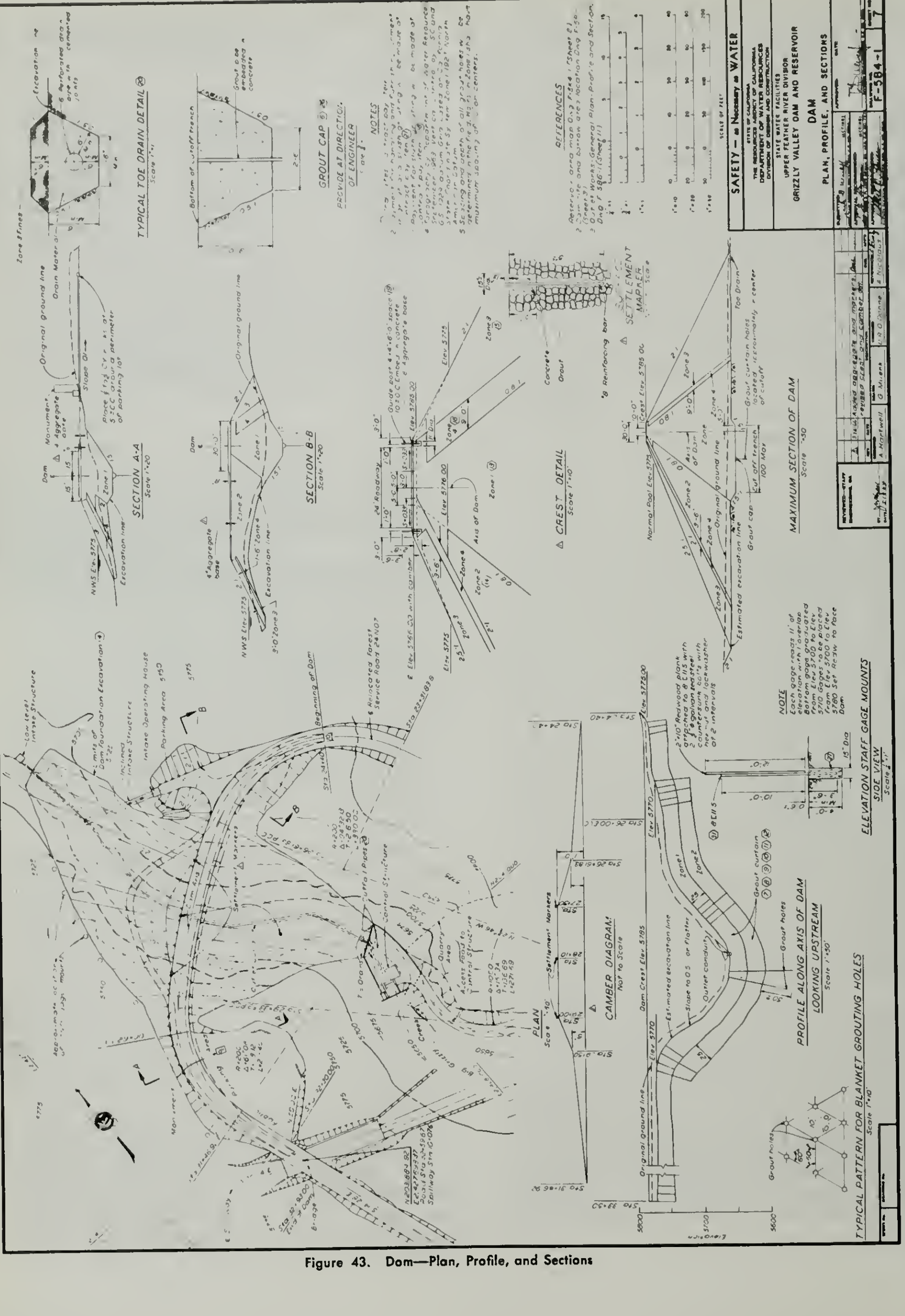

Figure 43. Dam—Plan, Profile, and Sections

length of the Dam and dike and holes 100 feet in depth in the streambed.

Instrumentation. Settlement monuments are provided on the edge of the crest and downstream face (Figure 44).

Outlet Works

Description. The outlet works (Figure 45) consists of a 36-inch-diameter reinforced-concrete conduit with a ¼-inch steel lining beneath the embankment along the base of the left abutment, a low-level intake, an inclined intake, and a downstream control structure.

The inclined intake is equipped with 30-inch butterfly valves at elevations 5,740 feet and 5,760 feet to permit withdrawal of water at selected levels for control of quality and temperature. The low-level intake is a free-standing tower with an opening at elevation 5,700 feet for drainage of the lower portion of the reservoir. A 24-inch butterfly valve is located in the connecting conduit at the base of the inclined intake to control flow from the low-level intake. A temporary 3-foot-diameter hole was provided through the base of the low intake for diversion during construction.

The downstream control structure contains a 10-inch butterfly valve for normal streamflow maintenance; a 30-inch butterfly valve for larger releases; and a 16-inch stubbed-off branch for connection of Grizzly Valley Pipeline, which was installed under a later contract by another agency. Both valves discharge into an impact-type dissipator structure.

Hydraulics. Outlet works components were sized to provide normal streamflow during the lower flow seasons and to allow drainage of the reservoir in one season with normal runoff. The outlet works rating curve is shown on Figure 46.

Structural Design. The low-level intake structure, approximately 37 feet high, was designed as a free-standing tower. The outside dimensions are 5 feet by 5 feet and the inside diameter is 36 inches. A concrete bulkhead that can be lowered to seal the tower opening is provided inside the trashrack for dewatering the conduit.

The inclined intake has a concrete-box section with inside dimensions of 6 feet by 6 feet. This structure was designed for external loading of the reservoir and embankment where applicable.

The concrete conduit was designed for external loads of the Dam and reservoir. The steel liner is capable of withstanding the internal pressure due to a full reservoir. Concrete cutoff collars are constructed outside the conduit at 40-foot centers along the reach passing under the dam core. Control houses are located at the toe and crest of the Dam.

Mechanical Installation. All valves are operated manually by handwheels or hydraulic cylinders.

Spillway

Description. The spillway is located on the right abutment about 250 feet from the main embankment. It consists of an unlined approach channel; an ungated broad-crested weir; a 30-foot-wide, rectangular, concrete chute; and a flip-bucket terminal structure (Figure 47). A bridge crosses the spillway at the crest connecting U. S. Forest Service roads.

Hydraulics. Storage above the spillway reduces the standard project flood from a peak inflow of 9,220 cubic feet per second (cfs) to a peak outflow of 1,190 cfs while retaining 4 feet of freeboard on the Dam and reduces the maximum probable flood from a peak inflow of 17,500 cfs to a peak outflow of 2,420 cfs with 0.6 of a foot of freeboard.

Structural Design. Crest walls were designed for normal earth loads plus live loads on the bridge and for normal earth loads plus a seismic acceleration of 0.1g. They were constructed monolithically with the crest.

At the crest, anchor bars were designed to resist uplift resulting from passage of floods. The upstream end is provided with a 6-foot-deep key. The crest was designed for loads transferred to it from the walls.

Walls and floors of the transition and chute sections were constructed monolithically. Shear keys were placed at each contraction joint. Design considered backfill loads and loads due to spillage.

The flip bucket discharges with a slight downward deflection 100 feet above streambed. The floor upstream of the cutoff (Figure 47) is anchored with two rows of No. 8 bars embedded 9 feet into the rock foundation at 7½-foot centers.

Grizzly Valley Pipeline

Grizzly Valley Pipeline supplies water for municipal and industrial use in Plumas County. The facility was designed to take 8.25 cfs with the water surface in Lake Davis at elevation 5,750 feet.

Grizzly Valley Pipeline was designed and constructed under the direction of the Plumas County Flood Control and Water Conservation District. The Department of Water Resources' role was limited to review and approval of the plans and monitoring of construction.

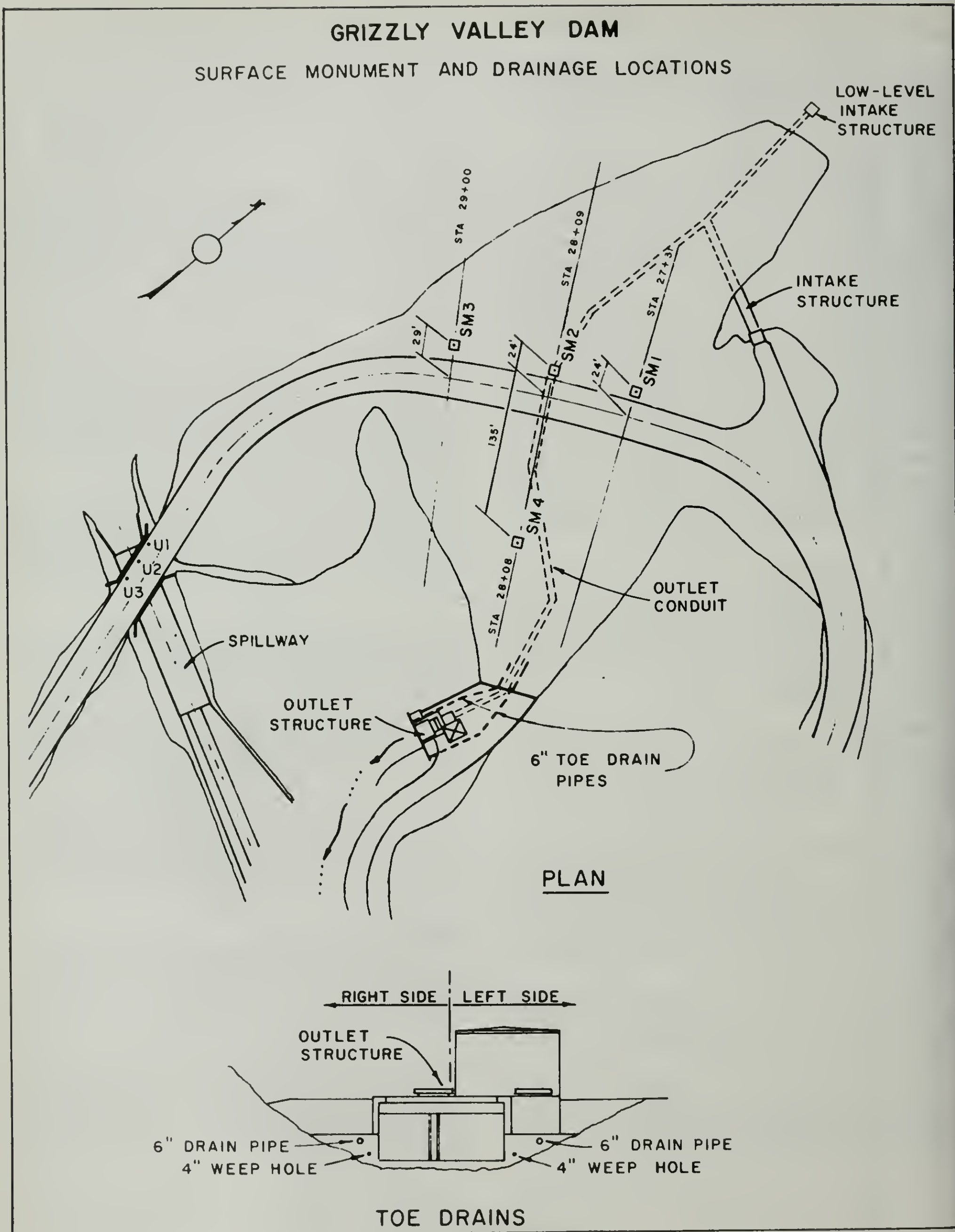

Figure 44. Location of Embankment Instrumentation

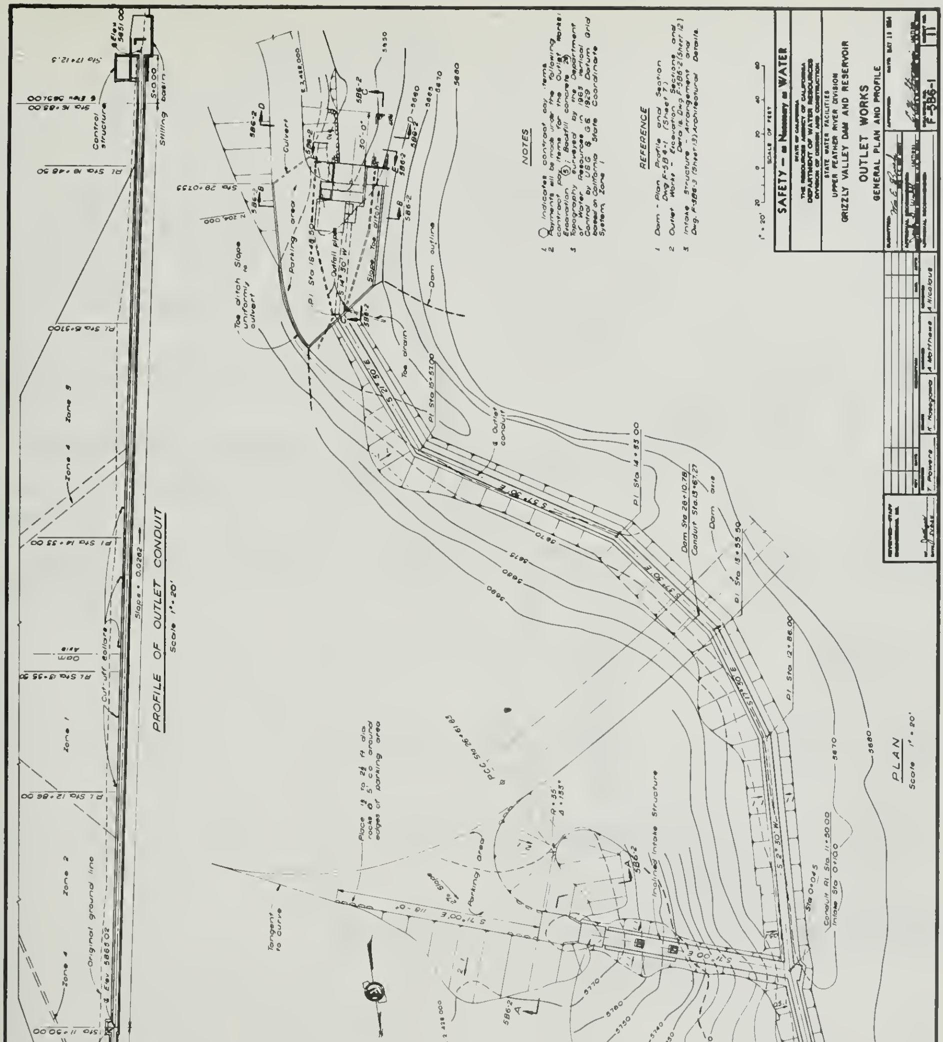

Figure 45. General Plan and Profile of Outlet Works

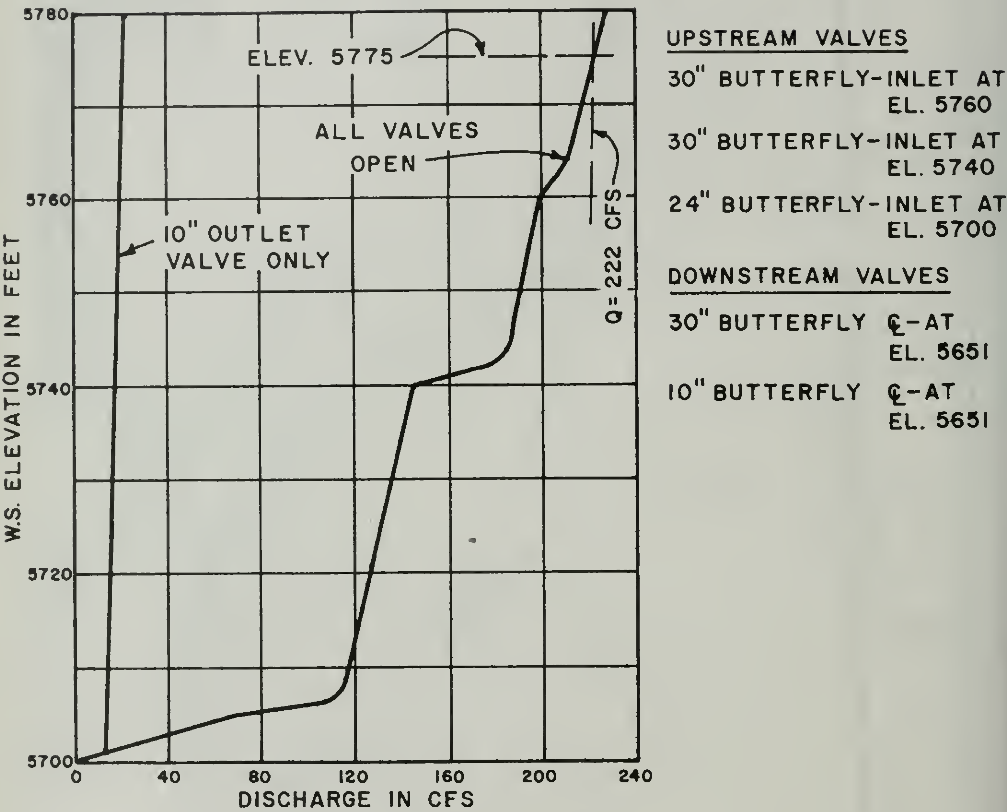

Figure 46. Outlet Works Rating Curve

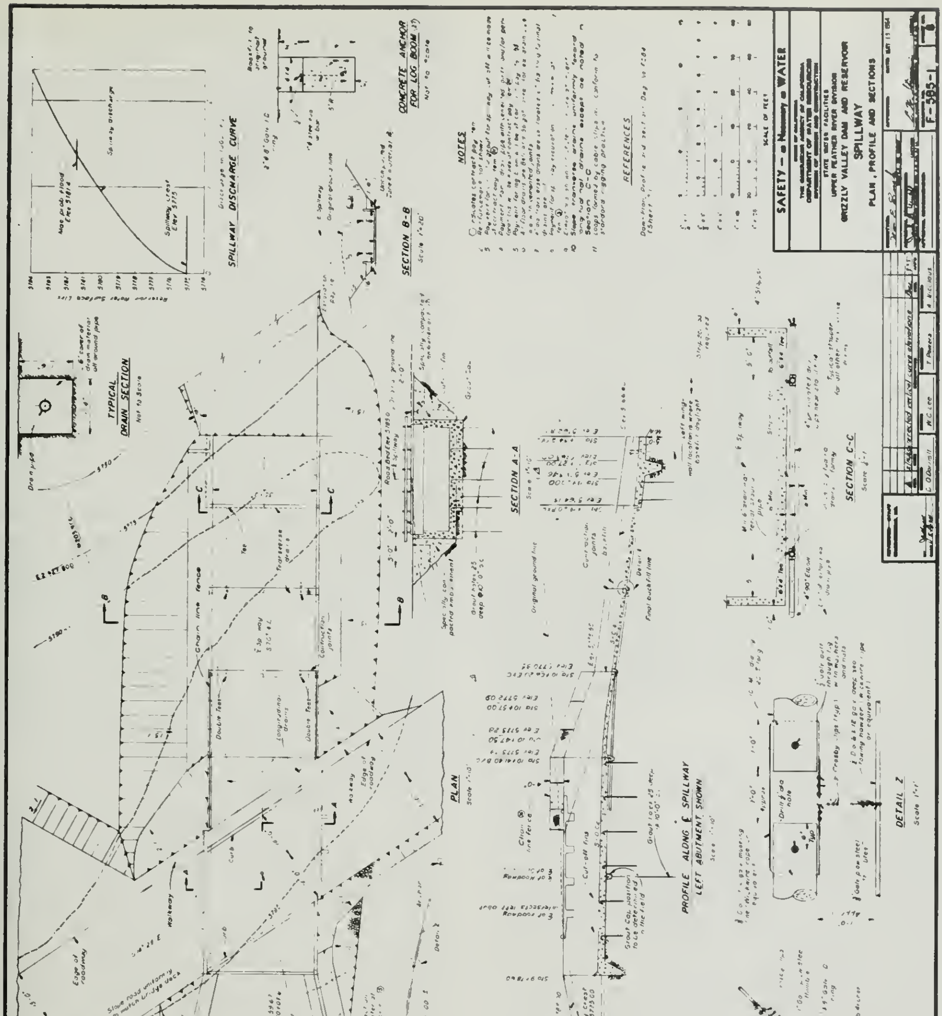

Figure 47. General Plan and Profile of Spillway

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Construction

Contract Administration

General information for the contract for the construction of Grizzly Valley Dam is shown in Table 7. The work was performed under the provisions of Specification No. 64-22. The principal features involved in the construction of Grizzly Valley Dam and reservoir were a zoned embankment consisting of an impervious earth core and rock shells with river gravel transition zones; a concrete, chute-type, ungated spillway; outlet works; the clearing of the reservoir; and the relocation of portions of existing U.S. Forest Service roads.

TABLE 7. Major Contract—Grizzly Valley Dam

Specification	64-22
Low bid amount	$1,833,131
Final contract cost	$2,269,382
Total cost-change orders	$66,020
Starting date	10/20/64
Completion Date	9/29/67
Prime contractor	Pascal & Ludwig, Inc.

Diversion and Care of Stream

Diversion of Big Grizzly Creek during construction was the responsibility of the contractor. During the first construction season, stream diversion was made through a temporary pipe. Diversion during the second season was through the outlet works. The 36-inch-diameter opening in the base of the intake tower was plugged with concrete after its use for stream diversion was discontinued.

Excavation for Dam and Dewatering Foundation

The left abutment was stripped with a "Yo-Yo" tractor anchored to a large tractor acting as an anchor.

In contrast to the left abutment, the right abutment consisted of many steep overhanging outcrops of fresh rock. The slope was shaped by blasting and backfilling with concrete. A cutoff trench was provided in the foundation under the Zone 1 core upstream of the axis. The total amount of foundation excavation was about 65,000 cubic yards.

Grouting was done in two stages. In the first stage, a curtain 25 feet in depth was extended across the entire length of the dam foundation. Below elevation 5,770 feet on the foundation, a second stage was extended to a depth of 50 feet. Two holes were extended to a depth of 75 feet. The total grout take of 1,837 cubic feet was about half the estimated 3,500 cubic feet.

After stripping and grouting were completed, artesian water problems were encountered. Holes 2½ inches in diameter were drilled into the foundation, pipes were grouted in, and the water table lowered by pumping. Drains up to 24 inches in diameter also were installed and pumped. After the fill material was compacted around the drains, they were sealed with concrete.

Handling of Borrow Materials

Borrow materials were obtained from the areas shown on Figure 48.

Impervious Zone 1, compacted clay, was disked in place to aid drying, spread on a stockpile, and disked again before being brought to the Dam. Excavation across layers varying from fat clay to decomposed granite was necessary to ensure that the embankment was homogeneous in texture and uniform in moisture content. A total of 101,750 cubic yards of clay was placed in the Dam.

Semipervious Zone 2, decomposed granite, was delivered directly to the fill. A total of 54,592 cubic yards was placed in the Dam.

Pervious material for Zone 3 and riprap were quarried from an andesite knoll on private land two-thirds of a mile south of the Dam and from the designated area in granodiorite adjacent to, and immediately downstream from, the Dam on the left bank. The bulk of Zone 3 rock came from the andesite knoll quarry where excavation was shifted between several areas but finally confined to an 80- by 150-foot swale surrounded on three sides by fines and overburden.

The material was processed by ripping, blasting, and raking to comb the rocks from the fines. The rock then was pushed into piles where a loader, equipped with a rock bucket, sifted the remaining fines from the piles and loaded the material into dump trucks for placement in the embankment. A total of 84,442 cubic yards of rock fill was placed in the Dam.

Zone 4 material, the transition zone, was produced by crushing, screening, and blending river-run sand with the andesite. This material was delivered to the Dam in dump trucks. A total of 13,953 cubic yards of transition zone material was placed in the Dam.

Embankment (Figure 49)

Zone 1 material was spread, leveled, scarified, and rolled with 12 passes of a sheepsfoot roller with each lane overlapping the last one rolled. Due to the confined working area in the bottom of the Dam, it was necessary to compact Zone 1 material in a direction normal to the dam axis. At elevation 5,680 feet, the working area had expanded sufficiently to permit placing and rolling to be done parallel to the dam axis.

Zone 2 material was spread by loaders, leveled, and watered before being compacted by a vibratory smooth-drum roller.

Zone 3 material was end-dumped on the Dam and dozed into place. To obtain proper filtration, Zone 3 material within 9 feet of the downstream edge of Zone 4 contained at least 20% material smaller than 4 inches in size.

Zone 4 material was end-dumped in position, then spread and compacted by a large tractor.

Outlet Works

Excavation. Blasting was required in most of the trench for the outlet pipe. Overexcavation was back-

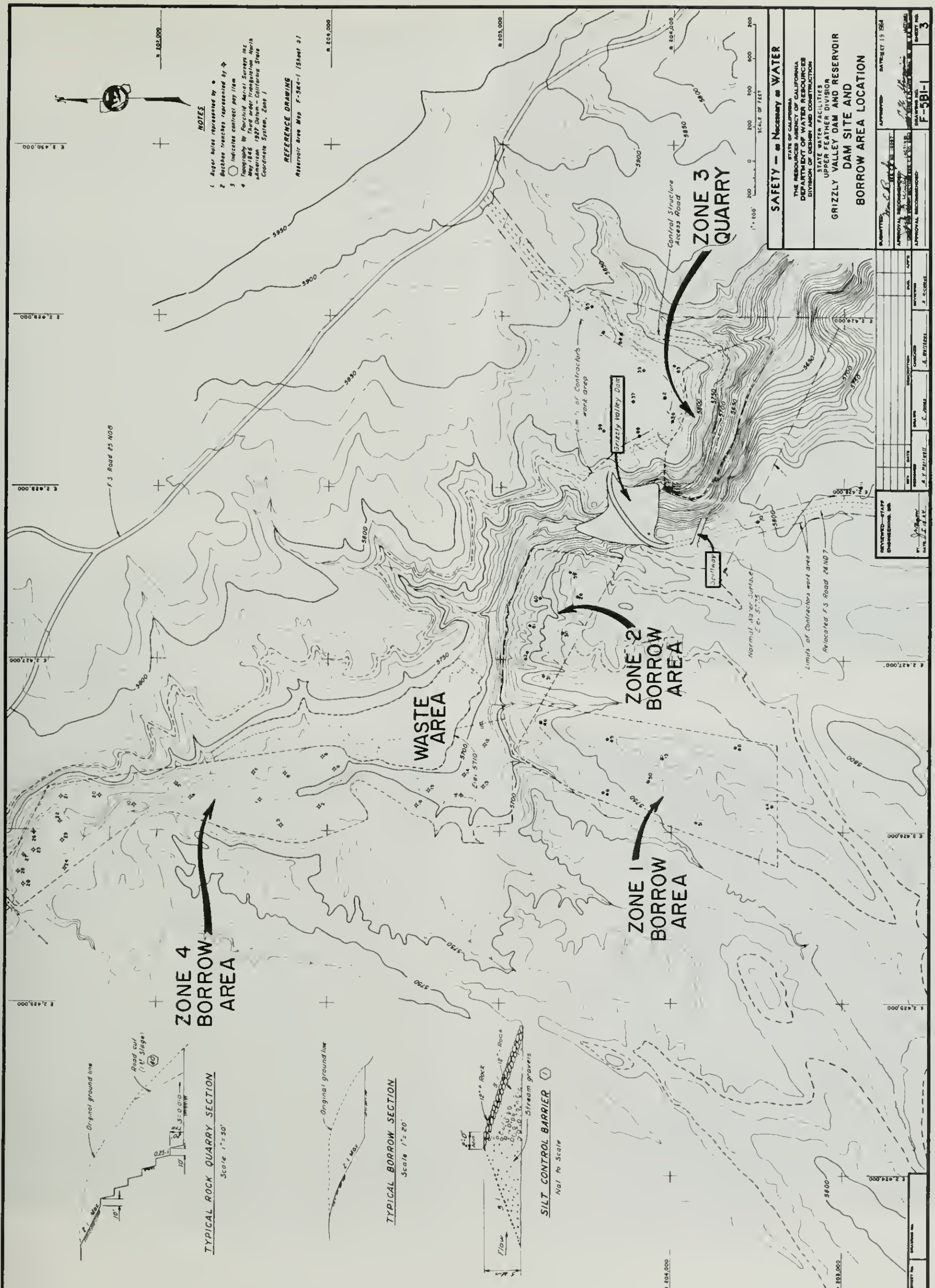

Figure 48. Location of Borrow Areas and Grizzly Valley Dam Site

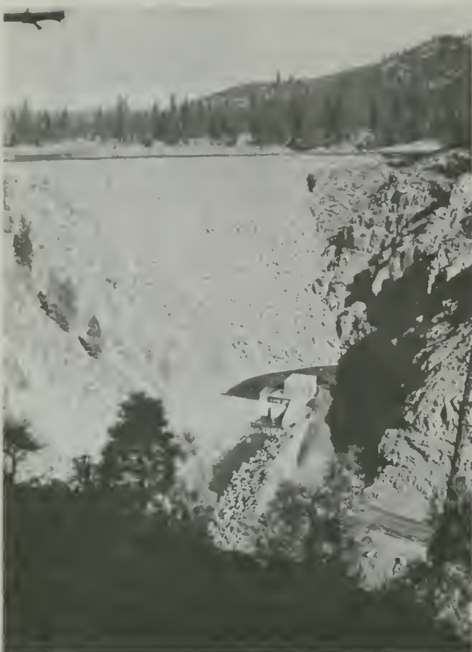

Figure 49. Completed Embankment

filled with concrete to the subgrade of the conduit. Metal chairs were set in the backfill concrete to support the steel liners.

Concrete. Dewatering of the trench was difficult, and a number of small pumps located in strategic areas were utilized to keep free water to a minimum during concrete placement. Concrete was cured with white pigmented compound and was protected with insulating blankets where necessary.

To avoid flooding of the conduit by winter runoff, both low-level and inclined intakes were placed under adverse weather conditions to elevation 5,694 feet. Mixing water was heated to maintain placement temperature at approximately 50 degrees Fahrenheit. After placement, the structures were tented with polyethylene and heated by salamanders. The balance of the concrete for the two structures was placed under favorable weather conditions and cured by normal methods. Both control houses are concrete block structures with reinforced-concrete slab roofs (Figures 50 and 51). A total of 1,412 cubic yards of concrete was placed in the outlet works.

Figure 50. Control House at Dam Crest

Figure 51. Control House at Dam Toe

Mechanical Installation

Butterfly valves in the sloping intake and the outlet structure (Figure 52) are operated directly by hand, while the 24-inch butterfly valve at the base of the sloping intake is operated by a hand-pump hydraulic unit. The concrete bulkhead inside the trashrack of the low-level intake is used for dewatering the outlet to allow visual inspection. It is raised and lowered by a diver using an A-frame hoist.

Spillway

Excavation. The spillway required little blasting. Overexcavation was backfilled to subgrade with concrete. The end key was excavated to sound rock.

Concrete. Spillway slabs were struck off with a slip form and hand-finished from plank bridges with rubber floats. Spillway walls were formed with metal forms. Due to temperatures below freezing, the concrete was cured using wet rugs covered by insulating blankets. This resulted in suitable curing temperatures for the slab and walls.

The spillway bridge was constructed during warmer weather; thus, only wet rugs were used in curing.

A total of 520 cubic yards of concrete was placed in the spillway, and 299 cubic yards were placed in the bridge and culvert headwalls.

The completed spillway chute is shown on Figure 53 and the spillway approach with log boom on Figure 54.

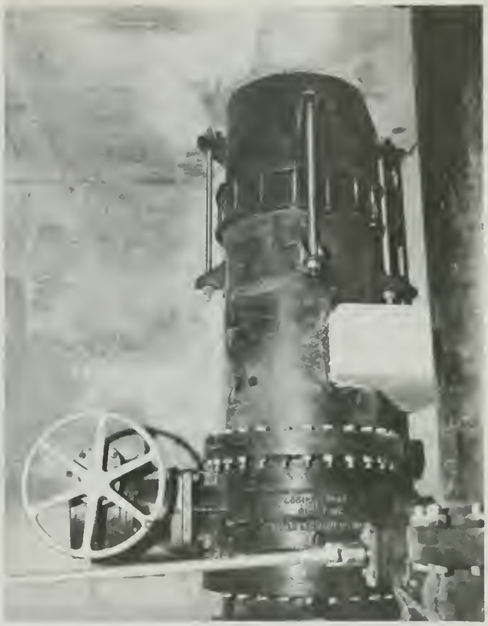

Figure 52. Outlet Works—Butterfly Valve in Outlet Structure

Figure 53. Spillway Chute

Figure 54. Spillway Approach and Log Boom

Concrete Production

Concrete was manufactured with a 3-cubic-yard batcher located about one-third of a mile upstream from the Dam site. During 1965, coarse aggregate was obtained from Graeagle, a small town about 20 miles from the Dam site. Because production at this plant was uncertain, in 1966 the contractor elected to import concrete aggregate from Oroville, about 95 miles away. Mixing water was obtained directly from Big Grizzly Creek. Concrete was mixed and transported to placement in 6-cubic-yard transit mix trucks.

An air-entraining agent was used in the concrete to provide resistance to deterioration due to freezing and thawing cycles which occur in the vicinity.

Reservoir and Other Clearing

Merchantable timber was felled and decked on the access roads, quarry areas, and Dam site in November 1964 and in the spillway area early in 1965. The remainder of the clearing was begun on March 29, 1966 and completed on September 29, 1967. Reservoir clearing was done to the normal high water line, elevation 5,775 feet. The downstream two-thirds of the shoreline was cleared of all vegetation between the 5,765- and 5,775-foot contours. Water storage in Lake Davis was begun in the fall of 1966 prior to completion of clearing, and the level rose higher than anticipated, inundating several of the burn piles below the 5,765-foot contour. The contractor was forced to use a great deal of hand labor during the summer of 1967 because of high water and wet ground.

Initial Reservoir Filling

No problems were encountered in filling the reservoir. Storage was started on October 18, 1966 and rose to 53,507 acre-feet in June 1967. In succeeding years, storage dropped during the summer months but increased during the rainy season. The first spillage occurred in January 1969, and storage has remained above 70,000 acre-feet since that time.

BIBLIOGRAPHY

California Department of Water Resources, Bulletin No. 59, "Investigation of Upper Feather River Basin Development: Interim Report on Engineering, Economic and Financial Feasibility of Initial Units", February 1957.

———, Bulletin No. 59-2, "Investigation of Upper Feather River Basin Development", October 1960.

———, Bulletin No. 117-3, "Lake Davis: Recreation Development Plan", July 1965.

———, Bulletin No. 128, "Lake Davis: Advance Planning Report", May 1965.

California Department of Public Works, "Northeastern Counties Investigation: Report on Upper Feather River Service Area", April 1955.

———, "Program for Financing and Constructing the Feather River Project as the Initial Unit of the California Water Plan", February 1955.

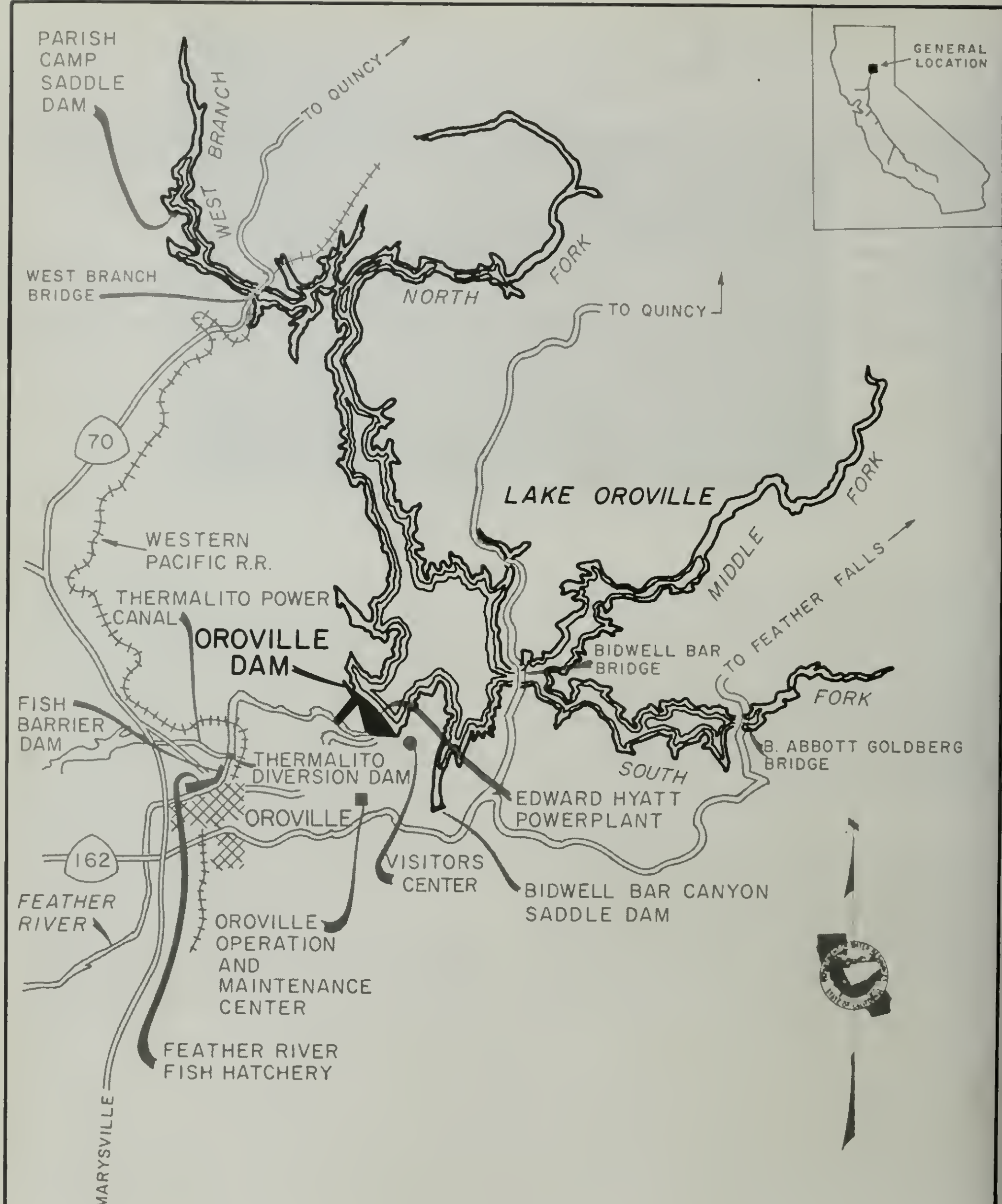

Figure 55. Location Map—Oroville Facilities

CHAPTER V. OROVILLE DAM AND LAKE OROVILLE

General

Description and Location

Oroville Dam, on the Feather River, is the highest earthfill dam in the United States. It rises 770 feet above streambed excavation and spans 5,600 feet between abutments at its crest. The 80,000,000-cubic-yard embankment is made up of an inclined impervious clay core resting on a concrete core block, with appropriate transitions and rock-filled shell zones on both sides.

The spillway, located on the right abutment of the Dam, has two separate elements: a controlled or gated flood control outlet, and an uncontrolled emergency spillway. The flood control outlet consists of an unlined approach channel, a gated headworks, and a lined chute extending to the River. The emergency spillway consists of a 1,730-foot-long, concrete, overpour section with its crest set 1 foot above normal maximum storage level. Emergency spill would flow to the River over natural terrain.

Two small embankments, Bidwell Canyon and Parish Camp Saddle Dams, complement Oroville Dam in containing the 3,537,577-acre-foot Lake Oroville. They are 47 and 27 feet high, respectively.

Most of the water released from the Lake passes through Edward Hyatt Powerplant, located in the left abutment of Oroville Dam. The plant's 678.75-megawatt output is achieved by means of three conventional generators rated at 123.2 MVA each, driven by vertical Francis-type turbines, and three motor-generators, rated at 115 MVA each, coupled to Francis-type reversible pump-turbines. The latter units allow off-peak pumped-storage operations. The underground powerhouse measures approximately 550 feet long, 69 feet wide, and 140 feet high.

The intake for the Powerplant is a sloping concrete structure located on the left abutment just upstream from the Dam. It consists of two parallel intake channels, one each for the two 22-foot-diameter penstock

Figure 56. Aerial View—Oroville Dam and Lake Oroville

tunnels. The intake openings are protected by stainless-steel trashracks. Beneath the trashracks and on top of the intake channels are guide rails for a shutter system. Each shutter is approximately 40 feet square. The setting of the shutters determines the level from which water is withdrawn from the reservoir and, in turn, the temperature of the water. Temperature of the water is critical for local agricultural purposes, mainly rice, and for the downstream fishery. Emergency closure of the penstocks can be accomplished by means of hydraulically activated roller gates positioned at the base of the channels at the entrance of the penstocks.

Discharges from the underground powerplant are conveyed to the Feather River by two 35-foot-diameter former diversion tunnels. The "river outlet" also is included in one of these tunnels. This facility, with a maximum release capability of 5,400 cubic feet per second (cfs), was used to make downstream releases until the reservoir reached the penstock intake level. It is being preserved to serve the same function in the event there is ever a prolonged total outage of the Powerplant.

Another outlet serves Palermo Canal on the left abutment at the Dam's mid-height. This tunnel outlet can release up to 40 cfs.

The Dam backs water into areas previously occupied by two railroads, a U. S. highway, and three county roads. These facilities had been principal arteries for access to and from the Oroville community and the State in general. In order to provide service to all who used these facilities, relocations were designed and constructed prior to the filling of Oroville Reservoir.

Table 8 shows the statistical summary of Oroville Dam and Lake Oroville, Figure 55 is a location map, Figure 56 is an aerial view, and Figure 57 contains the area-capacity curves.

Oroville Dam is situated in the foothills of the Sierra Nevada above the Central Valley and is 1 mile downstream of the junction of the Feather River's major tributaries. The Dam is located 5 miles east of the City of Oroville (Figure 55) and is approximately 85 miles north of Sacramento. Nearest major roads are State Highway 70, adjacent to the west city limits of Oroville, and State Highway 99, six miles farther west.

TABLE 8. Statistical Summary of Oroville Dam and Lake Oroville

OROVILLE DAM	
Type: Zoned earthfill	
Crest elevation	922 feet
Crest width	50.6 feet
Crest length	6,920 feet
Streambed elevation at dam axis	180 feet
Lowest foundation elevation	152 feet
Structural height above foundation	770 feet
Embankment volume	80,000,000 cubic yards
Freeboard above spillway crest	21 feet
Freeboard, maximum operating surface	22 feet
Freeboard, maximum probable flood	5 feet

LAKE OROVILLE	
Maximum operating storage	3,537,577 acre-feet
Storage at flood control pool	2,778,000 acre-feet
Minimum operating storage	852,000 acre-feet
Dead pool storage	29,638 acre-feet
Maximum operating surface elevation	900 feet
Surface elevation of flood control pool	848.5 feet
Minimum operating surface elevation	640 feet
Dead pool surface elevation	340 feet
Shoreline, maximum operating elevation	167 miles
Surface area, maximum operating elevation	15,805 acres
Surface area, minimum operating elevation	5,838 acres
Drainage area	3,611 square miles
Average annual runoff	3,500,000 acre-feet

SPILLWAY	
Emergency: Ungated ogee (left 930 feet) and broad crest (right 800 feet), discharge on hillside above river	
Crest elevation	901 feet
Crest length	1,730 feet
Flood control: Gated broad crest with lined channel and dispersion chute blocks—Eight submerged radial gates 17 feet - 7 inches wide by 33 feet - 6 inches high	
Sill elevation	813.6 feet
Total sill width	140.7 feet
Combined spillways: All 8 gates open	
Maximum probable flood inflow	720,000 cubic feet per second
Peak routed outflow	624,000 cubic feet per second
Maximum surface elevation	917 feet
Flood control: All 8 gates open	
Standard project flood inflow	440,000 cubic feet per second
Peak routed outflow	150,000 cubic feet per second
Maximum surface elevation	900 feet

POWERPLANT INTAKE	
Edward Hyatt Powerplant: Multilevel, twin, sloping intakes each with roller gate shutoff and 13 removable shutters	
Maximum generating release	16,900 cubic feet per second
Pumping capacity	5,610 cubic feet per second

OUTLET WORKS	
River outlet: Two 72-inch-diameter conduits through tunnel plug controlled by two 54-inch fixed-cone dispersion valves each of which is guarded by a 72-inch spherical valve—intake, diversion tunnel intake—auxiliary inlet, uncontrolled vertical shaft—discharge into tailrace tunnel	
Capacity	5,400 cubic feet per second
Palermo outlet tunnel: A 72-inch-diameter lined tunnel with a valve chamber and energy dissipator immediately downstream of grout curtain—control, 12-inch fixed-cone dispersion valve	
Capacity	40 cubic feet per second

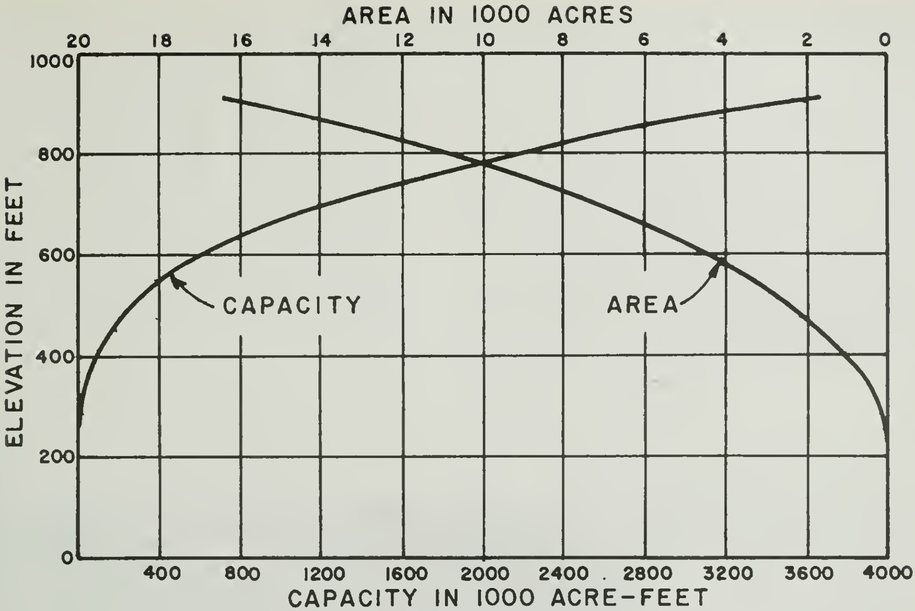

Figure 57. Area-Capacity Curves

Purpose

Oroville Dam and its appurtenances comprise a multipurpose project encompassing water conservation, power generation, flood control, recreation, and fish and wildlife enhancement. The Lake stores winter and spring runoff which is released into the Feather River as necessary, to supply project needs. A pumped-storage capability permits maximization of the value of power produced by these releases. The dependable capacity of Edward Hyatt Powerplant and downstream Thermalito Powerplant combined is 725 megawatts, with an energy output of over two billion kilowatt-hours per annum. The Federal Government shared in the cost of the Dam, which provides 750,000 acre-feet of flood control storage. The 15,805-acre surface of the Lake with a 167-mile shoreline provides water-oriented recreational opportunities.

Chronology and Alternative Dam Studies

Various plans for development of the Feather River have been studied by state and federal agencies beginning in the 1920s. The River, a major tributary to the Sacramento River in California's great Central Valley, has a particularly erratic streamflow record. In some years, it has been practically dry in the late summer and fall and, in other years, has produced devastating floods. A major project was needed to control the floods and to assure a firm water supply for the valley's irrigation needs. Later planning included exporting some of the conserved water to the State's south coastal area around Los Angeles. Since, historically, the flow of the Feather River has been well below normal for as many as four consecutive years, the storage had to be great enough to provide carryover storage from wet years to dry years.

In the late 1940s, the State compared development at the Oroville site with developments on the North Fork at Big Bend and on the South Fork at Bidwell Bar. The conclusion, quoting the August 1949 report (see Bibliography), was that "major storage capacity can most feasibly and economically be provided at the Oroville site".

Work then was concentrated on feasibility studies for the State Water Project (then known as the Feather River Project), which was authorized by the State Legislature in 1951. A concrete gravity dam similar to Shasta Dam was assumed at Oroville for these studies.

In 1956, the State Legislature authorized the preparation of final designs, plans, and specifications for Oroville Dam. First, the type of dam to be constructed at the site had to be selected. Initially, gravity, multiple-arch, straight-buttress, and arch-buttress concrete dams were studied (Figure 58).

One of these designs was of the concrete-buttress

Figure 58. Model of Multiple-Arch Concrete Dam

type consisting of a series of massive head buttresses 60 feet thick and spaced 120 feet center to center across Oroville Canyon. Later, to eliminate the extremely high buttresses that would have been required in the deeper part of the Canyon, an arch-buttress alternative was developed. The central deepest portion of the Canyon was to be spanned by an inclined arch abutting against a massive buttress on either side. The remainder of the dam would have consisted of a massive head buttress similar to that proposed for the straight-buttress dam. Preliminary design studies of this hybrid dam showed it to be economically and engineeringly attractive. Detailed design studies, which included a structural model, were undertaken. Designs for multiple-arch dams were suggested by the foremost concrete dam designers in the world at the time.

The Oroville Dam Consulting Board (discussed in Appendix A) advised the Department of Water Resources on these studies and on the final design and construction of the Dam and appurtenant structures.

Concurrent with these design studies, extensive damsite geologic and construction materials investigations were being undertaken. The materials investigations were centered in the vast fields of tailings located 10 to 15 miles downstream of the site that had been produced by dredgers working over the flood plain of the Feather River for gold. This gravelly cobbly material originally was explored for a source of concrete aggregate and later proved to be an ideal material for pervious shells for the earthfill dam. The dredger tailings, consisting of washed sands and gravels, varied in depth from 15 to 50 feet. Normal dredger operation stacked the gravel and cobbly material on top of the sand. The sand deposit was not usable for concrete aggregate but could be used to blend with the tailings to form transition zones for an earth or rockfill dam.

Preliminary analyses showed that an embankment dam utilizing the dredger tailings could be constructed at approximately the same cost as the most competitive concrete dam. Further exploration located a source of impervious core material near the tailings. An intensive study was undertaken to determine the feasibility of hauling the borrow materials an average of 11 miles to the dam. The use of a conveyor, a truck, and a rail haul was investigated. The high capital investment made a conveyor uneconomical. The cost of a high-speed road required for truck haul and the climatic conditions made trucks impractical. Daytime temperatures in the Oroville area reach 100 degrees Fahrenheit for more than 30 days each summer. At those temperatures, difficult service conditions would have been created for tires then available. In the winter, the area is frequently shrouded by fog which would have forced the trucks to reduce their speeds. The Department, in its economic studies, made the choice of rail haul based on the use of the old Western Pacific Railroad tracks for nearly one-half the haul distance. This was possible since the relocation of the railroad would be completed by the time construction of the dam commenced. This selection later was verified by the successful contractor, and all the other bidders adopted a railroad as the most economical means of transporting the materials to the damsite.

These design studies were completed and results gathered and reviewed in the fall of 1958. Preliminary costs estimated for the earth dam and the arch-buttress concrete dams were nearly the same when the power facilities were included, but continuing damsite exploration indicated that extensive foundation treatment would be required for a concrete dam. The conclusion was that an embankment dam should be constructed.

The next phase of design was to develop the section of the embankment dam. Included in these studies were preliminary designs of vertical, inclined, and sloping thin-core dams. The inclined core was selected as the section for final design.

Construction Schedule

First construction in the Oroville area was on U. S. Highway 40A (now State Highway 70) and Western Pacific Railroad relocations in 1957. The State Legislature annually appropriated limited funds to continue this work until 1960, when the California voters approved the bond issue (the "Burns-Porter Act") to construct the State Water Project.

Work at the Dam site started in the summer of 1961 with the award of a contract for constructing the first of the two diversion tunnels. The contract for construction of the Dam, including the second diversion tunnel, was awarded in the summer of 1962. The spillway, reservoir clearing, saddle dams, and other work were accomplished by separate contracts awarded one to four years after the main dam contract.

The embankment was topped out in October 1967. Storage in the reservoir commenced the following month with closure of Diversion Tunnel No. 1. The spillway was completed in early 1968 as was all other work, except the powerplant and cleanup contracts, which were completed within the following two years.

Regional Geology and Seismicity

Oroville Dam lies in the foothills on the western slope of the Sierra Nevada, a westerly tilted fault block with a core of granitic rock. A series of tightly folded, steeply dipping, metamorphic rocks overlies the granite core along its western and northwestern flanks.

Geologic formations in the Oroville area are grouped into an older, steeply dipping, "Bedrock Series" containing mostly dense, hard, metamorphosed volcanic and sedimentary rocks and a younger, overlying, "Superjacent Series" of nearly flat-lying, nondeformed, sedimentary rocks and volcanic flows.

During recorded history, the Oroville area largely has been unaffected by earthquakes. The most significant earthquake in the northern Sierra Nevada before 1934, when location of earthquakes by seismographs became routine, was the temblor of 1875 which probably occurred on the Mohawk Valley fault near Quincy, about 40 miles east of Oroville. Fissures 2 feet wide and rejuvenated hot springs along the fault trace were reported. A maximum intensity was not assigned for the epicenter because the area was uninhabited and consequently no damage reported. The intensity assigned the Oroville area was V (Modified Mercalli).

No known active faults are within 20 miles of Oroville Dam. The active Mohawk Valley fault is about 40 miles east of Oroville, and the San Andreas fault is about 130 miles west.

The foothill fault system parallels the Sacramento Valley for about 160 miles. An unnamed branch of this system is about 27 miles east of the project area. This fault system generally is regarded as inactive. Evidence suggests that the most recent movement occurred over 70,000,000 years ago.

Design

Dam

Description. Oroville Dam is a zoned earthfill structure with a maximum height of 770 feet above its lowest streambed excavation. The dam embankment crest, at elevation 922 feet, is 50.6 feet wide and approximately 5,600 feet long from the gated spillway to the left abutment. The embankment plan is shown on Figure 59. Selected sections and the profile of the Dam are shown on Figure 60. The materials used in each zone and the compaction methods were:

Zones 1, 1A, and 1B—Impervious core from the deposit adjacent to the pervious borrow areas consisting of a well-graded mixture of clays, silts, sands, gravels, and cobbles to 3-inch maximum size. Compaction was in 10-inch lifts by 100-ton pneumatic rollers.

Zones 2 and 2A—Transition zones consisting of a well-graded mixture of silts, sands, gravels, cobbles, and boulders to 15-inch maximum size (6% limit on minus No. 200 sieve material). Compaction was in 15-inch lifts by smooth-drum vibratory rollers.

Zone 3—Shell zone of predominantly sands, gravels, cobbles, and boulders to 24-inch maximum size; up to 25% minus No. 4 U.S. Standard sieve sizes permitted. Compaction was in 24-inch lifts by smooth-drum vibratory rollers.

Zone 4—Impervious core from selected abutment stripping contains between 15 and 45% passing No. 200 U.S. Standard sieve with 8-inch maximum size. Compaction was in 10-inch lifts by a 100-ton pneumatic roller.

Zone 4A—Buffer zone designed to compress, with same grading requirements as Zone 4 but less stringent compaction requirements. Compaction was in 15-inch lifts by a smooth-drum vibratory roller.

Zones 5A and 5B—Drainage zones consisting of gravels, cobbles, and boulders with maximum of 12% minus No. 4 sieve size permitted. Compaction was in 24-inch lifts by a smooth-drum vibratory roller.

The only processing of embankment materials required, other than a minor amount of moisture conditioning, was screening of the core material to remove plus 3-inch rock.

Elevation 900 feet was selected as the normal water surface while the concrete dams were being considered. Factors influencing the selection were: a 3.5-million-acre-foot reservoir was needed, the ridge near Parish Camp Saddle Dam is narrow, and this saddle and the ones at the spillway site and Bidwell Canyon were slightly below elevation 900 feet. In addition to these physical factors, the height of the Dam was unprecedented.

Configuration and Height. Because the shear strength of the core material is lower than the shear strength of the material in other major zones of the Dam, the thickness and relative position of the core played an important role in selection of the section for the Dam. Although the most economical design was determined to be a section with a thin vertical core, the consolidation characteristics of the embankment materials dictated the selection of an inclined core section. The core material is more than twice as compressible as the shell materials under the loads in a dam the height of Oroville. With a vertical core, there was the chance that resultant differential settlement at the shell-core contact could have caused "arching" and horizontal cracks through the vertical core. With an inclined core, these effects would be less likely to occur and the inclined core section did not affect materially the cost of the Dam. Settlement and stress measurements made in the embankment during construction verified the wisdom of the selection in that only a harmless degree of arching occurred even in the sloping core.

The curve in the axis of the Dam (Figure 59) improved the appearance and gained some possible arching to keep the core in compression.

A 400-foot-high cofferdam (Figure 61), which was incorporated into the final embankment, served to divert the Feather River and protect the Dam site during construction. This solution was the result of studying several sizes of diversion tunnels and embankment-placing schemes in which the standard project flood with a peak flow of 440,000 cfs was routed past the Dam. The plan for protecting the Dam during the first flood season by making allowance for the fill to be overtopped is discussed later in this chapter in describing the core block. During the second and third years, considerable storage was required behind the Dam to route the flow through the tunnels. It was doubtful if the impervious material in the main core could be placed at a rate great enough to provide the required dam height, particularly in the first full year of embankment placement. The high-cofferdam scheme reduced the required embankment placement rate during this shakedown year to little more than one-half the average rate required to complete the remainder of the Dam. When the need for the cofferdam

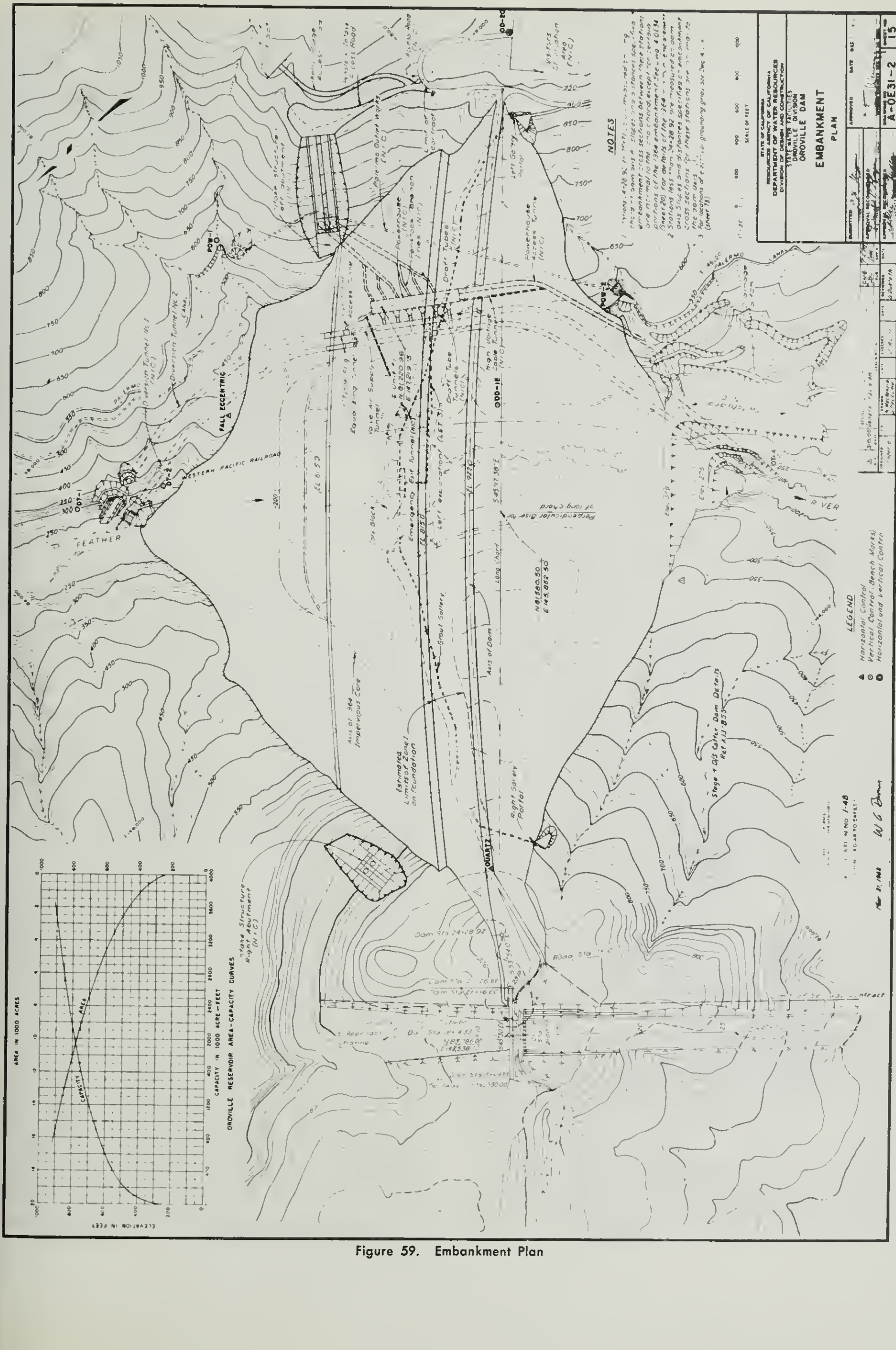

Figure 59. Embankment Plan

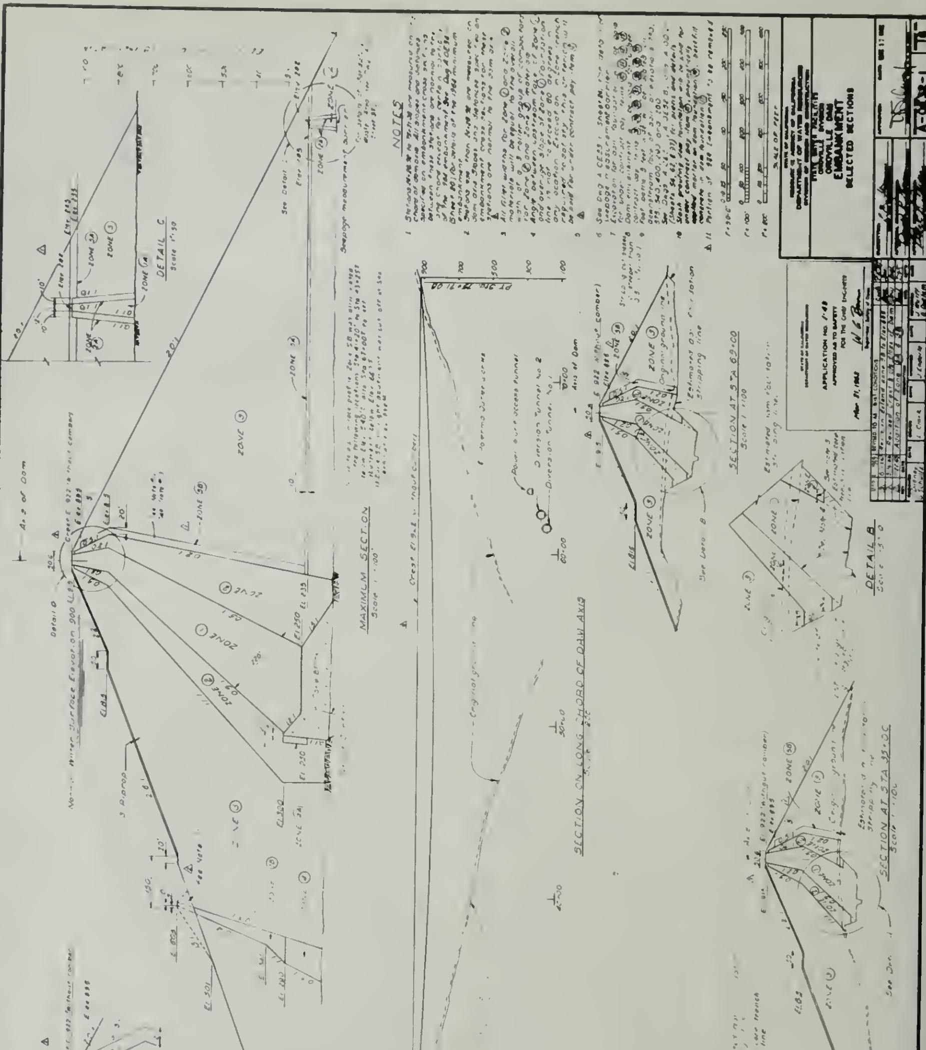

Figure 60. Embankment—Selected Sections and Profile

Figure 61. 1964 Cofferdam

had passed, the contractor elected to remove the bulge it caused in the upstream slope and to use the gravels in the Dam.

Construction Materials. Availability of suitable earth and rock materials was one of the key elements in the economy of an embankment dam at the Oroville site. Materials in and around the dredge tailing fields south and west of the City of Oroville (Figure 62) provided an abundant supply of all types of earth and rock materials for the major embankment zones.

The impervious borrow area was selected in favor of other areas explored because its proximity to the pervious borrow area would allow common transportation facilities and because a more uniform material could be obtained. The specifications required excavation by shovels on a vertical face to blend the material which was coarser at depth. The depth of the excavation was selected to produce the desired gradation.

Design exploration work in the Oroville dredger tailings was accomplished in two basic phases (see Bibliography). The first phase was accomplished in 1956–57 at the time when a concrete dam was proposed for the Oroville site.

The primary problem in exploration of the dredger tailings was that of obtaining representative samples from the loose gravels and sands occurring above and below the static water level. The coarseness of the material presented problems in sampling. Ordinarily, water level in the dredger tailings was located approximately at the interface between coarse gravels and sand. Large excavations by dragline were considered; however, the problem of obtaining representative samples, particularly beneath the water table, forced consideration of other means of exploration. Drilling of this kind of deposit appeared to be infeasible with standard types of equipment. However, a "hole excavator" manufactured and operated by Par-X Placer Equipment Company of Benicia, California, was located. This unique piece of equipment appeared to be suitable for excavating holes through these loose coarse gravels and into the sand beneath the water table. The excavator consisted of a carrier beam and attached clamshell bucket mounted on, and operated from, the back of a 2½-ton truck. A hydraulically controlled winch enabled the operator to raise the boom to the vertical from its horizontal transporting position, to raise or lower the boom, to crowd the bucket, and to open and close the bucket. The longest boom available was 41 feet, which limited the depth of hole using the clamshell bucket to 37 feet. The bucket could be locked in 90-degree positions or rotated freely, much as a percussion bit. The bucket was used to penetrate gravels and intervals of sand above the water table. Sand beneath the water table was excavated with a piston-operated cylindrical sucker which drew material through a hinged door on the bottom. The average operating capacity of the sucker was approximately 2 cubic feet. After this piece of equipment was removed from the hole, the sand was emptied into a riffled sluice box, which was used to prevent excessive washing and loss of fines.

A major problem with excavation of any type of hole in a tailing deposit is run-in or caving of the hole. To prevent these conditions, telescoping, continuous, and butt-jointed casings were used. In most cases, all three types of casing were used in each hole. Telescoping casing, consisting of six sections each 46 inches high and ranging from 62 to 50 inches in diameter, was used as a starter for all holes. A 12-foot length of 38-inch-diameter casing was used inside the telescoping casing to extend the depth of hole. While excavating in the gravelly materials, the casing had to be driven within a few inches of bottom in order to prevent excessive run-in and contamination of samples. Sucker casing was butt-jointed and fit inside the 38-inch-diameter casing. The inside diameter of the sucker casing was 17½ inches, and this casing was driven ahead of the hole when excavating sand beneath the water table.

Exploration procedures and equipment went through various periods of evolution, and attempts were made to analyze the results with regard to sufficiency and accuracy of data, samples obtained, and cost of conducting such types of exploration. During 1956–57, 70 holes on a 1,000-foot grid spacing were drilled through the dredge tailing deposit in an area which was selected because of location and apparent suitability. For practical considerations, the samples were split and run through screens at the test pit. Initially, only the minus ¾-inch material was sent to the laboratory.

Shortly after the investigation for concrete aggregates was completed, it was determined that Oroville Dam would be a zoned embankment type. In 1959, the second phase of the overall tailings exploration pro-

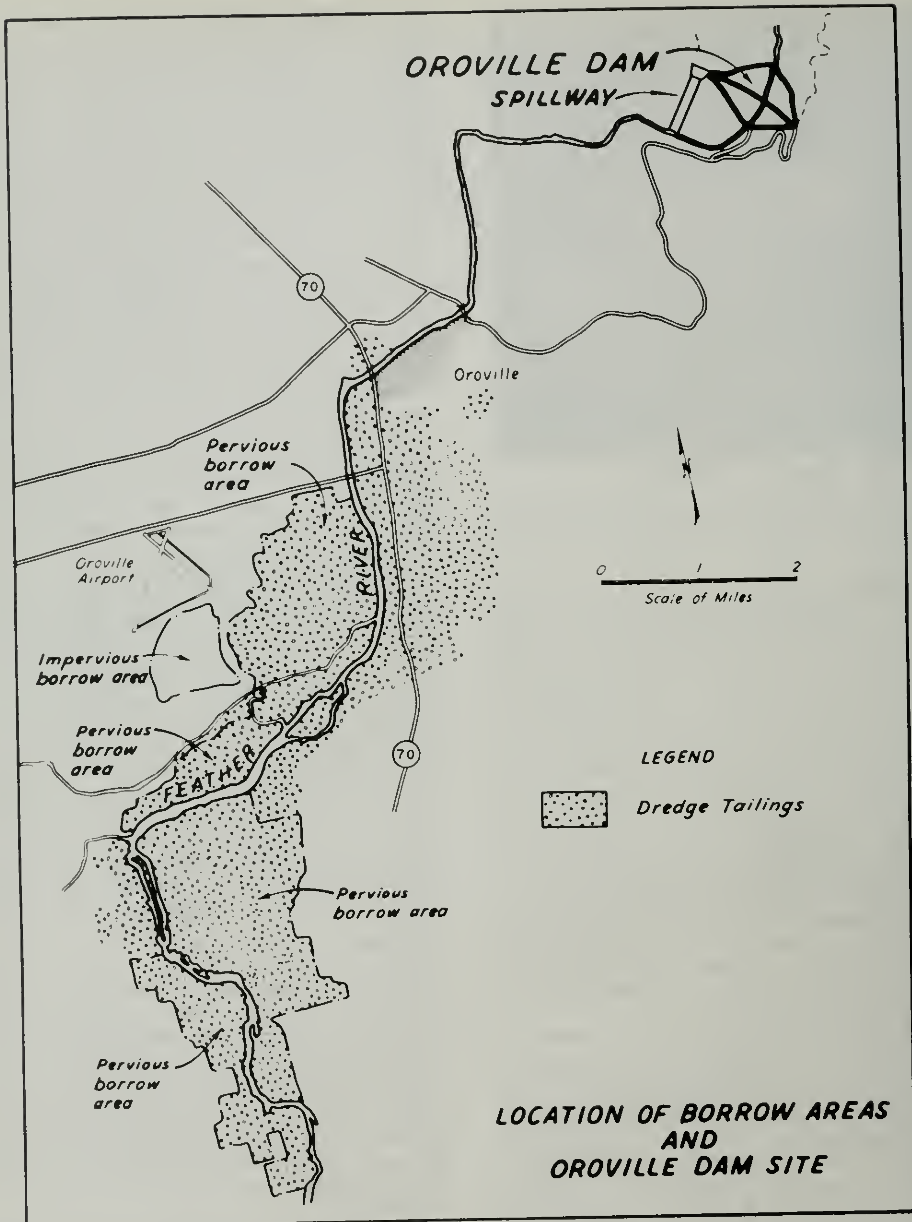

Figure 62. Location of Borrow Areas and Oroville Dam Site

gram was undertaken to verify the existence of the required additional volume of material suitable for the outer zones of the embankment-type dam and to determine its physical properties. Since quality and basic grading were known in at least one area as a result of concrete aggregate exploration, quantity became the prime consideration. Also, it was desirable and necessary to delineate broad areas of tailings with distinguishable and characteristic grading. The first step was to evaluate aerial photographs and block out similar-appearing areas of the tailings so as to plan an exploration program. Detailed gradings, as obtained during concrete aggregate investigations, were not required but immediate results were needed to provide necessary design data. It was decided that the hole excavator, used during concrete aggregate exploration, would be too slow; therefore, exploration by dragline pits and bulldozer trenches was selected. A program was planned in which approximately four pits would be excavated in each section of the dredger tailing field. A 1½-yard dragline and a large bulldozer were used, and the completed program consisted of the excavation of 71 dragline pits and 129 bulldozer trenches.

Procedure on the pit exploration started with the bulldozer leveling a site between two linear ridges of dredge tailings (Figure 63). By pushing the material from the ridge into the adjoining valleys at each side, an attempt was made to obtain an average thickness of gravel. This procedure reduced the depth of dragline excavation and gave relative assurance that the sand table would be reached. After leveling by the bulldozer, the dragline was moved in and test pits were excavated through the gravel and several feet into the sand so that an uncontaminated sand sample could be collected and an accurate elevation could be obtained

Figure 63. Dredge Tailings

on the sand table. After the pit was excavated, one large representative sample of the entire thickness of gravel penetrated was taken from the wall of the pit with the dragline bucket. This sample was placed in a separate pile adjacent to the pit. The sample pile contained approximately 10 cubic yards of gravel and, from this pile, several 40-gallon drum samples were taken with a backhoe. Drum samples were transported to the laboratory for analysis and testing.

It was intended to use the dozer to obtain supplementary information on gravel thickness, as well as to prepare test sites; however, cuts through the gravels proved impractical due to raveling of the material in cut slopes. Instead, most of the 129 trenches were used as an aid to a visual classification of the coarse tailings. In the visual classifications, the amount of No. 200 to No. 4 sieve size material and the amount of minus No. 200 sizes were estimated. Generalized classifications were used which consisted of clean, sandy, silty, and clayey gravels. These classifications were delineated on a base map and the volumes were computed for each classification, based upon the thickness of gravel determined during pit excavation. It was estimated from the exploration that a total volume of over 140 million cubic yards of coarse tailings were available. Test pits of the type excavated by dragline averaged approximately 20 feet deep.

Exploration was started in the impervious borrow area with a wide spacing of large-diameter holes. The Par-X excavator was tried; however, this method of excavating holes was abandoned for a faster method. Thirty-inch-diameter bucket augers were employed, and it was soon found that because of the compactness and gravelly nature of the borrow area, it was necessary to use heavy-duty drill rigs with extra-heavy kelleys. Also, it was necessary to use a variety of bucket types in order to continue holes when drilling became difficult.

To expedite the exploration and testing programs, it was desirable once again to do the coarse grading in the field and to transport only the finer materials to the laboratory for further testing. The entire borrow area was drilled with 56 holes on a 750-foot grid spacing, and materials were sampled and graded from each hole in the first phase. Holes averaged 42 feet in depth and were drilled to the static water level or to volcanic sediment underlying the borrow area. The deepest hole was 60 feet, and it was not necessary to case holes in this material. By carefully selecting materials from each 5-foot interval, it was not necessary to use the splitter or to process the entire sample being drilled. The 750-foot hole spacing was later split to 375 feet, requiring an additional 83 holes.

Laboratory Testing. The large materials made it impractical to attempt detailed laboratory testing on prototype material. However, it was considered desirable (1) to include the effect of maximum particle size on embankment design parameters to the extent practical from the standpoint of laboratory equipment design, and (2) to carry the testing into the range of loads expected within the embankment mass of the prototype structure. To this end a special, high-capacity, compression apparatus was designed and constructed at the Department's Bryte Laboratory with the capability of handling, in a cylindrical testing chamber, specimens up to 27 inches in diameter with varying heights up to 60 inches. This apparatus is capable of applying axial loads of up to 650 pounds per square inch (psi) through a hydraulic piston. In these test chambers, it was possible to test samples containing up to 6-inch rock sizes for compression, consolidation, and permeability characteristics under maximum estimated prototype load. As a secondary result, it was also possible to determine particle degradation under compressive loads representative of maximum embankment height.

For establishing the efficiency of the compactive processes used on the shell and transition materials, a special, laboratory, vibratory, density test was developed as a standard against which to compare the maximum density obtained by compaction equipment in the field. This test utilized a 27-inch by 30-inch cylinder and external vibration.

Maximum density for the impervious samples was determined in conventional laboratory compaction tests but modified to permit using various maximum particle sizes (¾-inch, 1½ inches, 3 inches, 4 inches, and 6 inches). A compactive effort of 20,000 foot-pounds per cubic foot was used.

An unusual undertaking of the laboratory investigations was the performance of triaxial shear tests on samples of materials for the three major zones, using samples up to 12 inches in diameter and lateral pressures up to 650 psi (see Bibliography). The equipment utilized had been constructed by the U. S. Army Corps of Engineers and was operating in its South Pacific Division Laboratory at Sausalito, California. In these studies, samples of materials containing particle sizes up to a maximum of 3 inches were tested using confining pressures of up to 125 psi. Samples containing particle sizes up to a maximum of 1½ inches were tested using confining pressures up to 550 psi for gravels and 650 psi for clayey gravels. Further testing of the embankment materials was undertaken after the Dam was under construction. These tests were run on even larger samples at the test facility constructed by the Department at the Richmond Field Station of the University of California (see Bibliography). The later tests showed the earlier results to be 1 to 2 degrees conservative.

Test Fills. It was desirable to verify the practicality of achieving laboratory densities and related physical properties in the field using prototype materials excavated, transported, and compacted in a conventional manner by commercially available modern construction equipment. To this end, an extensive test-fill program was conducted. Various compactors (sheepsfoot, pneumatic-tired, segmented-pad, pneumatic-vibratory, smooth-drum vibrator, crawler-tractor, and hydraulic-monitor) and various combinations of layer thickness, roller coverages, and moisture content ranges were tried on 14 different test fills. Over 150 different processes of placing and compacting material were tested. The fills were built in the proposed borrow areas using pit-run materials representative of the three major zones of the embankment.

Because of the large maximum particle sizes in the test fill materials, it was necessary to scale up the conventional field density determinations and, to accomplish this, the Department of Water Resources' field density determination method was developed. By this method, field measurements were made on samples weighing over 1,000 pounds taken from density holes to 6 feet in diameter. This test and the special laboratory density tests and equipment discussed earlier in this chapter were used to control construction of the Dam.

Stability Analyses. The slopes of the Dam were determined by use of the modified Swedish Slip Circle, the sliding wedge, and infinite slope methods of analysis. A 0.1g horizontal seismic acceleration was included in the conventional analyses to determine the factor of safety during an earthquake. Total stress or effective stress basis soil strengths were used, depending on the condition being analyzed. Results of these analyses indicate factors of safety substantially in excess of those the Department uses in the design of embankment dams of lesser heights. Properties used and the results of these analyses are summarized on Figure 64.

Because of the configuration of the cross section of Oroville Dam with the sloping core, it was found that the core material had relatively little effect on overall stability of the downstream slope. The predominant influence of the downstream shell, with a high friction angle and cohesion equal to zero, caused the minimum factor of safety for a downstream failure to occur under an "infinite slope"-type analysis. This fact was borne out in an extensive study by the Department and was confirmed in studies by Moran, Proctor, Mueser and Rutledge Consulting Engineers. Since an infinite slope-type failure in a coarse-grained material, such as Oroville Zone 3, would take the form of insignificant shallow raveling and would not involve deep-seated sliding, analysis of the downstream slope by deep wedges and circles was limited to only a few trials to compute the order of magnitude of safety factors which pertained.

The upstream slope stability was analyzed for three different cases of Zone 1 shear strength. Case 1 assumed the "effective stress" core shear strength of $\phi = 34$ degrees, $C = 0$, which did not include the effects of pore pressures due to shearing. Case 2 assumed the "total stress" core shear strength of $\phi = 14$ degrees, $C = 0.3$ tons per square foot, which included the effects of full pore pressures due to shearing. Case

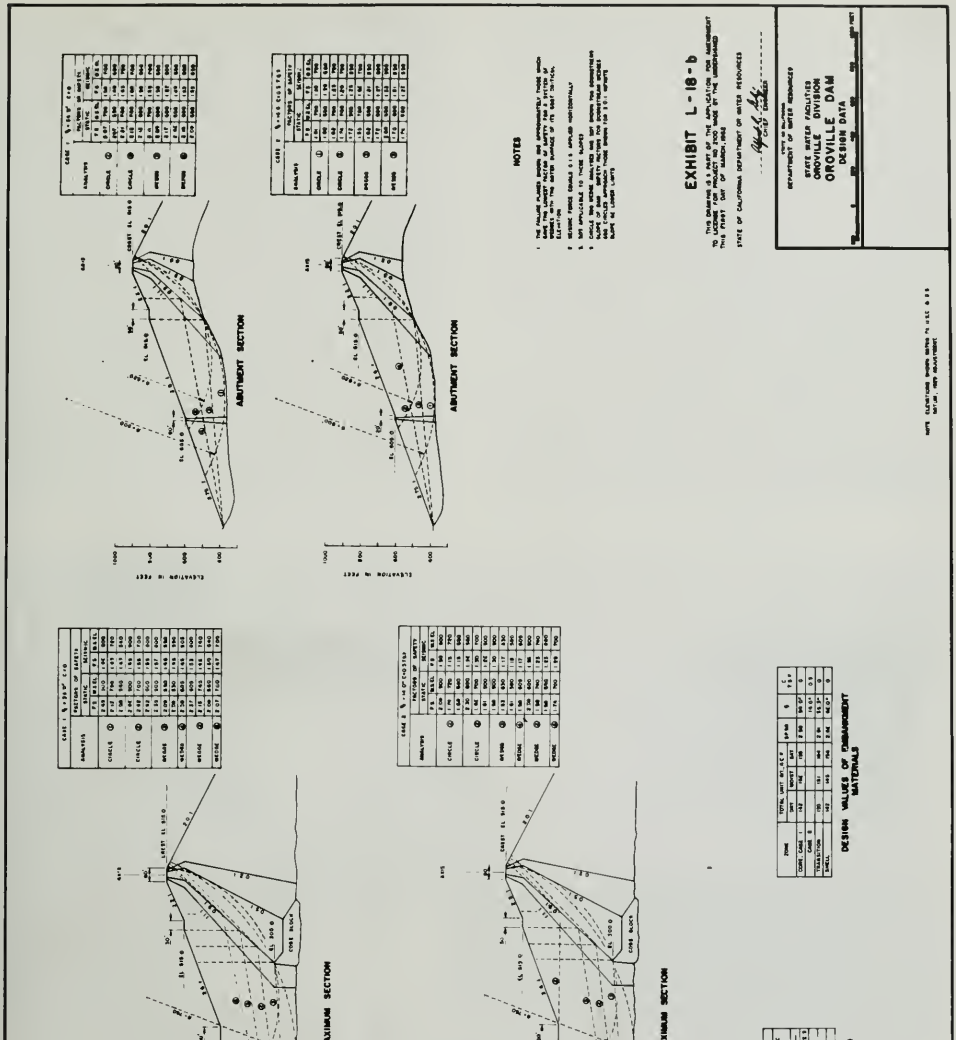

Figure 64. Stability Analysis Summary

3 assumed the unrealistically conservative core shear strength of $\phi = 0$, $C = 0$, or, in effect, a "fluid" core. This latter condition, a "limiting case", was analyzed only as a matter of interest and does not have any real status by comparison with the more realistic analyses of Cases 1 and 2.

In addition to these analyses, two additional conditions were analyzed by Moran, Proctor, Mueser and Rutledge Consulting Engineers. These additional conditions were (1) end of construction, and (2) rapid drawdown from elevation 900 feet to elevation 590 feet. Although the material properties used in these analyses differed slightly from the final values used by the Department, the results of this work demonstrated that the conditions studied were not critical to embankment stability and they were not pursued.

While Oroville Dam is located in a region of low historical seismic activity in California, design to resist earthquakes was a major consideration in development of the final embankment section. Analytical methods available in the early 1960s for earthquake-resistant design of earth embankments were, at best, only an approximation and somewhat empirical in their approach as to what actually happens in an embankment during an earthquake; therefore, it was decided to make further earthquake studies for Oroville Dam. In 1961 and 1962, a series of model tests was conducted for the Department by the Engineering Materials Laboratory of the University of California at Berkeley (see Bibliography) under the supervision of Professor H. B. Seed (Figure 65). These tests were conducted on a 1:400 scale model of the embankment, using horizontal earthquake accelerations up to 0.5g. They yielded some informative qualitative results but left some points unresolved due to the difficulty of scaling the influence of pore-water pressures during dynamic loading. If the prototype dam was subjected to a dynamic force which tended to cause movement along some potential shear surface, the deformation would be accomplished by either dilation or consolidation of the Zone 3 shell material, depending upon the load at any given location in Zone 3. The resulting dilation or consolidation would cause internal pore pressure changes which could not be included in a model study program but which could significantly affect embankment stability. However, since it was known from previous laboratory testing what range of

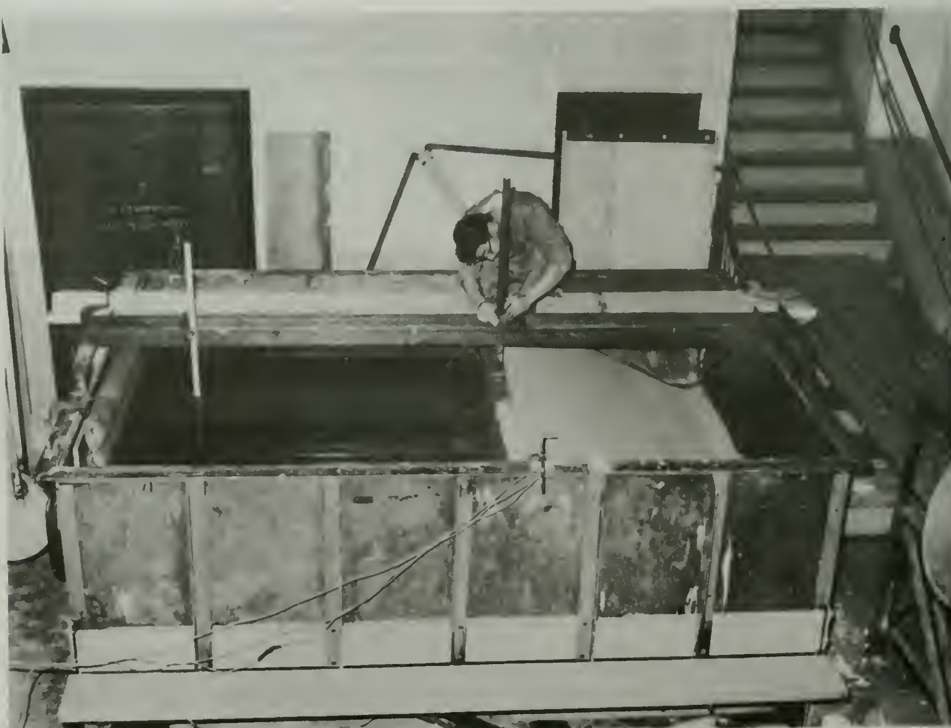

Figure 65. Embankment Model on Shaking Table

loading caused the Zone 3 material to change from dilation to consolidation during shear, it was concluded that additional analytical work could help resolve the uncertainties remaining from the model study program. One additional series of analytical studies was made for the upstream slope of the Dam. For this analysis, the undrained strength of Zone 3-type materials was utilized in conjunction with results of pulsating load test performed on sands at the University of California. Seismic coefficients of up to 0.25g were used. Also, the stability of the upstream slope of the Dam was analyzed for a design earthquake equal to the 1940 El Centro earthquake (peak acceleration 0.25g). Seismograph records of this earthquake were the best strong-motion records available at the time and were considered representative of the most severe ground motions which could be anticipated at the site.

The results of these studies supported the conclusions of earlier work that the dam embankment was designed conservatively with respect to earthquake loading in the light of currently accepted engineering practice.

Other Earthquake Considerations. It is interesting to note that inherent in the conventional embankment design were these additional earthquake-resistant features:

1. Dam embankment is founded directly on bedrock or, in the case of the outer shells, on a minor amount of sand and gravel with a density greater than that of the embankment, thus eliminating any possibility of foundation liquefaction.
2. The embankment zoning scheme provides a wide crest and wide transition zones of well-graded sand and gravel between shells and core. The transition is dense and relatively impervious.
3. The 22 feet of freeboard above normal maximum water level required for maximum floods is more than would normally be required for any possible combination of earthquake-caused reservoir waves and crest slumping.
4. Core material is a dense, plastic, erosion-resistant, extremely impervious material with a wide range of particle sizes. All material was placed at contacts with bedrock of concrete structures at an initial water content from 1 to 3% above optimum to ensure that a plastic zone is in contact with these more rigid elements. The sloping core was placed at or slightly wet of optimum to provide additional protection against potential cracking.

Settlement, Pore-Pressure, and Crest-Camber Studies. Settlement, pore-pressure, and crest-camber studies were performed for Oroville Dam to (1) determine if pore pressures developed during construction would be detrimental to embankment stability, (2) estimate how much of the overall settlement would occur during and after construction, and (3) determine what effects postconstruction settlement would have on the dam crest so that compensating design provisions could be made for crest camber. Several gradations and moisture contents of Zone 1 material were tested and used in the analyses to provide a range of possible results.

These studies indicated that construction pore pressures which could develop would not be significantly detrimental to embankment stability. They would not approach the $\phi = 0$, $C = 0$ Zone 1 condition, so no further construction stability studies were deemed necessary.

Settlement was studied at two important locations within the embankment: one at the maximum height of Zone 1 material, which was on a vertical line 242.5 feet upstream from the dam axis, and one at the dam crest, on a vertical line 10 feet upstream from the dam axis. These studies showed that regardless of grading, not over 2.5 feet of postconstruction dam crest settlement (due to simple consolidation) should be anticipated at the maximum dam section.

The third objective was to design crest camber to compensate for postconstruction settlement using the above results of the previously mentioned studies. Some additional factors are summarized as follows:

Postconstruction consolidation of Zone 1 due to embankment load	0.6 feet
Additional settlement of Zone 1 due to water load	0.3 feet
Additional settlement of Zone 2 and Zone 3 due to full load	0.9 feet
Vertical deformation due to embankment shear strain	1.8 feet
Safety factor	1.4 feet
Total design camber at maximum section	5.0 feet

The actual settlement after seven years is less than 1 foot but is still continuing at a slow rate.

Foundation

Site Geology. Oroville Dam is founded on an unnamed metavolcanic rock formation, one of several units within the "Bedrock Series". The rock is predominantly amphibolite, a basic rock rich in amphibole with abundant veins of calcite, quartz, epidote, asbestos, and pyrite. It is hard, dense, greenish gray to black, fine to coarse grained, and generally massive, although foliated or schistose structures are not uncommon. Average attitude of regional foliation strikes 12 degrees west of north and dips 77 degrees east. Rock at the site is moderately to strongly jointed and is transected by steeply dipping shears and schistose zones. Three prominent joint sets impart a blockiness to the rock, but individual joints are relatively tight. The depth of weathering was found to be substantial and varied greatly from place to place.

Two major shear areas exist beneath the Dam, which are about mid-height on each abutment. Both are steeply dipping and strike normal to the axis of the Dam.

Fresh rock was exposed on the bank of the river channel and in minor outcrops on the abutments. Weathering of rock approached 100 feet in depth in the sheared zones.

Exploration. Subsurface geologic exploration was initiated by the U. S. Army Corps of Engineers in 1944 with the drilling of two core holes, one on each abutment. In 1947, the U. S. Bureau of Reclamation drilled six core holes at the site. Explorations by the Department of Water Resources were begun in 1952 and those required for design were completed in 1959. They included the following:

1. Exploratory Adits—Four 5-foot by 7-foot adits were driven, two on each abutment, together with drifts and cross cuts, totaling 5,251 linear feet.
2. Core Borings—175 borings were drilled varying in size from EX to NX, and in depth up to 200 feet, totaling 18,600 linear feet.
3. Seismic Surveys—Supplementing a "depth of weathering" survey by the U.S. Bureau of Reclamation in 1950 was a 1957 program consisting of 4,350 linear feet of spreads to determine modulus of elasticity and depth of weathering.
4. Special Studies—Detailed evaluation of bedrock properties was made by carrying out the following special investigations:
   a. X-ray diffraction and solubility tests on clay gouge in shear planes
   b. Compression tests on rock cores
   c. Studies of blasting effects in the diversion tunnels
   d. Test grouting
   e. Rock-joint attitude mapping
   f. Measurement of ground water levels and spring flows
   g. Bedrock-stripping methods, eliminating ripping and blasting
   h. In situ rock modulus tests

Additional information was gained from exploration for the underground powerplant and other structures.

Excavation Criteria. The excavation criteria for the various parts of the foundation were:

Concrete Core Block—Sound hard rock consisting of fresh to slightly weathered rock, with unstained to slightly iron-stained fractures.

Embankment Core Trench—Sound hard rock that would be impervious after grouting. Trench slopes 1:1 or flatter downstream and ½:1 upstream. Seams and shear zones excavated to a depth approximately equal to their width. Irregular rock to be removed to permit compaction of the core.

Embankment Shells and Transitions—Weathered rock exhibiting definable rock structure of a strength equal to that of embankment materials placed thereon.

Figure 66. Grouting

Grouting. A single cement grout curtain of 200-foot maximum depth was provided in the foundation beneath the core (Figure 66).

Forty-foot-deep foundation drain holes with a maximum spacing of 80 feet were specified to angle downstream from the grout curtain discharging into the grout gallery.

Slush and shallow blanket grouting were provided to fill surface voids or to improve the strength of fractured areas of the core trench foundation.

Core Block

The 283,000-cubic-yard, lean-concrete, core block is composed of 18 monoliths (Figure 67). The primary structure has a flat top at elevation 250 feet. An upstream parapet rises to elevation 300 feet. Maximum height to the top of the parapet is about 120 feet. The base thickness at maximum section is about 400 feet and the crest length of the parapet is 900 feet.

The purposes of the core block were to (1) eliminate the need to compact impervious core material in the irregularly eroded inner gorge of the Feather River, (2) reduce possibility of transverse embankment settlement cracks, (3) reduce maximum height of the core, and (4) expedite embankment construction. The core block with a 50-foot-high parapet served as a potential overflow structure. This allowed the expedited placement of about 2,000,000 cubic yards of embankment in the upstream half of the stream channel in advance of the critical 1964 embankment construction season, in which the 400-foot-high cofferdam would be incorporated into the Dam to provide flood protection.

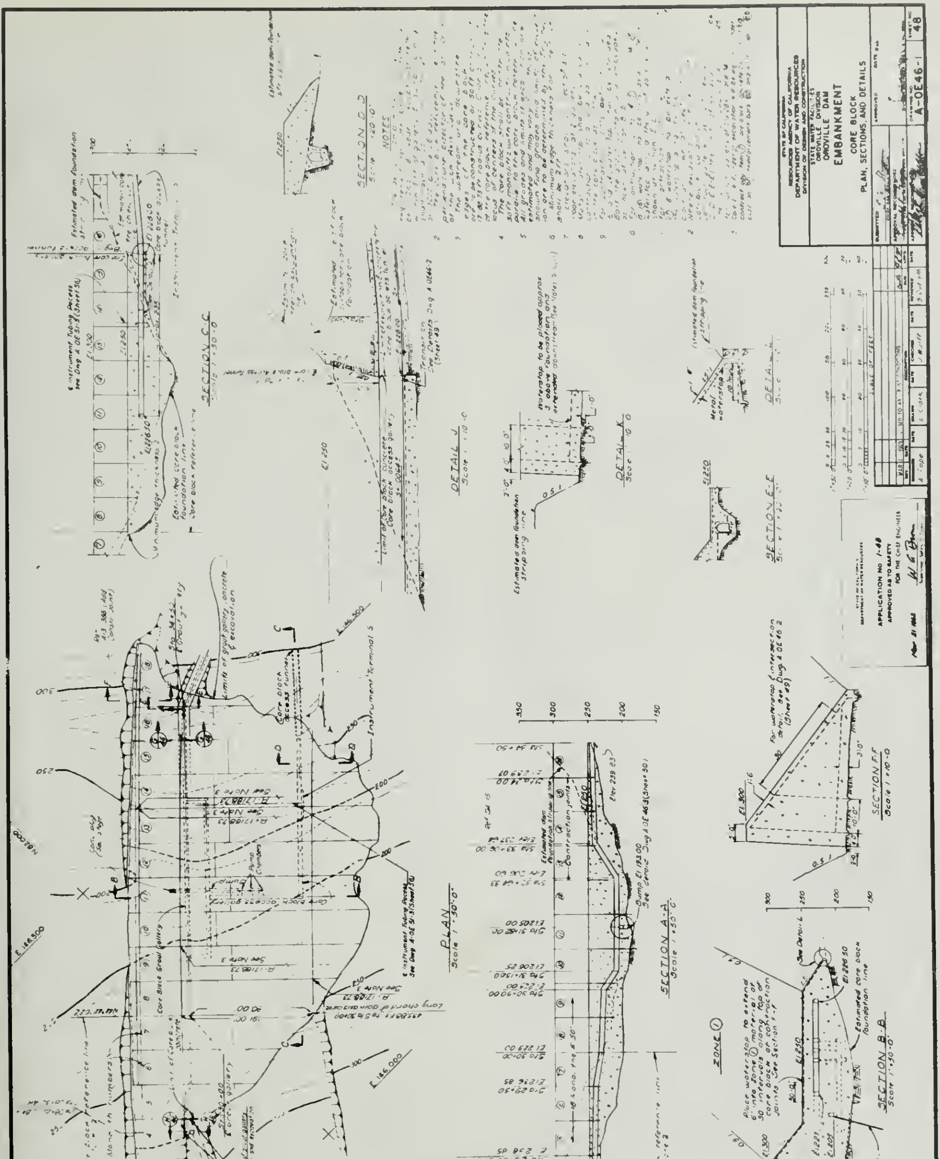

Figure 67. Core Block

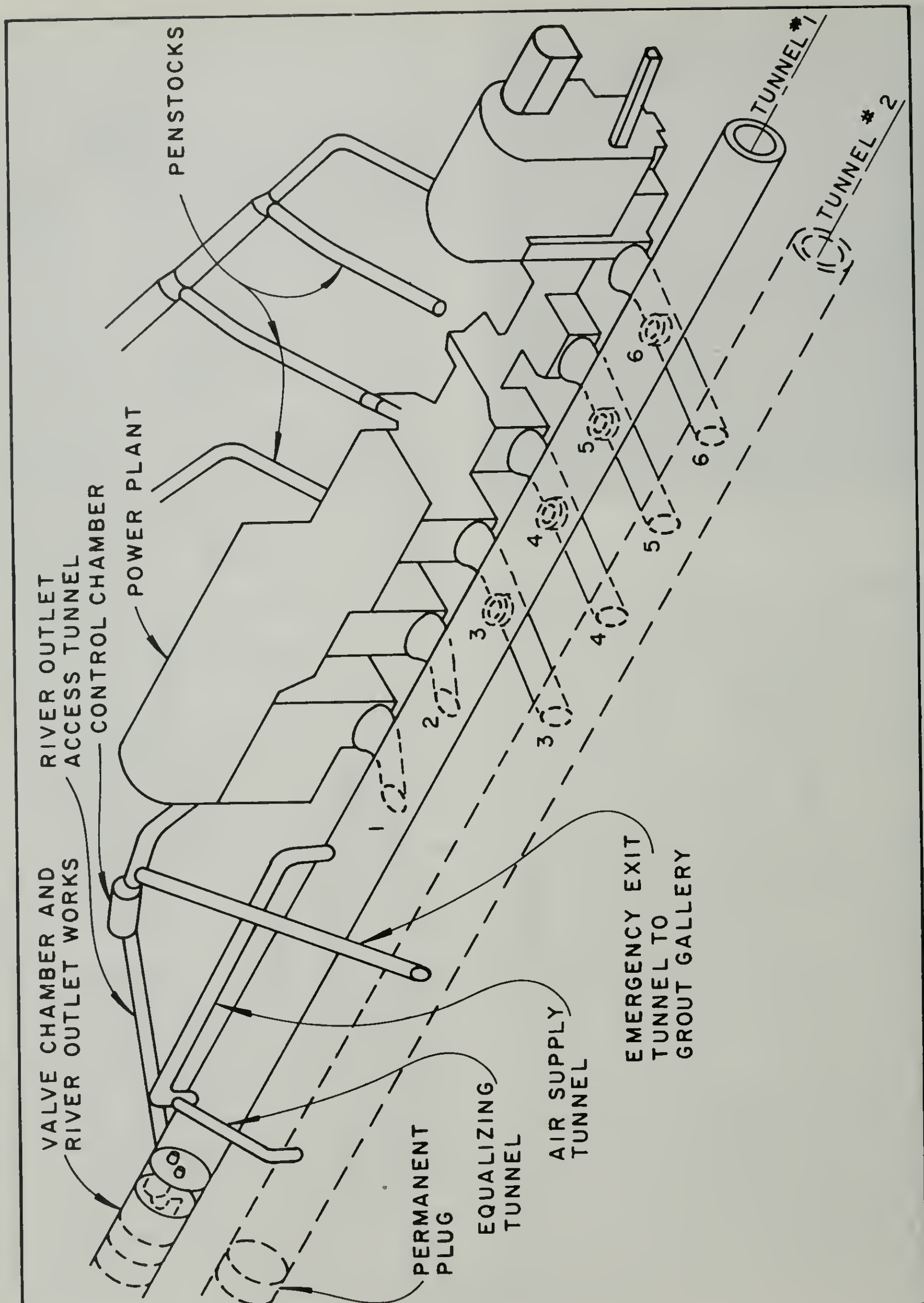

Figure 68. Diversion Tunnels Nos. 1 and 2—Draft-Tube Arrangement

Because of its position (buried under the dam embankment) and its low irregular shape, the core block as a whole did not lend itself to a simple structural analysis. Little information was available at the time of design on stresses that might be imposed on the structure by the zoned embankment. However, the parapet was recognized as the critical component. It was determined that, with certain assumptions made for embankment soil properties, the parapet would tend to separate from the remainder of the block. Under one loading condition, the parapet was the toe of the 1.6:1 downstream toe of the 1964 cofferdam. Compressible Zone 4A was included upstream of the rigid core block to provide some room for base spreading of this slope. The other loading condition had three contributing factors: (1) embankment base spreading caused by the core block being upstream of the crest, (2) tendency of the embankment loads to arch across the core, and (3) projection of the parapet.

Instrumentation was provided in the core block, particularly in the parapet, to observe stress and strains under these loading conditions. The parapet survived the first condition but cracked under the other, as is explained in the section on construction.

The core block contains an extensive gallery system. The grout gallery extends through the upstream portion. A bypass gallery connects it to a gallery that was constructed to provide access to the underground powerplant. Later, this connection was plugged to separate the dam gallery system from the Powerplant in case either flooded. A recess off one of the galleries contains one of the dam structural performance instrumentation terminals. Also included is a drainage system with three pump chambers. Two of the chambers contain 50-horsepower, vertical, turbine pumps with encapsulated windings to protect against water damage. The third contains a 75-horsepower submersible pump. A level control consisting of electrodes is located in a well within the No. 2 pump chamber. The submersible pump motor starter and a duplex sump pump controller are located at the connection of the grout gallery with the powerhouse emergency exit tunnel. All three pumps are connected to a 16-inch discharge pipeline. This pipeline uses the tunnel formerly connected to the Powerplant and galleries as the route to the terminus in the crown of Diversion Tunnel No. 2.

Grout Gallery

The reinforced-concrete, 5-foot by 7-foot, grout gallery is located under the core from the core block up the right abutment to approximate elevation 780 feet and up the left abutment to approximate elevation 820 feet. From these locations, each leg extends to the downstream face of the Dam (Figure 59). Under the core, the gallery generally is in a trench approximately 15 feet deep with a 10-foot bottom width and $\frac{1}{4}$:1 side slopes. In some areas, the trench is imperfect and the concrete projects a few feet into the core. The reaches extending downstream start the same as under the core and end fully projecting into the embankment.

Purposes of the gallery were to (1) reduce the conflict between the grouting and embankment-placing operations, (2) provide an exit for the foundation drain holes, (3) provide the capability to regrout under the higher portions of the embankment if necessary, and (4) provide access to the core block gallery system and the facilities located therein. Based on economic considerations, the grout gallery was terminated before it reached the crest of the Dam. It was determined that the remaining 100- to 150-foot height of dam remaining on the abutments did not justify the cost of the additional length of gallery.

The gallery in the trench is lightly reinforced considering the embankment loads on it, compared to structures designed using approximate methods. It was analyzed at the University of California using one of the first applications of the finite element method. Internal stresses and reactions of such a structure depend to a great degree on the relative stiffness of the concrete and the foundation rock. The finite element method readily handled these variables whereas other techniques available at the time could not. The result was a much more economical structure than the previously planned projecting gallery that was being analyzed by conventional methods or an overreinforced trenched gallery. More than six years of satisfactory performance have proven the validity of the analysis.

Tunnel Systems

A multiplicity of tunnels was required in conjunction with the construction and operation of Oroville Dam and Edward Hyatt Powerplant (Figure 68). Construction of the tunnels was accomplished under four major contracts. The first diversion tunnel and the Palermo outlet tunnel were let under separate contracts in 1961. The contract for the dam embankment included construction of the second diversion tunnel along with connecting portions of the powerplant draft-tube tunnels, a portion of the river outlet access tunnel, and the core block access tunnel. Tunnels directly serving the underground powerplant were constructed under the initial contract for the Powerplant. These tunnels, which include the powerhouse access tunnel, high-voltage-cable tunnel, penstock tunnels and branches, and the remaining portions of the draft tubes, are described in Volume IV of this bulletin.

All tunnels were excavated in the moderately jointed metavolcanic rock of the left abutment. A series of drill holes was used and exploration adits were excavated to locate major shear zones. The location of the powerplant machine hall, the major underground excavation of the complex, was established on the basis of this exploration. Location of the machine hall, in turn, was a major factor controlling the layout of the tunnel systems.

All tunnels in the complex are concrete-lined. The designed concrete-lining section is that thickness between the inside tunnel surface and a designated line called the "A" line. In this thickness, no materials were allowed to remain permanently which would reduce the integrity of the concrete section. No rock was allowed to project into the section, and all timber was required to be removed prior to concrete placement. Structural-steel tunnel support and other metalwork, which did not interfere with the reinforcement steel, was allowed to remain in the section.

Overbreak was anticipated in the excavation of the tunnels and a designated thickness of excavation and concrete lining (9 inches in the case of the diversion tunnels, 6 inches or less for smaller diameter tunnels) was paid for outside of the "A" line. The limit of payment was designated the "B" line.

Diversion-Tailrace Tunnels. The alignment and profiles of the two 35-foot-diameter, 4,400-foot-long, diversion tunnels were selected to (1) bypass the dam construction area, (2) provide for convenient connection to the underground powerplant for use as tailrace tunnels, and (3) keep the total tunnel length to a minimum. A circular section was chosen for the tunnels since they are subjected to high external hydrostatic heads.

Diversion Tunnel No. 1, nearest the Feather River, has an intake invert elevation of 210 feet and an outlet invert elevation of 182 feet. The center reach of the tunnel is depressed to permit connection of the draft-tube tunnels from generating units Nos. 3 through 6. The intake invert of Diversion Tunnel No. 2 is at elevation 230 feet and the outlet invert at elevation 207.5 feet. Draft-tube tunnels from units Nos. 1 and 2 connect directly to this tunnel and surge openings for units Nos. 3 through 6 are provided.

More than 50 years of good streamflow records were available on which to base the design of a sequence to provide flood protection during construction. They indicated the normal start of a 4½-month flood season was November 15. The following covers the highlights of the sequence as it actually occurred:

1. Diversion Tunnel No. 1 was completed in November 1963 allowing embankment to be placed in the stream channel upstream of the core block. Had the riverflow exceeded about 12,000 cfs that winter, this embankment would have been flooded and the core block parapet would have acted as a weir, preventing erosion of the embankment material already placed.
2. Diversion Tunnel No. 2 was completed in November 1964 in time to combine with Diversion Tunnel No. 1 and the 400-foot-high cofferdam, to protect the Dam from actual floods up to the U.S. Army Corps of Engineers' standard project flood (frequency about 1 in 400 years and about twice the flood of record). As discussed later in the section on construction, this combination withstood a new flood of record.
3. When no longer needed for diversion, the tunnels were plugged upstream of the reaches utilized for the tailrace. Plugging of Diversion Tunnel No. 2 took place in August 1966, when the embankment was high enough to protect against the standard project flood with only Diversion Tunnel No. 1 functioning.
4. Tunnel No. 1 was plugged in November 1967 when the embankment was topped out and the spillway essentially was complete. Gates were lowered at the intake to Tunnel No. 1 to dewater the tunnel for plug construction. This act marked the beginning of filling Lake Oroville.

Two criteria were considered for determining the size of the tunnels: (1) the flood conditions just discussed, and (2) the most economical diameter for use as a tailrace for power generation. The two 35-foot-diameter tunnels were required to control the design flood while holding the embankment placement rates to reasonable quantities. The most economical tailrace was estimated to be 31 feet in diameter but would have required a tailrace surge chamber. The two 35-foot tunnels allowed one tunnel to flow free and, by interconnections, eliminated the need for a surge chamber.

As velocities during diversion would reach over 100 feet per second, special effort was required to ensure a high-quality surface for flow. High-strength concrete was used, and a concrete finish was specified which allowed no abrupt irregularities and only minimal gradual irregularities. Tunnel intersections were plugged with the inside surface smooth and monolithic with the tunnel interior. The only irregularities allowed in the diversion flow path were the gate slots in the intake structure for Diversion Tunnel No. 1 and in the exit portal structures for both tunnels. The slots were constructed with offset downstream edges to minimize turbulence and negative pressures in the slots.

Bell mouths were used at each tunnel to reduce inlet head loss and prevent cavitation of the adjacent tunnel lining. Because Diversion Tunnel No. 1 required closure gates, a rectangular bell was formed and transitioned to the circular tunnel. Hydraulic model studies of the tunnels were conducted by the U.S. Bureau of Reclamation at Denver, Colorado (see Bibliography). Both diversion and tailrace modes were tested.

The concrete tunnel lining was designed for the maximum external hydrostatic head expected. Upstream of the plug locations, this was the maximum construction flood pool head minus the hydraulic gradeline for the peak diversion discharge (equivalent to velocity head plus intake head loss). Between the plugs and the dam grout curtain, full hydrostatic pressure of Lake Oroville would come to bear. Drain holes were therefore drilled through the lining into the rock to reduce local pressures, and an external loading of 50% of the lake head was assumed for design. Downstream of the grout curtain, the drain holes were continued and external hydrostatic head was assumed equivalent to the height of overburden. Any load from the disturbed surrounding rock was assumed to be borne by temporary supports and was not added to loading of the concrete lining.

Structural concrete for the lining was specified to attain an ultimate strength of 5,000 psi in one year. The one-year strength was specified since full loading would not occur until the reservoir filled.

Nominal reinforcement was used to minimize cracking due to shrinkage and temperature changes.

All portions of the tunnels are contact-grouted to ensure good contact between tunnel lining and the rock.

Pressure grouting from within the tunnel included a continuation of the grout envelope constructed around the Powerplant and consolidation grouting of the rock immediately surrounding the tunnel plugs. The grout envelope involves radial holes on a regular pattern throughout the reach near the Powerplant, grouted in an interval between 40 and 50 feet from the tunnel lining (Figure 69).

After the 150-foot-long concrete plugs were placed near the center of both tunnels, the knockout plugs for the draft-tube tunnel connections then were removed. The draft-tube connections, 18 and 21 feet in diameter, had metal form panels to partially outline the knockout plugs, facilitating their removal and minimizing the roughness of the opening.

Large ports connect Diversion Tunnel No. 2 with the draft-tube tunnels of units Nos. 3, 4, 5, and 6. This allows Tunnel No. 2 to act as a surge chamber to receive water from or supply it to the draft tubes and Diversion Tunnel No. 1 during load changes. Tunnel No. 2 operates at atmospheric pressure during all powerplant operation modes with the flow normally half filling the tunnel. To provide atmospheric pressure to the upstream end of Tunnel No. 1, an 8-foot-diameter pressure-equalizing tunnel connects the tailrace tunnels directly downstream from the tunnel plugs.

Maximum discharge through Tunnel No. 1 during generation is 12,000 cfs (picking up flow from units Nos. 3 through 6). Discharge through Tunnel No. 2 is 6,000 cfs (units Nos. 1 and 2).

River Outlet. The river outlet is located just downstream of the plug in Diversion Tunnel No. 2. Two 72-inch-diameter steel conduits were cast into the plug. Stream releases are controlled by two 54-inch fixed-cone dispersion valves that are backed up by 72-inch spherical valves. Access to the valves is gained through a tunnel from the Powerplant. The river outlet valves are capable of a combined discharge of 5,400 cfs with full reservoir. A steel liner is used inside the tunnel where discharges impinge and a baffle ring protrudes into the flow path to help still the discharge. An air supply for the valves is provided through a small-diameter tunnel above and parallel to the tailrace tunnel. This tunnel allows air to be drawn by the valves from an area downstream, clear of the valve discharge turbulence.

The 72-inch, spherical, shutoff valves have double seats, are hydraulic cylinder-operated, and are designed to sustain the maximum transient pressure without exceeding the allowable design stresses. The valves are provided with an electrohydraulic activating and control system. Each seat is separately controlled and operated by oil. The operating controls are arranged for the following operations:

1. Normal opening and closing of the valve and upstream and downstream seats from the valve chamber at elevation 233 feet.
2. Emergency remote closing of valve and downstream seat from equipment control chamber at elevation 290 feet.

The 54-inch fixed-cone dispersion valves (Howell-Bunger valves) are actuated by electric motors which can be operated locally or remotely from the equipment control center. Additionally, handwheels were provided for emergency operation of the valves.

The valves were carefully designed by the Department because of the severe service expected and the history of problems with similar valves experienced by other agencies. Special consideration was given to vane design; stiffness was emphasized in order to reduce any tendency toward vibration and possible fatigue failure. Smooth ground surfaces and faired edges of components exposed to high-velocity flow and full-penetration welds throughout were specified. Extensive nondestructive examination was performed on all welds during the construction phase.

Hydraulic model studies of the river outlet were conducted by the U.S. Bureau of Reclamation in Denver, Colorado.

Diversion Tunnel Intake Portal Structures. Intake excavations were shaped to efficiently train streamflows into the tunnel intakes during the diversion mode. Cut slopes in overburden were laid back to safe slopes to preclude slides, and steep cuts in rock were rock-bolted and covered with heavy wire mesh. The plan of the intake structures is shown on Figure 70 and sections on Figures 71 and 72.

The intake structure of Diversion Tunnel No. 1 is a rectangular bell mouth transitioning to the 35-foot-diameter tunnel. A center pier with an elliptical nose is incorporated into the structure to accommodate steel bulkhead gates to dewater the tunnel for plug placement. The gate slots are positioned as far back on the transition as practical to take advantage of arching of the partial circular section and thus its ability to resist external loads. Horizontal prestressed rods were installed in the pier and attached to the gate-slot assembly to carry gate loads forward to the more massive pier section. This assembly consists of 1-inch steel plates lining the slot and a steel grillage connecting the downstream slot faces so as to carry gate loads to the prestressed tendons. Heavy plate was used for the slot liner to maintain true alignment of the bearing surface and slot liner after prestressing loads are applied.

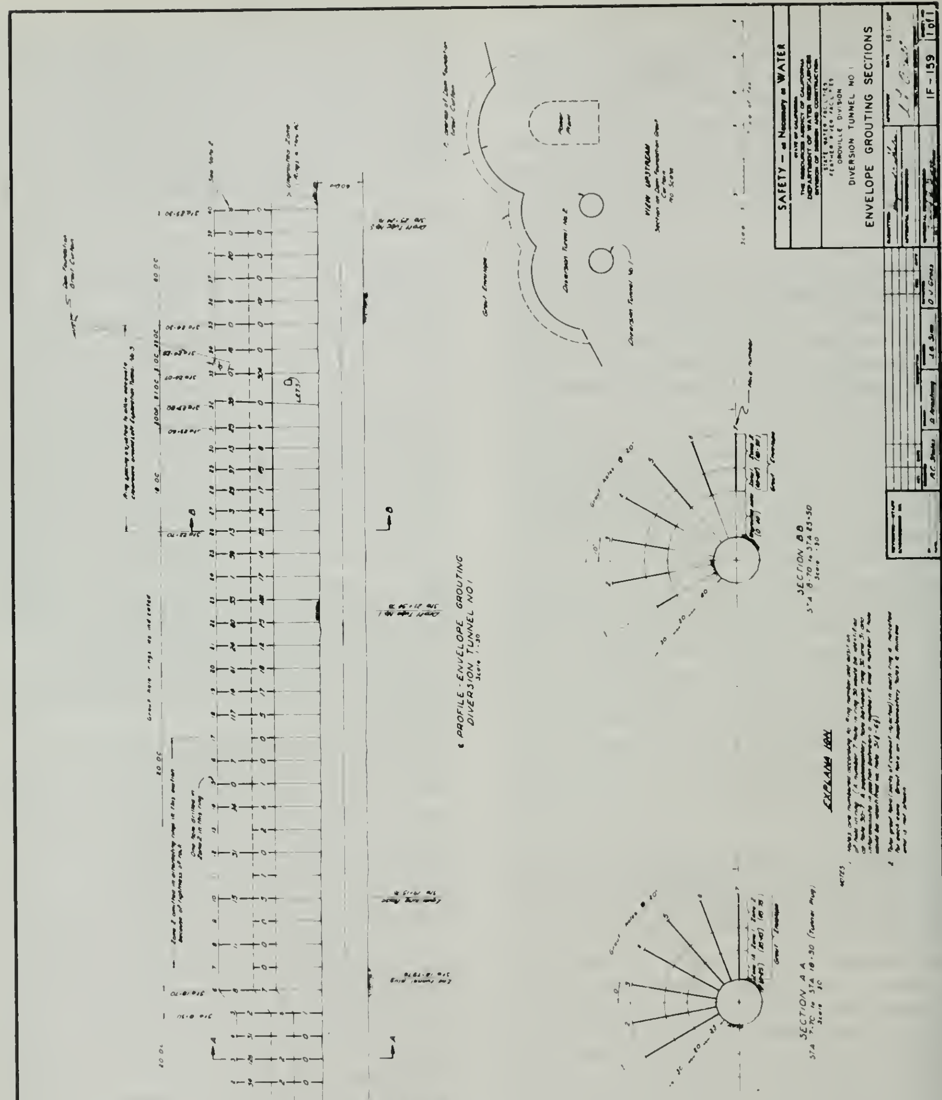

Figure 69. Grout Envelope

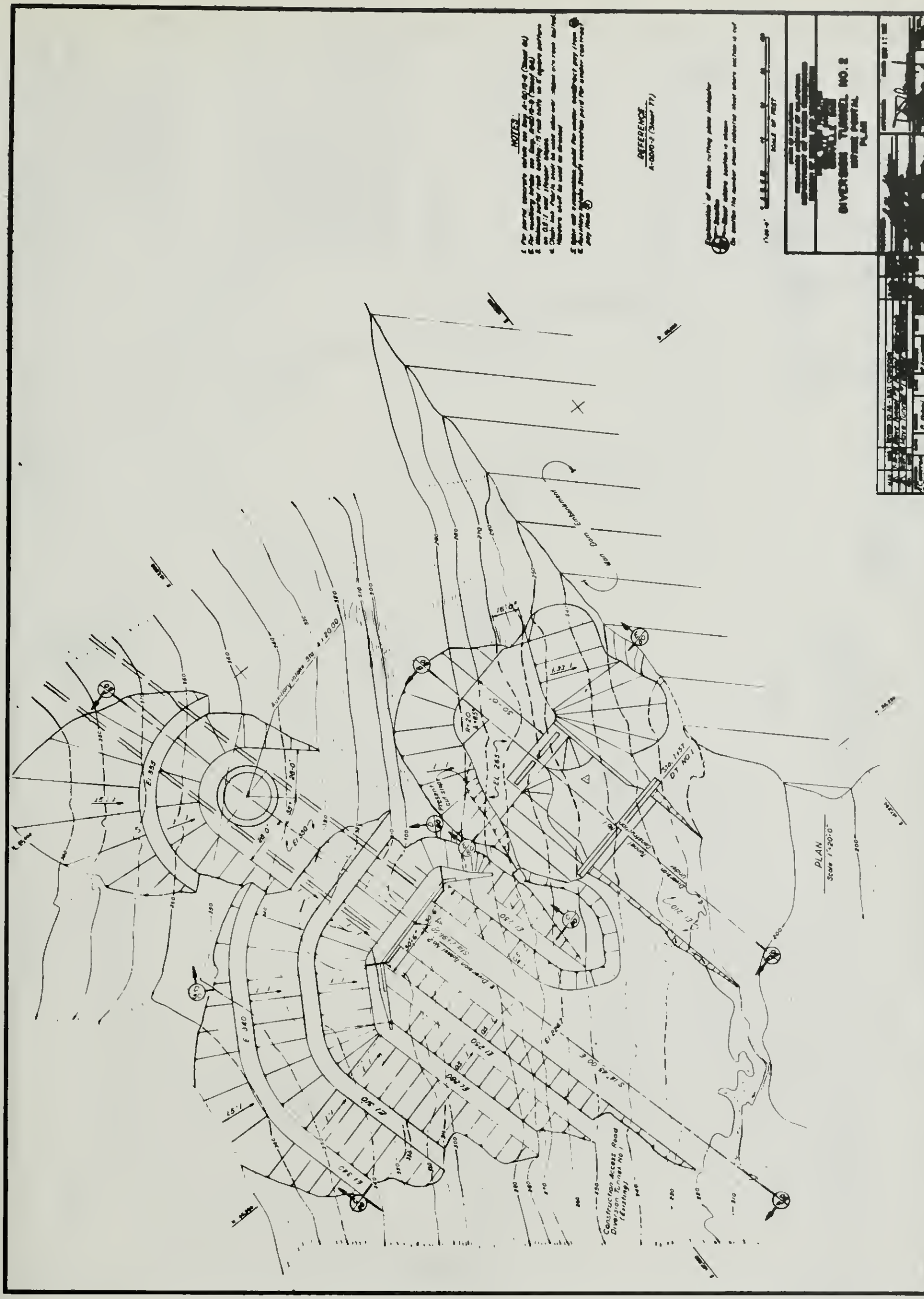

Figure 70. Intake Structures Plan

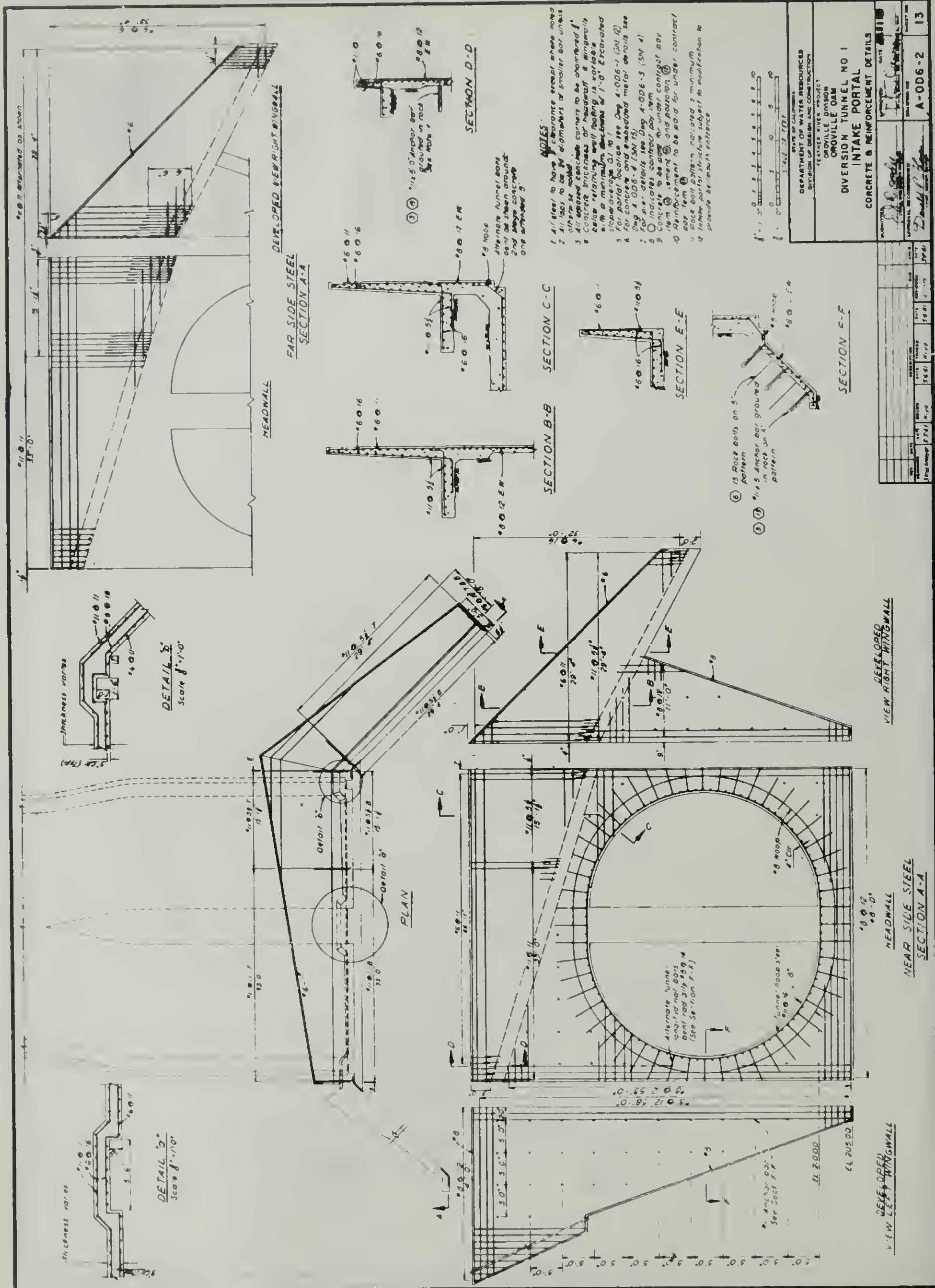

Figure 71. Diversion Tunnel No. 1—Intake Portal Sections

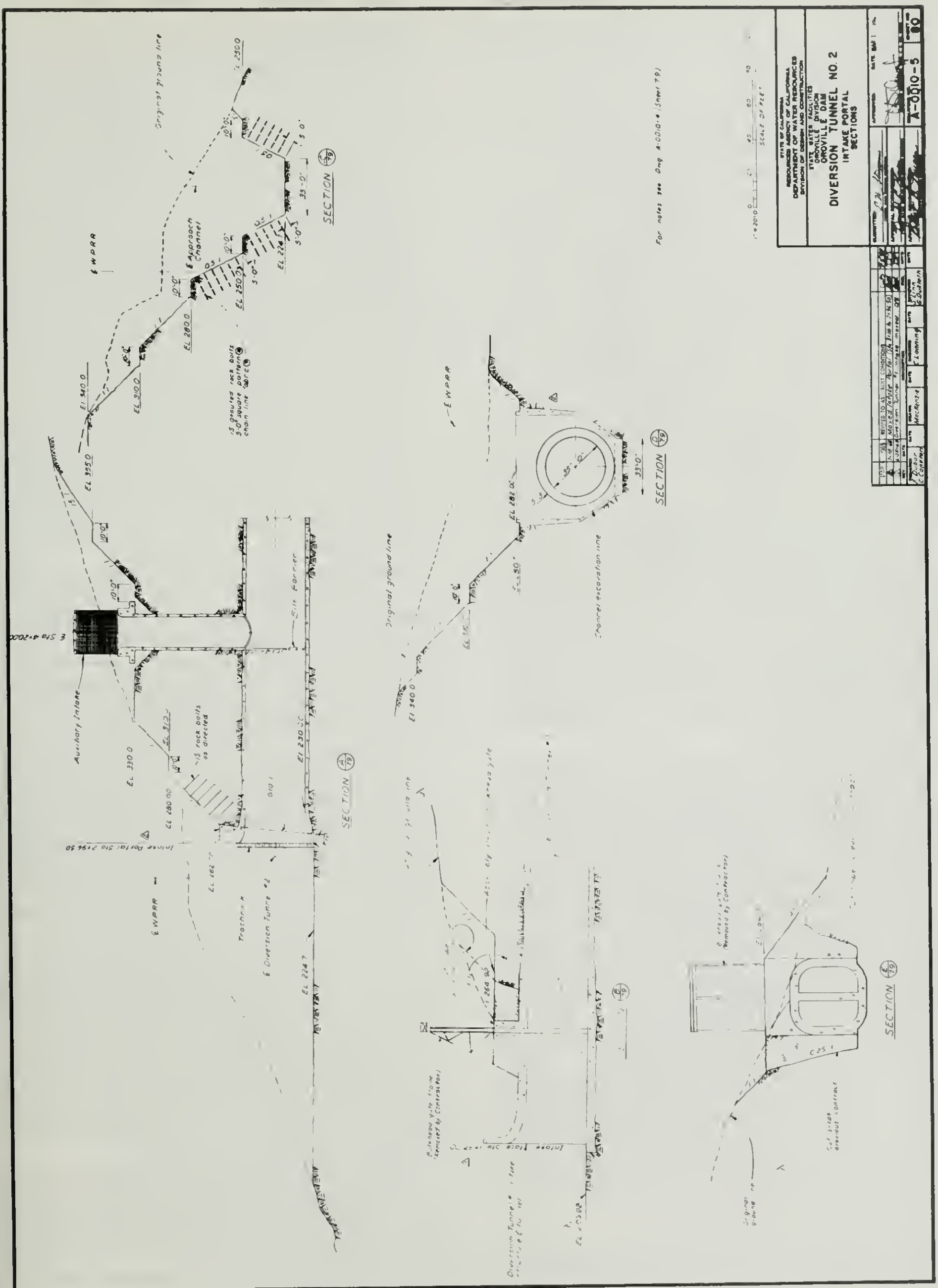

Figure 72. Diversion Tunnel No. 2—Intake Portal Sections

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The closure gates, 17 feet - ½ inches wide by 36 feet - 6 inches high, were built up from wide-flange structural sections and provided with rubber seals. They were positioned in a chimneylike structure above the gate slots prior to closure and lowered into place by a frame and cable system. Low streamflows were necessary to allow gate seating. A trial lowering was required for each gate, prior to final closure, to ensure that adequate sealing could be accomplished.

The intake walls and roof of Diversion Tunnel No. 1, upstream of the gate, were designed for an external hydrostatic head equal to the velocity head plus the entrance head loss during peak discharge while diverting the standard project flood. The closure gates and the structure downstream of the gates were designed for the maximum water surface in the reservoir during plug construction. Normal working stresses occur with a load of 440 feet of water, and yield is not exceeded with a full reservoir (elevation 900 feet).

Diversion Tunnel No. 2 intake is a circular bell mouth. The headwall and wingwalls are anchored to the rock face with grouted No. 11 bars. No provisions for gating were required as the intake is 20 feet higher than Diversion Tunnel No. 1 and was out of water during construction of the plug and river outlet (Tunnel No. 1 remaining open during this period). A concrete trashrack was constructed at the intake face to guard the river outlet valves from submerged debris. An auxiliary intake for the river outlet was placed at elevation 340 feet and connected to the tunnel by an 18-foot-diameter shaft. This intake supplies water to the river outlet should the tunnel intake be closed by silt. This intake also is provided with a concrete trashrack. The trashracks are designed to withstand a differential head of 20 feet.

Diversion-Tailrace Tunnel Outlet Portal Structures. Both tunnels have similar outlet portal structures consisting of a concrete headwall and a 50-foot-long concrete trough with a semicircular invert (Figure 73). Gate slots are provided in the troughs for installation of a bulkhead gate to dewater the tunnels. The gates extend from trough invert to normal tailwater surface, elevation 225 feet. The troughs are anchored to the rock with grouted bars and are designed to resist the forces of external head to elevation 225 feet with the interior dewatered. The headwalls also are anchored to the portal face with grouted bars. These bars are designed to resist a 10-foot hydrostatic head behind the wall.

Above the headwalls, working areas and concrete slabs are located on which trashrack hoisting equipment is mounted. The trashracks are held above the troughs on the portal face during generation of power and lowered into slots provided in the trough during the pumping cycle. Design of the trashracks is covered in Volume IV of this bulletin.

River Outlet Access Tunnel. The river outlet access tunnel (Figure 74) connects the valve chamber in Diversion Tunnel No. 2 with the northern end of the powerplant machine hall.

The 8-foot-diameter tunnel extends from elevation 262 feet in the powerhouse to the river outlet control chamber at elevation 290 feet, an enlarged portion of the tunnel which contains the control equipment. It continues to an 8-foot-diameter reach extending from the chamber to the diversion tunnel (invert elevation at intersection-elevation 233 feet). Metal stairs were installed in the inclined portions. The control chamber enlargement is 15 feet in diameter and 33 feet long. It was placed at elevation 290 feet to preclude flooding of the powerhouse with maximum tailwater during spillage of the maximum probable flood (maximum water surface elevation 287 feet). A panel downstream of the valve chamber is designed to pop out when a head of 15 feet bears on it; thus, the tunnel will not serve as a passage to flood the powerhouse should a failure of a river outlet component occur.

The concrete lining is 22 inches thick in the valve control chamber and 12 inches thick in the 8-foot-diameter tunnel. The lining will support the external hydrostatic head of full reservoir. As the tunnel is within the zone of influence of the powerplant drainage system, the actual head on the structure will be somewhat less.

Powerhouse Emergency Exit Tunnel. The purposes of the powerhouse emergency exit tunnel, shown on Figure 74, are to provide an alternate means of escape from the underground powerhouse in case of emergency and to act as an access to the grout gallery from the powerhouse. This tunnel, 8 feet in diameter and approximately 570 feet in length, is discussed in Volume IV of this bulletin.

Core Block Access Tunnel. The purpose of the core block access tunnel is to convey seepage water from the core block to Diversion Tunnel No. 2. It is a former exploration adit and was a planned low-level connection between the core block and powerhouse. It was plugged near the powerhouse wall to reduce the chance of flooding in either facility due to this interconnection. The tunnel is 7½ feet in diameter and approximately 780 feet in length.

A 16-inch-diameter steel pipe was installed in the tunnel to convey discharge from the core block drainage pumps. This pipe terminates in the vertical hole above Diversion Tunnel No. 2. The tunnel will convey water from the core block to the diversion tunnel should a failure of the drainage pumps occur.

The concrete tunnel lining is 10 inches thick. This thickness is sufficient to resist the external hydrostatic pressure of full reservoir and yet be convenient for placement of concrete.

Palermo Outlet Tunnel. Because the construction of Oroville Dam terminated the previous method of supply to Palermo Canal, a tunnel outlet (Figure 75) was designed to make releases into the Canal downstream of the Dam.

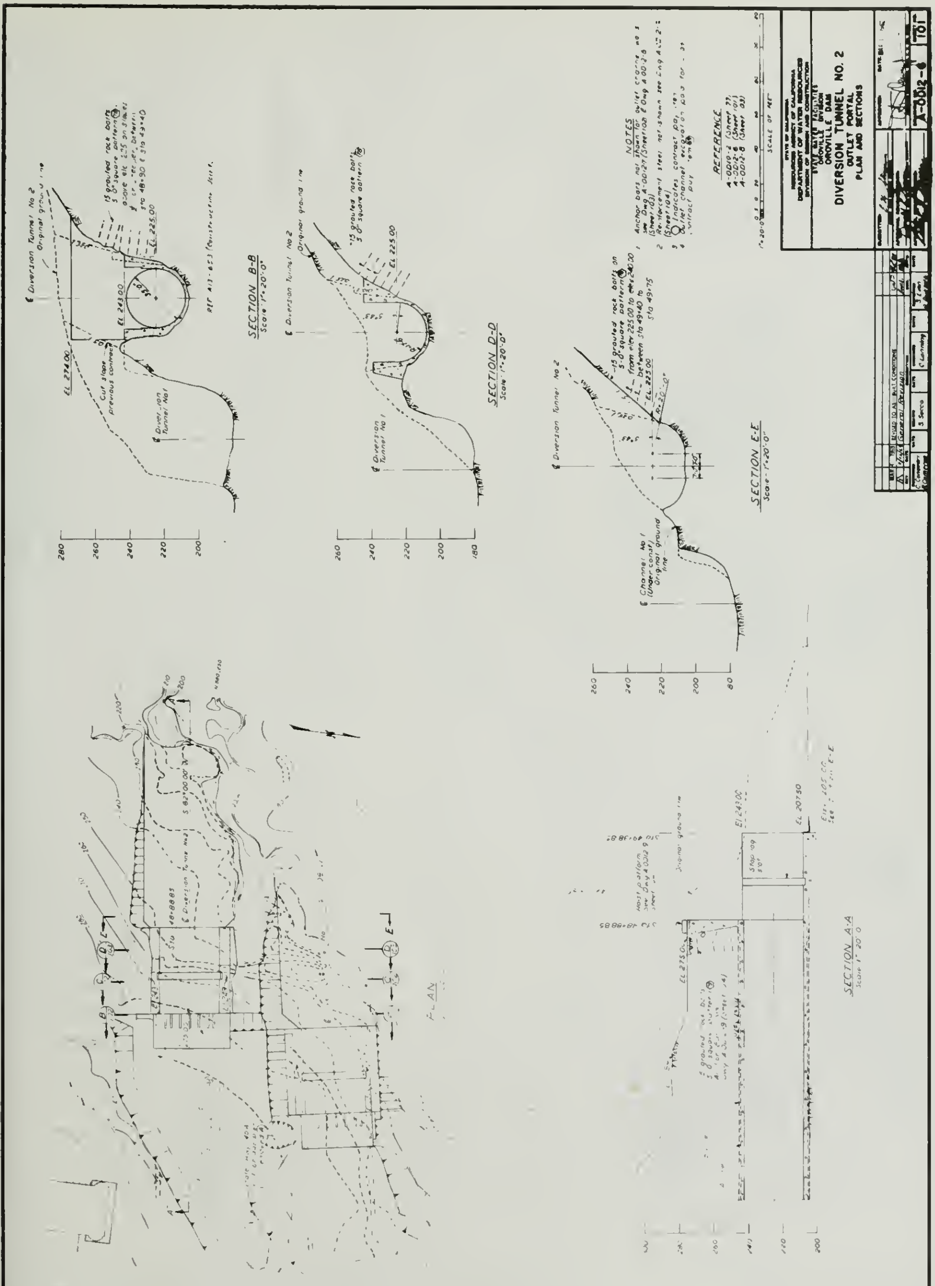

Figure 73. Diversion Tunnel Outlet Structures—Plan and Sections

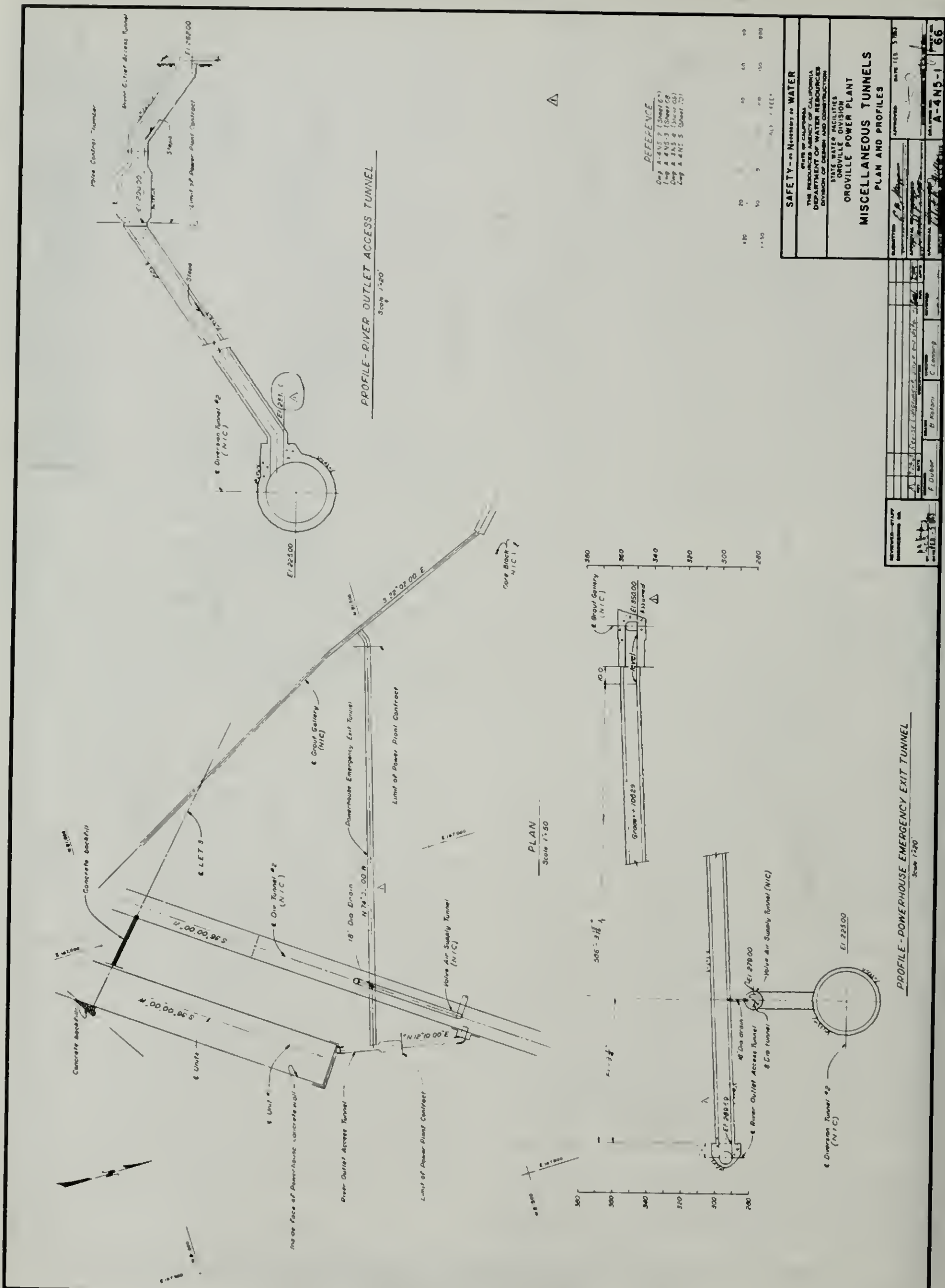

Figure 74. Miscellaneous Tunnels

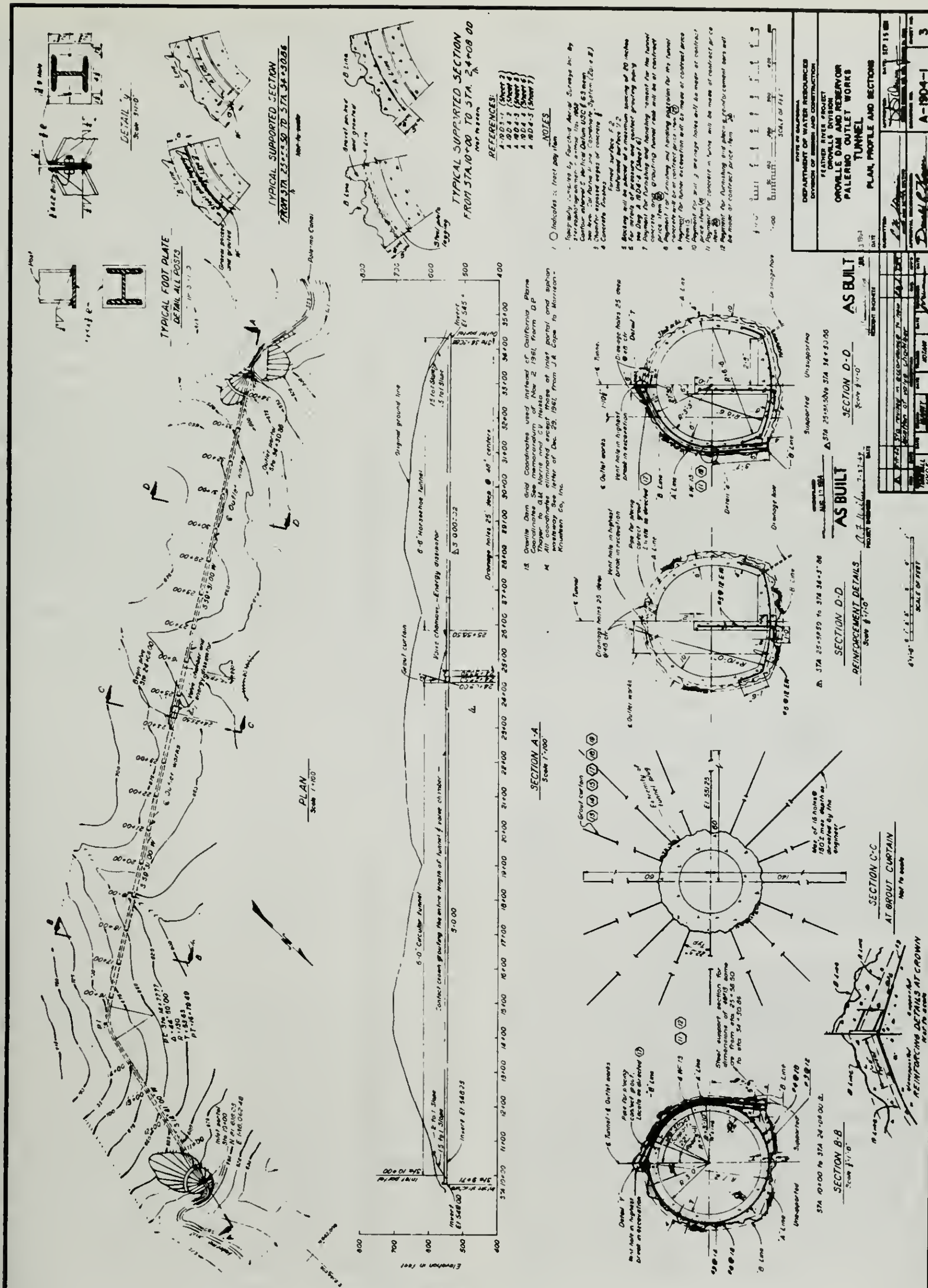

Figure 75. Palermo Outlet Tunnel

The outlet tunnel is concrete-lined and is approximately 2,430 feet long. The intake portal invert is set at elevation 548.25 feet. A 6-foot-diameter level tunnel connects the intake with a valve chamber located immediately downstream of the intersection with the dam grout curtain. Downstream of the chamber, a 6-foot - 6-inch horseshoe section continues to the downstream portal on a slight downhill grade. A wall divides the downstream reach into a water-carrying passage and a 3-foot-wide access walkway. The outlet valve is connected to a steel conduit which is embedded in a concrete plug immediately upstream of the valve chamber.

The intake portal structure consists of a short length (approximately 27 feet) of cut-and-cover tunnel section with invert elevation 549 feet. Slots are provided for a trashrack and bulkhead gate and a bulkhead gate is held in place above the intake. To dewater the tunnel, a diver is required to connect a hoist rope to the gate and remove the pins which hold the gate in its storage position.

Tunnel lining upstream of the dam cutoff was designed to resist the external hydrostatic head of full reservoir. Downstream reaches were designed to resist the maximum pressure used during tunnel grouting. This was assumed to be 25 psi. Drain holes were drilled through the lining in the downstream reach to ensure that ground water pressures on the lining do not exceed this design pressure. A grout curtain is provided upstream of the valve vault to mesh with the curtain under the Dam.

The outlet facility in the tunnel consists of a 12-inch-diameter rated discharge valve backed up by a 30-inch-diameter butterfly valve. Discharge is made through a steel hood into an energy dissipator.

The downstream portal structure consists of a concrete headwall and wingwalls paralleling the channel to retain material which may ravel from the cut slopes. Immediately downstream of the portal, a reinforced-concrete Parshall flume with a throat width of 5 feet was constructed in the channel and equipped with a recorder.

During construction of Oroville Dam, water for Palermo Canal was obtained from Oroville-Wyandotte Irrigation District's Kelly Ridge penstock, which is approximately 2,600 feet downstream of the outlet portal. A 16-inch turnout pipe connects to an access door in the penstock (elevation 583 feet) approximately 240 feet northeast of the point where the penstock crosses the Canal. The buried turnout pipe terminates near the Canal with a 10-inch-diameter, 40-cfs, rated, discharge valve enclosed in a reinforced-concrete energy dissipator. The valve is located at elevation 547 feet. If, in the future, there arises some emergency situation where water becomes unavailable from Lake Oroville in the required quantity, the needed water can again be supplied by Kelly Ridge penstock.

The outlet requirements of this turnout are the same as for the outlet tunnel, i.e., a maximum discharge of 40 cfs.

Spillway

The spillway for Oroville Dam (Figure 76) is located in a natural saddle on the right abutment of the Dam. This location allows spillway discharges to enter the River, well downstream of the toe of the Dam and powerplant tailrace. The spillway consists of a combined flood control outlet and an emergency weir.

The flood control outlet consists of an unlined approach channel with approach walls shaped to make a smooth transition to the outlet passage, a headworks, and a chute. The headworks structure (Figure 77) has eight outlet bays controlled by top-seal radial gates, 17 feet - 7 inches wide by 33 feet high. A concrete chute (Figure 78), 178 feet - 8 inches wide, extends 3,050 feet from the flood control outlet down the side of the canyon to a terminal structure (chute blocks) where the water plunges into the Feather River.

The emergency spillway is an ungated, concrete, overpour weir located to the right of the flood control outlet and is made up of two sections (Figure 79). The right 800-foot section is a broad-crested weir on a bench excavation. The left 930-foot section is a gravity ogee weir up to 50 feet in height. Except for a narrow strip immediately downstream of the weir, the terrain below the weir was not cleared of trees and other natural growth because emergency spillway use will be infrequent.

The flood control outlet was sized on the basis of limiting Feather River flow to leveed channel capacity of 180,000 cfs during occurrence of the standard project flood (peak inflow 440,000 cfs). This limitation applies at the confluence of the Feather and Yuba Rivers approximately 35 miles downstream of the Dam. It was estimated that a runoff of 30,000 cfs could be expected within this 35-mile reach of the Feather River during the standard project flood. Therefore, the flood control outlet was designed for a controlled release of 150,000 cfs. The normal reservoir water surface previously had been set at elevation 900 feet. To meet these criteria, a flood control reservation of 750,000 acre-feet was needed. The criteria also governed the size and location of the flood control outlet gates. The outlet must release 150,000 cfs at water surface elevation 865 feet to control the flood shown on Figure 80.

The standard project flood has a probability recurrence interval of approximately 450 years. If data received indicate a flood is developing greater than the standard project flood, release through the flood control outlet may be increased above 150,000 cfs but may not exceed 90% of the inflow. When the reservoir fills above elevation 901 feet, flow occurs over the emergency spillway. The emergency spillway, in conjunction with the flood control outlet, has the capacity to pass the maximum probable flood release of 624,000 cfs for the drainage area (peak inflow 720,000 cfs) while maintaining a freeboard of 5 feet on the embankment. The maximum probable flood has a probability recurrence interval in excess of 10,000 years. Hydrologic and hydraulic data are shown on Figure 80.

Various types of spillways were studied and modeled to arrive at the final structure. The original design consisted of a control structure with radial gates to pass the total spillway design flood. A short concrete apron was to extend downstream from the control structure, and then the flows were to be turned loose down the hillside in an excavated pilot channel. As the spillway would operate on the average of every other year, this plan was determined to be unacceptable based on the large quantities of debris that would be washed into the Feather River and could ultimately affect power operations. Adding a converging concrete-lined channel and chute to the original headworks structure created major standing-wave problems throughout the system. These problems were resolved by separating the flood control structure from the spillway structure as shown on Figure 76.

The rating curve for the flood control outlet (Figure 81) is based on these hydraulic studies.

Concrete for the spillway chute, weir, and flood control outlet structure above elevation 865 feet was specified to obtain a strength of 3,000 psi in 28 days; concrete for the lower portions of the flood control outlet, below elevation 865 feet, was specified to obtain a strength of 4,000 psi in 28 days; and concrete immediately behind the prestressed trunnion anchorages was specified to obtain a strength of 5,000 psi in 28 days.

Steel reinforcement conforms to intermediate or hard-grade billet steel as specified in ASTM Designation A15 or A408.

Post-tensioned tendons for the gate trunnion anchorages have an ultimate strength of 160,000 psi.

Structural steel for the main members of the radial gates and bulkhead gates conforms to ASTM Designation A441. Secondary gate members and trunnion beams are of A36.

Headworks. The top of the 570-foot-long headworks is coincident with the top of the Dam (elevation 922 feet). The gated outlet passages are placed in an excavated channel depressed from the emergency spillway approach channel. The invert of the outlet is elevation 813.6 feet.

Four bridges or service decks are provided: the crest of the structure is a roadway used for maintenance purposes, including placement of stoplogs; the radial-gate hoist deck is at elevation 886.5 feet; the spillway road bridge providing access to the right abutment by way of the dam crest is at elevation 870 feet; and a walkway for inspection of the gates and trunnions is at elevation 847 feet.

Because the headworks structure is founded on competent rock, sliding was not considered a factor in stability. The structure was analyzed for safety against overturning.

The embankment grout curtain was extended under the headworks. It consists of a single line with a maximum depth of 50 feet.

Drain holes were drilled into the foundation rock downstream of the grout curtain. Uplift pressures were assumed to be 100% of reservoir head at the upstream edge, reducing in a straight line to 33% at the drains and zero at the downstream toe.

Reinforced-concrete piers, 5 feet thick, separate the gated water passages. The piers also support the breast wall and service decks and provide anchorage for the gate trunnions. The pier noses are steel-armored to reduce wear, and guides for stoplogs are welded to the leading edges.

The crest service bridge is made up of reinforced-concrete slabs spanning between the piers. The total width is 21 feet. The spillway road bridge deck is of similar construction but the total width is 34 feet. Reinforced-concrete bents were required on the downstream end of the piers to support the bridge.

The maintenance deck involves individual reinforced-concrete slabs, 18 inches thick, supported on the piers in 4-inch-deep notches. Blockouts are provided in the deck for access to ladders which lead to the gate head seal assemblies. It was designed for a live floor load of 250 pounds per square foot. In addition to the live load, a seismic acceleration of 0.1g parallel to the axis of the structure was investigated assuming the deck acts as a strut between the piers.

The hoist deck, a downstream continuation of the maintenance deck, was designed to support the weight of the gate hoists, the maximum force caused by lifting the gates, and a live load of 250 pounds per square foot on the working surface.

The walkway for inspection of gates and trunnions, a 15-foot-wide reinforced-concrete slab spanning between the piers immediately downstream of the gate trunnions, also was designed to support a live load of 250 pounds per square foot.

The skinplates for eight 17-foot - 7-inch by 33-foot top-seal radial gates were designed as a continuous plate spanning horizontally between vertical supporting members. Bending stresses in the plate were determined by treating the member as a continuous beam; however, horizontal tension also was checked. The load on the skin assembly is transmitted to the trunnions through horizontal girders and canted arms. The outlet gate trunnion assemblies consist of welded-steel trunnion rings to which the three end-frame struts are connnected, steel shoes rigidly attached to the piers (e.g., by a trunnion beam), and trunnion pins.

Adjacent trunnion pins are not interconnected to the pier anchorage but are connected to a common

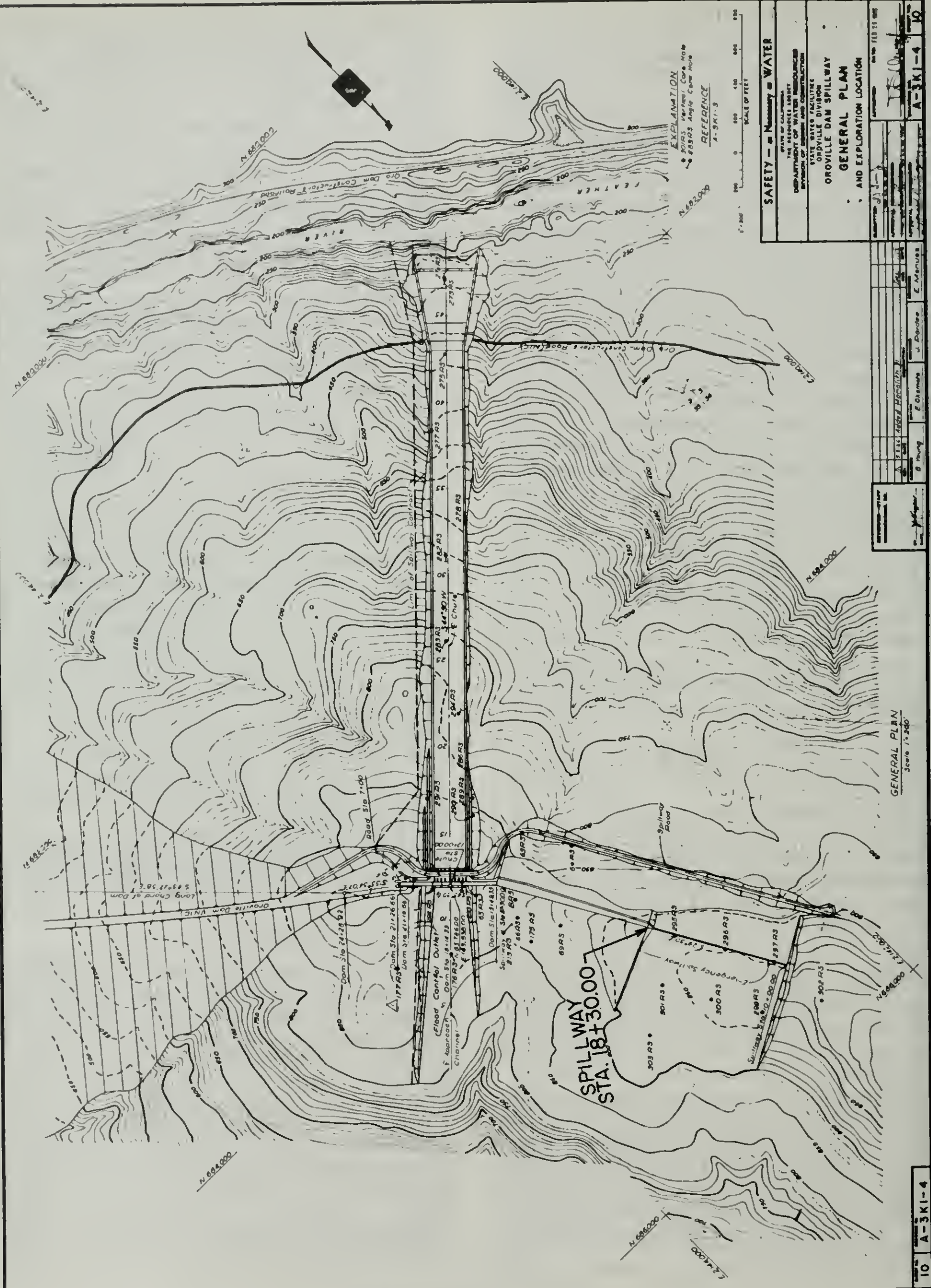

Figure 76. General Plan of Spillway

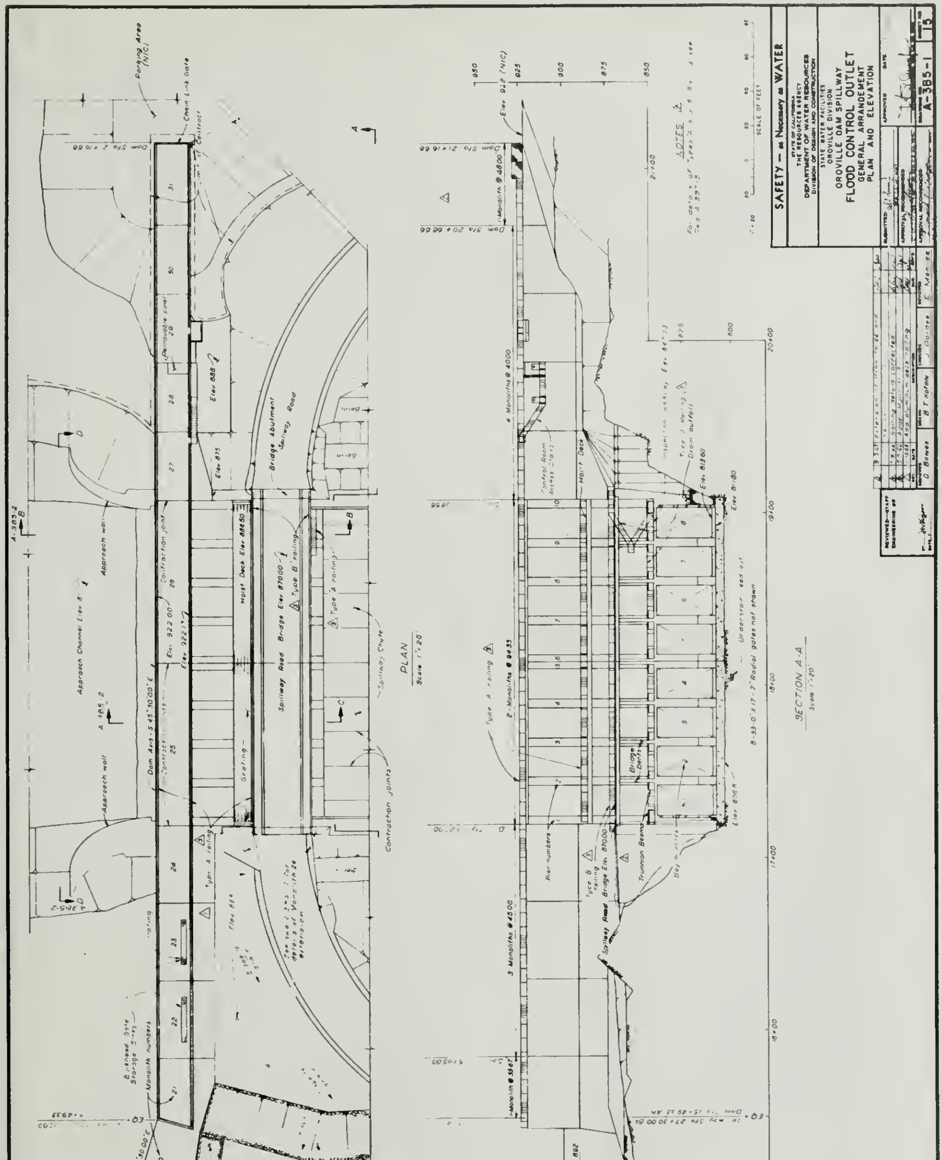

Figure 77. Flood Control Outlet—Plan and Elevation

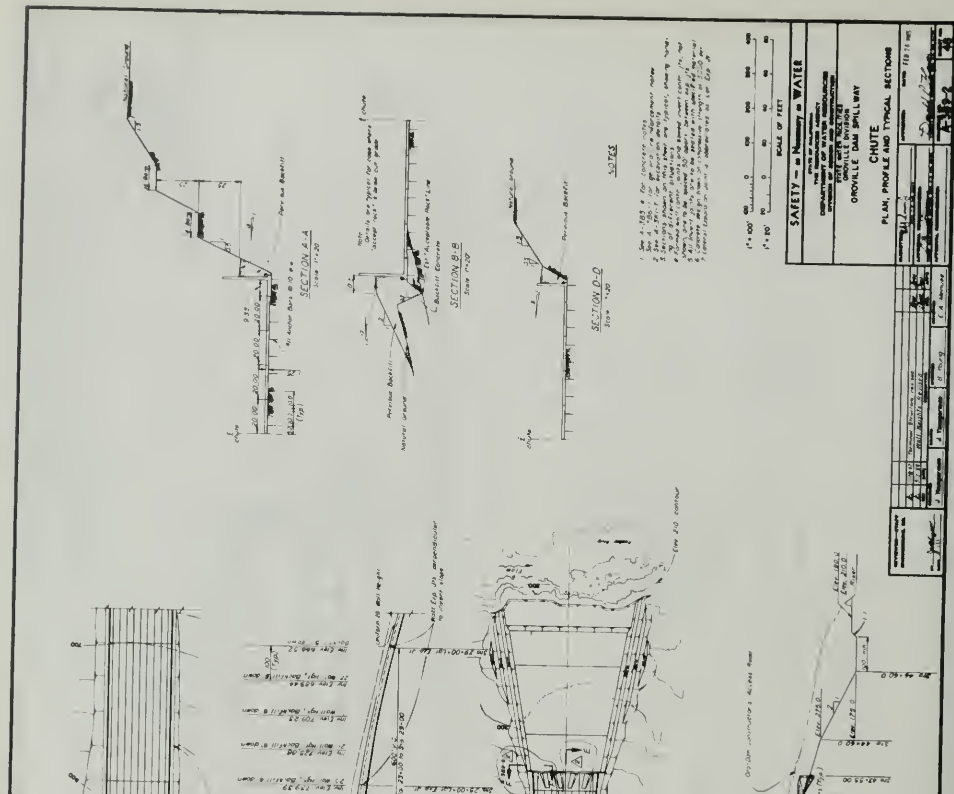

Figure 78. Spillway Chute—Plan, Profile, and Typical Sections

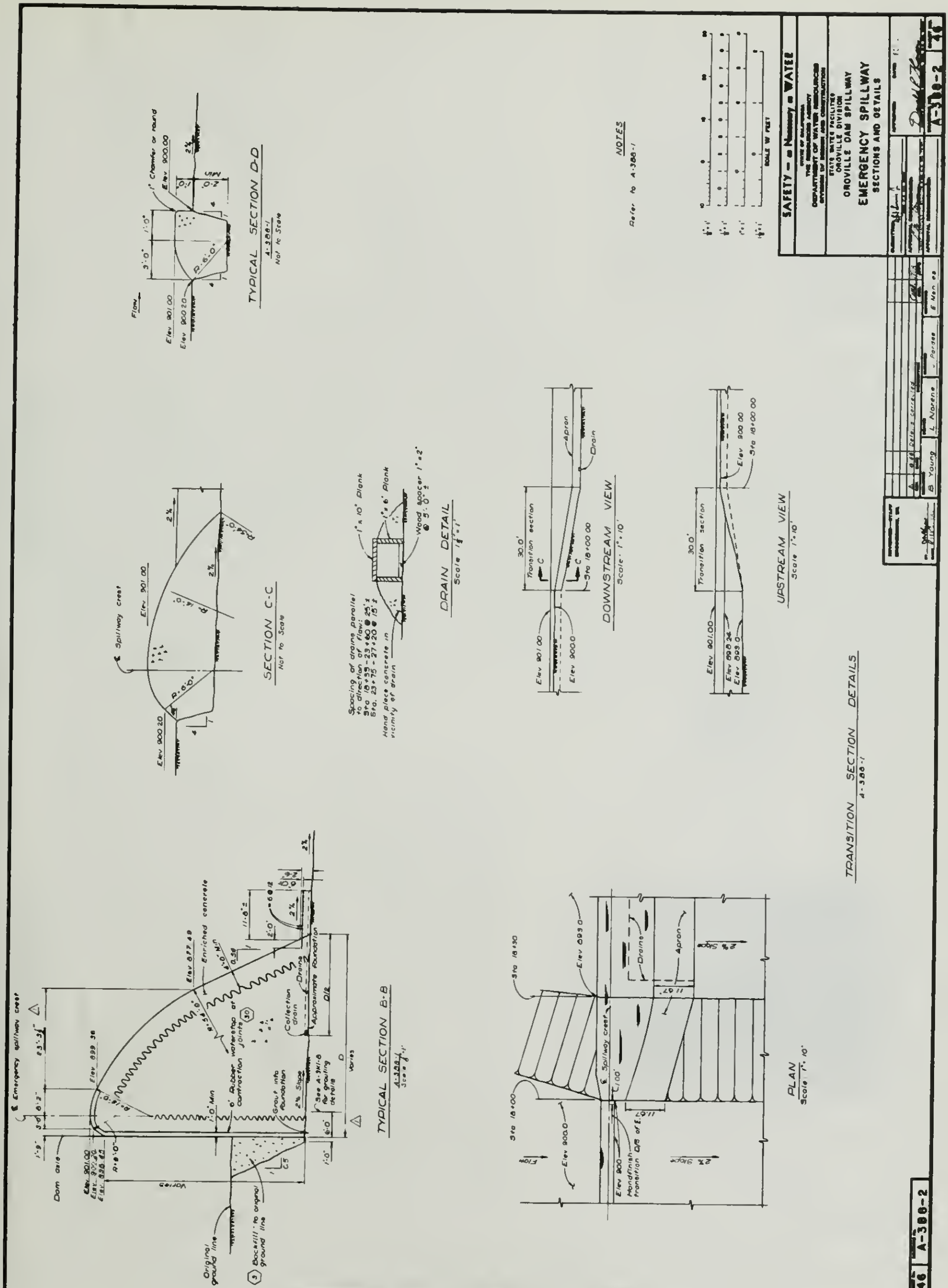

Figure 79. Emergency Spillway—Sections and Details

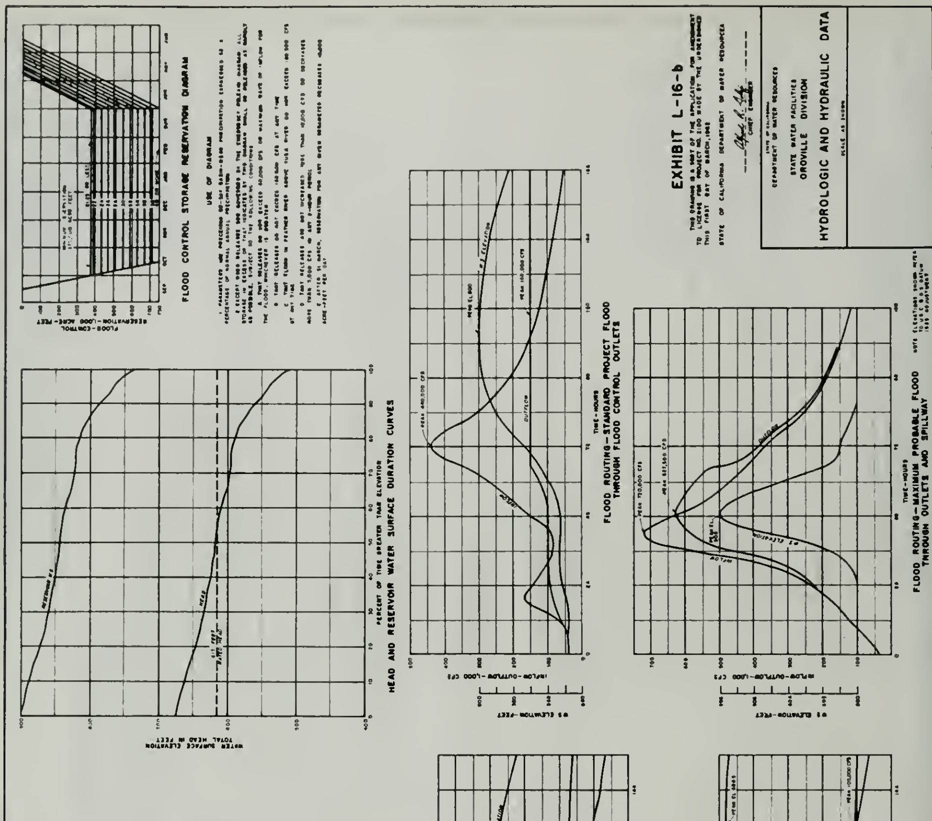

Figure 80. Hydrologic and Hydraulic Data

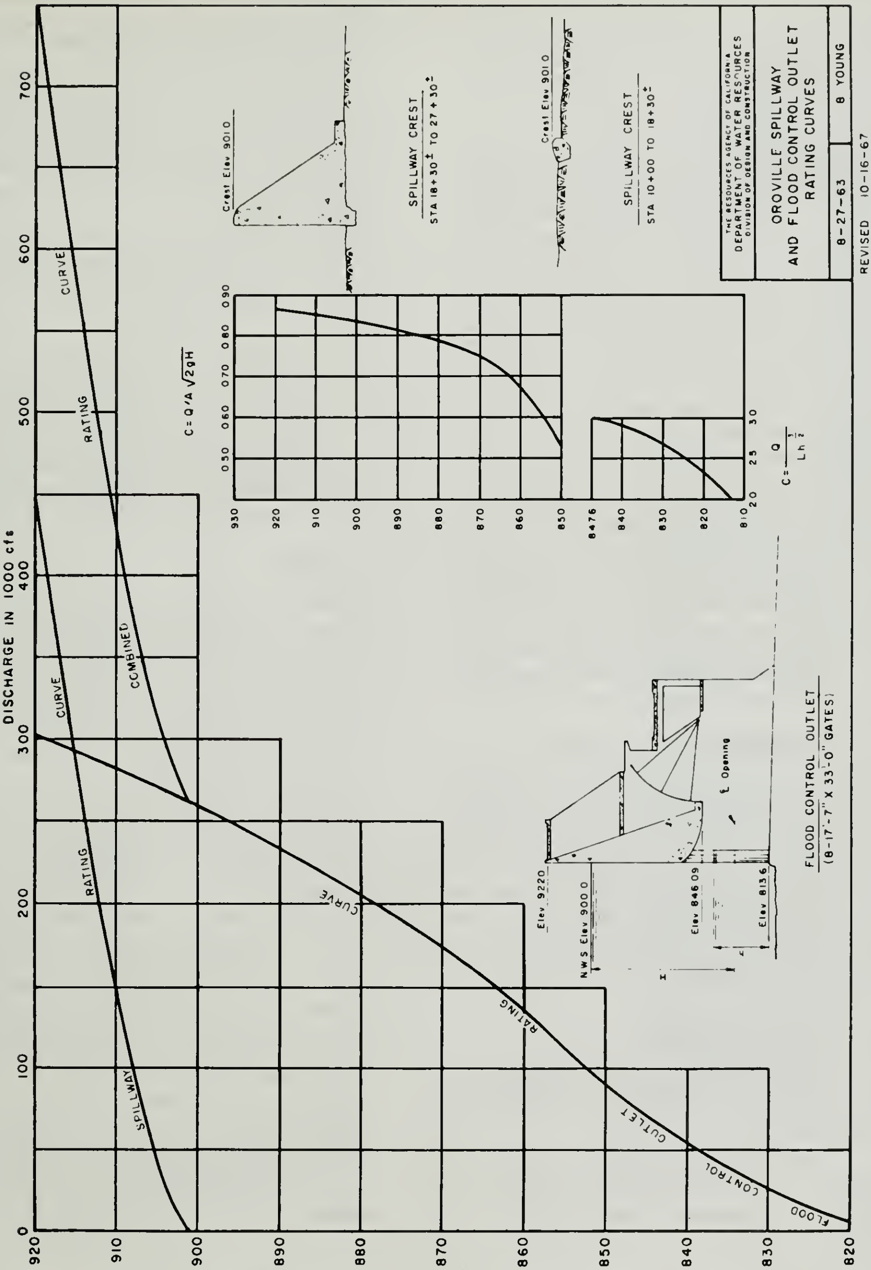

Figure 81. Spillway and Flood Control Outlet Rating Curves

support beam. At the end piers, support beams act as simple cantilevers. Post-tensioned embedded steel rods, at an alignment approximately parallel to the maximum resultant hydrostatic pressure, anchor the support beams to the piers. Twenty-four 1⅜-inch-diameter tendons anchored 37 feet - 6 inches into the piers are used for each gate trunnion.

As the radial gates are submerged (Figure 82), rubber seals are required on all four sides of the gate. The bottom seal is a rubber bar mounted in the sill plate. The gate rests on this bar, and its weight effects a tight closure. The side and top seals are hydraulically actuated, double-stem, rubber seals. Hydrostatic pressure is applied behind the seals by a direct connection to the reservoir with a two-way activated valve to relieve internal pressure when moving the gate. The side seals slide on and seal against embedded steel plates in the walls. Because sliding occurs when gates are opened or closed, the seal noses are teflon-clad. The friction factor assumed between the teflon-clad seals and the stainless-steel plates was 0.1.

The flood control outlet radial gates are operated by electric-motor-powered cable-drum hoists located on the hoist deck. The gates may be operated locally or remotely from the Oroville Area Control Center. Normal power for hoist operation is supplied through a buried distribution line from Edward Hyatt station service power system. Standby power is available locally in the form of a 55-kW generator operated by a liquid-propane-gas-fueled engine. Normal power supply is sufficient to operate all gates simultaneously.

A set of stoplogs is provided to dewater outlet bays to allow maintenance work to be performed on the radial gates.

Five interchangeable leaves are stacked in slots in the pier noses and make up the set required to close off one gate bay. The gate leaves are each 8 feet - 10 inches high and were constructed of ½-inch-thick skinplate and box-shaped ribs. Rubber seals are used on all four sides ("J" seals on the top and sides, and wedge-shaped seals on the base). Each leaf is provided with a 4-inch gate valve to refill the volume between the radial gate and stoplog and thus equalize the pressure for gate removal. The stoplog leaves were designed for a hydrostatic head of 87 feet (elevation 813 feet to elevation 900 feet).

The bottom stoplog seal was modified after initial use. A softer rubber bar-type seal was used to completely seal the bottom corners.

Also, a modified slide gate with regular ropes as operators was installed on the bottom stoplog to hasten the filling time of the space between the stoplogs and the radial gates.

Chute. Except for the terminal structure, the chute walls are cantilever type varying from 20 to 34 feet in height. The walls are structurally independent of the chute invert slab. The reinforced invert slab has a minimum thickness of 15 inches and is anchored to the rock with grouted anchor bars and provided with a system of underdrains.

The terminal structure, designed to dissipate large flows, is more massive and is keyed into the rock foundation to resist forces from the flow. Its walls are gravity type; its invert slab is thicker and keyed into the rock. Mounted on the invert are four 23-foot-high, 43-foot-long, chute blocks designed to separate the solid flow of water.

Energy of this separated jet is absorbed in a plunge pool excavated in the Feather River at the foot of the chute. No concrete lining is used in the plunge pool as rock of adequate quality exists near the ground surface.

Emergency Spillway. The grout curtain was continued under the left reach of the emergency weir near the upstream face, and formed drains are used under the downstream half. The crest of the emergency weir to the right, which is only 1 foot above the excavated channel, is keyed 2 feet into the foundation. Both weir sections were checked for overturning and shear friction safety factor and found to be satisfactory.

Saddle Dams

Location. Bidwell Canyon and Parish Camp Saddle Dams are low dams which complement Oroville Dam in containing Lake Oroville. Bidwell Canyon Saddle Dam is located at the head of the Bidwell Bar Canyon arm of the reservoir and is 1½ miles southeast of Oroville Dam (Figure 55). Parish Camp Saddle Dam is located on the narrowest ridge enclosing the Lake on the West Branch arm of the Lake and is 12 miles north of Oroville Dam.

Description. Bidwell Canyon Saddle Dam is an earth and rockfill dam consisting of two separate embankments. The main dam encompasses the former Miners Ranch Dike built by Oroville-Wyandotte Irrigation District as part of its South Fork project. The west dam meets the main dam at a knoll at the middle of Bidwell Canyon and extends to Kelly Ridge. In addition to containing Oroville Reservoir with a normal pool at elevation 900 feet on its north side, the main dam also contains Miners Ranch Reservoir with a maximum pool at elevation 888 feet on its south side. The plan and profile are shown on Figure 83, and sections and details are shown on Figure 84.

Parish Camp Saddle Dam is an earth and rockfill embankment which extends 260 feet across Lime Saddle, the maximum height being 27 feet. The plan, profile, and sections are shown on Figure 85. Construction of Parish Camp Saddle Dam included plugging a collapsed mine adit found near the Dam.

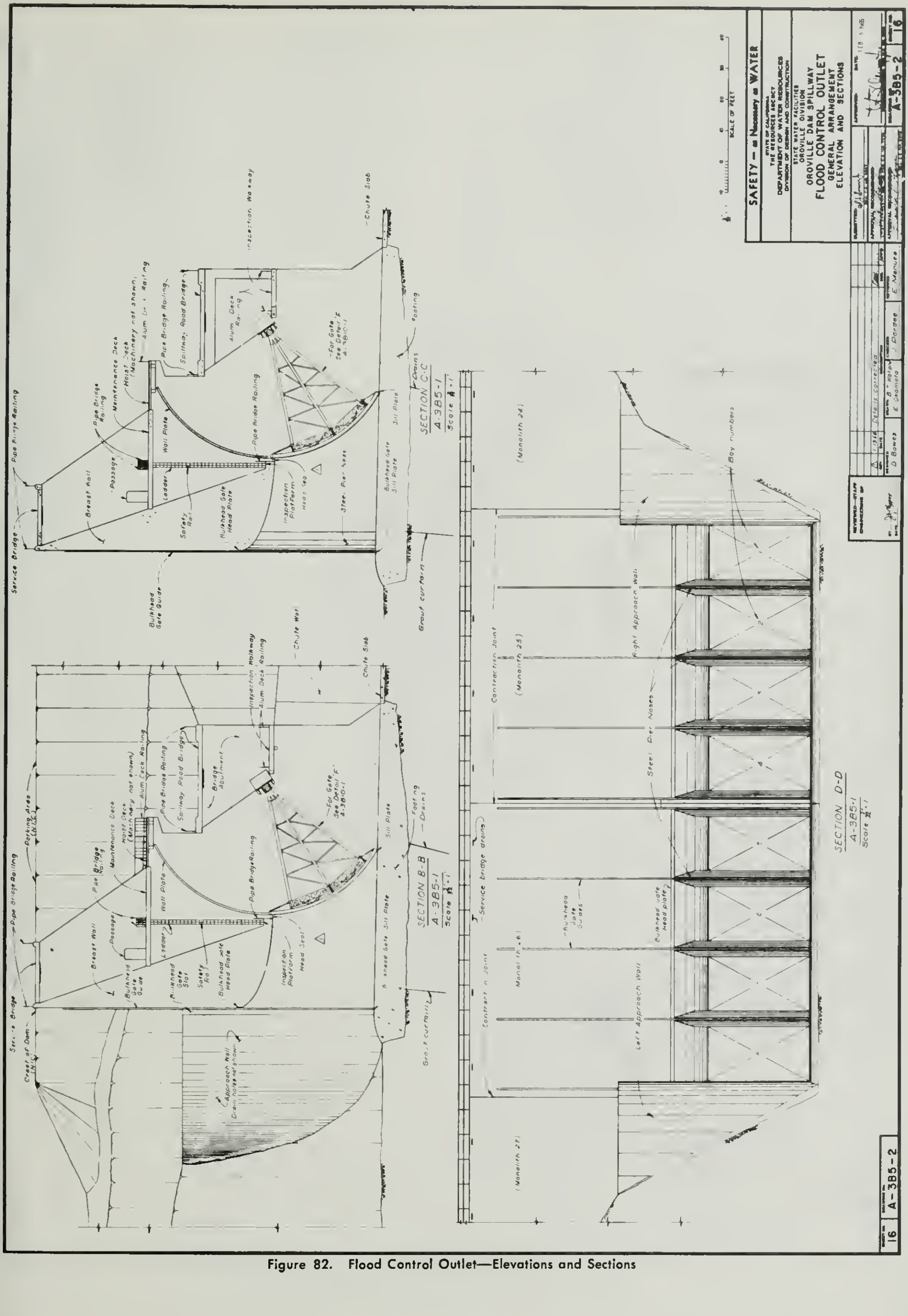

Figure 82. Flood Control Outlet—Elevations and Sections

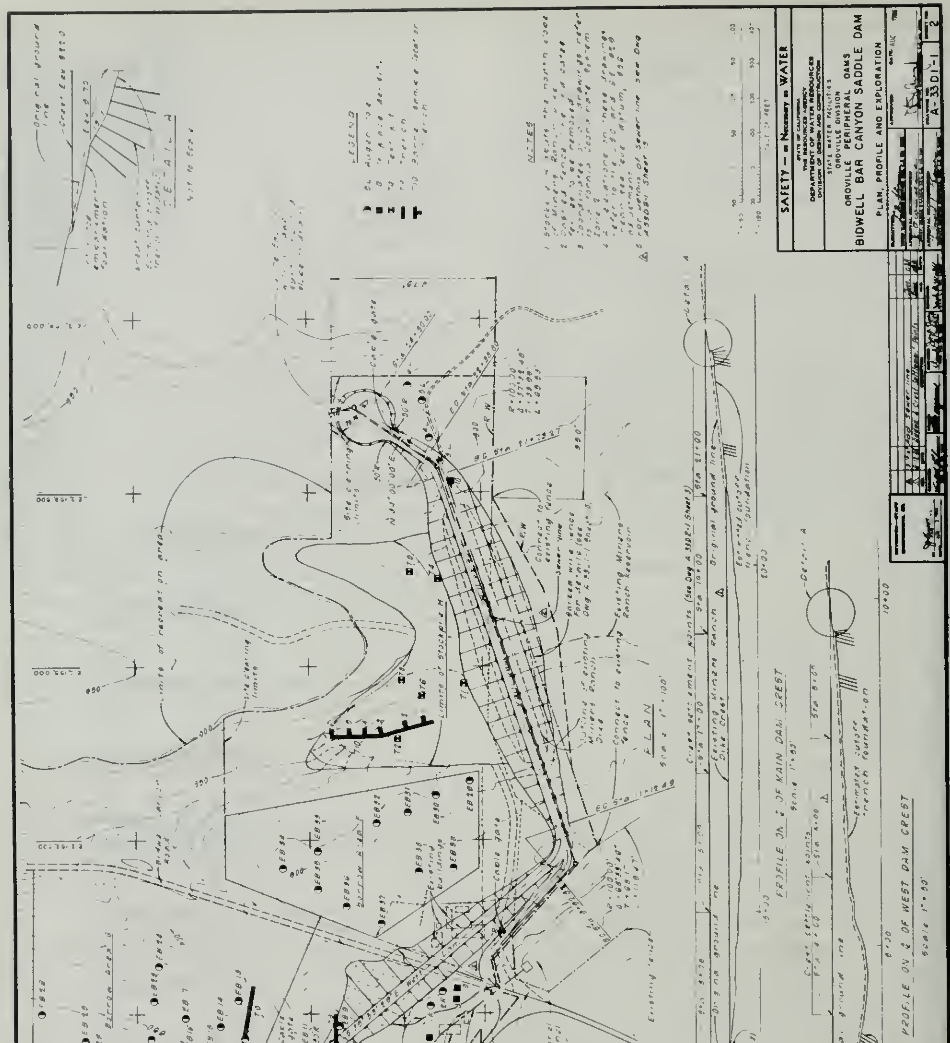

Figure 83. Bidwell Canyon Saddle Dam—Plan and Profile

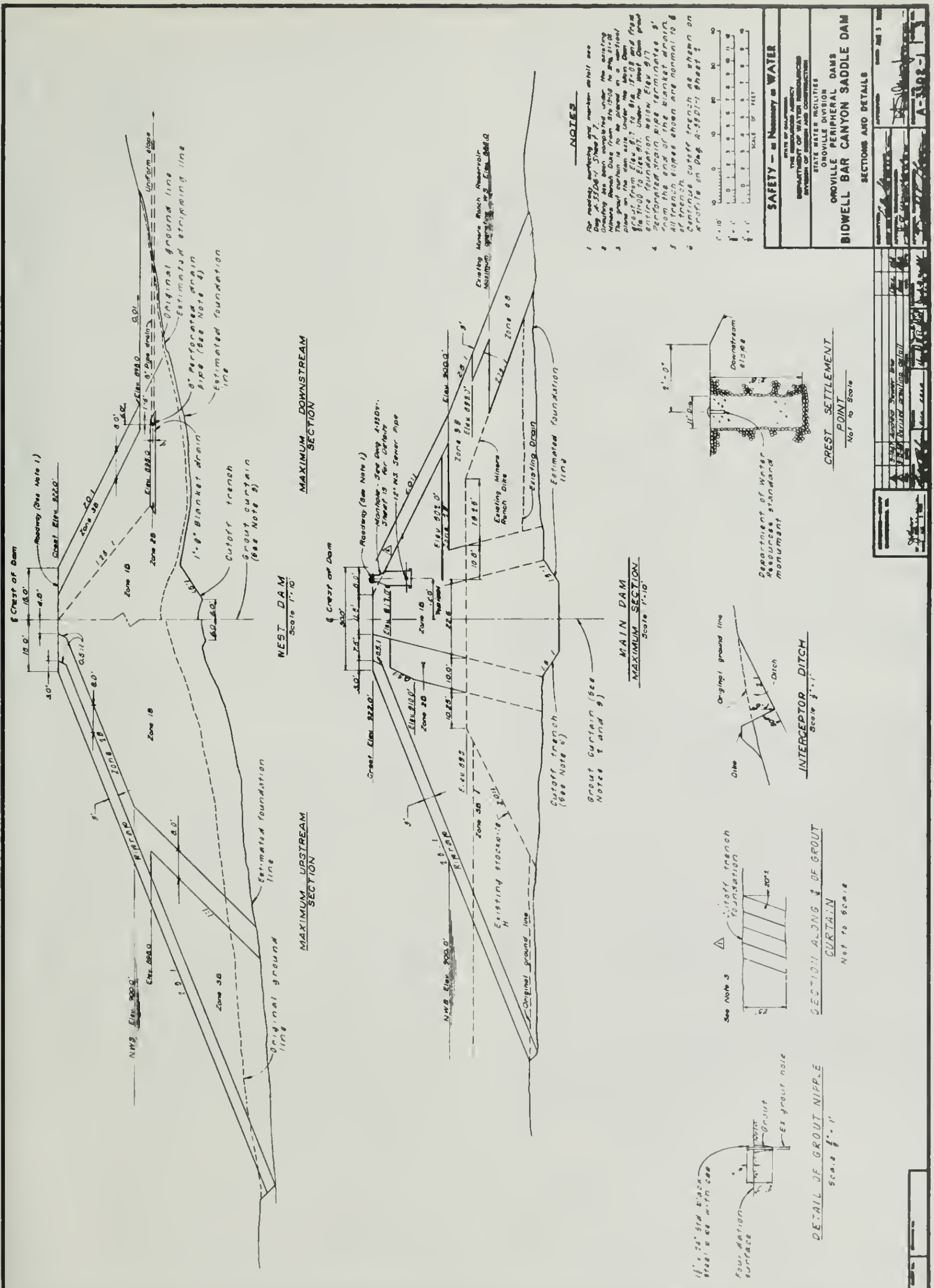

Figure 84. Bidwell Canyon Saddle Dam—Sections and Details

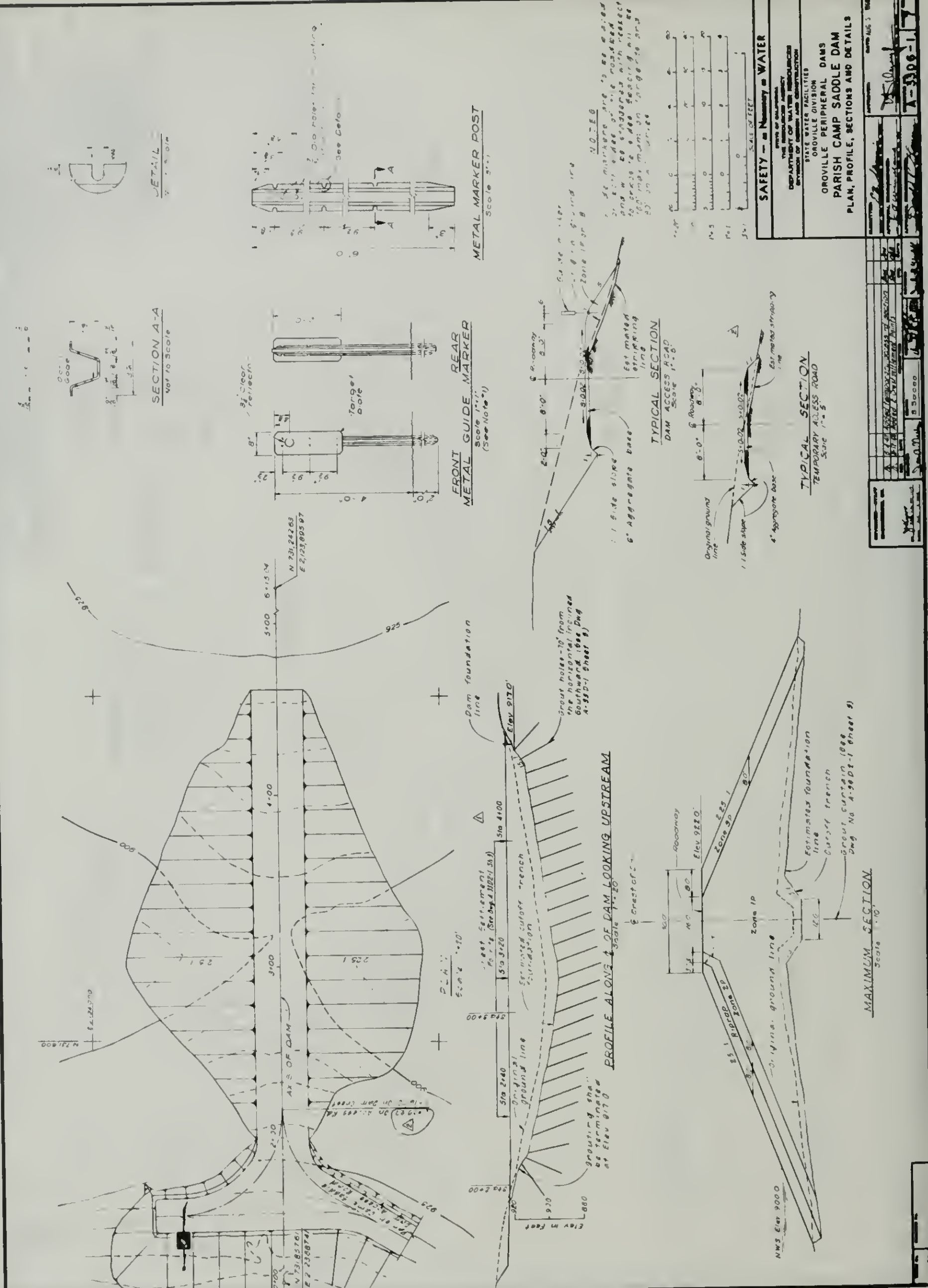

Figure 85. Parish Camp Saddle Dam—Plan, Profile, and Sections

Site Geology. Bidwell Canyon Saddle Dam is founded on a metavolcanic rock consisting mainly of a foliated amphibolite. Amphibolite varies considerably in its degree of weathering throughout the area, with depths ranging from 2 to as much as 18 feet. Fresh exposures generally are dense and tough with a dark bluish or greenish gray color.

Parish Camp Saddle Dam is founded on phyllite, a metavolcanic rock of the Calaveras group, with the exception of a small portion of the left abutment which has blocky metavolcanic rocks of the Oregon City formation. The phyllite is moderately weathered to decomposed, with the weathering ranging from 5 to as much as 16 feet. Weathered phyllite has a fairly high clay content. The Oregon City formation is fresh, dense, black, metavolcanic rock.

Embankment Design. Bidwell Canyon Saddle Dam's main dam was designed as a zoned embankment matching the Miners Ranch Dike's zoning, except that a "downstream wing" was added to the core late in the design period so that random material rather than the scarce rockfill material could be used.

Bidwell Canyon Saddle Dam's west dam was designed as a homogeneous structure except for a downstream horizontal blanket drain, a compacted rockfill upstream toe, and rock slope protection. The blanket drain is located 5 feet below the normal reservoir water surface and contains an 8-inch perforated collector pipe which drains into Miners Ranch Reservoir through an 8-inch pipe. The rockfill toe was included for stability purposes to avoid a flatter upstream slope which would have extended a considerable distance down the Canyon.

Parish Camp Saddle Dam was designed as a homogeneous dam with rock slope protection. The mine adit plug constructed near Parish is of common impervious embankment. Since the adit portal had caved, blocking access for exploration, the conservatively dimensioned plug was constructed to control possible seepage through the narrow ridge.

Foundation Grouting. The grout curtains for both dams are 25 feet deep in a vertical plane. The holes were inclined 20 degrees off vertical along the plane in order to cross the nearly vertical cleavage and schistosity. Selected grout holes were deepened to 50 feet to explore beneath the curtain. The curtain for the main dam is an extension of the curtain for the Dike. An initial hole spacing of 10 feet was used with intermediate holes drilled as necessary to close out the curtain.

Construction Materials. Borrow Areas F and G located outside proposed recreation areas just to the north of Bidwell Canyon Saddle Dam were chosen as the sources of impervious material (Zone 1B). This material consists of decomposed to strongly weathered schistose amphibolite.

Material excavated from Kelly Ridge tunnel approach channel of the South Fork project by Oroville-Wyandotte Irrigation District had been spoiled adjacent to the north side of Miners Ranch Dike. This material was suitable for Zone 3B and was designated Stockpile H (Figure 83).

The transition zone and drain material (Zone 2B), the downstream rock toe of the main dam (4B), and riprap were supplied by the contractor from approved outside sources.

Impervious material (Zone 1P) for Parish Camp Saddle Dam was developed by excavating the weathered phyllite in the two borrow sources not in a proposed recreation area. Excavation for a permanent access road, located through one borrow area, was designated as the primary source, with the remainder of the material supplied by Borrrow Area C immediately to the northwest of the Dam site (Figure 86).

Zone 2P, the transition, was supplied by the contractor from an approved outside source.

Material for the mine adit plug was obtained from Borrow Area E located in the reservoir area 400 feet north of the adit (Figure 86).

Stability Analysis. Stability of the embankments was determined by the Swedish Slip Circle method of analysis. Adequate safety factors were found for all loading conditions. Seismic forces of 0.1g in the direction of instability were included in the earthquake analysis. Analysis of the upstream slopes included several lake levels assuming a horizontal phreatic line at maximum pool elevation 900 feet. The downstream slope of the Bidwell main dam was analyzed for a steady-state seepage condition with the downstream tailwater at maximum water level in Miners Ranch Reservoir, elevation 888 feet. The narrow width and flat slope required for construction obviated the need for stability analysis of the mine adit plug.

Embankment properties used in the stability computations were determined by exploration and laboratory testing of the impervious zones, while standard properties were used for the transition zones, shell zones, and riprap.

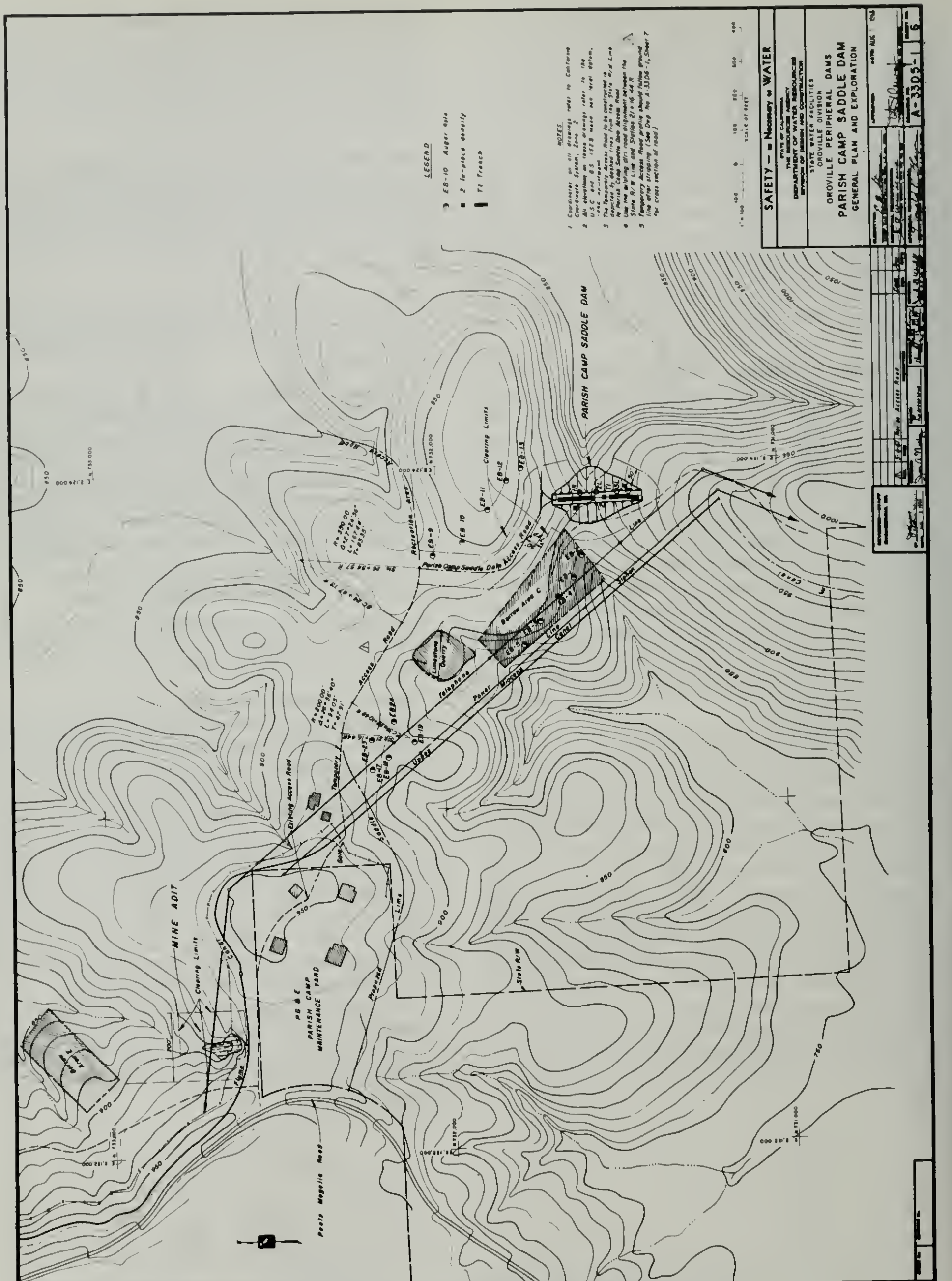

Figure 86. Location of Borrow Areas and Parish Camp Saddle Dam Site

Instrumentation

Dam. The following is a summary of the instrumentation in Oroville Dam:

Instrument	*Number*	*General Location*
Hydraulic Piezometers	54	In dam embankment
Hydraulic Piezometers	2	In dam foundation rock
Electric Pore Pressure Cells	2	In rock next to grout gallery
Dynamic Electric Pore Pressure Cells	6	In dam embankment upstream of Zone 1
Cross-arm Settlement Devices	2	In dam embankment downstream of Zone 1
Fluid Level Settlement Devices	36	In dam embankment
Surface Settlement Points	100	Along crest of dam and on upstream and downstream slopes
Horizontal Movement Units	Units 2	In embankment downstream of Zone 1
Soil Stress Meters, 7¼-Inch Diameter	32	Embedded in core block and grout gallery adjacent to fill material
Soil Stress Meters, 18-Inch Diameter		
Dynamic Soil Stress Cells, 30-Inch	27	In dam embankment
Diameter	15	In downstream Zone 3
Concrete Stress Meters	9	In core block and grout gallery adjacent to foundation rock
Concrete Deformation Meters	8	In downstream face of core block parapet
Accelerometers	2	In dam embankment
Accelerometer	1	In dam foundation
Resistance Thermometers	62	In core block and dam foundation
Extensometers	3	In core block

Spillway. Instruments used were strain meters, pore pressure cells, and pore pressure cells modified for use as piezometers. Strain meters were embedded in Pier 8 and in the breast wall above Bay 6 as the concrete was placed. Pore pressure cells were placed in holes drilled into the rock foundation under Bays 3 and 6 as the piers were constructed. Piezometers were placed in the concrete around Bays 5 and 8.

Saddle Dams. Crest settlement points are the only instrumentation in Bidwell Canyon Saddle Dam and Parish Camp Saddle Dam. There are eight of these at Bidwell and three at Parish Camp Saddle Dam (Figures 83 and 85).

Relocations

Western Pacific Railroad

Relocation of the Western Pacific Railroad out of the Feather River Canyon, site of Oroville Dam and Lake Oroville, was part of the initial work required before major construction for the Dam could start (Figures 55 and 87).

The 23 miles of relocated railroad begins at the easterly end of the City of Oroville. From that point, it crosses the Feather River about 1 mile upstream from the town, traverses northward along the westerly slopes of North Table Mountain, and then connects with the original railroad alignment along the easterly side of the North Fork Feather River. The relocated line passes through five tunnels and over three major bridges. The reconnection point is about one-quarter mile upstream from the shoreline of the maximum operating pool level of the reservoir. Four complete passing tracks approximately 7,000 feet in length are included in the relocated line. Station buildings and appurtenant facilities were built at a location near railroad tunnel No. 1.

The original reconnaissance for relocation of the railroad was performed during the early 1950s, with the final location survey being completed in 1955.

An agreement with Western Pacific Railroad Company established the criteria for design. The design was based on then-current railroad design data. These data called for a maximum grade of 1%. The grade was reduced at curves to compensate for curve resistance and at bridges and tunnels. The maximum degree of curve was 4°30′. Cut sections were constructed to a 40-foot minimum width and embankment sections were constructed to a 26-foot minimum width with an additional 1 foot of width provided for each 15 feet of fill over the initial 15 feet. Both cut and embankment sections were widened 15 feet for passing track and an additional 14 feet for storage track where required. All culverts were sized by the use of Talbot's formula, using a 4-inch rainfall and a C factor suitable to the area drained.

Tunnels. The length of the five railroad tunnels measured between portal faces is as follows: (1) 2,410 feet, (2) 2,923 feet, (3) 2,583 feet, (4) 4,407 feet, and (5) 8,825 feet. Due to the long lengths of tunnels Nos. 4 and 5, ventilating chambers were incorporated into the design. The typical portal wall, at each end of the tunnels, is a 2-foot - 6-inch-thick, reinforced-concrete, portal structure flanked by retaining walls at varying angles. The retaining walls are of different heights and lengths to fit the topography at each location.

The geometry of the tunnel cross section is typical for all five tunnels. The wall section above the springline is circular with a radius of 9 feet, which makes the clear width of the tunnel 18 feet. The height from the top of the rail to the springline measures 15 feet.

Eight-inch, wide-flange, steel ribs fitting the tunnel geometry were encased in the concrete wall along with other reinforcement, e.g., rock bolts with chain-link fabric, steel liner plates, and lagging materials as needed to support the various excavation conditions.

Concrete placement of the tunnel lining was performed pneumatically. The lining was constructed with contraction joints spaced at 40 feet maximum on centers. Four-inch formed drains were installed along each tunnel spaced at 8-foot intervals both vertically and horizontally. Refuge niches, car setoff niches, and

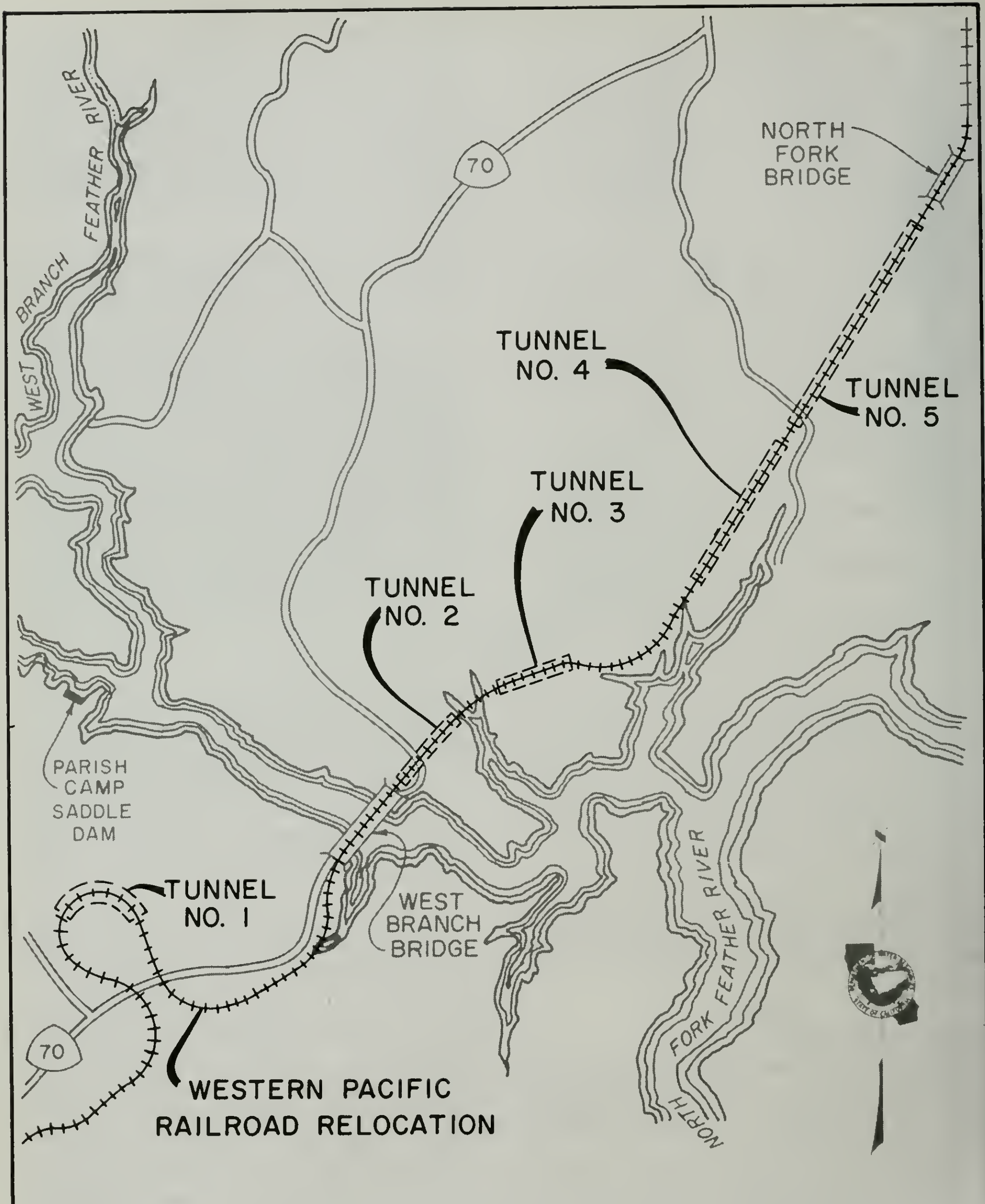

Figure 87. Western Pacific Railroad Relocation—Tunnel Locations

other safety and operating openings were provided along each tunnel.

The design and construction supervision was done by the Department except for track laying. Western Pacific Railroad approved all design drawings prior to award of construction contracts and laid the track.

Bridges. The three major bridges built in conjunction with the relocation of the railroad are: (1) Feather River Bridge, (2) a combination highway-railroad bridge across the West Branch Feather River, and (3) North Fork Bridge. In addition to the above, a short-span arch bridge was constructed across Dark Canyon, and many single-lane underpasses were built to provide access to property on both sides of the railroad.

All design was based on a Cooper E-72 live load with 0.1g earthquake loading.

Feather River Bridge. The crossing of the Feather River approximately 1 mile north of the City of Oroville was designated the Feather River Bridge (Figure 88). This bridge was designed by an engineering consultant to the Western Pacific Railroad Company using 1956 American Railway Engineering Association (AREA) specifications. Horizontal alignment of the Bridge is on a 3°00′ curve with a track grade of 0.88% ascending eastward.

A riveted-deck plate-girder structure was selected as the most economical design for this bridge. The centerline of track span lengths, center to center of piers, vary from 90 to 128 feet, for a total length of bridge of 1,012 feet in ten spans. Two plate girders were installed on 8-foot centers to support the 16-foot-wide, lightweight-concrete, ballast trough. The girders vary in length from 125 to 88 feet and their depths vary from 9 to 10 feet.

Supporting this superstructure are reinforced-concrete cylindrical piers founded on spread footings placed into bedrock, which vary in height from 44 to 77 feet. The top 44 feet of the cylindrical piers are 10 feet in diameter. They are 12 feet in diameter for the remainder of their length. Piers for a future track were installed 18½ feet upstream of the new bridge and on the same footings. These piers rise above the maximum normal water surface.

Principal considerations used in the design of the piers, due to their almost complete submergence in the Thermalito diversion pool, were: (1) mass to be as small as practicable, (2) form to require the acceleration during earthquakes of the least practicable amount of water, (3) superstructure square rather than skewed, (4) least practicable interference with and resistance to streamflow, and (5) as much duplication of forms as possible.

The consultant's plans and specifications were incorporated into contract documents, the work was advertised, and construction was supervised by the Department of Water Resources.

West Branch Bridge. The relocations of the Western Pacific Railroad and U.S. Highway 40A (now State Highway 70) converged at a common location to cross the West Branch Feather River. This crossing is located approximately 2 miles west of the confluence of the West Branch and the North Fork of the Feather River, 11 miles northeast of the City of Oroville. A combination highway-railroad bridge was the logical choice.

The West Branch Bridge (Figure 89) is a riveted-steel, cantilevered, Warren truss with verticals. The single-track railroad is on an 18-foot-wide, lightweight-concrete, ballast trough supported by steel

Figure 88. Feather River Bridge

Figure 89. West Branch Bridge

stringers and floor beams attached to the lower chord of the truss. The highway is four lanes on a 56-foot-wide concrete roadway supported by steel stringers and floor beams on the top chord of the truss.

The truss-span layout includes a 360-foot approach truss, two 432-foot anchor spans, and a 576-foot main span consisting of a 360-foot suspended span and two 108-foot cantilevers. In addition, there are 10 highway, welded-steel-plate-girder, approach spans. Six spans (79 feet, three at 80 feet, 135 feet, and 79 feet - 3 inches) are located on the south end and four spans (133 feet, two at 80 feet, and 79 feet - 3 inches) are located to the north. Total length of the structure is 2,731 feet.

The three main piers supporting the truss sections are 216 feet, 252 feet, and 257 feet in height. They are founded on spread footings placed against undisturbed rock excavated to near-vertical lines. The other piers and abutments are reinforced concrete on spread footings.

The Bridge Department of the Division of Highways, California Department of Public Works (now the Department of Transportation), designed and supervised construction of the Bridge. The contract documents were reviewed and approved by the Department of Water Resources and the Western Pacific Railroad Company.

Conventional construction methods were used in the pier-footing excavations and in constructing the piers themselves, except for the main piers. These piers were constructed using the slip form method of concrete placement.

Truss erection started at the southwest end of the Bridge (nearest Oroville) and proceeded toward the middle of the main span. Then, at the northeast end, erection proceeded until it connected with the already erected portion of the main span.

North Fork Bridge. In order for the relocated Western Pacific Railroad tracks to join the existing tracks at the North Fork of the Feather River, a bridge had to be constructed across the River (Figure 90).

The agreement between the Department of Water Resources and the Western Pacific Railroad Company required the railroad to design this bridge. The agreement called for either a cantilever truss structure or a reinforced-concrete arch bridge. A consultant for the railroad prepared contract documents for both proposals. These contract documents were furnished to the Department, who subsequently called for bids on both proposals. The bid for the reinforced-concrete arch bridge was the least costly and was selected for construction. The Department supervised the construction.

The reinforced-concrete arch structure has arch spans of 191.5, 308, and 257.5 feet, six east approach spans 22.5 to 28 feet long, and abutments for a total length of 1,010.5 feet. The 119-foot-high concrete arches vary in width from 20 to 30 feet. They are founded on concrete pedestals placed on bedrock.

Figure 90. North Fork Bridge

The single-track railroad is on 20-foot-wide, concrete, ballast trough with a parabolic soffit. The ballast trough is supported by walls rising from the arch. These walls vary in thickness from 9 feet at the piers to 2 feet at mid-span. They vary in height from 130 feet at the piers to approximately 5 feet at mid-span.

Feather River Railway

The Feather River Railway extended from the town of Feather Falls above the South Fork arm of Lake Oroville to its junction with the Western Pacific Railroad in the Feather River Canyon 2 miles upstream of Oroville Dam. As explained in Volume VI of this bulletin, the lumber formerly carried by the railroad is now trucked in accordance with a settlement between the Department and the owners.

Oroville-Quincy Road

The relocated Oroville-Quincy Road begins approximately 5 miles due east of the City of Oroville and extends 8.1 miles north to the original Oroville-Quincy Road (Figure 55). The north and south reaches of the new road are separated by an arm of the reservoir that is spanned by the Bidwell Bar Bridge. The general location of the entire road was dictated in part by the site selected for Bidwell Bar Bridge, which was built at the only feasible site for a major bridge in that area.

The portion of the old Oroville-Quincy Road now inundated by Lake Oroville originally crossed the Feather River Canyon at Bidwell Bar and continued generally north, rising from the canyon west of Mount Ratchel.

Forbestown Road to Bidwell Bar Bridge. From its southern end at Forbestown Road, the relocated route extends 3.8 miles north to Bidwell Bar Bridge. Location of its beginning was influenced by the location of the Miners Ranch Reservoir to the west. Beginning at elevation 850± feet, the Road rises to elevation 2,156 feet at its highest point. It joins the south approach to Bidwell Bar Bridge at elevation 950 feet. For part of its length, the Road follows the eastern limit of the Loafer Creek recreation area. Cross slopes range from moderate to steep. Several alignments for the southern reach were studied before the present alignment was accepted. Its precise location was the result of negotiations between Butte County and the Department and also reflected the wishes of the Department of Parks and Recreation for a road in the vicinity of the Loafer Creek recreation area. Subsequent to completion of the Road, Butte County relocated a portion of the southerly end of the Road so that it aligned with Olive Highway.

Inasmuch as this was a relocation of a Federal-Aid Secondary Highway with an average daily traffic count exceeding 400 vehicles, the criteria chosen are those appropriate for a road with an average daily travel of 400 to 1,000 vehicles in rolling or mountainous terrain.

Drainage structures were designed to carry runoff from 100-year storms as determined by the procedure defined in the "California Culvert Practice" of the Division of Highways. The calculations to determine the runoff were based on a 100-year record storm.

Bidwell Bar Bridge to Original County Road. The northern section of Oroville-Quincy Road extends 3.9 miles north from Bidwell Bar Bridge, following the configuration of the ground above Oroville Reservoir at elevations from 945 to 1,060 feet. The road crosses Canyon Creek on a 780-foot-long bridge and continues along the south side of Canyon Creek. It connects with the original county road about 1 mile northwest of the Canyon Creek crossing. Cross slopes for almost the entire length of the Road are extremely steep.

Butte County was responsible for design and construction of this part of the road relocation. The County contracted with its consultant, Porter, O'Brien and Armstrong Engineers, to prepare the plans and specifications for this portion of the Road. Both the Department of Water Resources and Butte County agreed on the design standards used. Total cost to the Department for design and construction work was not to exceed $4,900,000. The Department retained the responsibility for acquisition of right of way.

Roadway fills extending below the high water elevation of the reservoir were designed to resist the erosive action of 3-foot waves. The compacted embankment was protected by a 15-foot zone of pervious material and a 3-foot zone of rock slope protection.

The compacted embankment consisted of material with a gradation of 85% passing a ½-inch sieve, 50% passing a No. 8 sieve, and 10% passing a No. 70 sieve. This material, frequently found in the roadway excavation after overburden was stripped, was designed to give the compacted embankment below elevation 905 feet the free-draining characteristic needed for a reservoir drawdown rate of 0.0038 of a foot per minute. Pervious material selected or processed from roadway excavation, with no more than 10% passing a No. 4 sieve, was utilized below elevation 911 feet to act as filter material between the compacted embankment and rock slope protection.

Because of the steep transverse slopes where the road alignment is adjacent to the reservoir, viaducts or steep embankment supported by retaining walls were required. Following consideration of the viaducts and the many types of retaining walls, metal bin-type walls were selected.

Transverse members of the metal bin walls are spaced 10 feet apart, tying the front and rear walls of the bins. The walls are made up of modules 1.33 feet high. The bins were predesigned by the manufacturer for walls of six heights with corresponding bin widths. Base plates serve to establish the wall at proper line, grade, and batter. To facilitate adjustments, 8 inches minimum of loose material was placed below each footing. Native material was satisfactory for backfilling and was compacted in 1-foot layers. The bins were set on a 1:6 batter.

To provide adequate road width at gullies crossing the roadway, it was necessary to build concrete foundation walls on which the metal bin walls were placed. These concrete foundation walls were anchored to bedrock by dowels grouted into drilled holes in the bedrock.

Bidwell Bar Bridge. Bidwell Bar Bridge (Figure 91), the key feature of the Oroville-Quincy county road relocation, crosses the Middle Fork arm of Lake Oroville about 1 mile upstream from the former crossing at Bidwell Bar. The inundated Bidwell Bar crossing was the location of the historic Bidwell Bar suspension bridge, said to be one of the first suspension bridges erected in the West. The historical bridge was removed and is to be reassembled at a new site at the Kelly Ridge recreation area. The Bidwell Bar Bridge originally was designated the "Middle Fork Feather River Bridge". After construction, it was renamed after the geographical and historical site inundated by Oroville Reservoir.

The Department of Water Resources designed, and was the contracting agency for, the Bridge.

Selection studies for the Bridge were based upon an investigation of several types of existing structures. Types other than cantilever or suspension bridges were quickly eliminated due to site conditions, span length requirements, and economics.

Studies for a cantilever bridge emphasized the difficulties of fitting an economical structure to the steep slopes at the selected site. Two piers over 300 feet in height would have been necessary to support the bridge. Even with piers of such height, the balance between anchor, cantilever, and approach span was

Figure 91. Bidwell Bar Bridge

such that considerable uplift would have resulted at the abutment. The steep side slopes also would have required massive roadway excavations to make the near-right-angle turns in the approach roadway onto and off the bridge.

Preliminary studies in the design of the suspension bridge were made using the basic design approaches presented in "A Practical Treatise in Suspension Bridges" by D. B. Steinman.

The economics of locating the piers as far shoreward as possible was apparent. In addition to allowing the use of shorter piers, this geometry allowed the use of unloaded rather than loaded backstays. Use of unloaded backstays was necessitated by the requirement to use curved roadway approaches for smooth access to and from the Bridge.

It was apparent from preliminary cost estimates that a suspension bridge was the obvious choice. Aesthetical characteristics of the Bridge are an advantage which has been gained at no cost.

The main cables are covered with a plastic cable covering developed by the Bethlehem Steel Company. This bridge is the first to use a cable protection system other than the painted wire wrapping introduced by John A. Roebling in 1845. An obvious advantage in using the plastic (glass-reinforced acrylic) cable covering is found in comparing its estimated 50-year life without maintenance to the two- to three-year repainting cycle required by the painted wire-wrapping system.

The main cables are anchored in tunnels by utilizing prestressing tendons. This bridge is the first major structure to use this method of anchorage. The prestressed tunnel anchorage was the most economical type that could be used. Plain details, smaller members, and easier access resulted in a small tunnel requiring less excavation and concrete than would have been necessary with others. Another advantage offered by the prestressed anchorage is that the entire concrete tunnel plug is compressed. Consequently, there was no elastic elongation of the anchorage during the spinning of the cables, and an accurate zero point of movement for connecting the cable strands to the anchorage was available.

Canyon Creek Bridge. Canyon Creek Bridge spans the arm of Oroville Reservoir 2.75 miles north of Bidwell Bar Bridge and immediately south of the former confluence of Canyon Creek and its east fork. Studies indicated that the most economical structure for the Canyon Creek crossing would be a steel-plate-girder bridge. The crossing selected required a structure 780 feet long. The Bridge has four spans: 195 feet, 250 feet, 196 feet, and 134 feet in length. It was designed for two lanes of traffic with H20-S16-44 live loading (AASHO).

Pier footings were designed for direct bearing on sound rock. Excavation was planned to reach about 35 feet below original ground. Pier footings, 8 feet by 24 feet, were anchored to the hillside by 1¼-inch rock bolts. Piers 2 and 3 contained three horizontal bolts, three bolts at 15 degrees with the horizontal, and three at 30 degrees with the horizontal. A reinforced-concrete bench was installed at ground level to absorb the horizontal reaction of the tension bolts, thus avoiding a horizontal thrust on the base of the piers. In maintaining a length-to-depth ratio of not greater than 20, the outside dimensions of Piers 2 and 3 were 8 feet by 24 feet. Each pier consisted of a hollow interior shaft with dimensions of 16 feet by 5 feet - 4 inches. External vertical edges of the piers were rounded on a 4-foot radius.

Canyon walls rising 400 to 600 feet above Canyon Creek Bridge generally shield the Bridge from any high-velocity winds striking it in the normal direction. The only direct approach for the wind is from the southwest. Prevailing winds in the locality blow from either the southeast or the northwest, reaching a maximum velocity of 90 miles per hour. A wind load of 32 pounds per square foot was used in the design of the Bridge.

Seismic loading was based on the assumed period of vibration of historical earthquakes in the area. For Canyon Creek Bridge, this period was estimated to be 3.3 seconds. On this basis, as outlined in the Division of Highways' "Bridge Planning and Design Manual", the seismic force used to design the Bridge was .031 times the dead load. The dead load of the submerged portion of piers included the weight of a column of water 24 feet in diameter.

The reinforced-concrete deck is supported by two parallel steel girders spaced 20 feet apart. The deck, 34 feet wide and 10½ inches thick, cantilevers 7 feet from each side of the girders. A 30% impact load was used in the design of the deck.

The steel girders were designed as continuous members over Piers 2 and 3 and simply supported over Pier 4 and at the abutments. Expansion bearing assemblies at both abutments and at the stream channel side of Pier 4 allow for longitudinal movement of the girders. Fixed bearings support the girders at Piers 2 and 3 and at the bank side of Pier 4.

Interior cross frames were located as required at 25-foot maximum spacing to give lateral support to the girders. The frames were bolted or welded to the web stiffeners 6 inches in from the flanges. Diagonal lacing is used in the plane of the bottom chord of the frames. An inspection catwalk supported on the lower chord of the cross frames runs the full length of the Bridge.

Oroville-Feather Falls Road

The Oroville-Feather Falls county road relocation (Figure 55) is a 10-mile-long replacement for a portion of Lumpkin Road, inundated by Lake Oroville near Enterprise. It is located on the South Fork of the Feather River approximately 10 miles east of Oroville. The new alignment was based on least cost and location was governed primarily by the type and length of the bridge crossing. The structure ultimately was located 1⅜ miles upstream from the old road. Adoption of the route was effected through an agreement with Butte County which also established design criteria and provided for concurrent review of the design by the County. Although the character of the former road was extremely poor, the replacement design criteria were based on current design standards. The realignment was extremely advantageous from the standpoints of design, right of way, traffic interruption, and service.

The agreement with Butte County provided for passing lanes to compensate for the increased amount of truck traffic required to transport the lumber products of the Feather River Pine Mills that were formerly conveyed by railroad (see Feather Falls Railway).

The project was divided into two contracts because of the timing involved in both design and construction. The first contract included a portion of the road south of the B. Abbott Goldberg Bridge, and all of the road north of the Bridge, to the existing road. The second contract included that portion of the road from the beginning of the relocation to the first contract.

In terms of overall service, the relocation is 2½ miles longer than the original road.

B. Abbott Goldberg Bridge. Adequate bridge sites were limited due to the steep character of the terrain. The bridge site selected resulted from the best economic compromise between a narrow bridge crossing and the steep topography for the road alignment. A suspension bridge, a deck truss bridge, and a welded-plate-girder bridge were considered. No other types were appropriate for this site. The suspension bridge was eliminated due to the questionable characteristics of the decomposed granite in the anchorage areas.

A program for foundation exploration and testing revealed a deep layer of decomposed granite near the ends of the Bridge. Due to the instability of this material, it was more economical to span these areas. Span lengths necessary to avoid these critical end areas exceeded the economic limit for the plate-girder alternative; therefore, it was abandoned in favor of the deck truss design (Figure 92).

Bridge design was in accordance with the American Association of State Highway Officials "Standard Specifications for Highway Bridges", dated 1961, as supplemented by the State of California "Bridge Planning and Design Manual". Live load used was the H20-S16-44 AASHO loading and an alternative loading of two 24,000-pound axles 4 feet apart. A 15% impact factor was used for the truss design, and a 30% impact factor was used for the deck and approach spans.

Pier foundations required special consideration. The decomposed granite overburden is approximately 50 feet deep at the pier locations with the canyon slope at roughly 30 degrees from horizontal. Any yielding or movement in the slope at the piers will produce a

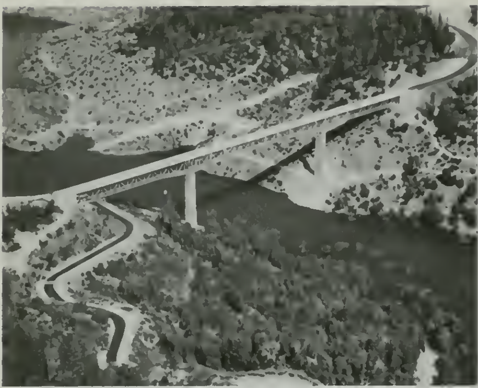

Figure 92. B. Abbott Goldberg Bridge

lateral force that must be resisted by the foundation. The design force against the pier from a potential slide in an active state was based on an effective wedge of soil that would be retained by the pier. An anchor shaft foundation, well socketed into fresh granite, was chosen as the most acceptable means of withstanding the large soil loads imposed on the piers. Closed-end cellular-type abutments were used to reduce the earth surcharge on the critical soil.

Interaction between the main truss members and the lateral bracing members was investigated using criteria recommended in the text "Modern Framed Structures" by Johnson, Bryan, and Turneaure. Participating stresses that would be developed were found to be quite significant.

Analysis of the truss also considered the loading conditions that would exist during various stages and types of erection. These calculations determined that eight members required selection based on erection stresses governing over other loading conditions.

A deck Warren truss with two side spans of 308 feet and a center span of 440 feet, for a total length of 1,056 feet, was selected for the superstructure. The superstructure uses two component trusses spaced at 20-foot centers. They are 24 feet in depth (center to center between the top and bottom cord) and are haunched to 36 feet in depth at the piers. All truss panel points are spaced at 22-foot intervals. The bridge roadway is 34 feet wide, consisting of two 14-foot traffic lanes plus a 2-foot safety curb and barrier railing on each side.

The truss expansion bearings and the slab roadway approaches are supported on the abutments. Fixed bearings for the truss are placed over the piers. Piers 2 and 3 are 12- by 36-foot hollow-shaft construction for the top 100 feet above the ground, changing to a 14- by 28-foot section for the remaining exposed portion, with approximately a 24-foot by 32-foot solid section below the ground. Pier 2 extends 149 feet above the ground surface and 102 feet below, while Pier 3 extends 140 feet above the ground surface and 75 feet below. Both pier shafts are anchored into rock.

U.S. Highway 40A

As discussed in Volume VI of this bulletin, the 21 miles of U.S. Highway 40A (now State Highway 70) that parallel the Western Pacific Railroad through the Feather River Canyon were relocated by the Division of Highways. Previously, it has been mentioned that this relocation shares a bridge with the railroad at the West Branch arm of the reservoir.

Nelson Bar County Road

Bennum and Lunt Roads modifications were necessary to improve traffic access across the West Branch arm of Oroville Reservoir and replace traffic patterns provided by the old Nelson Bar Bridge.

Bennum Road. Bennun Road modifications begin at Highway 70 near the northwest corner of Section 29, T.21N., R.4E. and extend northwest for approximately 1.65 miles along the west side of the West Branch arm of the reservoir to connect with the Pentz Magalia Road (a paved road connecting Highway 70 and Paradise). A channelized intersection on Highway 70 was incorporated with this facility to provide safety for vehicles turning left into Bennum Road.

Lunt Road. Lunt Road modifications begin at Highway 70 near the center of Section 4, T.21N., R4E. and extend northwest 0.65 of a mile to connect with the Nelson Bar County Road (a paved road serving the area in the vicinity of Concow School).

The above modifications to Bennum and Lunt Roads provide ready access via paved road from one side of the reservoir to the other in the vicinity of the West Branch arm, utilizing the West Branch Bridge and Highway 70 (relocated Highway 40A).

Bennum and Lunt Road modifications were designed and constructed by the Department. The channelization on Highway 70 at the intersection with Bennum Road was designed by the Division of Highways, incorporated into the Department's plans, and constructed at its expense. On completion, Bennum and Lunt Road modificiations were incorporated into the road system of Butte County.

U.S. Forest Service Roads

U.S. Forest Service roads to provide access for fire suppression and recreational use in the vicinity of the upper end of Oroville Reservoir on the North Fork of the Feather River were lengthened and improved.

Construction

Contract Administration

General information about the major contracts for the construction of Oroville Dam and appurtenances is shown in Table 9. Other contracts relating to the underground powerplant in the left abutment are discussed in Volume IV of this bulletin.

TABLE 9. Major Contracts—Oroville Dam and Appurtenances

	Specification	Low bid amount	Final contract cost	Total cost-change orders	Starting date	Completion date	Prime contractor
Tunnels Nos. 4 and 5, Western Pacific Railroad Relocation	57-03	$8,499,235	$10,442,272	$6,378	5/24/57	12/30/60	Peter Kiewit Sons' Co.
Bridge Over North Fork Feather River, Western Pacific Railroad Relocation	57-13	1,538,660	1,580,940	5,971	2/10/58	6/29/60	Pacific Bridge Co.
Bridge Over Feather River at Oroville, Western Pacific Railroad Relocation	58-01	1,169,265	1,293,159	50,068	4/29/58	3/16/60	John C. Gist
Roadway Structures and Tunnels Nos. 2 and 3, Western Pacific Railroad Relocation	59-16	5,720,320	6,255,568	155,824	8/11/59	9/ 8/61	Ball & Simpson, Inc.
Tunnel No. 1, Western Pacific Railroad Relocation	60-07	2,069,090	1,948,032	16,228	8/ 1/60	11/ 3/61	Frazier-Davis Construction Co.
Roadway and Structures, Oroville to West Branch Feather River, Western Pacific Railroad Relocation	60-08	3,612,652	4,074,149	153,271	9/15/60	4/20/62	Ball & Simpson, Inc.
Diversion Tunnel No. 1	61-05	6,193,685	7,675,552	407,498	8/18/61	12/27/63	Frazier-Davis Construction Co.
Palermo Outlet Works	61-15	724,261	801,118	23,910	11/16/61	5/17/63	Morrison-Knudsen Co.
Section Headquarters at Siding No. 3, Western Pacific Railroad Relocation	61-17	164,997	168,324	3,727	12/11/61	7/13/62	El Rey Builders, Inc.
Oroville Dam	62-05	120,863,333	135,336,156	7,503,716	8/13/62	4/26/68	Oro Dam Constructors
Oroville Seismograph Station	62-11	31,226	39,469	742	7/19/62	10/26/62	El Rey Builders, Inc.
Left Abutment Access Road	62-13	452,388	549,400	65,742	7/14/62	4/ 9/63	Piombo Construction Co.
Oroville Construction Headquarters	62-27	938,000	989,965	46,964	11/16/62	10/25/63	A. Teichert & Son
Middle Fork, Feather River Bridge	62-30	4,436,104	4,859,714	96,006	2/11/63	8/11/65	Bethlehem Steel Co.
Employee Housing	63-04	659,095	618,543	--	7/ 9/63	4/27/64	Nomellini Construction Co.
Oroville-Quincy Road Relocation, Oroville-Forbestown Road to Middle Fork Bridge	63-35	1,053,196	1,521,868	359,825	1/ 3/64	8/25/65	Piombo Construction Co.
Construction Overlook Modifications	63-38	138,483	138,196	1,063	12/18/63	5/15/64	A. Teichert & Son
Pioneer Cemetery and Grave Relocation	64-17	78,918	70,314	903	6/11/64	9/ 2/64	Frank P. Donovan
Headquarters and Employee Housing Landscaping	64-30	37,043	40,982	2,046	7/24/64	9/22/64	Frank M. Smith
Thermalito Power Canal Relocations	64-31	935,480	963,647	12,111	10/30/64	11/ 5/65	Osborn Construction Co.
Bidwell Bar Emergency Crossing	64-44	34,098	31,189	--	9/28/64	11/11/64	Frank P. Donovan
Temporary Access Road, Middle Fork Bridge North to County Road	64-51	327,904	379,909	8,303	12/ 3/64	6/ 5/65	Crooks Bros. Construction Co.
Clearing Oroville Reservoir Site	65-05	3,515,970	3,594,943	187,433	4/12/65	5/20/67	C. J. Langenfelder & Son, Inc.
Oroville Dam Spillway	65-09	12,249,850	13,702,871	615,781	6/25/65	2/20/68	Oro Pacific Constructors & George Farnsworth Construction Co.

TABLE 9. Major Contracts—Oroville Dam and Appurtenances—Continued

	Specification	Low bid amount	Final contract cost	Total cost-change orders	Starting date	Completion date	Prime contractor
Oroville-Feather Falls Road Relocation	65-23	$2,347,401	$2,565,225	$119,344	12/23/65	9/20/67	O. K. Mittry & Sons
Oroville Feather Falls Road Relocation, South Fork Feather River Bridge and Roadway, Station 422+50 to Lumpkin Road	65-26	2,773,236	2,919,468	152,926	8/10/65	1/11/68	Rothschild, Raffin & Weirick, Inc. & Piombo Construction Co.
Construction Overlook Relocation	65-59	123,887	123,657	863	1/19/66	6/17/66	Baldwin Contracting Company, Inc.
Bulkhead Gates for Diversion Tunnel No. 1	66-05	67,000	69,747	39	1/28/66	7/30/66	Berkeley Steel Construction Co., Inc.
West Branch County Road Modifications, Bennum and Lunt Roads	66-27	341,220	371,168	11,512	7/ 9/66	2/ 3/67	A. Teichert & Son, Inc.
Oroville Operations and Maintenance Center—Thermalito Annex	66-41	194,296	194,024	1,451	11/ 4/66	9/29/67	Baldwin Contracting Co., Inc.
Oroville Peripheral Dams	66-42	373,066	499,420	68,353	12/ 9/66	10/ 3/67	Harms Brothers
Oroville Operations and Maintenance Center	66-52	1,651,406	1,708,603	52,393	1/23/67	4/ 4/68	Christensen & Foster
Oroville Completion Contract No. 1	67-43	112,680	110,131	2,469	9/15/67	7/20/68	George R. Osborn Construction Co.
Butte County Road Relocation, Glen Drive Improvement	67-44	93,394	119,120	3,932	8/31/67	12/18/67	A. Teichert & Son, Inc.
Oroville Reservoir Service Area Boat Ramp	67-53	104,331	109,895	473	9/20/67	12/11/67	W. H. Lindeman & Sons, Inc.
Oroville Dam Crest Improvements	69-07	394,628	423,391	−9,965	3/10/69	7/21/69	Oman Construction Co., Inc.
Oroville Completion Contract No. 3	69-19	249,765	278,959	28,933	8/14/69	5/ 1/70	A. Teichert & Son, Inc.
Oroville Division Landscaping	69-34	66,655	69,521	1,967	1/ 8/70	3/24/70	Economy Garden Supply
Oroville Dam Spillway Fencing	71-20	14,952	14,947	--	9/ 3/71	10/13/71	Dalzell Corp.

Early Contracts

Relocations. The earliest contracts at Oroville were for relocations. Construction aspects of the relocations can be found with the design discussion. Costs are, however, contained in Table 9.

Diversion Tunnel No. 1. The contractor established an equipment and maintenance area on the right bank of the Feather River opposite the intake portal for the 4,400-foot-long 35-foot-diameter tunnel and erected a Bailey bridge to provide access to the work area. Later, another Bailey bridge was erected across the River to gain access to the work area at the downstream portal.

Open-cut excavation was begun at the intake portal on October 2, 1961 and, on November 14, 1961, a 12-inch-wide crack developed over and nearly perpendicular to the centerline of the tunnel. The partially completed portal collapsed shortly after the crack developed.

Twenty-five-foot rock bolts on 5-foot centers were installed on the left wall to increase the stability of the slope. One-inch-diameter slot and wedge-type rock bolts 20 and 25 feet long were installed on a 5-foot-square pattern and torqued to 250 foot-pounds. The rock above the portal was grouted, and most of the muck pile was left in place to buttress the portal slope while reinforcement of the rock progressed. Thirty-foot crown bars (No. 18 rebars) were installed in holes drilled on 12-inch centers over the arch of the portal face and grouted in place. Arch ribs were installed on wall plates to form an umbrella which was then lagged solid with timber. Timber cribbing was placed 30 feet up each side of the umbrella and backfilled with tunnel muck. The umbrella then was covered with approximately 1,800 cubic yards of well-graded dredger tailings which were allowed to assume their natural angle of repose (Figure 93). Tunnel excavation resumed on January 9, 1962. Open-cut excavation at the outlet portal began on July 10, 1962 in hard, strongly jointed, fresh to slightly weathered amphibolite. No problems were encountered with this portal.

A 38-foot section of the tunnel near the downstream portal was left unexcavated through the 1962–63 rainy season to prevent flooding of the tunnel from the downstream end. However, on October 13, 1962, a flood with a peak flow of 136,000 cfs overtopped the upstream training levee and flooded the tunnel from that end as well as taking out the downstream Bailey bridge. This flood occurred a month earlier than the start of the historic flood season. A flood with an even larger peak, 191,000 cfs, inundated the work and damaged the replaced bridge on January 31, 1963.

Rock conditions encountered were considered fair to good for excavation of a large bore. Problems with excavation and support were greatly minimized because the tunnel crossed geologic structures at nearly right angles to strike.

The contractor used the top heading method of driving the circular tunnel, excavating initially as a horseshoe section 28 feet high and 40 feet wide at springline. A 10-foot-square exploratory crown drift was driven 20 to 25 feet ahead of the main heading for the first 100 feet of tunnel from the upstream portal.

The tunnel was driven using diesel-powered rubber-tired equipment. Drilling of the top heading was done from two three-deck jumbos on truck chassis. The bench was removed for the entire length of the tunnel after excavation of the top heading had been completed. The average rate of advance for the top heading was 20.5 feet per 24-hour 3-shift day, while the average rate of advance for the bench was 83.2 feet. The tunnel was supported throughout with W12X58 (12WF58) four-piece ribs spaced from 1.5 to 6 feet on centers. Ground water was not a problem. The sum of all estimated initial flows into the tunnel during excavation amounted to 115 gallons per minute.

Figure 93. Diversion Tunnel No. 1 Intake Portal After Backfilling

Openings 5 to 10 feet long were excavated, rock-bolted, and concrete-lined for future connections to draft tubes Nos. 3 through 6. Concrete bulkheads then were installed.

A 20-foot-diameter 10-foot-high opening was excavated in the tunnel arch and rock-bolted prior to excavation of the 10-foot-diameter, 38-foot-high, vertical rise for later connection to the equalizing tunnel. This excavation was lined and a concrete bulkhead installed.

The tunnel lining reinforcement was modified in the future tunnel plug reach so that some of the concrete lining could be removed to key the plug.

Concrete was mixed in an on-site batch plant, transported by a variety of methods, and placed in four steps. A subinvert was placed to fill overbreak, make a good working surface, and hold steel arch forms. Curbs were placed on the subinvert to tie down the bottom of the arch form. The arch then was placed and finally the invert. The 48-foot-long arch form was mounted on a jumbo which moved on rails placed on the subinvert. Static forms were used for the invert. Pumpcretes and slicklines conveyed the arch concrete and belts conveyed the invert concrete.

The grouting program for the tunnel included envelope, contact, and consolidation grouting. High-pressure envelope grouting was done in the tunnel plug and draft-tube sections before lining the tunnel with concrete, as discussed later in this chapter. Contact grouting was done throughout the length of the tunnel after tunnel lining was completed. Consolidation grouting of the tunnel plug and a section at the inlet portal followed contact grouting.

Construction of the tunnel for the diversion mode was completed and water turned into the tunnel in November 1963.

Palermo Outlet Works. Initial work on the 2,430-foot-long 6-foot-diameter tunnel consisted of diverting drainage at the intake and outlet portals.

The upstream portal was cut in moderately to strongly weathered, moderately hard, blocky amphibolite and deep soil cover at slopes of 1½:1, and 2:1 above the bench. Because of the blocky nature of the rock at the intake portal, drill steel was installed to serve as crown bars to support the arch.

Downstream portal excavation encountered fresh to slightly weathered, hard, strongly jointed amphibolite with a thin soil cover. The cut was excavated at a slope of $1\frac{1}{2}$:1.

Both upstream and downstream portal cuts were excavated with a ripper-equipped tractor with only a minimum amount of blasting required to loosen large blocks.

Rock conditions encountered in the tunnel were considered good for normal tunneling operations in a small bore. The amphibolite was generally fresh, hard, moderately blocky and jointed,with support required at each portal and in only three zones of sheared weathered rock within the tunnel. Where support was required, W4X13 steel beams were used for 17% of the length of tunnel.

The tunnel was driven using rail-mounted equipment on a 24-inch-gauge track. Three hydraulically operated drifters were mounted on a jumbo for drilling the face. Mucking was accomplished with an air-operated mucking machine modified by shortening the bucket arms for operation in a small bore. Two muck trains were used for hauling the muck. Each train was composed of six 30-cubic-foot side-dump cars powered by a 55-horsepower, 6-ton, diesel locomotive.

The valve chamber and tunnel plug are located at the grout curtain for Oroville Dam tunnel Station 24+08 (Figure 69). The valve chamber was moved 25 feet upstream from the design location primarily to take advantage of better rock conditions but also to locate the tunnel plug and grout curtain coincident with the grout curtain for the Dam. (Construction of Palermo outlet started before design of the Dam was finalized.)

Curtain grouting was done from the tunnel to reduce or eliminate the need for deep high-pressure grouting adjacent to Palermo outlet works during curtain grouting on the main dam and to reduce reservoir pressure on the concrete lining of the tunnel downstream from the tunnel plug. To further reduce hydrostatic pressure on the concrete lining, drain holes were drilled along the tunnel downstream from the valve chamber. The curtain consisted of a single ring of EX grout holes oriented radially at intervals of $22\frac{1}{2}$ degrees in a plane that is transverse to the tunnel alignment and that dips 75 degrees upstream. Holes varied from 60 to 130 feet in length in order to reach elevations above and below the tunnel corresponding to the estimated upper and lower limits of the proposed grout curtain for the Dam.

Initial Dam Construction Activities

The prime contractor for Oroville Dam (including Diversion Tunnel No. 2 and Thermalito Diversion Dam) was notified to proceed with the contract work on August 13, 1962 and began preconstruction activities during that week. The first work performed, under the largest (in dollars) civil works contract up to that time, was the building of an access road to the outlet portal of Diversion Tunnel No. 2 and the construction of the piers for a Bailey bridge.

By the end of the month, the contractor's survey crews were staking out the locations for the conveyor belts to handle embankment materials.

Clearing of the Dam site was begun in September 1962. By November 1962, the clearing crew, equipped with ten logging dozers and chain saws, had completed the operation. Most of the area was covered by scrub oak, digger pine, and manzanita.

There were 31 exploration holes from 82 to 145 feet deep and five exploration tunnels in the dam foundation. The exploration holes which generally were under the core were filled with grout. Parts of the exploration tunnels under the core and transition zones were backfilled with concrete. In addition, the portals of all five tunnels were filled with embankment materials.

The Western Pacific Railroad had been relocated out of the reservoir area prior to the start of construction. Approximately 1.06 miles of abandoned track was removed and used in the contractor's rail facilities, and an interconnection between the relocated Western Pacific Railroad and the original line was constructed for an interchange yard.

Diversion of River and Dewatering of Foundation

The diversion of the Feather River and dewatering of the foundation were carried out in four stages.

For Stage 1 (Figure 94), the River was allowed to remain in its natural channel while the lower lifts of blocks 1 through 7 and 9 of the core block were being placed.

Stage 2 (Figure 95) was started on July 18, 1963, when the River was diverted through block 8 by means of an earth dike across the channel upstream of the core block. A similar earth dike downstream permitted dewatering of the remaining foundation.

Figure 94. Stage 1 Diversion

Stage 3 (Figure 96) of the diversion was provided by a 30-foot by 22.5-foot sluiceway through block 12. On September 4, 1963, the River was diverted through this opening allowing block 8 to be dewatered and its construction resumed.

The final diversion, stage 4, was made through Diversion Tunnel No. 1 on November 15, 1963. The sluiceway was backfilled with concrete in four lifts. The first three lifts were placed by means of a rubber-tired front-end loader. The crown lift was placed with a pumpcrete machine.

Foundation Preparation

Stripping. Stripping was started in the general area of the core block, the temporary diversion flume, and batch plant and cableway areas. The next goal was the area to be covered by the 1964 embankment, followed by the upper reaches of the right and left abutments. The final areas to be excavated were the areas downstream of the core block, from the river channel to approximate elevation 500 feet.

The principal method used was the loading of trucks by shovel for hauling to disposal areas. A part of the excavation was of sufficient depth to permit direct loading by the shovel. However, much of the area to be worked was in steep terrain, with shallow cuts. In this case, bulldozers were used to push the material downslope to the shovel. Gradalls were used in areas too small for dozers to operate. The most satisfactory method in the less steep areas was the use of scrapers. Depth of cut could be controlled, and the resulting surface was uniform. A minor amount of ripping was required to remove hard projections in weathered areas. The stripping for Zones 2 and 3 resulted in a foundation equal in soundness to that of the embankment to be placed thereon and consisted of the removal of topsoil, overburden, and weathered rock to a surface of definable rock structure.

Core Trench Excavation. Core trench excavation provided a foundation for the impervious Zone 1 embankment and included the removal of moderately weathered rock to expose a hard dense surface. The bulk of the excavation was systematically drilled and blasted and the material loaded by shovel.

Foundation preparation in the core trench, subsequent to the initial excavation and immediately prior to impervious embankment placement, consisted of a thorough cleanup of the sound rock surface. Surface preparation included handbarring and high-velocity air or air-water cleaning to remove all loose or unsound rock. Fissures, seams, shear zones, or small areas of adverse slopes were filled or corrected with backfill concrete. Slush grouting also was done where open seams were encountered.

Grout Gallery Excavation. Grout gallery excavation provided a trench within the core trench for the concrete grout gallery. As this excavation was generally in sound rock, drilling and blasting were required. Final cleanup was performed by washing downslope with water jets.

Core Block Excavation. All decomposed, weathered, or crushed rock or similar unsuitable foundation material were excavated to hard fresh rock which provided an impervious foundation after grouting. Final foundation cleanup was performed with water jets after the forms were in place and just prior to placing concrete. After initial stripping, surficial weathered material and foundation excavation for the core block including channel alluvium involved removing about 238,000 cubic yards of material. For comparison, the concrete volume of the core block was 283,000 cubic yards.

Figure 95. Stage 2 Diversion Earth Dike

Figure 96. Stage 3 Diversion

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Factors Affecting Contractor's Progress. Actual excavation overran estimated bid quantities by a substantial margin. Bid quantities are compared with current actual excavation quantities in the following tabulation:

Class of Excavation	*Bid Estimate (cubic yards)*	*Actual Pay Estimate (cubic yards)*
Stripping	2,860,000	4,844,300
Core Trench	690,000	1,112,200
Grout Gallery	30,300	55,500

In general, the requirement that excavation for stripping and core trench be carried to a surface defined in accordance with foundation requirements resulted in the removal of more weathered rock than had been contemplated. At least four slides developed in areas in which excavation was considered complete. The largest occurred on the left abutment, upstream from the core trench. A section approximately 150 feet in length and 40 feet high moved away from the hillside in a broken mass. Heavy rains in December 1964 saturated the material, and apparently small cracks above this slide became filled with water. This resulted in the formation of a much more extensive slide, the top of which extended above elevation 700 feet. Ultimately, about 100,000 cubic yards of material was removed from this slide area.

Core Block

The core block is a mass concrete structure 900 feet long and is positioned approximately 700 feet upstream of the long chord and parallel to the dam axis. Wood forms, constructed in place (Figure 97), were used for concreting on foundation rock, and their use continued until the blocks reached sufficient height to use steel forms. From this point, steel cantilever panel forms were used. A hydraulic crane, operating on top of the blocks, was used to raise and set forms (Figure 98). The crane was moved from block to block by means of a high line. The high line, the same one used at Glen Canyon Dam, was of 25-ton capacity (Figure 99). The main cable or "gut" line was 3 inches in diameter and spanned 1,400 feet. Two rail-mounted steel towers on opposite sides of the canyon provided the anchorage and support for the cable (Figure 100). The towers had a 400-foot travel parallel to the River. A tramway traveled on the cable and carried an 8-cubic-yard placing bucket. All points within the core block area could be reached with this piece of equipment.

The majority of the concrete in the core block was two-sack 6-inch maximum size aggregate with pozzolan to effect a lower heat of hydration for the mass concrete. In areas around the galleries and the upstream face of the core block, an enriched three-sack mix with pozzolan was used. Working limit of the slump as delivered was $1\frac{1}{2}$ inches and a maximum temperature of 50 degrees Fahrenheit was required. The temperature of the concrete for the core block was controlled by cooling of aggregate and by the use of cold water.

Figure 97. Wood Forms—Core Block

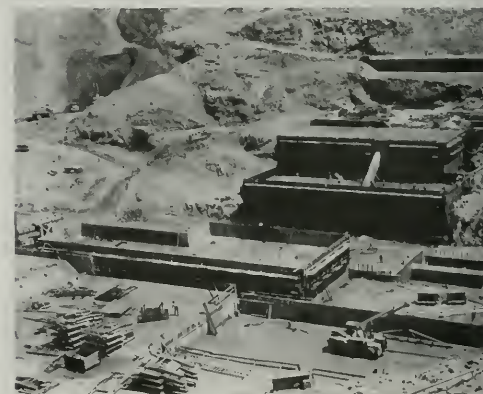

Figure 98. Steel Cantilever Panel Forms—Core Block

Figure 99. 25-Ton-Capacity Cableway

The cooling plant was composed of five silos, each of 210-ton capacity; three refrigeration units; and three boilers. Cooling of the rock was accomplished by evaporation. The silos were filled from the top with moist aggregate, sealed, and a partial vacuum applied. Under reduced pressure, the moisture from the aggregate evaporated, causing a cooling effect or heat loss. The cooled material then was discharged from the bottom of the silos onto an insulated belt and carried to the finish screens at the top of the batch plant. This cooling plant was more theoretical than practical and any deviation from ideal conditions caused a chain of troubles. It was found that the use of ice in place of water was more reliable than the elaborate aggregate cooling plant. On subsequent portions of the contract, ice was used in place of the aggregate cooling plant.

The concrete batch plant was a fully automatic unit with three 4-cubic-yard mixers. The cement silo contained 7,000 barrels and the pozzolan silo had 265 tons of storage. The mixers discharged into a "gob hopper" where the fresh concrete was held momentarily until dumped into the rail-mounted transfer cars. Then the concrete was transported from the batch plant to the cableway loading dock by rail. Two 8-cubic-yard hopper cars were constructed for this purpose, and each was pulled by a small diesel locomotive. An 8-cubic-yard pneumatic bucket on the high line was used for concrete placement.

Treatment of cracks which developed in the core block are covered in a later section of this chapter.

Grout Gallery

The grout gallery is located in rock below the final excavation in the core trench (Figure 101). Construction was initiated early in October 1963. However, the first 14 months of activity amounted to only a small effort and was used to take up work slack at the core block and Thermalito Diversion Dam which was included in the construction contract for the Dam.

With the Zone 1 embankment placement scheduled to begin in December 1964, the contractor accelerated gallery construction. However, heavy rains created difficult working conditions and slowed progress considerably. Despite weather shutdowns and associated difficulties, the contractor was able to construct the gallery from the core block up both abutments well ahead of the embankment placing operations.

Concrete for the grout gallery was delivered from the core block plant and the Thermalito Diversion Dam plant prior to construction of the grout gallery plant in October 1964. This plant was a fully automatic unit with a single 3-cubic-yard mixer.

During the mixing cycle, aggregate, sand, and ice were weighed in batch amounts and delivered to the mixer by belt. The cement and pozzolan were weighed separately in a weigh hopper mounted above the mixer and were discharged directly into the mixing drum. Admixtures were metered by commercial dispensers and entered the mixer with the mixing water. A 3-cubic-yard wet-batch hopper was located directly below the discharge gate of the mixer, and the fresh concrete was held there momentarily before being dumped into agitator trucks for transportation to the site. Two 1-cubic-yard buckets and a truck crane were used for placement.

The gallery segments were about 30 feet long and constructed in two lifts, which consisted of an invert placement and an arch placement. Interior forms were the same as had been used in the core block galleries. They were steel panels of 2-foot sections and were assembled at the placing site while the walls of the excavation served as the outside form for the trench section of the gallery. Exterior forms were required only in the projected section of the gallery. These were made of plywood or shiplap panels and assembled in place.

During the construction period, access ports were

Figure 100. Rail-Mounted Steel Towers

Figure 101. Grout Gallery

left open at various locations along the gallery for ease of access and to facilitate grouting operations. These were sealed as the embankment progressed upward.

At the left and right grout gallery portals, two fans powered by ¾-horsepower, squirrel-cage, induction motors were installed to provide ventilation through the gallery.

Core Block Access Tunnel

The core block access tunnel is located in the left abutment of Oroville Dam. It is approximately 780 feet long and has a finished inside diameter of 7.5 feet. At a point approximately 30 feet from the end of the tunnel, there is a drain shaft that empties into Diversion Tunnel No. 2.

Approximately 566 feet of left exploratory tunnel No. 3 were utilized for the access tunnel, and the remaining 214 feet were driven by a subcontractor.

The tunnel was lined by means of a pumpcrete setup using steel panel forms. The concrete with 1½-inch maximum size aggregate was supplied from a plant located on the right abutment, upstream from the inlet portal to Diversion Tunnel No. 1.

Production of Embankment Materials

As discussed in detail near the beginning of this chapter, material for construction of the Oroville Dam embankment came from borrow areas adjacent to the Feather River and the Oroville Airport, southwest of the City of Oroville (Figure 62). In addition to mining the material for the dam embankment, training dikes, weirs, and a spillway were constructed in the borrow areas to prevent dredger tailing sands from being carried into the river channel during high river stages.

Pervious. The contractor used three different and distinct methods of excavating the dredger tailings. These methods were:

1. A bucketwheel excavator with a conveyor system
2. Two dragline buckets and haul by bottom-dump wagons
3. A scraper spread push-loaded by dozers

Approximately two-thirds of the pervious material was excavated by the bucketwheel excavator. The remainder was removed by the other methods.

The bucketwheel excavator had a rotating wheel 30 feet in diameter and was equipped with eight buckets of 1.8-cubic-yard-capacity each (Figure 102). The excavator dug 30-foot-wide strips into the piles of dredger tailings, rotating through an arc of 90 degrees before moving along parallel to the face of the piles. For the first three passes, material was delivered to a field conveyor system from the belt mounted on the excavator (Figure 103). On the fourth and successive passes, when the distance from the excavation to the conveyor was excessive, a 200-foot-long transfer conveyor was joined to the excavator (Figure 104). When the width of excavation reached 300 feet, the field conveyor system was moved. Tractors, at intervals of 150 to 200 feet, moved along the length of the portable system and skidded the conveyor assembly approximately 2 feet sideways. This produced a severe bending in the rails which straightened out as the succeeding sections were skidded. As the tractors reached the end of the conveyor system, the direction of travel was reversed and another 2 feet of move accomplished. This procedure continued until a full 300-foot move was made and the conveyor was again adjacent to the excavator. The moving of the portable conveyor sys-

Figure 102. Bucketwheel Excavator

Figure 103. Initial Set-In of Excavation

tem usually was done on weekends when production was not needed for embankment placement.

The portable conveyor usually fed a fixed conveyor located at the edge of the borrow area being mined.

The fixed conveyor, which was up to 3 miles long, moved material into a hopper which discharged to an elevating belt that fed a shuttle belt which distributed material over ten hoppers at the loading station. These loading hoppers loaded 40-car trains (Figure 105). The train loading station was moved along with the permanent conveyor after the northern borrow areas were excavated.

Two draglines equipped with 11-cubic-yard buckets loaded a fleet of 100-ton bottom-dump trucks that hauled the material excavated by the draglines to a loading station. The trucks dumped into a hopper that fed the elevating belt; otherwise, the loading station was identical to the one used to load the trains.

The scraper spread was virtually the same spread used in the impervious material excavation and operated in the pervious area during "down time". The spread consisted of single- and double-unit pushcats assisting 20- to 50-cubic-yard-capacity scrapers in excavating, loading, and hauling the material.

An elevating belt loader was used by the contractor to mine areas inaccessible to other excavating units. The hopper of the loader was fed by several dozers. The load discharged directly into a hopper set over the fixed conveyor system or into the hauling units, which hauled to a loading station.

Impervious. The impervious material was of the Red Bluff formation deposited as a river flood plain in an area immediately adjacent to the Oroville Airport.

In March 1964, the contractor began excavating the material with a diesel power shovel equipped with a 3-cubic-yard bucket. A nearly vertical face was excavated with the material hauled by bottom-dump trucks to the loading station for processing. Processing consisted of screening the material to reject all rock and clay lumps in excess of 3 inches. This method of excavation proved to be unsatisfactory since the compacted clay close to the surface did not break down during excavation or screening.

In July 1964, the contractor was allowed to change the excavation method. Scrapers were push-loaded down a 3:1 slope, cutting a thin slice through each stratified layer to the full depth of the pit.

The scrapers dumped into hoppers feeding two vibrating screens where the plus 3-inch material was removed. This was the only processing of the impervious material required other than minor moisture conditioning. Material passing the screens was belt-conveyed to a shuttle conveyor that distributed to ten metal bins from which ten railroad cars could be loaded simultaneously (Figure 106).

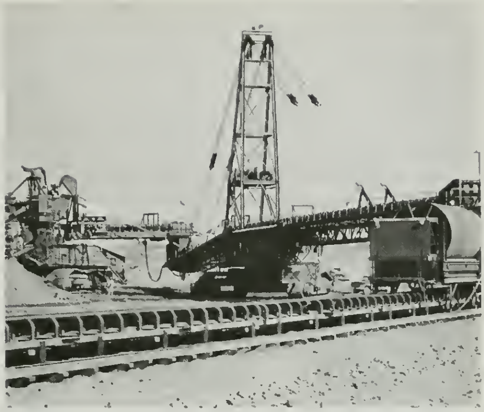

Figure 104. Transfer Conveyor and Bucketwheel Excavator

Figure 105. Pervious Loading Station

Figure 106. Impervious Loading Station

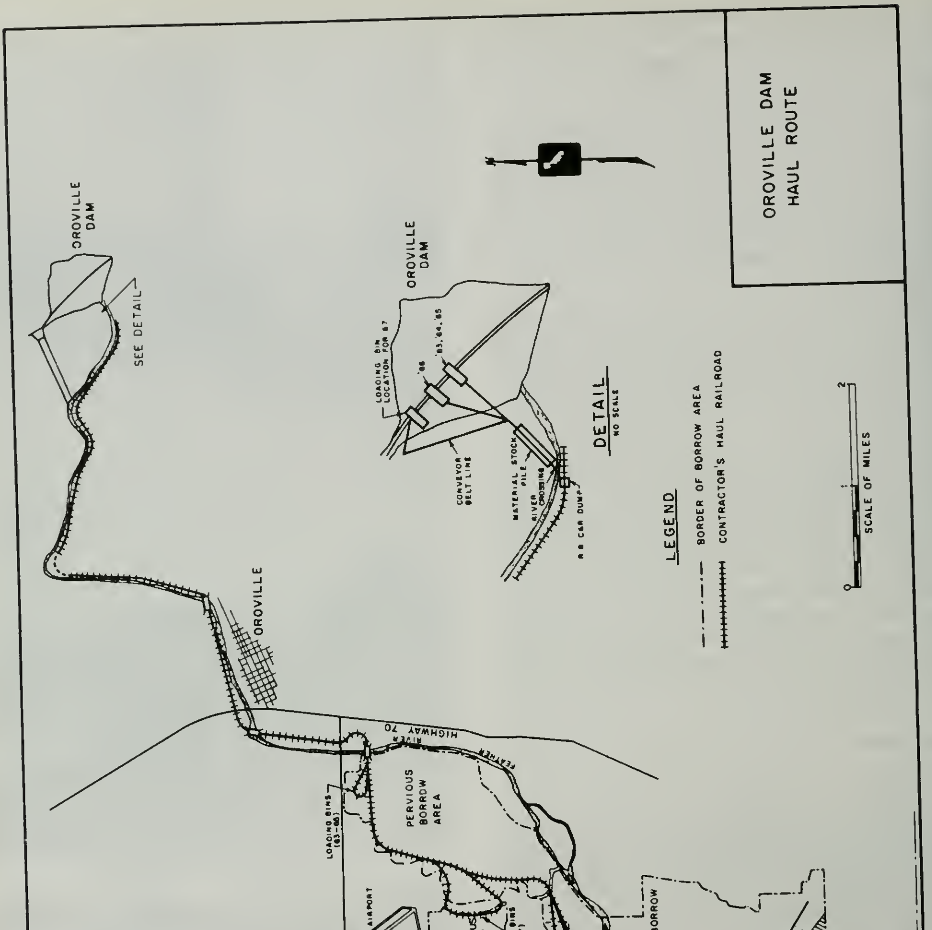

Figure 107. Haul Route

During the winter season, placement of clayey Zone 1 material occasionally was interrupted by rainfall; however, delays were minimized by protecting borrow material from saturation. A selected area was sloped to drain and covered for protection during the rainy season. During the 1964–65 rainy season, an 8-mil clear polyethylene cover was used. An asphalt membrane was used during subsequent rainy seasons.

Training Dikes. A system of training dikes was constructed around the pervious borrow areas to preserve and maintain the existing channel of the Feather River.

A sheet piling weir was included in each dike system so that the water levels on each side of the dikes would be equal and thus prevent washing of the dikes due to differential pressures.

Materials Hauling Facilities

The haul facilities consisted of a railroad (Figure 107) that ran through the borrow areas and along the Feather River to a car dumper at the Dam site, and a system of belts and transfer points that delivered the material to a truck hopper on the embankment. It took approximately 14 minutes to load a train under normal conditions at either type loading station and approximately 50 minutes for a train to make a round trip from the borrow area to the dumper and back. The average haul distance was about 12 miles.

When the train arrived at the Dam site, it would pull onto the dumper, spotting the first two cars over the dumper. After spotting, the locomotive would detach itself from the gondola cars, go to a tail track, pick up a train of empties, and return to the borrow area for reloading. The dumper inverted the two spotted cars (Figure 108) and then proceeded to empty the rest of the train two cars at a time. A "pusher bar" built into the car dumper moved the cars. The cars were equipped with rotating couplings so that it was not necessary to uncouple them for dumping. When a complete train was emptied, it was allowed to roll down an inclined grade to the tail track to await hauling back to the borrow area. Forty-five to fifty trains were dumped in a 24-hour period, a rate compatible with the production from the borrow areas. Embankment placement rates averaged nearly 500,000 cubic yards per week. The borrow and hauling functions frequently had to operate on Saturdays to keep up with the placement. The surge piles at the Dam provided the needed flexibility in the system.

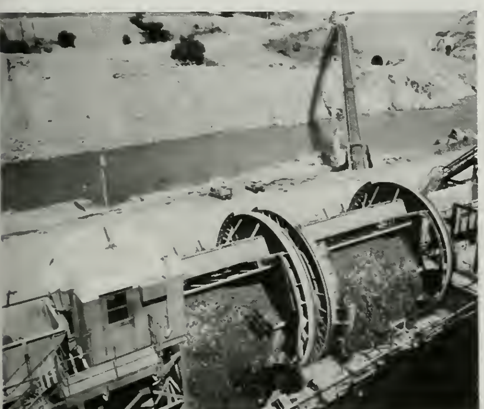

Figure 108. Automatic Car Dumper

The car dumper discharged onto a conveyor belt which carried the embankment material across the Feather River. This belt and other components of the conveyor system at the Dam are discussed in a subsequent section of this chapter.

Each train was powered by tandem, diesel electric, 2,500-horsepower locomotives pulling 42 railroad gondola cars. Although the contractor had four complete trains with 12 spare gondola cars, only three sets of locomotives were used at one time, with the fourth set being used for switch engines or for standby. This was possible since one train of gondolas was being unloaded while the other three were on the tracks.

Dam Embankment

The maximum section of the multizoned earthfill embankment is shown on Figure 60. Zones 1, 1A, 1B, 4, and 4A are impervious material. The mass of the Dam is made up mainly of Zone 3 (the pervious shell) and Zones 2 and 2A (transition zones). A drain system made up on Zones 5A and 5B was included in the downstream shell.

Chronology. The first embankment placement in Oroville Dam was made upstream of the core block on September 16, 1963. The maximum height for the embankment, prior to April 1, 1964, was planned to be elevation 290 feet, 10 feet below the parapet wall of the core block. This would allow a flood to pass over the core block without damage to the upstream embankment. The winter of 1963–64 was dry, work proceeded ahead of schedule, and the contractor was allowed to place fill above this elevation on the right abutment outside of the channel area. This action helped to assure that the 1964 cofferdam would be completed to elevation 605 feet prior to the 1964–65 winter, so that the River could be diverted through the diversion tunnels. In December 1964, less than a month after the 605-foot elevation was reached, a new flood of record (approximately 250,000 cfs) occurred and was controlled through the diversion tunnels with negligible downstream damage. Without the partially completed Dam, the disastrous floods of 1955, in which 38 lives were lost and $100,000,000 damage occurred, would have been exceeded. After the flood subsided, placement of the material downstream of the core block was started. Three years later, on October 6, 1967, Oroville Dam was topped out at elevation 922 feet.


Figure 109. Conveyor Across Feather River


Figure 110. Traveling Stocker at Reclaim Stockpile

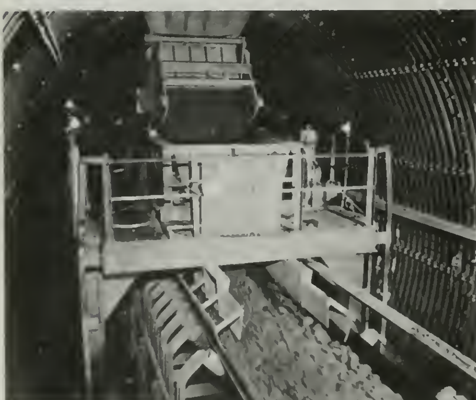
Figure 111. Reclaim Tunnel

Construction Equipment. The placing equipment on the embankment included:

14	100-ton bottom-dump trucks
3	Dozers
3	Loaders
2	6,000-gallon water trucks
1	Truck ballasted with steel plate to give 100-ton wheel-load equivalent
1	100-ton compactor
1	100-ton contractor-fabricated compactor
3	Compactor rollers used in triplex

In addition, several pieces of equipment not normally found on a construction project were used:

1. The conveyor carrying material over the Feather River from the car dumper was 2,380 feet long and carried up to 12,000 tons per hour on a steel cable belt 72 inches wide (Figure 109). It delivered to a traveling stacker that traversed 1,400 feet of stockpile; its belt, 96 inches wide, discharged at a distance of 60 feet over a reclaim tunnel (Figure 110).

2. The reclaim tunnel, constructed of 1,590 feet of multiple-plate arch, had an interior height of 18 feet. Gates and feeders in the tunnel roof consisted of 23 specially designed undercut gates, feeding one of two rail-mounted traveling chute cars, which fed a 72-inch belt (Figure 111). Selection could be made through 19 gates and vibrating feeders to proportion and blend sand into the cobbles to produce material for the transition zone.

3. Four multipurpose distribution conveyors (Figure 112) carried material from the stockpile to the 1,000-ton truck-loading hopper (Figure 113). Each of the conveyors was set up for a haul of 2,300 feet or less. Each was powered by two 1,000-horsepower motors. The conveyor was made up of steel sections 25 feet long.

4. A crawler-mounted pivoting and luffing transfer conveyor (Figure 114) fed the truck-loading hopper and completed the system for delivery of fill material. The transfer conveyor was designed initially to be set up in a 12-degree down position. Then, as the fill progressed, and by a series of ramps, the angle was increased to its maximum 16-degree up position. This provided for a possible range in the truck-loading hopper of 140 feet, with a single position of the feeding conveyors.

5. A bin stored up to 1,000 tons as a surge for truck loading. Through four hydraulically operated gates, it loaded two 100-ton trucks in as little as 8 seconds. The bin was portable, and it was usually kept above the elevation of the fill so that the trucks were loaded downhill to assist acceleration to maximum speed. This resulted in the minimum number of trucks needed for hauling.

Construction Operations. Zone 4, the impervious upstream core below elevation 290 feet, was constructed of selected fine material obtained from the abutment stripping operation. A 100-ton pneumatic roller was specified for compaction. The lift of Zone 4 covering foundation rock was placed 2 to 3 inches thick and hand-compacted with gasoline-powered whackers and air-operated "Pogo Sticks". The abutment rock was covered in the same way as the embankment progressed; then the 100-ton roller made four coverages (two passes equal one coverage) of all the material accessible to it.

Zone 3 was constructed of coarse dredger tailings which contained sound rock with a specific gravity of approximately 2.9. This material, which was mostly minus 6-inch with some larger cobbles, was suitable for use in its natural state. Bottom-dump trucks deposited their loads in 60-foot-long windrows which were spread by rubber-tired dozers. The required lift thickness was a maximum of 24 inches after compaction. Two coverages of the towed triple vibratory roller were used for compaction.

The downstream face of the Dam was constructed to an approximate slope by stepping in successive lifts. The final surface finishing was done with dozers trimming up the slope. Then, two 30-foot-long railroad rails were dragged down the slope.

Selected rock material from the Oroville Dam spillway excavation was placed in parts of the outer upstream Zone 3. Side-by-side test fills were constructed to compare the characteristics of the shot rock with the dredger tailings. Gradation, placing, and compaction of the spillway rock were similar to that required for Zone 3 dredger tailings, except the maximum size was decreased to 18 inches.

Zone 4A, a special compressible zone obtained from abutment strippings, was constructed just upstream of the core block to compress horizontally when high lateral soil pressures built up due to base spreading during the 1964 construction season. To obtain the moderate compaction required, only equipment travel was used since the specified coverage by the triplex vibratory roller gave too much compaction.

Zone 1 (the impervious core of the Dam) and Zone 1B (the 1964 cofferdam core) materials were conveyed to the embankment on the materials handling system. In order to deliver these materials to the embankment within the allowable moisture content limits (plus or minus 1½% of the designated moisture content of the minus U.S. No. 4 sieve fraction), water was added by spray bars at the conveyor transfer points and in the reclaim tunnel.

The material was deposited on a scarified and moistened surface in 100-foot-long windrows and spread by rubber-tired dozers. Placing and spreading were done parallel to the dam axis. Four coverages of a 100-ton pneumatic roller were required for compaction. Rolling of Zones 1 and 1B generally was done parallel to the dam axis. Areas inaccessible to the 100-ton roller were compacted in 6-inch lifts by a ballasted truck loaded to give a wheel load equivalent to the 100-ton roller. Areas inaccessible to the ballasted truck were compacted by hand-operated compactors. This

Figure 112. Distribution Conveyors on Right Abutment

Figure 113. Truck-Loading Hopper

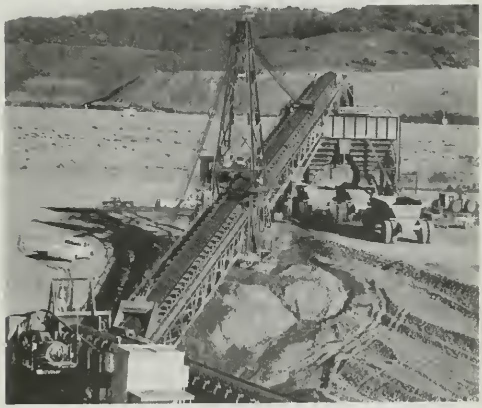

Figure 114. Transfer Conveyor

abutment contact material, with not less than 60% minus No. 4 sieve size, generally was manufactured on the embankment by running Zone 1 material over a portable vibrating screen.

Zone 2 serves as a transition filter zone between the fine-grained impervious Zone 1 and the coarse-grained Zone 3. This material is a combination of gravel and sand occurring naturally in the borrow areas or produced by blending the underlying sands with the gravels. Placing and compaction of Zone 2 were similar to Zone 3. The one exception was that the maximum lift thickness was 15 inches after compaction.

To impound and measure seepage into the downstream Zone 3 embankment at elevation 242 feet, a small earth barrier was constructed of Zone 1A material. Zone 2A filters Zone 1A. Materials and placing procedures were the same as for Zones 1 and 2.

Zone 5A, a horizontal drain, was constructed at elevation 235 to 245 feet from downstream Zone 2 to the downstream face. Zone 5B, a vertical drainage zone 20 feet wide, was constructed immediately downstream from Zone 2. These zones were added after embankment construction was underway because there was concern over the amount of fines in the pervious material that was being delivered to the Dam site. To ensure that the downstream shell would remain dry, these zones were incorporated into the Dam. Zones 5A and 5B were placed and compacted in the same manner as Zone 3.

A zone of riprap was placed on the upstream face of the embankment from elevation 605 to 922 feet and on the downstream face of the mandatory waste area at the downstream toe. Riprap for the upstream face was graded rock up to 1 cubic yard in size. At the downstream toe, rock fragments from ½ to 2 cubic yards in size were used. Most of the riprap was spillway rock placed as the outer 6 feet, measured perpendicular to the slope. The shot rock was hauled directly from the spillway to the Dam.

Electrical Installation

Electrical work on Oroville Dam consisted of the installation of two grounding grids along with two test grids and the installation of the lighting and power systems in the grout gallery, core block, and instrument houses.

Grounding Grids. Grounding grids (Figure 115) were installed in two areas in the embankment as part of the powerplant grounding system and were connected under a completion contract covered in Volume IV of this bulletin.

When the embankment reached elevation 290± feet, the area for the location of grounding grid No. 1 was bladed and "V" trenches were cut in the pattern of the grid. Electricians placed the copper cables in the trenches and cadwelded the cross connections forming the grid. Sand backfill was placed over the cables and embankment operations continued. Prior to the embankment reaching elevation 615 feet in the location for grounding grid No. 2, the grouting subcontractor drilled four cable drop holes from the embankment foundation to the crown of Edward Hyatt Powerplant. Using a "fish" line, the drops were fed into the holes from the Powerplant and pulled up to the foundation level using the winch of a ¾-ton truck. Prior to grouting the holes, the drops were supported from the top by wire mesh grips looped over 2-inch, schedule 80, steel pipe placed over the holes. Before grouting began, the bottom of the holes at the powerhouse crown were caulked. Using a small "tremie", about 25 feet of grout was placed in the bottom of each hole and allowed to set, forming a plug capable of holding the remaining grout needed to fill the hole.

When the embankment reached elevation 615 feet, grounding grid No. 2 was installed similar to grounding grid No. 1.

Lighting and Power Systems. In the grout gallery, the power system consists of a 480-volt, 3-phase, power distribution system which supplies power to each of six electrical equipment panels and two gallery ventilating fans. A 1½-inch rigid conduit was installed throughout the gallery with pullboxes located at 200-foot maximum intervals. Three No. 2 and one No. 12 RHW insulated conductors were pulled into the conduit. The No. 2 conductor provides power, and the No. 12 conductor is used as a switch bus connecting each of the lighting push-button stations for common lighting control.

The 480-volt 3-phase power is supplied to the system from the Powerplant through a distribution board located in the grout gallery at Station 36+75, the grout gallery connection to the emergency exit tunnel. Connected to grounding grid No. 1, a No. 4/0 bare copper ground cable was installed throughout the gallery system to provide grounding of all conduit and electrical equipment.

Structural Grouting of the Core Block

As mentioned in the discussion of the core block design, compressible Zone 4A material apparently limited upstream horizontal earth pressures on the unreinforced parapet protecting it through the initial loading condition. However, when the downstream fill was placed, pressures from that direction caused a rotation of the parapet and cracking as shown on Figure 116.

The presence of cracking was first indicated in October 1965, when contact was lost with piezometer No. 8, upstream of the core block at elevation 250 feet. Tubing to the piezometer had been routed through the core block concrete, near the top surface, and beneath the parapet. In April 1966, the last instrument routed beneath the "hinge point" was lost. Indications continued through the summer of 1967, although no adverse conditions were visible from within the galleries except for a joint in the sump which started leaking water at a rate of approximately 35 gallons per

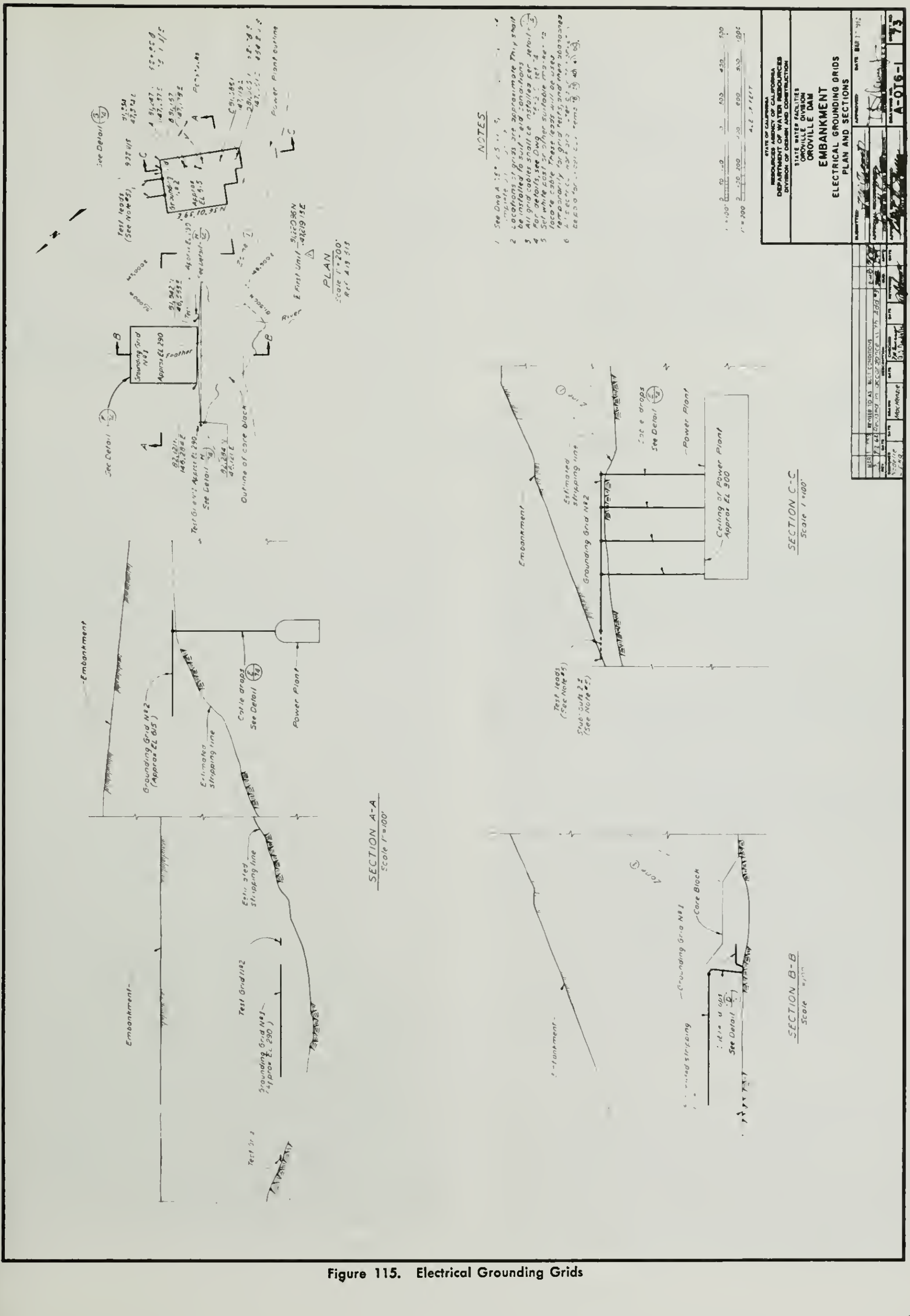

Figure 115. Electrical Grounding Grids

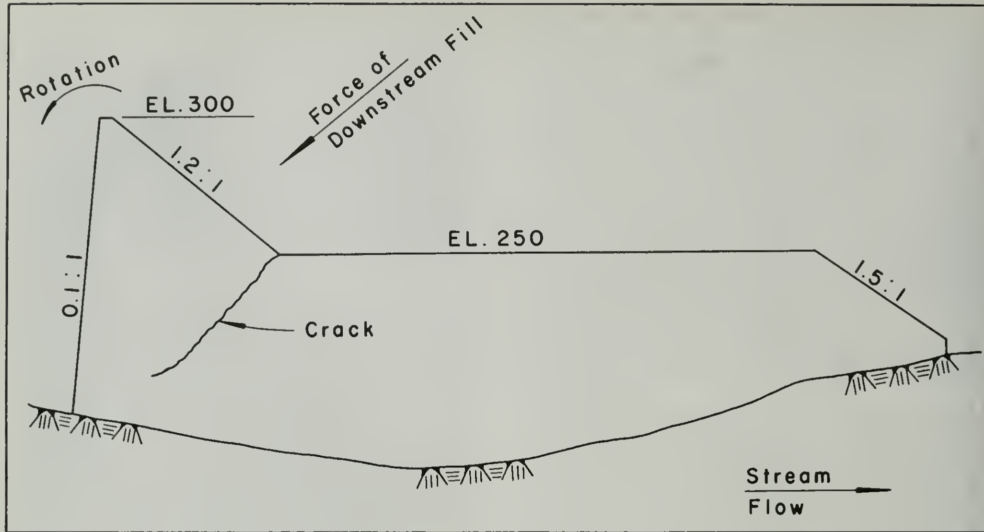

Figure 116. Cracking of Core Block

minute. Exploratory core drilling from the gallery system was initiated in the suspect area beneath the parapet wall. Open cracks, several inches wide, were estimated by drilling action and visually observed by a 20-foot-long periscope.

To minimize any further disruptive movements which might be produced by future reservoir loading, and to prevent reservoir seepage into the galleries and through the core block, the cracks and all monolith joints in the core block were filled with a neat grout. Six thousand sacks of cement were injected under pressure into the interlaced cracks and joint system. The grout mixes used varied from one sack of cement for one gallon of water to one sack of cement for five gallons of water. Drilling and grouting were done in stages from August 1967 through February 1968. Final check drilling, with 450 feet of water head in the pervious zones acting on the core block, indicated the cracks and joints were effectively filled and sealed off from any significant reservoir seepage. The grouted cracks and joints were instrumented to detect any future movements. There has been only minor harmless movement in the five years since the instruments were installed.

Diversion Tunnel No. 2

Diversion Tunnel No. 2 was excavated from the outlet portal to within 54 feet of the inlet portal to protect against possible floods. Open-cut excavation of the outlet portal channel began on January 3, 1963, disclosing that the left channel wall contained unsuitable rock; therefore, the slope was changed from ¼:1 to 1½:1. In addition, the left wall and headwall were both rock-bolted with expansion-shell groutable rock bolts before starting excavation of the crown drift. A 5-foot-square pattern with chain-link fabric and header steel was used. Thirty-foot-long crown bars (No. 18 rebars) were installed on 12-inch centers 2 feet above the tunnel "B" line and grouted.

Excavation similar to that for the outlet portal was started on the inlet portal in February 1964. Because of the extremely weathered rock at this portal, the contractor chose to drive an 11- by 11-foot exploratory crown drift through the 54-foot plug that was left in for flood protection. After a 1-inch-wide crack was observed over the portal at Station 3+20, the face was moved 3½ feet upstream in order to accommodate the revised structural support. The additional support steel and knee braces were anchored in concrete. Additional 15-foot rock bolts were installed above the portal face. Tunnel excavation resumed and removal of the rock plug was completed on July 31, 1964.

Tunnel excavation, concreting, and grouting were accomplished in the same manner as in Diversion Tunnel No. 1, except that the invert was concreted with a slip form and a steel liner was placed downstream of the river outlet location. To facilitate conversion of the tunnel into its role as a tailrace tunnel, the draft-tube stubs; auxiliary intake shaft and river outlet, air supply, and pressure-equalizer tunnels were excavated, lined, and capped off with removable concrete knockout plugs during the initial construction period. The main tunnel was completely excavated and lined prior to November 1964.

Diversion Tunnel No. 2 was in service as a diversion facility for the winter seasons of 1964–65 and 1965–66. During the summer of 1965, draft-tube stubs Nos. 5 and 6 were opened up, completely constructed, and resealed.

In the summer of 1966, Diversion Tunnel No. 2 was closed permanently by the installation of the mass gravity concrete plug. Streamflow continued through the adjacent lower Diversion Tunnel No. 1. Concurrent with this was the partial construction of the valve chamber and draft tubes Nos. 3 and 4 and removal of the knockout plugs from draft tubes Nos. 1, 2, 5, and 6. The knockout plugs for the auxiliary intake shaft and river outlet, air supply, and equalizer tunnels also were removed at this time.

Installation of the last 14 feet of 72-inch conduits and the valves was delayed for the 1966–67 winter season due to manufacturing problems with the conduit and valves. This necessitated installing bulkheads at the upstream end of the conduits to prevent flooding of Diversion Tunnel No. 2 and constructing a blockout for the valves and a portion of the conduit at the downstream end of the tunnel plug.

Wedge seats (Figure 117) for the plug were excavated from the tunnel lining by blasting. The deepest holes at the upstream end of the wedges were 1 foot, and succeeding holes progressing downstream in each wedge were shortened to develop the taper. All loose material was removed with chipping guns prior to placing concrete. Stainless-steel grout stop was installed around the full circumference of the plug at each end. This was done concurrently with the concrete work.

Concrete was placed in seven lifts of varying depths because of the installation of the two 72-inch conduits in the plug. An 18-foot-long blockout was left in the downstream end of the plug from elevation 223 feet up. This was to allow easy installation of the remainder of the conduits and the spherical valves.

Figure 117. Wedge Seat Removal in Diversion Tunnels

Under the specifications, the plug was to be 2½-sack 6-inch maximum size aggregate concrete. The contractor proposed to pumpcrete the placement, which would require 4½-sack 1½-inch concrete. This was approved with the requirement that the concrete be cooled to keep temperature rise equal to or less than that which would have been encountered using the mass concrete.

To accomplish this stipulation, the contractor installed 1-inch-diameter, thin-wall, steel tubing in all but the top lift of the plug which had been specified to be pumped. The tubing was on 3½-foot centers and circulated cold water from a cooling plant at the downstream portal. The cooling was discontinued after 14 days on each lift.

Equalizing Tunnel

The 8-foot-diameter pressure-equalizing tunnel connecting the two diversion tailrace tunnels (Figure 68) was excavated and concreted in conjunction with construction of Diversion Tunnel No. 2.

In the vertical portion of the tunnel, stopper and jacklog drills were used exclusively, working from a temporary platform. Fifteen-foot rock bolts were installed around the perimeter of the collar before excavation started, and additional 10-foot rock bolts were installed within the raise following each succeeding blast. The installation of the rock bolts stabilized the area as no movement was noted.

While the raise was being driven, the contractor decided to continue excavation up to the valve air-supply tunnel. The additional raise excavation at this time probably was faster and safer than sinking a shaft from the valve air-supply tunnel down to the equalizing raise.

The horizontal portion of the tunnel was mucked out with a slusher bucket attached to an air-operated tugger. Support consisted of rib steel left over from Diversion Tunnel No. 1 placed on 4-foot centers.

Valve Air-Supply Tunnel

The 8-foot-diameter, valve, air-supply tunnel (Figure 68) begins approximately 158 feet downstream from the equalizing tunnel, rises vertically 30 feet, then parallels the main tunnel until intersecting with the extension of the equalizing tunnel rise. Excavation and equipment used were essentially the same as used for the equalizing tunnel.

River Outlet Works

Between April and November 1967, the valve chamber was completed and the river outlet works was installed. The remaining portions of the conduits, two 72-inch spherical valves, and two 54-inch hollow-cone valves connected to the conduits through the concrete plug in Diversion Tunnel No. 2 were installed the previous summer and fall. The outlet works was put into service November 15, 1967, on the day following the closure of Diversion Tunnel No. 1.

The operating center for the river outlet is the Station 1 control cabinet within the river outlet control chamber. The 480-volt 3-phase power was provided to the cabinet by the Oroville Powerplant completion contractor. From this cabinet, lighting circuits were provided for the grout gallery, emergency exit tunnel, river outlet control chamber, river outlet access tunnel, and river outlet valve chamber. Power also was provided for the 220-volt, single-phase, spherical-valve-pit, sump pump. Power, control, and position indicator lamps for the fixed-cone dispersion valve operators were provided. Station 2 cabinets for local control of the spherical valves were mounted in the valve chamber.

River Outlet Access Tunnel

Convergence of the valve air-supply tunnel, equalizing tunnel, valve chamber enlargement, and the river outlet access tunnel in the same immediate area (Figure 68) presented a delicate excavation situation. Overbreak was experienced in the river outlet access tunnel collar, and many 15-foot-long rock bolts were installed along with wire fabric. The lower portion of the access tunnel was horizontal, and several 10-foot rock bolts were installed to stabilize this portion immediately behind the collar. The remaining 90% of the tunnel was inclined approximately 35 degrees and was excavated essentially the same as the other small tunnels. It was necessary to build a working platform from which to drill due to the steep slope. No structural steel was used.

Emergency Exit Tunnel

Portal excavation on the 8-foot-diameter emergency exit tunnel (Figure 68) was started May 14, 1964, after Oroville Dam grout gallery excavation essentially was completed beyond the portal location. Crown bars (No. 18 rebars) were installed on 12-inch centers and grouted to stabilize blocky, slightly iron-stained, fresh rock at the opening. Nine W4×13 steel ribs were installed on 4-foot centers in the portal area. Jacklegs were used to drill all holes, and an overshot mucker and shuttle car were used for removing shot rock. Five 8-foot-long rock bolts were installed on 4-foot centers in the arch throughout the 575 feet of tunnel. Numerous seams crossed the tunnel but did not create any problem.

Closure Sequence

By the fall of 1967, the river outlet in Diversion Tunnel No. 2, the connections between Diversion Tunnel No. 2 and the Powerplant, and the other tunnels were complete. Connections to Diversion Tunnel No. 1 had bulkheads so that releases could be made through the river outlet while Diversion Tunnel No. 1 was being converted to its tailrace mode. The main dam and saddle dams were topped out, the spillway was nearly completed, and the reservoir was cleared. Tunnel No. 1 remained to be plugged and converted to a tailrace (Figure 68).

In 1966, the Oroville Dam Consulting Board expressed two concerns about the closure of Diversion Tunnel No. 1, which started on November 14, 1967. The first was a concern that after the first bulkhead gate was positioned, the force of the flowing water might be too great for the second gate to seat completely. The second concern involved a question of whether the bulkhead gates were sufficiently strong in the event of rapid filling of the reservoir immediately after closure. The planned earlier closure had been delayed by several factors to the start of the normal storm season. Measures, including the addition of the stoplogs and the concrete plug behind the steel bulkhead discussed later, were added to the closure sequence to assure its success.

For the seating of the gates, concrete stoplogs (Figure 118) were placed across each opening during closure. By producing quiet water at the bulkhead gate, a diver was able to clean the slots so that the gate could be lowered without the pressure of running water. (By prior arrangement with upstream dam owners, the riverflow was held to less than 1,000 cfs.) After the first bulkhead gate was dropped into place, the stoplogs were removed from that side, moved to the other side, and the procedure was repeated. There was a substantial amount of leakage around both gates after they were dropped into place. The contractor succeeded in sealing the right gate by pumping a 1:1 mix of bentonite and Silvacel followed by fine sand into the edges of the gate. This was ineffective on the left gate. A section of 2-inch hose wedged between the slot and the gate on the left side succeeded in diverting the flow to the invert where it could be picked up in a drain.

After the water was controlled, work began on the building of the form for the 30-foot-long upstream

Figure 118. Lowering Stoplogs

plug to buttress the bulkhead gates. A 35-foot-diameter wood form was constructed adjacent to the trailing edge of the divider in the intake structure of the tunnel. The steel bulkhead gates served as the upstream form. The placing operation was continuous except for minor breakdowns of equipment and a few shutdowns to allow the air to clear in the tunnel. No attempt was made to completely seal off the top lift of the plug as its only function was to provide safe working conditions in the tunnel while the permanent plug was being constructed.

As soon as the upstream plug was completed, the contractor moved to the permanent plug near the midpoint of the tunnel. The wedge seats were removed in the manner used in Diversion Tunnel No. 2, and an 18-inch drain pipe was installed in the invert of the tunnel. The permanent plug then was constructed in five 7-foot lifts. Cooling pipes were installed in an identical method to Diversion Tunnel No. 2.

Prior to placing the last lift, the contact grout system was installed. A multiple header hookup was used on both pressure lines of the system with a return line connected to the vent pipe at the crown.

The 18-inch drain pipe installed in the plug at the invert was backfilled by placing a 90-foot piece of 10-inch thin-wall pipe inside the drain line and supporting it off the invert on chairs. Concrete was pumped through the thin wall into the drain pipe and forced to the upstream end. Pumping continued, forcing the concrete to reverse direction and flow back over the thin wall and out the downstream end of the drain pipe. The 10-inch line was cut and left in place.

While the tunnel plug was being completed, drain holes were drilled in the downstream half of the tunnel and, as soon as the tunnel plug was completed, the plugs connecting to the draft-tube and pressure-equalizing tunnels were removed. All work in Diversion Tunnel No. 1 was completed on March 15, 1968.

The steel bulkheads on the draft tube-tunnel No. 2 connections were removed from the floor of the tunnel shortly thereafter. Downstream demands were met by releasing water stored in the Thermalito facilities.

Spillway

Clearing. Approximately 115 acres, 40 in the spillway and chute and 75 in the emergency spillway area, were cleared of brush and trees. The area below the emergency spillway was not cleared.

Excavation. The three major methods used to excavate the spillway were as follows: bottom-loading scrapers and pushcats, a loader with cats feeding the belt and bottom-dump wagons hauling the material, and two large shovels.

In general, the scrapers were used to strip the area to rock. The shovels were used to excavate the rock after it was drilled and shot. The loader was used similarly to the scraper operation, the main difference being that it was possible to work this operation in rougher terrain as up to eight dozers were used to push material to the feeding hopper. A road was graded out below the hopper and the bottom-dump wagons could drive under the hopper to load.

All drilling for blasting was done by percussion-type drills mounted on tracks and powered by air. The patterns varied greatly from area to area. The most generally used pattern was 8 by 8 feet; however, patterns ranging all the way from 2½ by 2½ feet to 15 by 15 feet were used depending on the area, type of rock, and excavation objective. Excavation near structure lines had to be controlled to avoid damage to the rock to be left in place, and 840,000 tons of riprap for Oroville Dam had to be produced.

In part of the emergency spillway, an additional 10 feet of excavation was required to reach acceptable foundation rock, resulting in considerable additional time for excavation and placement of the backfill concrete to subgrade.

Approximately 90% of the chute foundation required blasting to reach grade. The only extra excavation directed was removal of a few clay seams in the foundation and a few areas where slope failures occurred.

The depth of overburden in the approach channel was deeper than estimated and the slopes had to be changed from ½:1 to 1½:1 to prevent sloughing as the excavation reached the final grade.

The slopes in the flood control outlet gate section proved to be of a lower quality rock than anticipated. There were several large seams running parallel with the chute. The planned anchor bars were replaced with grouted rock bolts, pigtail anchors, and chain-link surface covering in that area.

Drain System. The foundation drains designed for the spillway included nearly vertical NX holes drilled 65 feet into the foundation rock of headworks Monoliths 25 and 26 and extensive perforated pipe systems on the foundation surface under the headworks, chute, and higher portions of the emergency spillway weir. Much of the drain system on the foundation surface was modified during construction.

The original 4-inch-diameter, horizontal, pipe drains under the chute were redesigned in accordance with a recommendation from the Oroville Dam Consulting Board. The pipes were placed on a herringbone pattern to give them a downward slope and enlarged to a 6-inch diameter. The longitudinal collector system was enlarged proportionally and modified slightly. The effect of these modifications was to increase the system's capacity and its self-cleaning ability. The pipes remained on the foundation enveloped in gravel which projected into the reinforced-concrete floor of the chute.

Similar drain pipes were impractical to place on irregular rock surfaces under the headworks and emergency spillway weir. The contractor was allowed to substitute wooden formed drains of equal cross-sectional area. These forms were cut to fit the irregular rock surface and remained in place after the concrete was placed over them.

Concrete. Concrete placement started on January 26, 1966. Mass concrete was placed monolithically in all monoliths (other than 25 and 26, which contain the gates) between the transition section at Station 18+30 on the emergency spillway and the east end of the flood control outlet at dam Station 20+61.66. Mass concrete also was placed in the emergency spillway weir to the west of the transition. An on-site plant discharged into 4-cubic-yard concrete buckets positioned on low-boy trucks which hauled to the placing area where the concrete was handled and placed using a track-mounted crane. Six-inch vibrators were used to consolidate concrete. Concrete was placed in the monoliths in lifts of 7 feet - 6 inches. Wooden starter forms were used until 7-foot - 6-inch, cantilever, steel forms could be used. An adjustable steel form was used to form the curved or ogee section of Monoliths 1 through 20. The uppermost lift of the ogee section was formed using wooden forms. Seven screed ribs were shaped to the desired curve, between which panels were added as concrete was placed. When the concrete would hold its shape, the forms were removed and the surface finished. Structural concrete was placed in Monoliths 25 and 26 of the flood control outlet, the approach walls, the chute walls and invert, and the terminal structure.

Flood control outlet concrete was placed by a track-mounted crane. When the concrete reached an elevation that could not be reached by the crane, a conveyor-belt system was used. This conveyor system was used mainly in the Monolith 26 half of the flood control outlet. The conveyor-belt system was used less in Monolith 25 due to a more accessible area for a crane to be positioned at Monolith 24.

Concrete placing for the spillway chute invert began on September 8, 1966. A conveyor-belt system was used to transport the concrete from the chute banks to point of placement. Concrete was transported from batching plant to a conveyor reclaim hopper by "bathtub" trucks.

A 40-foot, steel-beam, slip form was used to screed the concrete to invert grade. This slip form, along with the discharge portion of the conveyor, rode on steel rails positioned along each side of the chute slab being placed. The slip form was propelled by the use of a winch geared to keep pace with the placing of concrete. This slip form was used chiefly on the steeper area of the chute. A smaller slip form was used to screed concrete for the chute wall footings.

Chute invert concrete in the area of lesser slope near the flood control outlet was placed by a truck-mounted crane and concrete bucket. Terminal structure concrete of the chute also was placed in this manner.

Rigid wood forms were constructed to contain the concrete in the chute walls. Windows were cut in the backs of the wall forms through which concrete was vibrated until it reached that level. A truck-mounted crane and concrete bucket were used to place the concrete in forms. A leveling platform was made from timbers to position the crane while working on the steep portion of the chute invert. The total volume of concrete placed was 165,000 cubic yards.

Electrical Installation. Lists were made of all electrical conduits and equipment to be installed prior to placing each lift of concrete. The lists were reviewed whenever necessary to ensure the installation of conduits in their proper places. The ground cable was completed as concrete was placed. All electrical conduits were in place before the electricians started cleaning the conduits and blowing lines into them for use in pulling the conductors through the conduits.

The standby generator was placed in the control room. After it was started and checked, it was used to operate the hoists because the temporary source of power had been removed.

As the radial gates were installed, temporary circuits were run to the hoist motors so the hoists could be operated and the gates checked before the controls were activated.

Gate Installations. The concrete was blocked out of the piers in the immediate vicinity of the trunnion beams. The trunnion beam was placed over the tensioning tendons and set to final alignment. The bearing plates then were brought into contact with the beams. When the trunnion beams and the bearing plates were positioned, concrete was placed in the blocked-out areas. After the concrete had achieved its design strength, the rods were tensioned to design loading and no rod failures occurred. After the tendons were tensioned, the voids between the rods and the sleeves were grouted.

The side-seal plates were aligned by swinging an arc with a chain from the centerpoint of the trunnion. After the plates were in proper position along the arc of the gate, the plates were adjusted parallel to the walls of the bay by transits. By the time the gates were to be placed, the radial wall plates were within ½ inch of their proper position.

The contractor chose to erect the gates in the bays in their operating position. This method was used because there was not enough work space to bring the gate skins into the bay in one piece. Erection of the radial gates was done as follows: (1) two halves of the skin with the side seals attached were moved into place; (2) horizontal and vertical members were bolted to the skins; (3) canted-arms and trunnions were bolted in place; (4) the gate was leveled and aligned and the two halves welded together; and (5) the gate was lowered to its closed position and final adjustments made to position the gate and seals.

Grouting Program

Foundation grouting for Oroville Dam consisted of curtain grouting a single row of holes the full length of the core trench and spillway crest, extending a maximum depth of 200 feet into rock; blanket grouting of the core trench and core block foundation; envelope grouting over Diversion Tunnels Nos. 1 and 2 and the Powerplant; and a minor amount of curtain grouting from the Palermo outlet works tunnel (Figure 66).

Grout Curtain. The grout curtain for Oroville Dam is located approximately along the upstream one-third point of the core trench on the abutments and the core block in the channel. This curtain was drilled and grouted from the grout gallery to where the gallery reached about elevation 750 feet, and from the rock surface between elevation 750 feet and the dam crest at elevation 922 feet. The dam and spillway curtains join on the upper right abutment at dam Station 21+29. The plane of the curtain dips 75 degrees upstream where grouted from the gallery section and is vertical where grouted from the rock surface. To control leakage and grout placement, the curtain was divided into three zones.

Zone 1 extended 50 feet into rock and was drilled and grouted in a minimum of two stages with a maximum hole spacing of 10 feet. Grouting pressures of 50 to 100 pounds per square inch (psi) were used in the gallery; 25 to 50 psi from the rock surface.

Zone 2 extended 100 feet below foundation elevation 500 feet; above elevation 500 feet, the depth equaled one-fourth the potential reservoir head and was drilled in one stage using packers with a maximum hole spacing of 20 feet. The normal grouting pressure was 150 psi.

Zone 3 extended 200 feet below elevation 500 feet, one-half the potential head above elevation 500 feet, and was drilled in one stage using packers with a maximum hole spacing of 40 feet. The grouting pressure ranged from 200 to 300 psi.

The total grout take was 23,763 cubic feet of cement and the average was 0.28 of a cubic foot per foot of hole.

Blanket Grouting. The blanket grouting program consisted of shallow low-pressure grouting of most of the core trench and core block foundation upstream from the grout curtain. This sealed the many joints and shear zones that cross the foundation and also consolidated areas of strongly fractured or weathered rock. Most blanket grouting in the core block was done after concrete placement.

Spacing and depth of blanket holes were governed by the nature and extent of foundation defects encountered and by grouting results as the operation progressed. In areas of weathered or closely fractured foundation, final hole spacing was as close as 5 feet and holes were arranged more or less in an equilateral pattern. In areas of fresh sparsely fractured foundation, such as portions of the core block and lower abutment core trench, holes were up to 40 feet apart. The general practice was to space the initial holes from 20 to 30 feet apart, then reduce the spacing if the initial holes took grout or if there was severe surface leakage. Final spacing of most holes upstream from the grout gallery ranged from 10 to 20 feet apart. In addition to grouting from the core trench surface ahead of embankment placement, 123 holes were connected to the gallery with pipes and grouted under an embankment cover of 100 feet to allow grouting at pressures of 50 to 100 psi and eliminate surface leakage. Most holes were inclined 40 to 70 degrees from horizontal and were aimed to cross steeply dipping joints and shears a few feet below the foundation surface. In general, holes ranged from 10 to 30 feet deep; a few holes were as deep as 70 feet. The total grout take was 13,745 cubic feet of cement and the average was 0.23 of a cubic foot per foot of hole.

Envelope Grouting. A grout canopy, extending 700 feet downstream from the tunnel plug section, was constructed over both diversion tunnels and around the underground powerplant to reduce inflow into the plant and protect the concrete lining of the diversion tunnels from a potential reservoir pressure of 300 psi. The grout curtain for Oroville Dam extends from the grout gallery to the downstream end of the envelope to produce a continuous curtain above the powerplant area (Figure 69).

Drilling and grouting were begun shortly after the top heading excavation was completed through the draft-tube section. All holes in each ring were drilled to the full depth of the first grouting zone as the drilling jumbo progressed downstream from ring No. 1. Grouting operations followed six to eight rings behind the drilling jumbo.

Reservoir Clearing

Clearing of the reservoir was planned to enhance recreational use. Vegetation was not removed in certain areas to be submerged to provide fishery enhancement. Public beach areas were grubbed of stumps and roots. The plan also included an agreement with the U. S. Forest Service and State Division of Forestry for the prevention and suppression of fires during clearing.

Elevation 640 feet was established as the lowest stage to which the reservoir would be drawn down under extreme conditions. Clearing of the area below elevation 640 feet consisted of removal of all loose floatable material. This included drift along the streams, logs, windfall trees, and miscellaneous improvements. The area between elevations 640 and 900 feet, with the exception of areas of vegetation retention, was cleared of all trees, brush, and improvements. All stumps and roots were grubbed out of the 840 acres of planned beach areas. In the remaining 7,660 acres above elevation 640 feet, stumps 6 inches or less in height were allowed to remain.

Approximately 1,100 acres of vegetation retention was provided above elevation 640 feet for the enhancement of the reservoir fishery. In these areas, all coniferous trees over 25 feet in height and deciduous trees over 60 feet in height were cut down and tied to their

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stumps with wire rope. A total of nearly 7,000 trees over 12 inches in diameter were cut and anchored with ½-inch galvanized wire rope, and a total of about 5,000 trees 12 inches or less in diameter were cut and anchored with ¼-inch galvanized wire rope.

The work of clearing timber and brush, between elevations 640 and 900 feet, was performed in three separate operations: (1) cutting timber and brush, (2) raking and piling cut material for burning, and (3) burning and final cleanup.

Cutting of timber and brush was performed with dozer equipment and chain saws. The method of cutting timber and brush was dependent upon roughness of terrain and access. Dozers equipped with sharp cutter blades, set at an angle, were used for cutting timber and brush on flat and moderately steep terrain. On steep terrain, a second dozer with a heavy-duty winch cable was used to control the downhill movement of a dozer with a cutter blade and to haul it back up the slope. This operation of using two dozers on steep terrain is known as a "yo-yo" method of cutting and stripping hillsides. Chain saws were used for cutting in areas not accessible to dozer equipment or for cutting large trees in heavily timbered areas.

The work of raking and piling cut timber and brush was performed by hand labor and by dozers equipped with brush rakes or regular dozer blades. On steep terrain, a "yo-yo" method was used to pile material in windrows. In areas not accessible to heavy equipment, chain saws were used to fell the timber and cut material into shorter lengths to facilitate hand piling and burning. Dozers and labor crews were used to collect and push together unburned chunks until completely burned.

Burning was restricted to the winter months or as permitted by the Division of Forestry. The work of raking and piling was contingent upon availability of equipment, manpower, terrain, and accessibility. At certain locations, the interval of time between cutting, piling, and burning actually spanned a period of six months or more, which allowed the cut timber to dry and brush to cover the ground during the late summer months. To prevent the spread of wild fires, and before piling and burning in these areas were permitted, the contractor was required to provide adequate fire breaks near the uncut timber and brush lines at elevation 900 feet.

Work of grubbing areas for recreation was performed after all clearing was completed. Grubbing was performed by dozers equipped with regulation dozer blades and with a single ripper tooth clamped and mounted on the left end of the dozer blade. The ripper tooth was mounted at one end of the blade so that stumps being rooted out would be visible to the operator.

The contractor cleared approximately 12 dwellings located at Bidwell Bar, Bidwell Bar Canyon, and Enterprise and disposed of them by burning.

The Department removed all buildings located along the Western Pacific Railroad and buildings located at Bidwell Bar. These buildings were burned in 1964 prior to the reservoir clearing contract.

Saddle Dams

Construction of the Saddle Dams commenced in December 1966. The contract specifications allowed 275 days to complete the work on both dams. Completion was timed to precede closing of the second diversion tunnel at Oroville Dam and commencing of storage in Lake Oroville. The planned date for completion of Oroville Dam and start of storage in the reservoir was October 17, 1967. The actual start was on November 14, 1967.

Foundation. All foundation excavation at both dams was done by common means, using scrapers and dozers with rippers. Excavation was started January 10, 1967 and essentially was completed on February 28, 1967.

The Bidwell Canyon Saddle Dam foundation outside the cutoff trench was stripped to expose strongly or moderately weathered rock. The cutoff trench was excavated to fresh rock or firm moderately weathered rock. The bottom of the trench was cleaned of loose materials with hand tools and air jets.

The downstream Zone 3 foundation within Miners Ranch Reservoir was not dewatered during embankment construction. It had been determined by probing, prior to construction of the Saddle Dam, that this area consisted of strongly weathered rock which was stripped of topsoil during construction of Miners Ranch Dike.

Ground water issued from the spoil pile adjacent to the upstream left abutment of the main dam during foundation excavation. Water also flowed from the toe of the Miners Ranch Dike. These flows were drained from the foundation area by placing a French drain of coarse rock at the base of Zone 3 and by digging a drainage trench through the spoil pile north of the Dam.

Excavation for Parish Camp Saddle Dam foundation outside the cutoff trench exposed strongly and moderately weathered phyllite over most of the area. The cutoff trench was excavated to fresh or firm moderately weathered phyllite.

The grout curtain for both dams was 25 feet deep in a vertical plane. Holes were inclined 20 degrees off vertical along the plane in order to cross the nearly vertical cleavage and schistosity. The ends of the curtain extended to foundation elevation 917 feet on all abutments. Stage grouting was used where necessary. In the more weathered areas where excessive amounts of grout might have leaked to the surface, a 10-foot-deep upper stage was drilled and grouted at 10 psi before drilling and grouting the remaining 15 feet of hole. In fresher rock, it was possible to grout the entire hole with a single hookup at a pressure of 25 psi without surface leakage.

Construction Materials. Impervious borrow for

Bidwell Canyon Saddle Dam was obtained by stripping near-surface, strongly weathered, decomposed amphibolite. Initially, material was taken from the two designated borrow areas north of the west dam. When these sources were depleted, one of the areas was extended farther northward and a new borrow area was opened near the east end of the main dam. These new sources were explored with bulldozer trenches excavated by the contractor at the direction of the Department. To help alleviate the shortage of impervious borrow, the design of the west dam was modified to reduce the quantity of impervious material required in the embankment. This modification permitted the use of more rocky material in the outer limits of the west dam impervious section. Such material could be obtained by excavating deeper in the designated borrow areas.

Pervious borrow initially came from the designated source, a rock spoil pile adjacent to the main dam made up of material from portal excavation of tunnels leading in and out of Miners Ranch Reservoir. When this stockpile was nearly depleted, a few thousand cubic yards of rock was hauled from a spoil pile of rock wasted during construction of the Oroville-Quincy Road relocation near Bidwell Bar Bridge. This source was abandoned when it was found to be more economical to rip and remove rock from one of the designated impervious borrow areas which had already been stripped of impervious material.

Filter material, Zone 2B, was trucked into the area from the Feather River dredger tailings and was produced by mixing the necessary ingredients, utilizing a dragline and a dozer. The proportions of different materials had to be changed somewhat when mechanical analysis tests indicated an increase in that portion passing the No. 200 screen. This condition was controlled by frequent testing of the material at the source. Zone 4B is clean, coarse, dredger-tailing cobble.

Impervious borrow for Parish Camp Saddle Dam was obtained from the weathered surface of the Calaveras phyllite, which was decomposed to gravelly clay and clay. The material came from excavation of the access road to the dam and from the designated borrow area just northwest of the dam. Pervious material was imported by the contractor.

A small amount of riprap for Bidwell Canyon Saddle Dam was obtained from the designated pervious borrow source adjacent to the main dam by separating out the larger rock fragments using dozers equipped with rock rakes. Most of the riprap was imported by the contractor from a department-owned spoil pile at Thompson Flat north of Oroville which contained rock excavated from a deep cut during construction of the Western Pacific Railroad relocation. This riprap also was separated out with dozer-mounted rock rakes. All riprap placed on the dams was fresh, hard, and durable metamorphic rock.

Mine Adit Plug. Work under the contract included placing compacted impervious embankment to cover the portal of an abandoned, collapsed, mine adit one-third mile northwest of Parish Camp Saddle Dam. This embankment was designed to prevent possible leakage from the reservoir.

The contractor stripped the area around the adit portal with dozers and a backhoe. The portal was found to be completely collapsed and was not opened during excavation. Water flowing from the adit at about ¼ of a gallon per minute was sealed off with placement of the compacted embankment.

Construction of Embankment. Impervious material (Zone 1B, 1P, and mine adit embankment) was excavated by scraper units which picked up both clay and weathered rock for placement. After placement, the lift was cut to a thickness that would produce a finished layer not more than 6 inches in depth by a grader and rolled by a 5-foot by 5-foot, double, sheepsfoot roller.

Filter material (Zones 2B and 2P), after being placed in layers that resulted in a thickness of not more than 12 inches after compaction, was moisture-conditioned to prevent bulking. Compaction was accomplished by four passes of a large tractor.

Pervious material (Zones 3B and 3P) was placed and compacted in the same manner as Zones 2B and 2P. Care was taken to distribute the embankment material to produce a well-graded mass of rock with a minimum of voids.

By agreement with Oroville-Wyandotte Irrigation District, Miners Ranch Reservoir was lowered as far as possible during Zone 4B placement. Underwater placement was accomplished by dumping, starting from the edge of the water and proceeding parallel to the longitudinal axis of the dam. This procedure was followed until the top of the material was 1 foot above the surface of the water. Thereafter, the material was placed in layers not to exceed 2 feet in thickness after compaction with a large tractor.

Riprap was placed to its full thickness in one operation. Care was taken not to displace the adjacent material. Riprap was placed on the upstream faces of the embankment with front-end loaders but had to be straightened and rearranged with a truck crane and a clam bucket to produce a well-graded mass with a minimum of voids.

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BIBLIOGRAPHY

California Department of Public Works, "Report on Comparison of Oroville, Big Bend and Bidwell Bar Reservoir Sites for Development of Feather River", August 1949.

California Department of Water Resources, "Basic Data Report on Test Fills of Proposed Embankment Materials for Oroville Dam", December 1961.

Chadwick, W. L. and Leps, T. M., "Inspection and Review of Oroville-Thermalito Project Facilities, Oroville Thermalito Diversion Dam, Thermalito Forebay Dam, Thermalito Afterbay Dam, Feather River Hatchery Dam", February 1973.

Daehn, W. W., "Development and Installation of Piezometers for the Fluid Measurement of Pore-Fluid Pressures in Earth Dams", ASTM, 1962.

Golzé, Alfred R., "Oroville Dam", Western Construction Magazine, April 1962.

Golzé, A. R., Seed, H. B., and Gordon, B. B., "Earthquake Resistant Design of Oroville Dam", Proceedings of ICOLD, 1967.

Gordon, B. B., Hammond, W. D., and Miller, R. K., "Effect of Rock Content on Compaction Characteristics of Clayey Gravel", ASTM, STP 377, June 1964.

Gordon, B. B. and Miller, R. K., "Control of Earth and Rockfill for Oroville Dam", ASCE Journal of the Soil Mechanics and Foundation Division, May 1966.

Gordon, B. B. and Wulff, J. G., "Design and Methods of Construction, Oroville Dam", Edinburgh, Scotland, Communication, 8th Congress on Large Dams, 1964.

Hall, E. B. and Gordon, B. B., "Triaxial Testing Utilizing Large Scale, High Pressure Equipment", ASTM, Special Tech. Pub. 361, 1963.

Kruse, G. H., "Instruments and Apparatus for Soil and Rock Mechanics", ASTM-392, pp. 131–142, December 1965.

Kulhawy, F. H. and Duncan, J. M., "Stresses and Movements in Oroville Dam", ASCE Journal of the Soil Mechanics and Foundation Division, July 1972.

Lanning, C. C., "Oroville Dam Diversion Tunnels", ASCE Journal of the Power Division, October 1967.

Marachi, N. D., Chan, C. K., and Seed, H. B., "Evaluation of Rockfill Materials", ASCE Journal of the Soil Mechanics and Foundation Division, January 1972.

O'Neill, A. L. and Nutting, R. G., "Material Exploration for Oroville Dam", 66th Annual Meeting, American Society for Testing Materials, June 1963.

Schulz, W. G., Thayer, D. P., and Doody, J. J., "Oroville Dam and Appurtenant Features", Journal of the Power Division, American Society of Civil Engineers, Volume 87, No. 902, July 1961.

Stroppini, E. W., Babbitt, D. H., and Struckmeyer, H. E., "Foundation Treatment for Embankment Dams on Rock", ASCE Journal of the Soil Mechanics and Foundation Division, October 1972.

Thayer, D. P., "Oroville Test Fill Experience in Compacting Granular Material", Proceedings 7th Congress on Large Dams, Volume IV.

Thayer, D. P., Gordon, B. B., and Stroppini, E. W., "Soil Mechanics Aspects of Oroville Dam", American Society of Civil Engineers, Water Resources Engineering Conference, May 1963.

Thayer, D. P. and Stroppini, E. W., "Hydraulic Design for Oroville Spillway", ASCE Hydraulic Conference, August 1965.

Tuthill, L. W., Adams, R. F., and Mitchell, D. R., "Mass Concrete for Oroville Dam", SP-6 Symposium on Mass Concrete, ACI, 1963.

United States Bureau of Reclamation, "Hydraulic Model Studies of the Diversion Tunnels for Oroville Dam", Hydraulics Branch Report No. HYD-502, January 18, 1963.

———, "Hydraulics Model Studies of the Flood Control Outlet and Spillway for Oroville Dam", Hydraulics Branch Report No. HYD-510, September 30, 1965.

———, "Hydraulic Model Studies of the River Outlet Works at Oroville Dam", Hydraulics Branch Report No. HYD-508, October 11, 1963.

Western Construction Magazine, "Big Show Starting at Oroville Dam", May 1963.

Wilber, A. F., and Mims, J. R., "Construction Procedure at Oroville Dam", ASCE Conference Paper, Denver, Colorado, May 16–20, 1966.

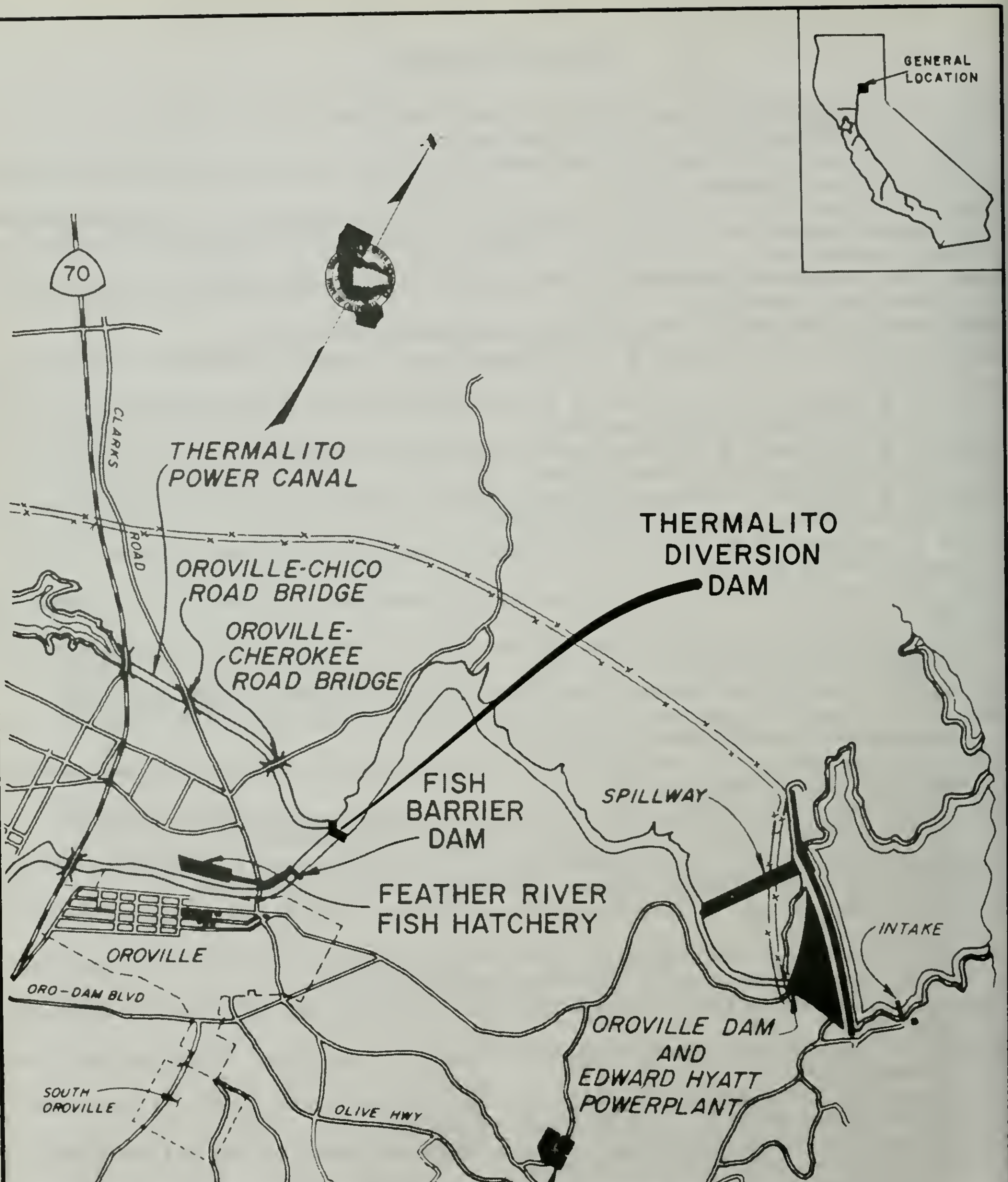

Figure 119. Location Map—Thermalito Diversion Dam

CHAPTER VI. THERMALITO DIVERSION DAM

General

Description and Location

Thermalito Diversion Dam is located on the Feather River approximately ¾ of a mile upstream from the Oroville-Chico highway truss bridge and 4½ miles downstream from Oroville Dam. The nearest major road, State Highway 70, crosses the Feather River about 2 miles downstream of the Dam (Figure 119).

Thermalito Diversion Dam (Figure 120) consists of a 625-foot-long, concrete, gravity section with an ogee spillway; a canal-regulating headworks structure; and an earth embankment section at the right of the canal headworks (Figure 121).

Purpose

Thermalito Diversion Dam diverts water into Thermalito Power Canal for power generation at Thermalito Powerplant and creates a tailwater pool for Oroville Powerplant. The impounded reservoir acts as a forebay when Edward Hyatt Powerplant is pumping water back into Lake Oroville. The reservoir also is used for incidental recreation.

Chronology

The concept of Thermalito Diversion Dam evolved in 1956 when a canal to a powerplant and offstream afterbay was first proposed. The Diversion Dam site formerly was considered for an afterbay. The pump-storage concept required more afterbay storage than was available at the Diversion Dam. Detailed design

Figure 120. Aerial View—Thermalita Diversion Dam

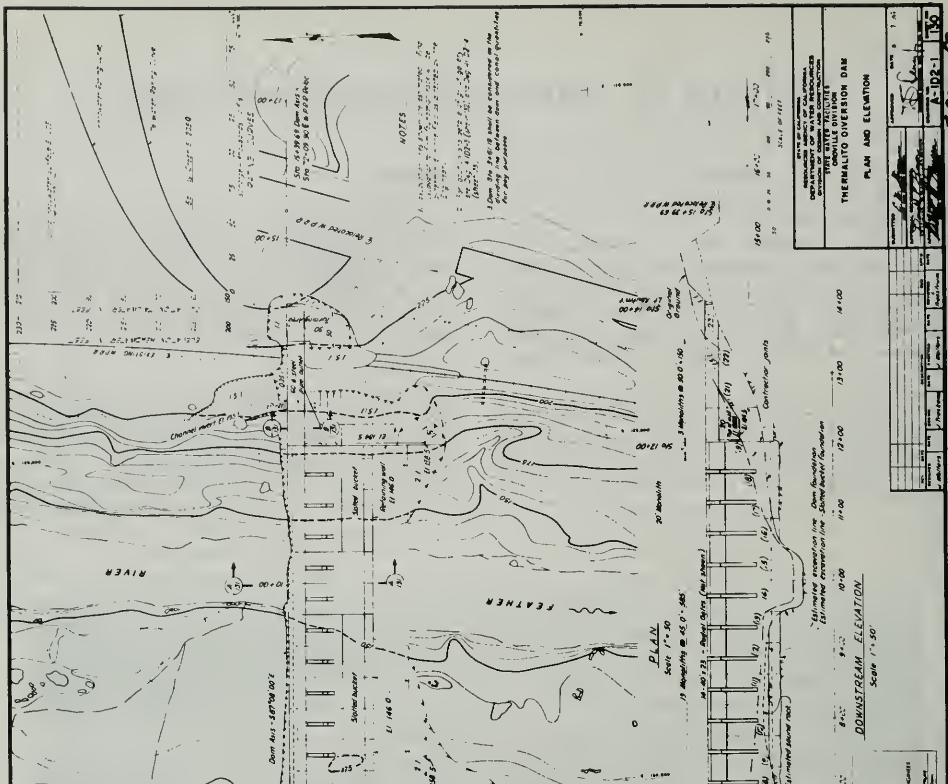

Figure 121. General Plan and Profile of Dam

work was initiated in 1961, and work on the Diversion Dam was started in October 1963 and completed in March 1968. A statistical summary of Thermalito Diversion Dam is shown in Table 10.

Regional Geology and Seismicity

The Dam site is approximately 4 miles west of Oroville Dam, and the section on regional geology and seismicity contained in Chapter V of this volume applies to the Diversion Dam as well as to Oroville Dam.

Design

Foundation

Site Geology. Rocks of the Jurassic Oregon City formation comprise the foundation of the Diversion Dam. This formation is composed of a sequence of metamorphosed volcanic and sedimentary rocks. Two rock types were exposed in the dam foundation: metaandesite, which is a massive, fine-grained, equiangular rock; and metaconglomerate, which contains volcanic cobbles in a medium- to coarse-grained ground mass. Both rock types are extremely hard, dense, and durable where fresh. Shear zones and well-developed joints cut the rock at various angles, creating a blocky or slabby appearance; however, no major shear zones were found in the foundation.

Strength. Inspection of the foundation rock, surface geologic mapping, and core hole logs indicated that foundation strength would not be a critical factor in the stability of a dam of this height. Therefore, no strength testing of the rock was performed. Grouting was necessary to consolidate the foundation and minimize seepage.

Overpour Section

Description. The spillway consists of a concrete, gravity, overpour section with a crest across the channel of the Feather River. An energy dissipator is provided at the toe of the spillway to prevent scouring of the channel and possible undermining of the Dam.

Hydrology. Spillway capacity of 320,000 cubic feet per second (cfs) was determined from a study of the economics, operational requirements, and physical limitations of the site. This capacity is in excess of the controlled releases for the standard project flood from Oroville Reservoir (150,000 cfs). Discharges in excess of 320,000 cfs will not overtop the abutments of the nonoverpour sections but will flow over the spillway bridge above the gates; however, no major damage will occur with discharges up to the peak spillway discharge of 650,000 cfs at Oroville Dam. The Ther-

TABLE 10. Statistical Summary of Thermalito Diversion Dam

THERMALITO DIVERSION DAM	
Type: Concrete gravity	
Crest elevation	233 feet
Crest width	24 feet
Crest length	1,300 feet
Streambed elevation at dam axis	105 feet
Lowest foundation elevation	90 feet
Structural height above foundation	143 feet
Volume of concrete	154,000 cubic yards
Freeboard above spillway crest	28 feet
Freeboard, maximum operating surface	8 feet
THERMALITO DIVERSION POOL	
Maximum operating storage	13,328 acre-feet
Minimum operating storage	12,090 acre-feet
Dead pool storage	5,849 acre-feet
Maximum operating surface elevation	225 feet
Minimum operating surface elevation	221 feet
Dead pool surface elevation	197.5 feet
Shoreline, maximum operating elevation	10 miles
Surface area, maximum operating elevation	323 acres
Surface area, minimum operating elevation	308 acres

SPILLWAY	
Type: Gated ogee crest with slotted-bucket energy dissipator—14 radial gates 40 feet wide by 23 feet high	
Top elevation of gates	226 feet
Ogee crest elevation	205 feet
Crest length	560 feet
Maximum probable flood inflow	650,000 cubic feet per second
Peak routed outflow	650,000 cubic feet per second
Maximum surface elevation	246 feet
INLET-OUTLET	
Edward Hyatt Powerplant	
Maximum generating release	16,900 cubic feet per second
Pumping capacity	5,610 cubic feet per second
Thermalito Power Canal	
Maximum generating flow	16,900 cubic feet per second
Maximum pumping flow	9,000 cubic feet per second
OUTLET WORKS	
To Oroville Fish Hatchery: 60-inch reinforced-concrete pipe with 60-inch slide gate shutoff on upstream side of dam—flow regulation at hatchery	
Capacity	100 cubic feet per second
River Release: 60-inch reinforced-concrete pipe—control, 42-inch fixed-cone dispersion valve—guard valve, 60-inch slide gate on upstream side of dam	
Capacity	400 cubic feet per second

malito Diversion Dam spillway rating curve is shown on Figure 122.

Structural Design. The spillway section (Figure 123, Sections D and E) was constructed of mass concrete with a 2-foot-thick surface of high-strength concrete on the spillway face. The vertical upstream face is recessed 6 feet downstream from the dam axis to reduce the volume of concrete required.

The 25-foot-radius, slotted-bucket, energy dissipator was designed to withstand the forces resulting from falling water during an occurrence of the design flood as well as to resist the tailwater loading. Uplift forces are resisted by the weight of the concrete bucket. Waterstops were placed at contraction joints to prevent the buildup of excessive uplift forces during spillway operation. Concrete gravity retaining walls at both ends of the energy dissipator contain the turbulent flow of the slotted bucket and prevent scouring of the abutments.

Thirteen 5-foot-thick piers and two 3-foot-thick training walls support the radial gate trunnion anchorage assemblies. The piers also support the bridge over the spillway section and will accommodate stoplogs. The spillway bridge, at elevation 233 feet, was designed to provide access across the Dam, serve as the hoist platform for the radial gates, support maintenance equipment, and allow placement of stoplogs. Removable railings were placed along the curb on the upstream side of the bridge to provide access to the stoplogs. Permanent railings are located on the downstream side of the bridge.

Spillway Gates and Hoists

Each of the 14 radial gates on the spillway crest consists of a curved skinplate welded to vertical ribs which in turn are welded to two horizontal girders. The loads are transmitted by radial arms which are bolted to the horizontal girders and to the trunnion assembly. The load from the gates is transmitted into the piers by prestressed anchorage assemblies. The 14 hoists are electric motor-operated, conventional, wire-rope type designed for 90 kips, with the load equally divided between the four wire ropes. The hoists can be controlled either on-site, at the local control building, or from the Oroville Area Control Center.

Stability

Stability analyses were performed on all sections of the Dam under various conditions of loading.

The monoliths are not keyed together, and no provisions were made for bonding the vertical joints. Waterstops are provided to prevent leakage through these joints.

Curtain grouting was done through a grout gallery in the Dam. Spacing and depth of the holes are described in the construction section of this chapter.

Although foundation drainage is provided, it was assumed, for conservative design purposes, that the drains would remain inoperative.

Water and pressure profiles over the spillway were calculated, and the loads resulting from the static and dynamic pressures on the crest, headwater, and tailwater and from velocity were incorporated into the stability analyses. No "over-turning factor" as such was determined; however, a sliding factor (ratio of vertical to horizontal forces) of 0.87 was calculated for the critical section. Due to the extremely uneven surface of the foundation, it was believed that sliding could not occur and, therefore, more emphasis was placed on the shear friction factor. Another factor which would resist sliding but was disregarded in the analysis was the bearing provided by placing the downstream toe of the Dam directly against the slope of the excavation.

It was assumed that earthquakes would not occur simultaneously with riverflows in excess of 150,000 cfs. Increases in stress due to a horizontal seismic acceleration factor of 0.1g were small and did not govern design in any case.

Nonoverpour Sections

A nonoverpour concrete section (Figure 123, Section F) with a 24-foot-wide crest at elevation 233 feet adjoins each side of the spillway section. The design and stability analyses were the same as for the overpour section. The crest accommodates vehicles and is provided with curbs and railings similar to the spillway bridge.

Three 60-inch-diameter outlets equipped with electrically operated slide gates are provided in the abutments. The one in the right abutment provides a water supply outlet for the Feather River Fish Hatchery of 100 cfs. The other two are in the left abutment: one providing for a minimum river release of 400 cfs, and the other for a possible future water demand. A 42-inch-diameter fixed-cone dispersion valve with an electric motor operator is located in a valve vault just downstream of the left abutment. It is used to make river releases. The valve discharge impinges onto the spillway apron parallel to the dam axis. Since the head and flow are constant, except for short periods, only on-site control was provided.

Thermalito Canal Headworks

Due to the extemely high value of head for power generation, it was imperative to minimize head loss. This was accomplished by increasing the velocity of the water at a constant rate through the approach channel.

Warped transitions were adopted both upstream and downstream from the headworks structure (Figure 121). This was done to minimize head loss through the structure and thus better meet pump-storage requirements. Sizing was based on a capacity of 16,900 cfs.

Piers were designed to support the gates, bridge, breastwall, and hoist platform and resist the resultant forces due to their respective critical loading conditions (Figure 124). Under normal loading conditions,

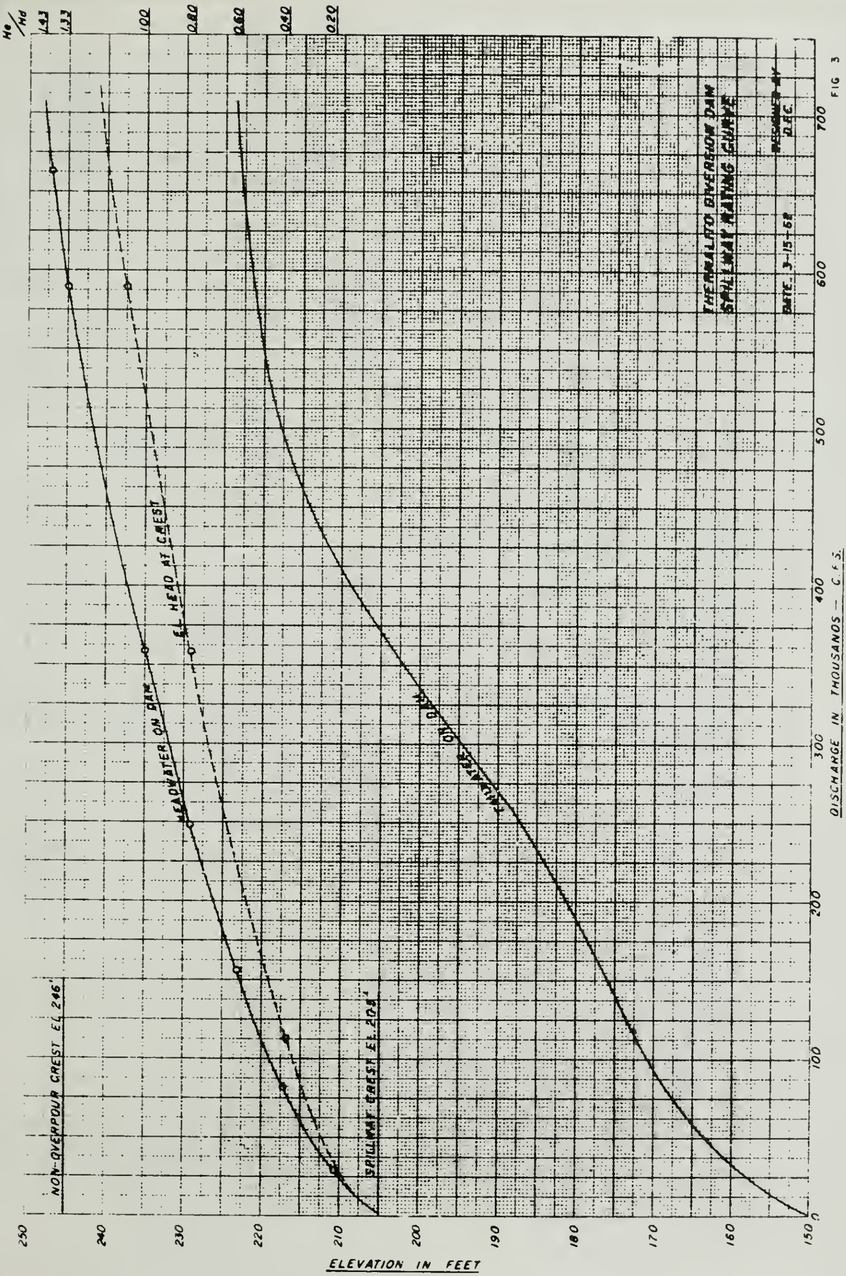

Figure 122. Spillway Rating Curve

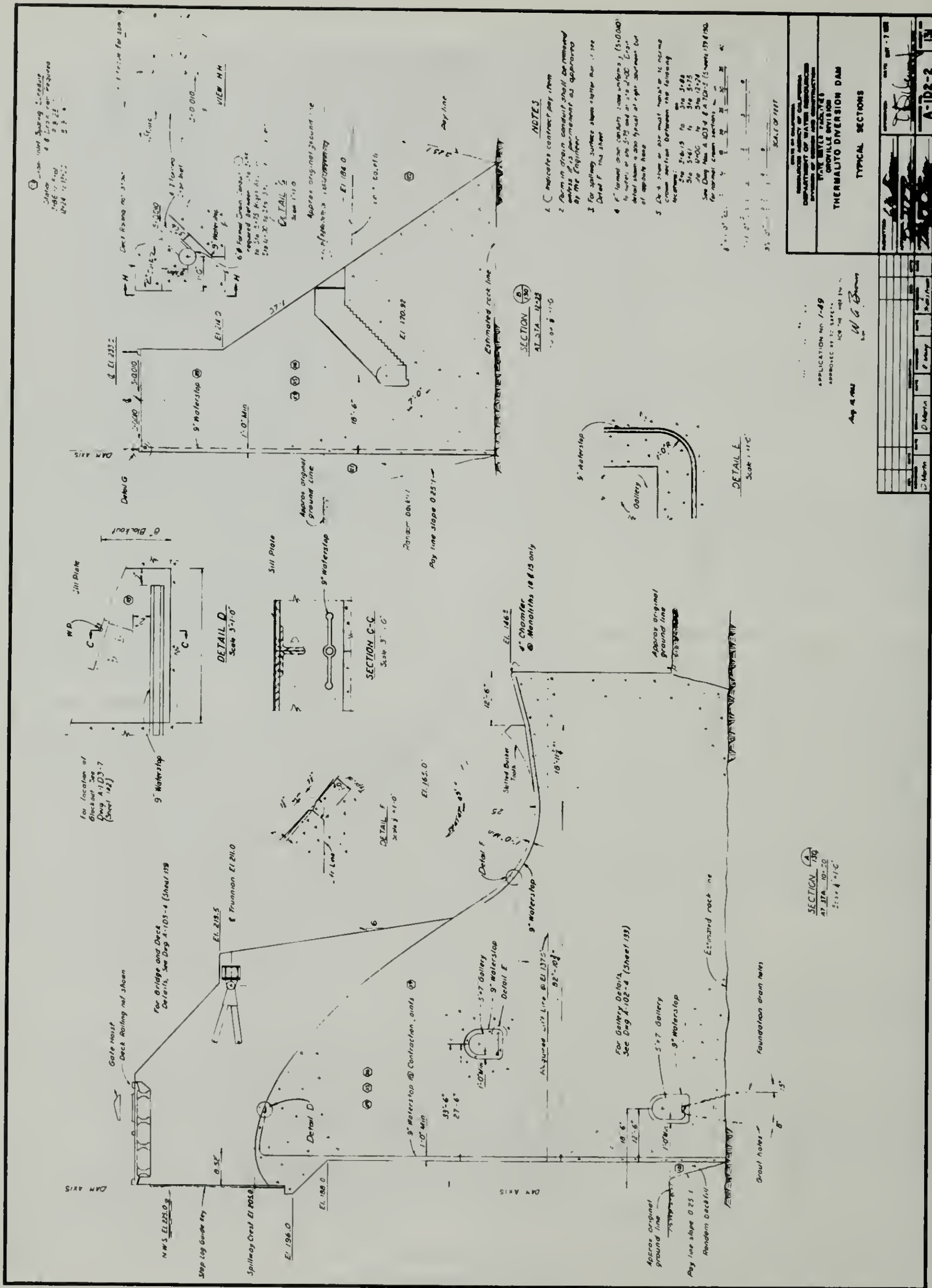

Figure 123. Spillway Sections

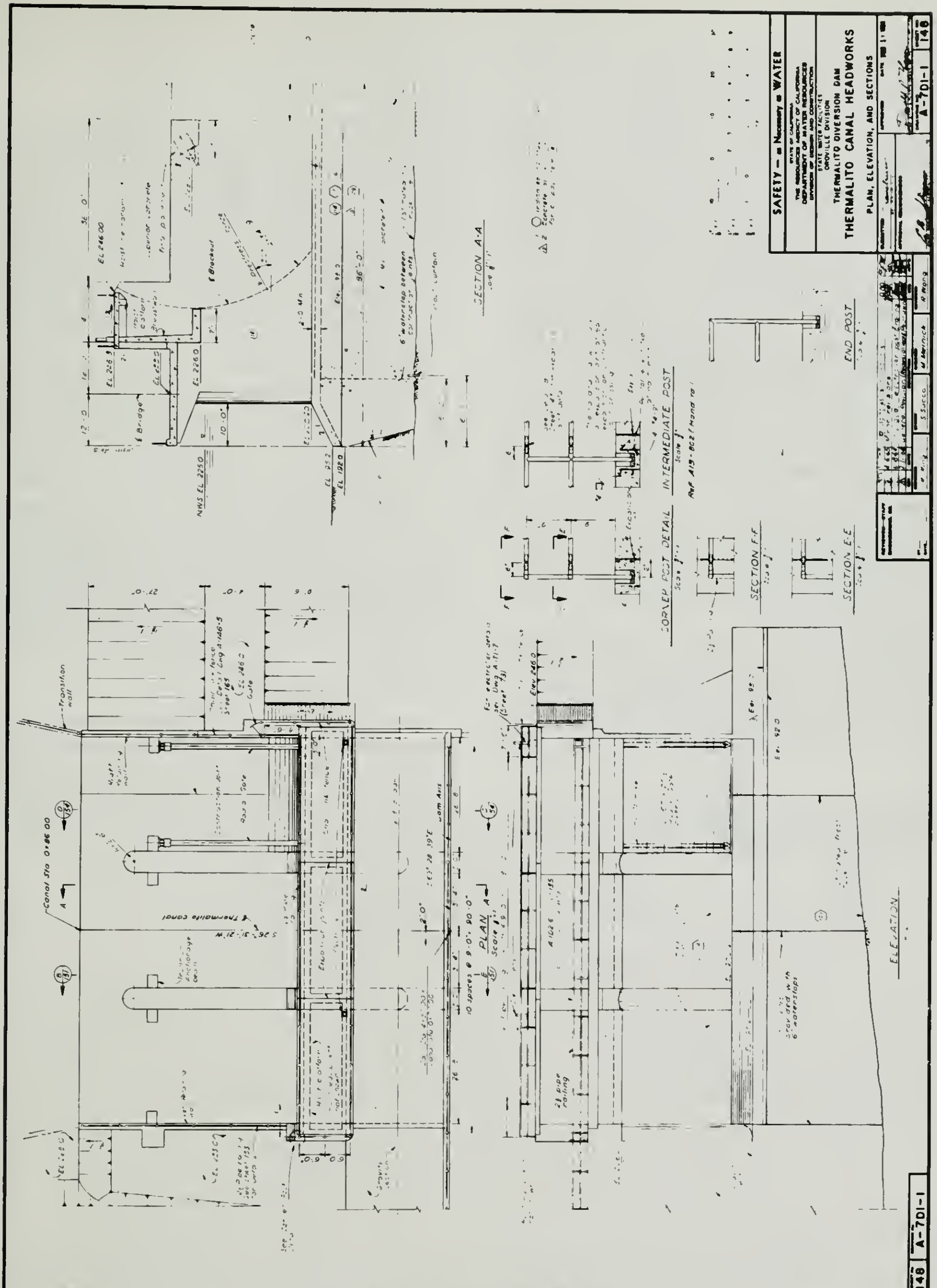

Figure 124. Power Canal Headworks—Plan, Profile, and Sections

seismic acceleration was included in the design.

Piers are 5 feet thick with vertical and horizontal reinforcing steel in both faces. Prestressed tendons are embedded to provide anchorage for the gates.

The left side of the headworks structure is flush with the face of Monolith 1 (Figure 121, Section A). A 4-foot-thick wall at the downstream face of the monolith connects the Dam to the left counterfort wall. The breastwall is formed by three reinforced-concrete vertical walls, simply supported between piers and endwalls (Figure 124, Section A). The purpose of the breastwall is to protect Thermalito Power Canal from high floodflows.

A bridge across the headworks structure provides access to the Dam and left bank of the Canal. It consists of a reinforced-concrete slab, continuous between the piers and walls. A platform, simply supported between the piers and walls, is situated at the top of the breastwall and is used to support gate hoist equipment. Live loads used for design included 100 kips for the gate, 25 kips for the hoist, 2.5 kips for the motor, and 25% of the gate weight for impact.

Radial Gates and Hoists. The three gates are of similar design to the spillway gates. The gate hoists are identical to the spillway gate hoists except for the spacing of the wire ropes. These gates were designed to be used either fully open or fully closed.

Foundation. The base of the canal headworks structure extends to sound rock of the same quality as the foundation of the Dam. Hydrostatic pressure is relieved by the placement of weepholes in the upstream transition walls and by underdrains emptying into the River on the downstream side of the structure.

Stability Analysis. Stability of the headworks structure was investigated with criteria similar to the Dam. Foundation conditions were considered comparable to the Dam. Grouting was performed directly from the surface and similar foundation drainage was provided.

Embankment Section

Description. The embankment section (Figure 123, Section J) of the Dam between the Canal and the right abutment provides freeboard only. The downstream toe is at elevation 228 feet, 3 feet above normal water surface of the Power Canal, and the crest is at elevation 246 feet. In addition to riprap for protection from wave action, a 4-foot concrete parapet with a top elevation of 250 feet is situated on the upstream side of the crest of the embankment. The crest serves as an access road to the hoist platform of the canal headworks.

Construction Materials. Pervious material for the embankment was obtained from foundation excavation and supplemented with dredger tailings from downstream borrow areas that were designated for use in construction of Oroville Dam. Impervious material was obtained from abutment stripping.

Stability Analysis. Stability of embankment sections was determined by the Swedish Slip Circle method of analysis, assuming reservoir-level criteria similar to that assumed for the gravity dam. A seismic acceleration factor of 0.lg was incorporated into the design forces. The slope stability safety factor did not fall below 1.1 for the most adverse condition analyzed.

Construction

Contract Administration

General information for the major contracts for the construction of Thermalito Diversion Dam is shown in Table 11. Thermalito Diversion Dam and its appurtenances were constructed as part of the Oroville Dam contract (Specification No. 62-05). Associated mechanical and electrical equipment was furnished under separate contracts.

TABLE 11. Major Contracts—Thermalito Diversion Dam

	Specification	Low bid amount	Final contract cost	Total cost-change orders	Starting date	Completion date	Prime contractor
Oroville Dam[1]	62-05	$120,863,333	$135,336,156[1]	$7,503,716	8/13/62	4/26/68	Oro Dam Constructors
Furnishing Slide Gates	64-25	20,876	16,081	--	6/29/64	3/ 4/69	Rodney Hunt Machine Co.
Furnishing Fixed Dispersion Cone Valve and Operator	64-26	24,004	25,255	838	6/25/64	10/18/66	Willamette Iron and Steel Co.
Furnishing Radial Gates and Hoists	64-43	751,000	778,830	--	12/11/64	9/22/66	Berkeley Steel Construction Co., Inc.
Electrical Equipment	65-47	103,963	155,393	38,448	1/12/66	4/15/68	Abbett Electric Corp.
Furnishing Stop Logs and Lifting Beam	67-40	43,755	45,270	--	7/17/67	1/ 4/68	Guntert and Zimmerman

[1] Thermalito Dam constructed under this contract. Estimated final cost $9,000,000.

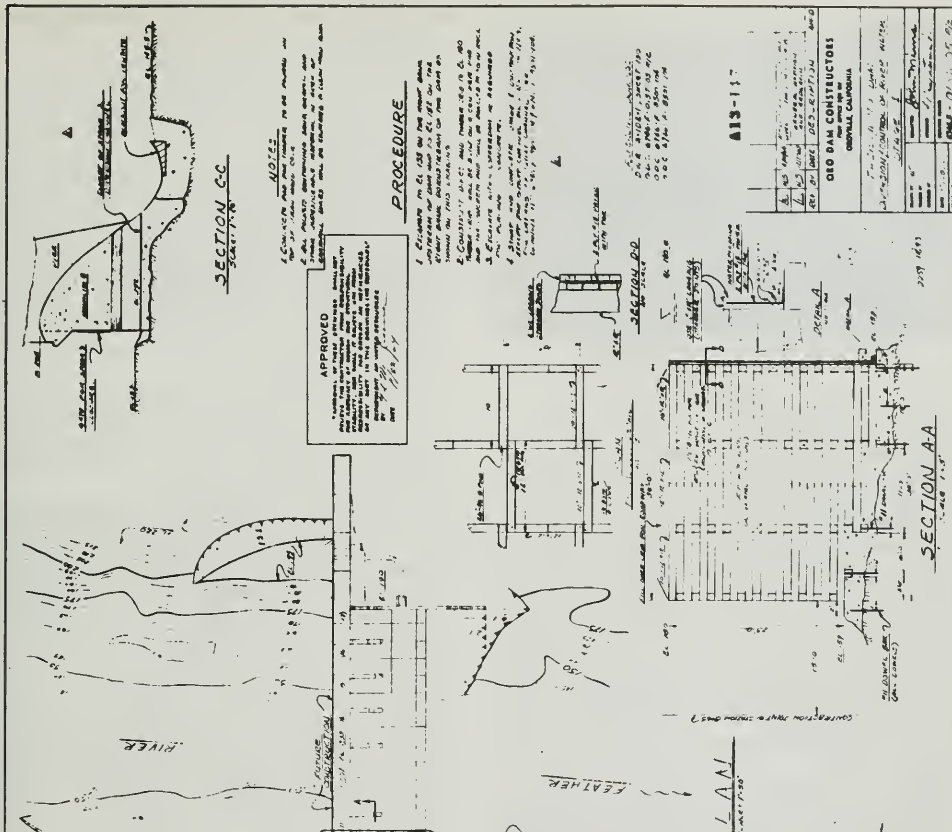

Figure 125. West Bank Diversion Plan

Figure 126. Channel Bypass (Aerial View)

Diversion and Care of River

West Bank. First-stage construction work for stream diversion consisted of an embankment and earth-filled timber crib which enclosed the right abutment and Monoliths 1 through 9 (Figures 125 and 126). This offered flood protection up to elevation 180 feet. A diversion channel through Monolith 8 also was excavated to prepare for second-stage diversion. The earth-filled timber crib was constructed in the foundation area of Monolith 10, and the remainder of the enclosure was an earth embankment extending upstream and downstream of the crib and tied into the right bank. This structure served as a precautionary measure in the event of a flood and did not actually encroach on, or divert, the normal flow of the Feather River.

Channel. Second-stage stream diversion (Figures 127 and 128) called for enclosure of the foundation area of Monoliths 10 through 19 and diversion of riverflow through a 15-foot by 15-foot opening through Monolith 8. Flood protection was to elevation 180 feet.

Timber cribbing was set adjacent to Monolith 9, both upstream and downstream. The earth section of the cofferdam was composed of rockfill ballast, clay core, and sheet piling driven vertically through the core to sound rock (Figure 127). It was constructed to extend out into the River as far as practical without losing material by erosion. Prior to the closure attempt, rock was stockpiled at the River's edge on both sides near the upstream closure point. The actual closure was made from the left bank. To accomplish this, all available earth-moving equipment was utilized. Progress was slow due to erosion of embankment material nearly as fast as it was trucked in. At one time, large rock was positioned by the high line to check the loss of fill material. Work continued around the clock for three days before the closure was successful. When the River was finally diverted, the remaining cofferdam construction was routine. Two 36-inch dewatering pumps were placed into operation near the downstream end of the cofferdam to take care of seepage water.

Closure. Third-stage diversion consisted of closing the 15-foot by 15-foot opening through Monolith 8, thus forcing the River to flow over the completed spillway structure.

Sudden closure of the 15-foot by 15-foot opening would have been a construction advantage but, due to flow requirements downstream of the Dam, this procedure was not permissible. As a result, the bulkhead gate was lowered in increments allowing gradual filling of the reservoir and at the same time, to a disadvantage, increasing steadily the hydrostatic pressure on the gate. Movement of the gate continued in this manner to within 16 inches of closure and, at that point, it became immovable.

Concrete blocks were then set upstream in the path of the opening. Progressively smaller rocks were dumped over the blocks, followed with application of earth and straw, until the flow was reduced enough to allow men to enter the opening.

To control the remaining flow, a box chamber was constructed along the downstream side of the bulkhead at the invert of the opening by welding two 15-foot by 2-foot by ½-inch steel plates together and to the bulkhead. A 10-inch gate valve was mounted on top at the end of a 5-foot pipe riser. A line then was connected and extended to the downstream end of the Monolith 8 opening, making it possible to open the valve and relieve hydrostatic pressure.

Water leaking through the chamber at the invert level was collected by two French drains positioned along the invert and piped into the access gallery prior to placing the first lift of concrete. Before placing the second lift, the 10-inch gate valve was closed and the downstream section of pipe removed.

After the third and last lift was placed and cured, grout was pumped into the French drains and also through a preinstalled grout system located in the crown of the 15-foot by 15-foot opening to complete the closure.

Cofferdam sheet pilings, which were easily pulled, were salvaged by the contractor, and the more difficult pieces were left in place at the time of the first high water. Rains came in late October and early November, flooding the construction area on November 2, 1964. More flooding occurred on November 9 and wet weather continued intermittently into December.

Heavy rain occurred December 18, 1964 and continued through December 23, resulting in a record peak flow of 253,000 cfs upstream of Oroville Dam. The peak flow at the Diversion Dam was reduced to 187,000 cfs by the restrictive action of the diversion tunnels at Oroville Dam. The highest elevation of the River at the Diversion Dam site was 206.7 feet.

When the River subsided, the remaining cribbing that had not been removed had been washed out. The downstream cofferdam embankment was eroded to

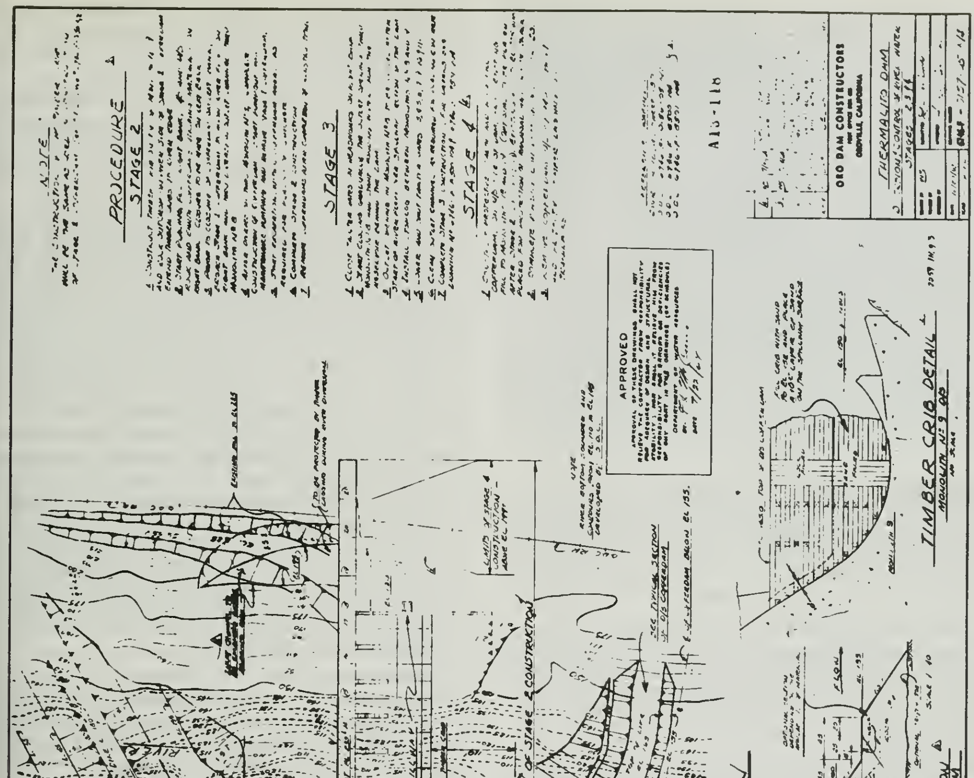

Figure 127. Channel Bypass, Closure, and East Bank Diversion Plan

Figure 128. Cofferdam Closure

elevation 150 feet and scattered along the river channel. A few pieces of twisted sheet piling protruding through the muck and debris marked the location of what was once the cofferdam. In midsummer of 1965, the upstream portion of the cofferdam was leveled to elevation 175 feet by the contractor.

East Bank. The fourth stage of stream diversion consisted of the placement of an earth dike to provide flood protection for future closure of the railroad bypass, which was located at the left abutment of the Dam.

Foundation

Excavation. At the abutments, the overlying soils and weathered material were removed and wasted in the designated waste area. The exposed rock was found to be badly fractured. Further excavation by drilling and shooting was necessary to reach a suitable foundation.

Streambed excavation consisted of removing river gravels that varied from 15 to 25 feet in depth. The exposed rock provided a satisfactory foundation, and it was not necessary to drill and blast as had been expected. A power shovel, end-dump trucks, and tractors were used to excavate and transport waste materials to a designated area upstream of the Diversion Dam. All material wasted within the reservoir was placed below dead storage, elevation 175 feet.

Canal headworks excavation included a portion of the Power Canal, upstream and downstream transition wall foundations, headworks foundation, and the intake channel. The excavated material was used in the first-stage cofferdam.

Preparation. The foundation at Thermalito Diversion Dam is a blocky, well-jointed, metavolcanic rock which was easily cleaned. The only difficult area was a fractured trough formed by two intersecting shears that cut diagonally across Monoliths 3, 4, and 5. The fractured material was excavated to a depth equal to the width of the shear zones, which varied from 3 to 5 feet.

Foundation cleanup was done using high-pressure water hoses to remove most of the loose material. Loose and detached rocks were removed by prying. Air-water jets and pneumatic siphons were used for final cleanup.

Grouting. Blanket grouting was used to consolidate areas of weathered or fractured rock and to seal open cracks and shear zones. Grout holes at spacings of 5 to 35 feet were drilled from 10 to 30 feet deep. Spacing and depth of holes were governed by the nature and extent of the foundation defects encountered. Most of the holes were inclined to penetrate steeply dipping seams and shear zones a few feet below the foundation surface. Holes normally were tested and grouted at 30 pounds per square inch (psi).

Curtain grouting was accomplished from a 5-foot by 7-foot horseshoe gallery within the Dam between Station 5+70 and Station 12+25 (Figure 121) and from the surface of the excavated foundation beyond the limits of the gallery. The maximum spacing of the holes was 10 feet. Zone 1 holes, drilled to a maximum depth of 25 feet, were stage-grouted at pressures up to 100 psi. Zone 2 holes, drilled to a maximum depth of 75 feet, were stage-grouted at pressures up to 200 psi.

Embankment Construction

Description. The embankment provides flood freeboard only. No unusual problems were encountered in the placement of embankment. A major portion of the material was obtained in borrow areas close to the job site. The embankment material was divided into five separate classifications.

Random Backfill. Random backfill with a maximum size of 24 inches, free of organic material, was placed on the upstream and downstream faces of the embankment. This material also was used as fill for the parking and storage area adjacent to the upstream face of the Dam and the service road adjacent to the downstream face of the Dam.

Impervious Core. Impervious core material was placed from the canal headworks to the right abutment. This material conformed to Oroville Dam Zone 1 embankment described in Chapter V of this volume.

Select Backfill. Select backfill was placed behind the retaining walls. The material conformed to Oroville Dam Zone 3 material with a maximum size of 12 inches. Zone 3 requirements also are described in Chapter V of this volume.

Consolidated Backfill. Consolidated backfill was used in the trench for the 60-inch water pipe leading to the Feather River Fish Facilities. This material was granular with a gradation specified for concrete sand.

Riprap. Riprap was hard, dense, durable, rock fragments free from organic, decomposed, and weathered material. It was obtained from canal excavation previously stockpiled and was placed on the upstream slope in such a manner that a well-graded blanket was produced.

Concrete Production

Construction Phases. Construction of Thermalito Diversion Dam was accomplished in four phases: (1) construction of Monoliths 1 through 9 and the canal headworks, (2) construction of Monoliths 10 through 20, (3) plugging the diversion opening through Monolith 8, and (4) filling the railroad pass through Monolith 20. During the second phase, Monoliths 11 through 15 were left low during the winter of 1964–65 for passage of floodflows. After the River had subsided sufficiently in the spring of 1965, these low monoliths were completed to grade.

Concrete Mixing and Materials. The concrete aggregates were stored in seven 320-cubic-yard timber bunkers. Clam gates at the bottom of the bunkers regulated the flow of the material onto a 30-inch conveyor belt. The cement used was Type II (low alkali) supplied by Calaveras Cement Company of Redding, California. Ice used as part of the mixing water for cooling the mass concrete was produced by an ice plant located on the job site.

The bulk of the mass concrete was a 2½-sack mix with 6-inch maximum size aggregate. Mixes with up to four sacks of cement were used on a selective basis (Figure 129).

The last day the batch plant was used to mix concrete for the Diversion Dam was October 6, 1965. From September 18, 1965 through November 9, 1965, concrete was transported from Oroville Dam batch plant. Transportation from this plant to the job site was by transit mix trucks.

For the remaining small-volume concrete items (curbs, control house, cable trench, radial-gate walls, and sill-plate blockouts), a portable concrete mixer was set up on the right abutment near the high line.

Concrete for the diversion opening plug in Monolith 8 and the railroad bypass in Monolith 20 was obtained from Oroville Dam spillway batch plant.

Concrete Placing and Formwork. Transportation of the concrete was accomplished in two stages. Eight-cubic-yard, rail-mounted, side-delivery, transfer cars transported the concrete from the batch plant to the high-line dock. Attached to the high line was an 8-cubic-yard bucket, which received the concrete from transfer cars and conveyed it to the placement area.

From October 1964 to May 1965, placing operations continued on a three-shift basis. With the beginning of summer weather and higher daytime temperatures, placing operations were restricted by state specification to two hours before sunset and two hours after sunrise. The contractor developed a nozzle that produced a fine mist or fog. Operation of this device over the concrete placement area reduced the surrounding air temperature by 10 to 15 degrees, allowing the resumption of a three-shift concrete operation.

The mass concrete in the Dam was placed in 7½-foot lifts. Steel forms cantilevered from the previous lifts were used on the spillway monoliths. The cantilever supports were supplemented by rods welded to pins embedded in the previous lift.

The slotted bucket was formed by using screed rails to hold the 2-foot by 6½-foot form panels in place. As placement progressed, more panels were added until the surface was completely formed. After the concrete had set sufficiently, the forms were removed in the order of assembly.

Wooden forms for the dentates were constructed at the job site.

Concrete was cured for ten days by continuous application of water with rainbird sprinklers and spray pipes. Concrete surfaces subject to direct sunlight were covered with burlap and kept wet by sprinklers.

The bridge over the spillway section has 56 prestressed beams. For fabrication, the contractor constructed a jig on the right abutment within reach of the high line for ease of handling. Four beams were cast at a time, using the post-tension method for prestressing.

Radial Gates and Hoists

Spillway Gates. Spillway gates are 40 feet wide by 23 feet high. The gates were assembled by setting the two horizontal beams on a level H-beam jig. The four skinplate sections were then assembled and bolted loosely to the horizontal beams with the upstream face up. The skinplate assembly then was squared up and dogged where necessary before butt welding the skinplate sections together. After the gates were fully assembled, the joints between the horizontal beams and the vertical ribs and the edges of the back-up strip on the downstream side of the plate were welded by a single filler pass. The gate was turned over so the downstream face of the skinplate was up. The diagonal braces between the beams then were welded in place, the bottom struts were welded bracing the gate bottom, and the gate arms were bolted on.

While setting the side seals to the required ⅛-inch compression tolerance, it was found that the $^{15}/_{16}$-inch adjustment of the side-seal angle bars did not provide the proper seal compression. The slots were elongated ⅜ of an inch for additional adjustment.

Canal Headworks Gates. The assembly and installation of the 26.67-foot-wide by 25.8-foot-high canal gates were similar to the spillway gates, except that the arms were bolted on after the skinplate assembly was in place in the canal bays. The presence of the breastwall (Figure 124) prevented setting the gate as a unit.

Hoists for Radial Gates. The first radial-gate hoist unit delivered to the job site was mounted on H-beams

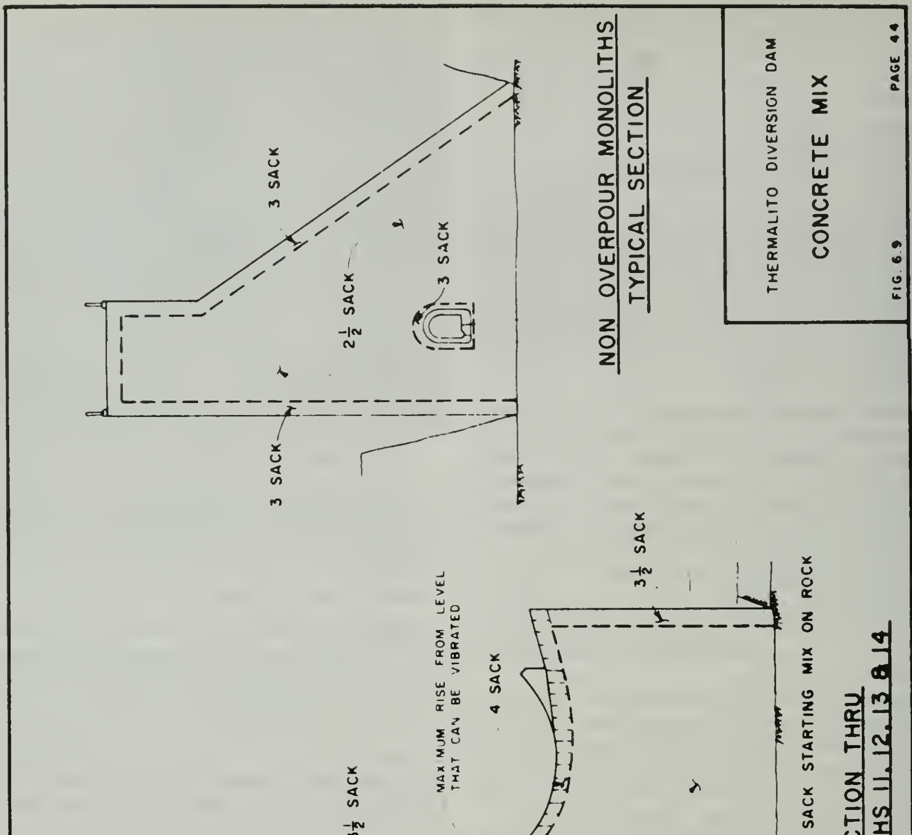

Figure 129. Location of Concrete Mixes Used in Dam Structure

and used as a portable test apparatus. Each gate was checked for operation, and final adjustments were made on the wall and sill plates. The plates then were secured with second-stage concrete. As the remaining hoists arrived, they were installed, serviced, and checked for operation.

Trunnion Beams and Stressing Operation. Trunnion beams were slipped over the tendons positioned in the piers and held to the correct elevation by field-fabricated pipe stands. The center section of the trunnion beams was to be filled with grout. Instead, two 3-inch holes were cut in the two horizontal stiffener plates and later filled with beam encasement concrete.

The trunnion beam tendons were post-tensioned by anchoring the jacking end of the tendon with grip nuts. This method of tendon anchorage proved satisfactory, as any loss of stress through seating of the wedge could be checked by returning the jack to the required pressure and checking the tightness of the nut.

Slide Gates and Operators. Three 60-inch slide gates were installed on the upstream face of the Dam. The one on the right abutment near the canal headworks releases water to the Feather River Fish Hatchery, and the other two are mounted side by side near the left abutment. One is a guard valve for the fixed-cone dispersion valve, and the other is connected to a pipe stub for future use. The gates are of the rising stem type provided with stem extensions and their installation was routine.

Other Installations

Installation of the outlet conduits, fixed-cone dispersion valve, sump pump system, ventilating system, and electrical connections also was routine.

Reservoir Clearing

Clearing of the Dam site and diversion pool area consisted mainly of removing scrub pine and oak.

Instrumentation

Instrumentation in Thermalito Diversion Dam consists of 64 foundation drains that are monitored quarterly and three crest monuments that are monitored annually.

BIBLIOGRAPHY

Amorocho, J. and Babb, A., "Thermalito Diversion Dam Model Studies", Department of Irrigation, University of California, Davis, 1963.

California Department of Water Resources, Specification No. 62-05, "Specifications Proposal and Contract for Construction of Oroville Dam", Addendum No. 1, 1962.

_______, Specification No. 64-25, "Specifications Bid Instructions and Bid for Furnishing Slide Gates for Thermalito Diversion Dam", 1964.

_______, Specification No. 64-26, "Specifications Bid Instructions and Bid for Furnishing Fixed Dispersion Cone Valve and Operator for Thermalito Diversion Dam", 1964.

_______, Specification No. 64-43, "Specifications Bid Instructions and Bid for Furnishing Radial Gates and Hoists for Thermalito Diversion Dam", 1964.

_______, Specification No. 65-47, "Specifications Bid and Contract for Electrical Equipment for Thermalito Diversion Dam," 1965.

_______, Specification No. 67-40, "Specifications Bid Instructions and Bid for Furnishing Stop Logs and Lifting Beam for Thermalito Diversion Dam", 1964.

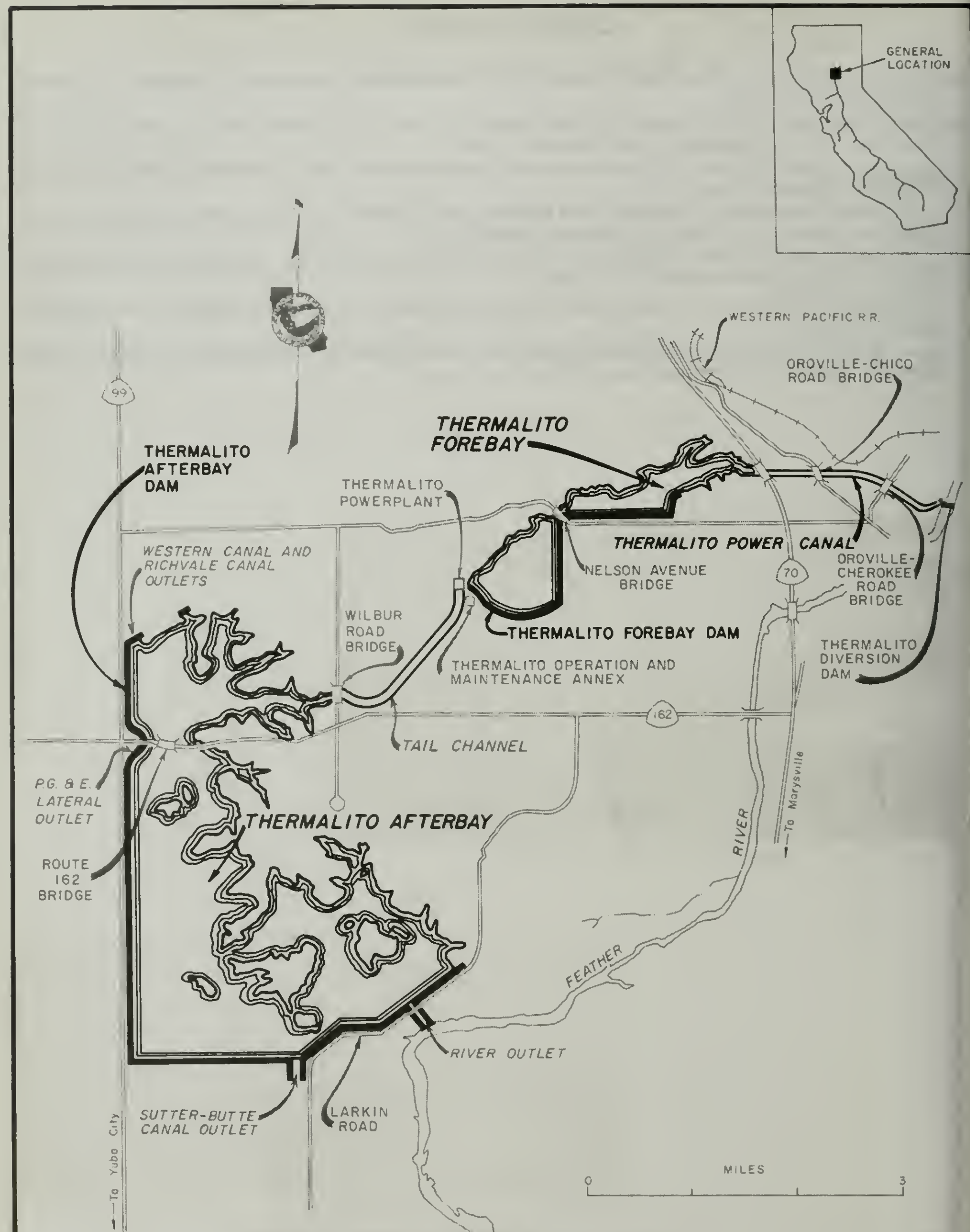

Figure 130. Location Map—Thermalito Forebay and Afterbay

CHAPTER VII. THERMALITO FOREBAY, AFTERBAY, AND POWER CANAL

General

Description and Location

Thermalito Forebay is an 11,768-acre-foot offstream reservoir contained by Thermalito Forebay Dam on the south and east and by Campbell Hills on the north and west. It is located approximately 4 miles west of the City of Oroville.

Thermalito Afterbay is a 57,041-acre-foot offstream reservoir contained by Thermalito Afterbay Dam on the south and west and by higher natural ground on the north and east. It is located approximately 6 miles southwest of the City of Oroville.

Thermalito Power Canal, which also is included in this chapter, is a concrete-lined canal about 10,000 feet in length. It conveys water in either direction between Thermalito Diversion Dam and Thermalito Forebay for power generation or pumping at Edward Hyatt and Thermalito Powerplants.

Excess material from the Canal and other excavations was placed in recreation areas and shaped to make the areas more productive for recreation use.

Nearest major roads are State Highway 99 bordering the west side of the Afterbay, State Highway 70 to the east of the Forebay and over the Power Canal, and State Highway 162 passing south of the Forebay and across the Afterbay (Figures 130, 131, and 132). Statistical summaries of the dams and reservoirs are shown in Tables 12 and 13, and area-capacity curves are shown on Figures 133 and 134.

Figure 131. Aerial View—Thermalito Forebay

Figure 132. Aerial View—Thermalito Afterbay

TABLE 12. Statistical Summary of Thermalito Forebay Dam and Forebay

THERMALITO FOREBAY DAM

Type: Homogeneous and zoned earthfill

Crest elevation	231 feet
Crest width	30 feet
Crest length	15,900 feet
Lowest ground elevation at dam axis	170 feet
Lowest foundation elevation	140 feet
Structural height above foundation	91 feet
Embankment volume	1,840,000 cubic yards
Freeboard, maximum operating surface	6 feet

THERMALITO FOREBAY

Maximum operating storage	11,768 acre-feet
Minimum operating storage	9,936 acre-feet
Dead pool storage	15 acre-feet
Maximum operating surface elevation	225 feet
Minimum operating surface elevation	222 feet
Dead pool surface elevation	185 feet
Shoreline, maximum operating elevation	10 miles
Surface area, maximum operating elevation	630 acres
Surface area, minimum operating elevation	592 acres

SPILLWAY

No spillway necessary

The capacity of Thermalito Powerplant bypass exceeds the peak maximum probable flood inflow. Should the bypass fail to operate, the entire volume of the maximum probable flood could be contained within the freeboard above the maximum operating elevation.

INLET-OUTLET

Thermalito Powerplant	
Maximum generating release	16,900 cubic feet per second
Pumping capacity	9,000 cubic feet per second
Thermalito Power Canal	
Maximum generating flow	16,900 cubic feet per second
Maximum pumping flow	9,000 cubic feet per second

OUTLET

Thermalito Powerplant Bypass	
Capacity	10,000 cubic feet per second

TABLE 13. Statistical Summary of Thermalito Afterbay Dam and Afterbay

THERMALITO AFTERBAY DAM

Type: Homogeneous earthfill

Crest elevation	142 feet
Crest width	30 feet
Crest length	42,000 feet
Lowest ground elevation at dam axis	105 feet
Lowest foundation elevation	103 feet
Structural height above foundation	39 feet
Embankment volume	5,020,000 cubic yards
Freeboard, maximum operating surface	5.5 feet

THERMALITO AFTERBAY

Maximum operating storage	57,041 acre-feet
Minimum operating storage	2,888 acre-feet
Dead pool storage	753 acre-feet
Maximum operating surface elevation	136.5 feet
Minimum operating surface elevation	123 feet
Dead pool surface elevation	113 feet
Shoreline, maximum operating elevation	26 miles
Surface area, maximum operating elevation	4,302 acres
Surface area, minimum operating elevation	2,190 acres

SPILLWAY

No spillway necessary

The river outlet capacity of 17,000 cubic feet per second exceeds the peak maximum probable flood inflow. Should the river outlet gates fail to operate, the entire volume of the maximum probable flood from both the Forebay and Afterbay could be contained within the freeboard above the maximum operating surface elevation.

INLET-OUTLET

Tail channel	
Maximum generating flow	16,900 cubic feet per second
Maximum pumping flow	9,000 cubic feet per second

OUTLET WORKS

River outlet: Gated structure through dam—control, five 14-foot by 14-foot radial gates
Capacity — 17,000 cubic feet per second

Sutter Butte outlet: Four 7-foot-wide by 6-foot-high rectangular conduits—control by slide gates on headworks—discharge over measuring weir into open channel
Design delivery — 2,300 cubic feet per second

PG&E lateral outlet: One 30-inch-diameter reinforced-concrete conduit—control by slide gate in wet well—discharge into stilling basin with measuring weir
Design delivery — 50 cubic feet per second

Richvale Irrigation District outlet: Three 72-inch-diameter reinforced-concrete conduits—control by slide gates on headworks—discharge through Dall flow tubes into open channel
Design delivery — 500 cubic feet per second

Western Canal outlet: Five 96-inch-diameter reinforced-concrete conduits—control by slide gates on headworks—discharge through Dall flow tubes into open channel
Design delivery — 1,200 cubic feet per second

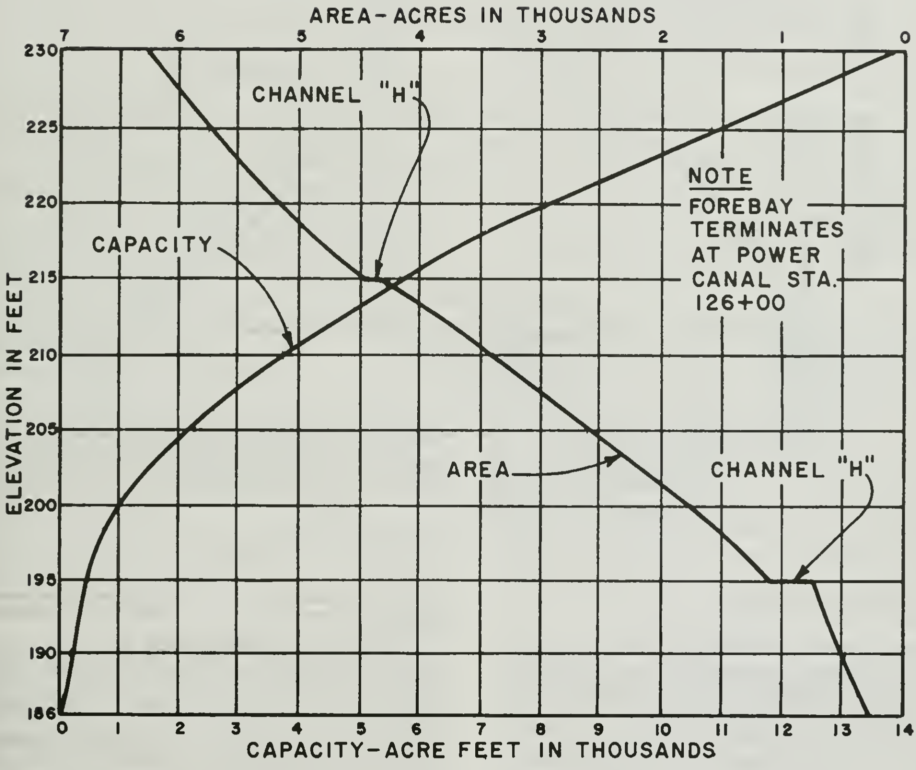

Figure 133. Area-Capacity Curves—Thermalito Forebay

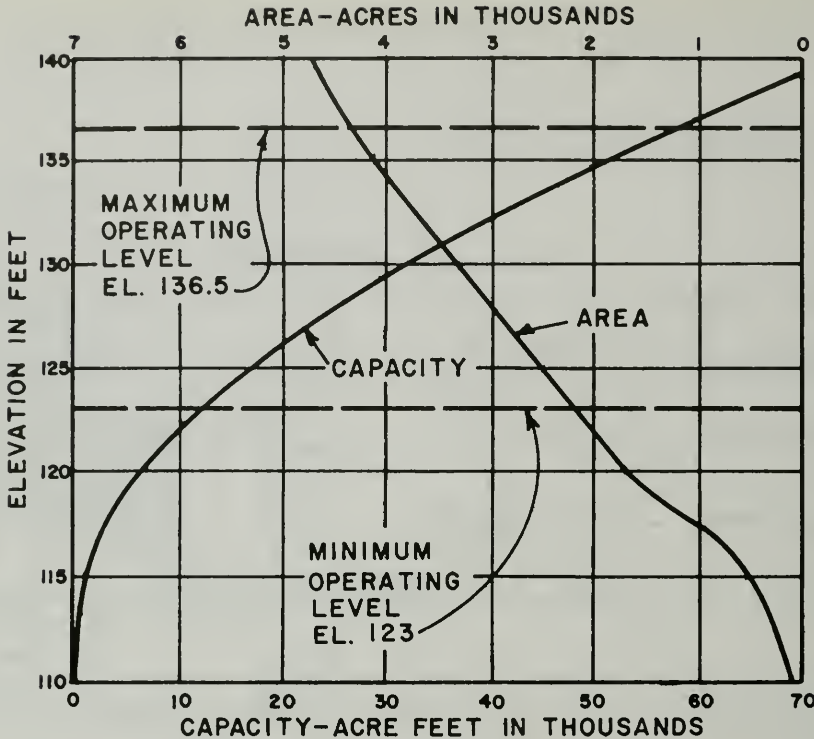

Figure 134. Area-Capacity Curves—Thermalito Afterbay

Purpose

The purposes of the Forebay are to convey generating and pumping flows between Thermalito Power Canal and Thermalito Powerplant, provide regulatory storage and surge damping for the Hyatt-Thermalito power complex, and provide recreation. Since Edward Hyatt and Thermalito Powerplants operate in tandem and are hydraulically matched, the regulatory storage is utilized only for start-up and shutdown flow mismatch.

The purposes of the Afterbay are to provide storage for the water required by the pumpback operation to Lake Oroville, provide power system regulation, produce uniform flow in the Feather River downstream from the Oroville-Thermalito facilities, and provide recreation. Outlets are provided to the Feather River and to furnish water to local districts where service from the River was interrupted by construction of the reservoir.

Chronology

Early in the 1950s, the original concept for power development downstream from Oroville Dam was to construct a number of small dams and plants within the Feather River channel. This concept later was modified to provide a diversion dam, a power canal, a power plant, and an offstream afterbay. A major consideration involved in selecting the offstream afterbay was that surges in the Feather River past the City of Oroville which would be caused by power plant releases could not be overcome in the onstream afterbay schemes. By 1957, a forebay was proposed for the development. In economic studies which followed, it was determined that any of three alternatives could be constructed for about the same cost: (1) the forebay as built, (2) a smaller forebay southwest of the Nelson Avenue Bridge, or (3) the originally proposed canal without any forebay. The forebay as built was selected because it could provide more operational flexibility, drainage problems could be minimized, and more recreation could be provided. A pumped-storage concept for the power facilities had been proposed in 1957 but was not considered in these alternative studies. Eventual adoption in 1958 of pumped storage and raising the crest of the Afterbay Dam by 5 feet to provide necessary additional storage did not affect the proposed forebay design.

Plans for Thermalito Forebay and Afterbay were completed in June 1965, a construction contract was awarded on October 18, 1965, and final inspection of the completed work was made on April 18, 1968. The Power Canal was constructed under a separate contract which was awarded on September 8, 1965. Work on this facility was completed in October 1967.

Within months after storage began in the Afterbay (November 1967), high piezometric levels were observed along U.S. Highway 99 and farther west. In February 1968, the water level in the reservoir was lowered to about elevation 119 feet, and a program of exploration and reservoir bottom sealing was initiated. As the work accomplished under this program later was found to be ineffective, a number of pumped wells were installed along the west and south sides of the reservoir. Operation of this system of wells, termed the Afterbay Ground Water Pumping System, successfully lowered the piezometric level in the surrounding land.

Regional Geology and Seismicity

The oldest rock exposed in the forebay and afterbay area is the Older Basalt formation, a series of basalt flows which form the Campbell Hills. Three separate basalt flow members of the formation have been designated: Lower, Middle, and Upper. Two interflow layers of volcanic sediments separate the flows.

Compact sediments of the Red Bluff formation overlie the basalt flows and form the dam foundations throughout most of the contract area. The Red Bluff formation is a flood-plain deposit ranging in classification from clay to gravel. Materials near the surface have weathered in place to clay, clayey sand, and clayey gravel. Leaching has created discontinuous iron-cemented horizons a few feet beneath the surface. Locally, streams have eroded through the surficial weathered zones, and excavations have shown that Red Bluff materials are cleaner at depth.

Unconsolidated fine- to coarse-grained alluvium occurs along stream channels such as Ruddy Creek in the Forebay and Grubb Creek in the forebay and tail channel areas. Approximately 150 acres of alluvium, known as Columbia fine sandy loam (silt and lean clay), lie within the southeast corner of the Afterbay. Most of the Columbia soil is underlain at 10 to 15 feet by relatively clean sandy gravel. Dredge tailings, consisting of loose gravel overlying sands, cover the area between the Columbia soil and the Feather River and extend in both directions along the River for a distance of approximately 5 miles.

The section on seismicity contained in Chapter V of this volume applies to the structures discussed in this chapter as well as to Oroville Dam.

Design

Dams

Description. Thermalito Forebay Dam is approximately 15,900 feet long with a maximum height of 91 feet and an average height of 25 feet. The 30-foot-wide crest is at elevation 231 feet, providing a 6-foot freeboard. This dam involves two relatively high sections joined by low reaches of embankment. The high portions are termed: main dam, located adjacent to Thermalito Powerplant, and Ruddy Creek Dam, located in the watershed near the Power Canal terminus. The plan of the main dam is shown on Figure 135, and sections and details of the entire dam are shown on Figures 136 and 137.

Thermalito Afterbay Dam is approximately 42,000 feet long with a maximum height of 39 feet and an average height of 24 feet. The 30-foot-wide crest, at elevation 142 feet, provides a 5½-foot freeboard at maximum operating pool. There is a 12-foot-high saddle dam approximately 1,000 feet in length at the northwest corner of the Afterbay. Sections and details of the Afterbay Dam are shown on Figures 138 and 139.

The design intent was to construct basically homogeneous dams with locally available materials. Zoned embankments with more dredger tailings and basalt rockfill were considered but deemed too complicated for the heights involved. Therefore, slopes were flattened and more local material was used.

The portion of the main Forebay Dam adjacent to the powerplant wingwall is the only location where a zoned section was required. The water side slope was steepened between the plant and Station 3+75F (Figure 135) to minimize encroachment into the approach channel. Transition to a flatter and more economical homogeneous section is completed at Station 5+50F. The land side transition from zoned to homogeneous section is completed at Station 2+00F. Remaining dam sections for the Forebay and all sections for the Afterbay are essentially homogeneous with protective facing. Downstream blanket and toe drains, including perforated pipe in a trench filled with pervious material, were added to the homogeneous sections.

Stability Analysis. Stability was determined mainly by the Swedish Slip Circle method of analysis. In zoned sections, the sliding wedge method of analysis also was used. The infinite slope method was used to check the stability of all outer shell slopes. In addition to the reservoir loading at the full level and at other critical levels, a seismic acceleration of 0.lg was applied horizontally in the most unfavorable direction to each section analyzed. Material strengths were based on soils testing.

Settlement. Foundation settlements were predicted to be negligible in all but the 91-foot-high main Forebay Dam. There, an embankment camber of 6 inches was provided.

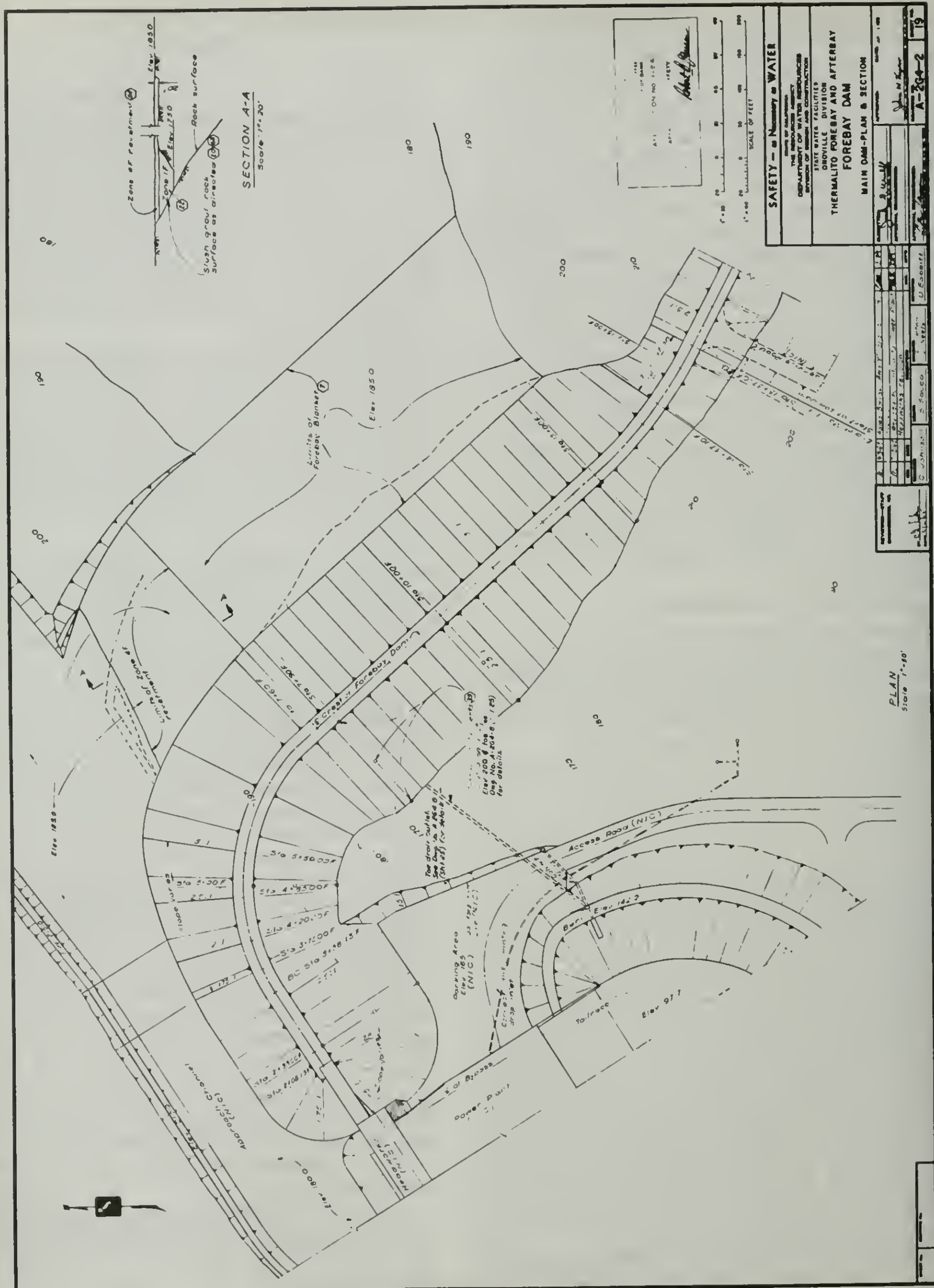

Figure 135. General Plan of Forebay Main Dam

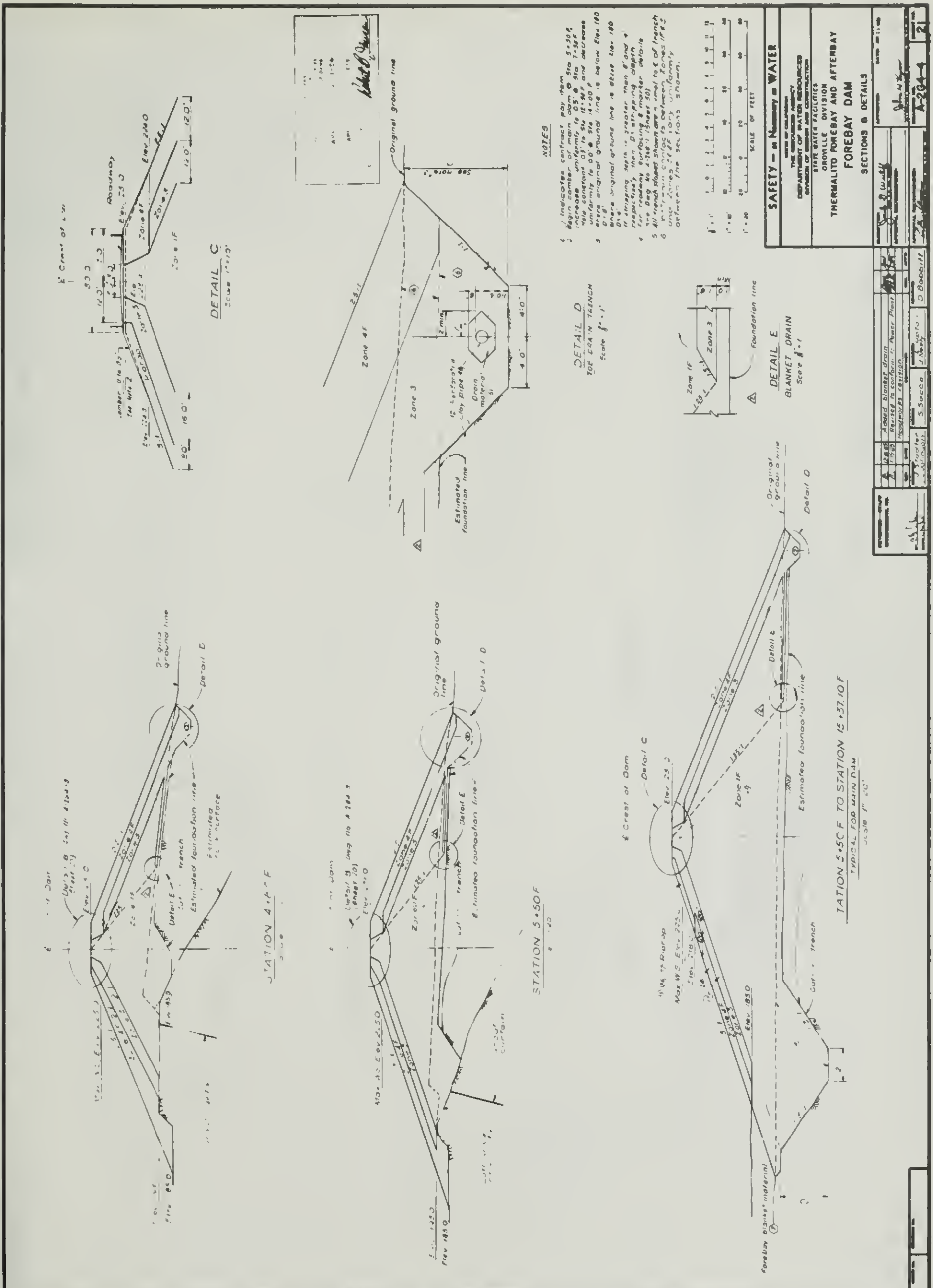

Figure 136. Forebay Dam—Sections and Details

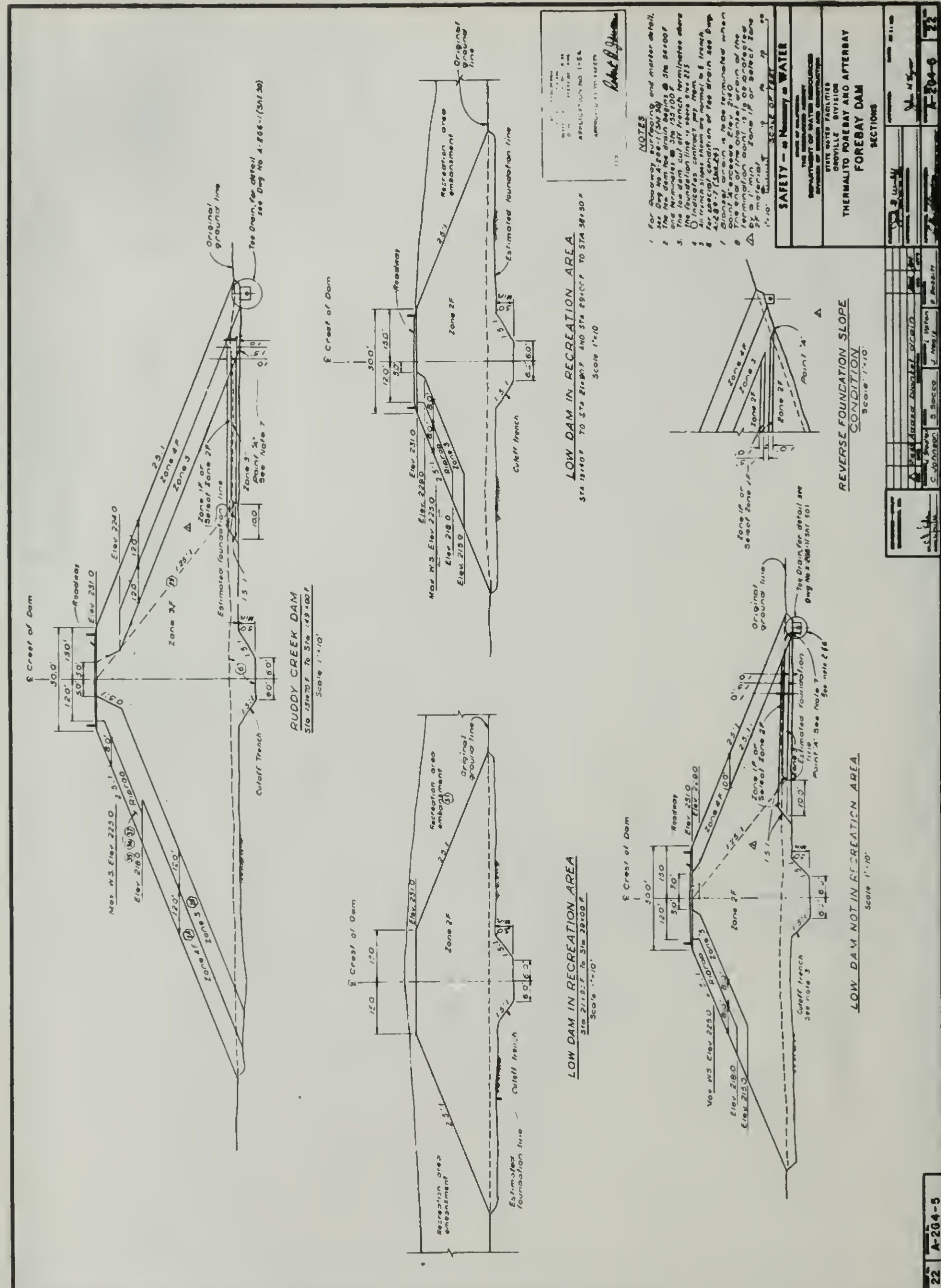

Figure 137. Forebay—Ruddy Creek and Low Dams Sections

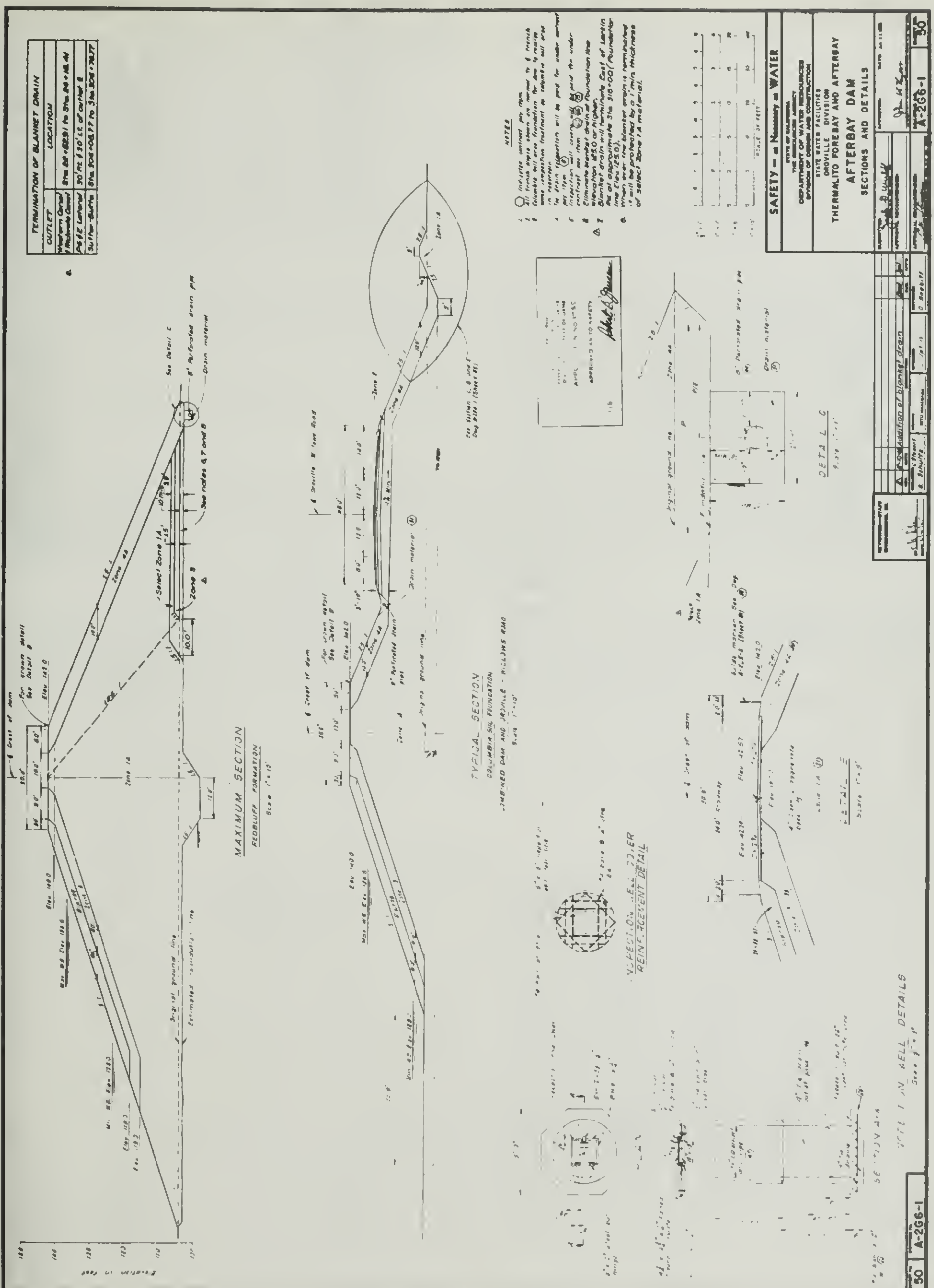

Figure 138. Afterbay Dam—Sections and Details

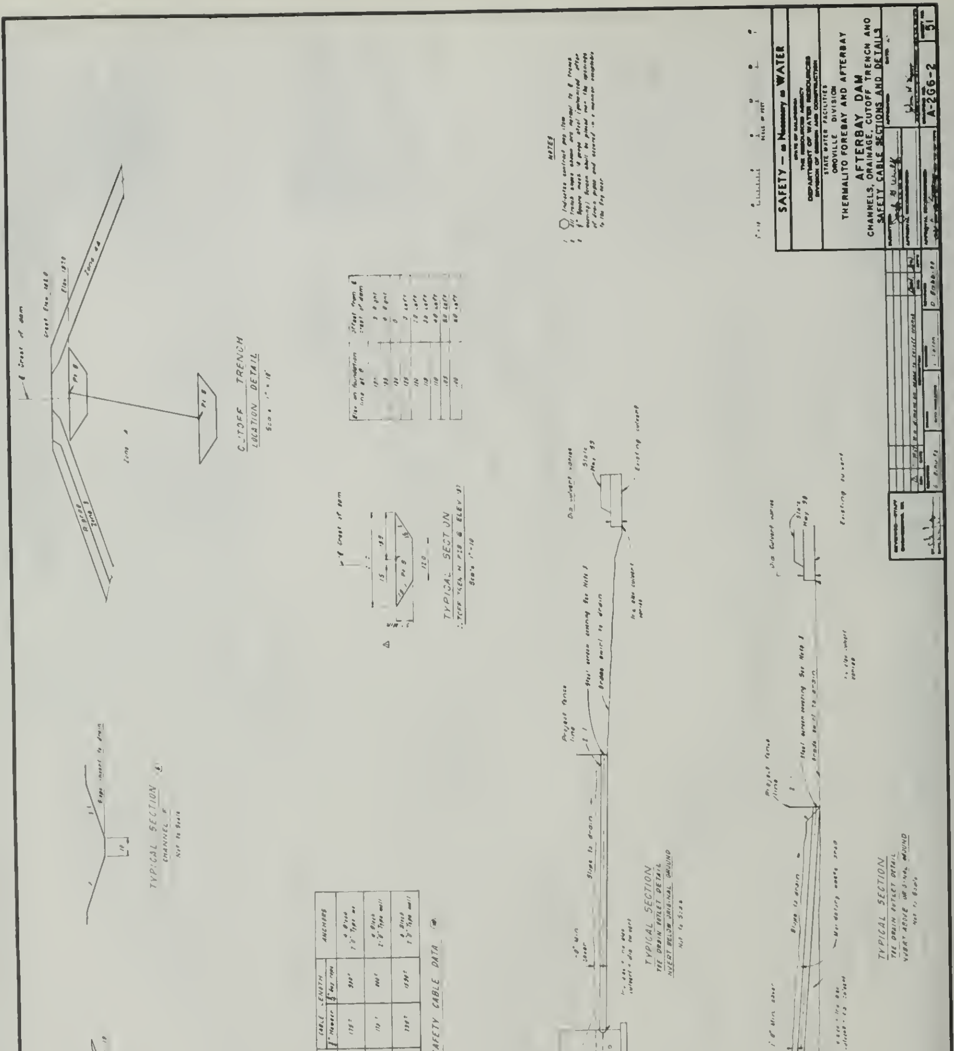

Figure 139. Afterbay Dam Details

Construction Materials. Materials used in embankments were obtained from the following sources (the letter F after the embankment zone number signifies Forebay Dam; the letter A, Afterbay Dam):

1. Impervious

Zone 1F—Designated stockpiled material from Thermalito Powerplant excavation.

Zone 1A—Mandatory afterbay excavation, optional channel excavation, dam foundation excavation, structure excavation, tail channel excavation, outlet channel and connecting channel excavation, and roadway excavation.

Zone 2F—Mandatory forebay excavation, optional channel excavation, dam foundation excavation, structure excavation, tail channel excavation, roadway excavation, and excess material from stockpile for Zone 1F.

Select Zones 1F and 1A are coarser materials selected from these sources to filter the blanket drains composed of Zone 3 material.

2. Pervious

Zone 2A—Transition material from Borrow Area Z (dredger tailings west of Feather River, south of Afterbay) or furnished by the contractor from outside sources.

Zone 3—Transition material from Borrow Area Z.

Zone 4F—Designated stockpiled rock material from Thermalito Powerplant contract, rock from mandatory forebay excavation, and Borrow Area Y (base of Campbell Hills within Forebay, near Thermalito Powerplant).

Zone 4A—Borrow Area Z and other designated areas of coarse dredger tailings.

3. Riprap

Designated stockpiled Thermalito Power Canal excavation (Rock Stockpile No. 1), designated stockpile Oroville Dam spillway excavation (Rock Stockpile No. 2), designated state-owned quarry 2 miles upstream of Thermalito Diversion Dam and approved contractor's sources.

Material design parameters for the construction materials are shown in Table 14.

Foundation. The forebay low dam is founded entirely on Red Bluff formation; the main dam, partially on basalt and partially on Red Bluff formation; and the Ruddy Creek Dam, on Red Bluff formation.

The Afterbay Dam is founded on Red Bluff formation except at the southeast corner of the reservoir where the foundation is Columbia loam.

The dam on the compact Red Bluff material was designed with a 5-foot-deep cutoff trench. On the Columbia loam, the trench was omitted, but the seepage path was lengthened by including relocated Oroville-Willows Road on a 50-foot-wide berm on the downstream side and a 100-foot-wide blanket on the upstream side. In addition, the entire reservoir floor was compacted.

Instrumentation. Instrumentation at Thermalito Forebay Dam consists of embankment settlement monuments and downstream open-tube piezometers and measuring wells.

Instrumentation at Thermalito Afterbay consists of

TABLE 14. Material Design Parameters—Thermalito Forebay and Afterbay Dams

Material	Specific Gravity	Unit Weight in Pounds Per Cubic Foot			Static Shear Strengths θ Angles in Degrees Cohesion in Tons Per Square Foot			
					Effective		Total	
		Dry	Moist	Saturated	θ	C	θ	C
Forebay								
Zone 1F	2.75	114	130	135	33	0	16	0.4
Zone 2F	2.75	109	126	134	30	0.15	--	--
Zone 3	2.91	155	161	164	45	0	--	--
Zone 4F	2.85	135	138	150	45	0	--	--
Riprap	2.85	125	127	144	45	0	--	--
Foundation	2.75	93	118	121	32	0.2	15	1.0
Afterbay								
Zone 1A	2.78	117	134	137	30	0.15	--	--
Zone 2A	2.91	155	161	164	45	0	--	--
Zone 4A	2.91	150	155	161	40	0	--	--
Riprap	2.85	120	122	140	45	0	--	--
Foundation in Columbia Soils Area:								
Southwest of River Outlet	2.80	96	122	124	30	0.10	--	--
Northeast of River Outlet	2.80	96	122	124	26	0.05	--	--
Foundation in Red Bluff formation was treated as basement rock								

109 open-tube piezometers (95 functional) located downstream of the dam and alongside the tail channel. In addition, piezometric level is monitored in 171 privately owned farm wells located in adjacent areas.

Forebay Inlet-Outlet

Flow into and out of the Forebay occurs in the normal course of power production. Thermalito Powerplant on the west and the Power Canal on the east each provide for these flows depending on whether generation or pumpback is taking place. A bypass gate is provided at the plant for release of water from the Forebay to the Afterbay in the event of powerplant shutdown. Flows up to 10,000 cubic feet per second (cfs) can be discharged.

Thermalito Power Canal

Thermalito Power Canal extends from a headworks structure, which is a portion of Thermalito Diversion Dam, to Thermalito Forebay (Figure 140). The canal invert is level to facilitate conveyance in both directions. It was designed to convey a maximum flow of 17,000 cfs for the generating cycle of pumped-storage operation. Maximum flow in the pumping cycle is 9,000 cfs. Canal cross-section dimensions were set for convenience of doubling the canal capacity at a later date in connection with a possible second Oroville Powerplant. Bridge pier foundations were deepened to allow for this expansion.

The area traversed by the Canal contained ground water which stood at a considerable height above invert for most of the length. Ground water levels were higher to the north of the canal alignment and generally followed the natural topography.

The Canal traverses an area with a wide variety of soil and rock material which vary considerably in strength from sound rock to weak clay.

Figure 140. Thermalito Power Canal

Canal Section. Preliminary design analysis indicated that for a canal with a flat invert at elevation 196.7 feet and 1½:1 side slopes, a 60-foot bottom width would yield the most hydraulically efficient section. The significant factor in selecting the bottom width was that it must be reasonably matched with possible second-stage construction. A 48-foot bottom width was chosen for initial development because it produced a favorable economic balance between the cost of additional head loss and savings in excavation and lining costs when considering future expansion. If the 60-foot section had been selected, widening problems would have been compounded for the second-stage canal to permit it to match the same water surface elevation. Typical sections are shown on Figure 141.

A transition section was provided between the Canal and the Forebay to minimize head losses. This section is 2,830 feet long.

Filter Subliner. Two types of material were used for the 9-inch filter blanket placed under the concrete lining. Type A (placed on soil excavation) was 65 to 90% sand with ¾-inch maximum size, 0 to 8% passing No. 100 sieve and nothing passing No. 200 sieve. Type B (for rock excavation) was 35 to 60% sand, with a maximum size of 1½ inches, and nothing passing No. 30 sieve. The filter was necessary to transport ground water into the canal lining weep holes and prevent piping of foundation materials.

Lining. Changes in pressures on the lining can result from fluctuations in ground water levels and from fluctuations in canal water surface elevation caused by changes in power operations. The lining, weep holes, and filters were designed to satisfy rapid fluctuations, even to the point of negative waves formed during emergency shutdowns.

Design of the canal lining specified 6-inch concrete slabs reinforced with No. 4, steel, reinforcement bars without expansion joints but with grooved joints placed at 15-foot centers to relieve expansion stresses. The lining is anchored at each end of the Canal to prevent movement at the ends. The top of the lining is at elevation 228 feet.

Weep holes were placed in the lining to allow ground water and water trapped behind the lining to flow freely into the Canal from the filter subliner. Holes were constructed from 1½-inch-diameter, molded, plastic pipe with a perforated conical tip embedded in the filter material. There are three weep holes for each top lining panel and one in each of the remaining 15- by 15-foot panels of canal lining.

Sides of the transition section into the Forebay were lined with 1 foot of stone protection over 6 inches of filter material. This stone material was specified to be 9 inches maximum and 1½ inches minimum size.

Drainage Structures. Interception of ground water flows and surface drainage on and above the cut slopes is critical to the stability of the canal side slopes. Surface drainage pipes receive intermittent use. Drop

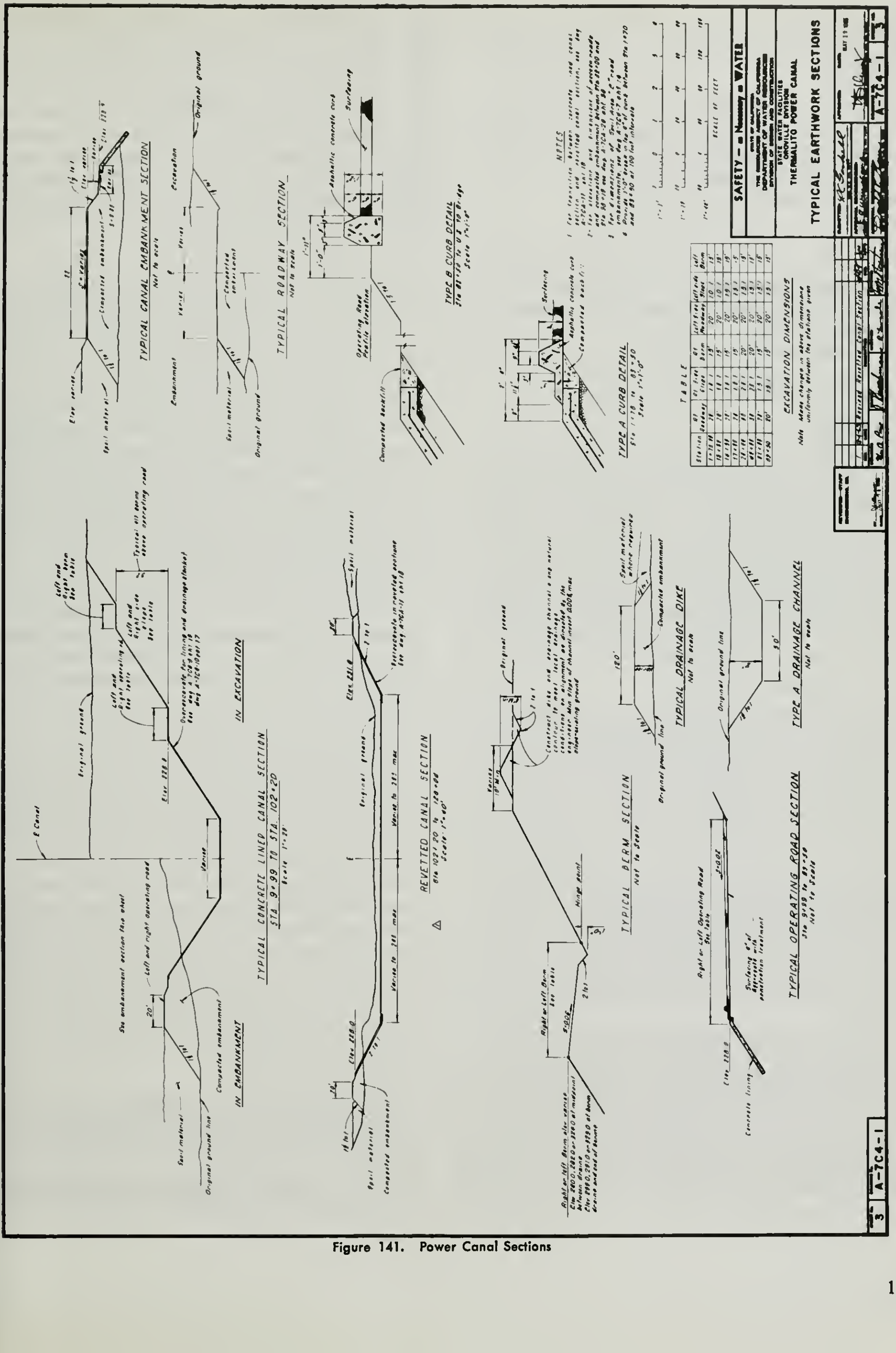

Figure 141. Power Canal Sections

inlets were installed with seepage rings to prevent seepage along the pipeline which would soon endanger the stability of the drainage structure and the canal section.

Horizontal drains were placed in localized areas to allow ground water in the cut banks above the canal lining to be collected and drained into the Canal (Figure 142). Each horizontal drain consists of a 2-inch-diameter, perforated, steel pipe connected to a 6-inch-diameter, asbestos-cement, collector pipe.

Turnouts. Two canal turnouts were provided (Figure 143). One is for California Water Service Company, and the other is for Thermalito Irrigation District. California Water Service Company conveyance and treatment facilities had to be relocated due to severance caused by the Canal.

The Thermalito Irrigation District outlet provides releases to replace water from its facility which was inundated by Lake Oroville.

Tail Channel

The tail channel hydraulically connects Thermalito Powerplant with the Afterbay. This channel is approximately 1½ miles long and was constructed with a 70-foot bottom width and 2:1 side slopes. It was sized to ensure that when Thermalito Powerplant is pumping and the Afterbay is at minimum water surface elevation, the water surface profile through the channel will be high enough to maintain adequate submergence on the pumping units. The channel section is lined with a 1-foot-thick layer of stone slope protection on a 6-inch layer of bedding.

Afterbay Irrigation Outlets

Irrigation outlet structures are incorporated into Thermalito Afterbay Dam to supply irrigation water to various water districts whose diversion facilities were taken out of service as a result of construction of the Afterbay. The irrigation outlet facilities (Figure 130) regulate, measure, and record deliveries. Sizes of the outlets were determined by the necessity to meet discharge requirements at minimum reservoir level and match existing water levels in the canal systems. These structures include Western Canal and Richvale Canal outlets combined in one structure, located in the northwest corner of the Afterbay; Pacific Gas and Electric Company (PG&E) lateral outlet, on the west side of the Afterbay south of State Highway 162; and Sutter-Butte Canal outlet, on the south side of the Afterbay.

Western Canal outlet consists of five 96-inch-diameter conduits through the dam. The Richvale Canal outlet parallels these with three 72-inch-diameter conduits, all resting on a concrete slab base (Figures 144 and 145). Each conduit is equipped with a slide gate at the upstream end to control flow and a Dall flow tube for measurement purposes. It was decided to use Dall flow tubes for measurement in this structure instead of a less expensive weir, as in the Sutter-Butte Canal outlet, because of the small head available when the reservoir is at minimum level. Conduits discharge into concrete-lined transition sections providing smooth flow conditions into the canals. Bulkheads are provided at the upstream and downstream end of the structure which enable the conduits and gated intakes to be dewatered. Stability analyses of the structure were made for several cases of loading, the more severe of which was for drawdown and for seismic loading. The minimum safety factor against sliding is 2.00 and against overturning is 1.47. Other stability results are considered to be adequate.

The PG&E lateral outlet (Figures 146 and 147) consists of a 30-inch-diameter conduit, a small intake training structure with provisions for bulkheading, a wet well upstream of the dam centerline containing a 30-inch-square slide gate, and an outlet stilling box with a weir for measuring flow.

The Sutter-Butte Canal outlet (Figures 148 and 149) consists of four 7-foot-wide by 6-foot-high rectangular conduits founded on a concrete base slab, slide gates, a headwall with provisions for bulkheading, training walls, and an outlet channel approximately 1,200 feet long connecting to the existing Sutter-Butte Canal. The concrete base slab overlies a 2-foot-thick drain blanket. Flow is measured by a control weir in the outlet channel approximately 400 feet downstream of the conduit outlet.

Stability analyses were made for several different cases of loading, with the two most severe being for drawdown and seismic loading. The minimum safety factor against sliding is 1.35 and for overturning is 1.33. All other stability results were considered to be adequate.

The controls for all irrigation outlets were designed to automatically adjust the slide gates to accommodate constantly changing head caused by afterbay fluctuations. Each gate may be operated from any of three control stations: at the gate hoist operator, in the separate outlet control houses, or in the remote control station located in the Oroville Area Control Center at the foot of Oroville Dam.

Each local control house is supplied with power from PG&E. The houses contain motor control centers, supervisory control cabinets, and standby engine-generator sets with appurtenances. The PG&E lateral outlet has a battery for emergency power rather than an engine-generator set.

Table 15 lists the pertinent data for the slide gates and electric motor-operated hoists installed on the various outlet structures.

Afterbay River Outlet

The river outlet (Figures 150 and 151) is situated in the southeast corner of the reservoir (Figure 130), a location most convenient for discharge to the Feather River.

Water is released from this structure for downstream project use, streamflow maintenance, and wa-

TABLE 15. Data for Gates and Hoists—Thermalito Afterbay

Outlet	Number of Gates	Size of Gates	Conduit Size	Gate Travel Speed Inches per Minute	Invert of Gate Opening Elevation	Flow Sensing Method	Gate and Operator Assembly Weight Each (pounds)
Western	5	96" sq.	96" dia.	6	105.0	Flow Tubes	13,000
Richvale	3	72" sq.	72" dia.	6	105.0	Flow Tubes	8,000
PG&E	1	30" sq.	30" dia.	3	109.2	Weir	3,000
Sutter-Butte	4	60" by 72"	72" by 84"	6	102.75	Weir	8,000

ter-right commitments. Streamflow maintenance requires a minimum release of 600 cfs at this point. The outlet discharges the net water used in power generation for each day uniformly over a 24-hour period. In winter months, high riverflows to Lake Oroville result in full generation, and afterbay releases can be as high as 17,000 cfs. During other months, only peaking generation occurs and releases are about 6,000 to 8,000 cfs. Releases to 8,000 cfs can be accomplished at any afterbay stage above minimum operating level, elevation 123 feet. Full discharge of 17,000 cfs is possible only with reservoir stage at or above elevation 127 feet. High flows in the Feather River restrict discharge through the outlet. Nearly full pool in the Afterbay is required for 17,000 cfs through the outlet with 150,000 cfs in the River (the standard project flood release at Oroville Dam).

Releases are controlled by five 14-foot by 14-foot radial gates with rubber "J" seals at the top and sides. The bottom is sealed by a rubber bar embedded in the invert. The gate position is automatically controlled and set remotely from the Oroville Area Control Center.

Outlet gates extend from invert elevation 105 feet to top elevation 119 feet. A concrete breastwall contains the reservoir above elevation 119 feet. Slots are provided both upstream and downstream of the radial gates to accommodate bulkhead gates to dewater each bay. Concrete counterfort walls form the sides of the outlet channel and retain the embankment at the headworks and channel. A service bridge crosses over the gates at elevation 142 feet, and a bridge for Oroville-Willows County Road spans the channel immediately downstream at elevation 135 feet.

Five electric motor-operated hoists are located on the machine deck of the headworks structure at elevation 142 feet to operate the radial gates. Each hoist consists of an operator with accessories mounted on a support base.

An unlined trapezoidal channel, with a bottom width of 160 feet and length of approximately 1,000 feet, carries the discharge from the headworks to the River. Concrete piers, in a pattern determined by hydraulic model study, were constructed in the concrete-lined headworks outlet channel to spread the flow and dissipate energy before the discharge enters the unlined trapezoidal channel. Approximately 800 feet downstream of the headworks, where the channel narrows, a weir was constructed to prevent anadromous fish from moving up the channel into the Afterbay and also for measuring discharge. The weir has an ogee-shaped crest at elevation 133 feet, with a length of approximately 168 feet. Metal racks were installed on the crest downstream of the weir crest to prevent migration of fish but were removed when subsequent operation indicated they were not required. A bridge and walkway provide access for service of the racks. Downstream of the weir is a paved channel about 117 feet in length which extends to the river channel.

The headworks structure and fish barrier weir were analyzed for overturning and sliding for both normal and seismic conditions. The minimum safety factors for the headworks are 1.92 against overturning and 1.32 against sliding. The minimum safety factors for the barrier weir are 3.4 against overturning and 1.66 against sliding.

A control house located near the outlet contains the motor control center, supervisory control cabinet, water-level and temperature recorders, and standby engine-generator set with its appurtenances. The control house is supplied with 3-phase, 480-volt, 60-cycle power.

Flood Routing

The volume of local inflow during the maximum probable storm was calculated to be 3,800 acre-feet for the Forebay and 5,200 acre-feet for the Afterbay. This would be produced by approximately 17 inches of rainfall in a 72-hour period. Ninety percent runoff would be expected due to the impervious surface of the drainage area.

Thermalito Powerplant bypass can discharge the forebay runoff into the Afterbay. The river outlet or one of the other afterbay outlets can discharge the combined local inflow, because the afterbay water surface would be at least 5 feet higher than the River or inundated lands in the area, even during the maximum probable storm over the entire Feather River drainage. Therefore, construction of spillways was unnecessary.

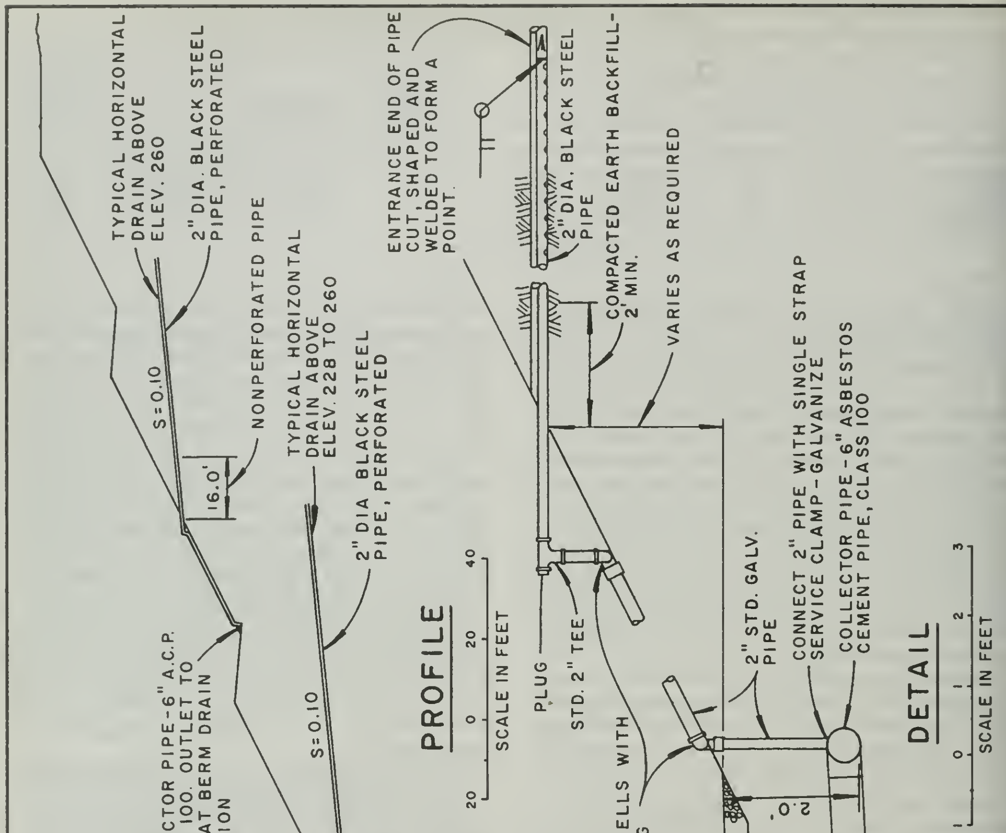

Figure 142. Typical Horizontal Drain—Thermalito Power Canal

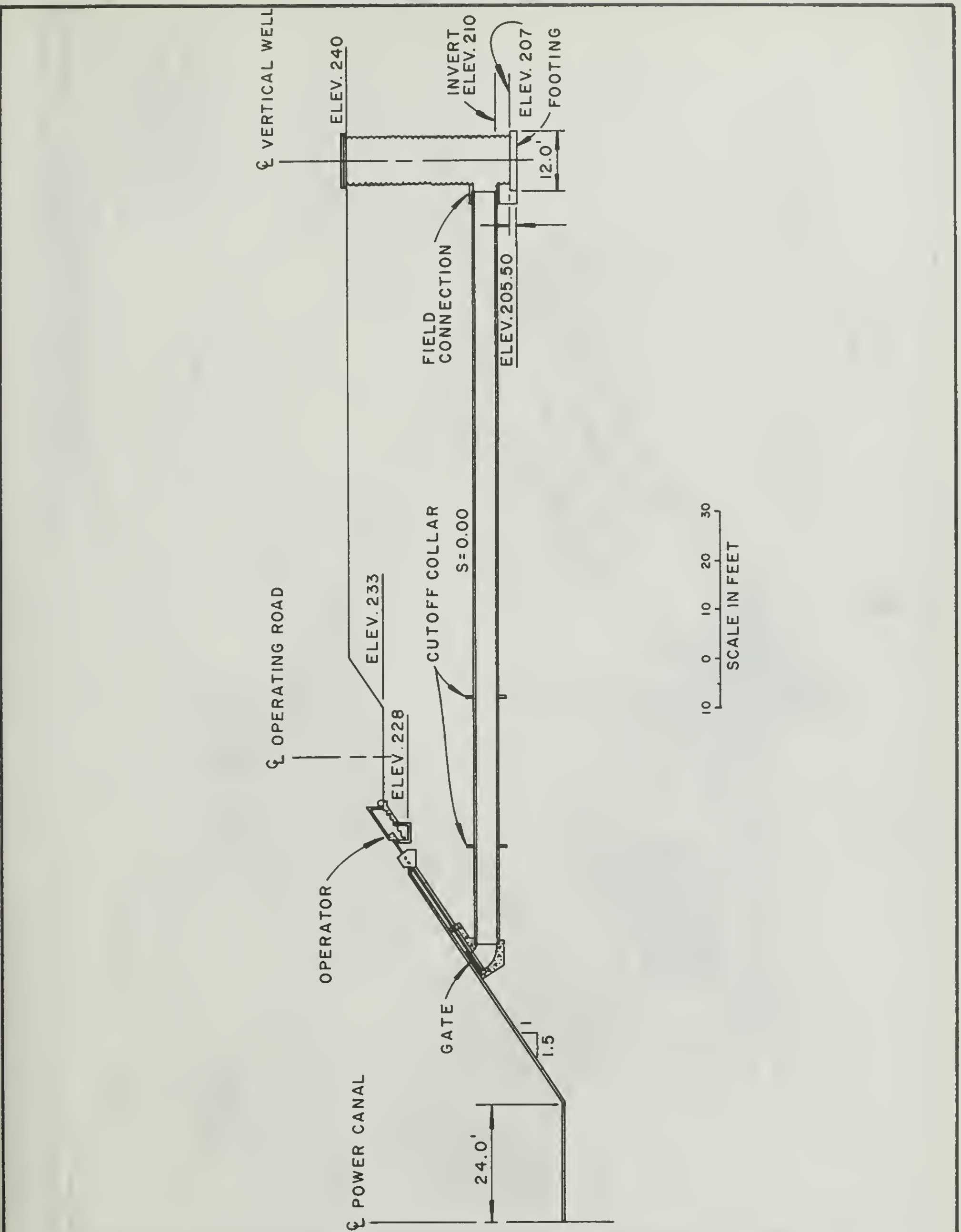

Figure 143. Typical Turnout—Thermalito Power Canal

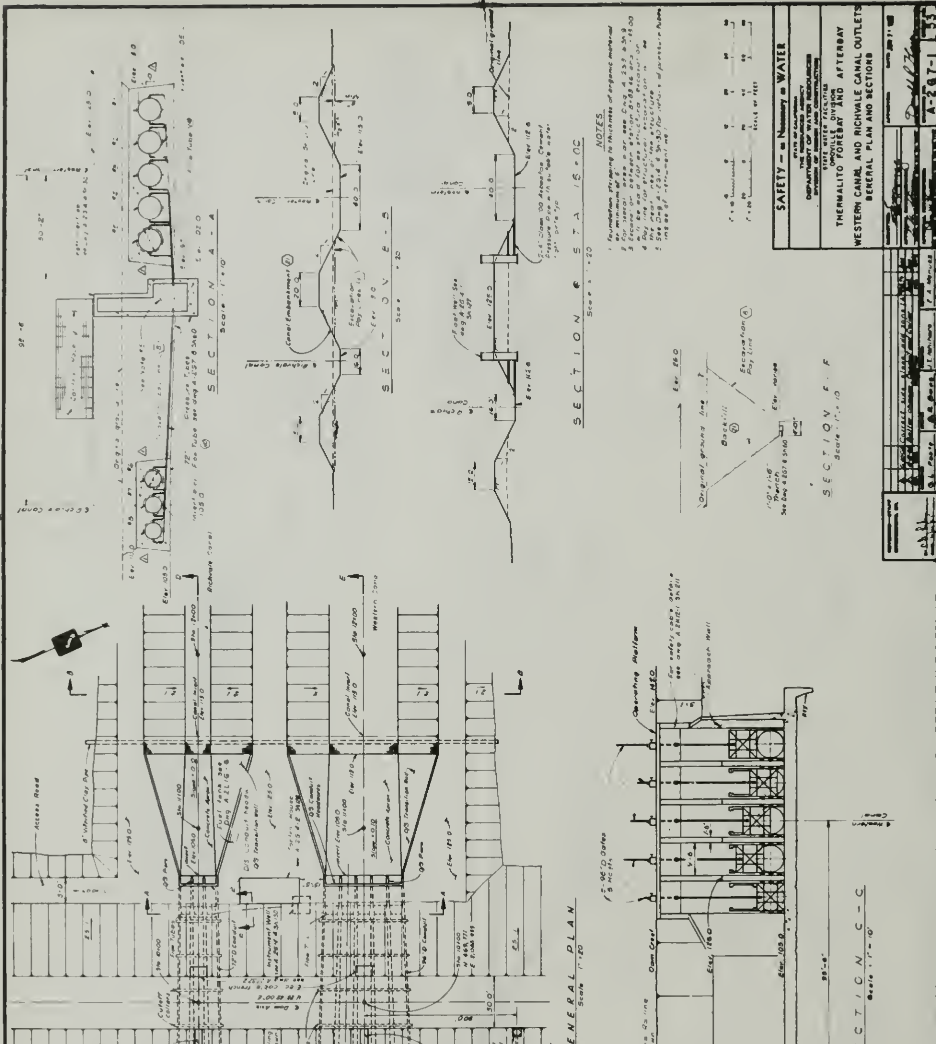

Figure 144. Western Canal and Richvale Canal Outlets

Figure 145. Western Canal and Richvale Canal Outlets—Isometric View

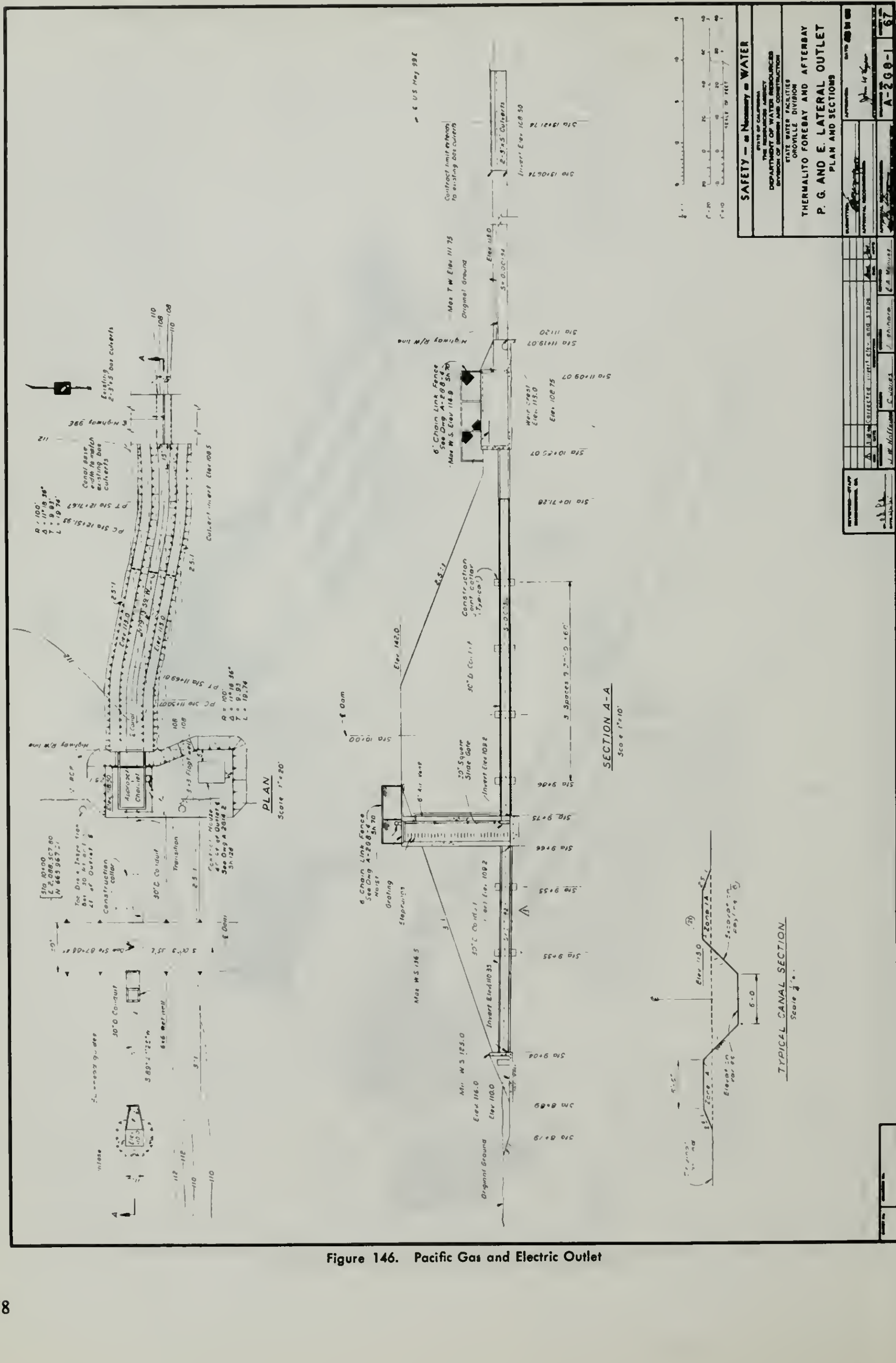

Figure 146. Pacific Gas and Electric Outlet

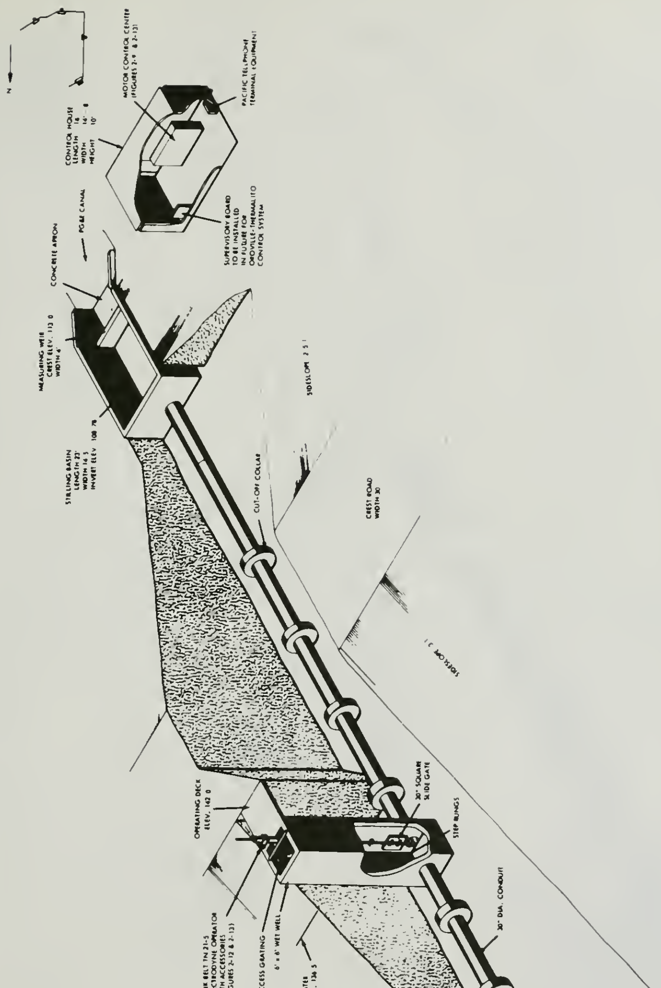

Figure 147. Pacific Gas and Electric Outlet—Isometric View

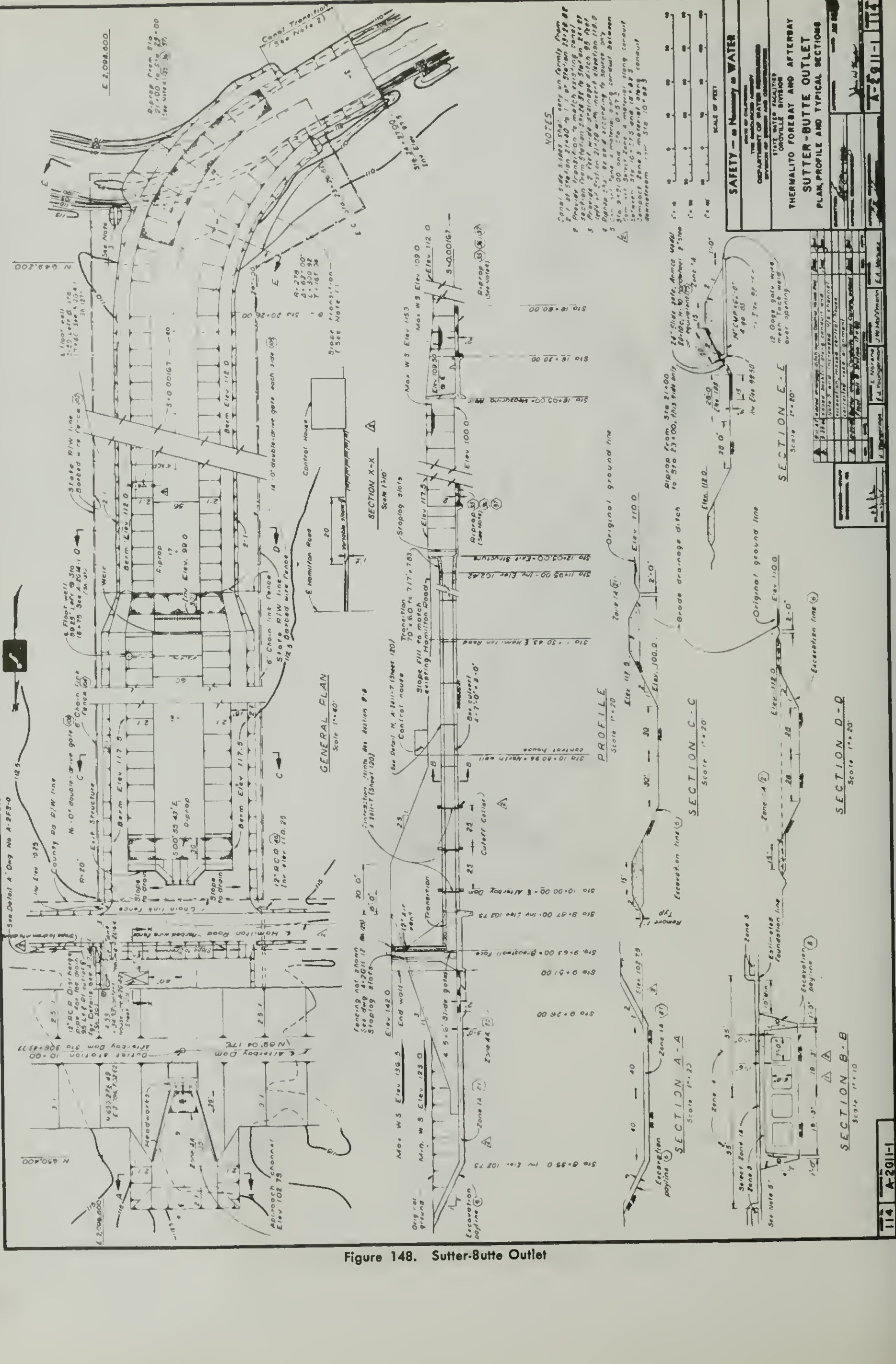

Figure 148. Sutter-Butte Outlet

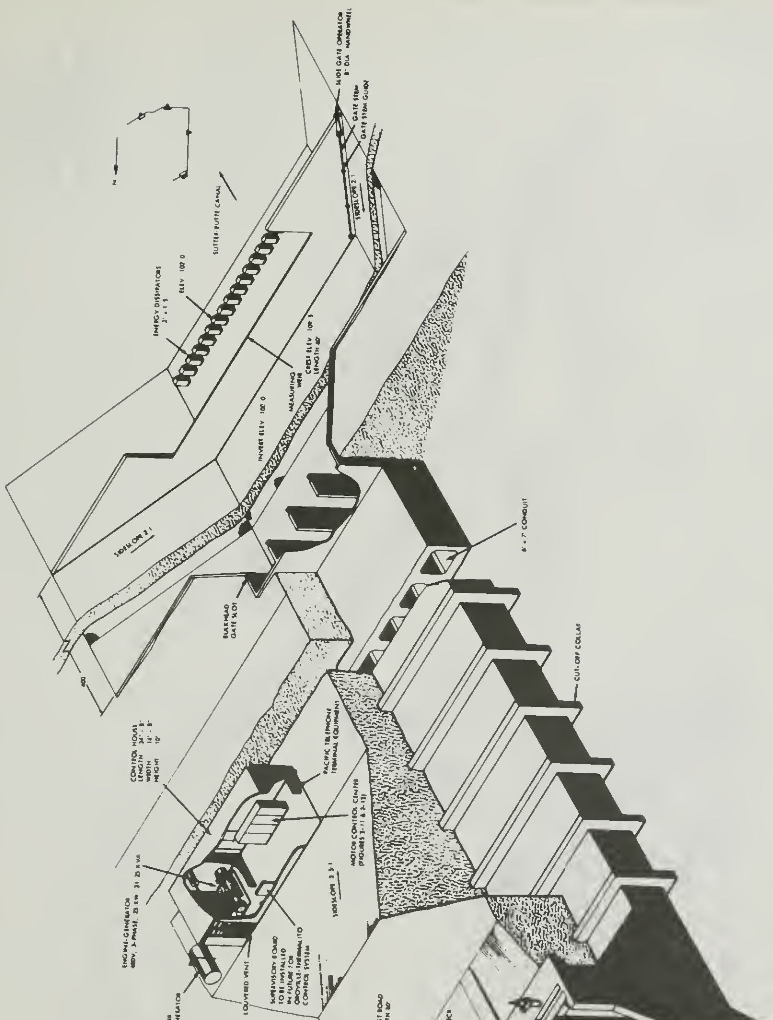

Figure 149. Sutter-Butte Outlet—Isometric View

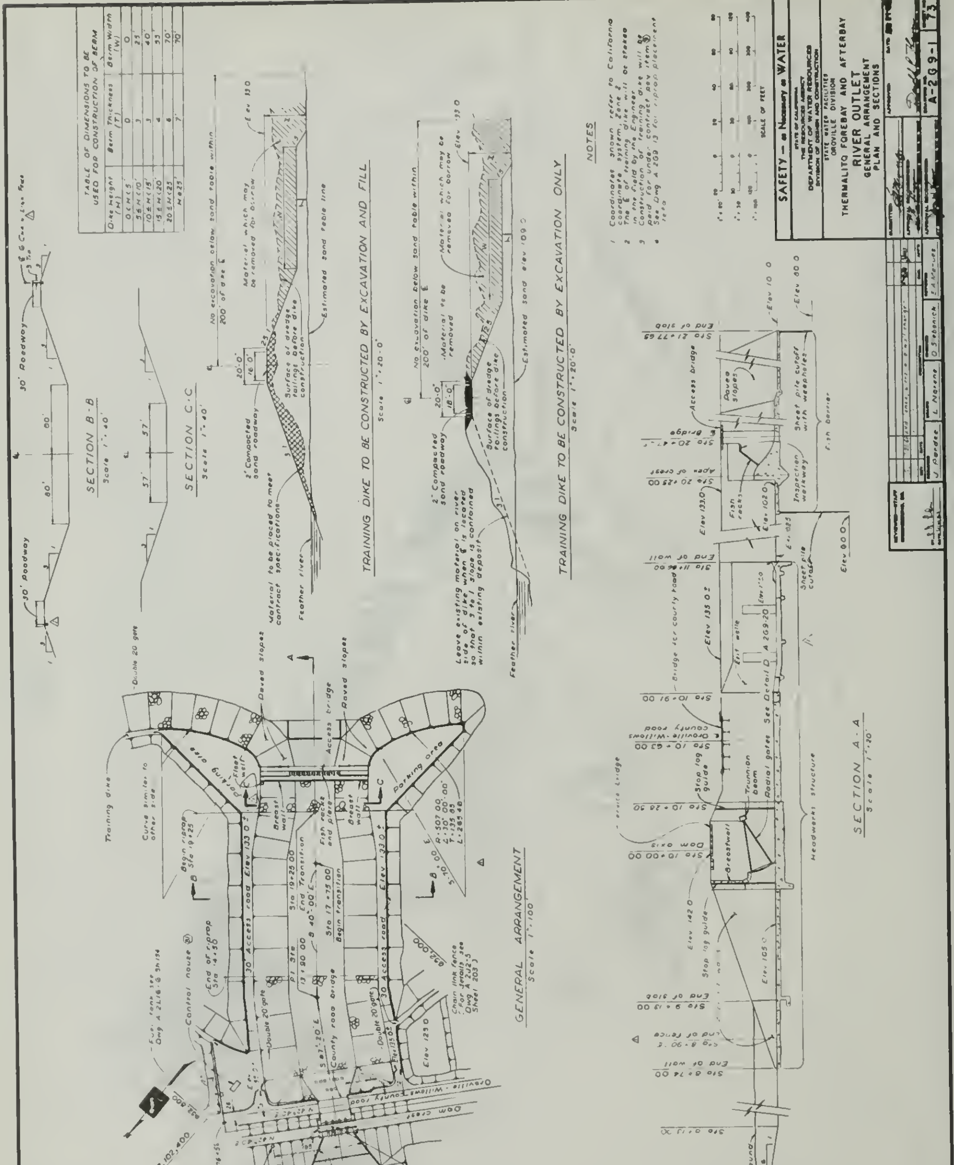

Figure 150. River Outlet

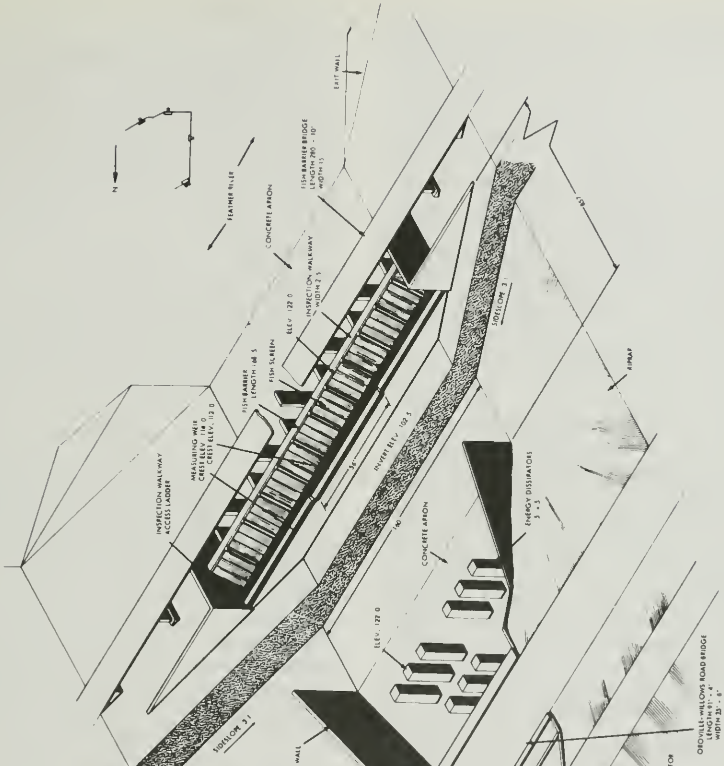

Figure 151. River Outlet Headworks and Fish Barrier Weir—Isometric View

Relocations—Thermalito Complex

Construction of the Thermalito complex required the relocation of many utilities and roads serving the area. It also required, where relocation was not made, structures to cross the complex or abandonment of little used roads. Where abandonment took place, other facilities were provided to handle the traffic (Figure 130).

Construction of the Power Canal required a bridge on the Oroville-Cherokee Road, the Oroville-Chico Road, and relocated Highway 40A (now State Highway 70). A bridge also was constructed across the Western Pacific Railroad relocation on the Oroville-Cherokee Road. Power Canal construction also required the relocation of the Thermalito water treatment facilities.

Construction of the Forebay required relocating about 4,600 feet of Nelson Avenue with a bridge across the Forebay.

Construction of the tail channel and Afterbay required relocation of Oroville-Willows Road, State Highway 162, and abandonment of sections of Tres-Vias and Larkin Roads.

All facilities were replaced in kind or updated to the then-current design standards. The bridges were designed and built to AASHO specifications and the California "Bridge Planning and Design Manual". They were designed for like loading of HS20-44 or H20-S16-44 with an alternate loading of two 24,000-pound axles 4 feet apart. The Department of Water Resources designed and constructed all the relocations except for State Highway 162, which was done by the State Division of Highways (now the Department of Transportation).

Following is a brief synopsis of the work performed on each road.

Oroville-Chico Road Bridge. The Oroville-Chico Road Bridge is a continuous, three-span, reinforced-concrete, box-girder structure supported on reinforced-concrete abutments and two reinforced-concrete piers. The two end spans are 60 feet long and the center span is 150 feet long, for a total length of 270 feet. Total deck width is 36 feet - 10 inches which includes two 14-foot - 0-inch vehicle lanes and a 5-foot -11-inch-wide curb on the east side. The boxgirder is a three-cell unit 28 feet - 5 inches wide from outside to outside and 8 feet - 0 inches deep from the roadway deck to outside bottom. The two abutments are founded on spread footings, and the two 8-foot - 0-inch-diameter columns are cast-in-place and extended to a maximum of 44 feet below canal invert with a minimum penetration of 8 feet into sound rock.

The entire structure was constructed prior to excavation of Thermalito Power Canal. This resulted in a shorter construction time with a minimum amount of false work required to support the cast-in-place superstructure.

The road alignment was shifted slightly to produce a more perpendicular and thus shorter bridge across the Canal.

Oroville-Cherokee Road Overhead Crossing. The Oroville-Cherokee Road overhead crossing of the Western Pacific Railroad relocation is a 140-foot-long bridge which has a three-span, continuous, reinforced-concrete, "T"-beam superstructure supported by reinforced-concrete piers and abutments. It provides a 32-foot roadway consisting of two 12-foot lanes and two 4-foot shoulders. A 3-foot - 2-inch safety curb with barrier railing is located along one edge of the bridge, and a 1-foot - 10-inch safety curb with barrier railing is located along the other edge of the bridge.

Oroville-Cherokee Road Bridge. The Oroville-Cherokee Road Bridge across the Canal is a continuous, five-span, reinforced-concrete, box-girder structure supported on reinforced-concrete abutments and four reinforced-concrete piers. The total length of the Bridge is 575 feet. One approach span is 65 feet long and the other is 110 feet. The lengths of the three interior spans are 110 feet, 144 feet, and 144 feet. Roadway width, sidewalks, curbs, and boxgirders are identical to the Oroville-Chico Road Bridge. The two abutments are founded on a cast-in-place concrete pile system. The four 8-foot-diameter reinforced-concrete piers are cast-in-place and extend 8 feet minimum into sound rock.

Nelson Avenue. Approach embankments and a bridge across the Forebay were constructed on an alignment north of the existing road to minimize the bridge length. The east approach embankment overlies the Forebay Dam.

The bridge has a six-span, reinforced-concrete, "T"-beam superstructure 430 feet long supported by reinforced-concrete piers and abutments. It provides a 28-foot clear roadway, a 5-foot sidewalk, a 2-foot safety curb, and barrier railings on each edge of the deck. Five piers have been provided for future construction of a parallel bridge.

Larkin Road. The Larkin Road Bridge was built to replace a county road which was severed by the tail channel.

The Bridge has a four-span, reinforced-concrete, "T"-beam superstructure 265 feet long supported by reinforced-concrete piers and abutments. It provides a 28-foot clear roadway, a 5-foot sidewalk, a 2-foot safety curb, and a barrier rail on each edge of the deck.

Except for a short section of this county road, Larkin Road was abandoned south of State Route 162. Traffic from this road was rerouted to the Oroville-Willows Road. (Recent maps indicate that Larkin Road has been renamed Wilbur Road and that old Oroville-Willows Road is now called Larkin Road.)

The county agreed that sections of Tres-Vias Road across the tail channel and the Afterbay could be abandoned if other access was provided. A single bridge crossing was used on Larkin Road to provide access to the Tres-Vias Road.

Oroville-Willows County Road. The Oroville-Willows County Road was relocated around the southern end of Thermalito Afterbay. The work consisted of slightly over 2 miles of new alignment with a bridge spanning the river outlet headworks.

The bridge is 96 feet long with a simple-span, composite, plate-girder superstructure supported at the ends by the walls of the river outlet headworks. It provides a 28-foot clear roadway, a 4-foot - 1-inch sidewalk, a 1-foot - 7-inch safety curb, and a barrier railing on each edge of the deck.

State Highway 162. State Highway 162 was placed on an embankment and a 670-foot-long bridge immediately south of its preproject alignment. The total length of the relocation was 4,300 feet. The embankment has adequate width for expansion to four lanes while the bridge will carry only the present two lanes of traffic. Embankment was included in the afterbay dam construction.

Construction

Contract Administration

General information about the major contracts for the construction of Thermalito Forebay, Afterbay, and Power Canal is shown in Table 16. There were two principal contracts. The first was for construction of Thermalito Forebay and Afterbay under the provisions of Specification No. 65-27. The most noteworthy features included forebay and afterbay embankments, tail channel, road relocations, outlet structures and gates, and installation of department-furnished equipment. The second was for the construction of the Thermalito Power Canal, Specification No. 65-37, which included earthwork, concrete lining, and turnouts for the Canal.

TABLE 16. Major Contracts—Thermalito Forebay, Afterbay, and Power Canal

	Thermalito Forebay and Afterbay	Thermalito Power Canal
Specification	65-27	65-37
Low bid amount	$14,452,680	$5,549,348
Final contract cost	$16,265,321	$7,061,410
Total cost-change orders	$1,387,113	$1,222,438
Starting date	10/25/65	10/7/65
Completion date	4/1/68	10/15/67
Prime contractor	Guy F. Atkinson Co.	Morrison-Knudsen Co.

Foundation

Stripping. Foundations of both the Forebay and Afterbay Dams were stripped of all organic material and Recent alluvium. Stripping was approximately 10 inches deep under the major portion of the dams and over 5 feet deep in Ruddy Creek, Grubb Creek, and other small drainage channels.

Excavation—Forebay. Foundation excavation in the Forebay was of two major types. One type was the Red Bluff formation and the other was basalt rock. Basalt rock extended from the Powerplant to Station 8+40F, while the remainder of the forebay dam foundation was Red Bluff formation. The foundation was excavated down to the basalt and curtain-grouted in this reach because it was economically feasible to do so. The grouting is discussed in a later section.

The main dam included a deep cutoff trench where it is founded on the Red Bluff formation to control seepage into the tail channel. A pervious water-bearing stratum was found near the bottom of design depth for the trench. The trench was deepened and extended to Station 15+37F, the southern limit of the Ruddy Creek channel. The pervious stratum still existed at that point, but any extension of the trench would have required considerable excavation. The deep trench was terminated since the tail channel slope was about 1,000 feet away. This treatment was effective and seepage into the tail channel has not caused any problems.

The method used to dewater the excavated trench was to place rock in drainage trenches on both sides of the cutoff trench invert and carry it above the limits of the wet zone. These trenches channeled water to 24- and 18-inch, perforated, riser pipes (sumps) from which water was pumped by submersible booster pumps until the fill reached within approximately 2 feet of the top of the pipe, at which point the water table was stabilized. The sumps were pumped dry, pumps removed, and immediately the riser pipes were backfilled with lean concrete. One and one-half-inch riser pipes, which were laid parallel to the slope and on 24-foot centers, penetrated the drain rock and extended to the surface. They were used to grout the drain rock when the fill reached the top of the cut.

Beyond Station 15+37F, the cutoff trench had a 12-foot bottom width with a depth of 5 feet except when pervious strata were encountered. Then auger holes were drilled to find the depth of the pervious strata, and deepening of the trench was accomplished if a satisfactory cutoff could be achieved within a maximum of 10 feet. If a satisfactory cutoff could not be achieved, a compacted impervious blanket 3 feet thick was added upstream, extending 100 feet from the toe of the dam.

Excavation—Afterbay. Foundation excavation in the Afterbay also comprised two major types. One type was the Red Bluff formation and the other Columbia loam. Red Bluff formation was treated in the same manner as in the Forebay beyond Station 15+37F.

The Columbia loam area in the southeast portion of the Afterbay and in the vicinity of the river outlet required close attention to prevent uncontrolled seepage into the River. Areas under the dam and the 100-foot blanket upstream of the dam were excavated to a depth of 3 feet to reduce the permeability, increase the strength, and assure removal of roots remaining from mature walnut trees that formerly grew in the area.

After excavating and grubbing operations were completed, the foundation was scarified, moisture-conditioned, and compacted with a 75-ton pneumatic roller prior to placement of Zone 1A compacted embankment.

Areas 400 by 2,000 feet, adjacent to Western Canal, were stripped to a depth of 2 feet and compacted. Once this area was compacted, the adjoining area received the same treatment, with the stripped material wasted on the previously treated area.

Grouting. The grout curtain under the main Forebay Dam was continuous with the powerplant grouting and included the length of the dam resting on basalt foundation as well as an extension 100 feet out onto the Red Bluff formation (Figure 152). In this last area, emphasis was placed on grouting the rubble lens at their contact. Eighty-five holes were drilled on the curtain line in three zones: (1) from the surface to 25 feet and pressure-tested at 25 pounds per square inch (psi); (2) from 25 feet to penetration of the interflow in the basalt and pressure-tested at 50 to 65 psi, depending on the depth of penetration; and (3) maximum depth of 100 feet and pressure-tested at 75 to 85 psi.

Grouting was conducted through a double-line (feed-return) system. Generally, the top 25 feet were tight; occasional holes took one or two sacks. Zone 3 holes also took only a few sacks of cement. Almost all of the grout was pumped into Zone 2 (interflow intercept) with one hole taking 614 sacks. Most mixes started at 7:1 and were reduced to 5:1 or 4:1.

In addition, 29 holes were drilled for blanket grouting fault areas and for contact grouting at the wingwall of the Powerplant. They were bottomed normally about 10 feet deep.

No grouting was performed in the Afterbay because only Red Bluff and Columbia soils were encountered.

Embankment Materials and Construction

Impervious—Forebay. Zone 1F and 2F material in the Forebay Dam was generally coarser and less uniform than Zone 1A material in the Afterbay Dam. Close attention was given to assure that the coarser portions of the Zone 2F materials were routed to the outer limits of the zones and Zone 1F materials were blended by controlled excavation to make them homogeneous.

Zone 2F material was compacted in all impervious portions of the embankment except for the designated Zone 1F. Zone 1F material was excavated from a stockpile located approximately 1,000 feet to the east of the main dam. Material was originally excavated from the Thermalito Powerplant area.

Zone 2F borrow areas contained many different strata of fine and coarse material. Excavation was performed normal to the strata with scrapers loaded downward on the slopes to assure blending of the strata. Sand lenses that were exposed in the Channel "H" excavation were blanketed.

Excavated material was transported to the embankment by scrapers and placed parallel to the dam axis in 8- to 10-inch loose lifts. It was then leveled and scarified with a bulldozer with a "Trinity Scratcher" or a motor patrol with a scarifier. The embankment lift was compacted with 12 passes of a sheepsfoot roller. All rolling was performed parallel to the dam axis, except where insufficient space prevented this procedure from being followed. Zone 1F material placed in contact with the powerplant wingwall was compacted with hand equipment (Figure 153). A relative compaction of 97% was required for Zone 1F and 95% for Zone 2F.

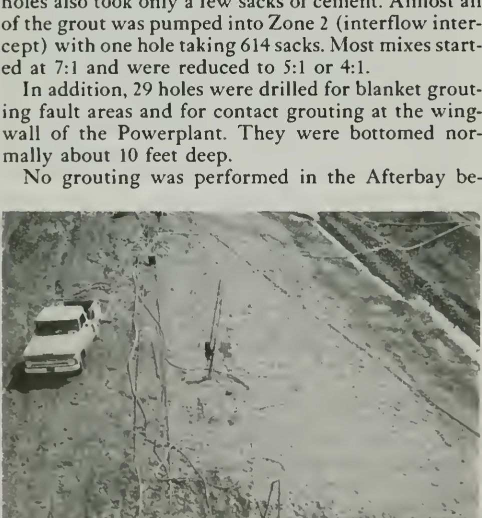

Figure 152. Grouting Foundation of Forebay Main Dam

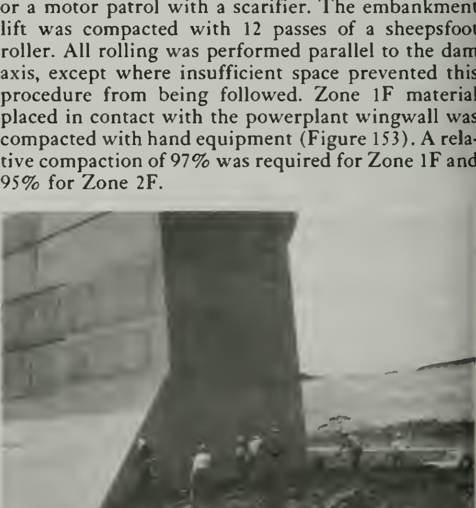

Figure 153. Hand Placement of Forebay Dam Adjacent to Powerplant Wingwall

Figure 154. Afterbay Dam Construction

Figure 155. Placement of Zone 4A—Afterbay Dam

When embankment showed an insufficient relative compaction, the areas were rerolled or the material removed and wasted.

Impervious—Afterbay. Suitable material excavated from the cutoff trench was utilized to backfill the already excavated cutoff trench or was compacted as Zone 1A embankment. The main source of material for Zone 1A embankment was required channel and structure excavation. Approved borrow areas selected by the contractor also were used. The specifications allowed the contractor to spoil required excavation and excavate for borrow at his own expense to shorten haul distances.

Transportation and placement of the Zone 1A material were performed in a manner similar to Zone 1F and 2F placement in the Forebay Dam (Figure 154).

Pervious—Forebay. Zone 3, a sandy gravel, was placed on the entire upstream face of the dam and at designated areas on the downstream side. Zone 4F material, basalt rockfill, was placed on the entire downstream face of the Forebay Dam and on the upstream face at designated areas. It was obtained from two places: Borrow Area Y, an extension of the inlet channel to Thermalito Powerplant, and stockpiled powerplant excavation.

Prior to placing Zone 3 and 4F material, the contractor elected to construct the Zone 1F or 2F embankment to about dam crest elevation. This resulted in a restricted working area which precluded compaction by the specified method of four passes of the treads of a crawler tractor. Tests of alternate compaction methods with vibratory rollers showed that one pass of a 72-inch vibratory roller on Zone 3 material and one pass of a 54-inch vibratory roller on Zone 4F material would result in compaction slightly higher than specified. Consequently, this method of compaction was adopted throughout the operation.

Both zones were loaded at the source with rubber-tired loaders and transported to the embankment in 16-cubic-yard, bottom-dump, highway trucks. A bulldozer was used to spread the material in 12-inch lifts.

Pervious—Afterbay. Placement of Zone 3 and 4A material consisted of a blanket of Zone 4A material on the entire downstream face of the dam (Figure 155) and Zone 3 material on the upstream face of the dam.

Zone 3 material was obtained from Borrow Area Z. Most of Zone 4A material was excavated from the river outlet structure area, with the remainder from optional borrow areas.

The contact point between Zone 1A embankment and Zone 3 and 4A material was watered and wheel-rolled with loaded trucks. Transportation and placement of material were accomplished in the same manner as for Zones 3 and 4F in the Forebay.

Riprap. Riprap was hauled to the dams utilizing end-dump highway trucks and trailer rigs and dumped in stockpiles along the upstream toe of the dams. Placement was performed with rubber-tired front-end loaders with chain wrapping on the front wheels for better traction (Figure 156). The loaders carried the rock from the stockpiles to the top of the slope and windrowed a lift for a distance of 50 to 100 feet longitudinally, then added another windrow below it. This procedure was continued until the bottom was reached.

"Top-Out" Operation. A small, self-loading, paddle-wheel scraper; a blade; and compaction equipment

Figure 156. Riprap Placement

Figure 157. Slide at Left Side of Power Canal

were employed to finish the top 2 feet of the impervious portion of the dams because they were too narrow for larger equipment to maneuver. The pervious portions of the dams were brought to grade at a later date. The top surface was fine-graded and a 4-inch layer of the aggregate base material was placed.

The aggregate base material was trucked to the embankment from a screening plant located at Borrow Area Z. Once at the embankment road, the material was dumped in two windrows, one on each half of the road; a spreader box was used to distribute the material to the required thickness and width of the road. After spreading was completed, the surface of the road was bladed and compacted with a three-legged roller. Final compaction was performed with a vibratory roller.

Thermalito Power Canal

Excavation. Canal excavation was started west of State Highway 70 using double-bowl diesel-electric scrapers. Excavation progressed eastward in a pioneering fashion, with the contractor constructing haul roads to and from various spoil areas prior to serious excavation in any particular area. Spreads of scrapers with bulldozers later were added to the excavation effort.

Nearly all of the excavated material required ripping prior to its removal with scrapers. Bulldozers operated singly or in tandem to push-load the nonelectric scrapers. Electric scrapers were self-loading with all eight wheels pulling but occasionally needed assistance. Softer earth material was excavated with the electric scrapers while the others were used mostly in weathered and decomposed rock, in wet areas, and in the small confined areas of the canal prism. The rate of production in the canal excavation averaged 25,000 to 28,000 cubic yards per eight-hour day.

Between the Oroville-Cherokee Road Bridge and the Oroville-Chico Road Bridge, areas of unsuitable material were encountered below the elevation of the canal invert and beyond the established slope lines above the operating roads. These areas were overexcavated and refilled. The invert was refilled with compacted embankment or with Type B filter material, depending on the subsurface water conditions encountered. Side slopes above the canal lining were filled with free-draining material from canal excavation to eliminate slide conditions.

Bedrock was drilled and shot. Typical drill holes were 20 feet deep on 8-foot centers. Shot rock was loaded into dump trucks with front-end loaders and, using the canal invert as a roadway, the material was hauled to the designated riprap stockpile area or wasted in spoil areas. The stockpiled rock was used as riprap on the forebay embankment under the Thermalito Forebay and Afterbay contract. More than half of the excavated rock broke into pieces too small to be used for riprap, even though normal rock excavation methods were used. The high loss probably resulted from weathered shear zones or closely spaced joints in the bedrock. A total of 207,000 cubic yards of excavated rock was stockpiled.

Beginning in early August 1966, the first indication of serious slide trouble became apparent when earth slides developed in both cut banks 1,000 to 2,000 feet west of Oroville-Cherokee Road Bridge. Fissures and cracks observed at various other locations along the canal gave evidence of possible future slides. These slides and indications of slides were not of any great magnitude at first; however, attempts to remove the slide material triggered further movement. The slides were attributed to failure, or "breaking down", of unsupported Ione formation clay, which was exposed during canal excavation (Figure 157).

After the first earth slides, seasonal rains commenced which further aggravated slide conditions. Existing slides became more extensive, and new slides developed in areas where the slopes had been standing satisfactorily. Additional movement of the earth slides west of the Oroville-Cherokee Road Bridge blocked the canal invert, causing surface drainage water to pond for a distance of one-half mile. Consequently, the contractor was directed to let the earth slides stand until studies were made. It was concluded that the slides should be removed and replacement of the slide material with compacted backfill began in early February 1967, on a two-shift-per-day schedule, five days a week. Later, the contractor went to a three-shift schedule, six days a week. Corrective work was completed in May 1967, and all canal excavation was completed in June 1967.

Horizontal Drains. In areas where ground water was found seeping through the cut slopes, horizontal drains were installed to intercept and remove this subsurface water. Average depth of the holes was 172 feet, with a depth range of 80 to 300 feet.

At several locations, horizontal drains were installed in the canal prism below the operating roads. These drains were not anticipated in the design, so a contract change order provided for a collector system and for disposing of the water. These drains extend through the canal lining.

Filter Subliner. Type A filter material was placed with a clamshell a few inches below final grade prior to the trimming operation, with the remainder being placed by the lining machine at trimming. Where Type A filter material was found to be above elevation 218 feet, it was removed and replaced with Type B filter by means of a cutoff plate attached to the front of the lining machine (Figure 158). Type B filter for rock sections also was placed on the slopes by a clamshell and trimmed with a lining machine. Filter material in the invert was spread with a motor grader and trimmed concurrent with trimming of the side slopes. Filter material was placed by clamshell and trimmed to line and grade by less automated methods in other areas inaccessible to the lining machine.

Concrete. The canal lining consists of 6-inch-thick reinforced concrete with transverse grooved joints on 15-foot centers. One longitudinal construction joint was placed at the canal centerline. Longitudinal grooved joints were placed on 15-foot centers on the side slopes and at distances of 7 feet and 20 feet on each side of the centerline in the invert.

Reinforcement steel for the canal lining consists entirely of No. 4 bars placed on 12-inch centers each way. Precast mortar spacers were attached to the steel, and the invert and side slope mats were placed on the filter blanket with a mobile crane (Figures 159 and 160).

Figure 158. Placing Type II Drain Pipe Along Toe of Slope and Type B Filter Material on Invert of Power Canal

Figure 159. Placement of a Slope Mat

Figure 160. Reinforcing Steel Being Placed in Canal Invert

The contractor's paving (lining) train was made up of a lining machine followed by a finishing jumbo and a curing jumbo. The jumbo spanned the full width of the paving operation, one side slope and half the invert.

All power to the lining machine was supplied by a diesel motor-generator mounted over the low driving track. Travel and support were provided by two independent tracks driven by an electric motor mounted on each unit. Line and grade were controlled by electric probes attached to the lining machine, activated by piano wire stretched along the invert and the operating road.

Concrete was delivered to the lining machine by 8-cubic-yard mixer trucks and deposited on a belt which conveyed the concrete to a 4-cubic-yard, traveling, dump car mounted on the front of the lining machine. Consolidation of the concrete was accomplished by horizontal vibrators running the full length of the lining machine and mounted in the bottom of the baffled hopper at the finish gradeline. Once the concrete was consolidated, it passed under a smooth pan located below the structural members of the machine. Beyond the smooth pan, there was a short gap; then the concrete passed under an 18-inch-wide floating pan.

The longitudinal grooves were cut first using cutting edges protruding 3 inches below the lining machine's leading pan. The transverse grooves were cut by transverse bars located at the rear of the lining machine which were forced into the concrete by hydraulic rams aided by vibration. The grooved joints obtained by this method were not satisfactory. They required more hand finishing than appeared practical, so other methods were explored. The contractor finally elected to place plastic strip joints both longitudinally and transversely. Observation of the breaks and appearance in the lining at the plastic strip joints indicated that good results were obtained.

Concrete placements, referred to as "hand lining" were made in areas inaccessible to the canal lining machine and in the transition area west of State Highway 70. A small paver (Figures 161 and 162) was designed by the contractor and used to place concrete in these areas.

Concrete placed was well consolidated and required only a small amount of hand finishing. The cove at the toe of the side slopes was placed first in alternating 15-foot sections. Side slopes then were placed in the remaining 15-foot-wide sections. Alternating the initial concrete placements provided a firm foundation and solid side forms for the paving screed during the final placements. The invert was placed last.

The operation that actually controlled construction progress on the canal lining was the concrete finishing. Concrete repair was required in a few areas that were placed in the early part of the lining operation.

Concrete for the canal lining was produced in an automatic batch plant located to the right of the canal,

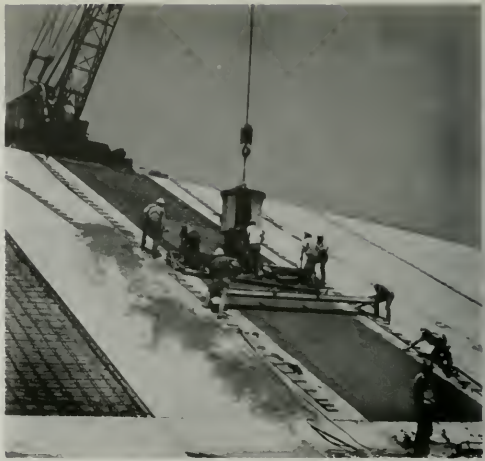

Figure 161. Slip Form Lining Operation on Thermalito Power Canal Transition Slopes

west of the Oroville-Cherokee Road Bridge, in Spoil Area B. Concrete temperature control was accomplished by the addition of ice to the batch water. The contractor was able to maintain the temperature at or below 80 degrees Fahrenheit.

Structural concrete was delivered to the site in transit mixers from a commercial plant.

Weeps. Due to anticipated problems and slow rate of progress in installing the canal weeps as designed, plastic tube-type weeps were substituted. The plastic weep consisted of a 2-inch-diameter by 12-inch-long tube, conical on the bottom and open at the top with an internally fitted plastic cap. Weeps were installed by placing each weep over a steel insert and driving it into the wet concrete.

Stone Protection. Where the canal section is 400 feet wide, the specifications gave the contractor the option of placing crushed stone or rock material obtained from the canal excavation or other sources. The contractor interpreted this as allowance for the use of uncrushed river gravel screened from local Feather River aggregate sources. However, it was the Department's intention to require the use of crushed or angular rock material—not round, uncrushed, river gravel. Consequently, a contract change order was issued clarifying the requirements.

Recreation Area. In cooperation with the California Department of Parks and Recreation, site preparation work for the north forebay recreation area adjacent to State Highway 70 was included in the power canal contract. This recreation area originally consisted only of a spoil area, but a contract change order provided for a swimming lagoon and beach adjacent to the spoil area and canal outlet.

Figure 162. Closeup of Slip Form Lining Operation on Thermalito Power Canal Transition Slopes

Figure 163. Bedding Placement on Tail Channel

Work in the area consisted of clearing and grubbing; stripping embankment foundations; placing compacted embankment for access roads, parking areas, and building pads; placing 24-inch-diameter, corrugated-metal, pipe culverts and culvert markers; final shaping and grading to the design contours; and the construction of a 24-foot-wide, reinforced-concrete, boat ramp. A separate contract change order provided for the application of a soil sterilizing agent to a beach area prior to placement of aggregate base material and processed sand.

Tail Channel

Excavation. Dewatering was performed with one 6-inch, one 8-inch, and one 12-inch, 50-horsepower, centrifugal pumps. Water was piped to the Van Gilder drain about 1½ miles west of Larkin Road.

In some areas, the bottom of the excavation became too saturated due to excessive ground water and would not support the weight of the scrapers and other equipment. In these areas, material was excavated with draglines. Where overexcavation occurred, the areas were brought back to grade with Zone 4F material. Usable material from the tail channel was placed in appropriate zones of the forebay and afterbay embankments and the remainder was wasted. Final trimming of the slopes was performed with a small bulldozer, and the invert was brought to grade with a blade.

Placement of Channel Protection. Upon completion of slope trimming, placement of the stone slope protection and bedding material commenced with a Gilli-K-Hike (Figure 163). Power to the belt was provided by a diesel engine-generator mounted adjacent to the hopper on a platform at the bottom of the machine, and the power to move the unit was provided by a bulldozer located on the operating road and a rubber-tired side-dump truck on the invert of the channel. Stone slope protection and bedding material were transported with side-dump trucks. The bedding material is dredge tailing sand and gravel which was processed at the Thermalito Powerplant batch plant. The slope protection is the same material as forebay Zone 4F. An average of approximately 1,000 feet of slope protection was completed per two-shift day.

Invert protection material, which is the same as slope protection material, was trucked with bottom-dump and end-dump, 18-wheel, highway trucks from Borrow Area Y and Zone 4F stockpile and spread with two large bulldozers. Final grading was performed with blades. Immediately after placement of the invert protection was completed, the sump pump located at Larkin Road was removed, and the ground water was allowed to fill the channel invert.

Miscellaneous Channel Excavation

Channels "A" through "G" are located within the limits of the Afterbay while Channel "H" is within the limits of the Forebay (Figure 164). They were excavated through high ground to provide hydraulic continuity in the reservoirs.

The contractor exercised his option of extending the minimum lines of Channels "A", "C", "D", and "H" to obtain suitable materials for embankment at reduced haul distances. Sand strata that were exposed during the excavation of several of the channels were blanketed with a layer of impervious material.

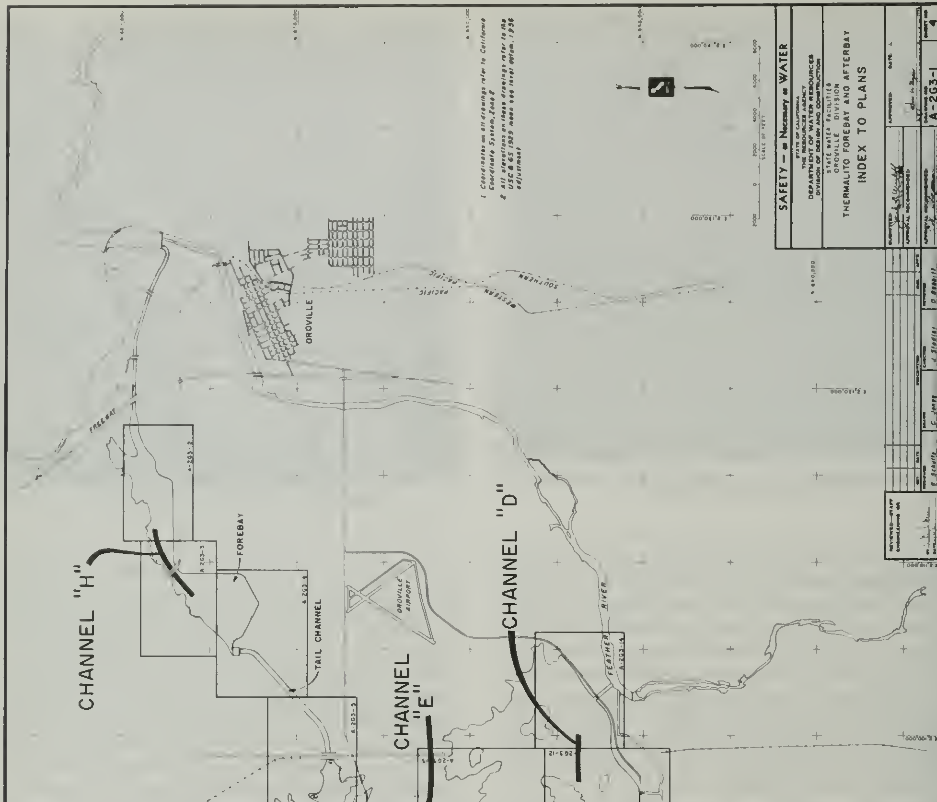

Figure 164. Location of Miscellaneous Channels

Irrigation Outlets

Western Canal and Richvale Canal. Western Canal and Richvale Canal outlet structures were built immediately north of the existing canal alignment so that the canal could be used during construction (Figure 165). At the upstream end, the structures are connected by a retaining wall, footing, and cutoff wall. The entire area between the two structures and the adjoining area for the structures themselves were excavated in one operation. This made dewatering easier since drainage channels could be cut across the entire area and fewer sumps were required.

Dewatering was accomplished by digging trenches around the structures. These trenches led to sumps where the water was pumped out and discharged into Western Canal. Two weeks after commencement of discharging water into Western Canal, it was discovered that a delta was developing in the Canal from sediments deposited from the pumped water. Subsequently, water from the excavation was discharged onto the ground where it either evaporated or seeped back into the ground. The foundation for the main cutoff wall was overexcavated and formed instead of placing concrete directly against the excavation.

The cylindrical forms for the five 92-inch-diameter (Western Canal) and three 72-inch-diameter (Richvale) Dall tubes had to be anchored securely because they were subject to extreme uplift pressure as the concrete placement was brought up.

It was difficult to place concrete under both the flow tubes and thimbles embedded in concrete at the barrel intakes without developing voids. These voids were filled by drilling a hole at the lowest and highest points of each void and pumping grout into the lower hole until it came out the upper hole. Holes in the metal were filled by threading the hole and screwing a plug into it. Plugs were ground until flush with the walls.

Each exit wall was placed in three sections. The first section was formed, the concrete placed and allowed to set, and the forms stripped, usually in about 6 days. Fill then was compacted behind the wall. In this case, however, the contractor was directed not to strip the forms until test cylinders showed a strength of 2,500 psi to assure that the stress at the base of this concrete section due to the backfill load would not be exceeded.

For the two sections most removed from the outlet structure, the slope was flatter and material was placed behind the walls to grade. Concrete then was placed from trucks and finished with a slip form that moved up and down the sloping surface.

PG&E Lateral. This structure is a single-barrel 30-inch-diameter conduit approximately 180 feet long, with a small intake and exit works.

Structural excavation was performed with a backhoe for the conduit section and with a bulldozer for the exit structure and weir. Minor adjustments were made by hand labor for the cutoff collars and the float well.

Concrete placement began with the bottom sections of the six cutoff collars. Subgrade slabs of the inlet, exit, and wet well were placed next.

Conduit concrete (Figure 166) was placed in a series of sections as determined by construction joints. Each section was placed in two lifts, the first of which was made with the inside conduit forms and all of the

Figure 165. Western Canal and Richvale Canal Outlets—Upstream View

Figure 166. Pacific Gas and Electric Lateral Outlet Conduit

reinforcing steel in place. The concrete was placed in the bottom section and brought up to a level an inch or two below the invert. This anchored the reinforcing and inside forms, thus preventing the forms from floating during the placement of the second lift. Outside forms then were installed and weighted in place and the second lift placed through the top.

Sutter-Butte Canal. Sutter-Butte outlet structure (Figure 167) consists of an approach and headworks section, four 6- by 7-foot rectangular conduits approximately 340 feet long, and an exit structure. A rectangular, modified V-notch, sharp-crested weir was constructed 370 feet downstream of the exit structure. Incorporated in the headworks section are 4- by 6-foot, steel, slide gates and hoists.

Canal excavation was performed in conjunction with the structural excavation. The small plug at the entrance to the old Sutter-Butte Canal was removed during the closure sequence.

Rough excavation was performed with scrapers, bulldozers, a motor grader, and a backhoe. Final trimming was performed with a small bulldozer, and cutoff collar excavation was performed with a small rubber-tired backhoe.

During structural excavation for the headworks structure cutoff collar, water-bearing pervious strata were exposed. As a result, the approach channel was blanketed with impervious material and the structure backfill modified to increase the length of the water percolation path. Dewatering was accomplished by excavating drainage ditches along the outside of the conduit slab and by utilizing the cutoff collar excavation as cross drainage.

Bottom slabs of the conduits were placed in alternate 25-foot-long sections for the full width of the structure. The walls and top slab of the conduit sections were placed next, in a like manner (Figure 168).

Placement of the downstream transition section included the first lift of the exit piers. Embedded in these piers were the downstream stoplog guides. During placing operations, movement of the forms pushed the right exit wall guide out of plumb by more than 1 inch parallel to the structure centerline. It was repositioned and epoxy-bonded concrete replacement was made in 2-foot lifts.

Construction of the slabs and walls of the upstream approach and headworks (Figure 169) was done similar to those of the exit works. Higher wall placements were made in two lifts, the maximum height being 14 feet. Headworks piers were made in three lifts with the breastwall concrete incorporated in the top two lifts. The counterforts were placed to full height in one lift.

River Outlet

The river outlet structure (Figure 170) is located in the southeast corner of the Afterbay (Figure 130).

Prior to commencement of excavation, sheet piles were driven and a cofferdam was constructed across the entire width of the canal just downstream of the fish barrier structure to keep the river water from entering the construction site.

Immediately upstream of the cofferdam, two electric centrifugal pumps, 8- and 12-inch, were installed to pump the ground water from between the fish barrier and the cofferdam into the River.

Another cofferdam was built just downstream of the headworks structure for flood protection purposes during the rainy season. During the winter months of 1966, the ground water between the two cofferdams was allowed to reach the water surface elevation of the River, thus equalizing the hydrostatic pressure about the cofferdam at the River. After the rainy season, the water once again was removed and construction proceeded normally.

Figure 167. Sutter-Butte Canal Outlet

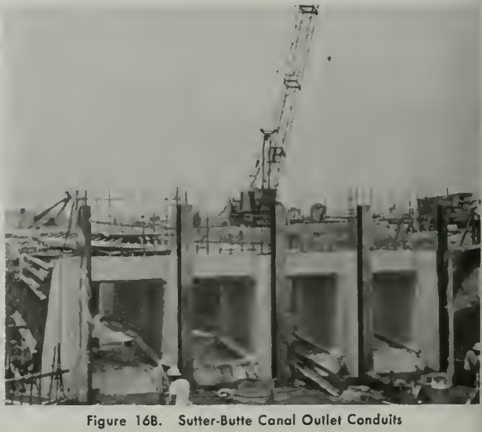

Figure 168. Sutter-Butte Canal Outlet Conduits

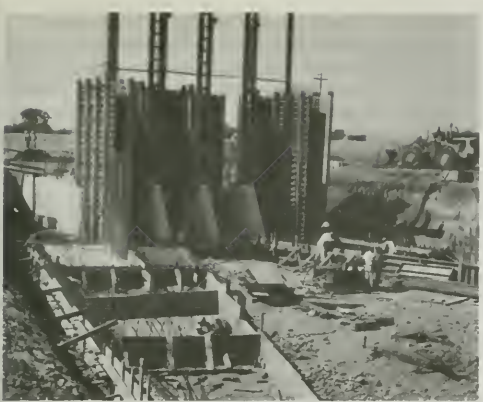

Figure 169. Sutter-Butte Canal Outlet Intake

Figure 170. River Outlet

Headworks. The headworks structure was built on in-place gravels. The foundation was excavated to elevation 95 feet and brought back to subgrade with compacted, semipervious, Zone 3A material. Subgrade was at elevation 101.0 feet for the entrance slab and at elevation 99.5 feet for the radial-gate slab. Blockouts were provided in the piers to receive the radial-gate wall plates. This permitted the wall plates to be adjusted to final position with the radial gates in place. Stoplog guides were installed upstream and downstream from the radial gates.

The county road bridge and service bridge slabs were hand-screened and finished without the aid of a mechanical finishing machine. Each bridge slab was covered with carpet and kept wet for curing.

The county road bridge abutments and the radial-gate end walls were counterforted without using horizontal construction joints.

Fish Barrier Weir. The fish barrier weir structure (Figure 171) is located on the bank of the Feather River approximately 860 feet southeast of the river outlet headworks.

A hammer, powered by compressed air supplied by a 900-cubic-foot-per-minute rotary compressor, was used to drive the sheet piling cutoffs under the weir. The downstream cutoff was driven to tip elevation 80 feet with head or cutoff elevation 99 feet in the channel invert. Upstream sheet piling was driven to tip elevation 60 feet with head or cutoff elevation 101 feet in the channel invert. No difficulty was encountered in driving the downstream sheet pile tips to elevation 80 feet but, when the upstream sheet pile tips reached elevation 70 feet, 1-inch penetration required 150 to 300 hammer blows. It was agreed that 150 blows per inch of penetration of the sheet piling would be accepted as refusal.

Figure 171. River Outlet—Fish Barrier Weir

The structural excavation was performed by a dragline with a 3-cubic-yard bucket and end-dump trucks. Excavated material was wasted in old dredger ponds or used for channel dikes between the fish barrier and the river outlet headworks.

Sheet piling was capped by the concrete slabs of the structure. Concrete for the 3:1 sloped exit slabs was placed using slip forms.

Face forms for the ogee crest of the weir were stripped within two hours after concrete was placed, and all surfaces received a hardwood float finish.

Falsework to support the service bridge slab was composed of 6- by 8-inch bents with 6- by 6-inch caps. Telltales were used to check any deflection during concrete placement. The maximum deflection was ¼ inch. The slab surface was hand-finished without any problems.

Concrete Production

Except for two sections of the river outlet approach, walls and minor drainage structures, all structural concrete used on the afterbay outlets was produced at the batch plant located at the site of Thermalito Powerplant. The remainder was produced at a plant in Oroville. Control of actual batching operations was done by department personnel.

Concrete was transported from the Thermalito Powerplant batch plant by agitator trucks with an 8-cubic-yard capacity. The remaining concrete was delivered to the job from Oroville by various transit mixers, ranging in capacity from 6 to 11 cubic yards.

Water cure was required on structures for a period of 10 days. This was accomplished by covering the structures with water-saturated carpets kept wet by soaker hoses. Carpets were kept in place for 4 days following the 10-day curing period.

Membrane curing was allowed on subgrade placements, surfaces to be backfilled, and scattered miscellaneous structures. Membrane curing compound was white-pigmented "Hunts".

Reservoir Clearing

Clearing was required within the entire construction area and the entire area below the maximum reservoir water levels. Within the cleared area all trees, structures, and obstructions were leveled to the existing ground line and all combustible material was burned.

Grubbing was required only under embankment foundations where tree roots, pipes, or other material were buried within the top 3 feet of the foundation. Trees were pushed over with a large bulldozer, and the roots were removed with a ripper attachment on the same equipment.

All known wells within the reservoir and wells that were discovered during the course of construction were backfilled in accordance with specifications. Designated wells located outside the reservoir were preserved for observation of piezometric levels.

Closure

Since no flowing stream or irrigation ditches existed in the Forebay, closure did not present any problems. In the Afterbay, the main closure problem centered around the Western Canal. The outlet structure was built adjacent to the existing canal and, upon completion in the spring of 1967, the Canal was rerouted so that water ran through the completed structure and the dam was constructed across the old canal alignment. The original inlet for Western Canal on the Feather River had to be maintained until the Forebay was filled and could feed Western Canal through the tail channel. At that time, October 1967, the Afterbay Dam was closed across the old Western Canal just north of the river outlet. Closure of the Sutter-Butte Canal presented no problems because it was possible to supply water through existing facilities outside the Afterbay during the period of closure. The PG&E lateral canal was shut down during the period of closure, and Richvale Canal was not yet in service when the Afterbay was constructed.

Instrumentation and Toe Drain Observations

Instrumentation was observed from the time of installation throughout the entire construction period.

After filling of the reservoirs, flows from the embankment toe drains were estimated to average less than ½ gallon per minute (gpm), and many drains remained dry. No unusual settlement or alignment deviations are evident in the embankment nor is there any evidence of deterioration of slope protection.

Seepage

Forebay. When the Forebay was filled, water in the piezometers between Stations 78+00 and 91+00 rose rapidly. Seven relief wells failed to alleviate the situation and, by April 1968, the piezometric surface was above ground level. The situation was corrected in September 1968 by drilling 11 more wells and collecting water in an open ditch, then pumping back into the reservoir. In June 1969, the system was improved by cutting off the well casings about 4 feet below ground level and installing a 10-inch, perforated, asbestos-cement pipe to interconnect the relief wells. A 100-gpm submersible pump now returns the water to the reservoir.

Afterbay. A similar situation was observed along Highway 99 when the Afterbay was filled. After lowering the water surface and an unsuccessful attempt to blanket probable sources of leakage with bentonite, the Afterbay Ground Water Pumping System was proposed and implemented.

The Afterbay Ground Water Pumping System involves 15 irrigation-type wells spaced around the west and south sides of the reservoir. Eleven of the wells are along the west side, situated between the dam and Highway 99. The remaining four are located on the south side along Hamilton Road. The total pumping capacity is about 28,000 gpm.

The purpose of the system is to mine the ground water on the immediate land side of the dam, thereby lowering the piezometric level in the surrounding ground. The pumped water is returned to the Afterbay.

Criteria used for setting the number of wells, their size, and location included data collected from the ground water monitoring program, topographic low points, known geologic data under the floor of the reservoir, results of two aquifer pumping tests, and availability of land. The total depth of each well drilled was based on judgment of interval of aquifer intercepted to produce an adequate drawdown. The depth of casing and perforation was determined from logs and resistivity tests on the drilled holes.

Each pump is controlled separately by start and stop probes set at the desired high and low ground water level.

Tail Channel. Excavation of the tail channel exposed an area of pervious material in the left bank just downstream of Thermalito Powerplant. This area has been monitored for seepage and signs of movement. Seepage from the Forebay enters the tail channel through this stratum but has not caused any damage or movement. This is considered beneficial since it lowers the water table in the area, alleviating the need for relief wells outside the Forebay adjacent to the Powerplant.

BIBLIOGRAPHY

Bucher, Kenneth G., "Hydraulic Investigation of the Thermalito Afterbay River Outlet", Water Science and Engineering Papers 1029, Department of Water Science and Engineering, University of California, Davis, June 1969.

California Department of Water Resources, Bulletin No. 117-6, "Oroville Reservoir, Thermalito Forebay, Thermalito Afterbay: Water Resources Recreation Report", December 1966.

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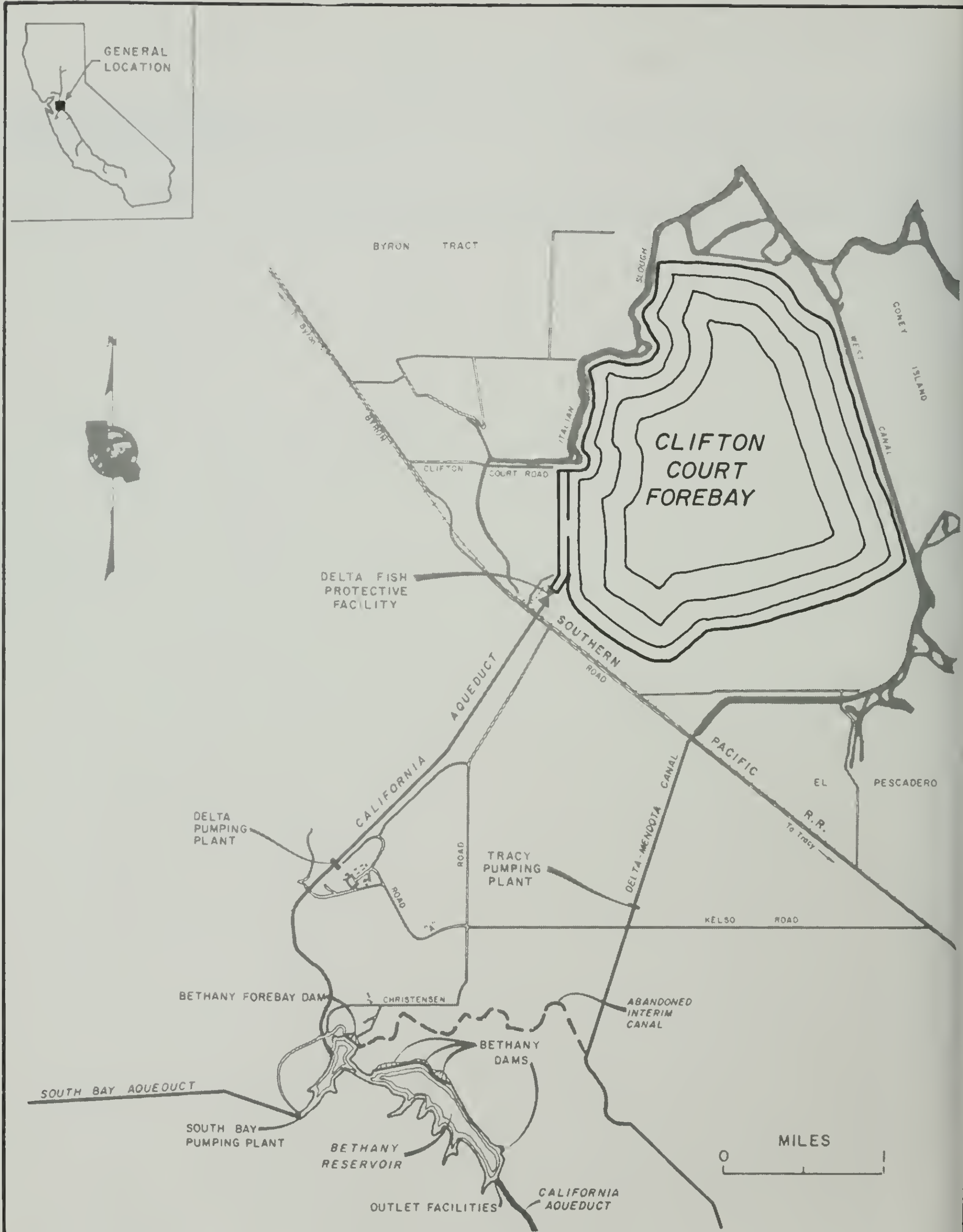

Figure 172. Location Map—Clifton Court Forebay

CHAPTER VIII. CLIFTON COURT FOREBAY

General

Description and Location

Clifton Court Forebay is a shallow 28,653-acre-foot reservoir at the head of the California Aqueduct. It was formed by constructing a low dam inside the levees of Clifton Court Tract. The Forebay is located in the southeast corner of Contra Costa County about 10 miles northwest of the City of Tracy adjacent to Byron Road (Figures 172 and 173).

A gated control structure connected to West Canal, a channel of Old River, allows Sacramento-San Joaquin Delta water to enter the Forebay. Water leaves the Forebay through a designed opening in the east levee of the Delta Pumping Plant intake channel just north of the Delta Fish Protective Facility. Until the latter connection was made, the intake channel was connected to Italian Slough, located on the west side of Clifton Court Tract, to furnish water for initial operation of the California Aqueduct. A statistical summary of Clifton Court Forebay is shown in Table 17.

Purpose

Clifton Court Forebay provides storage for off-peak pumping and permits regulation of flows into Delta Pumping Plant. This regulation dampens surges and drawdown which would be caused during peak pumping periods. When the water surface of the Delta channels falls below that of the Forebay because of tidal action, the control structure gates can be closed to prevent backflow. Inflows to the Forebay generally are made during high tides and can be controlled with gates to reduce approach velocities and prevent scour in the adjacent Delta channels. Sediment removal has proven to be an additional benefit of the Forebay which will result in reduced canal maintenance costs. Ultimately, the planned Peripheral Canal will supply the Forebay with water, bypassing the Delta channel system.

Chronology

Preliminary design of Clifton Court Forebay began in March 1965.

Exploration drilling for the Forebay commenced April 13, 1966, the design plans and specifications were completed on July 6, 1967, the contract was awarded on November 27, 1967, and all work was completed on December 17, 1969.

Regional Geology and Seismicity

Clifton Court Forebay is located at the southwestern edge of the Sacramento-San Joaquin Delta, where the flat Delta basins merge with the gentle slopes at the base of the Coast Range. These two physiographic regions intersect approximately along the sea-level contour in the southern and western edges of Clifton Court Tract.

The Forebay is entirely underlain by Quaternary alluvium consisting of deltaic sediments in the central

Figure 173. Aerial View—Clifton Court Forebay

TABLE 17. Statistical Summary of Clifton Court Forebay

CLIFTON COURT FOREBAY DAM	
Type: Zoned earthfill	
Crest elevation	
Where exposed to delta waterway	14 feet
Where not exposed to delta waterway	11 feet
Crest width	20 feet
Crest length	36,500 feet
Lowest ground elevation at dam axis	−10 feet
Lowest foundation elevation	−16 feet
Structural height above foundation	30 feet
Embankment volume	2,440,000 cubic yards
Freeboard	
Above maximum probable delta flood surface	5 feet
Above maximum operating surface	6 feet

CLIFTON COURT FOREBAY	
Maximum operating storage*	28,653 acre-feet
Minimum operating storage	13,965 acre-feet
Dead pool storage	not applicable
Maximum probable delta flood surface elevation	9 feet
Maximum operating surface elevation*	5 feet
Minimum operating surface elevation	−2 feet
Dead pool surface elevation	not applicable
Shoreline, maximum operating elevation*	8 miles
Surface area, maximum operating elevation	2,109 acres
Surface area, minimum operating elevation	2,088 acres

SPILLWAY	
No spillway necessary	
Top of embankment is above all surrounding ground	

INLET WORKS	
Control: Concrete structure with five 20-foot-wide by 25-foot - 6-inch-high radial gates	
Design flow	10,300 cubic feet per second
Design velocity	2 feet per second

OUTLET	
Intake channel connection:	
Design flow	10,300 cubic feet per second
Design velocity	2.5 feet per second

* Without Peripheral Canal.

and northern portions and alluvial fan deposits along the southern and western margins.

No known active faults occur at, or adjacent to, Clifton Court Tract. The area is located approximately 21 miles from the nearest known active fault, the Calaveras fault, and about 45 miles from the San Andreas fault. Seismic considerations similar to those used on the California Aqueduct intake channel and its related structures were applied to the Forebay.

Design

Dam

Description. The dam, which has a maximum height of 30 feet, has two basic compacted zones and is ballasted with uncompacted material (Figure 174). Zone 1 material, which consists of fairly uniform inorganic silty and sandy clays, was placed on the reservoir side of the embankment. The balance of the embankment proper, designated Zone 2, consists of inorganic clays, sands, and silts. Waste materials, such as peats and soft organics, were placed as ballast on the outside of the embankment where needed for stability and were designated Zone 3. Slopes are protected from wave action with soil-cement consisting of nine pounds of cement per cubic foot of soil.

Foundation. The dam alignment rests almost entirely on deltaic sediments which consist of nonorganic flood-plain deposits covered by a blanket of organic and peaty soils. The organic blanket ranges in thickness from less than 1 foot to over 12 feet. In general, the organic soils have low shear strengths and low densities. They include soft organic clays, organic silts, and peat in various stages of decomposition. At first it was thought that the organic soil should be removed, but the existing Clifton Court levees, which had been constructed on this soil at steeper side slopes than planned for the Forebay, showed that the organic soil was usable as a foundation. Reinforcing the same existing levees to serve as forebay embankments was ruled out because the strengths of both levee and foundation were indeterminable.

Construction Materials. Embankment materials were excavated from the floor of the Forebay (Figures 175 and 176). Zone 1 material was limited to Borrow Area A, Zone 2 material was selected from excavation throughout the Forebay, and Zone 3 material was excavation that proved to be unsuitable for placement in Zone 2. Sand and sandy silt for use in soil-cement were obtained from an old streambed within the Forebay, and the filter material was obtained from the Pacific Coast Aggregates Company located in the City of Tracy.

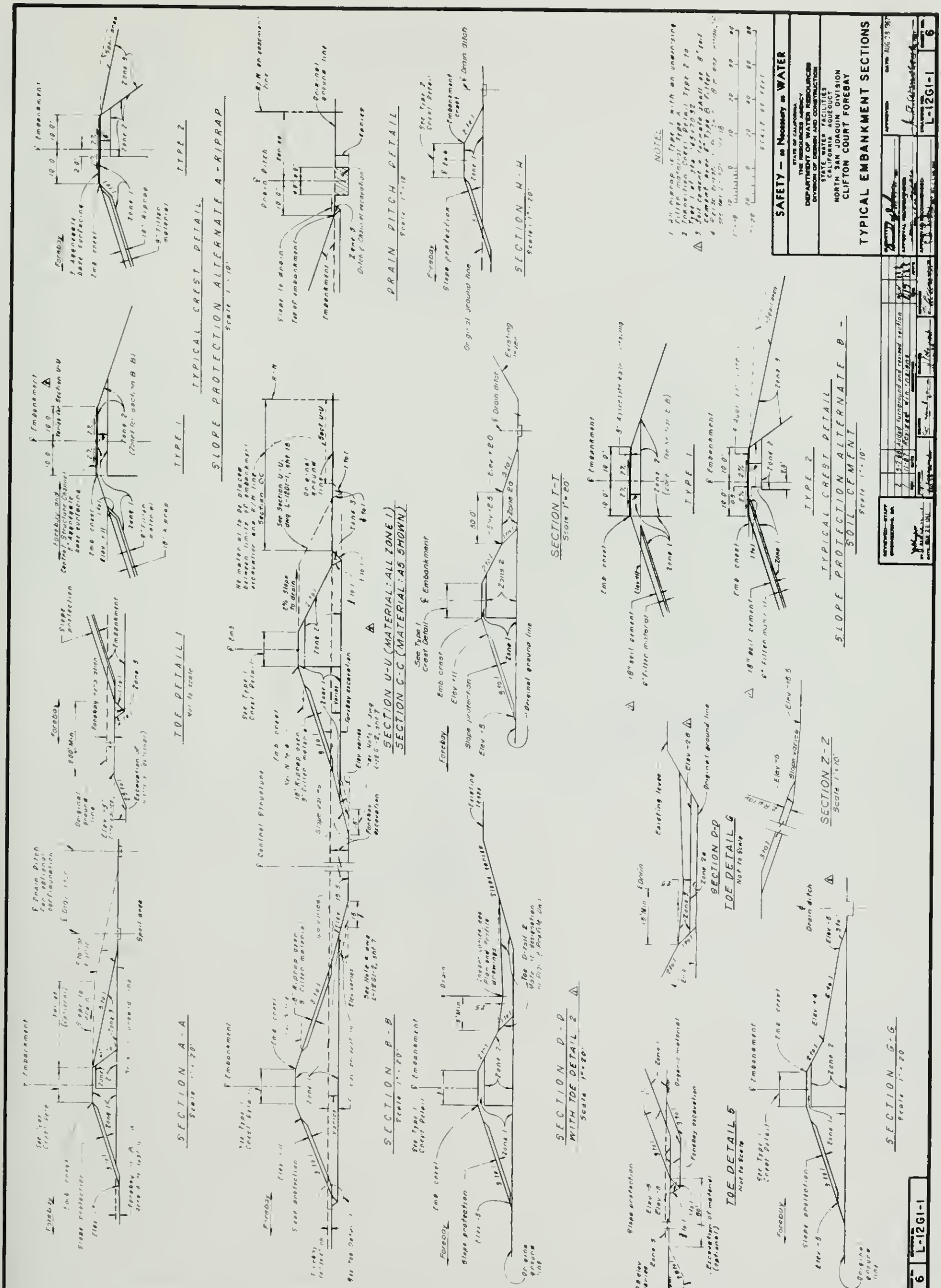

Figure 174. Embankment Sections

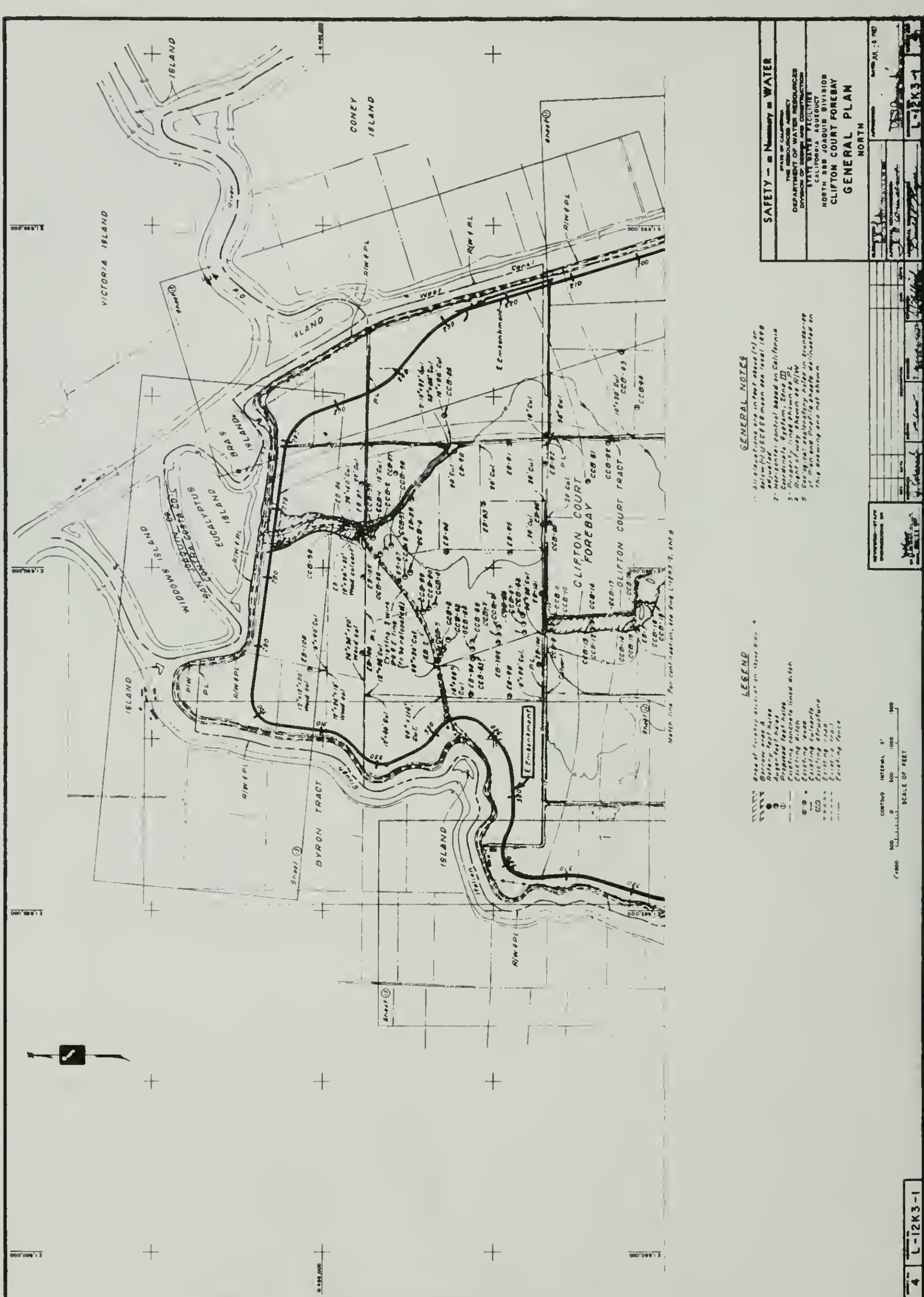

Figure 175. General Plan of Forebay—North

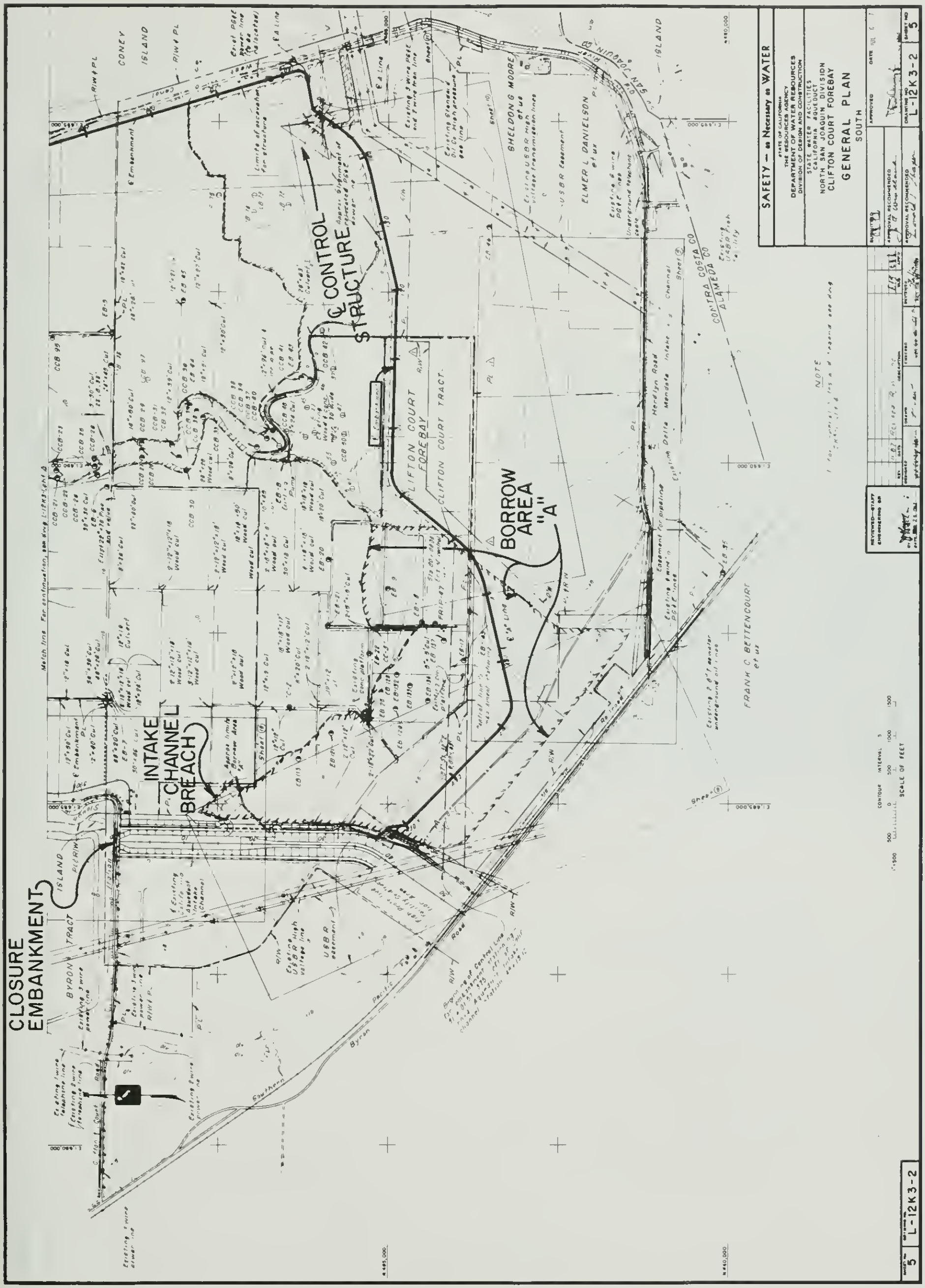

Figure 176. General Plan of Forebay—South

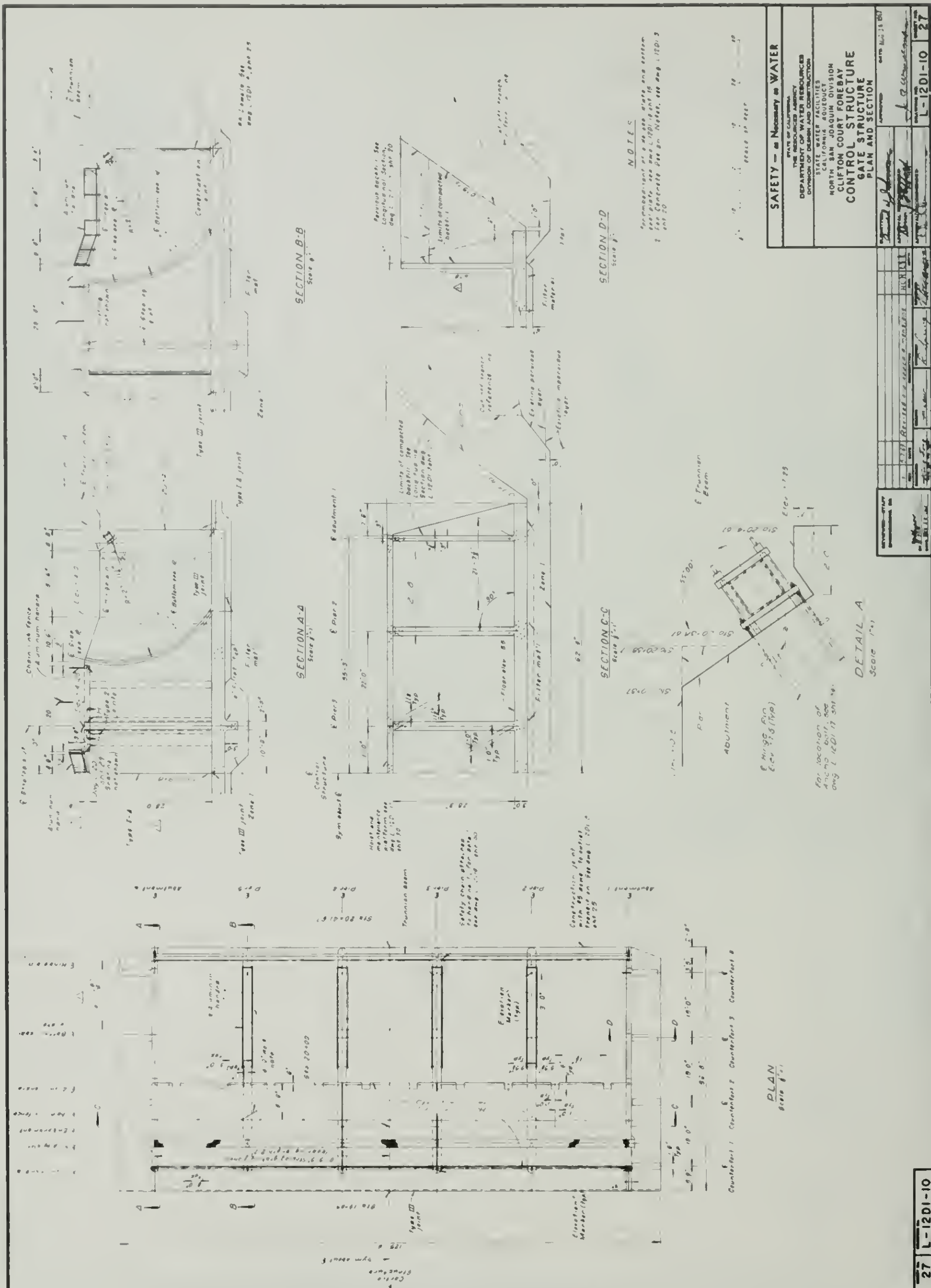

Figure 177. Gated Control Structure

Stability Analysis. The embankment was designed using the Swedish Slip Circle method of analysis employing a seismic force of 0.15g applied in the direction that would produce the lowest factor of safety for the condition being analyzed. Soil properties used in the analyses are given in Table 18.

Settlement. Probable total consolidation of the embankment foundation was computed to be from 1.4 feet to 2.1 feet, including a postconstruction consolidation range of 0.7 to 0.9 of a foot. Specifications required that the compacted embankment be constructed in stages of 4 feet and in several marginal areas in stages of 3 feet. It was required that a period of at least six weeks be allowed to elapse before making any additional placement upon a previous stage. This permitted consolidation to take place and minimized the buildup of pore pressures in the weak foundation material so that it could support the next embankment stage.

Control Structure

To regulate flow into Clifton Court Forebay and to isolate the Forebay from the Delta, a gated control structure was required (Figures 177 and 178). It will serve only as emergency control once the Peripheral Canal is completed. This structure consists of five 20-foot-wide by 25.5-foot-high radial gates, housed in a reinforced-concrete gate bay structure. The gate structure includes a 10-foot-wide bridge for vehicular traffic, one set of stoplog slots, and a hoist platform. A riprapped earthen transition extends from both the inlet and outlet of the structure. A 1,000-foot-long riprapped channel with a 300-foot base width connects the control structure to West Canal. A log boom was provided at the channel inlet to discourage boats from entering the area (Figure 178).

The control structure was designed for a maximum head differential across the gates of 6 feet of water under normal design stresses. For the extreme case, a head differential of 12 feet of water, one-third overstress, was allowed. The control structure inlet channel was sized to accommodate 16,000 cubic feet per second (cfs), the maximum tidal flow from West Canal, at an average velocity of 3 feet per second (fps). Riprap was placed in the portions of the earth transition where the average velocity could exceed 3 fps. The structure itself was designed to pass a continuous flow of 10,300 cfs with resulting velocities between 5 and 7 fps through the gate bays, depending on reservoir stage. If a high tide were to coincide with low water in the reservoir during the interim operating period, the full flow of 16,000 cfs could pass through the structure without causing any damage.

Figure 178. Control Structure and Inlet Channel Connection to West Canal and Old River (in background)

TABLE 18. Material Design Parameters—Clifton Court Forebay

Material	Specific Gravity	Unit Weight in Pounds Per Cubic Foot			Static Shear Strengths θ Angles in Degrees Cohesion in Tons Per Square Foot					
					Effective		Total		Construction	
		Dry	Moist	Saturated	θ	C	θ	C	θ	C
Foundation										
Clay										
0 to 8 feet	--	--	--	120	--	--	18	0.15	--	--
8 feet and lower	--	--	--	117	--	--	18	0.15	--	--
Sandy Silt	--	--	--	122	35	0	--	--	--	--
Sand	--	--	--	122	38	0	--	--	--	--
Organics	--	--	--	90	--	--	--	--	--	--
Embankment										
Impervious	--	--	118	125	--	--	25	0.2	--	--
Pervious	--	--	120	125	35	0	--	--	--	--
Riprap and Filter	--	--	--	120	35	0	--	--	--	--

The foundation consists of alternating layers of lean clay and sand below the organic and peaty soils. The structure was set on the uppermost layer of sand. To control uplift caused by high water levels in the Delta, it was necessary to excavate a cutoff trench at least 1 foot into the underlying clay strata. This trench completely encircles the control structure and is filled with compacted impervious material. It was judged desirable to construct the embankment adjacent to the structure at an early date to prevent differential settlement that could damage the structure. A surcharged fill was placed over the structure abutments to obtain most of the settlement before the structure concrete work was initiated. Pressure from this fill was 2,000 pounds per square foot (psf) greater than the proposed structural load to obtain more rapid consolidation of the embankment foundation. Preconsolidation duration was selected so that when structural excavation began, settlement rates for both the structure and the adjacent embankment foundations would be approximately equal.

The five 20-foot-wide gate bays were formed by counterforted abutment walls at each edge of the structure and by reinforced-concrete piers between the bays. The piers were designed as columns with cross-section widths sufficient to block out stoplog slots. The floor slab of the gate structure was designed as a one-way slab on an elastic foundation. The deck slab, which constitutes the hoist and maintenance platform and vehicular bridge, was designed as a simple span which would support hoist equipment and a H20-S16-44 bridge load. Studies indicated that for all static loading and uplift conditions, the resultant forces fall within the middle one-third of the foundation. For earthquake loading, the resultant falls within the middle one-half of the foundation. Five 20-foot-wide by 25.5-foot-high, tapered, radial gates are supported within the gate bays by means of the hoist cables and trunnion beams. A standby generator was provided for use in case of utility power failure.

Intake Channel Connection

The opening from the Forebay to the Delta Pumping Plant intake channel (Figure 172) was designed to pass 10,300 cfs, the maximum capacity of the plant, at an average velocity of 2.5 fps. The opening is an earth-lined channel extending into the Forebay with a level invert at elevation −15.5 feet (U.S. Coast and Geodetic Survey sea-level datum of 1929). The invert of the flare section within the Forebay slopes upward at 35:1 and daylights at original ground. The flare section, like the level portion, was designed for a maximum velocity of 2.5 fps.

Intake Channel Closure

The closure embankment (Figure 179) was provided to close permanently the earlier connection of the intake channel to the Delta at Italian Slough and provide a base for the permanent Clifton Court road crossing of the intake channel. It consists of granular fill topped with compacted Zone 1 material and a sheet-piling core in the center of the embankment extending from elevation 9 feet, the top of the granular fill, to elevation 23 feet, 8 feet below the channel invert. Existing riprap was removed for 10 feet on each side of the sheet piling. Granular material was used to assure speedy placement of a stable fill under water. The closure embankment was placed to elevation 15 feet, 1 foot above the adjoining embankment, to allow for postconstruction consolidation.

Piping and Drainage Systems

Four pump structures were installed between the embankment of Clifton Court Forebay and the original levee to drain accumulated surface water and seepage (Figure 180). Each structure consists of a vertical, 72-inch, reinforced-concrete pipe placed on a 1-foot-thick concrete pad; a 24-inch, reinforced-concrete, inlet pipe; a 900-gallon-per-minute centrifugal pump equipped with a 1,200-rpm electric motor; and automatic controls actuated by a metal float. Water is discharged into the Forebay from each pump through an 8-inch steel pipe installed through the embankment.

Construction

Contract Administration

General information about the construction contract for Clifton Court Forebay is shown in Table 19.

TABLE 19. Major Contract—Clifton Court Forebay

Specification	67-45
Low bid amount	$4,421,606
Final contract cost	$5,904,116
Total cost-change orders	$254,953
Starting date	12/12/67
Completion date	12/17/69
Prime contractor	Gordon H. Ball Enterprises

Dewatering and Drainage

Dewatering was the contractor's responsibility and the work was divided into two separate operations: first, maintaining the water surface in all drainage ditches at elevation −13 feet or lower until filling of the Forebay commenced; and second, dewatering and draining the area where the control structure was to be located to elevation −20 feet. Dewatering to elevation −13 feet was accomplished by cleaning and deepening existing ditches and using existing pumps. Interceptor ditches were constructed to tie in existing ditches.

In accordance with specifications, the dewatering operation around the control structure was extended more than 25 feet beyond the outer limits of the control structure. The contractor, in sequence, excavated a 1:1 trench in sandy material to elevation −26 feet, placed a 3-foot blanket of washed concrete sand in the trench, installed pervious concrete pipe 8 inches in diameter, and backfilled with another 3 feet of sand.

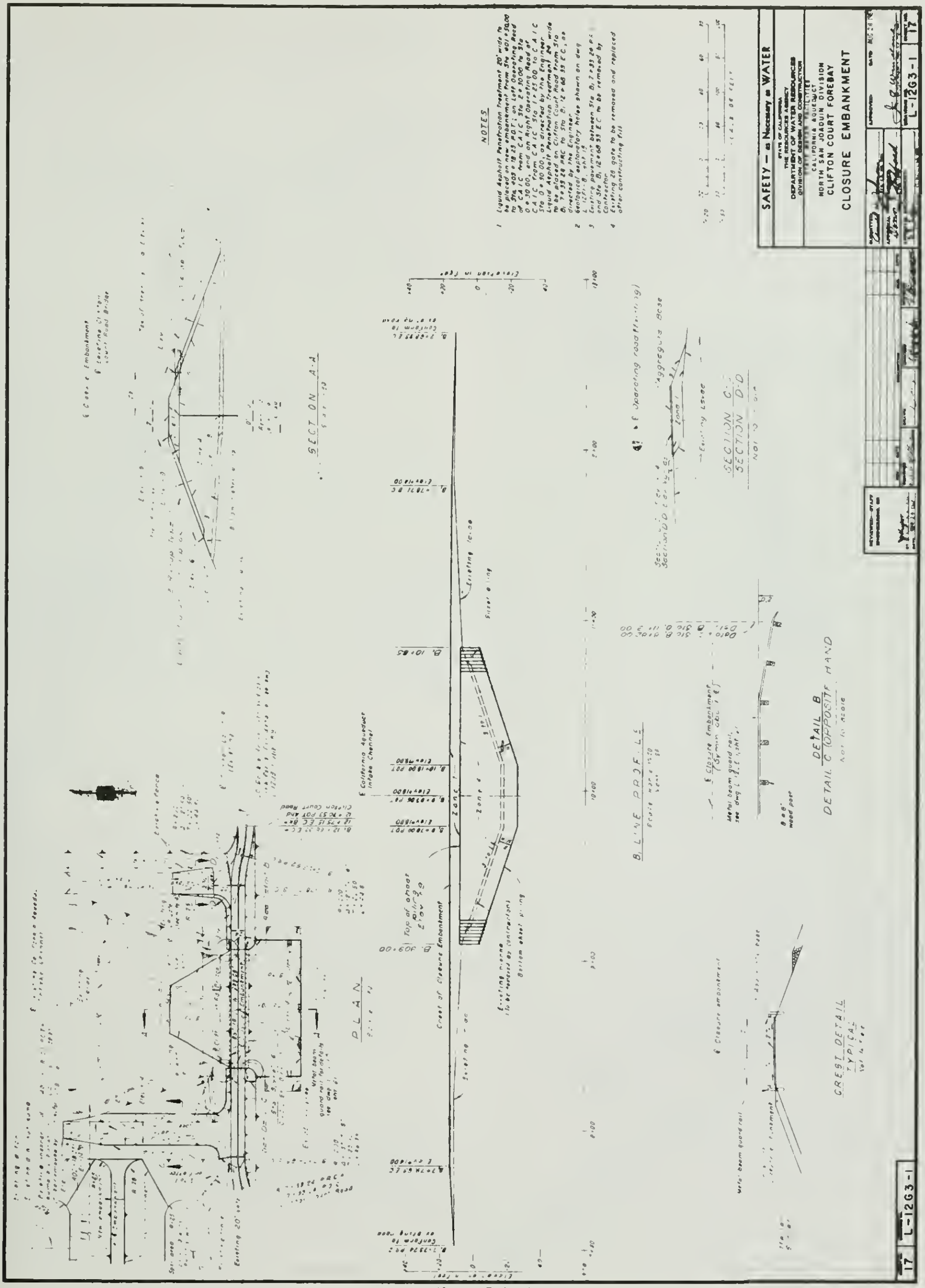

Figure 179. Closure Embankment

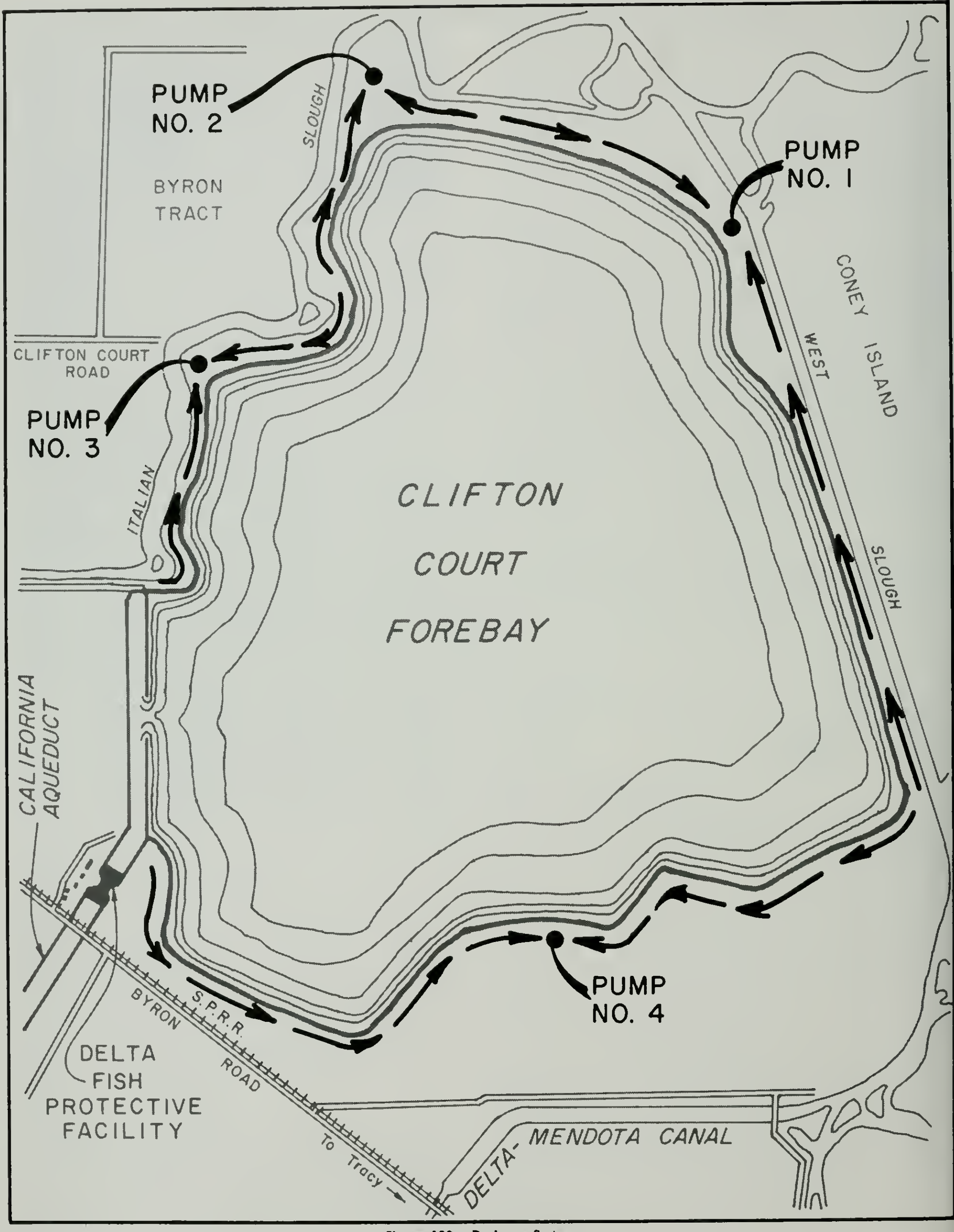

Figure 180. Drainage System

Within a day, the pipe had become completely choked and ceased to function. The piping project was abandoned in favor of open drainage with pumped disposal from two 30-inch-diameter riser pipes.

Reservoir Clearing

Clearing and grubbing, accomplished throughout the construction period, consisted of removing all trees, stumps, brush, culverts, downed timber, tanks, fences, buildings, privies, cesspools, leaching lines, discarded equipment, and debris. In addition, all existing ditches were backfilled. Grubbing was minimal because the Forebay was built on large parcels of open farmland.

Stripping was complicated by several factors. First, the rich soil, deep roots, warm days, and rains caused quick regrowth and second, the weak foundations necessitated the halting of restripping.

The contractor was required to pump, clean, and flush all septic tanks; remove all filter material from leaching lines; and dispose of the material outside of the Forebay. Pit privies were dug out and excreta deposits removed and disposed of outside the forebay area. Septic tanks and leach lines were dug out with small backhoes and the debris hauled to approved waste areas.

Large-diameter pipes through the old levees were excavated by a dozer, or a grader, and smaller pipes were removed by using a rubber-tired backhoe equipped with a loader bucket. Because of the critical nature of this work, removal of pipe from the old levees was paid for as structural excavation and backfill.

Excavation

Excavation was performed under four categories: Forebay, Borrow Area A, structural, and ditch and channel.

Forebay. Forebay excavation included material used for Zones 2, 2A, and 3, all of which was utilized in embankments and the various toe details or was spoiled. The contractor had two separate excavation operations: one for stripping surface organic materials which were placed in ballast fills or in waste banks, and another for the underlying material which was placed as Zone 2 and 2A embankment.

Forebay stripping excavation equipment consisted of scrapers, jeep tractors pulling scrapers, and double-engine scrapers. Large dozers were used exclusively as pushers at both the pit and the fill.

The other forebay excavation equipment spread consisted of a belt loader drawn by a large dozer. Bottom dumps were loaded from the attached conveyor belt. Because of soft ground, a dozer was needed to assist the bottom dumps in and out of the area. Where conditions were too wet or too soft for the belt loader to operate, the scraper spread was used to complete the required excavation.

Borrow Area A. Material from Borrow Area A (Figure 176), the southwest portion of the Forebay, was used in Zone 1 embankment. It was mostly impervious material and fairly plastic. The contractor was unsuccessful in excavating this material with the belt loader, and the bottom dumps were too heavy for use on the wet clay material. Most of Borrow Area A was excavated with a scraper spread.

Structural. The bulk of structural excavation consisted of the removal of control structure surcharge. Also included was removal of unsuitable material under the control structure surcharge and excavation for keys, cutoff trenches, concrete transitions, piping, drainage pump structures, float wells, and control building footings.

Ditch and Channel. Ditch and channel excavation was separated into two general types: that dug with scrapers, and that dug with backhoe or dragline equipment.

The trenches around the perimeter of the Forebay that drain water to one of the four pump sumps were excavated with scrapers and blades. These trenches were either vee trenches or vertical wall trenches and usually were constructed in Zone 3 material. Vee trenches were cut with a blade and are similar to a swale. Vertical wall trenches were excavated by using self-loading scrapers (paddle wheels).

The existing trenches to be crossed by the new embankment were first cross-sectioned and then excavated until suitable material was reached. This excavation, for the most part, was done with a Gradall which reached down and removed the mud and silt from the bottom until firm material was found. When these excavations were not filled quickly with Zone 1 material, fine silt particles carried by ground water tended to accumulate on the bottom of the trench, leaving an undesirable situation necessitating reexcavation. Once these trenches were backfilled with Zone 1 material, they did not cause any further trouble.

Handling of Borrow Materials

Impervious borrow material (Zone 1) was obtained from Borrow Area A. In place, this material was well above optimum moisture. When exposed to the wind and sun, the surface dried sufficiently to allow the top 18 inches to be taken at about 2% above optimum moisture. By the time it was transported, disked, and rolled, the material usually was at optimum moisture for compaction and little water had to be added at the embankment.

Zone 2 material was selected from various areas in the Forebay outside of Borrow Area A. This material was dry and required constant watering and disking to bring the moisture content up to optimum. Because of the availability of Zone 2 material in the northern part of the Forebay, it was used as ballast on the outside of the embankment from Station 229+80 to 363+80 (Figure 175) and was designated Zone 2A.

Zone 3 material was stripped from Borrow Area A and used as uncompacted ballast outside of the embankment around the southern portion of the Forebay.

Zone 4 material was supplied from an aggregate plant in Tracy and was used only in the closure embankment section.

Riprap required for purposes other than protection of the interior embankment slopes was obtained from department-owned stockpiles at Bethany Reservoir. Processing consisted of primary and secondary blasting and the use of a grizzly. Riprap was loaded with a loader, or wheel loader; a large dozer kept the material worked up into a pile at the pit. Eleven 10-cubic-yard dump trucks with special beds were used in the 4½-mile haul to the Forebay.

Embankment Construction

Compacted Material. Zone 1 and Zone 2 materials were placed in the same manner, except that Zone 2 material required constant watering at the embankment. The foundation was not strong enough to support the construction equipment, but it was found that when an uncompacted 3-foot lift was dozed out onto the foundation, equipment could travel over it. Once the initial uncompacted lift was placed, compaction proceeded smoothly. Material was brought to the fill by either scrapers or bottom dumps and placed in horizontal layers not exceeding 6 inches after compaction. Spreading equipment included a large dozer and a rubber-tired dozer which also pulled a larger hydraulic-operated double disc.

Compaction was accomplished by two pieces of equipment on each spread: first, a tractor with an attached dozer blade pulling a sheepsfoot roller; then a large, four-drum, electric power packer weighing 50 tons, the most effective piece of equipment. The power packer could be depended upon to obtain maximum densities in high fill areas. Relative compaction tests were taken in areas where compaction results were questioned, and good results were verified. The required relative compaction was 92%.

After completing each stage, the area was sloped to drain. When the 42-day waiting period was over, the embankment was disked and moisture-conditioned prior to placing another stage.

Uncompacted Material. Zone 2A and Zone 3 materials were placed in lifts not exceeding 12 inches and scrapers were routed over them. No difficulty was experienced with these placements.

Slope Protection. There were two alternatives for slope protection. Alternative A slope protection was riprap material which was to be imported as this large quantity was not available locally. Alternative B slope protection was soil-cement for which sand available in the Forebay could be used as aggregate. No contractor bid on Alternative A because of the higher cost.

The soil-cement mixing was done in a batch plant with automatic controls. Spreading was accomplished by a spreader box continuously fed by dump trucks. The method of compaction was by two passes with a steel-wheeled roller and no more than five passes with a pneumatic roller. The steel-wheeled roller was equipped with a special side roller that worked off the hydraulic system. It forced the small steel wheel into the side of the soil-cement, allowing it to roll the outer edge to obtain the desired stair-step effect. Curing was done by foggers that were installed on water trucks and worked off a high-pressure system on each truck.

The soil-cement slope protection, where subjected to severe wind wave action, has required maintenance since construction. Due to insufficient cement content and/or poor quality aggregate, certain layers of soil-cement have eroded badly, allowing the soil-cement slabs above to break up due to lack of support. Such locations have been repaired by placing riprap over the failure.

Closure Embankment. The closure embankment required removal and disposal of the existing bridge on Clifton Court Road that crossed the intake channel, removal of in-place riprap under the closure embankment, furnishing and placing the various categories of zoned closure embankment material, driving steel sheet piling, and rebuilding the road.

Bridge removal and pile driving were done from two large barges. One barge was equipped with a 40-ton crane with an 80-foot boom, and the other was equipped with a pile driver. The crane also was used to set and drive the sheet piling, and a clamshell bucket was used to remove the riprap.

A gravity drop hammer was used to place the steel sheet piling, and a vulcan, air-driven, single-action hammer was used to drive the sheets. These sheets were easily driven to the planned elevation, and Zone 4 material was then brought in on both sides of the sheet piling. Large amounts of the material had been stockpiled to facilitate the work, and trucks continued to bring material to the stockpiles as the closure was made.

After the Zone 4 material had been brought up to grade, Zone 1 embankment was placed. Aggregate base was spread on top, compacted, and oiled with 0.3 of a gallon of SC 250 per square yard. This area was covered with a layer of sand to allow automobile traffic to pass through the work.

Riprap was stockpiled on both sides of the closure and was placed as soon as the embankment had been topped out. After the riprap was placed, a guard rail was installed on each side of the road.

Construction of the closure embankment and breaching of the existing levee between Clifton Court Forebay and the California Aqueduct intake channel were coordinated. This was done so that water flowing into the California Aqueduct intake channel would not be less than 500 cfs for longer than any 3-day period, nor less than 2,000 cfs for longer than a 15-day period. The work was done at the end of the irrigation season. Work on the closure began on October 21, 1969 and was completed on October 30. The intake channel embankment was breached on October 31.

Breaching Levees. The West Canal levee was breached after forebay embankments, control structure, and riprap within the entrance to the Forebay were completed. Levee freeboard within the breached section was removed; then a section in the center of the breach was cut, thus enabling water to flow into the entrance to the control structure. The remainder of the levee was removed by a dragline, and riprap was placed underwater in the new passageway. When this was complete, the Forebay was filled to elevation −5 feet by allowing water to flow through the control structure at a rate of 500 cfs.

In order to breach the embankment between the intake channel and the Forebay, the water in the intake channel was lowered to elevation −5 feet. The breach was excavated to elevation −4.5 feet when the closure embankment cut off the flow from Italian Slough. Vertical openings of 150 square feet were made below elevation −4.5 feet and 700 square feet below elevation −2.0 feet to supply water to the intake channel from the Forebay. The Department controlled the water level in the Forebay to elevation +2 feet until the breach was completed. Excavation was done by a dragline, and riprap was placed in the opening underwater.

Control Structure

Concrete. The contractor used standard, ⅝-inch, exterior-grade plywood to form the inlet and outlet transition walls, but forms were not nailed to the studs in the conventional manner. Plywood was laid loose inside the 2- by 4-inch uprights, and only the vertical walers were nailed. This required closer spacing on the studs and walers but, according to the contractor, took less time and labor.

The contractor made good use of new products, such as the various developments in styrofoam and adhesives, which were used to seal the forms against grout leaks at the bottom, joints, and drilled holes.

Form work was done in the yard close to the structure. Concrete work on the control structure was begun on January 9, 1969 and completed on June 26, 1969.

Water cure for the general concrete work was tried in several ways. Use of carpets and plain burlap was not satisfactory, but cotton and burlap mats were heavy enough to stay in place. Water was introduced from the top by sprinklers to keep water flowing over the burlap. These mats could be draped over the wall immediately after the forms were stripped and cone holes dry-packed. A curing foreman worked seven days a week.

Mechanical Installation. The gates were erected in sections by the subcontractor in his Hayward, California yard. He supplied the five motor-operated gate hoists complete with gear motor and gear-motor brakes, shoe brakes, gear reducers, limit switches and drives, couplings, shafting, shaft bearings, drum support bases, and wire ropes.

It was necessary to modify the bottom seal plate to make it fit the side seal. Bottom seal plate bolts, which were installed earlier, had to be removed. This involved chipping out concrete to a sawed line, drilling in cinch anchors and bolts, and grouting as required by the plans. The gates were tested in the field by both the contractor and the Department.

Concrete Production

The contractor set up a batch plant approximately 700 feet from the control structure. The 1⅜-cubic-yard batches were discharged into 1½-cubic-yard buckets which were hauled two at a time by truck to the control structures, where a crane hoisted them to the work. Vibration was done by 3-inch electric and pneumatic vibrators. Sixty-cubic-foot-per-minute compressors supplied the air.

Twenty-eight-day strengths of the 1½-inch maximum size aggregate 5¼-sack mix used in the control structure averaged 4,300 pounds per square inch (psi), ranging from 3,310 to 5,360 psi. The design value for 28 days was 3,000 psi. The 7-day strengths averaged 2,400 psi, ranging from 1,890 to 3,210 psi, and the 91-day strengths averaged 4,980 psi, ranging from 4,130 to 5,770 psi. These figures were based on 38 tests taken from 3,219 cubic yards of concrete between January 8, 1969 and June 26, 1969.

Seven tests were made on four other mixes used in the 133 cubic yards of miscellaneous concrete. Test results indicated that the concrete was of satisfactory strength.

Electrical Installation

Electrical installation at Clifton Court Forebay included work at the control structure, control building, and pump structures. Cathodic protection and a standby generator also were provided. Excavation for the conduit between the control house and the control structure was done by a rubber-tired backhoe with a loader bucket. Initially, compaction was obtained by using whacker compactors. Later, the entire length of the trench was compacted by a large air tamper which straddled the trench and pounded the earth with equipment operating much like a pile driver.

Instrumentation

Instrumentation of Clifton Court Forebay was accomplished by using (1) settlement gauges, (2) slope indicators, (3) plastic tubes, and (4) structural monuments.

To monitor settlement of the embankment foundation during construction and subsequent operation, 64 settlement gauges (Figure 181) were installed at specified locations. During construction, areas which experienced large settlements were watched for

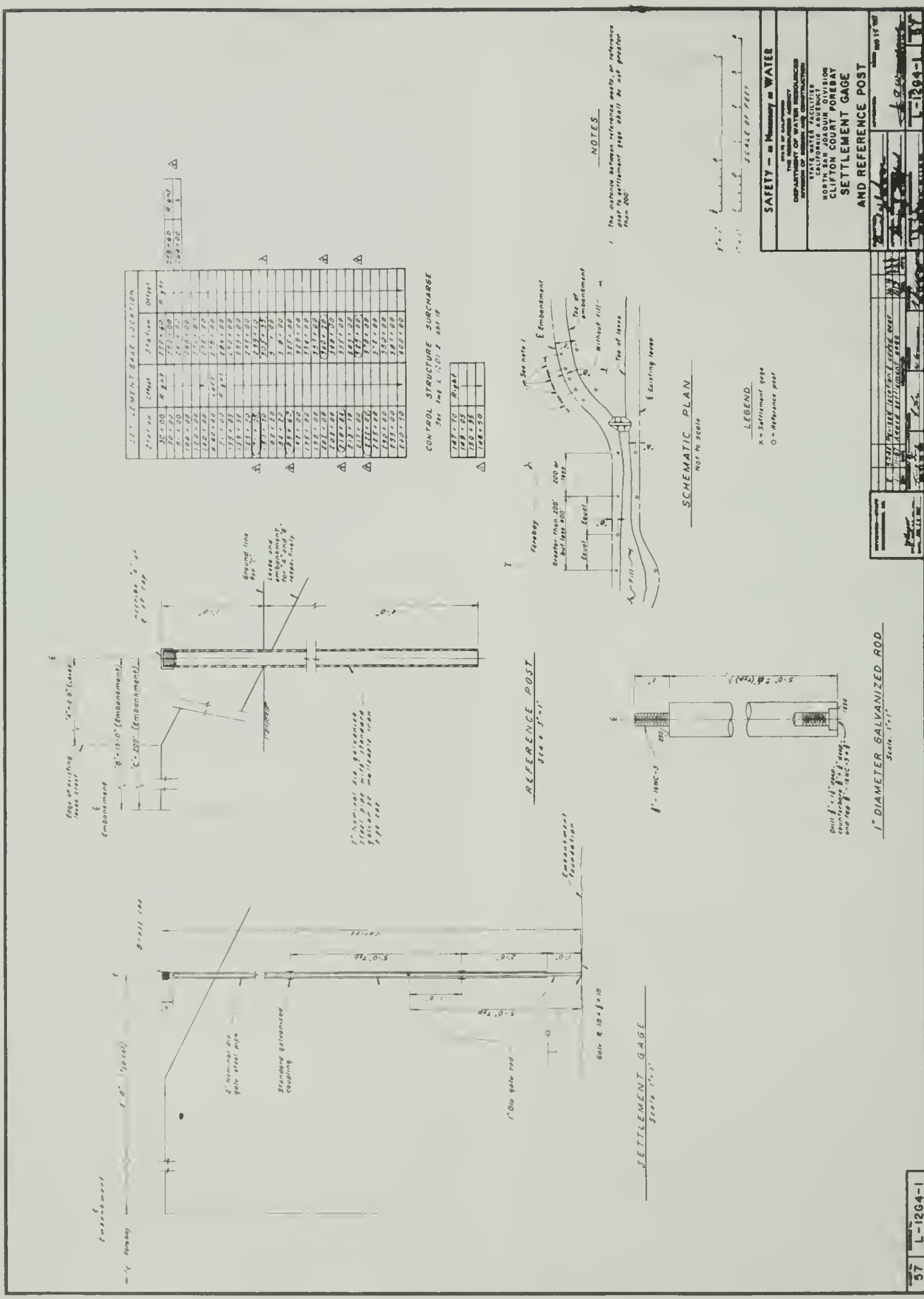

Figure 181. Test Installatians

evidence of potential failure and the rate of fill placement was controlled accordingly. Settlement during the period of construction ranged generally between 0.5 to 2.0 feet with a maximum of 2.5 feet at Station 252+00 (Figure 175). Following completion of construction and filling of the Forebay in late October 1969, settlement rates became nominal. During a seven-month period, from May to December 1970, maximum settlement was 0.07 of a foot.

During construction, 8 slope indicators and 83 plastic tubes were installed at seven locations on the forebay side of the embankment to detect and monitor possible horizontal ground movement caused by embankment construction. The plastic tubing was installed at 75-foot centers along two parallel lines at distances of 100 and 180 feet from embankment centerline. Although movement was noted in both the slope-indicator pipes and plastic tubes, none was considered indicative of horizontal movement in the foundation. Movements were erratic and frequently reversed direction, indicating the probable influence of adjacent drainage ditches and heavy-equipment operation. All of these instruments are now inundated.

Permanent bench marks installed on the control structure have been monitored periodically since July 1969. During the period July 1969 to October 1969, when the structure became operational, settlement of 0.14 of a foot occurred.

BIBLIOGRAPHY

California Department of Water Resources, Specification No. 67-45, "Specifications Bid and Contract for Clifton Court Forebay, State Water Facilities, North San Joaquin Division, Contra Costa County, California", 1967.

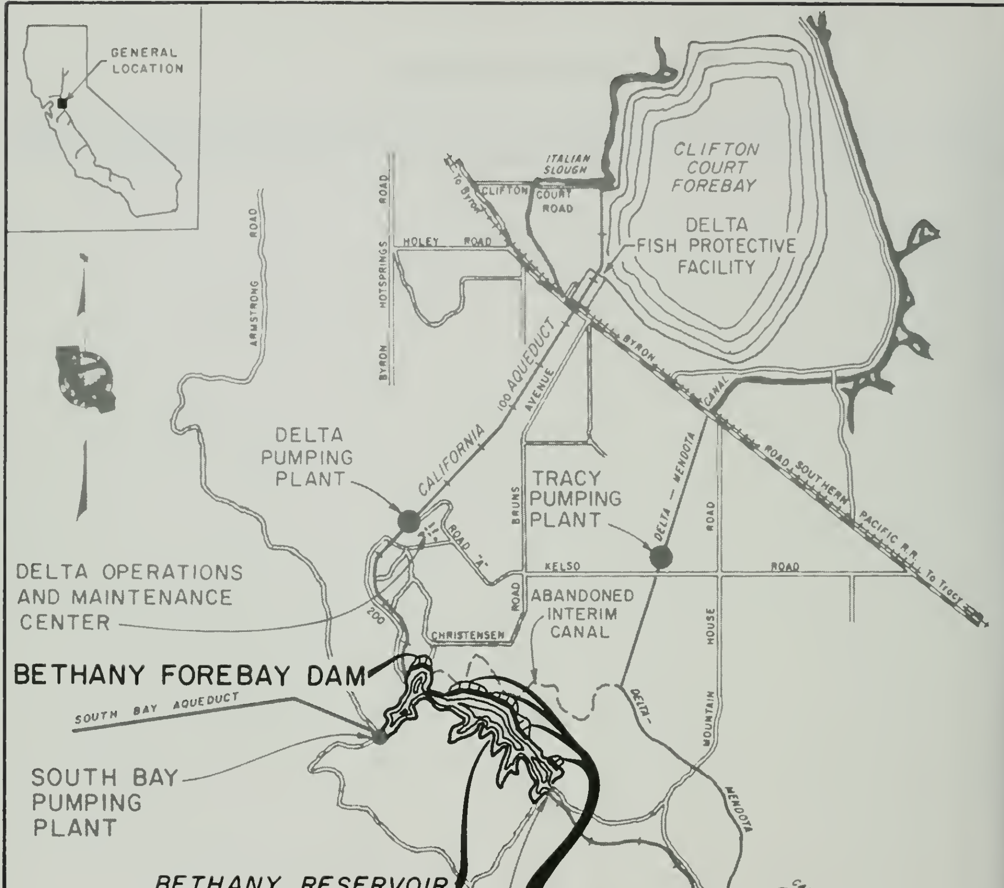

Figure 182. Location Map—Bethany Dams and Reservoir

CHAPTER IX. BETHANY DAMS AND RESERVOIR

General

Description and Location

Bethany Reservoir is a 4,804-acre-foot pool on the California Aqueduct, 1½ miles down the canal from Delta Pumping Plant. The Reservoir is impounded by five earth dams.

The facility is located approximately 10 miles northwest of the City of Tracy in Alameda County. The nearest major roads are U. S. Highway 50 and Interstate 580 (Figure 182).

A statistical summary of Bethany Dams and Reservoir is presented in Table 20 and the area-capacity curves are shown on Figure 183.

Purpose

Bethany Reservoir serves as a forebay for South Bay Pumping Plant and as an afterbay for Delta Pumping Plant. The Reservoir also serves as a conveyance facility in this reach of the California Aqueduct and provides water-related recreational opportunities.

TABLE 20. Statistical Summary of Bethany Dams and Reservoir

BETHANY DAMS	
Type: Homogeneous earthfill	
Crest elevation	250 feet
Crest width	25 feet
Crest length	3,940 feet
Streambed elevation at dam axis	170 feet
Lowest foundation elevation	129 feet
Structural height above foundation	121 feet
Embankment volume	1,400,000 cubic yards
Freeboard above spillway crest	5 feet
Freeboard, maximum operating surface	7 feet
Freeboard, maximum probable flood	2 feet
BETHANY RESERVOIR	
Maximum operating storage	4,804 acre-feet
Minimum operating storage	4,200 acre-feet
Dead pool storage	150 acre-feet
Maximum operating surface elevation	243 feet
Minimum operating surface elevation	239 feet
Dead pool surface elevation	190 feet
Shoreline, maximum operating elevation	6 miles
Surface area, maximum operating elevation	161 acres
Surface area, minimum operating elevation	150 acres

SPILLWAY	
Type: Ungated broad crest with unlined channel	
Crest elevation	245 feet
Crest length	100 feet
Maximum probable flood inflow	6,410 cubic feet per second
Peak routed outflow	1,560 cubic feet per second
Maximum surface elevation	248 feet
INLET	
California Aqueduct from Delta Pumping Plant	
Capacity	10,300 cubic feet per second
OUTLET WORKS	
Emergency outlet: Reinforced-concrete conduit beneath Forebay Dam at base of right abutment, valve chamber at midpoint—discharge into impact dissipator	
Diameter: Upstream of valve chamber, 60-inch pressure conduit—downstream, 48-inch steel conduit in a 78-inch concrete horseshoe conduit to manifold—24-inch pipe from manifold to dissipator	
Intake structure: low-level, uncontrolled	
Control: 24-inch butterfly valve at manifold—48-inch butterfly guard valve in valve chamber	
Capacity	121 cubic feet per second
OUTLETS	
South Bay Pumping Plant	
Capacity	330 cubic feet per second
North San Joaquin Division of California Aqueduct	
Capacity	10,000 cubic feet per second

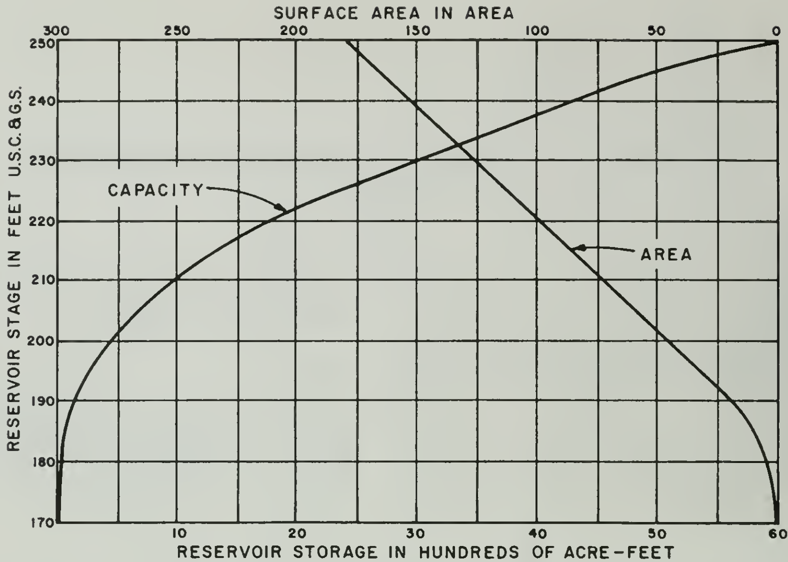

Figure 183. Area-Capacity Curves—Bethany Reservoir

Chronology

The five dams were built under two separate contracts. A single dam, designated the Forebay Dam, was constructed initially to create a reservoir to supply South Bay Pumping Plant (Figure 184). This initial pool was designated Bethany Forebay. After a few years of operation and with the construction of the California Aqueduct underway, the second contract was awarded for construction of four smaller dams southeast of the Forebay Dam. These four dams allowed expansion of the initial reservoir and provided the most economical conveyance facility for this portion of the California Aqueduct (Figure 185). The resulting pool is designated Bethany Reservoir.

The initial dam designation, Forebay Dam, has been retained; the later dams are referred to as Bethany Dams No. 1 through No. 4, or adjacent dams.

During the initial operational period of the South Bay Aqueduct (before December 1967), water was pumped into the Forebay from an interim canal joining the Delta-Mendota Canal. This interim canal was abandoned upon completion of the reservoir enlargement, and service from the California Aqueduct commenced.

Detailed design of the Forebay Dam was started in 1958, and construction was completed in 1961. Design of the four adjacent dams and expanded reservoir was started in 1965, and construction was completed in 1967.

Regional Geology and Seismicity

Bethany Reservoir is located in an area on the east flank of the Altamont anticline of the Diablo Range. Recent alluvium and sedimentary rocks of the Upper Cretaceous Panoche formation are the two geologic units present. Recent alluvium consists of clay and sandy clay with minor lenses of silt, sand, and gravel. The Panoche formation consists of interbedded shales, sandstones, and siltstones with occasional hard,

Figure 184. Bethany Forebay

Figure 185. Bethany Reservoir

calcareous, boulder concretions within the sandstone beds. The strongest earthquakes in the area in historic times are thought to have originated either on the San Andreas, Hayward, or Calaveras fault systems, all of which pass through the San Francisco Bay area. It is doubtful that any of these earthquakes in the project area exceeded an intensity of VII on the Modified Mercalli scale.

Design

Dams

Description. The 119-foot-high Forebay Dam was designed as a homogeneous rolled earthfill with internal sloping and horizontal drains located downstream from the axis of the dam. The plan, profile, and sections of the Forebay Dam are shown on Figure 186.

Adjacent dams, 50 to 121 feet in height, are of the same design except for the drainage system. These dams have strip drains comprised of granular materials surrounding perforated drain pipes in place of the Forebay Dam's continuous pervious blanket drain. The plan, profile, and sections of each of the four adjacent dams are shown on Figures 187 through 190.

Stability Analysis. Stability of the embankment sections was investigated by the Swedish Slip Circle method of analysis. Cases analyzed included full reservoir and critical lower reservoir levels combined with earthquake loading. Earthquake loading was simulated by the application of a horizontal acceleration factor of 0.1g in the direction of instability of the mass being analyzed.

Settlement. No detailed settlement analysis was made. Consolidation tests indicated, however, that practically all settlement would occur during construction. A camber of 1% of the fill height was provided for each of the five dams.

Construction Materials. On the basis of surface examination and auger holes, a knoll adjacent to the right abutment of the Forebay Dam was selected as the primary borrow area for that dam (Figure 191), and another knoll between Dams Nos. 3 and 4 was selected as the borrow area for Dams Nos. 1 through 4. Suitable materials from required excavations also were used in the dams. In the areas above the water table, natural moisture was well below the 15% optimum while, in a few areas where water was encountered, moisture ranged from 11.1 to 28.6%. A summary of material design parameters is shown in Table 21. Soils were tested for material parameters by the Department of Water Resources. Pervious materials were unavailable on the site and were imported.

TABLE 21. Material Design Parameters—Bethany Dams

Material	Specific Gravity	Unit Weight in Pounds Per Cubic Foot			Static Shear Strengths θ Angles in Degrees Cohesion in Tons Per Square Foot					
					Effective		Total		Construction	
		Dry	Moist	Saturated	θ	C	θ	C	θ	C
Foundation										
Forebay Dam	2.74	92	114	121	--	--	20	0.25	8	0.50
Adjacent Dams	2.72	105	122	130	32	0	15	0.6	5	0.25
Embankment										
Forebay Dam	2.73	115	132	136	--	--	23	0.25*	23	0.25
Adjacent Dams	2.73	105	123	131	32	0	18	0.6	22	1.3

* Value for static case; increase to 0.50 of a ton per square foot for seismic case.

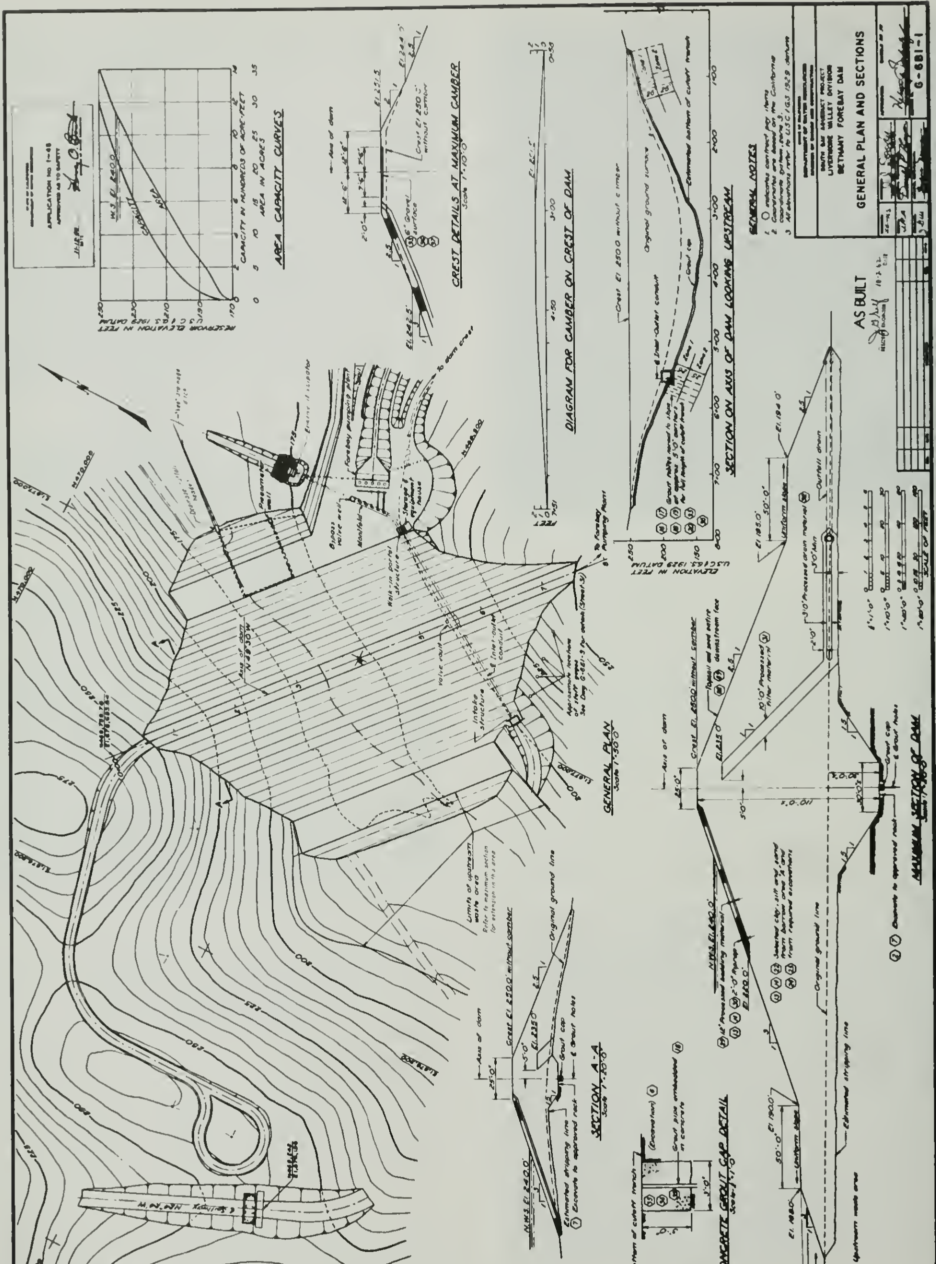

Figure 186. Bethany Forebay Dam—Plan, Profile, and Sections

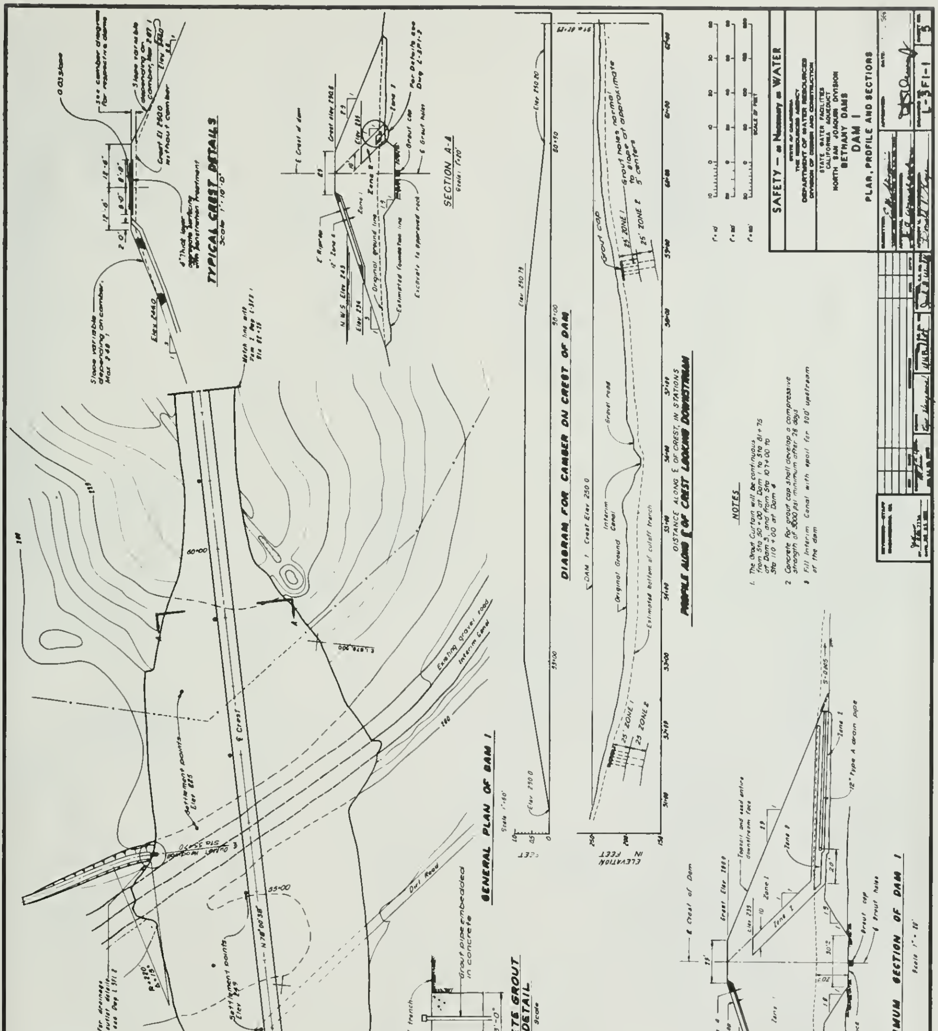

Figure 187. Dam No. 1—Plan, Profile, and Sections

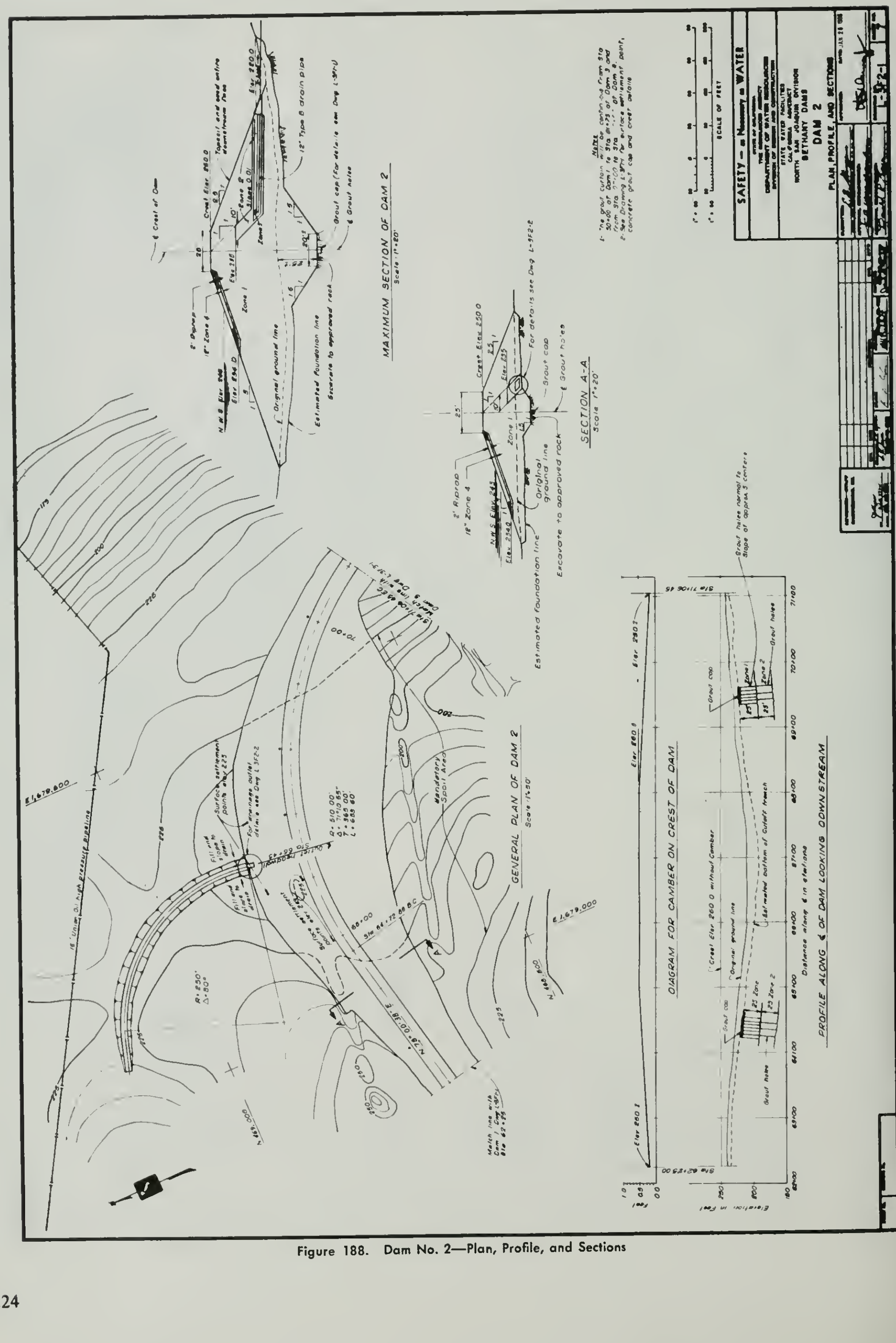

Figure 188. Dam No. 2—Plan, Profile, and Sections

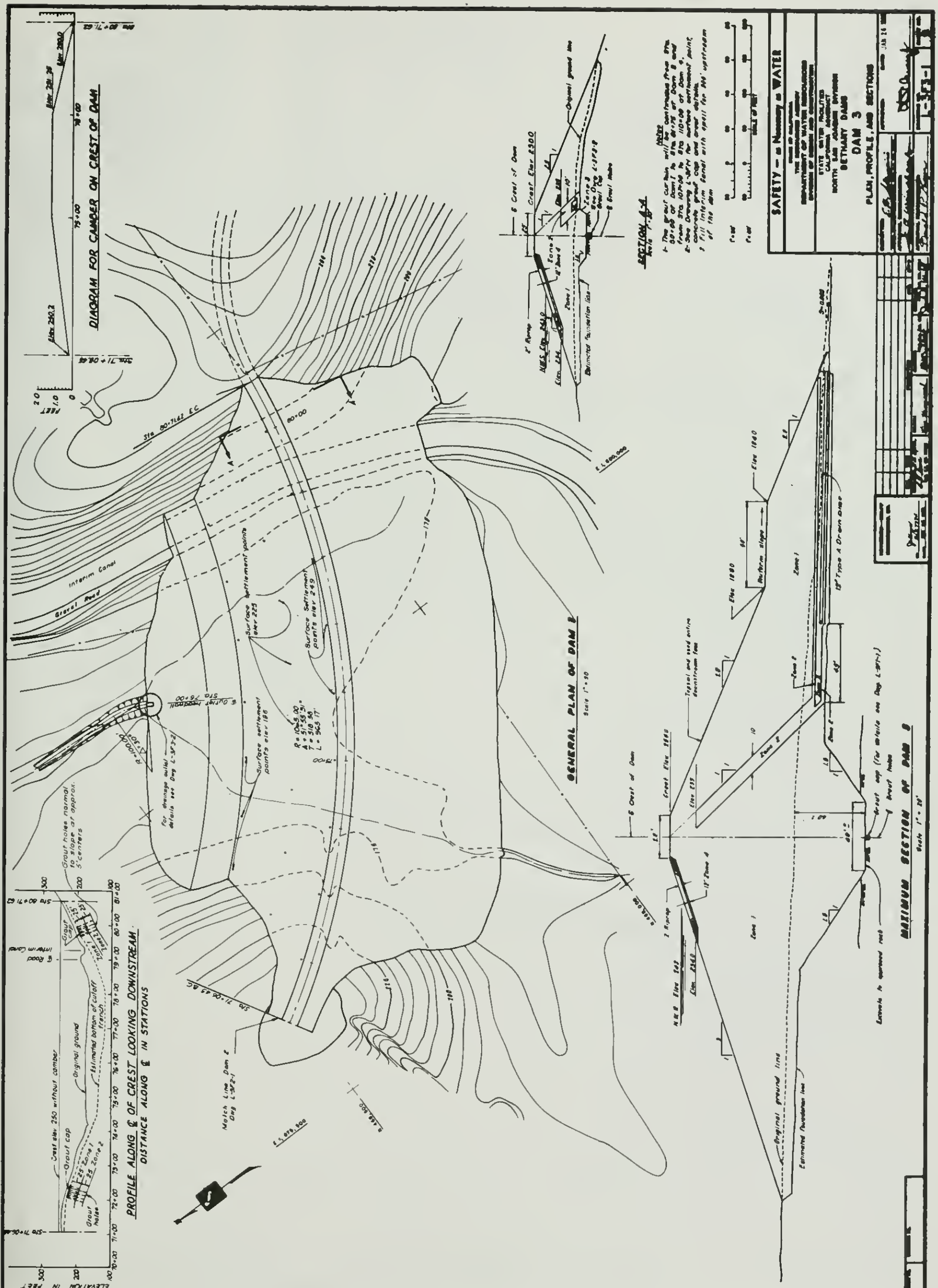

Figure 189. Dam No. 3—Plan, Profile, and Sections

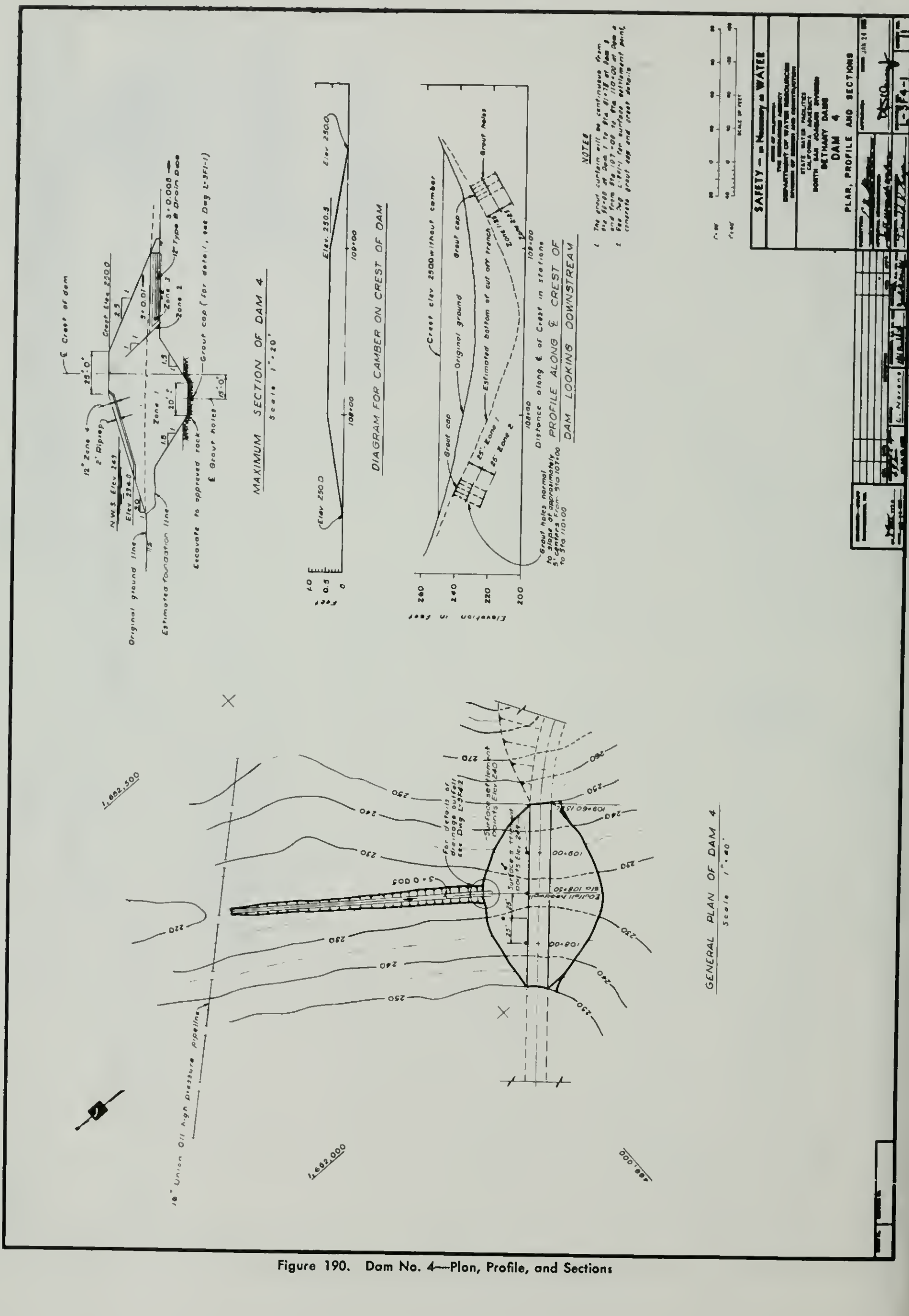

Figure 190. Dam No. 4—Plan, Profile, and Sections

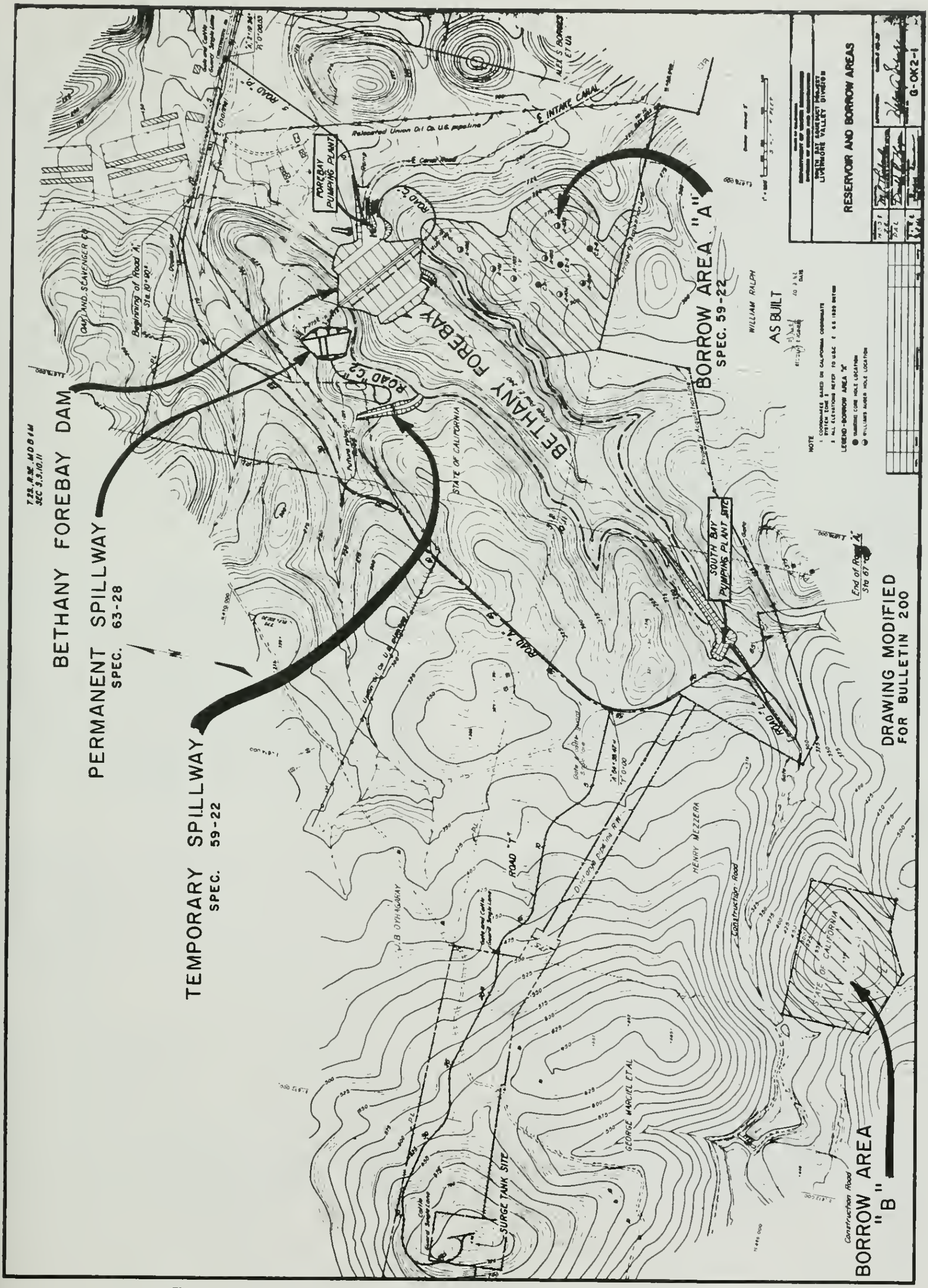

Figure 191. Location of Borrow Areas and Bethany Forebay Dam Site

Foundation. Abutments of all five dams are in sandstones and shales of the Panoche formation. Foundations in the channel sections are either Panoche formation or alluvial deposits consisting mainly of clay. Panoche formation shales and sandstones are strongly weathered, jointed, and fractured and part easily along bedding planes. Many joints, fractures, and bedding planes were coated heavily with iron and manganese oxides, and a few joints and seams are gypsum-filled. The alluvium is weak and, except at Dam No. 3, does not increase appreciably in strength to the depths penetrated.

All organic material was removed. It ranged in depth from less than 1 foot on the abutments to about 8 feet in the channel sections. In addition to this stripping, cutoff trenches were excavated through alluvium into stable rock of the Panoche formation. This rock was susceptible to air slaking when exposed. All foundation areas in this formation were excavated within 12 to 15 inches of final grades, and final excavation was made just before embankment placement. The foundation excavation plan for Dam No. 3 (Figure 192) is illustrative of all dam foundations.

A shear seam extends through the channel portion of the forebay dam foundation; however, there is no evidence of recent geological movement. The right abutment contains a joint system that appears to be the result of shearing stresses produced by regional folding.

Instrumentation. Four types of instrumentation were installed in the Forebay Dam (Figure 193): (1) surface settlement points, (2) hydraulic piezometers, (3) a cross-arm settlement unit for measuring differential settlement at different embankment levels, and (4) base-plate installations for measuring foundation settlement.

The piezometers are connected to an instrument panel in a well at the downstream toe of the Forebay Dam.

Surface settlement points were installed on all adjacent dams, and both Dams Nos. 1 and 3 are instrumented with two porous tube-type piezometers.

Outlet Works

Forebay Dam. In the first phase of operation, water was diverted by gravity from the Delta-Mendota Canal, about 2 miles away, through an interim canal to the right abutment toe of the Forebay Dam. It was then pumped into the Forebay through the outlet conduit.

In the second phase of operation, water is delivered by gravity to Bethany Reservoir through the California Aqueduct. The Aqueduct enters the Reservoir through a topographic saddle about 750 feet to the southwest of the left abutment. The original outlet works of the Forebay Dam will be used only to empty the Reservoir in case of emergency.

The outlet works is located on the right abutment. It consists of a low-level intake structure, with trashrack; a 60-inch-diameter, reinforced, cast-in-place, concrete conduit upstream of a valve chamber; and a 48-inch steel pipe installed in a 96-inch, horseshoe-shaped, concrete, access conduit between the valve chamber and the downstream portal structure. A 48-inch butterfly valve in the valve chamber provides shutoff for dewatering the downstream facilities.

Beyond the portal structure, five 24-inch lines branch from the 48-inch steel conduit and extend to the former interim forebay pumping plant. The 48-inch steel conduit and the manifold are encased in concrete. These five branch lines were capped when the interim pumping plant was removed from service.

A 24-inch, steel pipe blowoff extends from the end of the manifold to a reinforced-concrete energy dissipator in the stream channel. This blowoff is controlled by a 24-inch gate valve.

An investor-owned utility company supplies the power to a load center for valve chamber lighting and ventilating and for driving the valve operators.

The plan and profile of the outlet works are shown on Figure 194, and the rating curve is shown on Figure 195.

Outlet to South Bay Aqueduct. An unlined intake channel at the southwest margin of the Forebay supplies water to South Bay Pumping Plant.

Outlet to California Aqueduct. A connecting channel, designed to carry 10,000 cubic feet per second (cfs) at a velocity of 2 feet per second, supplies water to the expanded Bethany Reservoir (Figure 196). The outlet from Bethany Reservoir into the California Aqueduct consists of a check structure located at the southwest edge of the expanded reservoir.

Spillway

Since the Reservoir was constructed in two phases, two spillways also were constructed. The temporary forebay spillway (Figure 197) was constructed under the Bethany Forebay Dam contract at the location where the California Aqueduct later entered the Reservoir. This spillway was replaced by a permanent spillway constructed under the canal embankment Specification No. 63-28.

The permanent spillway consists of a straight, unlined, trapezoidal, earth channel located in a saddle about 200 feet beyond the left abutment of the Forebay Dam. The control structure is a reinforced-concrete broad-crested weir located midway along the spillway alignment. The spillway channel is unlined because the need for its operation is extremely unlikely. The maximum probable flood (peak inflow 6,410 cfs) can be accommodated fully by the outlet facilities to the California Aqueduct (capacity 10,000 cfs).

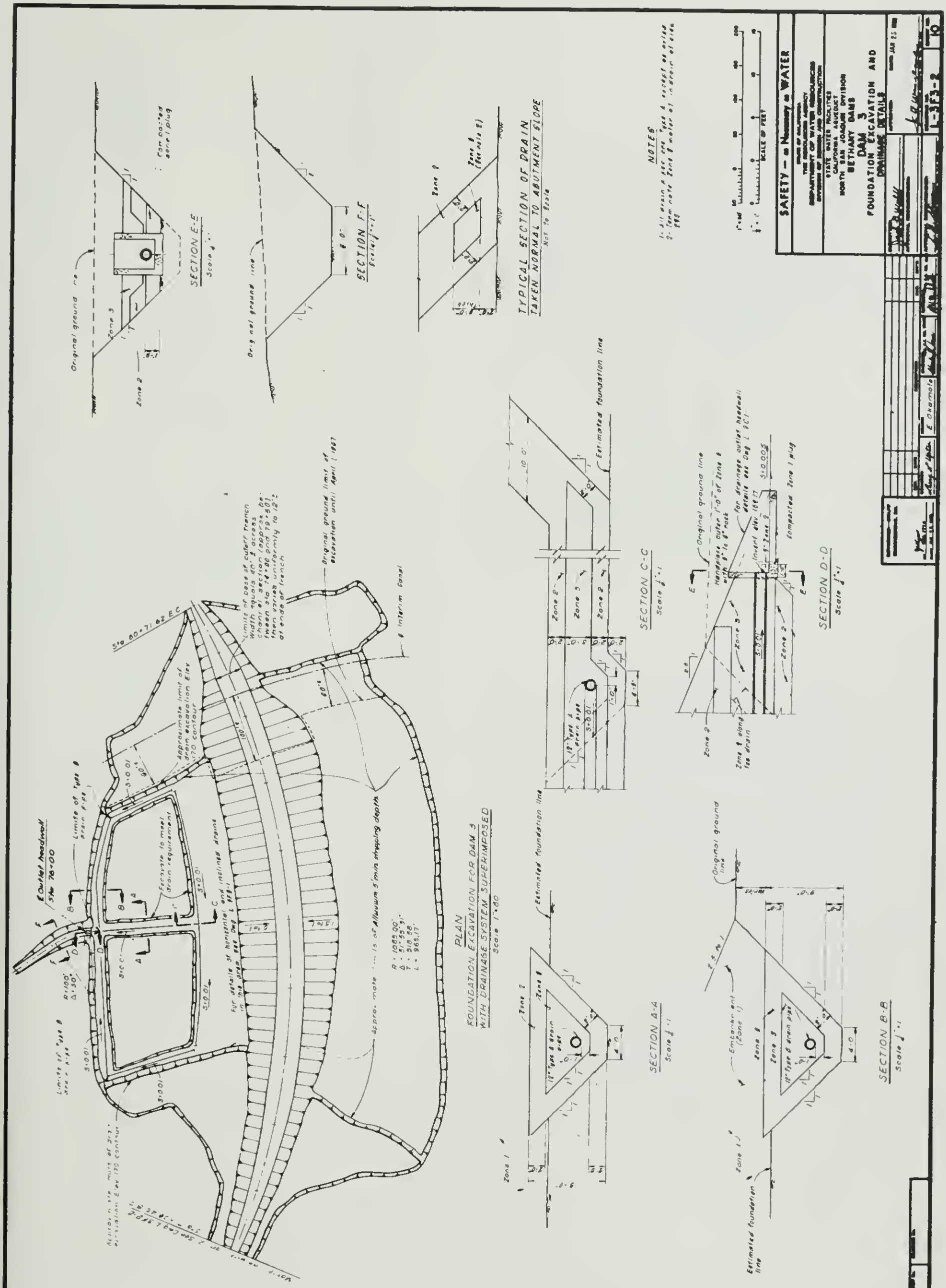

Figure 192. Foundation Excavation and Drainage Details—Dam No. 3

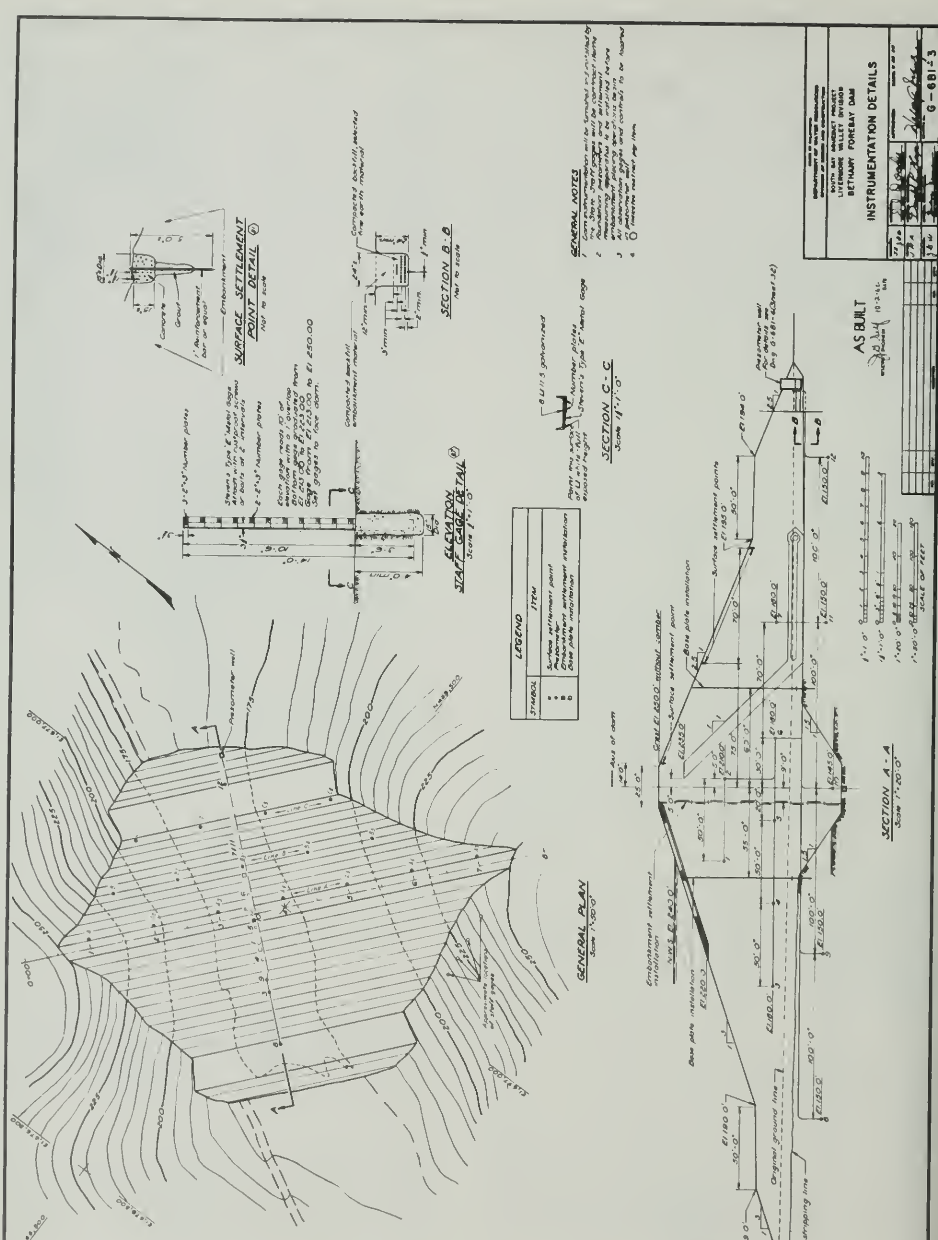

Figure 193. Location of Forebay Dam Instrumentation

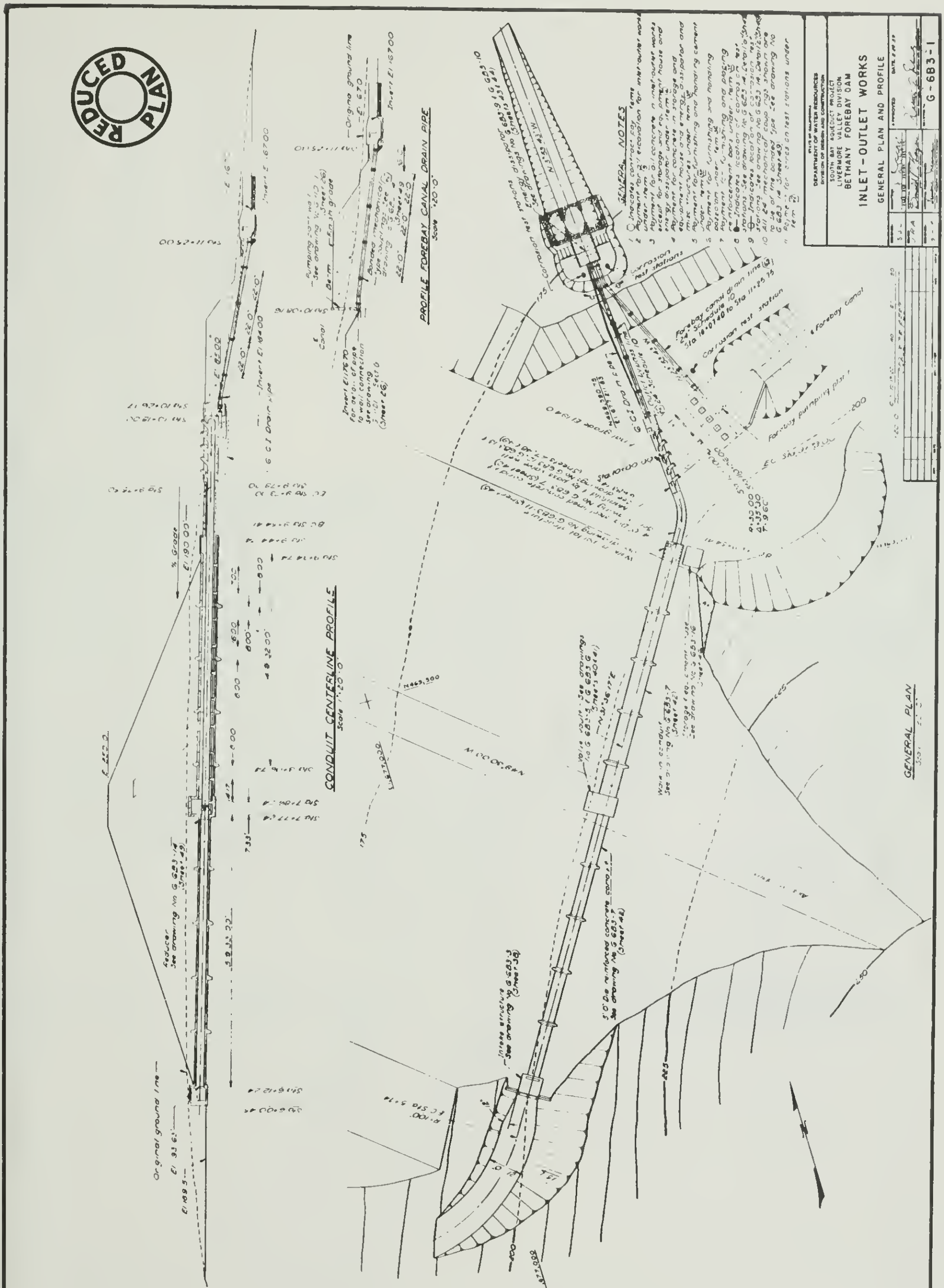

Figure 194. Outlet Works—Plan and Profile

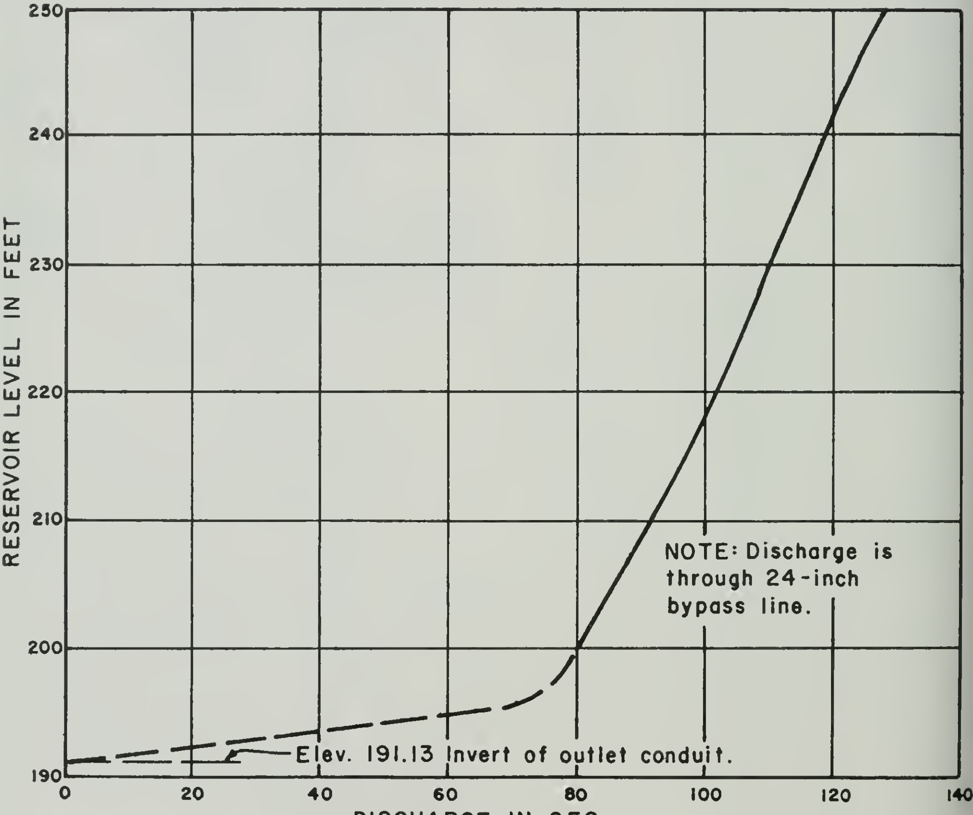

Figure 195. Outlet Works Rating Curve

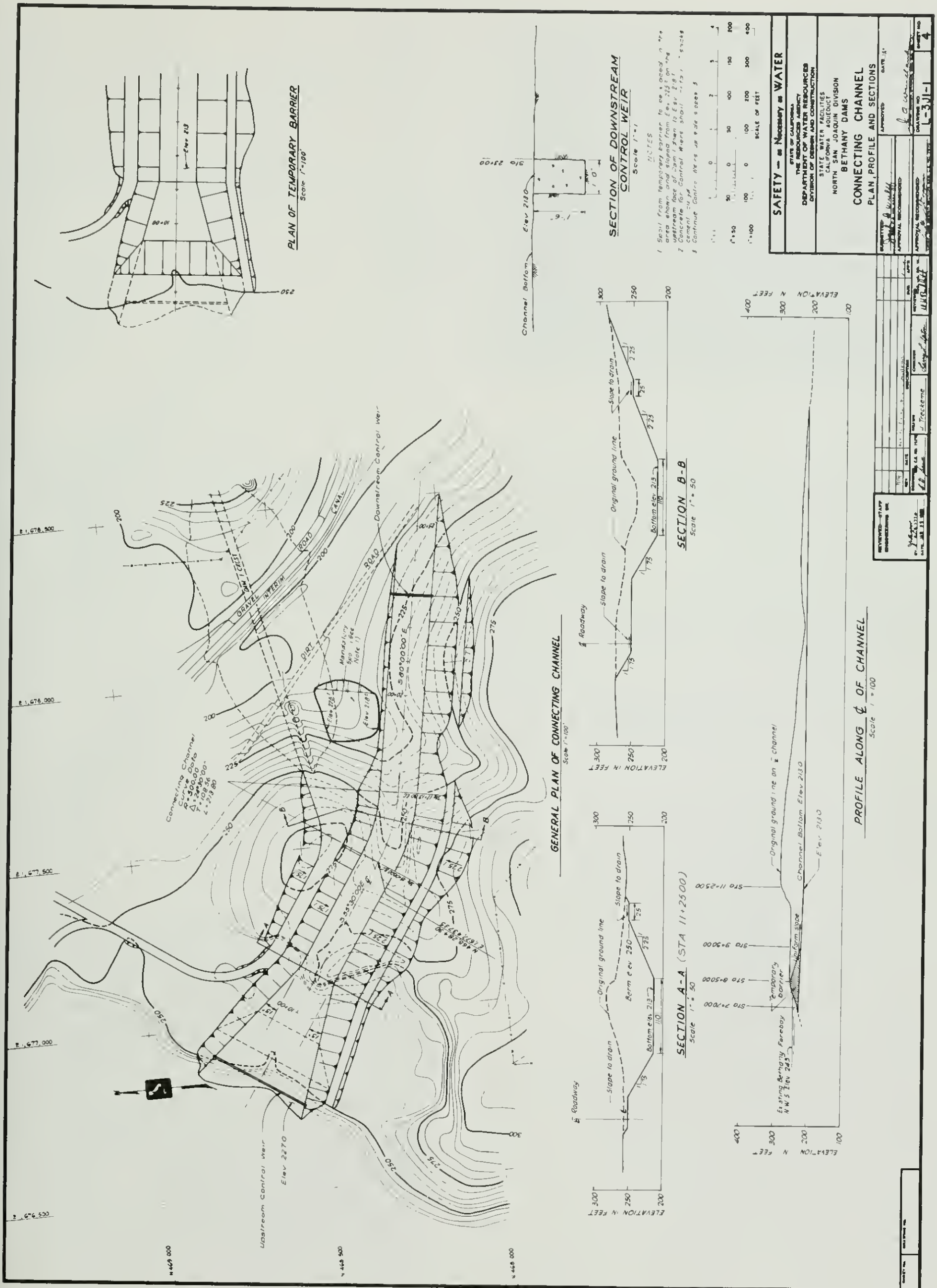

Figure 196. Connecting Channel—Plan, Profile, and Sections

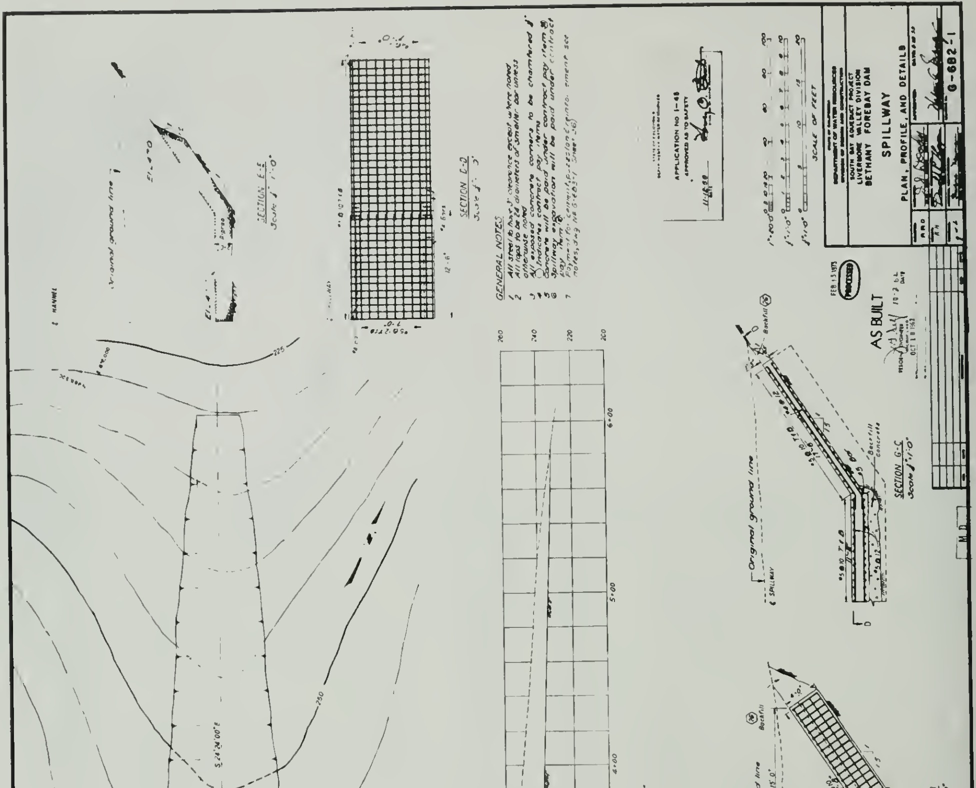

Figure 197. Temporary Spillway

Construction

Contract Administration

General information about the major contracts for the construction of Bethany Forebay Dam and Bethany Dams is shown in Table 22.

TABLE 22. Major Contracts—Bethany Forebay Dam and Bethany Dams

	Bethany Forebay Dam	Bethany Dams
Specification	59-22	66-17
Low bid amount	$877,870	$1,716,650
Final contract cost	$876,339	$2,057,838
Total cost-change orders	$26,456	$87,448
Starting date	11/25/59	5/10/66
Completion date	3/9/61	12/13/67
Prime contractor	O.K. Mittry & Sons	Rivers Construction Co. Inc.

Diversion and Care of Stream

Streamflow through the Forebay Dam site (Specification No. 59-22) was controlled during construction by a small earth embankment and retention pond approximately 1,300 feet upstream from the Dam site.

A drainage ditch was provided around the South Bay Pumping Plant site during excavation of the intake channels and later was converted for permanent hillside drainage and erosion prevention of the channel cut slopes.

Diversion facilities were unnecessary for the four dams constructed under Specification No. 66-17.

Foundation

Material from all dam foundation excavations suitable for use as compacted embankment, riprap, or topsoil was stockpiled for later use. Unsuitable material was wasted in the reservoir area. A total of 64,000 cubic yards was excavated for the foundation of the Forebay Dam and 333,000 cubic yards for the foundation of Dams Nos. 1, 2, 3, and 4 (Figure 198).

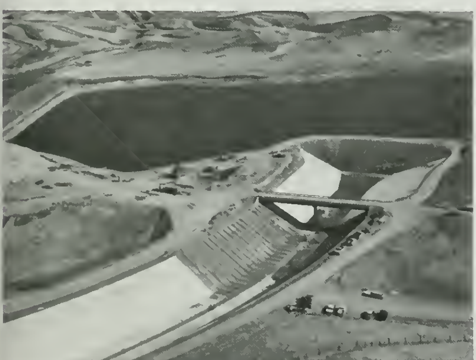

Figure 198. Bethany Forebay and Excavation for Adjacent Dams

Figure 199. Foundation Grouting—Bethany Forebay Dam

Seepage water accumulated in the middle 100 feet of the cutoff trench of the Forebay Dam. This was controlled by embedding a slotted 2-inch-diameter pipe in a gravel-filled trench and pumping from an 8-inch riser set at the low point. One-inch vent pipes were installed at the one-third points of the 2-inch pipe on each side of the riser. After the embankment was placed 10 feet above the foundation, sand-cement grout was forced into the drain system through the 8-inch riser to force out the remaining water and seal the system.

Water was encountered in the vicinity of the interim canal crossing under Dam No. 1 and was controlled by pumping.

Grouting

A concrete grout cap, 3 feet wide by 3 feet deep, was placed in the foundation of each cutoff trench along the centerline. The trench for the grout cap was excavated mainly with a rotary bucket wheel trencher which worked well in the soft shales and sandstones and usually left a smooth uniform cut. Because the shales tended to air slake severely, the concrete grout cap was placed as soon as possible after trench excavation. Grout nipples were set on 5-foot centers in the grout cap.

The split-spacing method was used for drilling and grouting the curtain along the entire length of Bethany Dam (Figure 199) and the adjacent dams. Primarily, holes were drilled and grouted at 40-foot spacing, and secondary holes were set midway between the primary holes. The spacing similarly was "split" twice more, so that the final spacing of the grout curtain holes was 5 feet except on the higher portions of the forebay dam abutments where the spacing was 10 feet. The specification provided for the holes on 10-foot spacing to be 50 feet deep and the intermediate holes to be 25 feet deep.

Both stage and packer grout methods were used.

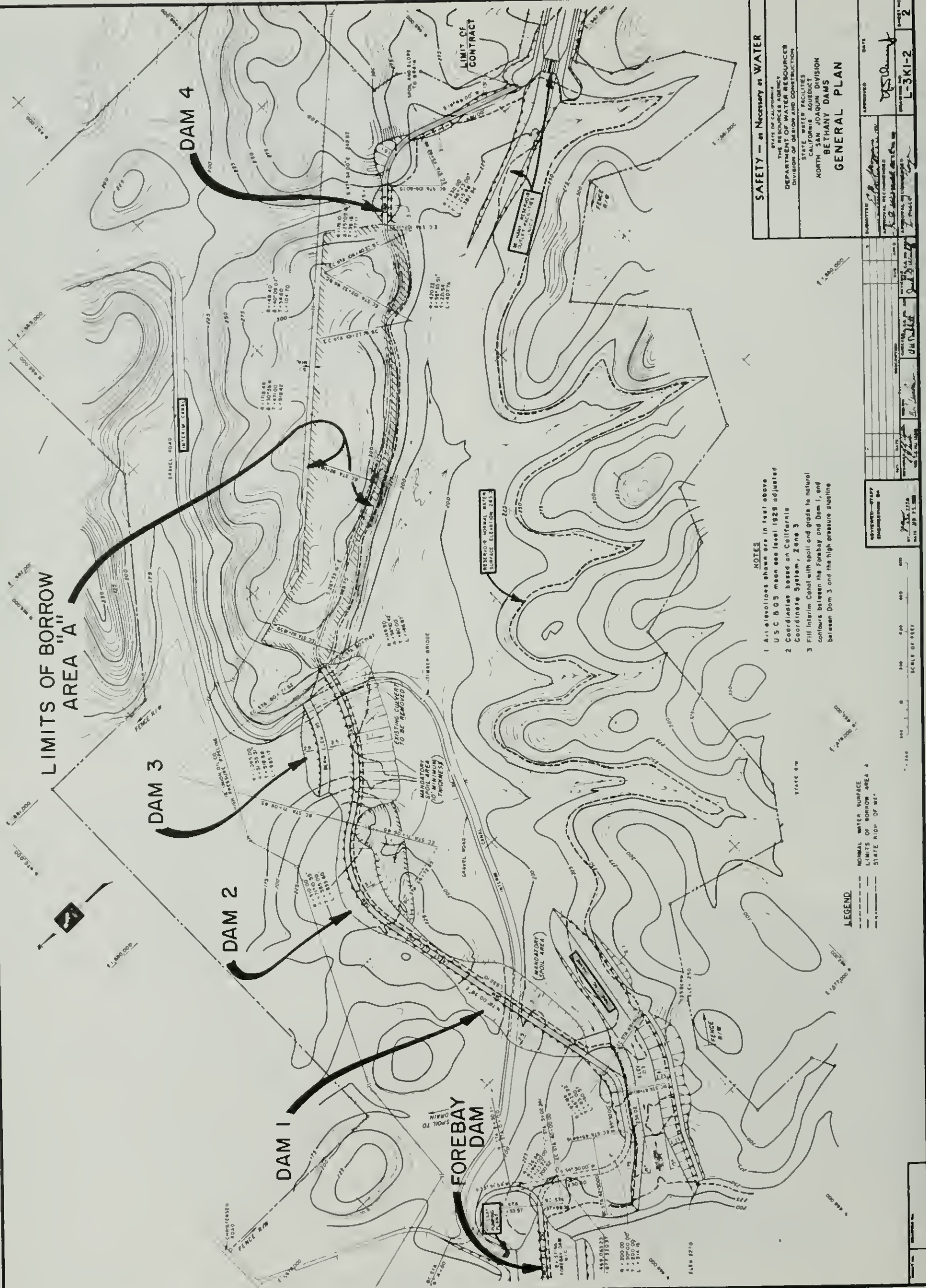

Figure 200. Location of Borrow Areas and Adjacent Dams

During stage grouting, holes were drilled to partial depth, grouted, and then cleaned out by flushing with water before the grout in the hole had set. After the grout in the surrounding rock had set, the hole was drilled an additional interval, stage-grouted, and flushed out again. This process was repeated until the required depth was reached. During packer grouting, the hole was drilled to full depth, and then packers were set at depth in the hole to isolate selected intervals for grouting.

Handling of Borrow Materials

The borrow materials for all dams were obtained from designated borrow areas, foundation, spillway, and connecting channel excavations (Figures 191 and 200). Where possible, the material was placed directly in embankments being constructed; surplus material was stockpiled for future use. It was unnecessary to go to Borrow Area B under the forebay dam contract because all material was obtained from other areas of surplus. A total of 285,079 cubic yards of excavation in borrow areas plus suitable material from the foundation and spillway excavation yielded 289,712 cubic yards of compacted embankment for the Forebay Dam. A total of 1,037,338 cubic yards of compacted Zone 1 embankment from designated channel excavations and foundation excavations was placed in the four adjacent dams.

Impervious borrow areas (Zone 1 material) were moisture-conditioned by sprinkling to near-optimum moisture content prior to excavation. At times during construction of the Forebay Dam, the premoistening operations were poorly executed, delaying and complicating embankment placement and compaction.

The processed filter and drain materials (Zones 2 and 3) for all dams were obtained from commercial aggregate sources in the Tracy area.

There was enough suitable rock in borrow areas and other areas of required excavation to supply all of the riprap and the riprap bedding (Zone 4). Rock was stockpiled as excavation progressed, then hauled to the dams by 5-cubic-yard, rubber-tired, front-end loaders.

Embankment Construction

Zone 1 embankment, the bulk of all the dams, was placed by controlling the distribution and gradation of materials throughout the fill to avoid lenses, pockets, streaks, and layers of material differing substantially from surrounding fill. Embankment materials were spread in successive horizontal layers, not exceeding 6 inches in thickness after compaction. As hauling and spreading of the material had a drying effect, an adjustment to the moisture content was made with a water truck, followed immediately by disking for mixing just prior to compaction.

The contractor was required to keep the compacted materials adjacent to the abutments 2 to 3 feet higher than the rest of the fill to provide a good seal between the abutment and the fill.

Good compaction results were obtained when moisture control, rolling of lifts, and rock removal were done properly. Compaction improved as the work progressed. The low densities found early in construction of the Forebay Dam were not considered critical, and it was reasoned that the overburden weight of the upper material would sufficiently consolidate the low-density areas. Twelve years after the completion of the Forebay Dam, settlement gauge readings showed a maximum settlement of 5 inches, well within the anticipated amount. The Forebay Dam during final construction stages is shown on Figure 201.

Figure 201. Bethony Foreboy Dom Construction

The overall average in-place dry density for Bethany Dams Zone 1 was 107.3 pounds per cubic foot (pcf), ranging from 95.2 to 120.4 pcf. The average relative compaction was 98%, ranging from 91 to 106%.

Zone 2 (filter) material was placed in layers not more than 12 inches thick after compaction, and Zone 3 (drain) material was placed in layers not more than 15 inches thick after compaction.

In Dam No. 2, obtaining the proper density of the sloping drain was difficult. A maximum of 4% moisture before compaction was specified. This was increased to 8% before obtaining the required 70% relative density. The increased moisture content was supplied throughout the construction of all four of the adjacent dams. The moisture content for the sloping drain zone was not specified in the forebay dam contract, and the zone was placed without incident. As the Forebay Dam became higher, the drain became longer and placement lagged. Until additional trucks were used, the elevation of the drain zone fell below the other zones.

The average relative density for Zone 2 material in the Bethany Dams was 71%. Forty-seven (63%) of the 74 tests taken were above the 70% relative density specified. Zone 3 material was placed with little difficulty.

Zone 4 (riprap bedding) material was dumped and spread on prepared surfaces in layers not exceeding 12 inches after compaction.

Spillway

Excavated material from the temporary forebay spillway, where suitable, was placed in the embankment. The remainder was wasted in the reservoir area. This spillway structure site was overexcavated and backfilled to grade with concrete.

After the permanent spillway for the extended reservoir was built, the Aqueduct was excavated on the alignment of the temporary forebay spillway. Suitable material from this excavation was used in the aqueduct embankment, while the remainder was spoiled in designated areas.

Outlet Works

Excavation for the outlet works of the Forebay Dam followed excavation of the dam foundation. Outside the cutoff, alluvial material was removed to sound rock. From the intake structure to the walk-in portal, shotcrete protective coating was applied to prevent slaking; then the trench was backfilled to subgrade with concrete. Downstream of the walk-in structure, the trench was backfilled to subgrade with compacted impervious embankment.

The outlet works conduit, including 2-foot by 6-inch cutoff collars at the construction joints, was monolithically cast in 32- and 33-foot sections. Six-inch, "dumbell", rubber, water stops and ½ inch of expansion-joint filler were placed in the construction joints. A 48-inch-inside-diameter steel pipe was placed on concrete saddles spaced at 16 feet inside the 8-foot-diameter, walk-in, horseshoe conduit.

Concrete Production

Concrete was supplied by a commercial ready-mix plant and was mixed and transported to the placement site in transit mix trucks. The manually operated, 3-cubic-yard, batch plant was located 2 miles north of the City of Brentwood on State Highway 4. Transit mix trucks varied in capacity from 5 to 9 cubic yards. The haul distance to the site was about 20 miles, and the average haul time was 45 minutes, with an average unloading time of 15 minutes.

Mechanical and Electrical Installations

A ventilating fan, operated by a ⅛-horsepower motor with an output of 406 cubic feet per minute, was installed in the storage and equipment house. This ventilating equipment was connected to the portal structure by an 8-inch-diameter cast-iron pipe. Air was blown into a 9-inch-diameter aluminum duct through the walk-in conduit and discharged at the valve vault.

A 50-amp load center also provides power for the lighting system and the 24-inch valve operator.

Reservoir Clearing

The reservoir area below elevation 240 feet was cleared of all trees, brush, rubbish, fences, and a timber bridge. Trees were cut off within 1 foot of the ground.

Placing Topsoil and Seeding

Topsoil placed on the downstream slopes of all dams was selected from the stockpiles that contained the most fertile loam. Ammonium sulfate fertilizer was spread evenly at the rate of 400 pounds per acre, and seed was sown at the rate of 60 pounds per acre. Immediately following seeding, the seeded areas were covered uniformly with layers of straw and anchored by rolling the entire area with a punching-type roller.

Embankment Test Installation

The instrumentation described earlier in this chapter was observed continuously during construction and now is observed on a scheduled basis.

BIBLIOGRAPHY

California Department of Water Resources, Bulletin No. 117-12, "Bethany Reservoir Recreation Development Plan", December 1970.

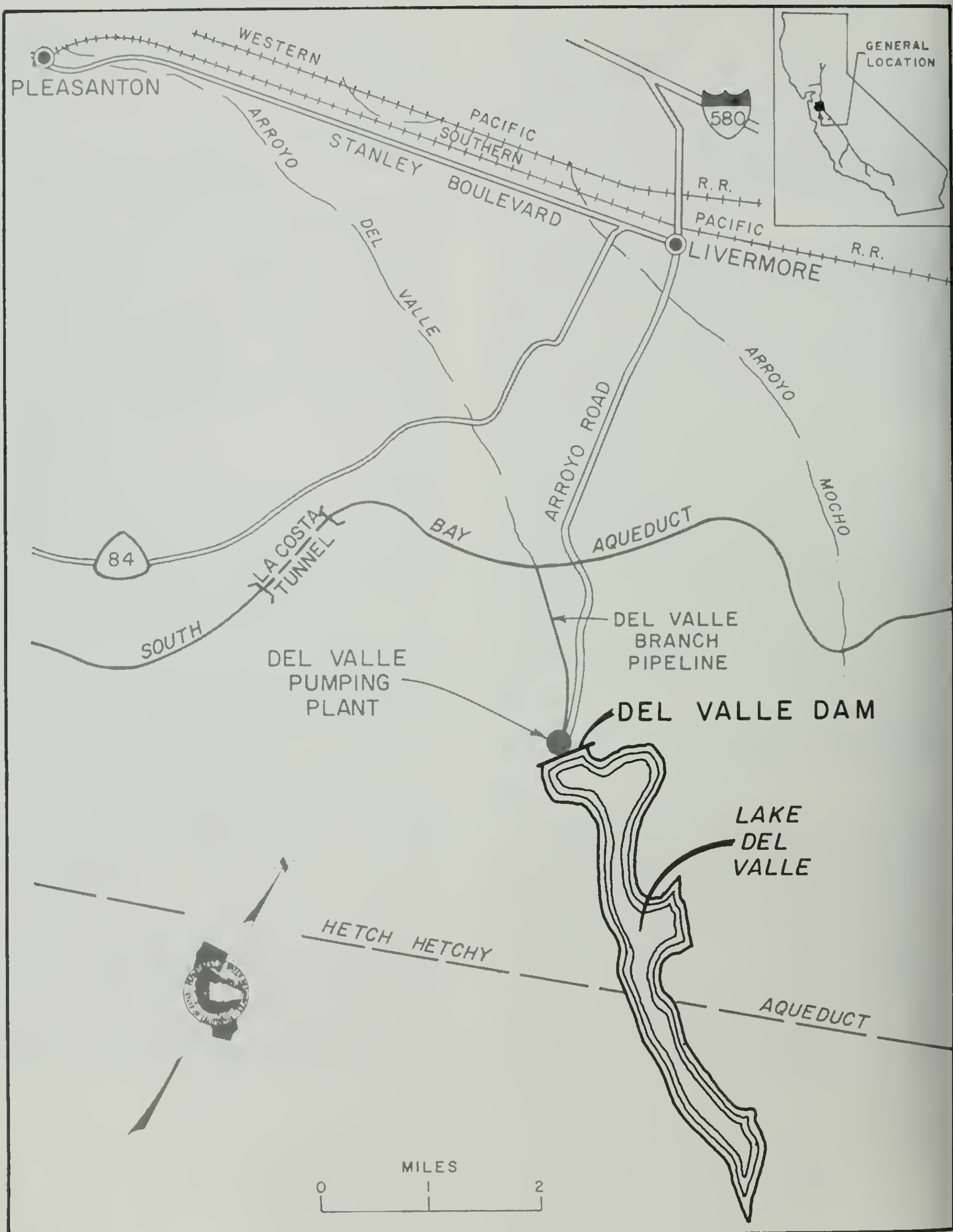

Figure 202. Location Map—Del Valle Dam and Lake Del Valle

CHAPTER X. DEL VALLE DAM AND LAKE DEL VALLE

General

Description and Location

Del Valle Dam is a 235-foot-high zoned embank-nent containing 4,150,000 cubic yards of material. The spillway control structure is an 84-foot-diameter, ungated, glory-hole intake located 1,600 feet southeast of the Dam beyond the right abutment. The spillway ntake discharges into a 30-foot-diameter vertical shaft oined by an elbow transition to a 28-foot-diameter, ,893-foot-long, nearly horizontal tunnel. An 18-foot-liameter, flood control, outlet works tunnel dis-harges into the spillway tunnel at the elbow. High-ressure slide gates located at the elbow control the low through the flood control outlet. A smaller con-ervation outlet is located between the spillway and Dam. It consists of an inclined, multilevel, reinforced-concrete, intake structure; a 78-inch-diameter pressure tunnel; a valve vault at the axis of the Dam; and a 60-inch-diameter steel pipe inside a 9-foot - 6-inch, horseshoe-shaped, walk-in tunnel beyond the valve vault.

Lake Del Valle has a capacity of 77,106 acre-feet, a surface area of 1,060 acres, and a 16-mile shoreline.

The Dam and Lake are located in Arroyo Del Valle, just south of Livermore Valley, approximately 4 miles from the City of Livermore in Alameda County. Arroyo Road, a paved city street and county road, affords access from Livermore to the Dam site. The nearest major roads are U. S. Highway 50 (also Interstate 580) and State Highway 84 (Figures 202 and 203).

Figure 203. Aerial View—Del Valle Dam and Lake Del Valle

A statistical summary of Del Valle Dam and Lake Del Valle is shown in Table 23, and the area-capacity curves are shown on Figure 204.

The 320-foot-deep Del Valle Shaft, which served as access to the Hetch Hetchy Aqueduct tunnel, was plugged with a combination of concrete plugs and fill material under Specification No. 67-55. The Shaft and its access were submerged by Lake Del Valle. The Shaft had been used for access during construction of the tunnel and was considered for the same purpose for construction of a future parallel tunnel. With the existence of Lake Del Valle, this future construction scheme was abandoned.

Purpose

The purposes of the project are to provide regulatory storage for the South Bay Aqueduct (30,000 acre-feet), flood control for Alameda Creek (38,000-acre-foot reservation), conservation of storm runoff, recreation, and fish and wildlife enhancement.

The U. S. Army Corps of Engineers has made payments of approximately $4,900,000 toward the construction and the capitalized operation and maintenance costs of the flood control features of the Dam and reservoir. Possible additional appropriations are pending.

Local storm runoff is impounded by the Dam and later released at the convenience of downstream water users. The Department of Water Resources has received payment for these conservation benefits from the water agencies involved.

Chronology

The State started preliminary surveys at the site in 1957 and final design in 1964. Dam construction was begun in March 1966 and completed in 1968.

Regional Geology and Seismicity

The Dam and reservoir are located in the Diablo Mountain Range, a part of the Coast Ranges. Geologic formations in the area consist mainly of sedimentary rocks folded into northwest-trending anticlines and synclines. The oldest rocks in the area, Jurassic graywackies, cherts, shales, and serpentines of the Franciscan group, crop out in an extensive area south of the reservoir. Younger Cretaceous sandstones and shales of the Panoche formation are present at the Dam site and extend for several miles west where the Panoche formation is in fault contact with Franciscan rock along the Williams fault. Above the right abutment of the Dam, soft sandstones and siltstones of the Miocene

TABLE 23. Statistical Summary of Del Valle Dam and Lake Del Valle

DEL VALLE DAM	
Type: Zoned earthfill	
Crest elevation	773 feet
Crest width	25 feet
Crest length	880 feet
Streambed elevation at dam axis	550 feet
Lowest foundation elevation	538 feet
Structural height above foundation	235 feet
Embankment volume	4,150,000 cubic yards
Freeboard above spillway crest	28 feet
Freeboard, maximum operating surface	69.8 feet
Freeboard, maximum probable flood	8.4 feet

LAKE DEL VALLE	
Storage at spillway crest elevation	77,106 acre-feet
Maximum conservation storage	40,000 acre-feet
Storage at flood control pool	39,000 acre-feet
Minimum conservation storage	9,863 acre-feet
Dead pool storage	3,317 acre-feet
Maximum conservation surface elevation	703.2 feet
Surface elevation of flood control pool	702 feet
Minimum conservation surface elevation	638 feet
Dead pool surface elevation	609 feet
Shoreline, spillway crest elevation	16 miles
Surface area, spillway crest elevation	1,060 acres
Surface area, maximum conservation elevation	708 acres
Surface area, minimum conservation elevation	285 acres

SPILLWAY	
Type: Glory hole with concrete-lined tunnel and stilling basin	
Crest elevation	745 feet
Crest length	264 feet
Crest diameter	84 feet
Tunnel diameter	28 feet
Maximum probable flood inflow	64,000 cubic feet per second
Peak routed outflow	44,200 cubic feet per second
Maximum surface elevation	764.6 feet
Standard project flood inflow	23,500 cubic feet per second
Routed outflow	7,500 cubic feet per second
Water surface elevation	749.7 feet

INLET-OUTLET	
Del Valle Pumping Plant	
Capacity, in or out	120 cubic feet per second

OUTLET WORKS	
Conservation: Lined tunnel under right abutment, valve chamber at midpoint—upstream of valve chamber, 78-inch-diameter pressure section—downstream, 60-inch steel conduit in a 144-inch concrete horseshoe tunnel—intake, five-level inclined structure with 42-inch shutoff butterfly valves—downstream control, 42-inch fixed-cone dispersion valve—discharge into spillway stilling basin—66-inch butterfly guard valve in valve chamber	
Capacity	400 cubic feet per second
Flood control: 18-foot-diameter lined tunnel under right abutment—intake, bell-mouth entrance—transition to 28-foot-diameter spillway tunnel—control in transition by two pairs of 6-foot-wide by 7-foot - 6-inch-high, high-pressure, slide gates in tandem	
Capacity with surface elevation at spillway crest	7,000 cubic feet per second

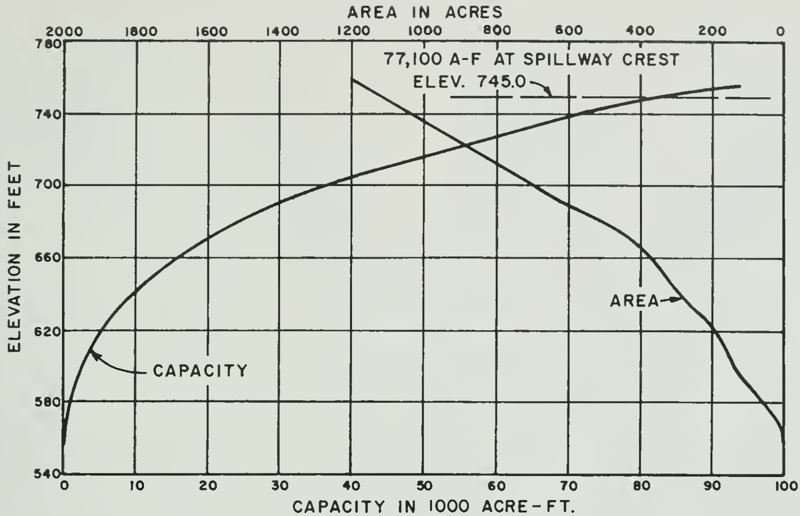

Figure 204. Area-Capacity Curves

Cierbo formation rest unconformably upon the Panoche sandstones and shales. Overlying the Cierbo formation are the Plio-Pleistocene Livermore gravels which extend for about 5 miles northeast of the Dam.

Del Valle Dam is near seismically active regions in California. There are three major active fault zones within 30 miles of the Dam: Calaveras (8 miles), Hayward (12 miles), and San Andreas (30 miles).

Design

Dam

Description. The Dam is a rolled earthfill strucure consisting of a central impervious core, granular hells, and random stability zones upstream and lownstream. The plan of Del Valle Dam is shown on igure 205.

Internal embankment drainage is provided by an nclined drain downstream of the impervious core and lanket drains on the downstream abutments which onnect to a drain in the stream channel. Protective ilters are provided between the core and downsteam nclined drain, between the core and upstream shell, nd between the channel and abutment foundations nd drains (Figure 206).

Stability Analysis. Selection of the preliminary ection was based on geologic investigations, availabilty and characteristics of materials, known characterstics of foundation materials, and seismic coniderations. After preliminary design was completed, nore extensive drilling and testing for final design evealed poorer foundation conditions and strengths han had been anticipated during preliminary designs. s a result, the embankment slopes were flattened and stability sections of random fill were added upstream and downstream.

Stability of embankment sections was analyzed by the Swedish Slip Circle and sliding wedge methods of analysis. Adequate factors of safety were calculated for the final embankment sections under all cases of loading. These cases of loading included full reservoir and other critical reservoir levels along with earthquake loads. Earthquake loading involved a horizontal force equal to the weight of the soil mass being analyzed, multiplied by an earthquake acceleration factor.

Because of the close proximity of major faults and less than desirable foundation conditions, the Department's Earth Dams Consulting Board recommended (1) the use of an earthquake acceleration of 0.15g, (2) conservative embankment and cut slopes, (3) wide impervious embankment sections, and (4) freeboard design to include consideration of landslides and seiches.

Settlement. Measurements of settlement in prototype dams of impervious material indicated that most of the settlement occurs during fill placement. A nominal camber of 1% of the fill height was provided to allow for postconstruction settlement.

Foundation. The Dam is founded upon sandstones and shales of the Panoche formation. Geologic structure is complex owing to faulting and folding. Most of the shears or faults are small but, in the right side of the channel, there was a shear large enough that soft sheared material was required to be removed to prevent seepage under the core along the shear zone. Beds in the left abutment either are overturned

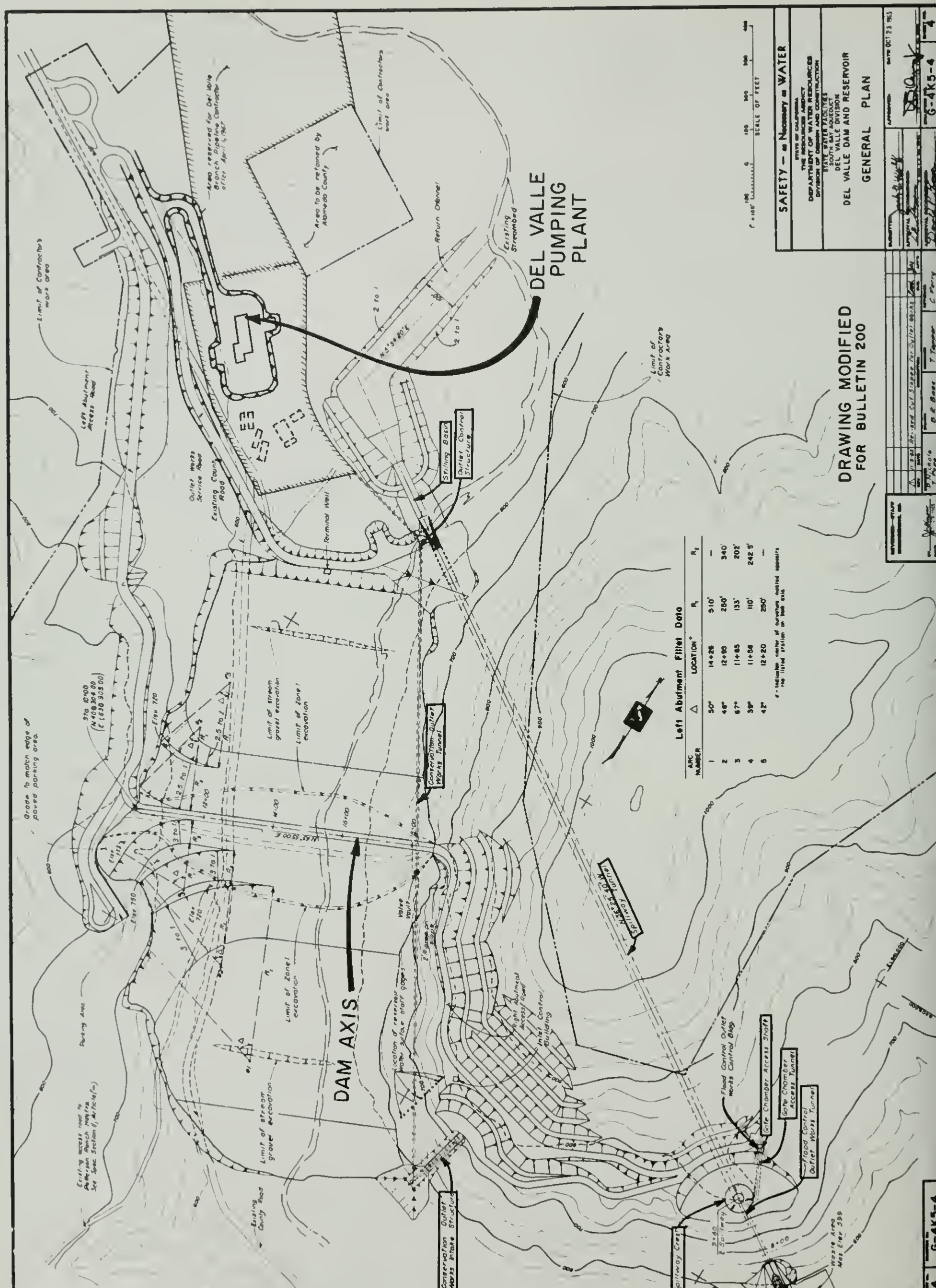

Figure 205. General Plan of Dam

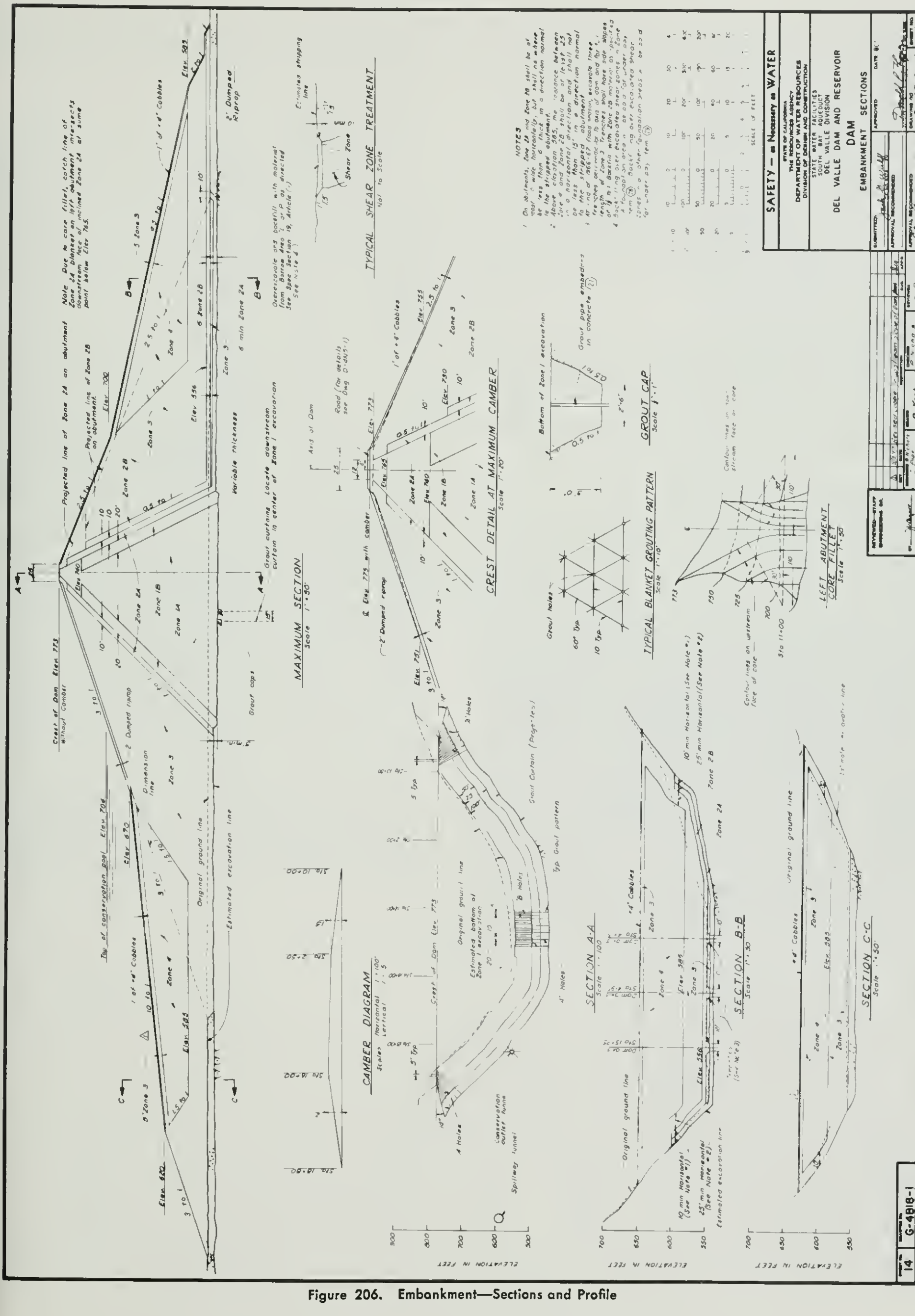

Figure 206. Embankment—Sections and Profile

or steeply dipping into the abutment. Beds in the right abutment dip from 34 to 63 degress into the abutment. Sandstones become increasingly abundant in the upper right abutment, and many of the sandstone beds have open cracks. The lower flank of both abutments had considerable volumes of terrace material and slopewash that were unsuitable for foundation.

Foundation excavation involved removal of weathered or otherwise weak materials from the abutments, removal of the stream gravels and terrace materials, and clean-out of soft materials from the shear zone described above. The excavation under the core was specified to be about 3 feet deeper than that for the remaining foundation so as to expose less permeable material.

A grout curtain, consisting of two rows of holes, was provided at the centerline of the core contact. The grout holes were spaced at 5-foot intervals and were up to 100 feet in depth. Blanket grouting of shallow holes on a 10-foot grid was required in areas where weak permeable material was identified beneath the core.

Construction Materials. Construction materials for the Dam were obtained from various sources. Material design parameters for these materials, as determined by soils testing, are presented in Table 24.

Zone 1A, the impervious core, consists of clayey soils of the Livermore formation obtained from a location on the north arroyo slope upstream from the Dam. Zone 1B flanks Zone 1A on each side and was obtained from an alluvial terrace located between the aforementioned borrow area and the stream channel.

Zones 2A, 2B, and 3 were composed of streambed gravels excavated as far as 5 miles upstream of the Dam. Zone 2A is the processed transition between the core and drain, Zone 2B is the processed gravel drain, and Zone 3 is pit-run gravels forming the embankment shells. Zone 4 is a random material from mandatory excavations.

Instrumentation. Instrumentation at Del Valle Dam consists of 31 piezometers, 8 Carlson pore-pressure cells, 9 porous-tube piezometers, 37 surface monuments, and 8 slope indicators. This instrumentation was designed to monitor pore pressures, settlements, and horizontal movements (Figure 207).

Conservation Outlet Works

The conservation outlet works (Figure 208) is used to release natural streamflows in Arroyo Del Valle and convey regulated inflow and outflow of South Bay Aqueduct water between Del Valle Pumping Plant and the reservoir. During construction of the Dam, the conservation outlet tunnel was used to divert natural streamflows around the Dam. After construction, the diversion intake was plugged with concrete from its inlet to the elbow at the base of the inclined intake.

The conservation outlet works consists of a multilevel, inclined, reinforced-concrete, intake structure on the right abutment; an upstream reach of 78-inch-diameter, concrete-lined, pressure tunnel; a valve vault near the axis of the Dam; and a 60-inch-diameter steel pipe inside a 114-inch-diameter, horseshoe-shaped, walk-in tunnel extending from the valve vault to a control structure near the left wall of the spillway stilling basin.

The inclined intake structure (Figure 209) consists of a 7-foot-square reinforced-concrete conduit from elevation 710 feet to elevation 600 feet; a transition to a 78-inch-diameter circular conduit; and an elbow and thrust block near streambed level. To provide for selective level releases for water quality control, 42-inch-diameter butterfly valves and trashracks were installed at elevations 690, 670, 650, 630, and 620 feet.

The tunnel transitions from a 78-inch diameter to a 60-inch diameter with a steel liner immediately upstream of the valve chamber.

The valve vault contains a 60-inch butterfly valve for emergency shutoff, two air-vacuum valves, and a mechanical-type coupling for ease in assembly of the valves and steel pipeline. Dimensions of the valve vault are sufficient to install, operate, and remove the butterfly valve and operating mechanism.

TABLE 24. Material Design Parameters—Del Valle Dam

Material	Specific Gravity	Unit Weight in Pounds Per Cubic Foot			Static Shear Strengths θ Angles in Degrees Cohesion in Tons Per Square Foot			
					Effective		Total	
		Dry	Moist	Saturated	θ	C	θ	C
Zone 1A	2.72	114	131	135	24	0	14	0.6
Zone 1B	2.72	114	131	135	24	0	14	0.6
Zone 2A	2.76	139	143	151	38	0	38	0
Zone 2B	2.78	116	--	136	38	0	38	0
Zone 3	2.76	146	152	156	38	0	38	0
Zone 4	--	114	129	134	29	0	16	0.6
Foundation	--	104	--	--	30	0	16	0.6

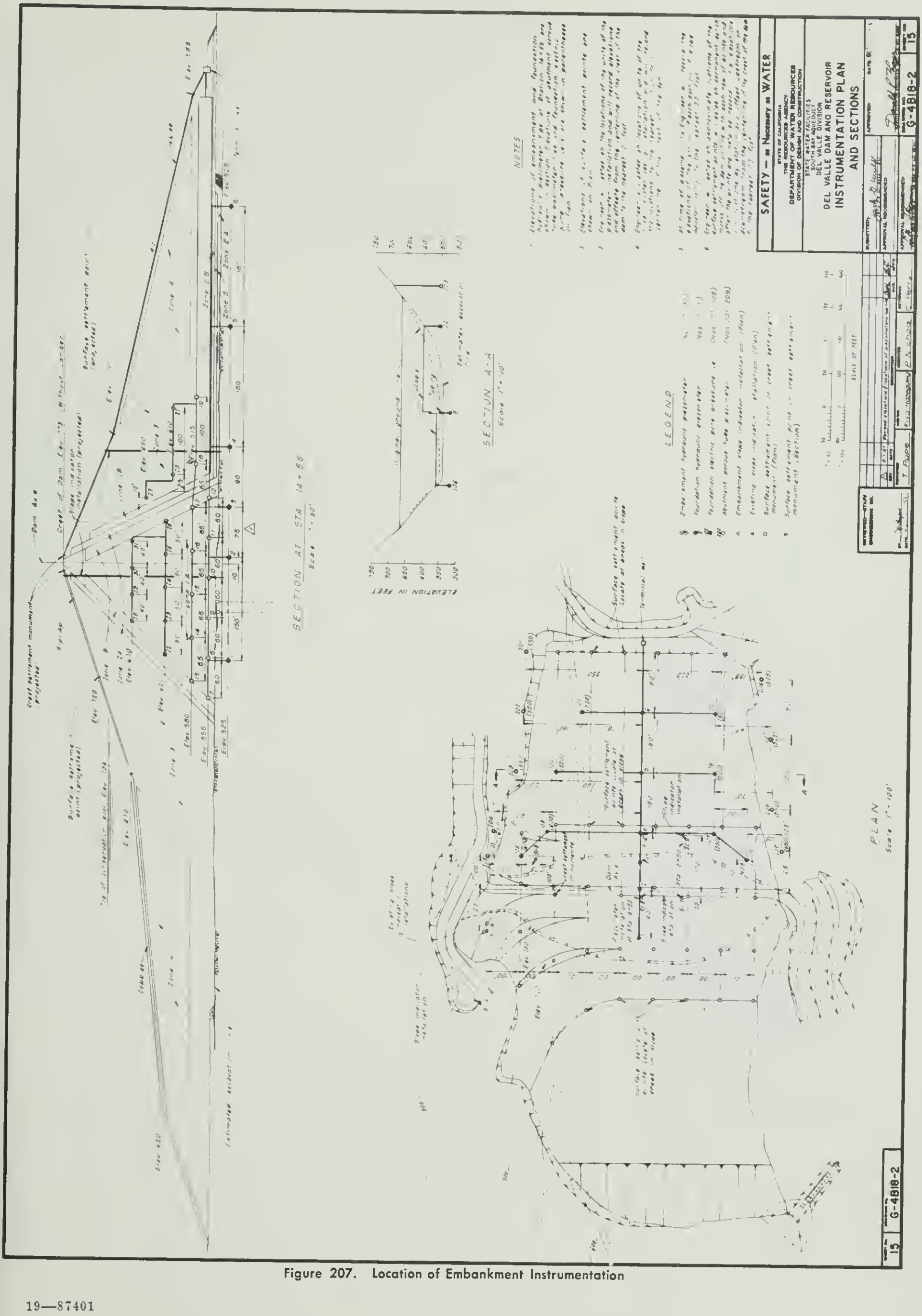

Figure 207. Location of Embankment Instrumentation

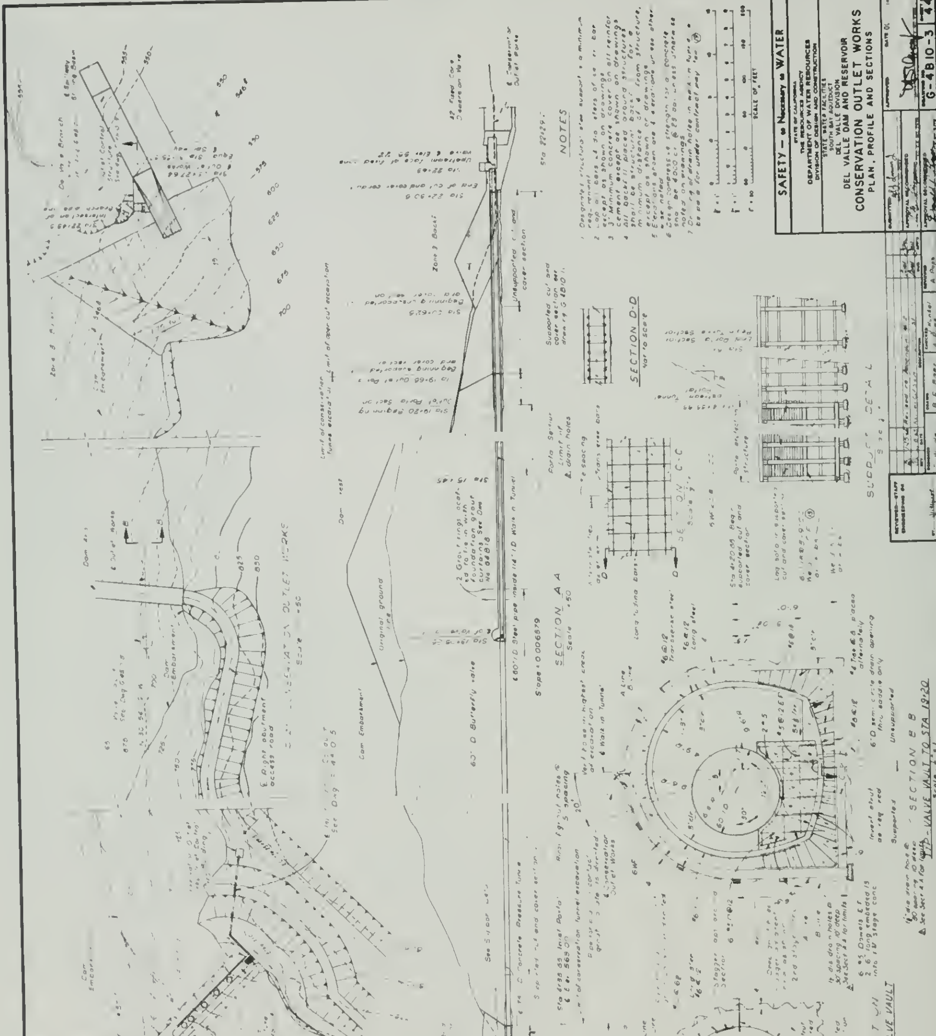

Figure 208. Conservation Outlet Works—Plan, Profile, and Sections

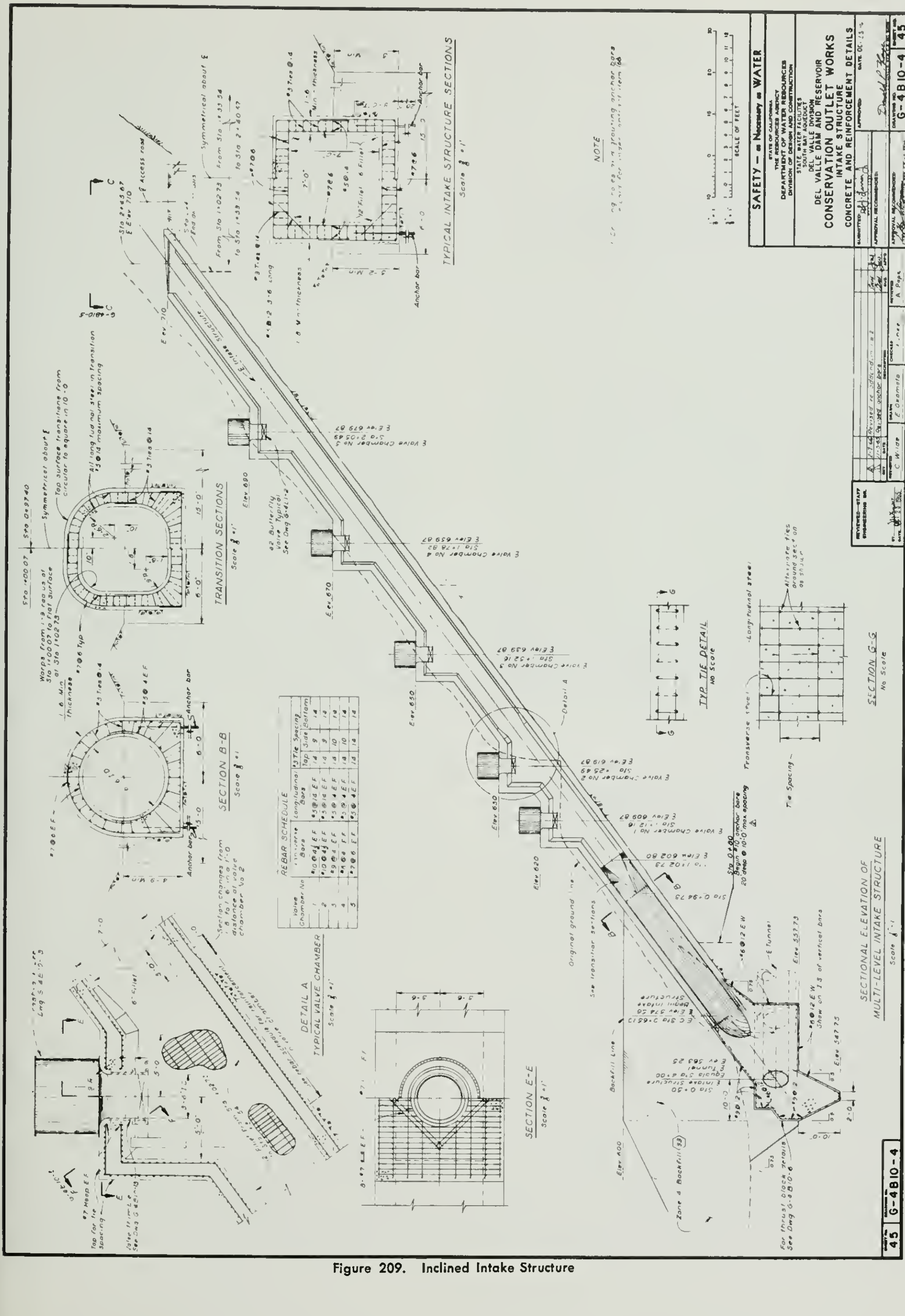

Figure 209. Inclined Intake Structure

The walk-in conduit is a concrete-lined, 114-inch-diameter, horseshoe-shaped section with a concrete walkway and concrete pipe supports. This walkway provides room for removal of the emergency shutoff butterfly valve. Peripheral drain holes are placed between Stations 15+45 and 19+65 (Figure 209) at 30-foot spacing to relieve the external hydrostatic pressure.

A sump and sump pump were installed at the downstream end to remove seepage entering the walk-in conduit. A ventilation system was provided, consisting of a fan near the downstream control structure and an air duct running from the fan to the valve vault.

The downstream control structure was constructed as an integral part of the left wall of the spillway stilling basin. It consists of a wye branch and thrust block, a 60-inch by 42-inch reducer, and a 42-inch fixed-cone dispersion valve. The valve allows the flow to discharge into a 10-foot by 9-foot chamber which confines the spray and directs the flow into the stilling basin. A control house was constructed above the valve to provide room for (1) valve operating equipment, (2) sump pump operating equipment, and (3) walk-in tunnel ventilating fan.

The conservation outlet works was designed to (1) release 400 cubic feet per second (cfs) at the minimum conservation pool, water surface elevation 638 feet, and (2) to pass a flow of 120 cfs between the reservoir and Del Valle Pumping Plant. To reduce pumping cost, project water from the South Bay Aqueduct can be discharged by gravity through the branch pipeline into the downstream end of the outlet and into Arroyo Del Valle to replace natural flow which is being stored in the reservoir.

The 42-inch butterfly valves in the sloping intake were designed to operate either fully open or fully closed, and the 60-inch butterfly valve in the vault beneath the dam crest remains open except in an emergency. Releases to the stream are controlled by the 42-inch fixed-cone dispersion valve at the downstream end. Flow between the reservoir and Pumping Plant is controlled in the plant. The rating curve for the conservation outlet works is shown on Figure 210.

The sloping intake may be dewatered for inspection and was designed for this condition with an external water load imposed by a reservoir water surface at elevation 710 feet. The thrust block was designed to spread the load of the intake structure at the base of the structure near streambed level.

Maximum allowable load on the foundation was 4,000 pounds per square foot. Trashracks are removable and were designed for yield stress at a differential head of more than 20 feet of water. Steel bulkheads were provided to allow closure of the openings should repair of the butterfly valves be necessary. The bulkheads were designed to resist the water load with the reservoir at elevation 710 feet.

The pressure tunnel was designed to resist the full reservoir head both internally and externally (applied individually) assuming no support from the surrounding rock. The valve vault was designed for an external pressure of a full reservoir. The walk-in tunnel from the valve vault to the tunnel portal was designed to resist the external water pressure, which was assumed to vary linearly from 68 pounds per square

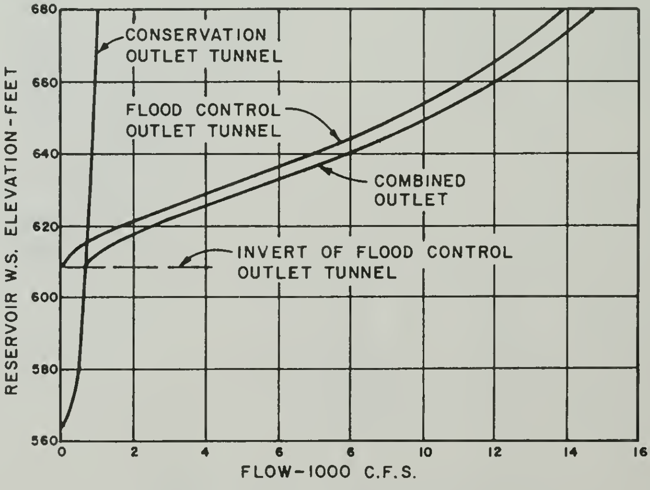

Figure 210. Conservation Outlet Works Rating Curve

inch (psi) at the valve vault to 0 psi at the downstream control structure. Rock loads from the tunnel roof were assumed to be resisted by the support installed during construction and were not added to the load on concrete sections. The walk-in conduit from the downsteam tunnel portal to the control structure was designed to support the load of the pervious backfill assuming the vertical load is equivalent to the weight of the overburden and the lateral load is one-third the vertical load.

The outlet control structure contains the welded steel wye, embedded in concrete, which connects to the discharge valve and connects the outlet works with the Del Valle Branch Pipeline and Pumping Plant. Retaining walls on each side of the control structure were designed as cantilever walls. The wall on the left side contains the fill on top of the structure, while the wall on the right side forms the stilling basin wall.

The 60-inch-diameter steel pipe and branches were fabricated from steel plate which has a minimum yield strength of 30,000 psi and an allowable stress of 15,000 psi. Saddle supports are spaced at 7-foot centers to eliminate the necessity of stiffener rings on the steel pipe. An expansion joint was placed near the midpoint of the pipe alignment to permit expansion and contraction of the steel pipe due to a change in temperature of up to 30 degrees Fahrenheit.

Flood Control Outlet Works

The flood control outlet works is located in the right abutment and connects with the spillway tunnel (Figure 211). It consists of a reinforced-concrete trash frame, a bell-mouth entrance, an 18-foot-diameter tunnel 216 feet long, and a junction structure with a 28-foot-diameter spillway tunnel. A concrete plug in the junction structure contains a transition to two sluiceways for the purpose of making flood control releases. Each sluiceway has two 72-inch by 90-inch, high-pressure, slide gates in tandem for regulation and emergency shutoff. These sluiceways discharge through a 10-foot by 12-foot conduit directly into the spillway tunnel. A gate chamber housing the hydraulic operators was provided above the sluiceways. Access to the gate chamber is by a stairway in a 9-foot-diameter, concrete-lined, vertical shaft from the spillway overlook parking area to a 9-foot-diameter, concrete-lined, access tunnel which slopes at 0.5% to the gate chamber. The control house over the vertical shaft contains electrical panels, ventilation equipment, and a standby engine-generator for emergency operation of the outlet works.

The flood control outlet works was designed to release up to 4,400 cfs with the reservoir surface at elevation 702 feet. To provide for emergency reservoir drawdown, the design discharge is 7,000 cfs with the reservoir water surface at the spillway crest, elevation 745 feet. The 4,400-cfs flow corresponds to the capacity of Arroyo Del Valle below the Dam, and the 7,000-cfs flow can be carried with minor damage. The intake trash frame was designed so that the velocity through the net area would not exceed 5 feet per second for a flow of 7,000 cfs. Except for a case of extreme emergency, the slide gates will not be operated at more than 90% open.

The intake structure was placed directly on Zone 3 fill rather than on existing rock formations to minimize excavation into the hillside. Two expansion joints were provided in the conduit between the tunnel portal and the intake structure to allow for differential settlement between the intake structure and the tunnel. The trash frame was designed for a yield stress at 20 feet of differential head.

The 18-foot-diameter, reinforced-concrete, tunnel lining from the stoplog slot in the intake structure to approximately 50 feet downstream of the tunnel portal was designed to support 30 feet of overburden, and the reinforcement in the concrete lining is sufficient to resist the full reservoir head. The flood control tunnel and spillway junction was designed to withstand the internal pressure of full reservoir head, assuming no support from the surrounding rock.

The gate chamber was designed for full hydrostatic head, with normal allowable stresses for water at the lip of the spillway, elevation 745 feet, and no more than one-third overstress for water at the spillway design flood elevation of 765 feet. Allowance was made in the gate chamber dimensions to give sufficient clearance for installation, operation, and removal of slide-gate bonnets, hydraulic operators, and gate leaves. The access tunnel and shaft were designed to support 30 feet of highly fractured overburden, and the concrete lining was designed for full hydrostatic head due to a water surface elevation of 765 feet.

Spillway

Description. The first type of spillway investigated was an open chute on the left abutment. This alternative was abandoned because of an extensive shear zone that made the location undesirable for a spillway structure.

Both side-channel and glory-hole spillways were considered for the right abutment, but these also were abandoned because of the high cuts that would be necessary. A failure of the cut slope could cause rock to slide into the intake structure and render the spillway inoperable.

The design selected was a glory-hole spillway located on a knob 1,600 feet southeast of the Dam.

At this location, cut-slope heights were minimized. The spillway intake structure was located 50 feet from the base of the cut slope at its nearest point (Figure 211) in order to provide a space to catch small slides should they occur. The unlined approach apron is on a relatively flat plane at elevation 738 feet and is sloped at 5% in two directions for drainage. The inlet consists of an 84-foot-diameter ungated crest at elevation 745 feet. Antivortex vanes are placed at the one-third points on the circumference. A log boom prevents large debris from entering the glory-hole spillway in-

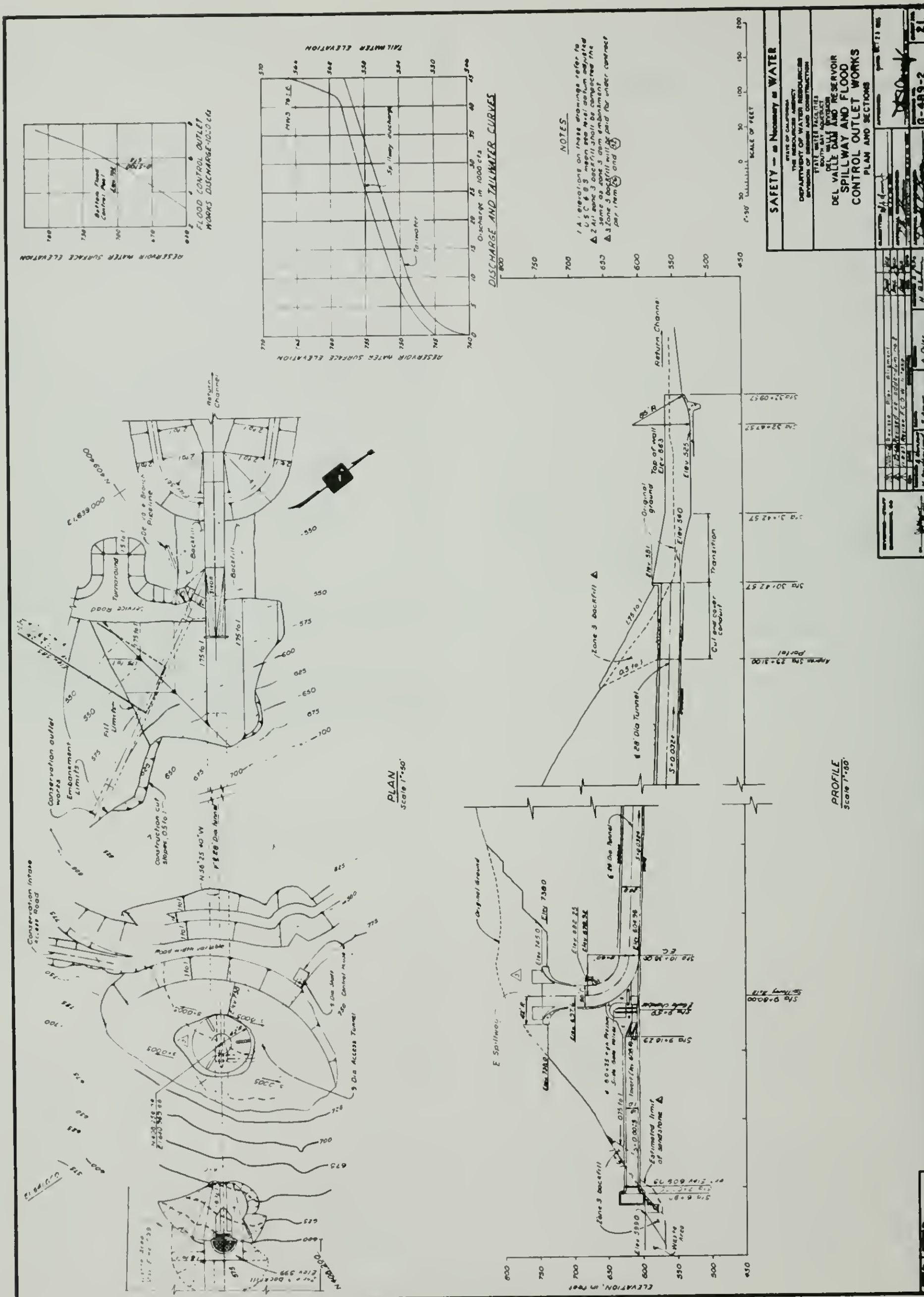

Figure 211. General Plan of Flood Control Outlet Works

take. A transition connects the ogee crest to a 30-foot-diameter shaft at elevation 697.60 feet, which extends to an elbow at elevation 678.92 feet. The elbow reduces the diameter uniformly to 28 feet at the point where it connects with the spillway tunnel. The flood control outlet works, previously described, connects at this elbow. An air vent leads from near the top of the elbow to the top of one of the antivortex vanes. The spillway tunnel extends downstream 1,893 feet from the elbow to the downstream portal, and a 28-foot-diameter cut-and-cover conduit, 111 feet in length, ends at the transition to the stilling basin. The transition is 100 feet long and changes from a semicircular section to a rectangular section. Within this length, the invert elevation falls 15 feet.

Hydraulics. The Federal Government, acting through the U. S. Army Corps of Engineers, participated in the flood control aspects of the project and initially required that the maximum discharge during the standard project flood be not more than 7,000 cfs (the capacity of the downstream channel allowing only slight damage). This limitation was placed on an occurrence of the standard project flood with reservoir level at the spillway lip and the flood control outlet not being used. Routing of the standard project flood (spillway weir elevation 745 feet) resulted in a peak discharge of 7,500 cfs. To reduce the peak discharge to 7,000 cfs, it would have been necessary to raise the weir elevation and the dam crest. The Corps decided that the additional cost of construction to lower the discharge was not warranted and set the final criteria for the maximum discharge during the standard project flood at 7,500 cfs. Discharge during the maximum probable flood would be 44,200 cfs with a maximum water surface of about 765 feet, leaving 8 feet of freeboard above the resulting water surface. Flood hydrographs are shown on Figure 212.

The spillway stilling basin (Figure 213) was designed so that the hydraulic jump would not move upstream into the conduit. To avoid negative pressures on the floor of the basin, the vertical curve was made flatter than the trajectory of a free-discharging jet. The stilling basin will contain a full hydraulic jump for all flows up to 7,500 cfs. Model studies indicate a partial jump is contained for flows up to 10,000 cfs. A flip sill was placed at the downstream end of the stilling basin to create a discharge trajectory with impact greater than 80 feet from the structure for flows ranging between 10,000 cfs and 44,200 cfs and to keep the dynamic load on the sill to a minimum. The excavated portion of the return channel was designed to carry flows up to 4,400 cfs without expanding to an overbank condition. This is the capacity the natural downstream channel can carry without causing flood damage. Water is released through the flood control outlet works when the reservoir surface reaches elevation 702 feet during the flood season.

Structural Design. The crest structure and approach structure are located in the Panoche formation. Bearing tests conducted on similar foundation material indicated a maximum allowable foundation pressure of 3,000 pounds per square foot.

The crest structure was analyzed as a gravity structure. The loads on the weir were calculated with the water surface at elevations 745 and 764.6 feet. Full uplift pressures along with horizontal and vertical seismic forces were considered. The concrete in the throat transition and shaft was analyzed as horizontal rings subject to uniform lateral rock loads and hydrostatic pressures. The antivortex piers are concrete structures cantilevered vertically from the spillway crest.

The concrete tunnel lining and stilling basin designs considered static and dynamic forces of floodflows as well as loading due to the reservoir and backfill. Where dynamic loading is critical, one-third overstress (one-quarter in end sill) is allowed and, where static loading is critical, normal stresses are allowed.

Structural design of the stilling basin considered loading of the backfill as well as static and dynamic loading of water during flood discharge. One-quarter overstress was allowed in the end sill when dynamic loads were considered. A drainage system under the transition and stilling basin, consisting of interconnected longitudinal and transverse vitrified clay pipes, relieves uplift pressure, distributes pressures uniformly, and provides a drainage path for the water.

Mechanical and Electrical Installations

Power to operate the mechanical equipment at Del Valle Dam is supplied from Del Valle Pumping Plant to the outlet control house, located at the downstream toe. From there, power is extended to the other facilities at the Dam (Figure 214). The Pacific Gas and Electric Company supplies 480-volt, 3-phase, 60-cycle power to Del Valle Pumping Plant. Only the motors that drive the gate operators, sump pumps, and ventilators can make direct use of this power. For all other uses, the power is transformed to 120 volts, single phase.

Flood Control Outlet Works. The high-pressure slide gates in the flood control outlet can be opened or closed in 15 minutes by a motor-driven hydraulic operator. The control panel is in the gate chamber. Air for ventilation is supplied through an 18-inch aluminum conduit. Air for the slide gates is furnished by two 24-inch aluminum ducts routed through the access tunnel and shaft from the intakes in the control house located over this shaft. An emergency generator, capable of producing the power to operate the flood control outlet works in case of power failure, was installed in this control house.

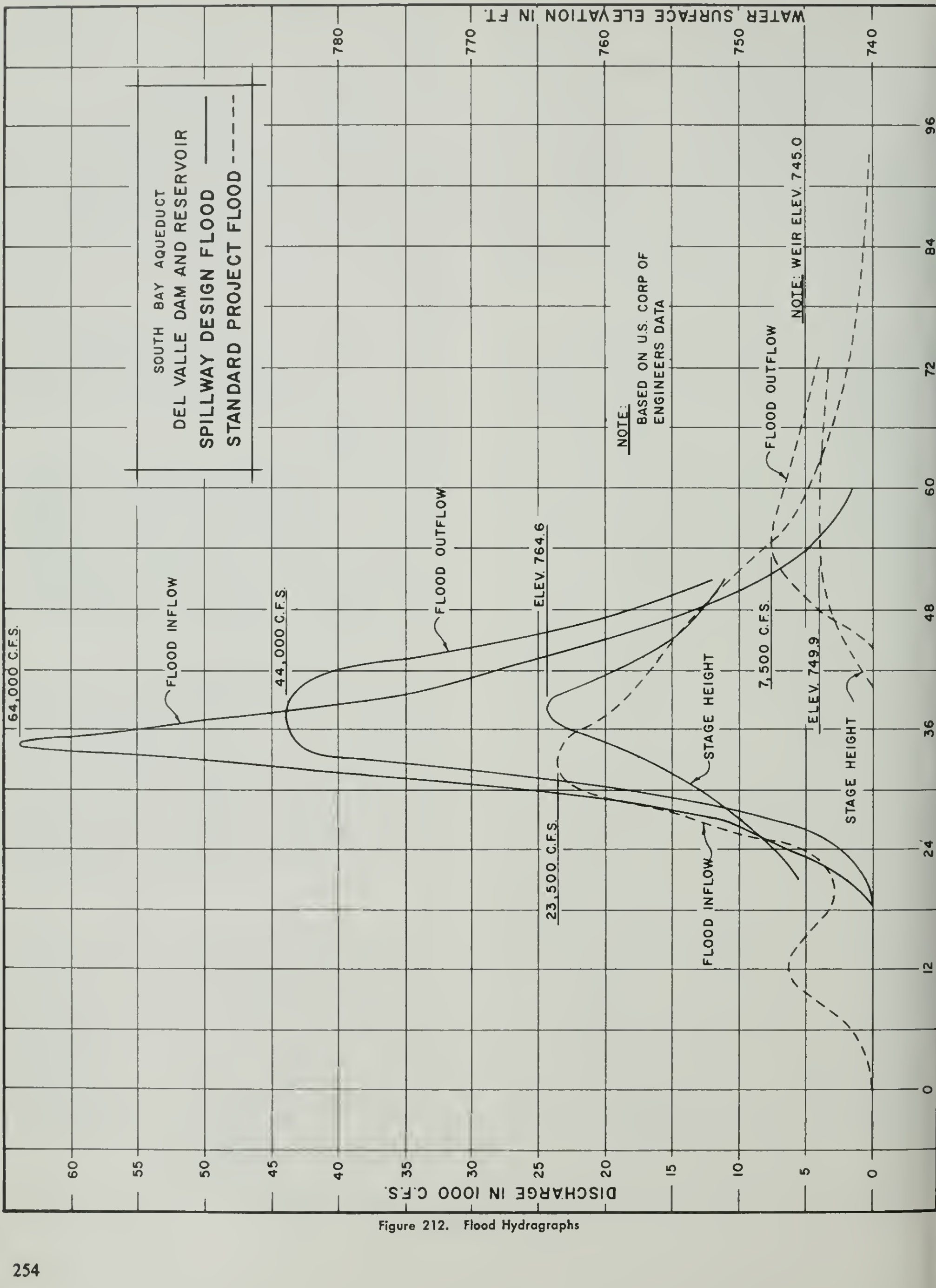

Figure 212. Flood Hydrographs

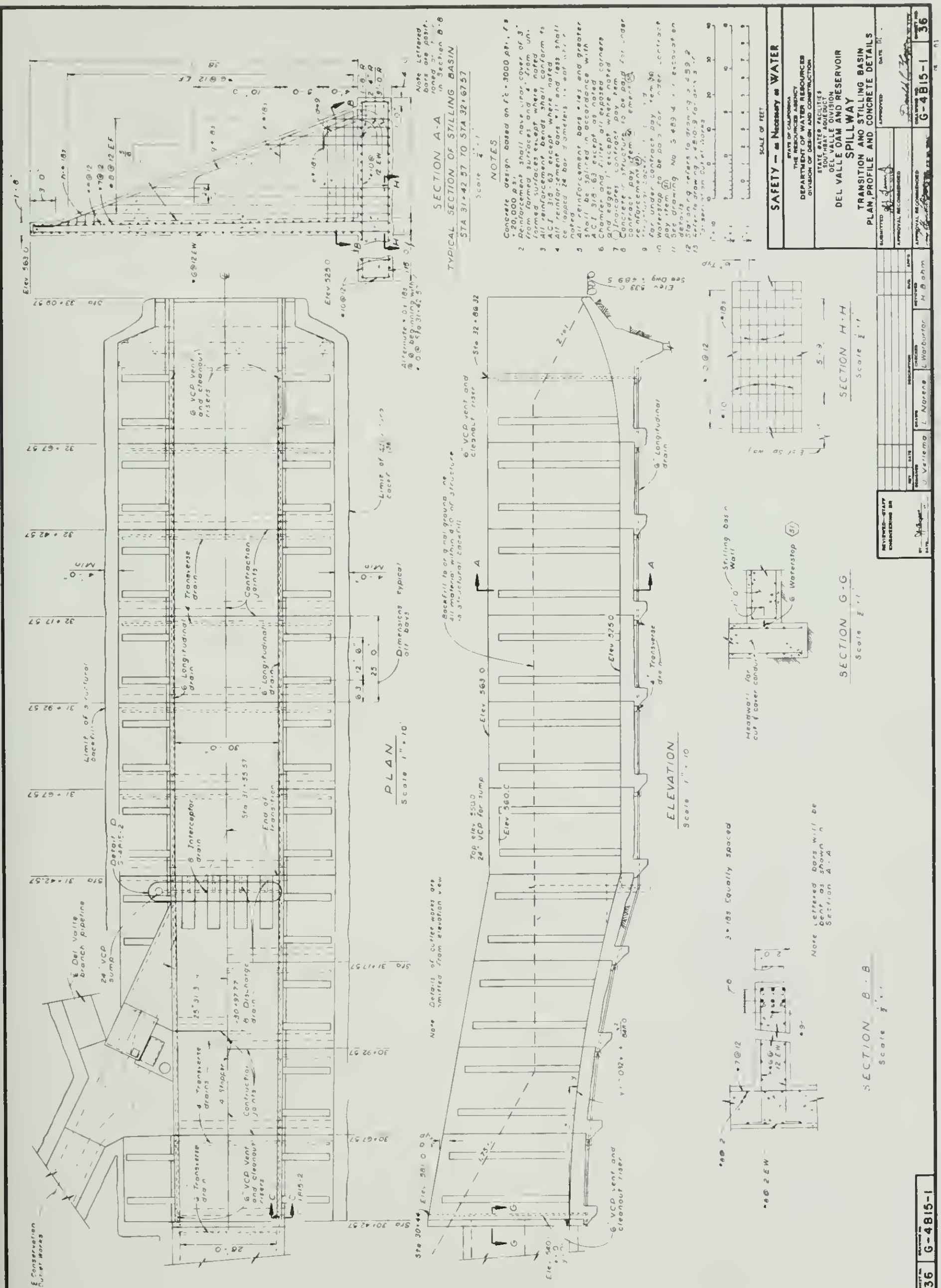

Figure 213. Spillway Stilling Basin

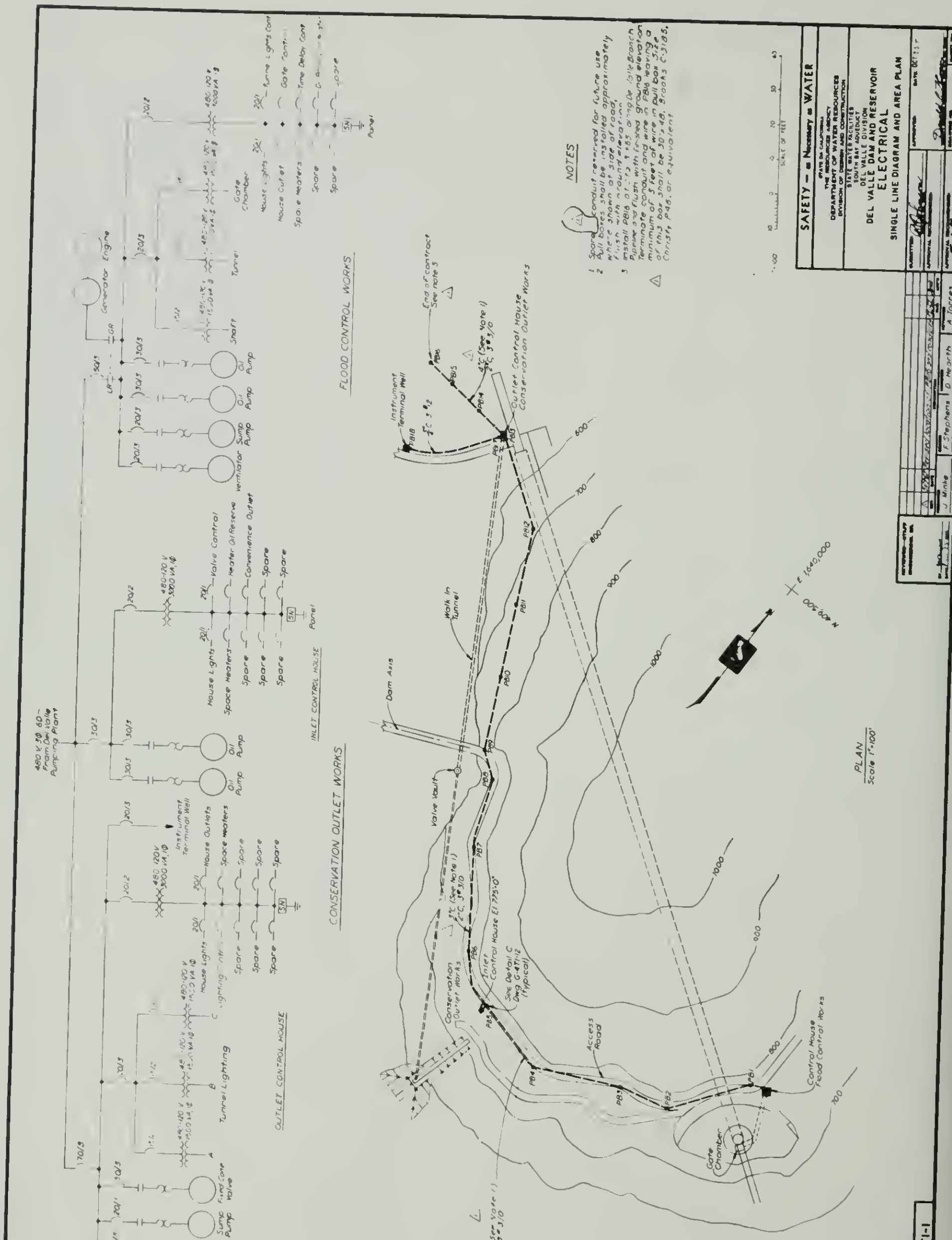

Figure 214. Single-Line Electrical Diagram

Conservation Outlet Works. The 42-inch butterfly-valve operators in the sloping intake are activated from a motor-driven pump in a control house located above the intake. The operator for the 60-inch butterfly valve is activated by a motor-driven pump located in the conservation outlet works valve chamber. Operating time for all butterfly valves is four minutes. Two 10-inch vacuum valves were installed on the downstream side of the 60-inch butterfly valve within the valve chamber. Air for ventilation is supplied by a blower in the conservation outlet works through an 8-inch aluminum conduit in the walk-in tunnel.

The fixed-cone dispersion valve in the conservation outlet works can be opened or closed in five minutes by a motor-driven hydraulic operator in the outlet control structure. A sump and sump pump that drain the walk-in tunnel are located on the right side of the outlet control structure and empty into the spillway basin through a 2½-inch, galvanized, steel pipe. The 6-inch, manually operated, blowoff valve for the conservation outlet works empties into the same chamber as the 42-inch fixed-cone dispersion valve.

Construction

Contract Administration

General information about the contract for the construction of Del Valle Dam and reservoir, Specification No. 66-01, is shown in Table 25. This contract included the construction of the Dam and its appurtenant structures.

TABLE 25. Major Contract—Del Valle Dam and Reservoir

Specification	66-01
Low bid amount	$16,577,802
Final contract cost	$16,520,404
Total cost-change orders	$242
Starting date	3/28/66
Completion date	9/17/68
Prime contractor	Green Construction Co. & Winston Bros. Co.

Diversion and Care of Stream

First Construction Season. During the summer of the first construction season, 1966, a trench was excavated across the valley at the upstream toe of the Dam. Collected water was pumped into a 12-inch pipe running along the county road and discharged into a settling basin below the Dam site. A ditch was excavated from the settling basin to the stream for return flow. The original streambed between the return point and the downstream toe of the Dam then was used as a spoil area for tunnel muck from both the flood control and conservation outlets.

The foundation for the Dam was excavated, the curtain grouted through a grout cap, and embankment placed to the original level of the streambed. A flood control channel was excavated from the downstream toe of the Dam to return floodflows to the original channel. That winter, a maximum streamflow of 5,600 cfs caused only minor damage.

Second Construction Season. In the spring of 1967, a cofferdam was installed around the inlet end of the conservation tunnel, and 36-inch and 24-inch corrugated-metal pipes were installed through the cofferdam into the tunnel. The contractor's plan was to divert the stream through the conservation tunnel when the flow fell below 50 cfs so embankment placement could begin. This occurred on May 11, 1967.

The only water flowing downstream through the conservation outlet works during the summer was leakage through the cofferdam which was diverted by a 12-inch pipeline through the tunnel to facilitate the remaining work on the outlet.

By the fall of 1967, the Dam had been topped out, but the flood control outlet works was not yet operable. The diversion plan for the winter utilized the reservoir as a temporary detention basin by releasing the flows through the conservation outlet works. This diversion operation would control a 100-year flood with the maximum reservoir stage at the invert elevation of the flood control outlet works, 40 feet above the crown of the conservation outlet works. A steel bulkhead was available to place in the stoplog slot of the flood control intake should a larger flood occur. The empty reservoir capacity would have contained the entire maximum probable flood volume well below the crest of the spillway. The winter of 1967–68 was exceptionally dry, and stream diversion during this period did not present any problems.

Third Construction Season. Following an extremely dry winter, the streamflow was low enough to allow placement of the permanent plug in the diversion portion of the conservation outlet works on May 16, 1968. The storage of water behind Del Valle Dam essentially began on that date.

Foundation

Dewatering. During streambed excavation and curtain grouting, the diversion across the channel at the upstream toe of the Dam adequately dewatered the foundation. Before any embankment could be placed, it was necessary to cut off all flow into the excavated area. This was accomplished by installing a system of French drains just downstream of the cutoff trench. These drains joined a rock-filled sump 50 feet downstream of the upstream toe of the Dam. A 15-foot length of 24-inch corrugated-metal pipe was placed vertically at the low point of the sump, and the water collected was pumped into a 12-inch discharge line by a float-controlled pump.

Excavation. The first area excavated was the foundation for Zones 1A and 3 and upstream Zones 1B and 2A. The excavated material was hauled to Zone 3 and 4 stockpiles. Inspection of the exposed foundation revealed that it was suitable for acceptance of Zone 1A material without the additional 3-foot depth of excavation required by the plans and specifications.

The second area excavated was in the vicinity of the upstream and downstream portals of the conservation outlet works and most of the foundation area below elevation 668 feet between the two portals. A bench was constructed along the right abutment to facilitate construction of the conservation outlet works during the winter season. The excavated material was loaded at the base of the abutment and hauled to the Zone 4 stockpile.

When excavation of the lower part of the right abutment was completed, isolated pockets in the Zone 1A area of the streambed were cleaned out and the material hauled to the Zone 4 stockpile downstream of the Dam.

The third major area excavated was the streambed between the downstream limit of Zone 1A and the downstream toe of the Dam. All material suitable for Zone 3 was placed directly in upstream Zone 3 and remaining material was hauled to the Zone 4 stockpile.

The shear zones were excavated and filled, completing the preparation of the streambed portion of the foundation on October 10, 1967. The embankment for the Dam then was placed to the original level of the streambed in accordance with the plans and specifications, except that Zone 2A material was temporarily substituted for Zone 2B material. Otherwise, silt from the winter flows would have infiltrated Zone 2A material, destroying the drainage characteristics. After the flood season, surplus Zone 2A was trimmed back and replaced with Zone 2B material.

In January 1967, work commenced on the fourth area to be excavated, the upper portion of the right abutment below the access road. Dozers side-cast material from the cut where it was loaded at the base of the abutment and hauled to the upstream spoil area. Boulders were separated and hauled to the upstream riprap stockpile by end-dump trucks.

In February 1967, stripping commenced on the fifth and last area, the left abutment. The material was hauled to Zone 3 and 4 upstream stockpiles. In May, a surface crack appeared at Station 33+00 normal to the dam axis revealing a slide potential on the abutment. Four thousand cubic yards were placed and compacted between Stations 29+50 and 37+50 extending from the streambed (elevation 555 feet) to elevation 620 feet to stabilize the abutment and prevent a slide.

Flood Cleanup. Cleanup of foundation debris resulting from the first season flooding was completed on May 27, 1967. Material suitable for Zone 3 was spread to dry in upstream areas, and material suitable for Zone 4 was hauled to a stockpile.

Dam Foundation Grouting

Dam foundation grouting included blanket grouting to seal near-surface voids and curtain grouting to form a barrier against seepage beneath the Dam. The blanket holes were drilled and grouted before the curtain holes to control surface leaks while grouting the curtain holes.

The starting grout mix (water-cement ratio) for both blanket and curtain grouting was originally 7:1, gradually increasing in thickness to 1:1 or ½:1 as necessary. The starting mix was later changed to 5:1.

Grouting pressures were controlled carefully because the foundation could be deformed easily with grouting pressures exceeding 1 psi per foot of hole depth. The pressures were limited to 0.50 to 0.75 psi per foot of hole depth.

Blanket Grouting. Drilling and grouting of the blanket holes were done in one or two stages, depending upon the water-pressure test results. The holes were drilled to a depth of 15 feet, washed, and water-tested at a maximum pressure of 10 psi. If the water loss exceeded 0.5 cubic feet per minute, this interval was grouted and then the interval from 15 to 25 feet was drilled and grouted. If the water loss was less than 0.5 cubic feet per minute, the hole was completed to a depth of 25 feet, then washed, water tested, and grouted. In areas of very disturbed rock, no water testing was done.

Initially, the blanket holes were arranged in a triangular pattern over the entire Zone 1 foundation area. Later, the plan was modified to concentrate the blanket grouting near the grout curtain and the upstream portion of the Zone 1 foundation. Two additional rows of blanket holes were added: one 10 feet upstream from the curtain with holes on 10-foot centers and the other 20 feet downstream from the curtain with holes on 20-foot centers.

Curtain Grouting. The curtain grout holes were arranged in two parallel lines, about 15 feet apart, running the full length of the Dam near the center of the Zone 1 embankment foundation. Initially, primary holes 100 feet deep on 40-foot centers were drilled and grouted; then 75-foot-deep secondary holes between the primary holes were drilled and grouted. In areas of relatively high grout take, the spacing was split even farther, with additional holes 50 feet deep. Final spacing was 2½, 5, or 10 feet for the curtain holes. The curtain holes were drilled and grouted in stages of 25 feet (Figure 215).

To test the effectiveness of the curtain grouting, 32 holes were drilled and grouted in a plane midway between the two grout curtains. These holes were drilled at angles of 40 to 60 degrees with respect to the curtain holes. Most of these holes were tight when water-tested or they had negligible grout takes.

The grout take was far less than anticipated. Only 47,163 cubic feet of the estimated 150,280 cubic feet were used.

Embankment Materials

Impervious. Zone 1A material was acquired from Borrow Area L located approximately 4,000 feet east of the Dam site on terrain that slopes into Arroyo Del

Figure 215. Left Abutment Excavation and Curtain Grouting

Valle (Figure 216). This material is composed of clays and sandy clays from the Livermore formation. Prior to excavation, the contractor prewet Borrow Area L to moisture-condition soils which would be used in the embankment. During initial excavation in the southwest corner, unsuitable sands and gravels were encountered. The contractor then moved to other areas and obtained acceptable materials although the borrow was heterogeneous at times. Scrapers were loaded downhill so that materials from the different strata would be mixed, resulting in a more homogeneous embankment.

Transition. Zone 1B is the impervious transition between the Zone 1A and Zone 2A filter. Zone 1B is more coarsely graded than Zone 1A and was obtained from alluvial soils in Borrow Area P, located about 3,000 feet upstream of the Dam site adjacent to the southern limit of Borrow Area L. This material was old alluvial soils from stream-terrace deposits. The borrow deposits occurred in two zones: an upper zone of impervious soils derived primarily from the Livermore formation and a lower zone of semipervious terrace deposits. The alluvium in Borrow Area P contained erratic lenses of gravel and clay. Satisfactory mixing and blending was achieved by prewetting the surface prior to excavation and crosscutting the alluvial fans. The bulk of Zone 1B was taken from the northeast portion of the borrow area.

Filter. Zone 2A in the downstream portion of the embankment is the filter between Zone 1B and the pervious drain, Zone 2B. In the upstream portion of the Dam, Zone 2A acts as a drain during reservoir drawdown. Material for Zone 2A was processed from materials obtained in Borrow Area S, which extended from the Dam site to a point about 5 miles upstream. The processing plant was established approximately 3 miles upstream from the Dam site, in the Arroyo Del Valle stream channel. The processed material, fairly well-graded from the ¾-inch size to the No. 200 sieve size, was hauled from the plant to the embankment in 25-cubic-yard-capacity bottom-dump wagons. Material in Borrow Area S was, in general, a homogeneous sandy gravel composed of subrounded to rounded particles of resistant sandstone, cherts, schists, and assorted igneous and metamorphic rocks. The material particle size generally was less than 8 inches.

Drain. Zone 2B is the drain zone designed to convey seepage from the core and abutments to the downstream toe. Zone 2B is a coarse material with 100% passing the 4-inch sieve with no more than 5% permitted to pass the No. 4 sieve. This material was processed by the same plant previously described. Material was obtained from Borrow Area S. Due to the gradation of material in Borrow Area S, an excess of Zone 2B resulted while obtaining the necessary quantity of Zone 2A and cobbles. By change order, the contractor was permitted to place the 30,000 cubic yards of surplus Zone 2B material in the Zone 3 embankment above elevation 755 feet at no additional cost.

Outer Shell. Zone 3 forms a portion of the outer shells of the Dam. Zone 3 comprises the bulk of materials placed in the embankment, some 2,171,834 cubic yards. Material for Zone 3 was stream-channel gravels excavated from Borrow Area S. Some Zone 3 material also was acquired from foundation excavation.

Random. Zone 4 is a random zone forming portions of the upstream and downstream shells. This zone accounted for the second largest quantity of material placed in the embankment. Materials for Zone 4 were acquired from stockpiled material from the dam foundation, open-cut excavations, spillway tunnel and shaft excavations, conservation tunnel excavations, and from the specified borrow areas after stockpiles were exhausted. Zone 4 materials are clayey gravels, sandstones, shales, and talus.

Slope Protection. Cobbles also were obtained during the processing of materials from Borrow Area S and were placed on the embankment slopes for erosion protection. By change order, the upstream cobble thickness was reduced to 1 foot. The reduced quantity conformed more nearly to the amount available after obtaining the necessary quantities of Zone 2A and Zone 2B material. The specifications required all material to be coarser than 3 inches.

Riprap required for upstream slope protection, stilling basin outlet, and downstream channel was obtained approximately 26 miles from the Dam site. The source was existing stockpiles of large rock removed during excavation of the California Aqueduct. The rock was reduced to proper size by a pneumatic im-

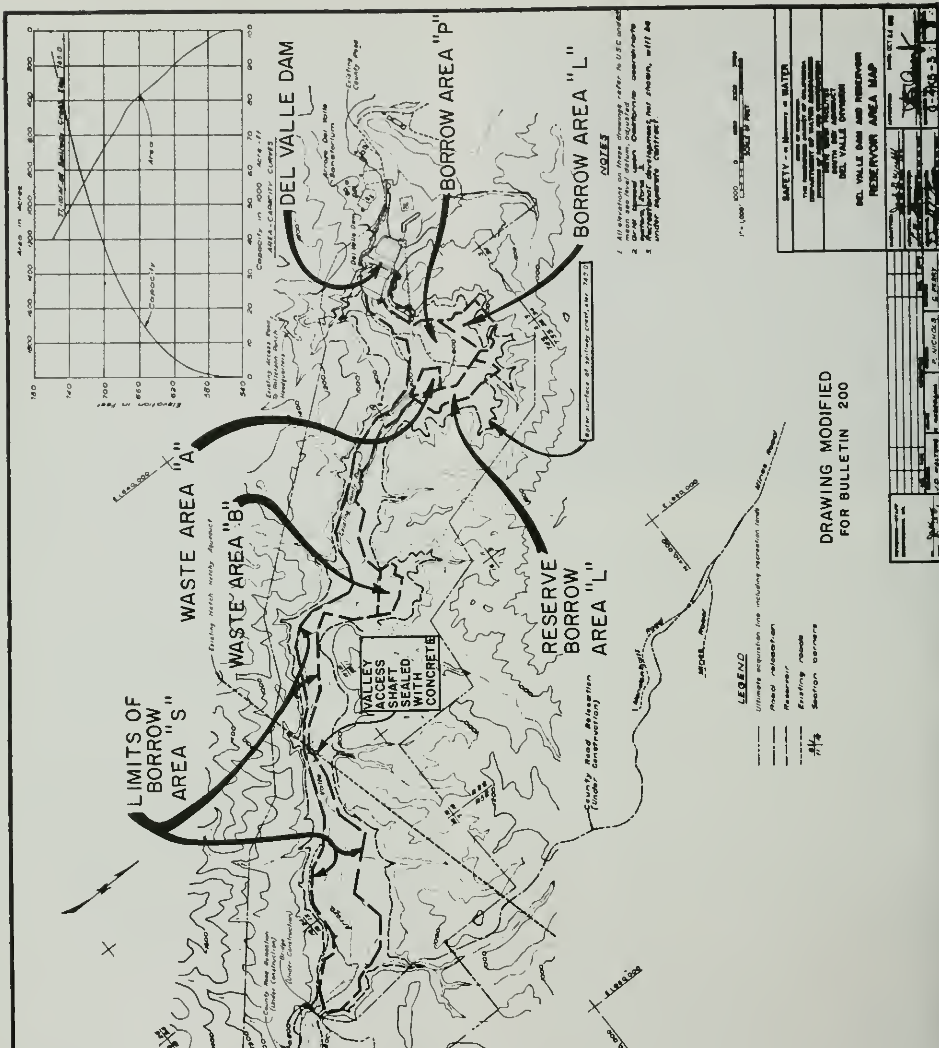

Figure 216. Location of Borrow Areas and Del Valle Dam Site

TABLE 26. Compaction Data—Del Valle Dam

	1A	1B	2A	2B	3	4
Compaction—percent	97.6	97.6	79.4	110.5	101.5	97.0
Standard deviation	2.6	2.6	13.3	19.5	4.7	2.7
In-place dry density—pounds per cubic foot	121.2	131.8	130.3	114.6	143.2	116.4
Standard deviation	4.8	5.2	5.7	3.9	5.8	6.3

pact hammer mounted on the boom of a backhoe. It was transported in 12-cubic-yard truck-trailer units. A minor amount of riprap was obtained from stockpiled boulders from foundation excavation.

Embankment Construction

Table 26 shows the as-built relative compaction and in-place dry density of the various zones within the Dam. Placement and compaction of embankment materials is shown on Figure 217.

Impervious and Transition. Placement of Zone 1A material began during October 1966 following completion of the backfill of all shear zones in the foundation. The bedded borrow materials were loosened prior to and during excavation by a tractor-mounted ripper. In some areas, the material was very hard and rock-like in nature but was sufficiently broken down by the disking and compacting operations to be removed easily. The contractor transported and spread Zone 1A material using a scraper operation. Zone 1B construction procedures were the same as for Zone 1A. Placement of Zone 1B began during September 1966 by backfilling overexcavated foundation shear zones. Zones 1A and 1B were placed, spread, and compacted parallel to the dam axis in layers not exceeding 6 inches in compacted thickness.

Following placement of each layer, the materials were blended and mixed by means of a disc pulled by a tractor. Any necessary moisture corrections were made prior to rolling by use of 10,000-gallon water wagons. Material placed against abutments generally was placed with a higher moisture content to obtain more plasticity and wheel-rolled with a rubber-tired tractor.

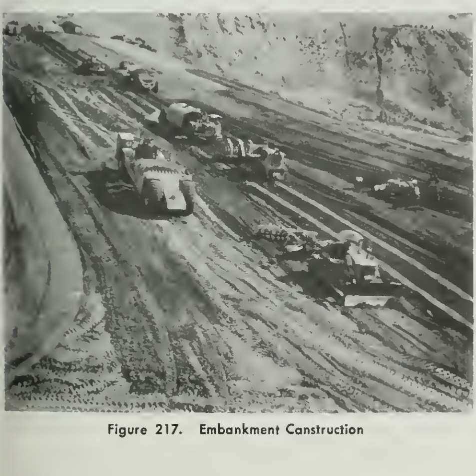

Figure 217. Embankment Construction

Compaction of Zones 1A and 1B was achieved with two tandem-roller tractor combinations. The four rollers were double-drum, sheepsfoot, tamping rollers, 5 foot by 5 foot in size.

The standard deviation for in-place densities for the first period of Zone 1A and 1B placement was 4.6 compared to the overall value of 2.6 at the end of the job. A standard deviation of 3.0 or less was considered acceptable control.

Compaction test data indicated that moisture control was good. Prewetting in the borrow area and efficient disking of the fill contributed to the good control achieved.

Filter and Drain. Zone 2A material was loaded from stockpiles at the processing plant with a 2½-cubic-yard-bucket rubber-tired loader. Then, it was hauled to the embankment in 25-cubic-yard loads with a bottom-dump wagon powered by a rubber-tired tractor.

Zone 2B material was loaded at the processing plant with a 4-cubic-yard-bucket rubber-tired loader and hauled to the embankment by scrapers powered by rubber-tired tractors.

Both Zones 2A and 2B were placed in layers not exceeding 15 inches after compaction with sufficient moisture content to preclude bulking. Following dumping on the embankment, the Zone 2A and 2B material was leveled to required thickness with a rubber-tired dozer and moisture was added. Sufficient time was allowed to permit penetration of added moisture. Zones 2A and 2B then were compacted with two passes of vibratory rollers. The compaction equipment consisted of three smooth-drum vibratory rollers connected in a triangular pattern and drawn with a large tractor.

Outer Shell. Initial placement of Zone 3 began during September 1966, with material from the downstream Zone 2 foundation excavation being hauled directly into upstream Zone 3. After this source was exhausted, Zone 3 material was excavated from Borrow Area S using scrapers pushed by large tractors. The material then was hauled and placed in the embankment.

Zone 3 was placed in layers not exceeding 15 inches in compacted thickness. Moisture conditioning was required on materials containing over 15% finer than the No. 4 sieve size. Compaction was achieved with two passes of the vibratory compactor used for Zones 2A and 2B.

Moisture conditioning was required on the embankment for most Zone 3 material. In order to achieve the required density, the contractor was directed to saturate the area prior to compaction. Satisfactory results were obtained using this method.

Random. Zone 4 material was placed in 8-inch compacted layers with moisture conditioning provided in a manner similar to Zone 1A and Zone 1B. It was compacted by six passes with a sheepsfoot tamping roller having the same characteristics mentioned under Zone 1A. Zone 4 material had been stockpiled both downstream and upstream from the embankment. Material was excavated, hauled, and placed in the embankment with scrapers. Moisture was added on the stockpiles and embankment with 10,000-gallon water wagons.

The in-place dry density of Zone 4 averaged 116.4 pounds per cubic foot. Considering the range of materials used, the density was uniform.

Slope Protection. The cobbles and riprap were dumped and spread on the slopes. No testing was required for these materials.

Instrumentation. Reading of the embankment instruments (Figure 207) commenced upon installation and continued during the construction period. The responsibility was transferred to operating personnel after the Dam was completed.

Shortly after installation, 24 of the 31 hydraulic piezometers located in the embankment core gave questionable readings. The remaining 7, along with the foundation piezometers, however, were believed to be able to provide adequate information to monitor the performance of the Dam.

Conservation Outlet Works

Outlet Portal. Excavation of the outlet portal of the conservation outlet works was started in April 1966. Because of the close proximity of the spillway tunnel outlet, the portals for the two tunnels were excavated at the same time (Figure 218).

To avert loss of the portal by a landslide, a unique construction technique was employed. The first 100 feet of the cut-and-cover tunnel adjacent to the portal resembled a fully supported tunnel. The excavation was made, 6-inch WF tunnel supports with invert struts were installed on 2-foot centers, and invert concrete was placed to anchor the supports. Solid lagging was placed entirely around the outside of the supports, and grout was placed in the narrow space between the lagging and the excavation in those locations where compaction of backfill would be difficult. Then structural backfill was placed, and the

Figure 218. Combined Outlet Works

crown was placed later from the inside as in a tunnel section. The driving of the tunnel was not started until after the backfill over the first 100 feet of cut-and-cover sections was completed. There is little question that this method of construction protected the tunnel portals from a major slide. During excavation, a crack developed just downstream of the tunnel portal and the formation was shored with heavy timbers. Tunnel supports and struts then were placed between timbers, invert was placed, backfill was completed, and shoring was removed without a slope failure.

Inlet Portal. Excavation for the inlet cut-and-cover section commenced on July 1, 1966, following a procedure similar to the one used at the outlet portal. The 18 feet of cut-and-cover conduit adjacent to the tunnel portal was supported and the remaining 32 feet was unsupported.

Tunnel. The tunnel was driven on a three-shift-per-day basis from the outlet end, through the valve vault area, to Station 9+53. Excavated material was stockpiled downstream for use in the Dam. The remainder of the tunnel was driven from the inlet end. Seepage water was encountered in all tunnel sections and was controlled by pumping through a 4-inch line. Continuous pumping was required during concreting of the tunnel.

Concrete. Invert placement for the walk-in tunnel included the lower 9 inches of the walls and was formed with steel channels set on wooden headers. The form was an open structure utilizing a manually operated strike-off to maintain the invert radius. The crown form was a steel horseshoe structure supported on an overhead needle beam. Concrete was placed by a pumpcrete machine through an 8-inch line. Internal vibrators were used inside the form, and form vibrators were spaced along the form surfaces. An invert section was placed in the afternoon and the corresponding crown section the following morning.

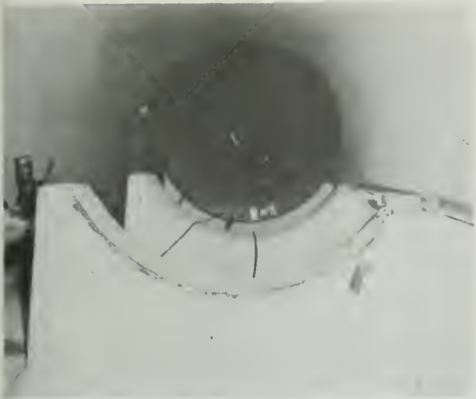

Figure 219. Concrete Saddles for 60-Inch Pipe

The 9-foot-diameter pressure section was monolithically cast in 30-foot sections using forms split into three sections. Concrete was placed in the same manner as in the crown of the walk-in tunnel.

The final 180-foot downstream section of the walk-in tunnel was cut and cover. The invert was placed as in the supported section, and the same interior horseshoe steel form was used for the crown. Exterior forms were 30-foot-long steel panels that extended just above the springline. This portion of the forms was first filled with concrete, then steel trusses with sloping panels were attached to the side forms, and concrete placement was resumed through the nonformed area at the crown.

Grouting. Grout pipes were installed throughout all supported sections of the tunnel. Contact grouting filled the voids behind the concrete lining for the entire length of supported tunnel. Average grout take for both the 6-foot - 6-inch pressure tunnel and the 9-foot - 6-inch walk-in conduit was 3½ sacks per linear foot of tunnel.

Consolidation grouting for the purpose of cutting off the flow of water in lenses parallel to the tunnel extended from Station 4+70 to Station 15+42 and was tied into the grout curtain of the Dam. Four hundred and thirty-five holes, 15 and 25 feet in depth, totaling 7,335 feet in length were used. The average grout take was 0.49 of a cubic foot of grout per linear foot of grout hole.

Installation of 60-Inch Steel Pipe. Forms for the concrete saddles (Figure 219) which support the 60-inch pipeline in the walk-in section were prefabricated in the contractor's carpenter shop. Reinforcing steel also was formed into cages for easier installation. Placement started at the upstream end and continued progressively downstream. Concrete was transported to the placement area by buggies and placed by hand shoveling.

A single railroad rail was installed on the pipe supports and fastened to the supports. The rail supported the special equipment carriage used to transport the 60-inch pipe and butterfly valve. The carriage was equipped with two flanged rail wheels on one side and two plastic-tired wheels on the opposite side. The plastic-tired wheels traveled along the walkway of the tunnel, and the railroad wheels guided the carriage along the railroad rail. The 60-inch conduit was installed in 20-foot - 10-inch sections that were loaded onto the special equipment carriage and hand-pushed to the placement position. Dry packing was inserted between the asbestos friction surface and the pipe supports as pipe was placed.

The prefabricated bifurcation unit, connecting the 60-inch conduit in the walk-in section to the 60-inch Del Valle Branch Pipeline, was encircled with straps welded to the reinforcing steel protruding through the invert slab to prevent flotation during concrete placement. The conduit trench was overexcavated from 2 to 9 inches and backfilled with selected bedding material. Spark gap tests were made of the coal-tar-coated exterior as well as the wrapped joints, and all defects were repaired prior to backfilling. A blind flange was bolted to the end of the Branch Pipeline at Station 3+35.

Intake Structure. The trench for the intake structure (Figure 220) was excavated on a 1½:1 slope and was 5 feet deep by 10 feet wide. The trench was excavated with a backhoe starting at the top. A dozer pushed the material down the slope where it was removed during foundation cleanup. Because the foundation was composed of highly fractured sandstone and shale, consolidation grouting was necessary to minimize the possibility of settlement and to prevent erosion of the foundation due to fluctuations of the water surface.

Figure 220. Sloping Intake Structure

The excavated surfaces were covered with 1 inch of gunite, anchor bars were placed in drilled holes (but not grouted), and consolidation grouting was started. While injecting consolidation grout, cross-flow into the anchor bar holes grouted many of the bars in place. The remaining bars were grouted in place before any concrete was placed. The first concrete was placed in the thrust block. Ten linear feet of 1¼-inch grout pipe was installed through the block for consolidation grouting in that area.

The slab was placed with a 5-foot-long by 7-foot-wide, weighted, steel panel used as a slip form. The form was pulled up the slope with a pneumatic hoist and was guided by steel channels welded to the No. 10 grouted anchor bars. The concrete was dumped and vibrated above the leading edge of the form. Four placements were required to complete the invert.

The inclined sections were monolithic placements. Walls were placed first with an hour lapse in time allowed to minimize shrinkage before placing the top slab. The heavy network of reinforcing steel within the forms necessitated use of ¾-inch maximum size aggregate in the concrete mix.

Hydraulic Testing. After the 42-inch fixed-cone dispersion valve and the 60-inch butterfly valves were installed, the entire system was tested hydraulically by filling the intake structure with water to elevation 700 feet. Leakage that developed at the flexible coupling expansion joint in the walk-in section of the conservation tunnel was repaired and the line was satisfactorily retested.

Testing Butterfly Valves. After all five 42-inch butterfly valves were installed (June 27, 1968), the operator and hydraulic lines were tested. A leakage test of each butterfly valve also was conducted by filling the space above the valve with 1 foot of water. Minor leaks resulting from improper seating were corrected.

Inlet Control Building. The conservation outlet works intake structure is operated from the inlet control building. The building is located above the intake structure on the conservation outlet works service road and is connected to the intake structure by the inlet control conduit.

The building rests on a concrete slab foundation and is of masonry block construction, with a metal roof deck and built-up, mopped-on, roof surface.

Machinery installed in the building includes a power package with hydraulic pumps; two 44-volt, 3-phase, 60-cycle, dripproof, electric motors; a 20-gallon, heated, hydraulic fluid reservoir; and a control panel.

Flood Control Outlet Works and Spillway Tunnel

Description. The 2,340-foot-long spillway tunnel and flood control outlet works are located in the right abutment of Del Valle Dam. The complex is composed of three sections: the 215-foot-long, 18-foot-diameter, pressure tunnel extending from the trashrack at the inlet to the gate chamber; the gate chamber and 141-foot-high shaft; and the main 2,004-foot-long 28-foot-diameter tunnel extending from the gate chamber downstream to the stilling basin. The entire system is lined with reinforced concrete.

Inlet. The abutment at the inlet portal was stripped to firm sandstone, the face of the tunnel portal was cut, and tunnel supports of the cut-and-cover section were placed. Prior to starting tunnel excavation, thirteen No. 18 bars were placed in holes drilled 30 feet horizontally into the abutment 1 foot - 8 inches outside and around the crown of the tunnel. They

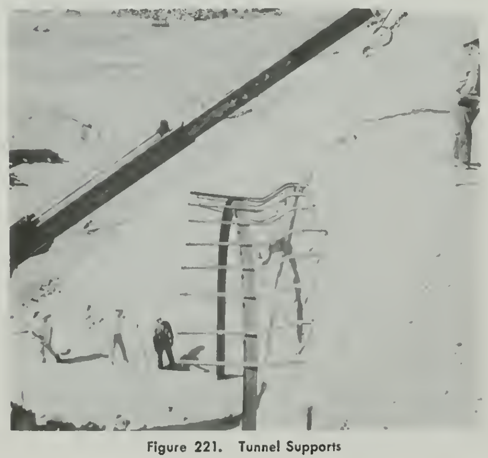

Figure 221. Tunnel Supports

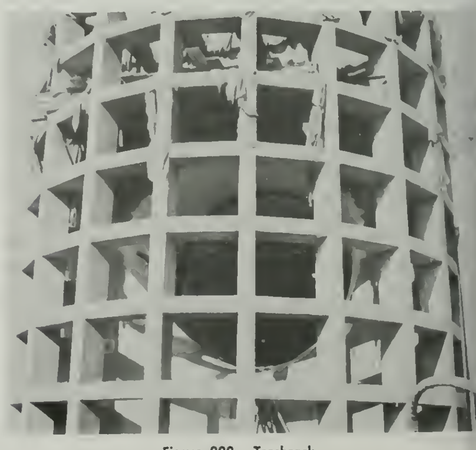

Figure 222. Trashrack

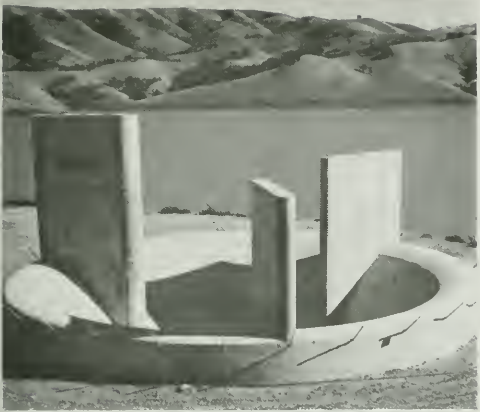

Figure 223. Spillway Crest

Figure 224. Spillway Shaft Excavation Support

were then pressure-grouted in place forming a protective shield in the abutment at the portal. By having these bars extend 5 feet out of the abutment and welding the first three structural steel supports of the cut-and-cover section of the tunnel portal to them, they acted as anchors and gave additional protection to the portal during the tunnel operations (Figure 221). Tunnel supports were placed for the full length of the portal protection structure and lagged solidly.

Pressure Tunnel. The pressure section was excavated by conventional drilling, shooting, and mucking methods. Steel posts and arches, with 4-inch timber lagging, supported the crown and walls while excavation of the tunnel was in progress. Concrete was placed in the pressure section, utilizing an 18-foot-diameter, full-circle, 24-foot-long, steel form. The form was split at the invert and hinged at the quarter points, allowing the lower halves to be raised or lowered by hydraulic jacks. The form was moved on an overhead rail trolley, raised into final position by jacks, and tied down to the structural steel posts by steel rods welded to the form. The ends of the form were bulkheaded against the rock sides with lumber. Concrete was placed with a pumpcrete machine through two 8-inch slicklines discharging at the 10 o'clock and 2 o'clock positions on the crown.

Contact grouting of the flood control outlet works was completed on January 9, 1968. Consolidation grouting was started at the completion of contact grouting and continued until completed on January 22. Consolidation grouting of the gate chamber started on January 23 and was completed on February 9, 1968. A portion of the pressure tunnel, near the portal, was grouted from the ground surface with good results.

Trashrack. The flood control trashrack is a reinforced-concrete structure 34 feet high composed of a 16-foot-radius, semicircular, grid frame with 2-foot-thick beams and columns and 4-foot-square openings (Figure 222). It is joined to the upstream end of the tunnel with a transition section and headwall.

The foundation of the trashrack structure was excavated to firm sandstone. Zone 3 material was placed and compacted to the required density and elevation. A 2-foot-thick, reinforced-concrete, footing slab then was placed and the trashrack constructed upon it. Concrete placements were made in 8-foot-high lifts using a mobile crane bucket. The concrete was cured by wrapping with burlap and sprinkling with water.

Spillway Crest, Shaft, and Gate Chamber. A 5¼-inch hole was drilled on the centerline of the vertical spillway shaft from the bench at approximate elevation 738 feet downward to the elevation of the gate chamber. A 1-inch steel cable was lowered through this hole and attached to a 6-foot-square metal platform. With this platform, an 8-foot-square shaft was raised to the surface at elevation 738 feet. The spillway shaft was sunk from this elevation. Holes were drilled, loaded, and shot on concentric circles. The material then was dozed into and dropped through the 8-foot-square shaft and removed through the spillway tunnel from the lower level.

A 4-foot-wide concrete collar with an inside diameter of 64 feet then was placed around the perimeter of the excavation at elevation 734.5 feet. A concrete crest structure (Figure 223) was placed on top of the collar at this elevation. The collar was used to anchor the structural-steel shaft supports. Trimming of the shaft then proceeded from the top. Successive structural steel rings (Figure 224) were suspended, lagged, and braced as the shaft was sunk to elevation 652 feet. Here, at the elevation of the crown of the gate chamber, a 33-inch, wide-flange, steel, ring beam was installed. Excavation of the gate chamber and elbow then commenced. Arch sets bracing the walls and crown of the gate chamber were installed and welded to the ridge beam. Tunnel muck was removed through the spillway tunnel.

Concrete placement in the gate chamber, elbow, and shaft was made with four 12-inch and two 8-inch steel pipe trunks spaced at intervals around the circular spillway shaft. The trunks were positioned between the inner and outer layer of reinforcing steel. Each trunk was attached to an individual hopper at the top of the shaft. Concrete was dumped into the hoppers and then dropped through the pipes into final position. A concrete mix with $1\frac{1}{2}$-inch maximum size aggregate and a 4- to $5\frac{1}{2}$-inch slump was used. The mix fell a maximum of 5 feet from the end of the trunk. No gates were used and no segregation was noted. Concrete placement in the shaft was made in three lifts.

One foot of first-stage concrete was removed from the invert of the transition between the gate chamber and the tunnel. A pneumatic impact hammer mounted on a diesel wheel tractor with backhoe attachment was used to remove the bulk of the concrete. Fine grading was done with hand-operated pavement breakers. In preparing the joint for second-stage concrete, the top 2 inches of first-stage concrete was sawed.

A prefabricated form section was lowered down the spillway shaft and skidded into place on a heavy timber track placed on pipe rollers. To prevent this form from floating, No. 8 reinforcing bars were grouted into the invert of the gate chamber and welded to steel plates secured to the main structural members of the form. She-bolts in the sides and crown of the gate chamber were attached to the form in a similar manner. Interior timber bracing running from crown to invert and from side to side also was installed.

The drilling and grouting of the spillway shaft was accomplished from a platform suspended from a crane. Horizontally oriented rings with 8 to 10 grout holes each were located at elevations 727, 717, 707, 697, and 673 feet. A ring of 19 grout holes was located at elevation 662 feet and a ring of 23 grout holes at elevation 652 feet. In addition, 18 grout holes of various orientations were located near tunnel Station 9+82, in the elbow area.

Downstream Portal. With one exception, the downstream portal of the spillway tunnel was constructed in the same manner as the upstream portal of the flood control outlet tunnel; that is, as soon as the lagging of the umbrella section was completed, structural backfill was placed from the footings to the springline.

Spillway Tunnel. The tunnel was driven by the top heading and bench method. First, all material was removed between the crown and 2 feet below springline from the outlet portal to the gate chamber, a distance of 1,893 feet. Structural steel sets were placed and lagged as the tunnel advanced. The material below springline then was removed commencing at the outlet portal. As the bench was removed, the steel supports were extended to the invert elevation. This extension was made by temporarily supporting the top heading posts and arches with a 6-inch by 18-inch by 30-foot-long soldier beam.

Concrete tunnel lining was placed in two phases: the invert and crown. The invert was formed in 28-foot sections with an 8-foot-long, 90-degree-arc, metal slip form. Guide rails 40 feet long allowed the 8-foot slip form to clear the end of each 28-foot section. The form was pulled along the guide rails by two air winches.

Concrete was placed in the invert by a swivel belt conveyor system discharging into the forms and supplied directly by transit mix trucks. Concrete first was placed to the top of the reinforcing steel mat and vibrated. The next lift was placed over the mat and forced ahead of the leading edge of the slip form. Finishers worked from a wooden platform spanning the slip form rails. Finishing started as soon as the concrete cleared the form.

Crown placements were formed with two identical 28-foot-long steel forms (Figure 225). The forms were moved on a rail-mounted jumbo and positioned to line and grade with hydraulic jacks and an electrically operated "Tugger" hoist.

Crown concrete was placed by two pumpcrete machines discharging through 8-inch slicklines at the 2 o'clock and 10 o'clock positions on the crown. The concrete was consolidated with form vibrators mounted on the form ribs at spacings of 7 feet longitudinally and 6 feet circumferentially. Fourteen to sixteen vibrators were used for the 56-foot-long section. Two and one-half-inch internal vibrators also were used between the form face and rock walls.

Figure 225. Tunnel Crown Placement Forms

Complete filling of the crown was achieved by inserting 4-inch pipes through the lining at each end of the arch form and injecting sand-cement mortar after all concrete was placed. A total of 310.5 cubic yards of mortar was injected behind the arch section from Station 10+36 to Station 29+17. Mortar was injected prior to tunnel contact grouting.

Contact grouting commenced near the downstream portal and progressed upstream. Leakage at construction joints and grout flow to other holes occurred as far as 60 feet from the injection hole. The relatively high grout take of almost ten sacks of cement per linear foot of tunnel is probably attributable to the excessive fracturing and overbreak along the tunnel crown during the driving of the tunnel.

The initial spacing and location of consolidation grout rings in the spillway tunnel and shaft were based on recommendations of the project geologist. Several additional grout rings were installed in areas of abrupt change in bedding attitude and areas of excessive overbreak. Two multiring grout curtains were located in the prolongation of the dam axis. Each curtain consisted of six rings of 16 holes each, with 5-foot spacing between rings.

Gates. Flow through the flood control outlet works is controlled by four hydraulically operated slide gates located in the gate chamber.

The cylinders, bonnets, and frames were lowered down the spillway shaft by a mobile crane onto a metal cart which transported them to the final location.

Prior to positioning the gate bodies, the hydraulic cylinders were hoisted to within 4 feet of the gate chamber crown with individual 15-ton coffin hoists and tied off with 1-inch cable safety straps to the pad eyes embedded in the crown. The cylinders remained suspended until the concrete deck of the gate chamber was placed. They then were lowered into position on top of the frames.

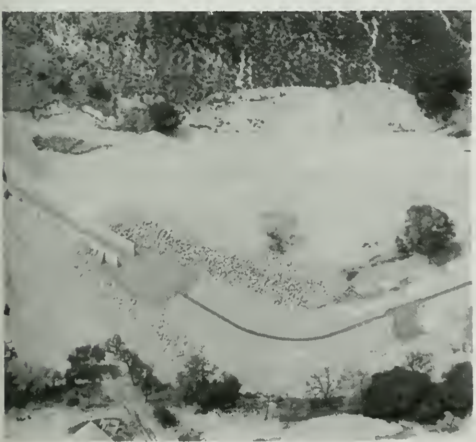

Figure 226. Stilling Basin

Gate Chamber Access. Entrance to the flood control outlet works gate chamber is through a concrete-lined shaft and tunnel. The upper level of the shaft terminates at elevation 773.5 feet in the flood control outlet works control building located near the north end of the right abutment access road.

The contractor first placed a 12-foot-diameter concrete ring at the shaft location and started excavation by hand. As the shaft was sunk, loose material was removed by hand loading into a clamshell bucket. Conventional drilling, shooting, structural-steel ring placing, and lagging methods were used. Waste material was loaded into dump trucks and hauled to a Zone 4 stockpile. The tunnel invert, elevation 640 feet, was reached on October 13, 1966. The tunnel then was excavated for a distance of 9 feet. The first two structural steel rings were set and lagged. Excavation of the tunnel then was started from within the gate chamber area. A shuttle buggy was used to remove material from the tunnel. Material was loaded into the shuttle buggy with an air mucker, dumped into the gate chamber, and hauled out through the flood control tunnel with a front-end loader.

The first concrete was placed in the vertical elbow section to elevation 652 feet, or 12 feet from the shaft base. Concrete was placed up the shaft in 20-foot lifts. A 9-foot-diameter, full-circle, steel form was lowered into place wth a truck crane and secured by welding to the structural steel rings. Because of the limited work area, all reinforcing steel for the entire 130 feet of shaft was installed prior to the first concrete placement. Concrete was placed by mobile crane bucket.

After completion of tunnel concreting, a concrete walkway was formed and placed along the length of the tunnel.

The same form used in the shaft was used in the tunnel. Concrete in both the tunnel and horizontal elbow section was delivered and placed by a double-ram pumpcrete machine positioned just outside of the flood control intake.

Grout pipes were installed as the concrete was placed, and grouting of the access shaft and tunnel started on September 25, 1967 and was completed on October 12, 1967. Grout takes generally were low.

Stilling Basin. The stilling basin (Figure 226) for the spillway and flood control outlet tunnel is a reinforced-concrete structure 267 feet long, 30 feet wide, and 38 feet high. The basin is supported on a 62-foot-wide, drained, concrete footing. The stilling basin foundation was soft sandstone partially saturated with water. Continual seepage during construction of the basin drains made installation difficult. The sandstone tended to disintegrate and become suspended in the water. The sandy water filled the sumps and plugged the dewatering pumps which had to be changed frequently.

The stilling basin was excavated during July 1967. Concrete placement started on August 10, 1967 and was completed on December 14, 1967. Construction of the basin drains proceeded simultaneously with the concreting. The final concrete placements were deferred until all construction equipment was removed from the spillway tunnel.

Wall sections were placed in two lifts, each 25 feet in length. A 6-inch polyvinylchloride waterstop was placed in all construction joints. The floor slab was concreted in alternate panels staggered on either side of the centerline. A 6-inch waterstop also was installed in the slab construction joints.

All concrete in the structure was placed by a mobile crane bucket. The floor slab was cured by flooding and the walls by draping with carpets kept moist by soaker hoses.

Return Channel. The channel from the end of the stilling basin flip bucket extends approximately 800 feet downstream in line with the tunnel from where it turns in a northerly direction and merges with Arroyo Del Valle. The upstream portion of the channel is confined between dikes constructed of Zone 3 material, cobbles, and riprap. Downstream from the turn, the channel is confined between banks excavated in stream gravels. The elevation of the channel invert is 535.6 feet at the discharge end of the stilling basin. Grade of the invert rises to elevation 540 feet at its intersection with the original creek channel.

A small 15-foot-wide channel is located along the center of the return channel for conveying reservoir releases to supplement the streamflow.

The excavation, placement of cobbles on the invert, and placement and compaction of Zone 3 material in the dikes proceeded without difficulty. Placement of riprap in the return channel, however, was a slow and tedious process. Material was dumped as close as possible to the final location. Each rock then was picked up individually with an orange-peel bucket and deposited in final position.

Concrete Production

Concrete for the major portion of the project was supplied from a portable batch plant operated by a subcontractor.

The prefabricated plant was automatic and interlocked, and 12 different mixes could be set on the plant controls for immediate selection. The mixer was a 4-cubic-yard tilt mixer. Additional equipment which was part of the plant included a storage building for block ice and a crusher, pozzolan and cement bulk storage silos, and an overhead shuttle conveyor system for transporting truck-loaded aggregates to stockpiles. There also was an underground reclaiming tunnel and conveyor system to feed aggregates from the stockpiles through washing and finishing screens to the weigh hoppers and finally to the tilt mixer.

Mixed concrete was transported from the plant to the placement locations with three mixer trucks equipped with tilt mixers.

The remainder of the concrete was supplied by a commercial plant in Pleasanton, 8 miles from the Dam site.

Reservoir Clearing

Clearing operations started on the hillside above the flood control inlet and outlet portals, moved to the dam foundation and abutments, and then progressed upstream to the borrow areas and the reservoir.

Heavy grubbing was done with three large dozers equipped with rippers and brush blades. Small brush was removed by laborers using power saws and machetes. Stumps were removed with the dozers and later buried in designated waste areas. Brush and burnable logs were piled and burned. Small logs were removed and piled.

Closure

The permanent plug closing the diversion opening in the conservation outlet works was placed on May 16, 1968, without incident. All work was completed on September 17, 1968 and, by the end of the year, the water was 23 feet below the invert of the flood control outlet works, the reservoir's lowest outlet. The first flood control releases were made on February 25, 1969, and since then the reservoir has been operated within the planned range.

BIBLIOGRAPHY

California Department of Water Resources, Bulletin No. 117-2, "Del Valle Reservoir: Recreation Development Plan", December 1966.

_______, Bulletin No. 132-72, "The California State Water Project in 1972", June 1972.

_______, Bulletin No. 132-72C, "The California State Water Project SUMMARY: nineteen-seventy-one", Appendix C.

_______, Specification No. 66-01, "Specifications Bid and Contract for Del Valle Dam and Reservoir, State Water Facilities, South Bay Aqueduct, Del Valle Division, Alameda County, California", 1966.

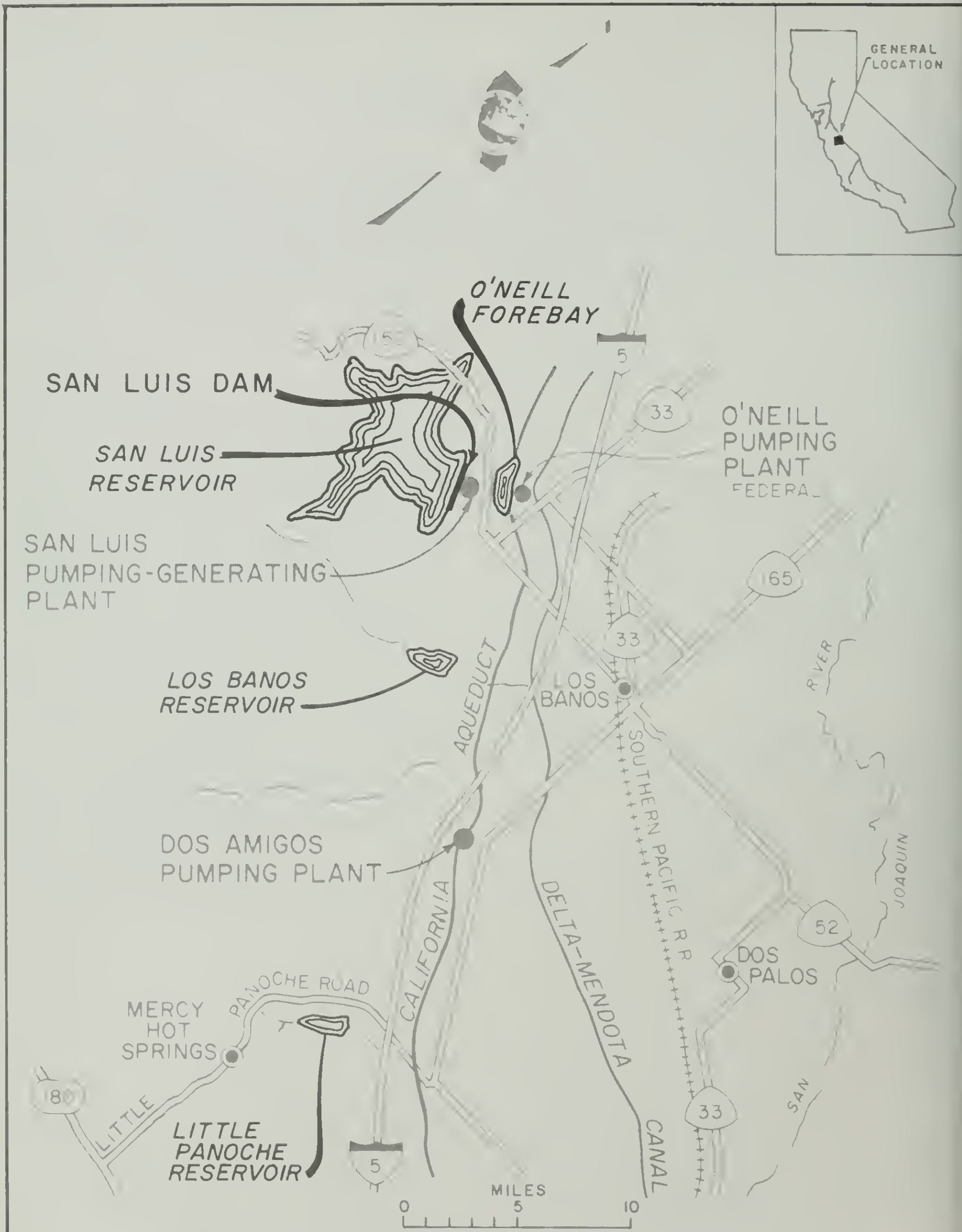

Figure 227. Location Map—San Luis Joint-Use Storage Facilities

CHAPTER XI. SAN LUIS JOINT-USE STORAGE FACILITIES

General

Description and Location

The San Luis Joint-Use Facilities serve the U.S. Bureau of Reclamation's San Luis Unit of the Federal Central Valley Project and the State Water Project (Figure 227). They extend along the west side of the San Joaquin Valley from O'Neill Forebay to Kettleman City, about 106 miles. The joint-use storage facilities consist of San Luis Dam and Reservoir (Figure 228), O'Neill Dam and Forebay, Los Banos Detention Dam and Reservoir, and Little Panoche Detention Dam and Reservoir (O'Neill Dam and Forebay formerly were identified as San Luis Forebay Dam and Forebay).

The route for conveying project water to state service areas in the San Joaquin Valley and Southern California is through the 500,000-acre federal San Luis service area on the west side of the San Joaquin Valley.

Figure 228. Aerial View—San Luis Dam and Reservoir

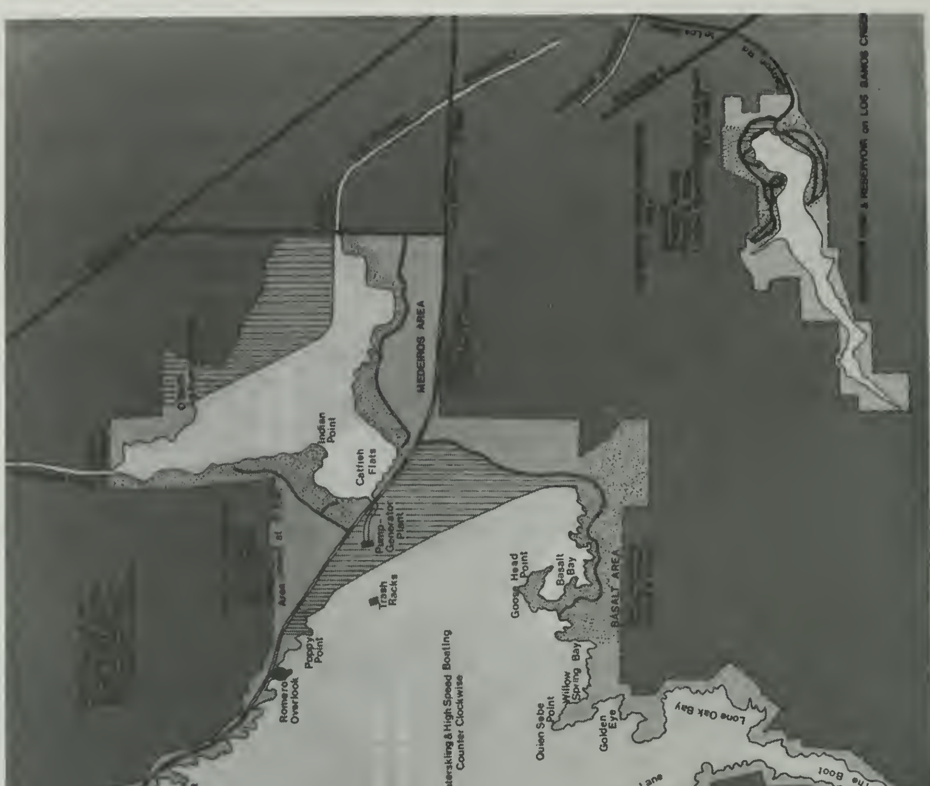

Figure 229. San Luis Reservoir Recreation Areas

The San Luis Unit of the Central Valley Project was authorized by Act of Congress, PL 86-488, which became law on June 3, 1960 (74 Stat. 156). Each project required development of the San Luis Dam site, west of Los Banos, for storage of surplus flows pumped from the Sacramento-San Joaquin Delta. Therefore, the best development for California and the United States was to integrate the storage, pumping, and conveyance facilities for coordinated operation. An agreement between the State and the United States was entered into on December 30, 1961 to achieve those results.

The agreement provided that the Bureau of Reclamation would design and construct the Joint-Use Facilities. It was agreed that cost sharing would be on the basis of approximately 55% state and 45% federal. The State was granted responsibility for operation and maintenance under the same approximate cost-sharing formula.

The Bureau of Reclamation publishes reports on its projects, similar to the Department of Water Resources' Bulletin 200. Reports covering the Joint-Use Facilities are or will become available in the near future. Accordingly, this chapter is intended to provide only sufficient information for continuity in Bulletin 200 coverage of the State Water Project.

San Luis Dam is located at the base of the foothills on the west side of the San Joaquin Valley in Merced County. It is 12 miles west of the City of Los Banos on San Luis Creek and 2 miles west of O'Neill Dam. Several highways traverse the locality. Located to the east are Interstate Highway 5 and State Highway 33.

Relocated State Highway 152 extends through the site between San Luis and O'Neill Dams and across the intake channel leading to the San Luis Pumping-Generating Plant. This highway connects Los Banos and the Gilroy-Hollister area over Pacheco Pass. Los Banos Detention Dam is 7 miles southwest of the City of Los Banos in Merced County. The nearest highway is Interstate 5, about 1 mile to the east of the Dam site. Little Panoche Detention Dam is located in Fresno County 20 miles southwest of the City of Los Banos. The nearest major highway is Interstate 5, about 3 miles to the east of the Dam site. The Dam is located at a stream constriction about 2 miles west of the San Joaquin Valley floor.

Purpose

San Luis Reservoir and Lake Oroville are the two key conservation features of the State Water Project. San Luis Reservoir provides offstream storage for excess winter and spring flows diverted from the Sacramento-San Joaquin Delta. In periods of excess runoff, water is pumped into San Luis Reservoir from O'Neill Forebay via San Luis Pumping-Generating Plant. San Luis Reservoir is sized to provide seasonal carryover storage. Hydroelectric power generation, on a nondependable schedule, is a project benefit but only an incidental purpose. There are extensive recreational developments around the reservoirs, as shown on Figure 229, as well as fish and wildlife benefits.

Los Banos Detention Dam provides flood protection for San Luis Canal, Delta-Mendota Canal, City of Los Banos, and other downstream developments. In addition, a 470-acre lake is provided for recreation.

San Luis Dam is a 385-foot-high 77,645,000-cubic-yard embankment with a crest length of 18,600 feet. The gross capacity of San Luis Reservoir is 2,038,771 acre-feet of which 1,067,908 acre-feet are allocated to the State. The spillway consists of a glory-hole inlet, shaft, conduit, chute, and stilling basin. San Luis Pumping-Generating Plant serves as the inlet-outlet works for the Reservoir. Four tunnels near the left abutment connect the plant to an inlet-outlet structure in the Reservoir. A control shaft and reservoir intake for the Pacheco tunnel, a future outlet to the west, were constructed to serve the San Felipe Division of the Central Valley Project. This tunnel stub is approximately 2 miles long.

O'Neill Forebay, with a gross storage capacity of 56,426 acre-feet, has several inlets and outlets. San Luis Pumping-Generating Plant functions as both an inlet and an outlet. The California Aqueduct entering from the north and O'Neill Pumping Plant on the left abutment of O'Neill Dam serve as inlets. Finally, the San Luis Canal reach of the California Aqueduct starts on the southeast edge of the Forebay. O'Neill Dam contains 3,000,000 cubic yards of embankment. It has a crest length of 14,350 feet and a maximum height of 88 feet. The forebay spillway is a glory-hole inlet discharging into a cut-and-cover conduit.

Los Banos Detention Dam is a 2,100,000-cubic-yard embankment with a structural height of 167 feet and crest length of 1,370 feet. The spillway is a concrete chute located on the left abutment. The outlet works consists of a vertical intake tower, a tunnel with a gate chamber, and a discharge line with hydraulically operated slide gates. Los Banos Reservoir has a gross capacity of 34,562 acre-feet.

Little Panoche Detention Dam is a 152-foot-high embankment with a crest length of 1,440 feet. The embankment volume is 1,210,000 cubic yards. Parallel cut-and-cover conduits for the spillway and outlet works are located under the embankment on the right abutment. The spillway has a glory-hole inlet, while the intake to the outlet works is a vertical tower. Little Panoche Reservoir has a gross capacity of 13,236 acre-feet.

Tables 27 through 30 are statistical summaries of the dams and reservoirs.

Little Panoche Detention Dam provides flood protection for San Luis Canal, Delta-Mendota Canal, and other downstream developments.

TABLE 27. Statistical Summary of San Luis Dam and Reservoir

SAN LUIS DAM

Type: Zoned earth and rockfill

Crest elevation	554 feet
Crest width	30 feet
Crest length	18,600 feet
Streambed elevation at dam axis	241 feet
Lowest foundation elevation	169 feet
Structural height above foundation	385 feet
Embankment volume	77,645,000 cubic yards
Freeboard above spillway crest	10.1 feet
Freeboard, maximum operating surface	11.2 feet
Freeboard, maximum probable flood	8.2 feet

SAN LUIS RESERVOIR

Storage at spillway crest elevation	2,038,771 acre-feet
Maximum operating storage	2,025,795 acre-feet
Minimum operating storage	79,200 acre-feet
Dead pool storage	8 acre-feet
Maximum operating surface elevation	542.8 feet
Minimum operating surface elevation	326 feet
Dead pool surface elevation	281.1 feet
Shoreline, spillway crest elevation	65 miles
Surface area, spillway crest elevation	12,700 acres
Surface area, maximum operating elevation	12,520 acres
Surface area, minimum operating elevation	3,600 acres

SPILLWAY

Type: Glory hole with reinforced-concrete conduit and stilling basin

Crest elevation	543.9 feet
Crest length	95.2 feet
Crest diameter	30.3 feet
Conduit diameter	9.5 feet
Peak maximum probable routed outflow	1,030 cubic feet per second
Maximum surface elevation	545.8 feet

INLET-OUTLET

San Luis Pumping-Generating Plant: Four vertical intake towers each with a 23-foot - 3½-inch-wide by 23-foot - 7-inch-high roller gate shutoff and a 24-foot - 6-inch-wide by 29-foot-high emergency bulkhead gate in tandem

Maximum generating release	13,120 cubic feet per second
Pumping capacity	11,000 cubic feet per second

OUTLET

Pacheco Tunnel

Length (initial contract)	1.8 miles
Length (ultimate)	10.3 miles
Diameter	13 feet
Capacity (when completed)	670 cubic feet per second

TABLE 28. Statistical Summary of O'Neill Dam and Forebay

O'NEILL DAM

Type: Homogeneous earthfill

Crest elevation	233 feet
Crest width	30 feet
Crest length	14,350 feet
Streambed elevation at dam axis	167 feet
Lowest foundation elevation	145 feet
Structural height above foundation	88 feet
Embankment volume	3,000,000 cubic yards
Freeboard above spillway crest	8 feet
Freeboard, maximum operating surface	9 feet
Freeboard, maximum probable flood	5 feet

O'NEILL FOREBAY

Storage at spillway crest elevation	56,426 acre-feet
Maximum operating storage	53,730 acre-feet
Minimum operating storage	35,700 acre-feet
Dead pool storage	10,220 acre-feet
Maximum operating surface elevation	224 feet
Minimum operating surface elevation	217 feet
Dead pool surface elevation	202 feet
Shoreline, spillway crest elevation	12 miles
Surface area, spillway crest elevation	2,700 acres
Surface area, maximum operating elevation	2,670 acres
Surface area, minimum operating elevation	2,450 acres

SPILLWAY

Type: Glory hole with reinforced-concrete conduit and stilling basin

Crest elevation	225 feet
Crest length	184.4 feet
Crest diameter	59.0 feet
Conduit diameter	11.8 feet
Peak maximum probable routed outflow	3,250 cubic feet per second
Maximum surface elevation	228 feet

INLET

North San Joaquin Division of California Aqueduct

Capacity	10,000 cubic feet per second

INLET-OUTLET

San Luis Pumping-Generating Plant

Maximum generating release	13,120 cubic feet per second
Pumping capacity	11,000 cubic feet per second

O'Neill Pumping-Generating Plant

Maximum generating release	3,600 cubic feet per second
Pumping capacity	4,200 cubic feet per second

OUTLET

San Luis Canal

Capacity	13,100 cubic feet per second

TABLE 29. Statistical Summary of Los Banos Detention Dam and Reservoir

LOS BANOS DETENTION DAM

Type: Zoned earthfill

Crest elevation	384 feet
Crest width	30 feet
Crest length	1,370 feet
Streambed elevation at dam axis	228 feet
Lowest foundation elevation	217 feet
Structural height above foundation	167 feet
Embankment volume	2,100,000 cubic yards
Freeboard above spillway crest	30.5 feet
Freeboard, maximum recreation surface	56.2 feet
Freeboard, maximum probable flood	5.8 feet

LOS BANOS RESERVOIR

Storage at spillway crest elevation	34,562 acre-feet
Maximum recreation storage	20,600 acre-feet
Dead pool storage	8,000 acre-feet
Maximum recreation surface elevation	327.8 feet
Dead pool surface elevation	295.2 feet
Shoreline, spillway crest elevation	12 miles
Surface area, spillway crest elevation	623 acres
Surface area, maximum recreation elevation	500 acres
Surface area, minimum recreation elevation	380 acres

SPILLWAY

Type: Ungated ogee crest with lined chute and stilling basin

Crest elevation	353.5 feet
Crest length	20 feet
Peak maximum probable routed outflow	8,600 cubic feet per second
Maximum surface elevation	378.2 feet

OUTLET WORKS

Type: Lined tunnel under left abutment, valve chamber at midpoint—discharge into stilling basin

Diameter: Upstream of valve chamber, 78-inch pressure conduit—downstream, 70-inch steel conduit in a 126-inch concrete horseshoe tunnel

Intake structure: Uncontrolled tower

Control: Downstream control, two 42-inch-square, high-pressure, slide gates—guard valve, one 60-inch-wide by 72-inch-high, high-pressure, slide gate in valve chamber

Capacity	1,255 cubic feet per second

TABLE 30. Statistical Summary of Little Panoche Detention Dam and Reservoir

LITTLE PANOCHE DETENTION DAM

Type: Zoned earthfill

Crest elevation	676 feet
Crest width	30 feet
Crest length	1,440 feet
Streambed elevation at dam axis	556 feet
Lowest foundation elevation	524 feet
Structural height above foundation	152 feet
Embankment volume	1,210,000 cubic yards
Freeboard above spillway crest	34.5 feet
Freeboard, maximum probable flood	5.6 feet

LITTLE PANOCHE RESERVOIR

Storage at maximum probable flood	13,236 acre-feet
Dead pool storage	315 acre-feet
Dead pool surface elevation	590 feet
Shoreline, maximum probable flood	10 miles
Surface area, maximum probable flood	354 acres
Surface area, dead pool elevation	30 acres

SPILLWAY

Type: Glory hole with reinforced-concrete conduit and stilling basin

Crest elevation	641.5 feet
Crest length	88.5 feet
Crest diameter	28.5 feet
Conduit diameter	9.5 feet
Peak maximum probable routed outflow	3,200 cubic feet per second
Maximum surface elevation	670.4 feet

OUTLET WORKS

Type: Reinforced-concrete conduit beneath dam at base of right abutment—discharge into stilling basin

Diameter: 9 feet - 6 inches

Intake structure: Uncontrolled tower

Control: None

Capacity	1,040 cubic feet per second

Chronology

Final design of the San Luis features started in 1961. The first construction contract for relocation of State Highway 152 was awarded in August 1962. The official San Luis Unit ground-breaking ceremonies were held on August 18, 1962. The event was highlighted by the presence of President John F. Kennedy (Figure 230). The contract for construction of San Luis and O'Neill Dams and the Pumping-Generating Plant was awarded on January 8, 1963.

Construction of San Luis and O'Neill Dams was completed on August 4, 1967. Water was first pumped to storage in San Luis Reservoir on April 12, 1967. Essentially, it was filled for the first time on May 31, 1969.

The detention dams were completed and put into operation in 1966.

Operation

Primarily during winter and early spring, surplus water in the Sacramento-San Joaquin Delta is pumped into the North San Joaquin Division of the California Aqueduct and flows by gravity into O'Neill Forebay. The federal diversion is pumped into the Forebay from the Delta-Mendota Canal by O'Neill Pumping Plant. From the Forebay, water either flows south through the San Luis Canal or flows through the intake channel to the San Luis Pumping-Generating Plant, where it is pumped into San Luis Reservoir. The maximum static head on the Pumping-Generating Plant is 327 feet.

During the remainder of the year, when downstream water demand is greater than direct Sacramento-San Joaquin Delta diversions through the California Aqueduct and the Delta-Mendota Canal, water is released from San Luis Reservoir to augment the flow. Power is generated as the flow passes through the Pumping-Generating Plant into the Forebay. Additional power for the Federal Central Valley Project is generated by reversal of flow through O'Neill Pumping Plant from the Forebay to the Delta-Mendota Canal.

The reservoirs are operated to minimize electrical energy costs for pumping and to deliver water on demand. They also are operated at a normal maximum operating level 1 foot below the maximum storage elevation to prevent loss of water through the spillways by wave overtopping.

Under the coordinated operation to satisfy the future maximum demands on the state and federal projects, there will be large storage withdrawals from San Luis Reservoir. The hydraulic machinery of the Pumping-Generating Plant was designed to function with the reservoir level reduced to elevation 326 feet, a fluctuation of 218 feet.

Operational limitations on storage in Los Banos Reservoir are shown on the flood control diagram (Figure 231). Between September 20 and March 15 of any year, 14,000 acre-feet of space will be maintained insofar as possible for control of flood waters under the following conditions:

1. Releases from Los Banos Reservoir shall be restricted, insofar as possible, to flows which will not exceed 1,000 cubic feet per second (cfs) in Los Banos Creek below Los Banos Detention Dam.

2. The rate of change (increase or decrease) of flows in Los Banos Creek below the Dam shall not exceed 200 cfs during any two-hour period.

During the remainder of the year, inflow from Los Banos Creek may be used to raise the recreation pool above elevation 327.8 feet.

The outlet works for Little Panoche Reservoir is ungated. Water is stored behind the Dam above current dead storage only during the period that the inflow from Little Panoche Creek exceeds the capacity of the outlet works.

Figure 230. President John F. Kennedy and Governor Edmund G. Brown Pushing Plungers to Detonate Explosives for Ground Breaking

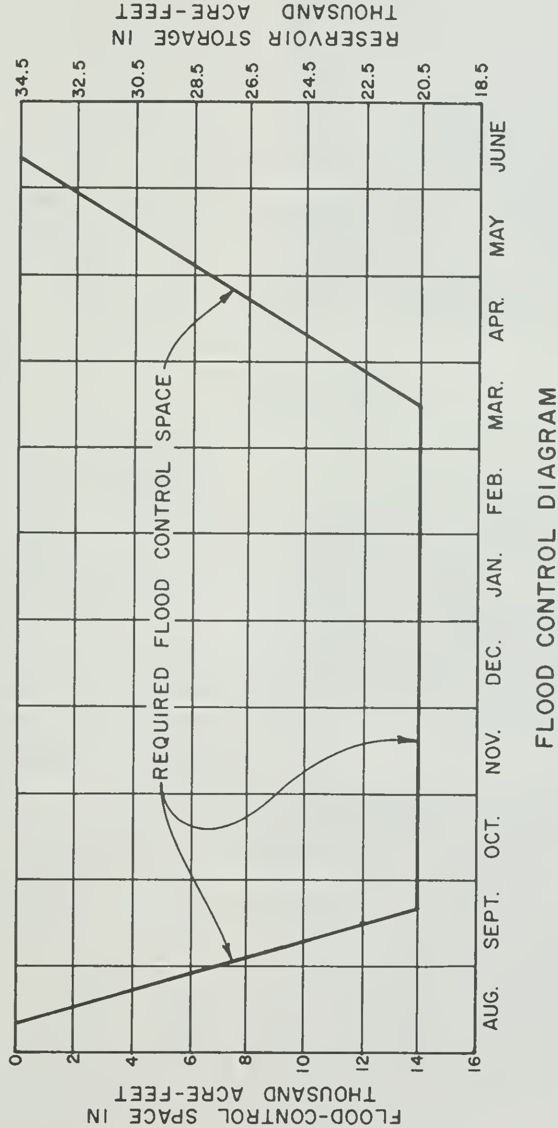

Figure 231. Flood Diagram—Los Bonos Reservoir

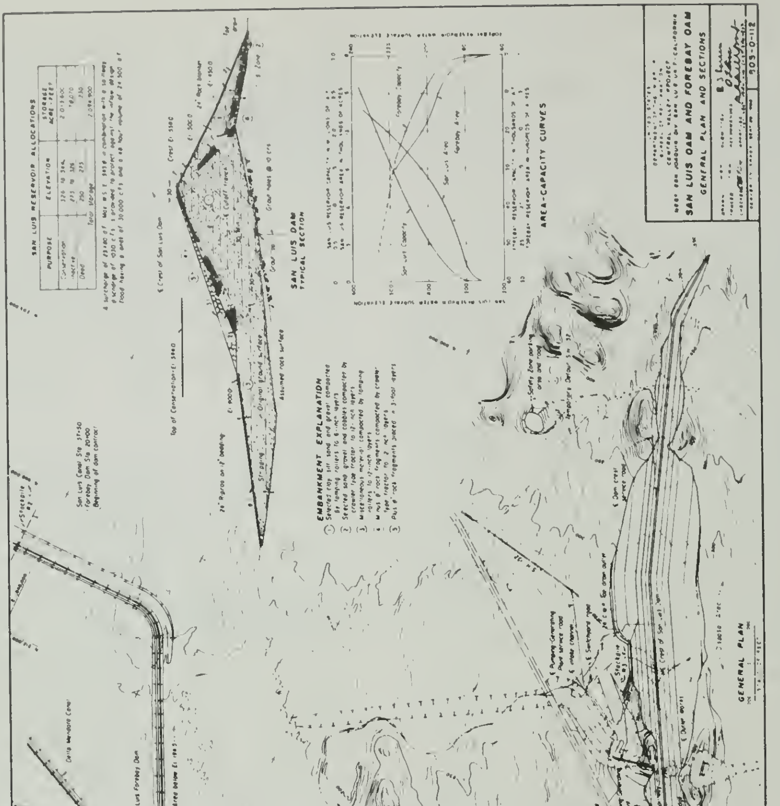

Figure 232. General Plan and Sections of San Luis Dam and O'Neill Forebay

Design

Dams

San Luis. The 385-foot-high San Luis Dam is a zoned earthfill (Figure 232) with a rock face section on a 3:1 slope upstream above elevation 400 feet, and an 8:1 slope of random material below elevation 400 feet, steepening to 2½:1 below elevation 310 feet (Figure 233). The downstream slope is 2:1 above and 2½:1 below elevation 450 feet, flattening to a 6:1 slope below elevation 400 feet. The Dam incorporates a chimney drain below elevation 500 feet connected to a foundation blanket drain. This drain system terminates in toe drains. The dam crest is at elevation 554 feet.

The foundation for San Luis Dam, in the channel of San Luis Creek, consisted of up to 160 feet of alluvium. Excavation for the Dam was in excess of 100 feet in depth into the alluvial deposits terminating on a competent, firm, clayey, gravel formation extending to Panoche bedrock.

The double foundation trenches involved the removal of 5,570,000 cubic yards of material. The foundation trenches provide for a proper gradient to control seepage underflow, a foundation capable of supporting the superimposed loads, and an embankment section capable of resisting static and dynamic sliding forces. The abutments mostly consist of interbedded shale and sandstone units. There are conglomerates in the lower left abutment which were penetrated by the four inlet-outlet tunnels.

The rock foundation was grouted with primary holes at 10-foot centers. Where grout takes were heavy, the spacing was split to obtain satisfactory results. The curtain extended to a normal depth of 160 feet with a maximum depth of 300 feet. The deepest holes were drilled to intercept faults in the outlet works tunnel alignments to improve conditions for their excavation. The average grout take was nearly 0.4 of a cubic foot of cement per foot of hole.

Embankment materials for Zone 1 were the alluvial valley fills from within both the San Luis and O'Neill reservoir basins. The largest portion was excavated by a large wheel-type loader (Figure 234). The sand, gravel, and cobbles of Zone 2 were obtained from channel deposits along San Luis Creek, beginning upstream of San Luis Reservoir. Zone 3 consisted of miscellaneous material, including that removed in foundation excavation which was moved directly to placement in the embankment. Material for rock fill, bedding, and riprap was quarried from Basalt Hill, high on the right abutment. It was separated at the 8-inch size with some of the material passed through a crusher to manufacture bedding for riprap (Figure 235). Zone 4 consists of the fraction less than 8 inches and Zone 5 that fraction greater than 8 inches.

Figure 233. Placing and Compacting Embankment—San Luis Dam

Figure 234. Wheel Excavator Cutting on 50-Foot-High Face

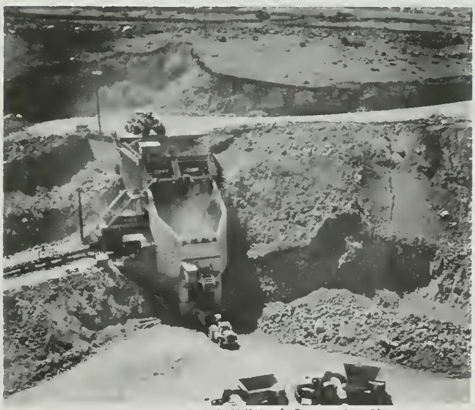

Figure 235. Basalt Hill Rock Separation Plant

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O'Neill Forebay. This 88-foot-high dam is homogeneous with riprap protection from 2 feet below minimum pool to the dam crest on the upstream face. The upstream slope is $2\frac{1}{2}$:1 above elevation 215 feet and $3\frac{1}{2}$:1 below. The downstream slope is constant at 2:1. The downstream section has a foundation blanket drain and a toe drain.

The Dam is founded entirely on alluvium except for the abutment at O'Neill Pumping Plant. Exposed gravels in San Luis Creek upstream of the Dam were blanketed with 5 feet of impervious material. Grouting was done at the abutment, but seepage control otherwise is obtained by the cutoff trench. The embankment materials are from the same sources as for San Luis Dam.

Los Banos Detention. Figure 236 shows the plan and sections of 167-foot-high Los Banos Detention Dam.

The foundation consists of shales, sandstones, and conglomerates. The channel of Los Banos Creek contained alluvial deposits which were removed under Zone 1 of the Dam.

Foundation preparation included construction of a grout cap. Grout holes were drilled at 10-foot centers to a maximum depth of 110 feet. The grout take was small, averaging 0.19 of a cubic foot of cement per foot of hole.

The embankment contains three zones: Zone 1, the core, contains a clay, silt, and gravelly material to 5 inches maximum size; Zone 2 is stream sand and gravel from the reservoir area; and Zone 3 is miscellaneous material from required excavations and suitable site stripping. Riprap was obtained from a basalt quarry about 17 miles west of the project.

Little Panoche Detention. Figure 237 shows the plan and section of Little Panoche Detention Dam.

Foundation for the Dam is the Panoche formation consisting of sandstone, siltstone, and clay shale. A cutoff trench was excavated to fresh formation. To prevent air slaking, the final 1 foot of excavation for the Dam and concrete structures was deferred until covering could proceed expeditiously.

The foundation was not grouted. Grouting was unnecessary because water will not be in reservoir retention longer than five days at a time, which is not long enough to allow seepage to become established. Furthermore, siltation is expected to form a bottom blanket.

The embankment contains two zones. The majority of Zone 1 embankment was obtained from selected material from required excavation, with the balance taken from the reservoir area.

Zone 2 embankment generally was that material which was unsuitable for Zone 1. Mostly, it is sand, gravel, cobbles, and fragments of sandstone and, in the reservoir borrow area, normally was overlain by Zone 1.

The contractor chose to develop a quarry on private land southeast of Ortigalita Peak to obtain riprap. It was in greenstone of the Franciscan formation.

Inlets, Outlets, and Spillways

San Luis Reservoir. The San Luis Pumping-Generating Plant is the only inlet-outlet for San Luis Reservoir. Four $17\frac{1}{2}$-foot-diameter concrete and partly steel-lined tunnels, located in the left abutment, connect the plant to the four trashrack tower structures (Figures 238 and 239). Each of these tunnels services two pump-turbines.

The tunnels are about 2,150 feet long and were driven from both portals simultaneously. The geology of the site, and particularly the existence of faults, influenced the method of construction. The normal concrete lining is 2 feet - 5 inches thick. The thickness at special sections through faults, and in areas of low rock cover, is a minimum of 3 feet - 6 inches. The downstream portions of the tunnels are steel-lined.

As inlets, the tunnels can convey 11,000 cfs and, operating as reservoir outlets, they can discharge 13,120 cfs.

The spillway, with a capacity of 1,030 cfs at water surface elevation 545.8 feet, is sized to pass the maximum probable floodflow. However, because the San Luis Pumping-Generating Plant has a capacity to discharge a flow of 13,120 cfs while generating, the spillway will be used only if the Pumping-Generating Plant is inoperative during a flood. Under such conditions, the flow would pass into a glory-hole-type inlet structure, through a 9-foot - 6-inch conduit under the Dam on the left abutment, and into a rectangular chute. At the downstream end of the chute, the flow would enter a stilling basin, then discharge into the approach channel of the Pumping-Generating Plant.

O'Neill Forebay. As stated previously, there are four inlets and outlets for O'Neill Forebay: (1) San Luis Pumping-Generating Plant, (2) O'Neill Pumping Plant, (3) North San Joaquin Division of the California Aqueduct, and (4) San Luis Canal. O'Neill Pumping Plant has six reversible units. Each has its own 10-foot-diameter discharge line to the Forebay which carries water at a 700-cubic-foot-per-second rate. The inlet from the North San Joaquin Division portion of the California Aqueduct is a simple transition, while the outlet to the San Luis Canal essentially is a canal check structure.

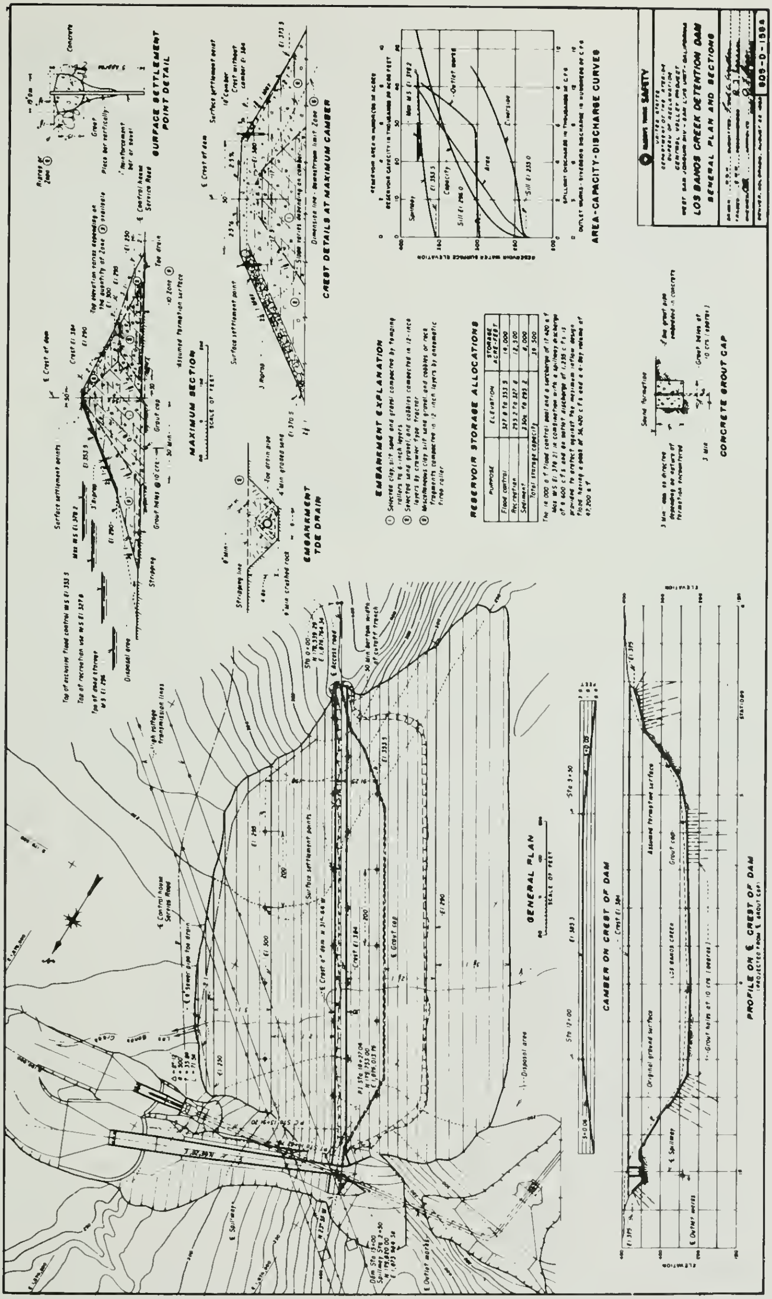

Figure 236. Los Banos Detention Dam—Plan, Profile, and Sections

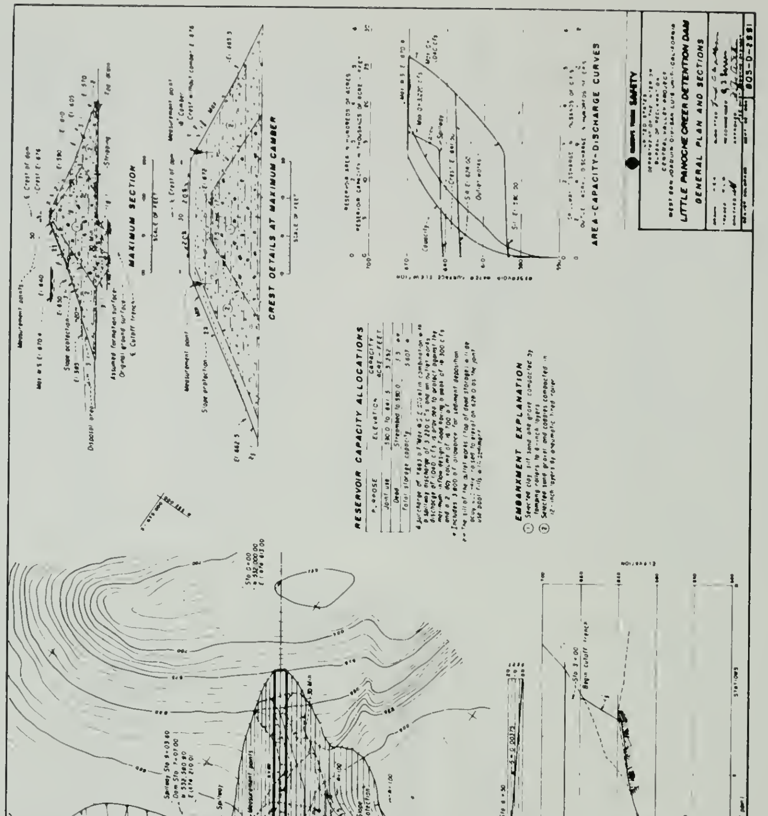

Figure 237. Little Panoche Detention Dam—Plan, Profile, and Sections

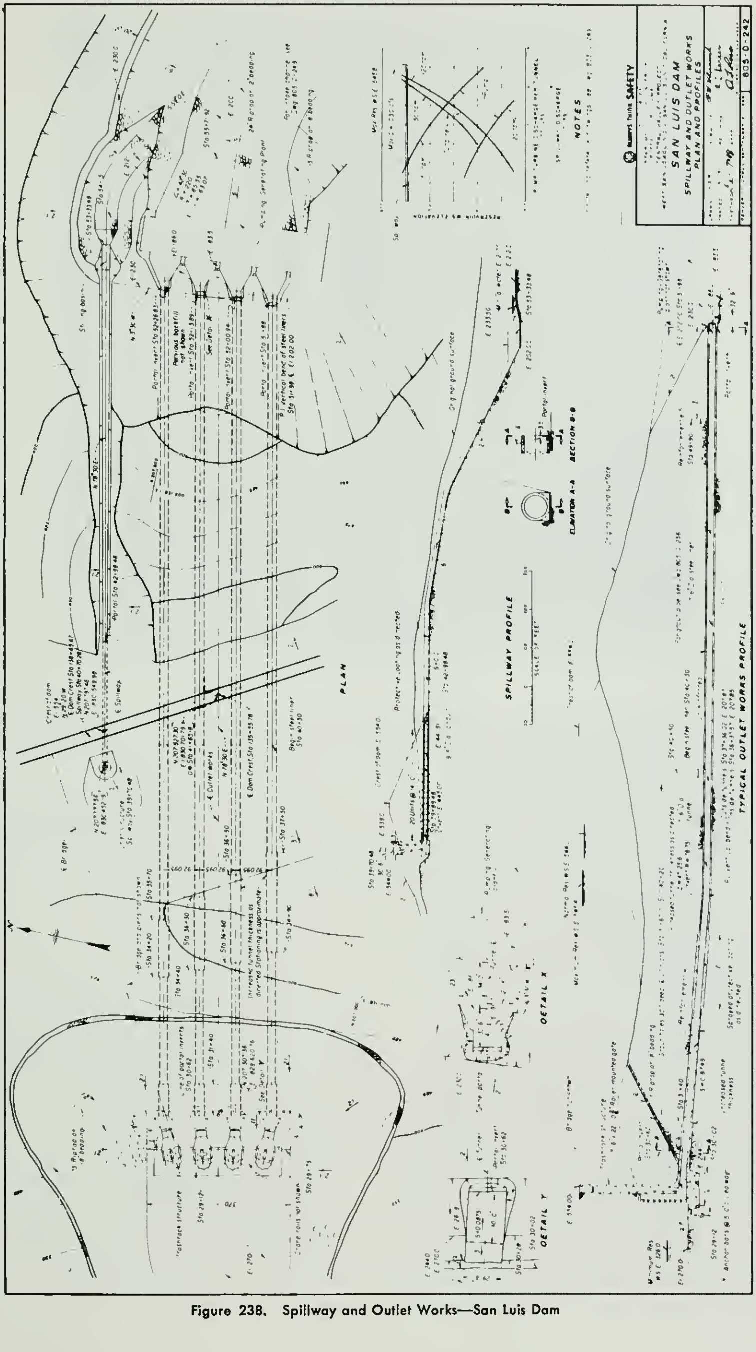

Figure 238. Spillway and Outlet Works—San Luis Dam

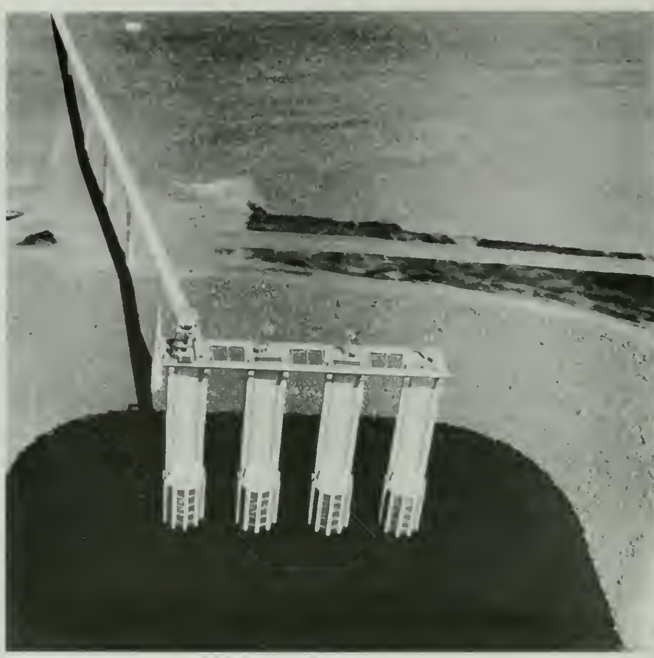

Figure 239. Trashrack Structure and Access Bridge—San Luis Dam

The spillway (Figure 240), with a capacity of 3,250 cfs at water surface elevation 228 feet, is capable of carrying the maximum probable floodflow from the Forebay. Also, it is expected to be used infrequently because the reversible O'Neill Pumping Plant has a capacity to discharge 3,600 cfs while generating, which is well in excess of the spillway design flow. During a plant outage, floodflows will pass into a glory-hole spillway adjacent to the Pumping Plant, through an 11-foot - 9-inch-diameter conduit under the Dam, and into a stilling basin at the toe of the Dam. After being stilled, the flow will enter the forebay wasteway and pass through a siphon under the Delta-Mendota Canal. Here, it is returned to the channel of San Luis Creek.

Los Banos Reservoir. The outlet works located in the left abutment of the Dam will discharge a maximum of 1,225 cfs and consists of an 84.5-foot-high, trashracked, intake structure; a 6.5-foot-inside-diameter (ID), 510-foot-long, circular tunnel; a gate chamber housing one 5-foot by 6-foot, hydraulically operated, high-pressure, emergency gate; and a 70-inch ID, 497-foot-long, steel conduit (within a 10.5-foot ID, concrete, horseshoe tunnel) which branches into two outlets pipes, each controlled by a 3.5-foot-square, hydraulically operated, high-pressure, slide gate. The outlets discharge the water into a stilling basin which, in turn, empties into the existing channel of Los Banos Creek downstream from the structure. Plan and profiles of the outlet works are shown on Figures 241 and 242.

The concrete chute spillway has a discharge capacity of 8,600 cfs and is located in the left abutment of the Dam. It has an uncontrolled 20-foot-long ogee crest at elevation 353.5 feet. Water reaches the spillway crest through an inlet channel to the crest, then flows through an open chute of variable width down to the stilling basin located near the toe of the Dam.

Little Panoche Reservoir. The outlet works, located adjacent to the spillway near the right abutment of the Dam, will discharge a maximum of 1,040 cfs and consists of a 90.5-foot-high, trashracked, intake structure and a 6-foot - 6-inch ID, 592-foot-long, ungated, reinforced-concrete pipe that empties into a stilling basin adjacent to the spillway stilling basin. These stilling basins, in turn, flow into Little Panoche Creek channel downstream of the structure. The plan and profile of the outlet works are shown on Figure 243.

The spillway, with a glory-hole-type inlet has a discharge capacity of 3,220 cfs and is located in the embankment on the right abutment. It discharges through a 9-foot - 6-inch-diameter conduit into a stilling basin at the toe of the Dam.

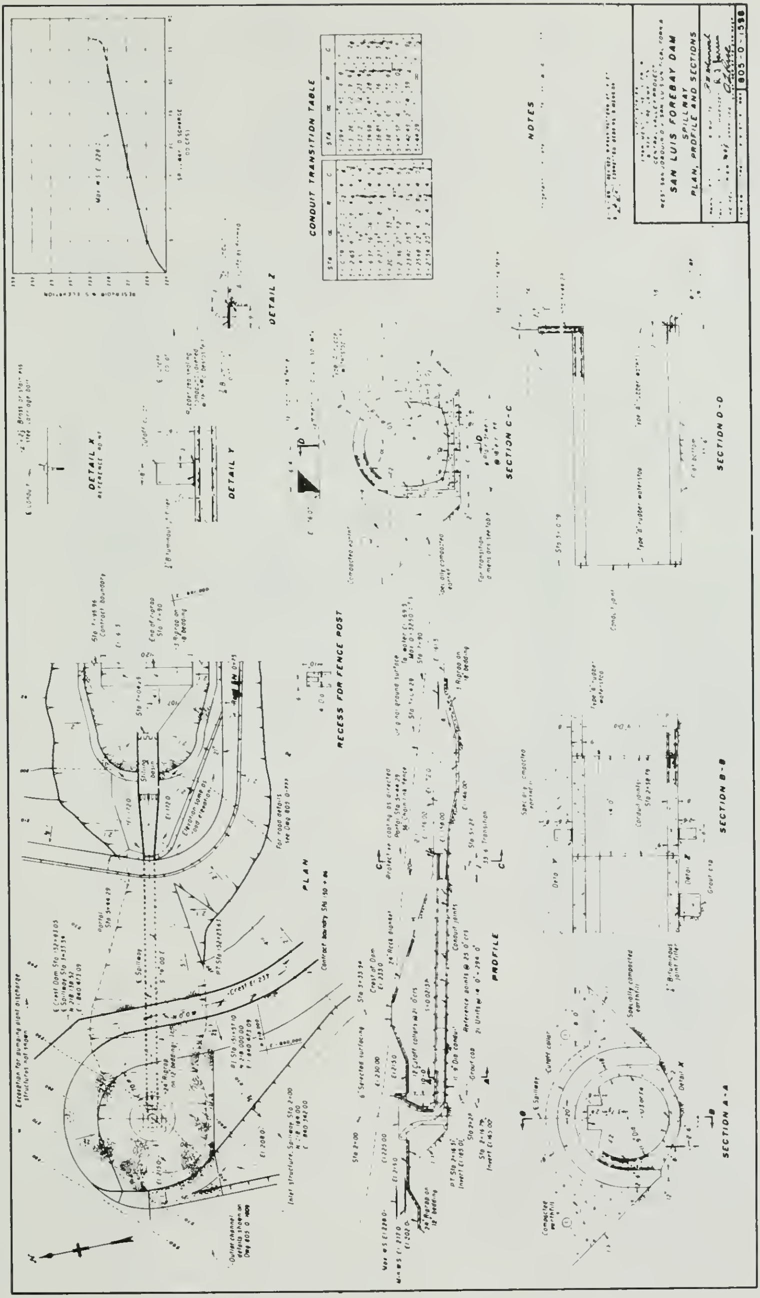

Figure 240. Spillway—O'Neill Forebay Dam

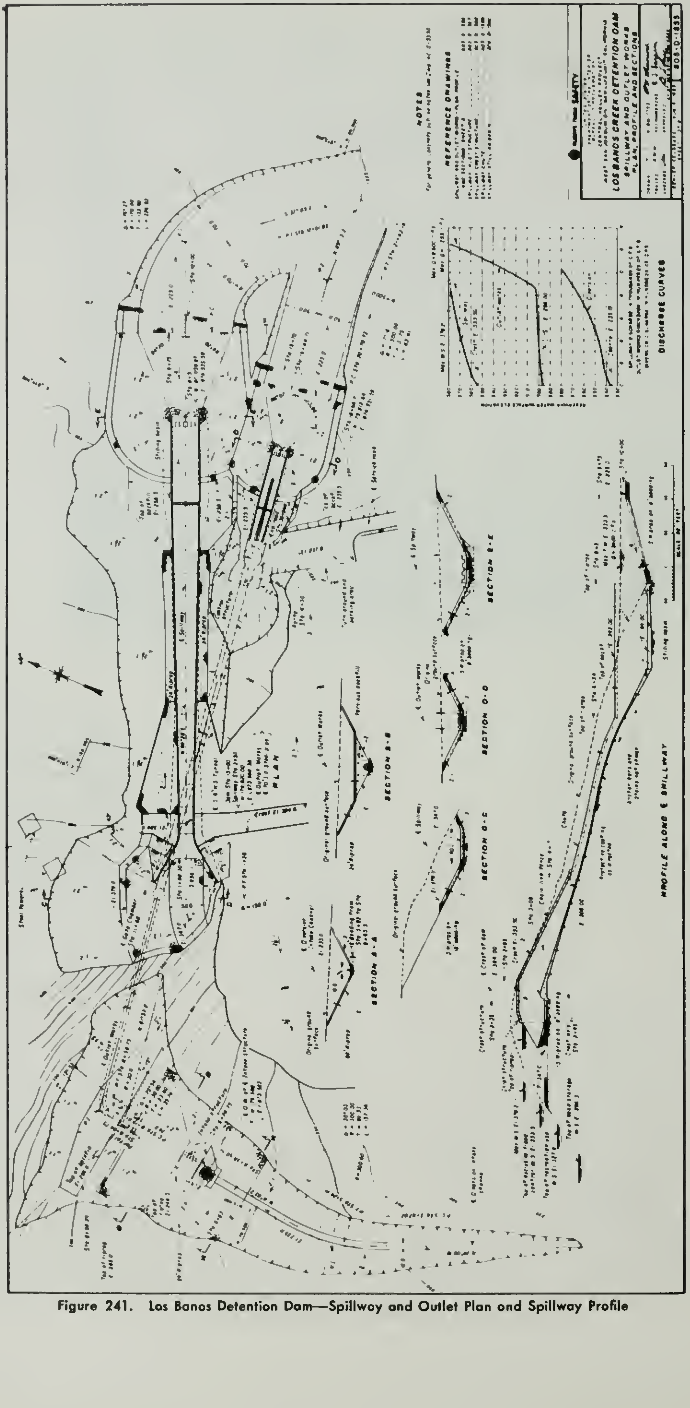

Figure 241. Los Banos Detention Dam—Spillway and Outlet Plan and Spillway Profile

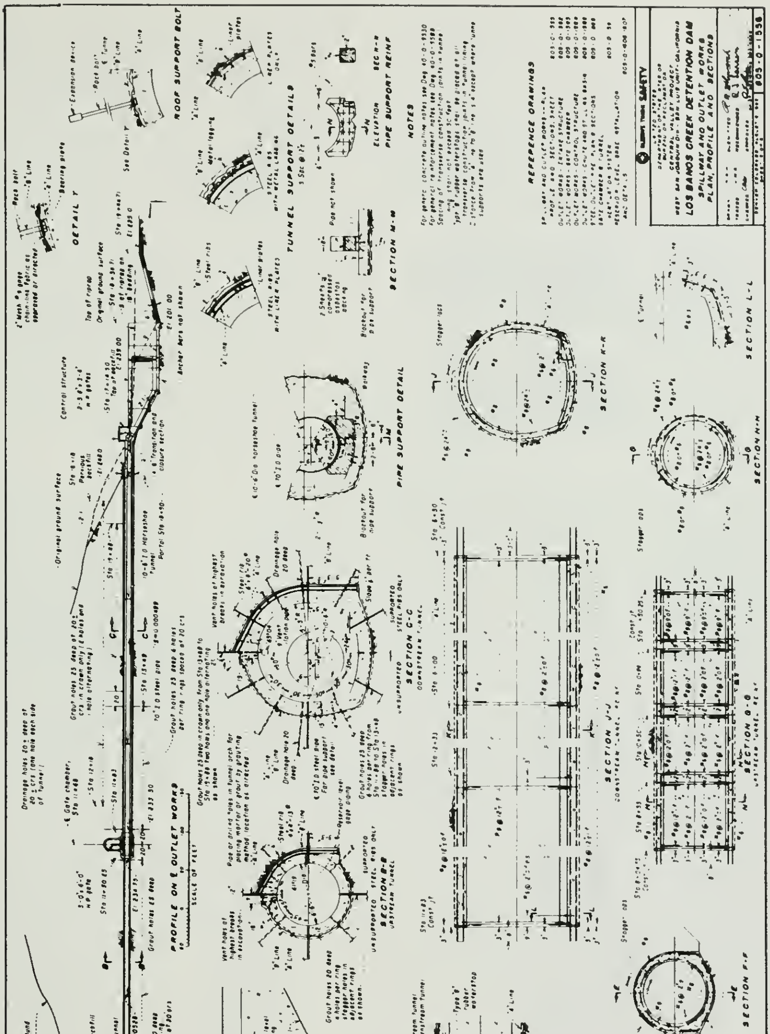

Figure 242. Los Banos Detention Dam—Outlet Profile

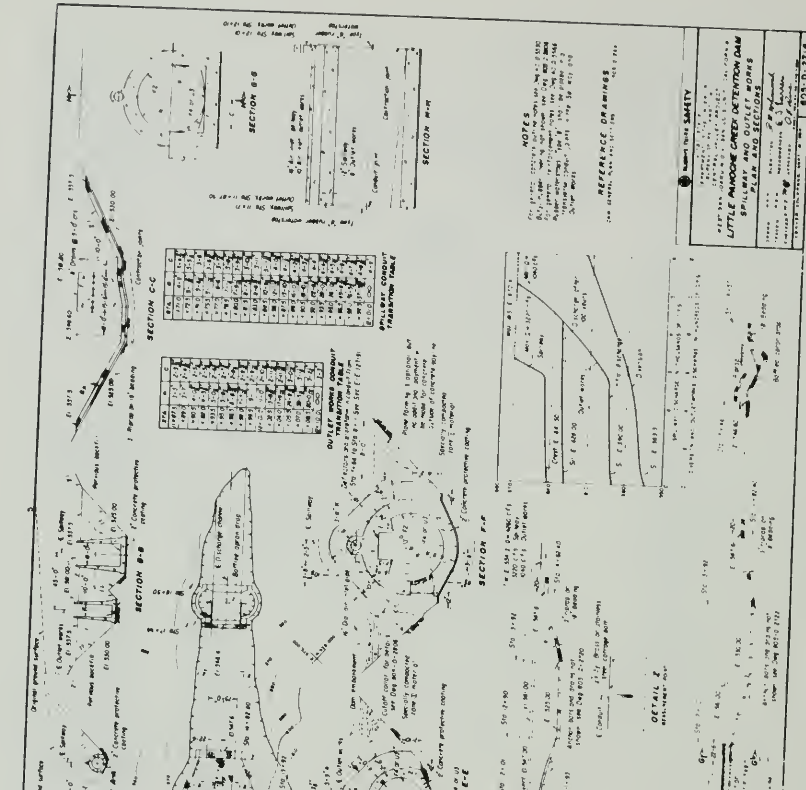

Figure 243. Little Panache Detentian Dam—Spillway Plan and Sections

Instrumentation

Instrumentation at San Luis Dam and O'Neill Dam consists of the following:

San Luis Dam

Monumentation	
Surface settlement	117
Spillway tunnel	14
Spillway chute and basin	110
Trashrack structure	10
Hydraulic piezometers	119
Internal vertical and horizontal movement devices	4
Base plates—foundation settlement	3
Observation wells	16
Toe drains	8
Seepage measurements	5
Slide monitoring	1
Area control	60

O'Neill Dam

Monumentation	
Embankment	42
Spillway conduit	13
Spillway chute and basin	16
Toe drains	3
Ground water observation wells in agriculture settings	34
Ground water observation wells at dam and appurtenant features	15

The location of certain instrumentation at San Luis Dam is shown on Figure 244.

The following instrumentation was installed at Los Banos Detention Dam and Little Panoche Detention Dam:

Los Banos

Monumentation	
Embankment	20
Spillway	46
Outlet works	20
Abutments	14
Abutment piezometer wells	2
Toe drain	1
Weir, outlet tunnel drain	1

Little Panoche

Monumentation	
Embankment	23
Spillway conduit	18
Spillway stilling basin	25
Outlet conduit	24
Outlet stilling basin	21
Abutment	16
Toe drain	1

The location of the embankment monuments and the toe drains is shown on Figures 237 and 240.

Recreation

Although the drawdown of San Luis Reservoir (discussed under "Operation") will not be an annual occurrence, it affects the recreation potential of the Reservoir. Therefore, to realize the greatest return from multipurpose benefits, proportionally greater outlay is invested in recreation facilities at O'Neill Forebay where maximum reservoir fluctuation will be only 8 feet.

During construction of San Luis Dam, some of the embankment materials were excavated from the forebay reservoir area. This excavation was performed with the objective of improving the forebay topography in the interest of creating better beach and bottom areas for the recreational environment. Also, it created a greater storage capacity to minimize drawdown. It is projected that the Forebay will receive considerably greater recreation patronage because of the more stable water surface.

Native vegetation in the area is predominantly grasses, with scattered trees at the higher elevations around San Luis Reservoir. Mainly, the land had been used for grazing and livestock production. For the most part, the locations chosen for recreation were barren. At an early stage, the State planted trees to provide areas around the reservoirs more conducive to recreation enjoyment.

The California Department of Parks and Recreation is responsible for project recreation. It funds the onshore improvements and manages the recreation facilities. The Department of Water Resources provides interpretive services to the public at Romero Overlook, an attractive visitor facility which is adjacent to State Highway 152 and beyond the left abutment of San Luis Dam.

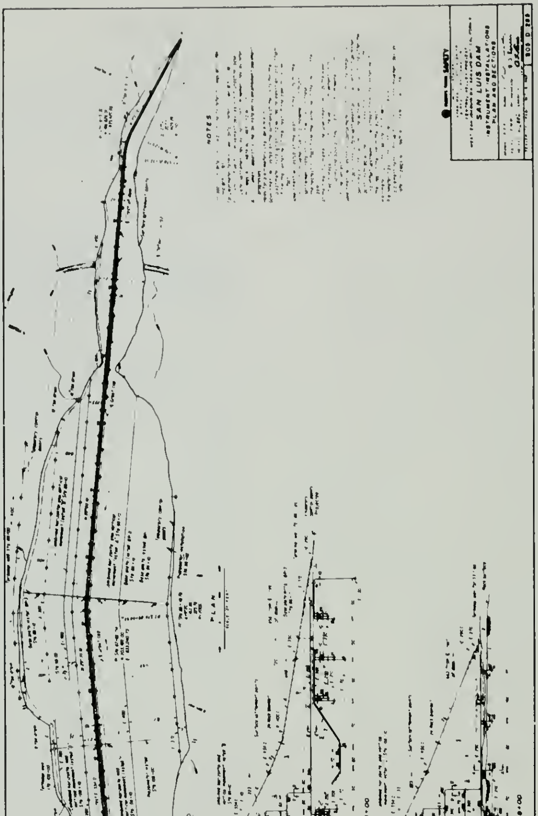

Figure 244. Location of Instrumentation—San Luis Dam

BIBLIOGRAPHY

United States Bureau of Reclamation, "Agreement Between the United States of America and the Department of Water Resources of the State of California for the Construction and Operation of the Joint-Use Facilities of the San Luis Unit", Contract No. 14-06-200-9755, December 30, 1961.

———, "Final Construction Report, Completion of Dos Amigos (Mile 18) Pumping Plant Switchyard and Appurtenant Works", Specifications No. DC-6255, April 1970.

———, "Final Construction Report, Completion of San Luis Pumping-Generating Plant and Switchyards", Specifications No. DC-6185, June 1970.

———, "Final Construction Report, Dos Amigos (Mile 18) Pumping Plant", Specifications No. DC-5982, April 1969.

———, "Final Construction Report, Forebay Pumping Plant and Appurtenant Works and Forebay Dam Spillway", Specifications No. DC-6033, December 1971.

———, "Final Construction Report, Little Panoche Creek Detention Dam", Specifications No. DC-6236, May 1972.

———, "Final Construction Report, Los Banos Creek Detention Dam", Specifications No. DC-6080, December 1966.

———, "Final Construction Report, San Luis Dam and Pumping-Generating Plant and Forebay Dam", Specifications No. DC-5855, February 1970.

———, "Final Construction Report, San Luis Canal, Reach 1", Specifications No. DC-5900, September 1970.

———, "Final Construction Report, San Luis Canal, Reach 2", Specifications No. DC-5977, August 1971.

———, "Final Construction Report, San Luis Canal, Reach 3", Specifications No. DC-6148, December 1969.

———, "Final Construction Report, San Luis Canal, Reach 4", Specifications No. DC-6280, June 1971.

———, "Final Construction Report, San Luis Canal, Reach 5", Specifications No. DC-6344, July 1971.

———, "Final Construction Report, Pacheco Inlet Channel and Tunnel to Station 198+55", Specifications No. DC-6160, April 1971.

———, "History, General Description, and Geology", Volume I, San Luis Unit Technical Record of Design and Construction.

———, "San Luis Dam and Pumping-Generating Plant, and O'Neill Dam and Pumping Plant—Design", Volume II, San Luis Unit Technical Record of Design and Construction.

———, "San Luis Dam and Pumping-Generating Plant, and O'Neill Dam and Pumping Plant—Construction", Volume III, San Luis Unit Technical Record of Design and Construction.

———, "Dos Amigos Pumping Plant and Pleasant Valley Pumping Plant—Design", Volume IV, San Luis Unit Technical Record of Design and Construction.

———, "Dos Amigos Pumping Plant and Pleasant Valley Pumping Plant—Construction", Volume V, San Luis Unit Technical Record of Design and Construction, September 1974.

———, "Waterways and Detention Dams—Design", Volume VI, San Luis Unit Technical Record of Design and Construction.

———, "Waterways and Detention Dams—Construction", Volume III, San Luis Unit Technical Record of Design and Construction.

———, "Designer's Operating Criteria, San Luis Canal", July 1966.

———, "Pile Driving Report on San Luis Canal, Reach 3", Specifications No. DC-6148.

———, "Preliminary Design of Canal Sections, San Luis Canal", July 1961 (revised September 1961).

———, "Final Embankment and Grouting Report, San Luis Dam and Pumping-Generating Plant and Forebay Dam", Specifications No. DC-5855, December 1968.

———, "Final Report, Test Apparatus Installation, San Luis Dam", Specifications No. DC-5855, November 1967.

An Act of Congress Authorizing the San Luis Unit, Central Valley Project, Public Law 86-488, 86th Congress, S. 44, (74 Stat. 156), June 3, 1960. (United States Bureau of Reclamation)

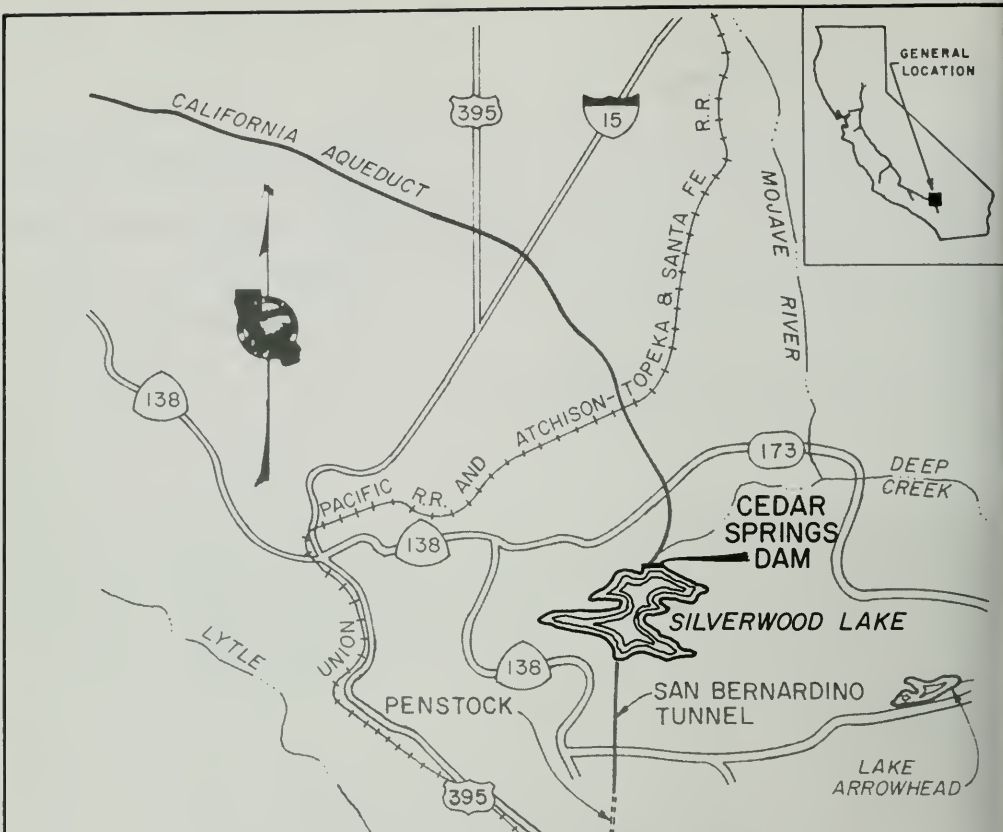

Figure 245. Location Map—Cedar Springs Dam and Silverwood Lake

CHAPTER XII. CEDAR SPRINGS DAM AND SILVERWOOD LAKE

General

Description and Location

Cedar Springs Dam is a rolled earth and rockfilled dam with a structural height of 249 feet and a crest length of 2,230 feet. The spillway is an ungated concrete chute with the outlet works located in a tunnel directly beneath; both have a common energy dissipator. The reservoir is supplied project water from Mojave Siphon which terminates at the inlet structure on the left abutment of the Dam directly west of the spillway (Figures 245 and 246). Construction work on the Dam was started November 10, 1968 and completed during the summer of 1971. Initial reservoir filling started on January 5, 1972.

The embankment consists of a zoned earth-rock section that utilizes thick shells of rolled rockfill and dumped large rock, both upstream and downstream of the impervious core. Alignment of the embankment is buttressed by a downstream knoll so that downstream deflection compresses the core. In addition, the core is thickened in such critical areas as in the convex downstream curvature of the Dam and throughout the left abutment area where a shear zone exists.

Figure 246. Aerial View—Cedar Springs Dam and Silverwood Lake

Special consideration for earthquake loading was given to all zones, including the filter and transition. The embankment was constructed to accommodate a shear movement of 5 feet and still retain a sufficient zone thickness to function as designed. Zone 2, located next to the core, consists of silty and gravelly sands of the Harold formation and is used to provide both a sealing material in the event of rupture and a usable transition section.

In 1968, Cedar Springs Reservoir was officially renamed Silverwood Lake. At about the same time, plans were being completed for the construction of large-scale recreation facilities.

Large quantities of borrow waste and channel excavation material were utilized during construction to form beaches and boat ramps. On the west side of the reservoir, rerouted State Highway 138 incorporates several scenic, overlook, parking areas. On the opposite side of the reservoir from Highway 138, an emergency fire road was constructed to a greater width than normally required to include a portion of the California riding and hiking trail. When possible, the reservoir water level will be maintained within a range of 30 inches during any seven-day period from March 1 to September 1. A statistical summary of Cedar Springs Dam and Silverwood Lake is shown in Table 31, area-capacity curves on Figure 247, and a Dam site plan on Figure 248.

Cedar Springs Dam lies 10 miles due north of the City of San Bernardino on the West Fork of the Mojave River near the junction of State Highways 173 and 138. The reservoir drainage area covers 34 square miles.

Purpose

Cedar Springs Dam and Silverwood Lake are part of the California Aqueduct and provide regulatory and emergency storage. The reservoir's primary purposes are to firm deliveries to water users along the Aqueduct, provide recreation, and assure continuity of discharge through Devil Canyon Powerplant.

TABLE 31. Statistical Summary of Cedar Springs Dam and Silverwood Lake

CEDAR SPRINGS DAM

Type: Zoned earth and rockfill

Crest elevation	3,378 feet
Crest width	42 feet
Crest length	2,230 feet
Streambed elevation at dam axis	3,165 feet
Lowest foundation elevation	3,129 feet
Structural height above foundation	249 feet
Embankment volume	7,600,000 cubic yards
Freeboard above spillway crest	23 feet
Freeboard, maximum operating surface	23 feet
Freeboard, maximum probable flood	5 feet

SILVERWOOD LAKE

Storage at spillway crest elevation	74,970 acre-feet
Maximum operating storage	73,031 acre-feet
Minimum operating storage	39,211 acre-feet
Dead pool storage	3,967 acre-feet
Maximum operating surface elevation	3,353 feet
Minimum operating surface elevation	3,312 feet
Dead pool surface elevation	3,235 feet
Shoreline, spillway crest elevation	13 miles
Surface area, spillway crest elevation	976 acres
Surface area, maximum operating elevation	962 acres
Surface area, minimum operating elevation	690 acres

SPILLWAY

Type: Ungated ogee crest with lined channel and stilling basin

Crest elevation	3,355 feet
Crest length	120 feet
Maximum probable flood inflow	51,000 cubic feet per second
Peak routed outflow	32,250 cubic feet per second
Maximum surface elevation	3,373 feet
Standard project flood inflow	27,500 cubic feet per second
Peak routed outflow	21,000 cubic feet per second
Maximum surface elevation	3,368 feet

INLET WORKS

Open concrete-lined channel and chute from terminus of Mojave Siphon to flip bucket on reservoir floor

Capacity	1,990 cubic feet per second

OUTLET WORKS

Stream release facility: Lined tunnel under spillway, valve chamber at midpoint—discharge onto spillway

Diameter: Upstream of valve chamber, 13-foot pressure tunnel—downstream, 13-foot concrete horseshoe tunnel

Intake structure: Uncontrolled tower with provision for steel plug emergency bulkhead

Control: Stream maintenance, 30-inch fixed-cone dispersion valve and two sets of two 5-foot-wide by 9-foot-high, high-pressure, slide gates in tandem within valve chamber

Release at minimum storage	5,000 cubic feet per second

Las Flores Pipeline: Stream release from Mojave Siphon

Capacity	23 cubic feet per second

OUTLET

San Bernardino Tunnel: Lined tunnel 12 feet - 9 inches in diameter—six-level intake tower—inlets controlled by 60-inch butterfly valves

Capacity	2,020 cubic feet per second

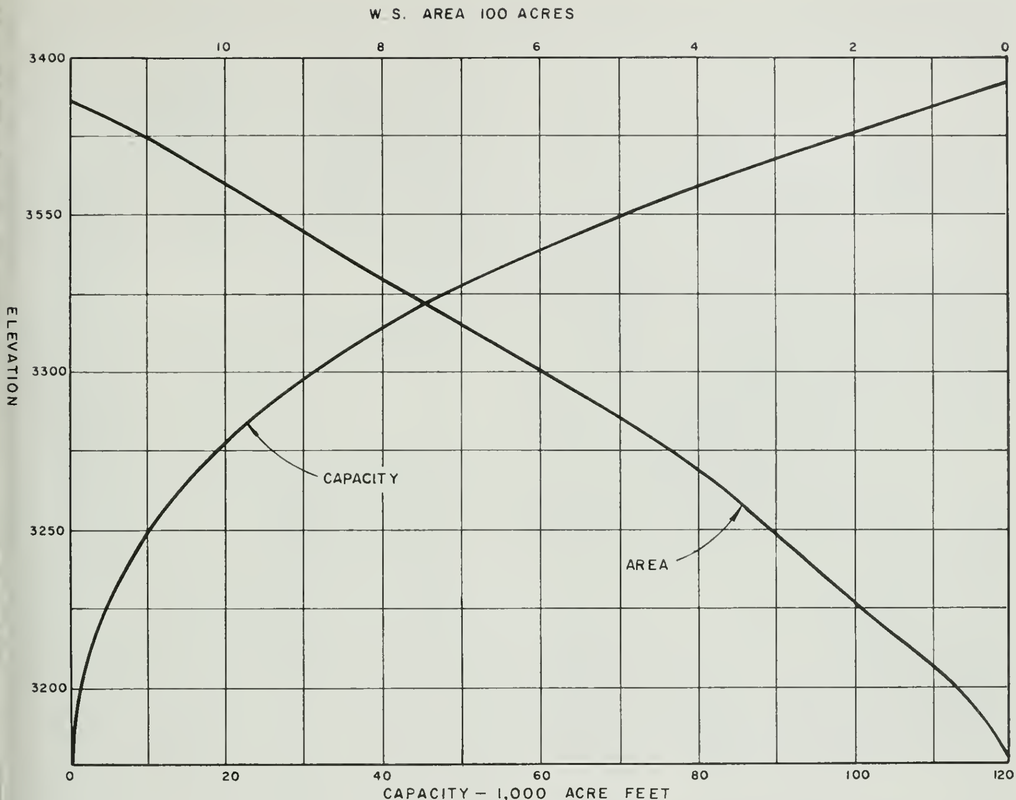

Figure 247. Area-Capacity Curves

Cedar Springs Dam has features which were designed to enable it to withstand overtopping by wave action in the event it should occur. The central core and adjacent zones have been capped by a paved crest road. The downstream section, which is constructed of massive rocks, is capable of passing large flows safely. Added safety from a seiche overtopping the Dam is provided by the reservoir's irregular shape and the 23-foot surcharge and freeboard distance between the normal water surface and the dam crest.

The outlet works and spillway were constructed on the left abutment of the Dam and utilize a common energy-dissipating basin and river return channel. Downstream release controls consist of two pairs of 5- by 9-foot, high-pressure, slide gates and a 30-inch fixed-cone dispersion valve located inside a tunnel constructed at streambed elevation. The outlet tunnel was used during construction of the Dam to divert floodflows around the dam embankment. In case of an emergency, a full reservoir can be dewatered in approximately one week.

The dam spillway is an ungated, 120-foot-wide, rectangular, lined chute that is located directly over the outlet works tunnel. Because the spillway crosses a fault zone, extra precautions were taken in the design to minimize loss of operating capabilities if a movement should occur. These included having the wall steel continuous through the concrete joints, constructing shear keys into the fault zone material instead of anchor bars, and providing sufficient area in the spillway to prevent backwater on the weir in the event there is local movement downstream in the chute.

Chronology

Final design of a 216,000-acre-foot storage reservoir and dam commenced during July 1964. In April 1965, the Department of Water Resources' Consulting Board for Earthquake Analysis concluded that, while the probability of a fault offset occurring through the damsite was small, it would be assumed that such a fault displacement could happen. As a result, it was

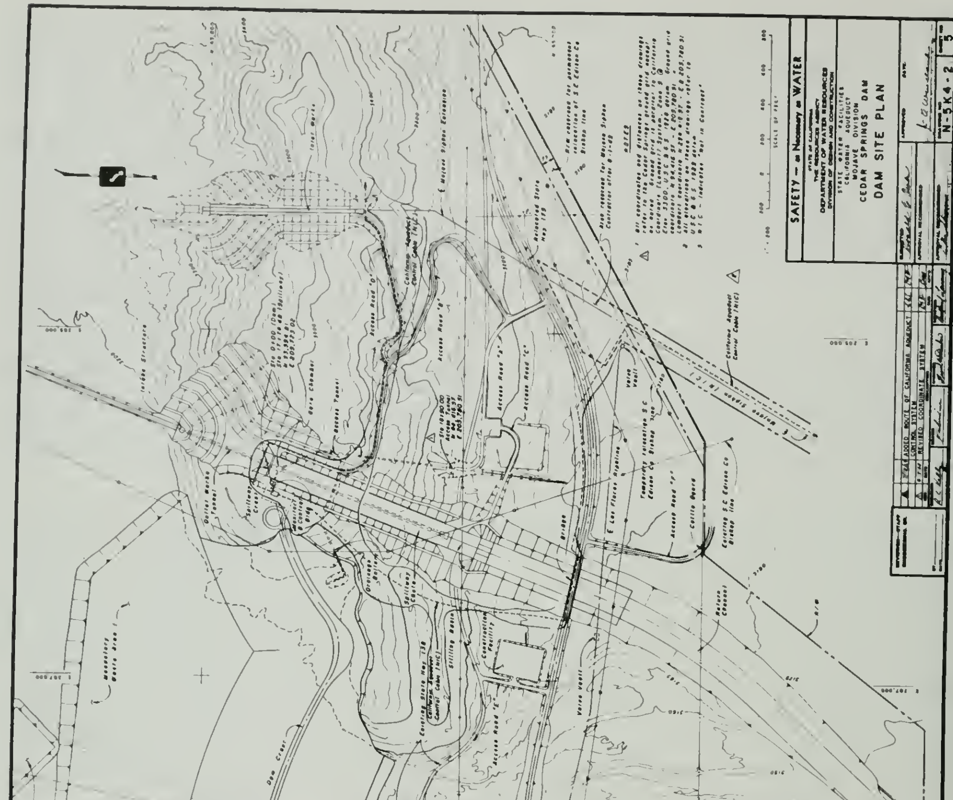

Figure 248. Dam Site Plan

decided to limit the size of Silverwood Lake to approximately 75,000 acre-feet.

To make up the resulting loss of storage capacity, other alternative and supplemental sites to Cedar Springs Dam were studied. In all cases, the cost per acre-foot of storage was considerably more than for the original plan. Therefore, the most practical solution was to build a dam on the original site to a size deemed safe for the foundation conditions and to adjust the system elsewhere. The reduced size of the dam then became a factor in the final sizing of the Aqueduct to Lake Perris, which is discussed in Volume II of this bulletin.

Design of the Dam and reservoir was completed August 9, 1969, and construction began shortly thereafter. The dam embankment was completed during March 1971. The spillway and outlet works essentially were completed by April 1971.

Regional Geology

Cedar Springs Dam is in the northwest portion of the San Bernardino Mountains, near the boundary of the Transverse Ranges geomorphic province with the Mojave Desert. Bounded on the south by the San Andreas fault and on the north by a series of east-west faults, the San Bernardino Mountains form an east-west-trending block about 55 miles long and up to 30 miles wide. Rock types are mainly gneissic and granitic rocks of the Mesozoic Age. Pre-Cambrian gneiss, which includes some scattered bodies of marble, is dominant in the southwest portion of the range. Tertiary to Quaternary continental sediments and older and recent alluvium are found locally along structural troughs and valley floors.

Cedar Springs Dam is 7 miles from the San Andreas fault and 10 miles from the San Jacinto fault. Several faults, some associated with the San Andreas, pass through the reservoir area. These faults have steep dips and trend about N 70° W. However, two faults near the Dam site strike roughly N 60° E and N 40° E. The Cleghorn fault, which offsets older alluvium in Cleghorn Canyon, is about 2 miles south of the Dam.

Seismicity

It was necessary to consider the effects of a fairly large earthquake in design of the Dam and appurtenant structures because of the Dam's proximity to the San Andreas and San Jacinto fault zones. Instrumentally determined epicentral data are available from 1937, and records of earthquake intensities are available from 1857 to date (1974).

Major Faults. During the 1857 earthquake, the ground along the San Andreas fault was fractured from San Bernardino to Cholame Valley, a distance of about 200 miles. An estimated 20 feet of horizontal displacement of the land surface occurred near Gorman, 95 miles northwest of the Dam.

San Jacinto fault was the source of two earthquakes during the past 70 years estimated to have Richter magnitudes of 6.8 and 6.3. These caused severe damage near Hemet, San Jacinto, and San Bernardino—47, 43, and 18 miles southwesterly of the Dam, respectively.

Cleghorn fault trends in a general northwest-southeast direction in the reservoir area about 1½ miles south of Cedar Springs Dam.

Sierra Madre fault zone is located about 20 miles southwest of Cedar Springs Dam. It is considered to be active, but it is improbable that earthquakes associated with this fault would damage Cedar Springs Dam.

Two presumably active faults were discovered during excavation of the foundation material near the axis of Cedar Springs Dam (Figure 249) and have been traced a distance of from 1 to 3 miles. Vertical displacement of 3 to 5 feet has been measured at an exposed fault in the exploration trench. It is highly improbable during the life of the Dam that displacement along these faults in any direction will exceed 5 feet. Separation of the fault wall is not expected, even if movement should occur. The basis for these assumptions can be found in the Bibliography.

Design Criteria for Maximum Credible Accident. The maximum credible accident is defined as a major earthquake producing a 3- to 5-foot displacement in the foundation, occurring when the reservoir is at maximum water surface with spillway and outlet works operating at maximum discharge.

Six possible conditions that could result from the maximum credible accident were analyzed, resulting in 20 specific provisions being incorporated into design to protect against their effects. Provisions included selecting a material for the impervious core that can deform plastically without cracking, thickening tran-

Figure 249. Fault Uncovered During Excavation

sition zones to accommodate a 5-foot displacement along a fault while maintaining sufficient thickness to operate as designed, not placing the core on known faults, providing additional freeboard to guard against overtopping, and locating the spillway so that the chute does not cross a fault until it is 400 feet downstream of the crest.

Design Criteria for Embankment. The dam embankment was designed to withstand earthquakes by including a horizontal force in the stability analyses. This force was equivalent to an acceleration of 0.15g multiplied by the mass of the material in a sliding wedge or slip circle and was applied in the direction of greatest instability. The embankment was designed to meet the required factor of safety against sliding with this horizontal force and the normal static forces applied.

Two conditions resulting from the maximum credible accident were applied in the design stability analyses for the dam embankment: (1) a full reservoir was assumed against the face of Zone 3 downstream of the impervious core, and (2) upstream Zone 2 was assumed liquefied with all loss of strength. These conditions were additive to the normal static forces plus the force due to the 0.15g horizontal earthquake acceleration.

Design Criteria for Structures. Earthquake loads applied to the structures of the complex were scaled as to the importance of the structure and the consequences of its failure. The earthquake accelerations that were used in design are listed in Table 32.

Design

Dam

Description. The dam embankment is a zoned rockfill structure with a core consisting of an impervious, imported, plastic material (Zone 1) flanked by narrow zones of nonplastic native materials (Zone 2). Figures 250, 251, and 252 show the embankment plan, sections, and profile, respectively. Transition zones are provided on each side of the core consisting of either streambed sands and gravels or fines produced from the processing of material for the downstream shell (Zone 3). The upstream shell is rolled quarry-run rock (Zone 4A). The inner portion of the downstream shell is rolled processed rock (Zone 5).

A plastic clay was chosen for the core because of its ability to deform rather than crack if the foundation should settle or shift during an earthquake. Even if the clay should crack, it has the added characteristic to resist erosion.

Because of the seismic hazard, the core and filters were made thicker than usual. A wide cross section resulted at the elevation of the normal reservoir water surface. Rather than have a wide crest, the slopes were extended upward to a narrower higher crest to provide additional freeboard above the normal water surface. Additional embankment was relatively inexpensive and provided greater safety against overtopping by a reservoir seiche, as well as allowing a narrower, less expensive spillway.

Blankets of streambed gravels were provided under the upstream and downstream rock shells of the Dam. The primary reason for these blankets was to provide a filter over the minor faults passing through the foundation. This will prevent piping through the rockfill embankment downstream should seepage develop through the shear zones of the faults. Upstream, the filter material could enter any developing crevice, thus helping to develop a seal within the crevice.

Foundation. The downstream two-thirds of the dam foundation is granitic rock and the upstream one-third is slightly indurated silty sand of the Harold formation. The Harold formation is in a nearly vertical contact with the granitic rock. Movement along both the granite rock-Harold formation fault and intersection fault displaced alluvial stream gravels, indicating the fault system was active during recent geologic time. As a precaution, the Dam was designed

TABLE 32. Design Earthquake Accelerations—Cedar Springs Dam Complex

Structure	Foundation	Percent San Andreas Design Earthquake	Relative Importance	Applied Horizontal Acceleration
Spillway				
Crest (includes approach and crest walls and crest structures)	Granite	50	100	0.25g
Spillway chute and stilling basin	Granite shear zone, Harold formation	75	40	0.15g
Outlet Works				
Inlet tower and trashrack	Granite	50	100	0.25g
Gate chamber	Granite	50	100	0.25g
Mojave Siphon Inlet Works				
Chute	Granite	50	40	0.10g

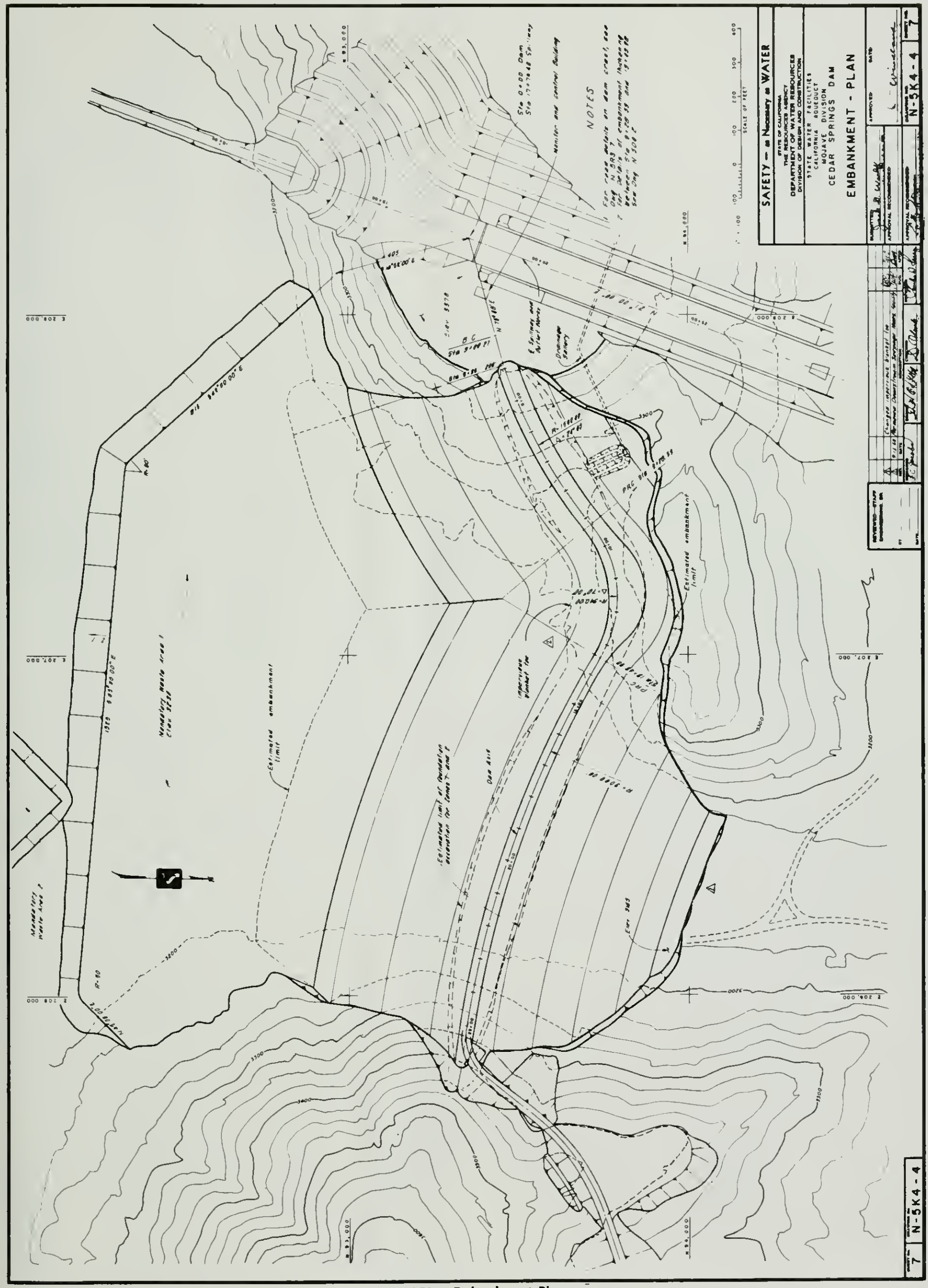

Figure 250. Embankment Plan

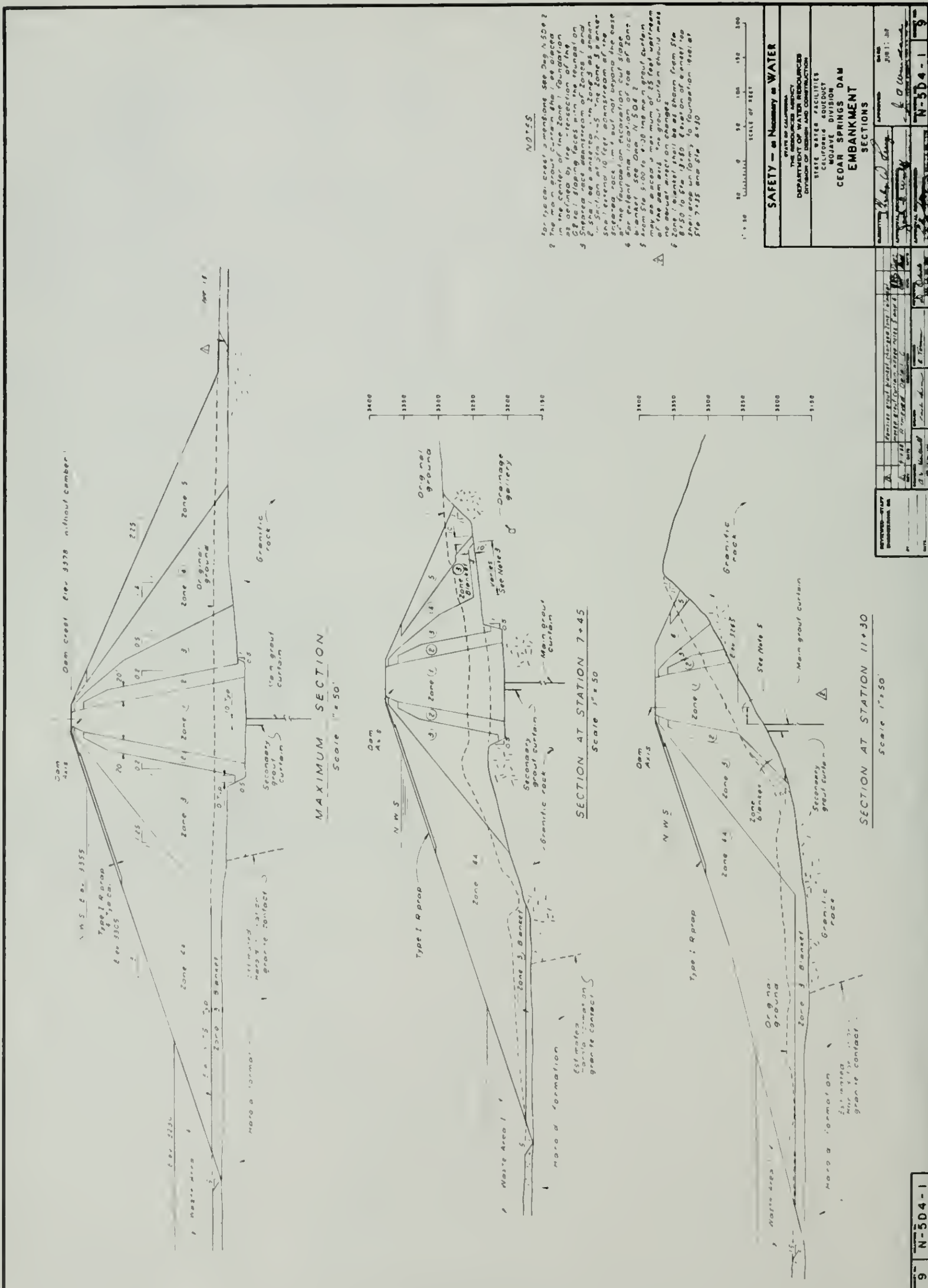

Figure 251. Embankment Sections

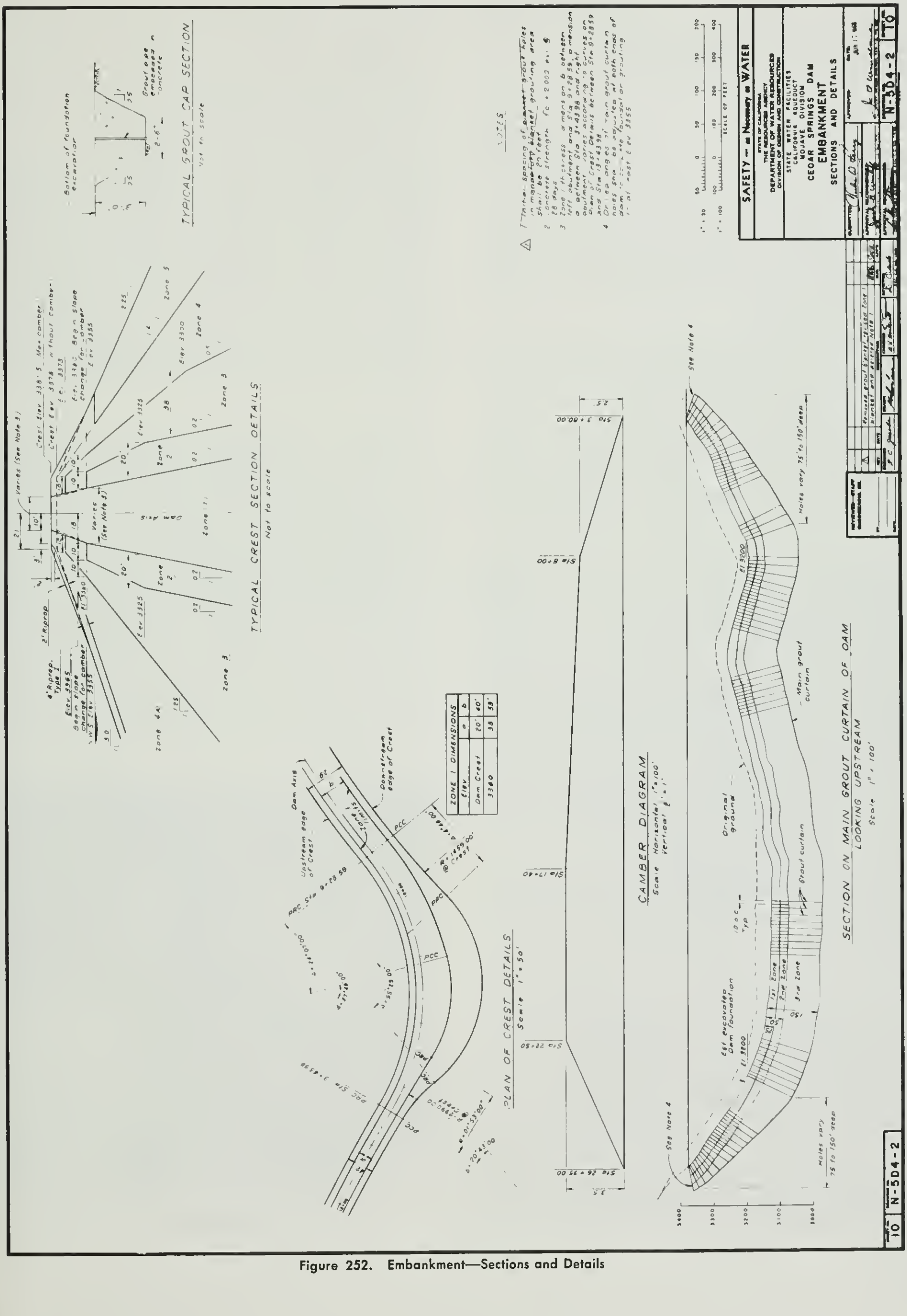

Figure 252. Embankment—Sections and Details

to maintain a minimum distance of 200 feet between the dam axis and the granite rock-Harold formation fault contact. Removal of the alluvium, colluvium, and highly weathered and badly fractured bedrock was specified under the embankment to ensure a foundation at least as strong as the embankment rockfill.

Embankment Layout. To minimize potential cracking, the core was located entirely on the granite rock portion of the foundation. This condition required the dam axis to be bent sharply near the center. The core at the bend was thickened and buttressed against the knoll located between the left abutment and the river channel. The axis then was arched toward the reservoir on both sides of the bend to further ensure against cracking due to embankment creep under reservoir pressure.

Grouting. To ensure the imperviousness of the foundation, a grout curtain up to 150 feet deep was required beneath the Zone 1 material, with a secondary grout curtain 25 feet deep upstream of the main grout curtain. Blanket grouting (holes up to 25 feet deep) of the foundation under embankment Zones 1 and 2 was specified to strengthen and seal the foundation where fracturing might be a problem. The specifications provided for slush grouting, if necessary.

Construction Materials. Zone 1, clay material, was required to be highly impermeable, plastic enough to withstand large deformations without cracking, and sufficiently workable to be used with heavy equipment.

Initial explorations showed that no clay material was located in the lake area. An extensive search was conducted to find suitable material that would have a Plasticity Index greater than 15 and have the Liquid Limits fall between 30 and 50. Bucket auger holes were drilled at Summit Valley and in dry lakebeds as far as 40 miles away. An unnamed lakebed 20 miles northeast of the Dam site was selected since it showed the desired clay quality (Figure 253).

Zone 2 material (silty sand), used for transition between pervious and impervious material, occurs naturally in the Harold formation as a weak sandstone lying between the ridges of the granite mountains in the area.

River deposits throughout the reservoir area were found to contain adequate material for Zone 3 (sand and gravel). Natural river deposits smaller than 18 inches generally produced a satisfactory filter between Zones 2 and 4 and between Zones 2 and 4A.

Zone 4A is the rolled, quarry-run, upstream, rockfill shell. Zone 4 was designed as a processed, coarse-graded, rolled rockfill to provide stability and free drainage downstream. Zone 5, the coarsest of all, was placed as the farthest downstream shell and was intended to be dumped because of its size. Riprap was required to be of a more select gradation and functions as slope protection for Zone 4A. Fines resulting from the production of Zone 4 and 5 material were used in Zone 3.

Stability Analysis. Embankment stability was analyzed by the infinite slope, sliding wedge, and slip circle methods. The soil properties used in the stability analysis are shown in Table 33.

Zones 1 and 2 and the Harold formation in the foundation were unable to immediately dissipate construction pore pressures. Because of this, unconsolidated undrained strengths for these materials were used for postconstruction stability determination.

Flow nets used in stability analyses assumed Zones 3, 4, 4A, and 5 were infinitely pervious, except during drawdown when Zone 4A was assumed to be as permeable as Zone 3. Zones 1 and 2 were assumed to have the same permeability with a horizontal-to-vertical permeability ratio of 16. For rapid drawdown, the upstream phreatic line was determined by Casagrande's approximate method with a minimum drainage time of six days.

Seismic Considerations. Design considerations for seismic activity necessitated placing large shells of rolled rockfill both upstream and downstream of the core, removal of all alluvial material under the Dam, and provisions of ample freeboard above the normal water surface. The large size dense rock used provided the Dam with highly stable zones as the major supporting elements. Removal of all alluvial materials eliminated any possibility of liquefaction of foundation materials during an earthquake. With the rolled rockfill shells, settlements and deflections due to earthquake shaking will be minimized, and the amount of slumping to be expected will not lower the crest to the elevation of the maximum probable flood water surface.

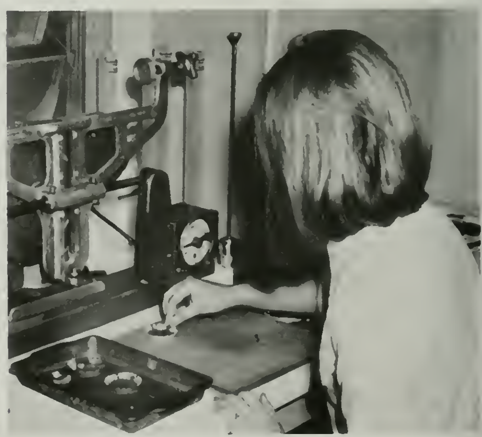

Figure 253. Determining the Plasticity Index Number of the Impervious Zone

TABLE 33. Material Design Parameters—Cedar Springs Dam

Material	Specific Gravity	Unit Weight in Pounds Per Cubic Foot			Static Shear Strengths θ Angles in Degrees Cohesion in Tons Per Square Foot					
					Effective		Total		Construction	
		Dry	Moist	Saturated	θ	C	θ	C	θ	C
Zone 1	2.78	105	126	130	24	0.2	16	0.2	4	1.5
Zone 2	2.70	117	131	136	35	0	16	1.0	*	*
Zone 3, Quarry	2.71	127	137	143	38	0	24	1.0	*	*
Zone 3, Streambed	2.71	127	137	143	40	0	26	1.5	*	*
Zone 4A	2.70	130	--	144	43	0	26	1.5	*	*
Zone 4	2.70	120	--	--	42	0	*	*	*	*
Zone 5	2.70	105	--	--	38	0	*	*	*	*
Foundation, Harold formation	2.69	124	--	141	36	0	27	1.5	23	1.0

* Free-draining material, use effective stress values.

Design criteria included the possibility of an earthquake seiche. Configuration of the reservoir is such that seiche waves probably will be of short duration. Design features providing for seiches include a substantial freeboard (23 feet) above maximum operating water surface, an ample zone of large open-graded rockfill (plus 2-foot sizes) on the downstream slope of the Dam, and a paved road on the crest of the Dam.

Settlement. Laboratory consolidation testing was performed on Zone 1 material to determine the amount of settlement that could be expected from the consolidation of the central impervious core. Tests were run on this material at 1% above optimum moisture content compacted to 95% of maximum dry density. The consolidation tests on these as-compacted specimens showed an initial buildup of a large amount of compression in a short period of time followed by a slow uniform rate of compression buildup for the duration of the test. Tests that were postsaturated indicated that approximately 1% settlement would take place in the dam core. The dam embankment was increased in height by approximately 1.5% to compensate for long-term settlement.

Instrumentation. Test apparatus and facilities installed in the foundation and embankment of Cedar Springs Dam (Figures 254 and 255) made possible the gathering of data to study its structural behaviour during construction. A monitoring program now has been made part of the regular maintenance procedure. Instruments which were installed include piezometers, slope-indicator casings, surface settlement points, and crest settlement monuments.

Six pneumatic piezometers were installed in the foundation of the Dam and 15 in the embankment. These piezometers are observed from instrumentation terminal well No. 1 on top of the Dam. Seven foundation piezometers are observed from instrumentation terminal well No. 2 (Figure 256).

Three slope-indicator installations were made for Cedar Springs Dam and are located upstream of the centerline of the Dam.

Seventeen crest settlement monuments are located on the crest of Cedar Springs Dam, and 15 surface settlement points are located on the upstream slope of the Dam.

A seepage measuring weir has been constructed at the toe of the main dam to monitor and record embankment and foundation seepage.

Two instrumentation terminal wells were constructed under the main dam contract. The wells are located on the dam crest. These structures contain terminal facilities for the foundation and embankment piezometers and include the measuring panels, air tanks, filters, and all appurtenant equipment for operation of the piezometer system.

A reservoir head-level recording and indicating system was installed and consists of a servo-manometer, two mechanical counters, a gas purge tube, and a sensor tip anchored to a concrete bench mark monument. The system records and indicates reservoir water surface level from elevation 3,230 feet to elevation 3,370 feet.

Debris Barriers

Description. Debris barriers were placed upstream on either side of the deeply cut San Bernardino Tunnel approach channel. They were designed to protect the tunnel approach channel from debris originating in Miller and Cleghorn Canyons (Figures 257, 258, and 259). The debris capacity was based on either a 10-year rainstorm coming after the San Bernardino Tunnel approach channel was excavated and before reservoir filling, or a 100-year storm following 10 normal storm years after the reservoir has been filled.

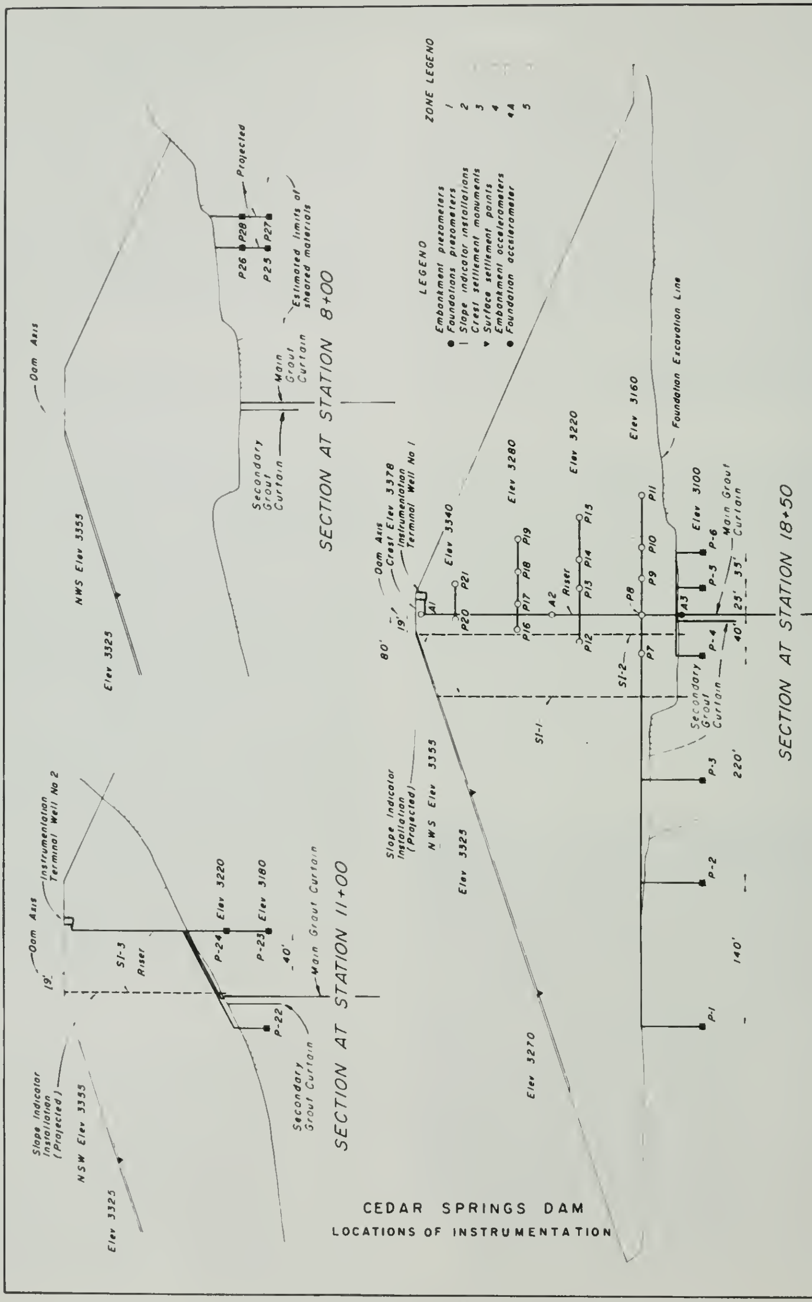

Figure 254. Location of Instrumentation—Sections

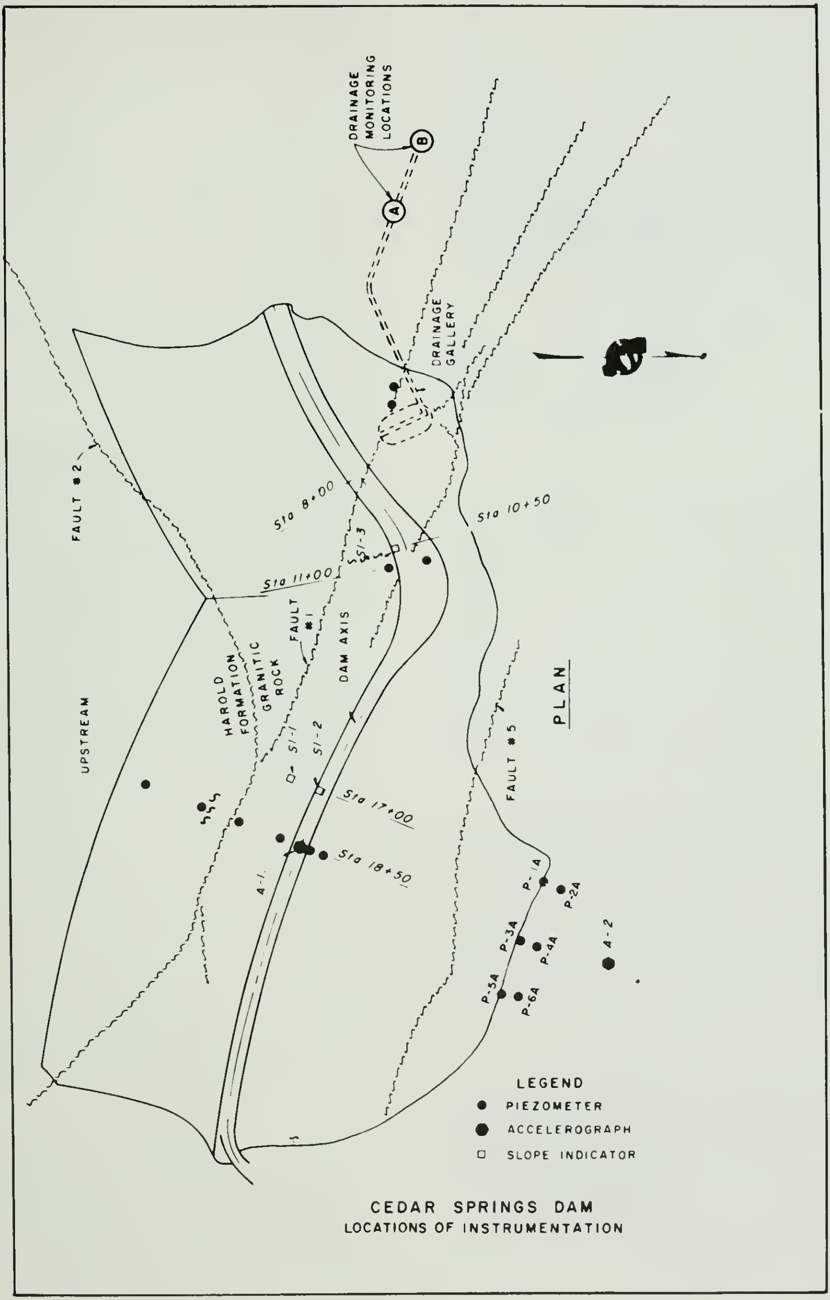

Figure 255. Location of Instrumentation—Plan

Figure 256. Installation of Piezometer Tubing

Miller Canyon. This barrier (Figure 257) is L-shaped with a crest length of 980 feet at elevation 3,304 feet. It was built of random fill material excavated from the San Bernardino Tunnel approach channel and has a downstream rock face. The crest width is 30 feet, and upstream and downstream slopes are 2½:1. A rock-covered, 130-foot-wide, spillway section was included in the right abutment to pass floodflows around the San Bernardino Tunnel approach channel.

Cleghorn Canyon. This 580-foot-long barrier (Figure 258), with crest at elevation 3,300 feet, was built of random fill excavated from the San Bernardino Tunnel approach channel and has a rockfill downstream face.

San Bernardino Tunnel Approach Channel

The San Bernardino Tunnel approach channel (Figure 259), a 10-foot-wide trapezoidal section, connects the intake tower approach channel, constructed under the intake tower contract, to the reservoir floor at elevation 3,230 feet. The San Bernardino Tunnel approach channel was excavated in granitic material beneath Borrow Area B-3 at Station 7+50 and joins the intake tower approach channel at Station 40+00. At Station 28+00, the material encountered changed from granite to Harold formation.

Drainage Gallery and Access Tunnel

Exploration Adit. An exploration adit was constructed in the left abutment of Cedar Springs Dam to (1) determine the site geology by paralleling the fault inside the left abutment of the Dam, (2) cross the main shear zone in two locations for inspection and fault monitoring, (3) determine tunnel support requirements, (4) create a permanent drain between the fault and the dam embankment, and (5) provide access to a valve chamber that would convert the diversion tunnel to an outlet works.

This adit, which is now the drainage tunnel and part of the access tunnel, was constructed 1,480 feet in length as an unlined horseshoe-shaped section. The first 710 feet were aligned along the centerline of the Cedar Springs Dam access tunnel and served as the top drift for that tunnel. Under the main dam contract, the remaining 770 feet of exploration tunnel in the left abutment of the Dam was converted to a concrete-lined drainage gallery (Figures 260 and 261).

A standard 4-inch by 4-inch, wide-flanged, structural-steel rib was selected as the support to carry the varying loads. Rib spacing was on 4-foot centers.

Drainage Gallery. The main drainage gallery prevents ground water buildup in the left abutment of the Dam, particularly the area upstream of a local shear zone that runs east-west. This shear zone acts as a barrier that would develop a hydrostatic head on the downstream toe of the left abutment if no provisions were made for drainage.

A drainage pocket exists in the downstream foundation formed by the core, the left abutment, and the knoll. Positive drainage was provided through the left abutment by adding a short tunnel to the drainage gallery which terminates under the embankment rockfill at the base of the pocket. An 8-foot-long concrete plug was designed for the end of the gallery which had openings large enough to carry water freely but small enough to prevent the rockfill from passing through.

The drainage gallery is a concrete-lined horseshoe-shaped section 5 feet by 6½ feet. The length of the gallery is 723 feet with an invert slope of .005 toward the access tunnel. Drain holes, approximately 100 feet in length, were drilled on 20-foot centers alternating between sets of four with two vertical and two horizontal holes and sets of four with all four holes at 45 degrees. The invert contains a collection ditch which permits the seepage to drain into one of two weir boxes. The first weir box is located directly above the outlet works tunnel and empties through a 14-inch drain hole into the outlet tunnel at the crown. The other weir box is located at the access tunnel entrance, Station 19+90, and empties into an outside drainage ditch.

The reinforced-concrete tunnel lining was analyzed as an elastic arch supporting a hydrostatic load of 50 feet, which was estimated to be the equivalent of 19 feet of rock load on the lining. This loading resulted in a concrete lining 12 inches thick, with only a nominal amount of temperature steel reinforcement.

Access Tunnel. Besides drainage, the main purpose of the access tunnel is to provide outside access to the outlet works gate chamber. Design was similar to the drainage gallery except for the length of the drain holes. Drainage from the gate chamber and part of the drainage gallery flows along the access tunnel floor to the portal where it is measured in a Parshall flume.

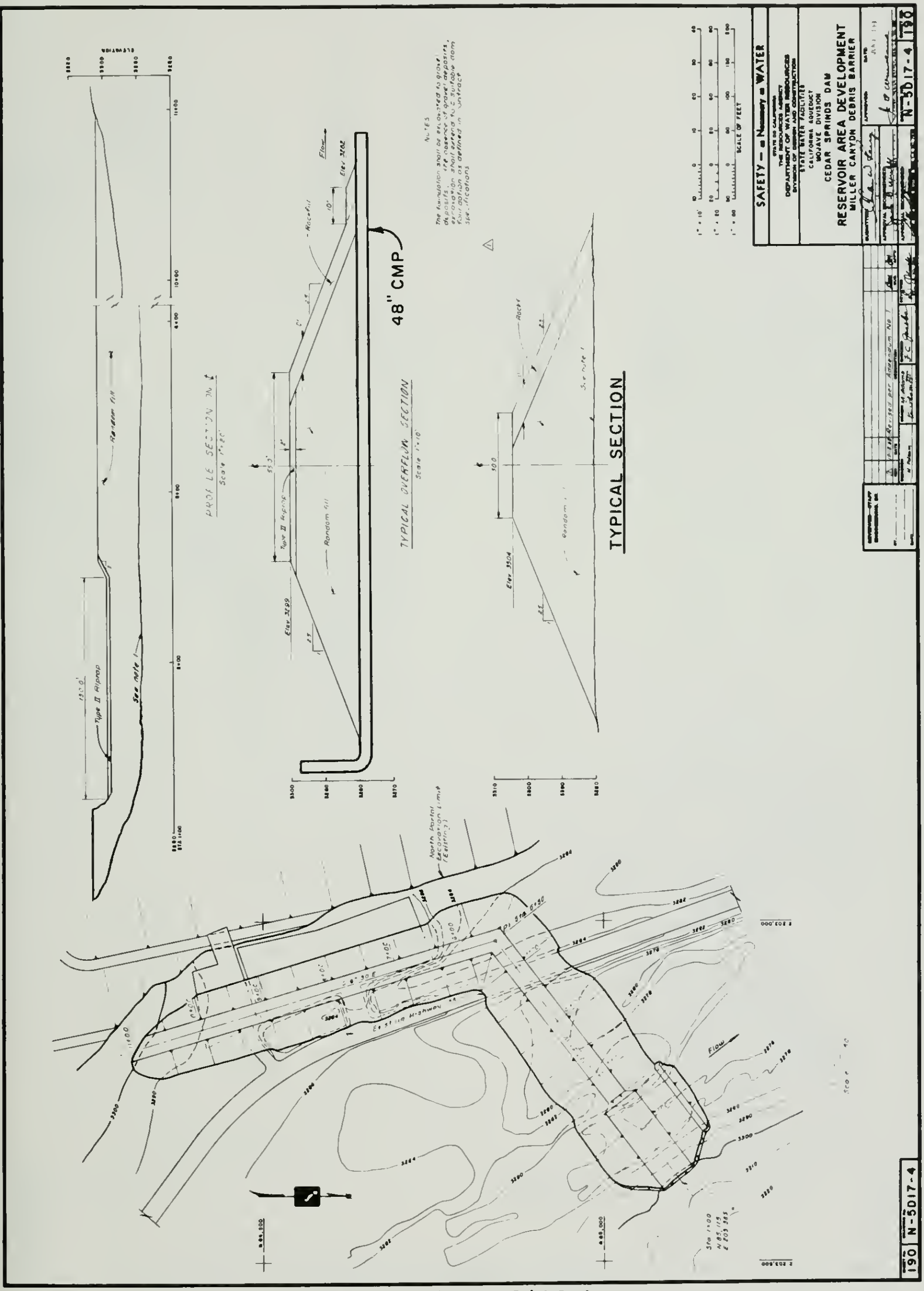

Figure 257. Miller Canyon Debris Barrier

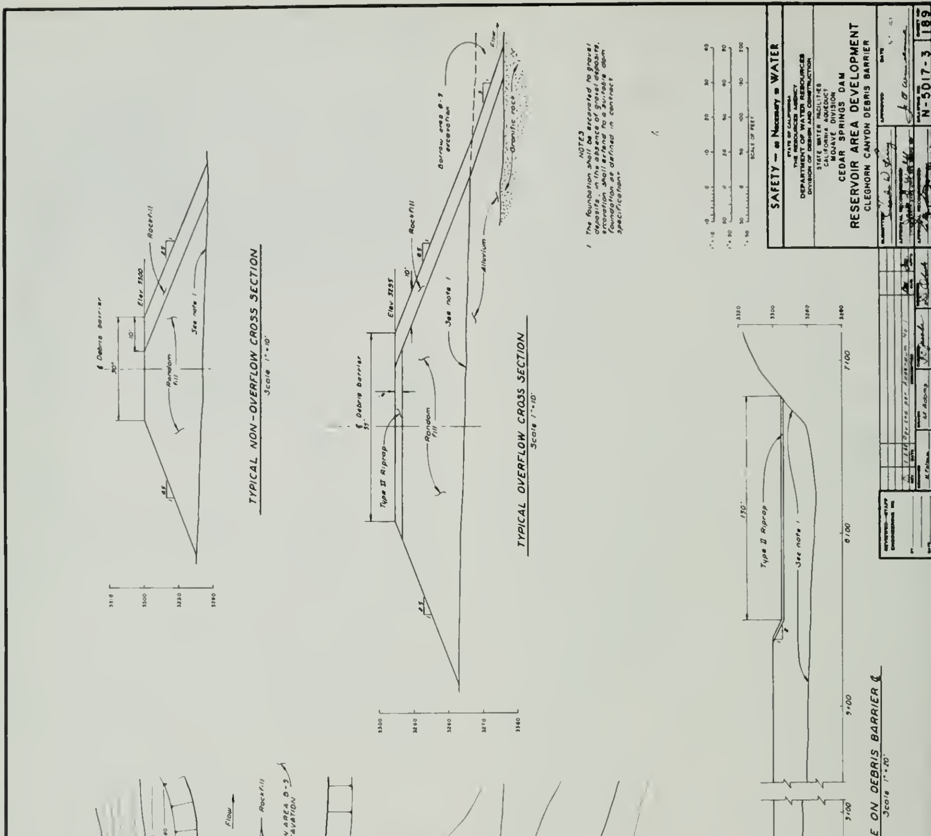

Figure 258. Cleghorn Canyon Debris Barrier

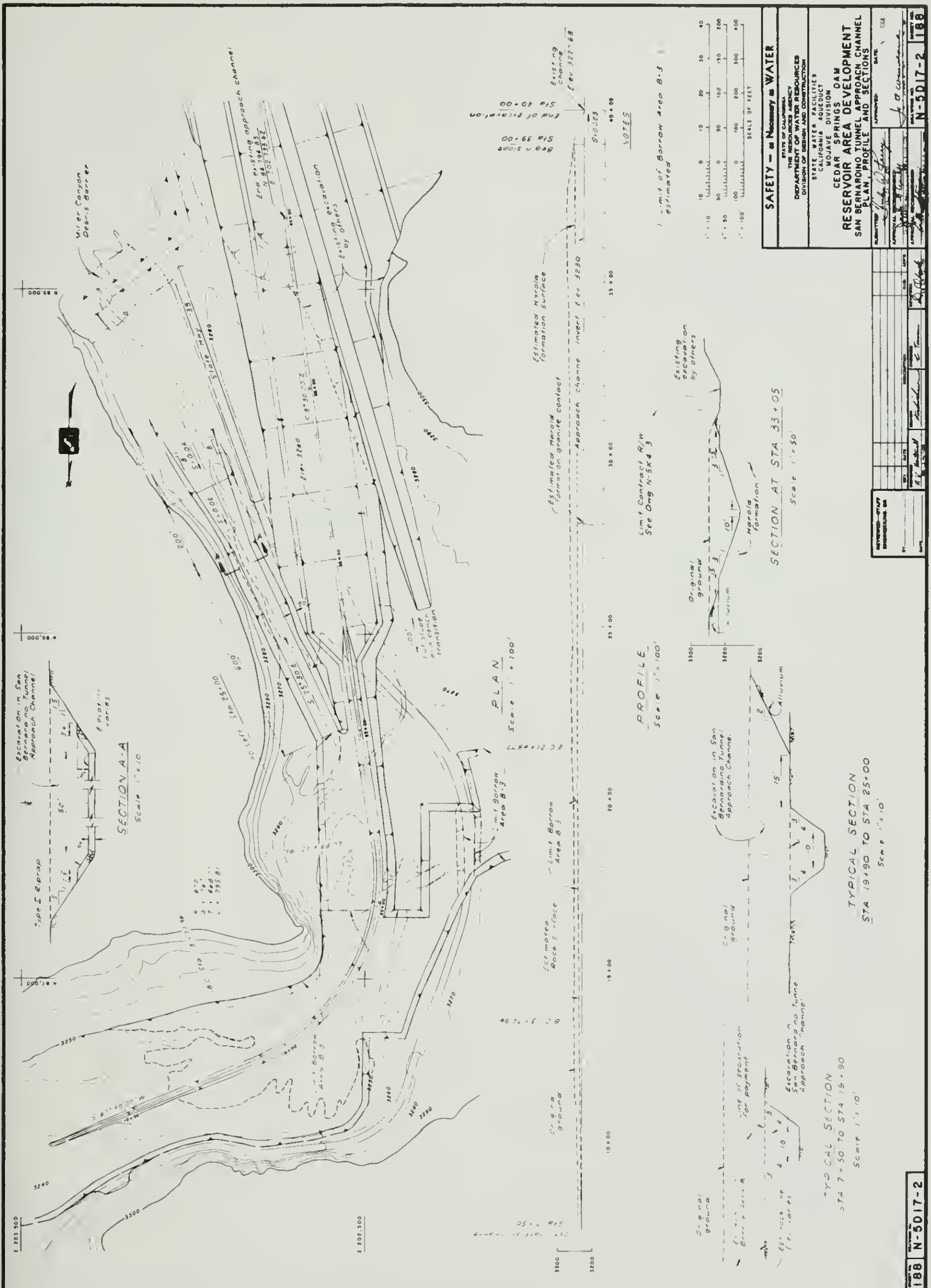

Figure 259. San Bernardino Tunnel Approach Channel

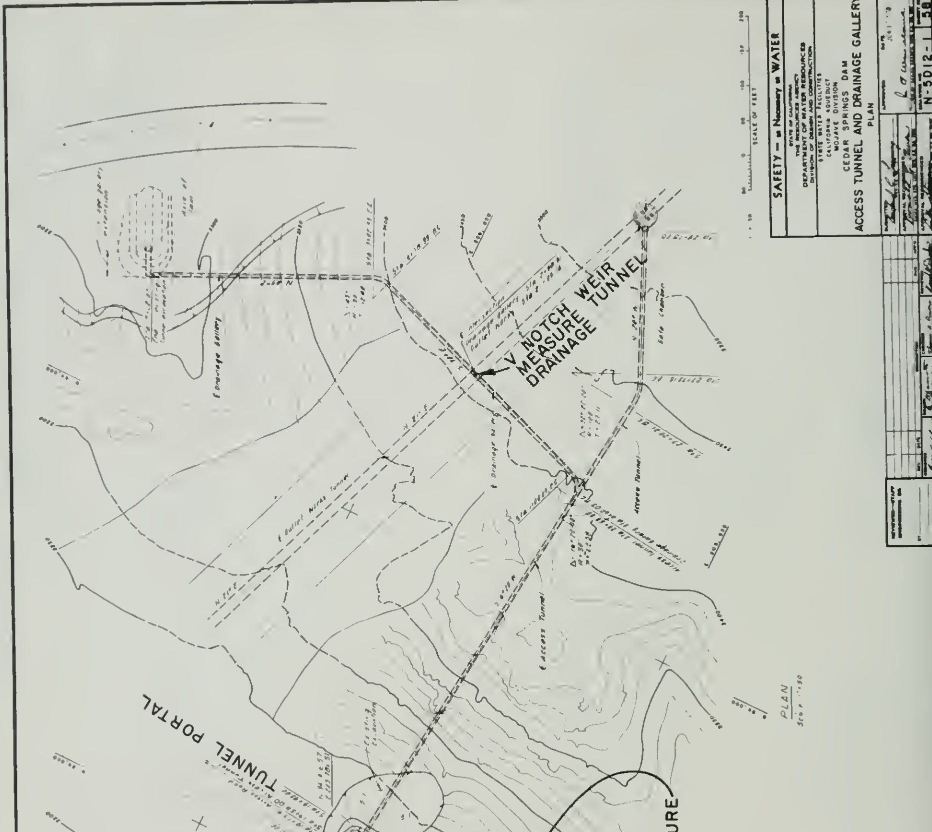

Figure 260. Access Tunnel and Drainage Gallery Plan

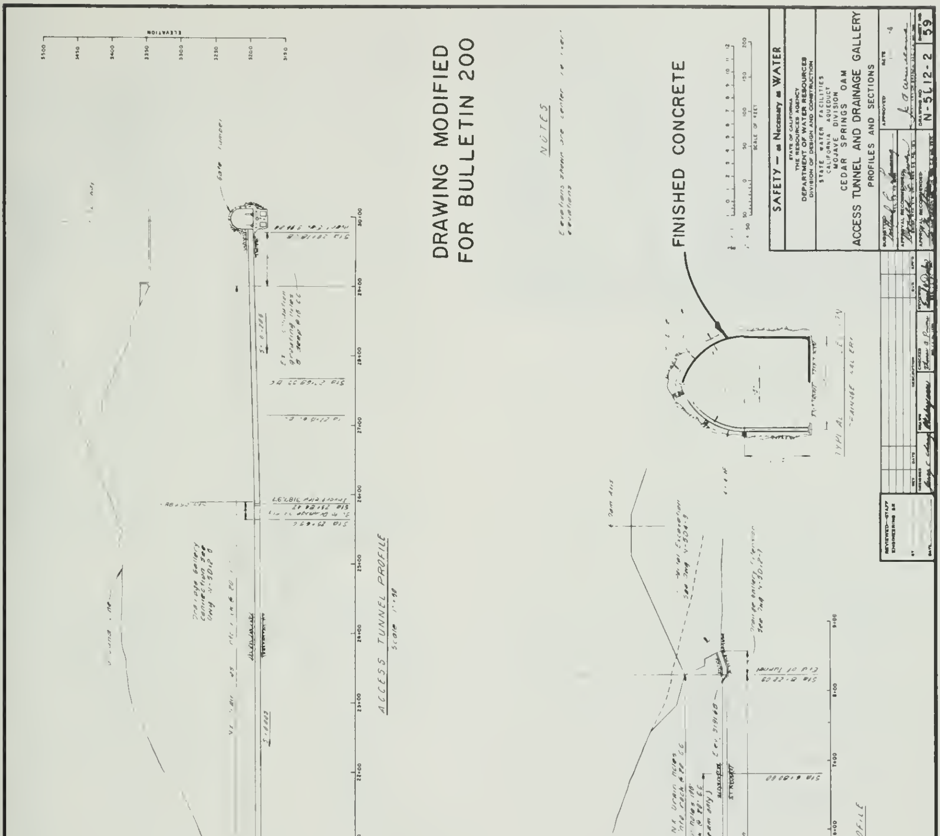

Figure 261. Access Tunnel and Drainage Gallery—Profiles and Sections

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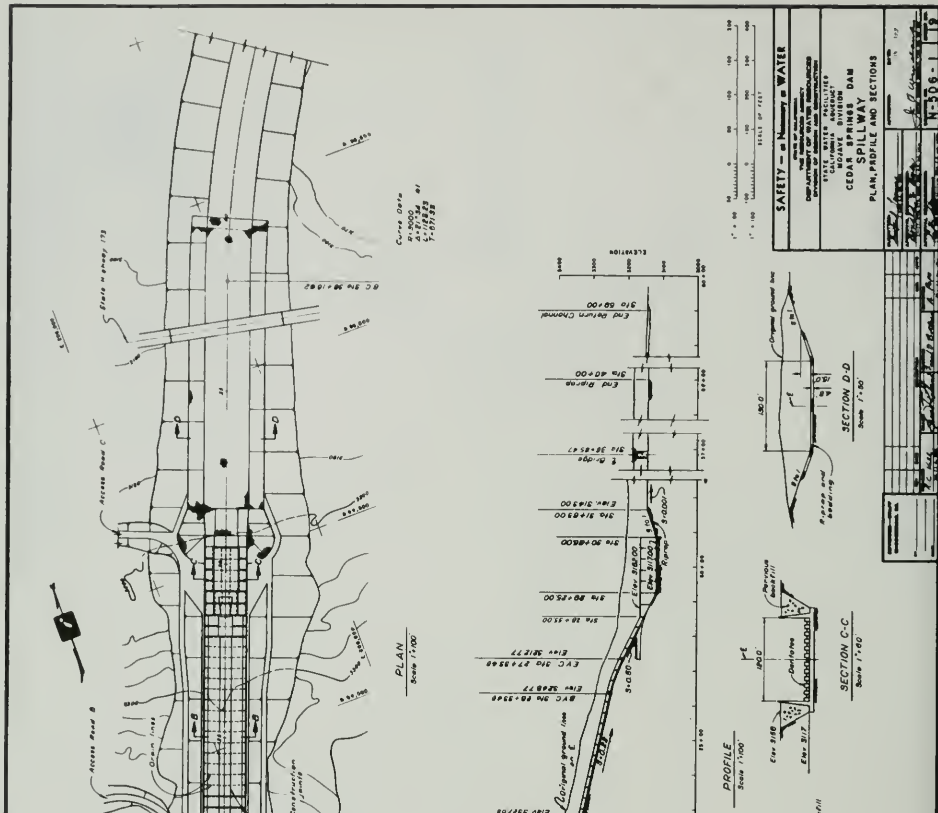

Figure 262. Spillway

Spillway

Description. The spillway (Figure 262) for Cedar Springs Dam is located on the left abutment, approximately 300 feet from the embankment, and was designed to convey all floodflows around the Dam site and back to the natural river channel. It consists of an approach channel, crest structure, chute, stilling basin, and return channel.

The approach channel to the spillway weir is trapezoidal in cross section and is 120 feet wide at the bottom with 1½:1 side slopes. The channel is unlined except for the first 65 feet immediately upstream of the weir. This reach is lined with concrete to block downstream leakage and reduce uplift.

The spillway chute is rectangular in cross section, concrete-lined, and 120 feet wide. Side walls vary in height from 10 to 13 feet. The outlet works tunnel enters the spillway chute just upstream from the stilling basin. The hydraulic jump-type stilling basin is 120 feet wide, is 160 feet long, and has cantilever side walls 45 feet high.

The spillway return channel is trapezoidal in cross section and is 130 feet wide at the bottom with 3:1 side slopes. A bridge carries State Highway 173 traffic across the channel downstream from the stilling basin.

Excavation. Spillway excavation upstream from Station 32+00 was in granitic rock. Downstream from that point, excavation was in alluvium, slopewash, and Harold formation. Except for that portion of the chute that crosses a small east-west fault (Station 22+70 to 23+60), the spillway is founded on hard rock. The excavation was all in open cut and had a maximum depth of 200 feet at the weir. The granitic rock portion was used in the dam embankment.

Backfill. Impervious backfill, which corresponds to the requirements for Zone 2 dam embankment, was placed behind the spillway approach and crest structure walls. This backfill prevents seepage around the spillway. The remainder of the walls were backfilled with pervious material.

Drainage. A drainage system is located under the spillway floor slabs and along the wall heels in the chute and stilling basin to help relieve uplift. The chute drainage system consists of lateral-perforated drain pipes placed in a herringbone pattern with the perforations down and surrounded by drain material (Figure 263). In the stilling basin, the drainage system consists of perforated drain pipes bedded in concrete in a rectangular pattern, with the perforations up and drain material placed over the pipe.

Hydraulics. The spillway was designed to carry the maximum probable flood with 5 feet of freeboard between the maximum reservoir water surface and the crest of the Dam. For design, it was assumed that, prior to the maximum probable flood, the reservoir would be completely full, the spillway discharging the

Figure 263. Perforated Drain Pipes in Spillway Chute

base flow of the stream (1,000 cubic feet per second), and all outlet facilities inoperable. The maximum routed outflow for these conditions is 32,250 cubic feet per second (cfs) with the reservoir water surface at elevation 3,373 feet.

The 120-foot-long spillway weir is ogee-shaped and has a 45-degree, sloping, upstream face to improve discharge efficiency. The downstream surface consists of circular arcs which approximate the profile of the lower nappe of the standard project design flood discharge of 21,000 cfs. This ogee shape was developed to prevent negative pressures that would cause cavitation on the surface of the weir for flows up to the maximum probable flood.

Wall heights for the chute vary from 10 to 13 feet and were chosen to allow for freeboard above the water surface profile of the maximum probable flood. The vertical curves of the invert of the chute were designed to be flatter than was required to conform to the trajectory of a free-discharging jet. This was done to insure that positive pressures are maintained on the chute invert.

The stilling basin is used to still both the spillway and the outlet works flows. Chute blocks are provided at the upstream end, and dentates are provided at the downstream end of the basin. Model studies showed that the selected basin would operate satisfactorily for all flows up to maximum probable flood. These model studies also verified the riprap size required, both adjacent to the stilling basin and in the downstream return channel.

The return channel is lined with riprap to Station 40+00 (Figure 262) just downstream from State Highway 173 bridge and is unlined from that point to the end. Riprap was not placed downstream from Station 40+00 because erosion of the channel in this reach would not have any adverse effects. Heavy riprap is provided at the stilling basin to protect the channel from turbulence caused by the hydraulic jump. Heavy riprap also is provided at the downstream end of the lined portion of the channel to provide a control section for tailwater if the downstream channel erodes.

Structural Design. The spillway approach channel walls between Station 17+47 and Station 17+87 (Figure 262) are slabs that are anchored to the excavated slope. The approach walls between Station 17+87 and Station 18+12 are a combination of anchored slabs and cantilever walls. This second section of walls affects the transition of the side slopes of the approach channel from 1½:1 to vertical and are backfilled with impervious material. Reinforcement is continuous through all the construction joints and waterstop was placed in the designated construction joints.

The weir is a concrete gravity structure anchored to the foundation with grouted anchor bars. Reinforcement is continuous and waterstops are in all of the construction joints.

Crest structure walls are cantilever with a portion of the weir acting as a wall toe. To prevent spillway leakage, the walls were backfilled with impervious material. Weakened contraction joints are provided in the crest walls at Station 18+12 and Station 18+44. Reinforcement is continuous through them and waterstop was placed in the joint at Station 18+12.

Spillway chute walls are cantilever and decrease in height as the water gains velocity. A transition of pervious backfill that meets Terzaghi's filter criteria was placed behind the walls to prevent drain water washout. The toe of the spillway walls forms a portion of the chute floor and was designed to have the water load offset the overturning moment of either the soil or water load. The longitudinal steel in the walls, except for four expansion joints, is continuous for the entire length of the chute. This unorthodox construction was included to prevent spillway wall breakage resulting in possible chute blockage in the event the fault crossing the spillway was to move.

Chute slabs are 15 inches thick and are anchored to the foundation. Where the chute crosses a local shear fault, shear keys are provided because grouted anchors alone would not be effective in the broken rock and soil (Figure 264). Longitudinal construction joints, with waterstop and continuous reinforcement, are placed at a spacing of approximately 33 feet. Lateral construction joints with continuous reinforcement were placed as required, except in the fault zone where a joint was placed at each shear key.

The outlet works transition to the spillway transfers the flow from the outlet works tunnel to the spillway chute. It consists of a rectangular box section of varying height and width. The width gradually widens from 20 feet at the tunnel portal to 35½ feet over a distance of 111 feet. The top of the sections is a slab 2 feet thick and was designed as a beam fixed at the ends, supporting a uniform load equivalent to 6 feet of water. This load accounted for the depth of flow over the spillway from the maximum probable flood plus an allowance for vibration and impact. The bottom slab varies in thickness from 2 feet at Station 27+96 to 3 feet at Station 28+55. This slab was designed as a beam fixed at the ends with a uniform uplift applied due to a water surface at the elevation of the spillway chute drains. This assumes the French drain below the bottom slab is plugged. The slab that covers the portal excavation at Station 27+96 was designed as a two-way slab fixed on three sides supporting a uniform construction load of 12 feet of wet concrete. Walls of the box section are essentially gravity walls and carry nominal reinforcement.

A transition section, 70 feet long and 120 feet wide, with walls varying in height from 10 to 45 feet connects the chute to the stilling basin.

The stilling basin is a rectangular structure 120 feet wide and 160 feet long, consisting of two parallel cantilever walls with a concrete floor between the toes of the walls. Its walls are anchored to the rock foundation and are divided into seven sections by contraction joints placed approximately 33 feet apart to minimize cracking due to temperature stresses. Floor slabs are independent of the wall sections and are anchored to the foundation with grouted anchor bars. Floor slabs between the cantilever walls were designed to withstand the uplift loads imposed on them from the hydraulic jump for the standard project flood. The end

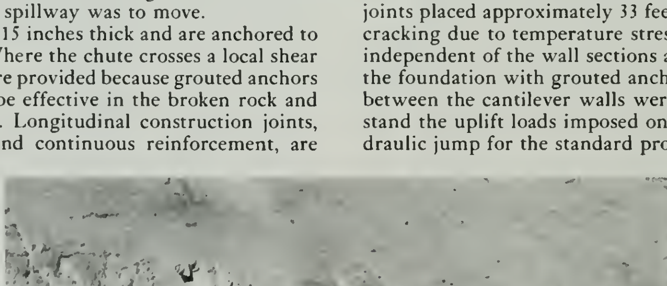

Figure 264. Shear Keys in Spillway Chute

slabs between Station 30+53 and Station 30+85 were designed to resist the dynamic effects of the flow impinging on the dentates. The slabs are divided by three transverse contraction joints located at Station 28+55, Station 28+90, and Station 30+53 and by three longitudinal contraction joints located on the centerline and at the toes of the wall sections. All contraction joints are waterstopped due to the high differential heads produced by the hydraulic jump.

Chute blocks before the stilling basin and dentates at the end of the basin were designed to resist the dynamic effect of the flow impinging on them.

Outlet Works

Description. The outlet works (Figure 265) consists of an intake tower, an upstream pressure tunnel, a gate chamber, a downstream tunnel that discharges into the spillway chute, and an air intake that also acts as an emergency exit. The primary purpose of the outlet works is to release normal reservoir inflow and emergency reservoir drawdown. During construction, the outlet tunnel was used to divert the river around the dam embankment.

The outlet works tunnel was driven through granitic rock which varies from moderately weathered to fresh. A local shear zone crosses the outlet works at approximately a right angle. The fault zone extends approximately from Station 22+70 to Station 23+50. Drain holes are located in the horseshoe tunnel downstream from the gate chamber to lower the hydrostatic head on the tunnel lining.

This was the first contract in which the Department specified that construction of all tunnels was to be by lump-sum bid item. This, in effect, placed all responsibility for tunnel support systems on the contractor.

Hydraulics. Downstream water rights for the Mojave River require that all natural inflow to Silverwood Lake be released. Downstream river releases between ½ and 23 cfs generally are made from the Mojave Siphon via the Las Flores Pipeline (Figure 248). Flows between 23 and 5,000 cfs usually are released through the outlet works. Flows above 5,000 cfs either are stored in the reservoir for later release or discharged over the spillway.

Flows through the outlet works up to 300 cfs are controlled by a 30-inch fixed-cone dispersion valve. The valve was designed to deliver the required flows at minimum operating pool, elevation 3,312 feet. Two 5-foot by 9-foot slide gates are used to control flows between 300 cfs and 5,000 cfs. Both the valve and the gates are located in the gate chamber (Figure 266).

Structural Design. The intake structure was designed to meet both construction and operation conditions. Loading conditions for construction were analyzed with and without backfill along with consideration for an earthquake acceleration of 0.25g horizontally and 0.125g vertically. The operating condition considered was with the reservoir water level at maximum operating pool, elevation 3,355 feet. Dead load consists of tower weight, submerged surrounding backfill, and weight of water. Earthquake force, location of resultant force, and allowable increase of normal working stresses for loads of short duration were considered the same as during construction.

The pressure tunnel upstream of the control gates consists of a steel-lined branching conduit 39 feet long, 26 feet of tunnel transition, and a circular tunnel 504 feet in length. The portal section was designed to resist a combined external load of 43 feet of backfill plus a hydrostatic load due to the reservoir at maximum operating pool. Between Stations 12+27 and 14+00, the tunnel section was designed for separate internal and external hydrostatic loads due to the reservoir at maximum operating pool. The internal load was considered separately to allow for rapid initial filling of the reservoir when insufficient time is available to develop the full external water load. Between Stations 14+00 and 17+81, the tunnel section was designed for an external hydrostatic load with the reservoir at maximum operating pool.

The transition section and rectangular conduits between Stations 17+81 and 18+35 were designed for separate internal and external hydrostatic loads when the reservoir was at maximum operating pool. Concrete lining for the gate chamber was designed to withstand full hydrostatic pressure head when the reservoir was at maximum flood pool, elevation 3,373 feet; a rock load due to 20 feet of rock; and a concentrated load of 20 tons acting through any of the ten lifting hooks. A steel liner extends from the beginning of the rectangular conduit at Station 17+07 to the upstream end of the gate body and from the downstream end of the gate body to Station 18+84. Its primary purposes are to protect the concrete lining from cavitation by high-velocity flows and prevent seepage between the upstream pressure tunnel and the downstream horseshoe tunnel. The upstream steel liner and stiffeners were analyzed as a rectangular frame and designed for a hydrostatic head of 178 feet.

The horseshoe tunnel at Station 18+67 was designed as an integral part of the gate chamber. The load transmitted from the gate chamber was assumed to be uniformly distributed onto the tunnel arch. This load was combined with an external hydrostatic load that decreased from a hydrostatic head of 178 feet at Station 18+84 to 25 feet at Station 19+17. The remaining downstream portion of the tunnel was designed for an external hydrostatic load of 25 feet above its invert. External hydrostatic pressure on the tunnel lining in the later reach of tunnel is relieved by drain holes drilled through the concrete lining into the surrounding rock.

The reach of tunnel crossing the shear zone is provided with extra reinforcement and a thicker concrete section.

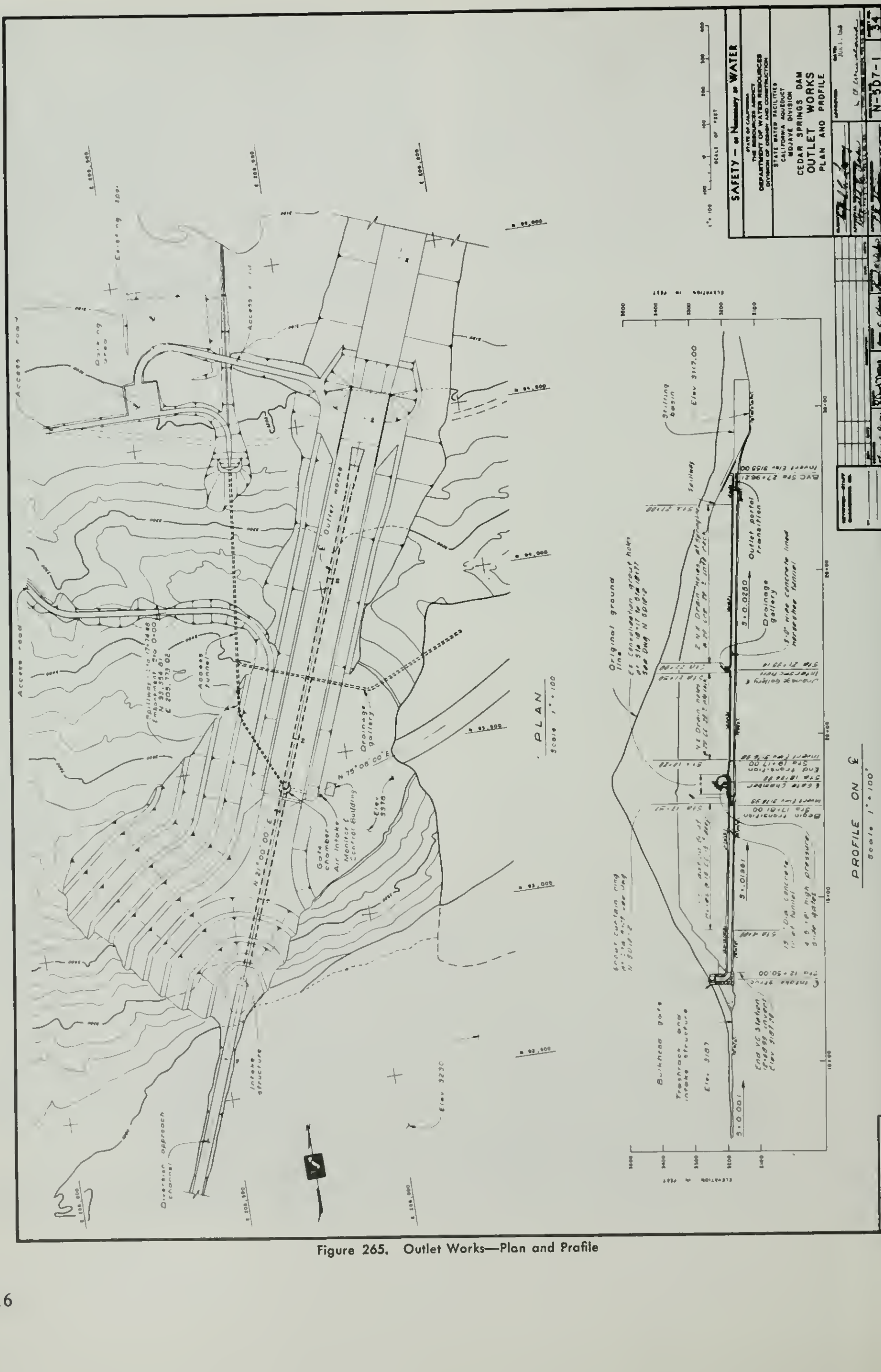

Figure 265. Outlet Works—Plan and Profile

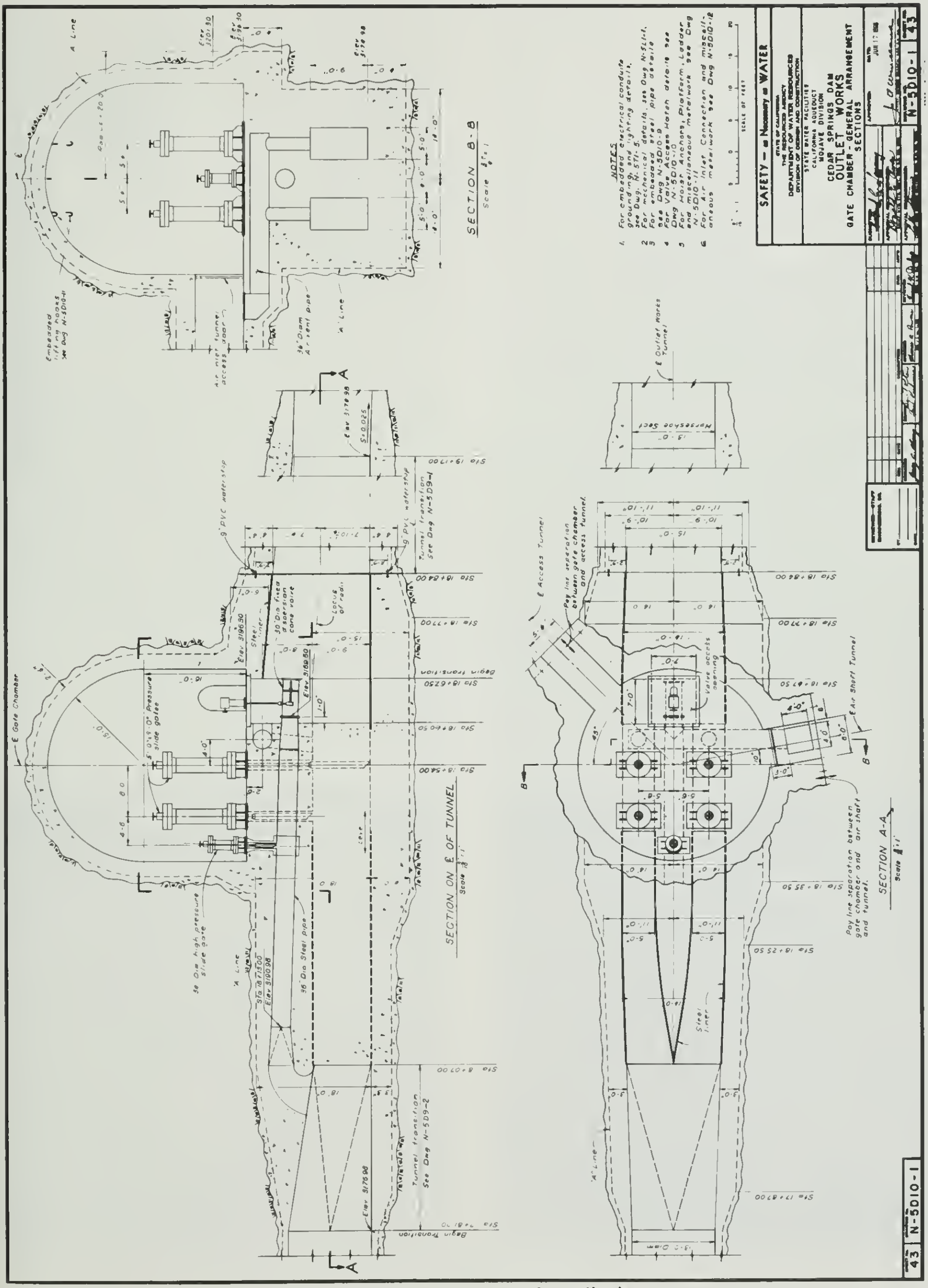

Figure 266. General Arrangement of Gate Chamber

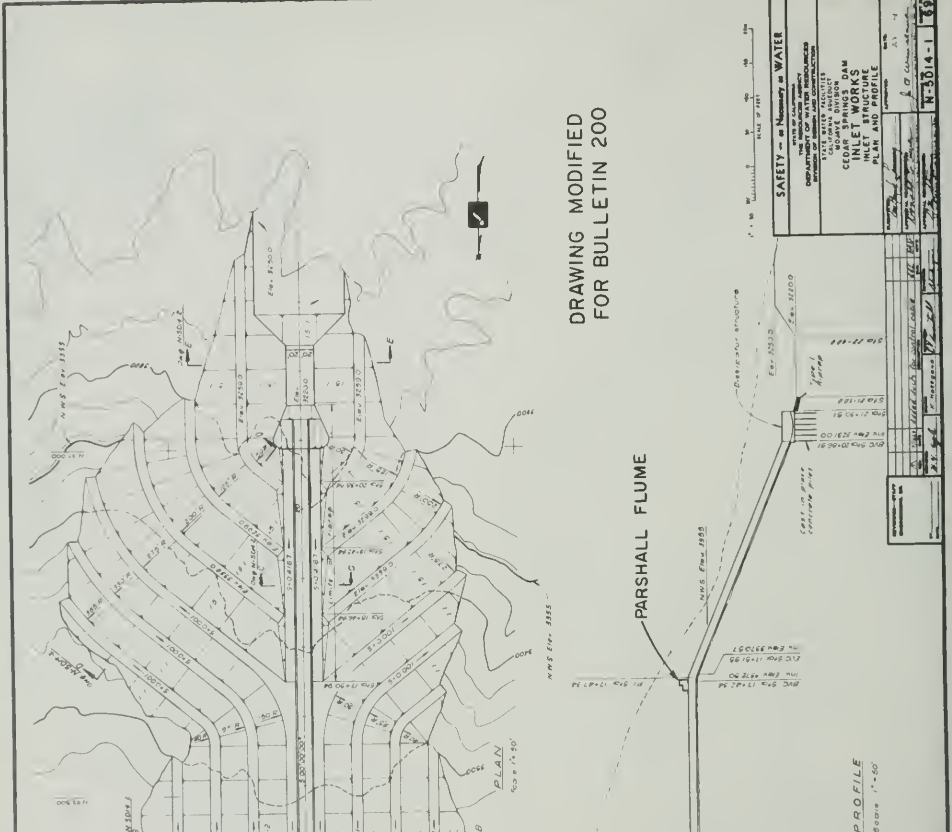

Figure 267. Inlet Works—Plan and Profile

Mojave Siphon Inlet Works

Description. Project water is supplied to Silverwood Lake through the Mojave Siphon, a portion of the California Aqueduct which crosses under the Mojave River. The Mojave Siphon enters the Lake on the left abutment ridge approximately 1,500 feet west of the spillway and consists of a transition structure, concrete chute, and a flip-bucket energy dissipator (Figures 267 and 268). The concrete chute through the ridge is on a slope of approximately 0.1% toward the reservoir. It follows down the ridge to a flip-bucket energy dissipator at elevation 3,231 feet. This chute was modified in 1974 at the grade change to form a Parshall flume section to measure inflow. The picture was taken during initial reservoir filling but does not show the Parshall flume (Figure 269).

The siphon barrels have transitions to square conduits that open into an open rectangular section which narrows to the 20-foot chute width. A center pier was constructed in the open transition to stabilize discharges and provide slots for a bulkhead gate to allow dewatering of the individual siphon barrels. Only the 1,200-cfs barrel of the Mojave Siphon has been completed. The second barrel with a capacity of 800 cfs has not been scheduled for construction. The summit of the inlet works, elevation 3,373 feet, occurs at the end of the open transition. This allows 2 feet of freeboard above the maximum reservoir stage when the maximum probable flood is routed through.

Hydraulics. To deliver the full design discharge of 1,990 cfs, a maximum energy gradeline is required at the end of the Siphon at an elevation of 3,388 feet. The chute width of 20 feet was established to meet the above requirement while allowing the chute invert to be placed above the maximum reservoir stage.

Subcritical flow is maintained in the upper portion of the chute, with the flow passing through critical depth at the chute vertical curve. A vertical-curve radius was selected to ensure positive pressure at all points on the invert.

Four feet of freeboard was provided on the walls at the siphon outlet and 2 feet for the remainder of the chute. A flip-bucket energy dissipator was used at the chute terminus to eliminate erosion in the vicinity of the structure during initial filling of the reservoir and in cases of extreme drawdown.

Structural Design. The closed portion of the transition structure was designed for the following loading cases:

1. External fill load to elevation 3,378 feet plus a surcharge of 2 feet to account for vehicular loads and internal hydrostatic head to elevation 3,386 feet.
2. External loads above only, with no internal hydrostatic head.

The open section of the transition was designed as a "U" channel section with the same external fill and internal hydrostatic design loads.

A bulkhead gate was designed for the maximum water depth while one barrel is operating (12 feet). Yield strength of the material is not exceeded if the gate is loaded to its top (13 feet).

Weakened-plane contraction joints are located at 25-foot intervals along the upper chute and 30-foot intervals down the steep portion. Expansion joints are located at 100-foot intervals along the entire chute. The upper portion of the chute was backfilled with pervious material and was provided with drains, while the lower portion of the chute was backfilled with riprap and bedding to a depth of 8 feet. The chute was

Figure 268. Flume and Chute

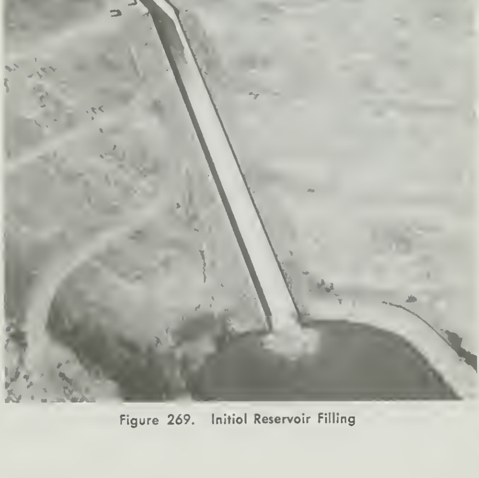

Figure 269. Initiol Reservoir Filling

designed as a "U" section with the following loading cases:

1. No water in chute, saturated backfill, uplift equal to 50% of fill height.

2. Water in chute, saturated backfill, no uplift.

The flip-bucket energy dissipator was partially backfilled with riprap and bedding material was used to stabilize the lower chute section.

Loading on the structure includes:

1. Fill pressure taken as an equivalent fluid weighing 50 pounds per cubic foot.

2. Hydrodynamic load of the design discharge on the flip bucket.

3. Uplift due to the maximum tailwater surface with supercritical flow within the dissipator.

Cast-in-place concrete piles were used under the structure to resist uplift and to support the structure should erosion or softening of the foundation take place.

Las Flores Pipeline

Las Flores Pipeline (Figure 248) provides water to the downstream water right owners and to the Department's operation and maintenance buildings. It is a buried, mortar-lined, 18-inch-inside-diameter, welded-steel pipeline that diverts water from the Mojave Siphon and delivers it to an energy dissipator structure located adjacent to relocated State Highway 173 below Cedar Springs Dam. Components of Las Flores Pipeline include the steel pipe, two valve vaults, two blowoff structures, and an energy dissipator structure.

Hydraulics. The 23-cfs design flow of Las Flores Pipeline was calculated from the maximum capacity of an irrigation system that was removed to construct the Dam. One of the design requirements was to deliver water downstream of the Dam at an elevation high enough to be diverted into an existing Las Flores Ranch ditch. The terminal end of Las Flores Pipeline was designed with a river turnout so that no water rights were established or denied by replacing parts of the Las Flores Ranch's previous irrigation system.

Valve Vaults. There are two underground, concrete, valve vaults located on Las Flores Pipeline. One is located at Mojave Siphon and the other at a high point in the Pipeline just east of the return channel. They contain the emergency, regulatory, vacuum relief, and air valves. Pervious material was backfilled against the vault walls. The vault has an aluminum roof which can be taken off should removal of a valve be necessary.

Blowoff Structures. There are two blowoffs: one at the West Fork of the Mojave River and one at the return channel. They consist of a standard AWWA 18-inch tee, a blind flange bolted to the tee, a 4-inch steel riser pipe welded to the blind flange, and a 4-inch steel gate valve screwed onto the riser pipe.

Energy Dissipator Structure. The impact-type energy dissipator was constructed as a box with an internal beam baffle. Water leaving the energy dissipator structure is turned into the Las Flores distribution pipe to flow by gravity to Las Flores Ranch. An overflow weir returns excess flow to the Mojave River via a small riprapped channel.

Mechanical Installation

Functions of the mechanical features are to control the release of water to the Mojave River and to provide the essential station services for operation and maintenance personnel and for safeguarding the operation of the outlet works.

Outlet Works. In the gate chamber (Figure 266) of the outlet works, there are two 5-foot by 9-foot, regulating, high-pressure, slide gates; two 5-foot by 9-foot, emergency, high-pressure, slide gates; a 30-inch-diameter fixed-cone dispersion valve; and a 36-inch-diameter, emergency, high-pressure, slide gate.

The emergency gates, installed upstream of the regulating gates and the fixed-cone dispersion valve, are used for isolation purposes. They will shut off the flow if the regulating gates or the fixed-cone dispersion valve malfunction and also will permit normal maintenance and repair of the downstream waterways, regulating gates, and fixed-cone dispersion valve.

Controls for the 36-inch-diameter shutoff valve (slide gate) and the fixed-cone dispersion valve, which are in tandem, were designed to move each individual valve to either a fully open or fully closed position by a single push-button command. Each valve may be stopped in any position and then moved in either direction. The shutoff valve cannot be operated unless the fixed-cone dispersion valve is closed, and the fixed-cone dispersion valve cannot be operated unless the shutoff valve is open.

Slide Gates. Although the slide gates are of two different sizes and serve two different functions, all five slide gates, for reasons of economics and uniformity, were constructed under the same specifications with a few minor exceptions. The guard gates do not have an air vent for air admission to the downstream side of the gate leaf nor gate creep adjustment, and the regulating gates do not have a 2-inch bypass line across the gate body. The major components of the gates are the gate body, the seats, the gate leaf, and the operator.

The gate body is of welded-steel construction and was designed to resist the internal water pressure or external pressure caused by reservoir seepage. The body is embedded in concrete, except the bonnet cover section. The bonnet cover section was designed to withstand the full reservoir head. The 5-foot by 9-foot guard and regulating gates are bolted together at their faying surface and "O" rings added for watertightness. All bolted joints in the bodies become a permanent assembly after the gates are embedded, except the bonnet joints, which remain accessible.

The seats, which are renewable, are bronze and are bolted to the body to function as top and side seals.

The gate leaf also is of welded-steel construction and is secured to the gate stem by a nut, tightened against an adjusting nut on the stem. The leaf is fitted with bolted bronze seats on the top and sides and is overlain with stainless steel on the bottom edge to function as the bottom seal. The seats bear on the corresponding seats in the body.

The seating surfaces are metal-to-metal and their watertightness depends on their close contact when forced together by the hydrostatic pressure and by the downward force of the moving parts. A small amount of leakage is expected.

The bottom seals of the two 5-foot by 9-foot, regulating, slide gates and the 36-inch-diameter emergency gate were changed to rubber seals. This change was made to reduce water leakage and prevent damage to the lip area of the leaf.

The sliding seating surfaces are lubricated by means of the centralized, automatic, lubricating system in the gate chamber. Application of lubricant to the seating surfaces before each operation provides a lubricating film to reduce friction, lessen the possibility of scoring, and improve the degree of sealing obtained.

The operator is a hydraulically driven cylinder which is bolted to the gate bonnet. The cylinders for the 5-foot by 9-foot gates have a 26½-inch diameter with an 8-inch-diameter rod and, for the 36-inch-diameter gate, the cylinder has a 13-inch diameter with a 4-inch-diameter rod. A hydraulic system installed in the gate chamber provides power for the operators.

To mechanically hold the gate leaf in the fully open position, a handwheel is provided at the top centerline of the cylinder for screwing a stud into the top end of the piston rod, thus holding the stem piston and the gate leaf in a fixed position. If, in an emergency or by accident, hydraulic pressure is applied above the piston when the stud is engaged, the stud will break and allow the gate to close. Stud breakage should occur at approximately 150 pounds per square inch (psi) and does not result in damage to the equipment. The cylinder must be dismantled and a new stud installed after breakage occurs.

A mechanical, gate-leaf, position indicator is operated by a vertical rod attached to a stuffing box in the bonnet cover. Position of the gate leaf can be observed in the gate chamber by a pointer on the rod and an attached scale. Position of the regulating gate leaves also can be read remotely by means of the rotary shaft encoders and remote display unit.

Gate-leaf creep adjustment is provided for the regulating gates. The mechanisms to achieve the adjustment are part of the hydraulic system which provides power to the cylinder operators.

Before adoption of two slide gates and one fixed-cone dispersion valve to control the release of water, a large fixed-cone dispersion valve, fixed-wheel gate, and butterfly valve also were considered. They were rejected chiefly because of higher overall cost.

Based on agreements with the water users and technical considerations, the gates were designed according to the following criteria:

Design Flow and Head Criteria

1. Two 5-foot by 9-foot regulating gates shall pass 300 to 5,000 cfs at 173 feet of head and 8,000 cfs at 200 feet of head under flood condition.
2. All five gates shall be capable of operating under a maximum static head of 200 feet.
3. The gate body for all gates shall be capable of withstanding 77 psi of static internal pressure and 77 psi of static external pressure plus all additional surge pressure which may result from gate operation.

Operation Criteria

1. The regulating gates shall control the flow rate with an accuracy of approximately $\pm 2\frac{1}{2}\%$ at all gate positions within the normal range of regulated flows.
2. The gate shall be operable both locally in the gate chamber and remotely from the monitor control building.
3. Opening and closing time for each 5-foot by 9-foot gate shall be 15 minutes and for the 36-inch-diameter gate it shall be 3 minutes.
4. If required, two gates may be operated simultaneously.
5. Both local and remote indication of regulating gate position shall be provided.

Design Stresses and Loading Criteria

1. The design stresses shall be governed by the applicable provisions of the ASME Boiler and Pressure Vessel Code, Section VIII. Where a material is not listed in the Code, a design stress of ⅕ of the ultimate or ⅓ of the minimum yield strength of the material shall be used.
2. A seismic loading of 0.3g shall be included in all design loads.
3. All parts shall be designed in consideration of buckling. No part shall have a l/r ratio of more than 80, where l is defined as the unsupported length of the part and r is the "least radius of gyration" of the part.
4. Gate-leaf deflection shall not exceed 0.005 of an inch.

The rectangular configuration of the 5-foot by 9-foot slide gates was based on the optimum width-to-height ratio of 0.5:0.6 published in the U. S. Army Corps of Engineers' design manual, "Hydraulic Design of Reservoir Outlet Structures", and the Tennessee Valley Authority's Technical Report 24. The 36-inch-diameter slide gate was determined by configuration of the pipe required for the fixed-cone dispersion valve. It should be noted that the gate dimensions refer to the cross-sectional area of the passageway of the gate. The cross-sectional area was based on the flow to be passed through at a specified head and a discharge coefficient of 0.7.

The design of the slide gates, in many respects, followed the design for those gates installed at the Department's Del Valle Dam and reservoir. The gate leaves were constructed in a welded-box-girder form for rigidity. The bottom of each gate was tapered at 45 degrees. Model studies made by the Corps of Engineers indicated that a 45-degree lip was the best configuration to reduce vibrations, downthrust, negative pressures, and turbulence.

Fixed-Cone Dispersion Valve. This 30-inch valve is a cylindrical-type, free-discharge, regulating-cone valve commonly known as either a fixed-cone dispersion valve, a hollow-cone valve, or Howell-Bunger valve. The major components of the valve and operator are the cylindrical valve body, sliding gate, worm gear units, bevel gear drive assembly, screw operator assembly, body sleeve, and valve operator.

Seating is accomplished by a fixed seat attached to the central cone in the body and a movable seat, which is part of the sliding gate. To open the valve, the gate is moved upstream by the screw assembly which, in turn, is rotated by the worm gear units on the sides of the valve. These are driven by two sloping shafts through the bevel gear drive assembly to the input shaft, which is driven by the valve operator.

The valve was designed based on the following:

1. The 30-inch-diameter fixed-cone dispersion valve shall pass a minimum of 305 cfs at a minimum total discharge head of 120 feet.
2. A maximum static head of 185 feet was specified.
3. The valve shall be operable both locally in the gate chamber and remotely from the monitor control building.
4. Opening and closing time shall be three minutes in each direction.
5. The valve and the companion flange shall be designed in accordance with all applicable requirements of the ASME Boiler and Pressure Vessel Code, Section VIII, Unfired Pressure Vessels, unless otherwise specified.
6. A liberal factor of safety shall be used throughout the design of the valve and its components. For components for which allowable stress state or factor of safety is not established, the factor of safety based on the minimum ultimate tensile strength of the material and most adverse conditions to be encountered is not less than the following:

Cast Steel	6
Cast Iron	10
Forged Steel	6
Stainless Steel	6
Bronze	6
Structural Steel Plate (for brackets and supports)	5

The design of the body and the ribs comply with the following minimum requirements:

1. Under maximum head, all stress combinations fall within a maximum shear stress theory of failure envelope having coordinate boundaries of $\frac{1}{6}$ of the minimum tensile strength or $\frac{1}{4}$ of the minimum yield strength of the material, whichever is less.
2. Internal pressure corresponding to maximum head acting on the body shell plus a differential pressure of 15 psi acting across the ribs shall not produce any stress combination which will fall outside of a minimum shear stress theory of failure envelope having coordinate boundaries of the endurance limit of the material. The endurance limit shall be defined as a stress equal to 45% of the minimum ultimate tensile strength of the material.
3. The minimum body shell and rib thickness are 1 inch.

The complete drive mechanism, including shafting, was designed for the following conditions:

1. Under normal operating torque, all stress combinations are within the maximum shear stress theory of failure envelope that has coordinate boundaries which provide the minimum factors of safety as specified earlier in this section.
2. Under a stalled motor torque at maximum operating torque, plus the torque due to the inertia of the rotating parts, all stress combinations in the drive fall within a maximum shear stress theory of failure envelope having coordinate boundaries of 75% of the minimum yield strength of the material.
3. The maximum twist of the shaft under normal operating conditions will not exceed one degree in a length equal to 20 shaft diameters.

The valve was made of various types of material to best suit the requirements for the different valve components. The materials used are as follows:

Part	*Material Used*
Valve body; bolting flange; radial ribs; valve gate; stiffening ring; seat ring	A285 Firebox Grade C, Plate Steel; A515, Firebox Grade 65, Plate Steel
Companion flange	A27, Grade 70-36 cast steel normalized and tempered

The dispersion-cone surface is made straight to prevent cavitation due to the creation of low pressure areas on the surface or seat. Cavitation and pitting of valve parts will occur if the valve is operated at less than 10% of its gate opening.

Two pilot tubes installed at the leading edge of one rib and one static pressure tap installed in the valve body are used to measure the flow through the valve.

The valve is operated by a SMB-00 limitorque valve control operator having a 15-foot-pound, 3-phase, 60-hertz, 440-volt, 1,800-rpm, weatherproof, continuous-duty motor. The torque limit switch and the limit switches automatically will stop the motor operator in the event of an obstruction in the gate path or the failure of a limit switch.

Shop tests consisted of a hydrostatic test on the valve at 120 psi for 60 minutes for any evidence of valve distress, a leakage test at 80 psi for 30 minutes with leakage not exceeding one-half gallon per minute, and a stroking test of three full opening and three full closing strokes. Field tests consisted of three full opening and three full closing strokes in the installed position.

These tests were conducted to ensure that the equipment was free from defects prior to acceptance by the Department.

Lubrication System. A centralized lubrication system was installed in the gate chamber to meet the lubrication requirements for the five slide gates and the fixed-cone dispersion valve. The system works automatically in conjunction with the gate and the valve operating systems through an electrical interconnection. Whenever a gate or valve is energized for operation, the electrical system simultaneously actuates the lubrication system. The lubrication system also is capable of being manually controlled.

The system consists of a central station, piping, valve manifolds, valving, control mechanism, and accessories.

Service Facilities. The service facilities consist of (1) heating and air-conditioning system, (2) ventilation system, and (3) domestic water supply system. These facilities are mainly in the monitor and control building and gate chamber of the outlet works. They are provided for maintaining and safeguarding the operation of the outlet works.

Heating and Air-Conditioning System. A central heating and air-conditioning system is provided in the offices and equipment room of the monitor and control building to maintain comfortable working conditions for personnel and proper ambient temperature for the electronic equipment.

The cooling unit has a capacity of 37,000 BTU per hour and is supplied with 208-volt, single-phase, 60-hertz power from the motor control center and uses R-22 refrigerant. The electric heater has a capacity of 48,000 BTU per hour and is supplied with 480-volt, 3-phase, 60-hertz power from the same center. The fan used in both the heating and the cooling phases has a capacity of 1,460 cubic feet per minute (cfm) and is supplied with 120-volt, single-phase, 60-hertz power. Selection of the components for the air-conditioning system was based on (1) an inside cooling temperature of 80 degrees Fahrenheit and an inside heating temperature of 65 degrees Fahrenheit, and (2) consideration of reliability and minimum maintenance since the electronic equipment in the equipment room is highly sensitive to heat and moisture.

Ventilation System. The ventilation system consists of individual fans and blowers installed in the gate chamber, drainage gallery, and two instrumentation wells for the protection of equipment.

The system was designed to provide a sufficient number of air changes to avoid any danger of asphyxiation in addition to removal of heat and moisture. An airflow switch was installed at the drainage gallery to operate an alarm which will warn individuals entering the gallery when the blower is not providing proper ventilation.

The blower located in the gate chamber is designated as Blower 1. It ventilates the gate chamber by drawing air into it through the access tunnel and then discharging it through the ventilation shaft. The blower located at the remote end of the drainage gallery, designated as Blower 2, takes its air from the gallery and discharges it into the outlet works tunnel. Both of these fans are rated at 1,000 cfm, ½-inch S.P., 1,174 rpm, and are provided with 480-volt, 3-phase, 60-hertz power.

An 8-inch centrifugal exhaust fan is provided to ventilate each instrumentation terminal well. These fans are rated at 175 cfm and operate on 120-volt, 3-phase, 60-hertz power.

Domestic (Potable) Water Supply System. The potable water system provides treated water in the monitor and control building for domestic use.

Raw water is taken from the Las Flores Pipeline and collected in a sump in the valve vault at Station 19+16. It is pumped to the treatment plant located in the monitor and control building through a steel pipeline.

The treated water is stored in a 350-gallon tank in the same building ready for dechlorination and distribution. The treated water has turbidity of 1 part per million and chlorine level not below 0.2 of a part per million. The water pressure at the fixture is between 20 to 50 psi.

Las Flores Pipeline. Mechanical equipment for the Las Flores Pipeline is located in two valve vaults at Stations 1+98 and 19+16. The equipment includes a shutoff valve, flow metering, control valves, and an air-vacuum valve. They are required for proper operation of, and measuring the flow through, the Pipeline (Figure 248).

Shutoff Valve. The shutoff valve is located in the first valve vault at Station 1+98 and is used for closing off all flow to Las Flores Pipeline during periods of maintenance and repair. It is an 18-inch, 150-pound, manually operated, rubber- seated, butterfly valve and is flanged at both ends.

A butterfly valve was selected because it is more economical and simpler in operation and maintenance than gate and plug valves.

Flow Metering. The flow-metering equipment installed in the second valve vault at Station 19+16 is used to measure all flow through Las Flores Pipeline. The metering equipment consists of a 14-inch and a 6-inch flow tube in parallel complete with pressure tap connections and flow transmitters.

The flow transmitters, which transmit the electrical signals to the monitor and control building for recording and totalizing the flow, are mounted on the vault wall.

Control Valves. Two control valves, one 12-inch and one 6-inch, were installed downstream of the 18-inch and 6-inch flow tubes inside the valve vault at Station 19+16. They regulate the flow through Las Flores Pipeline and control the amount of flow through each flow tube for proper metering.

Air-Vacuum Valve. There are two 2-inch air-vacuum valves installed in Las Flores Pipeline to release air during pipeline filling and to admit air into the Pipeline if a vacuum condition occurs. The air-vacuum valves are 150-pound rated with both ends flanged. Each valve is complete with a 2-inch, 150-pound, flanged, solid-disc, gate valve for shutoff.

Electrical Installation

Normal electrical service is supplied by a utility company and standby electrical power is provided by an engine-generator set for all electrical features at the Cedar Springs Dam and reservoir facilities. Electrical power is fed to the intake gate chamber and access tunnel, Las Flores outlet vault, outdoor lighting, and instrumentation well buildings from the control building.

Operating Equipment. All the equipment necessary for remote monitoring and control of Cedar Springs Dam and reservoir are contained in the control building. Data, status, and alarm information are transmitted to the Castaic Area Control Center. Remote closing and opening of the intake slide gate is provided from the control building and Castaic Area Control Center by means of the digital comparator module, supervisory equipment, and site processor.

Local controls and annunciations for Las Flores outlet and cone valves are located at the valve sites and duplicated at the control building. All other local controls are only at the sites although duplicate annunciations are provided at the control building.

Emergency Engine-Generator Set. An engine-generator set is provided as an emergency source of power for the operation of the entire facility, including the slide-gate hydraulic system, fixed-cone dispersion valve operator, lubrication system, air-conditioning system, ventilating fans and blowers, instruments, controls, and lights. The engine-generator set is located in the engine-generator room of the monitor and control building.

The engine is a 4-cycle, 6-cylinder, water-cooled engine. It has overhead valves, 339-cubic-inch displacement, and produces 105-brake-horsepower at 1,800 rpm. The engine is equipped with a built-in flyball close regulating governor for speed regulation. The engine operates satisfactorily with liquid propane gas (LPG) fuel at an elevation of 3,400 feet.

The generator was designed for a continuous output of 55 kW at 0.8 power factor, 277/480 volts, 3 phase, 4 wire, 60 hertz at 1,800 rpm.

The fuel for the engine-generator set is stored in a 500-gallon storage tank which is sufficient to meet the full-load fuel requirements of the engine for a period of 96 hours.

Engines using diesel, natural gas, and LPG fuels were investigated. An LPG-fueled engine was selected because of the low initial cost of the unit and also because LPG fuel is easily stored and remains usable indefinitely. LPG-fueled engines are easier to start under extreme temperature conditions, which is the case at Cedar Springs Dam site.

Equipment. The following electrical equipment was installed at the Cedar Springs Dam and reservoir facilities:

Control Building

1. Motor control center
2. Servo-manometer-type water-level indicating and recording instruments
3. Supervisory and site processor consoles
4. Telephone system equipment
5. Engine-generator set
6. Miscellaneous electrical equipment such as lighting, air-conditioning and heating, ventilation, and potable water-pump starter

Gate Chamber

1. Hydraulic consoles and lubricating equipment
2. Motor control panels
3. Air-shaft ventilation equipment
4. Access tunnel lighting

Instrumentation Well Buildings

1. Seismic equipment
2. Panel boards and transformers
3. Lighting and ventilation equipment

Las Flores Outlet Vault

1. Flow-metering equipment
2. Motor control panels
3. Lighting

A single-line diagram of Cedar Springs Dam facilities is shown on Figure 270.

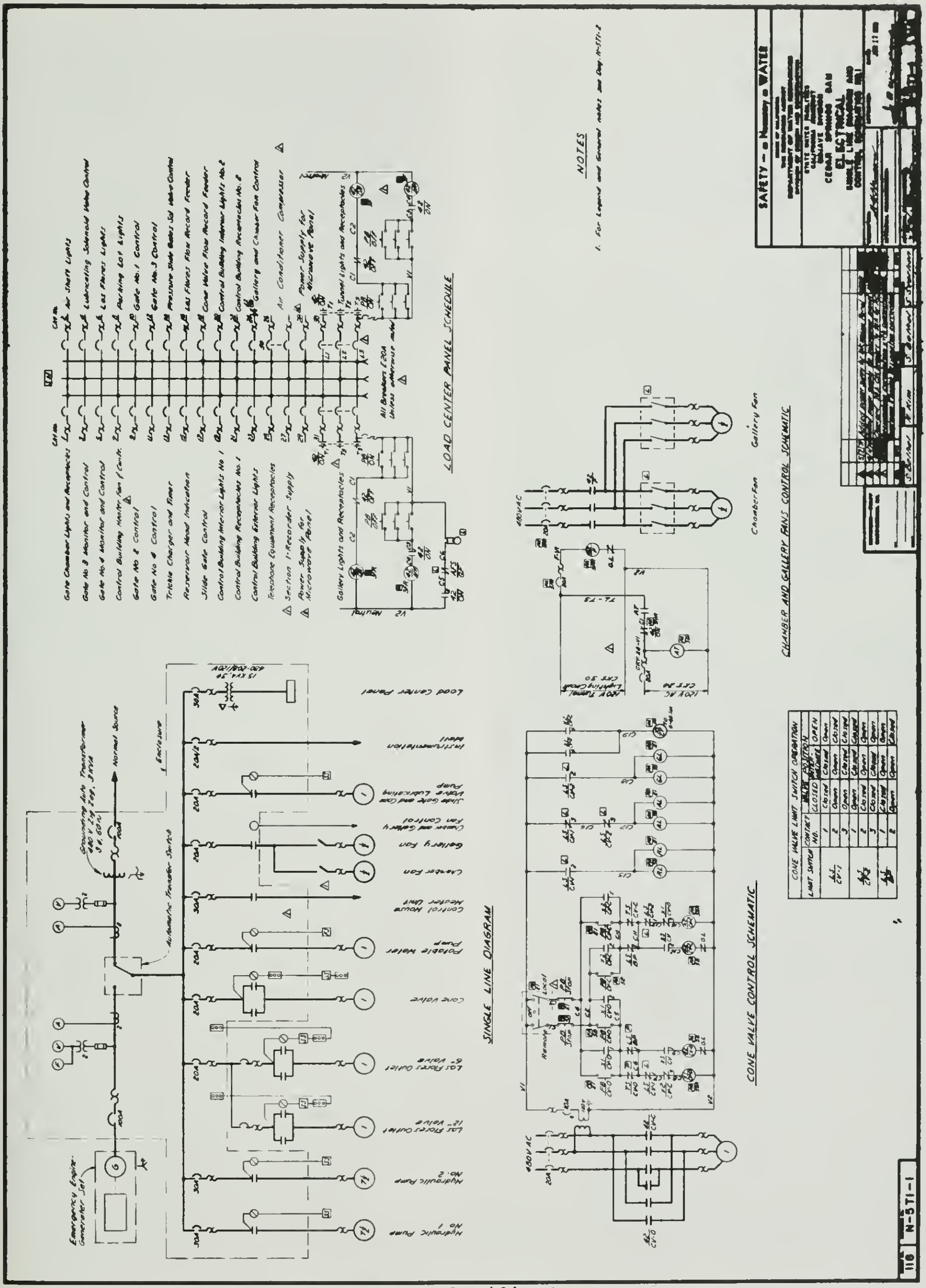

Figure 270. Control Schematics

Construction

Contract Administration

General information about the major contracts for the construction of Cedar Springs Dam and appurtenances is shown in Table 34. The principal contract, Specification No. 68-30, included the Dam, relocation of forest service roads, part of State Highway 173, access roads, inlet works, a portion of the Mojave Siphon, and the outlet works tunnel and control structure.

Exploration Adit

Portal excavation for the exploration adit, which later became the access tunnel, began in September 1967. A trench was dug extending north from the portal across State Highway 138 (Figure 248). Temporarily, the Highway had to be relocated around the north end of the trench while a conduit pipe for access to the proposed tunnel was being placed in the trench and the Highway rerouted over the top. Tunnel excavation began in October 1967 and was completed in March 1968. Two jacklegs were used to drill holes 4 to 7 feet deep. The drilling pattern was a 26-hole burn cut and a 14- to 26-hole "V" cut. Muck was loaded into muck carts with an air-powered mucker.

Tunnel support from the portal to Station 26+03 (Figure 260) consisted of temporary timber sets placed on centers of 4 and 8 feet. Wood sets consisted of 10-inch by 10-inch caps and 6-inch by 8-inch posts. A 36-foot section between Stations 22+48 and 22+84 was driven 14 feet wide and left unsupported. It was used as a car pass area. From Station 26+03 to the end of the adit, W4X13 (4WF13) horseshoe steel sets were used for support. While driving the adit, support generally followed the face by less than 10 feet but, in places, as much as 70 feet of tunnel was left unsupported for several days.

Diversion and Care of River

During the first winter (1968–69), two delays amounting to 24 calendar days were a result of two storms. The first storm caused flood conditions so severe the entire vicinity was declared a disaster area by the Governor of California. All construction work, including excavation for the spillway and reservoir clearing, was halted by this storm. A second storm

Figure 271. Embankment Construction

closed all routes to the job site and washed out haul roads to the spillway. Major excavation operations were resumed on March 3, 1969.

After April 1, 1969, construction began on a small dam at the site of the Las Flores diversion structure to bypass summer streamflow around the construction area. A 36-inch-diameter corrugated-metal pipe was laid through the Dam site. Since no rainfall occurred during the summer of 1969, the system remained dry. Because streamflow in the Mojave River was insufficient to meet construction needs, the Department developed a 2,250-gallon-per-minute well in the Mojave River, 9 miles downstream, and pumped water to the area of use.

During the first phase of dam construction, the River was diverted through the Dam site west of the right abutment to allow completion of right abutment excavation and foundation grouting. By October 1969, right abutment excavation, core trench excavation, and foundation grouting had progressed sufficiently to permit commencement of construction of the flood diversion channel located along the right abutment (Figure 271).

The completed flood channel was triangular in shape and was formed by the excavated right abutment slope and a 5:1 filled slope on the left. The invert of the channel was at approximate elevation 3,165 feet.

By April 1970, the first-stage embankment reached elevation 3,305 feet. Because the contractor demon-

TABLE 34. Major Contracts—Cedar Springs Dam and Appurtenances

	Exploration Adit	Interim Water Supply Facility	Cedar Springs Dam	Stage II Pump	Quarry Irrigation System
Specification	67-36	67-50	68-30	69-13	71-09
Low bid amount	$251,130	$395,353	$25,376,422	$34,238	$24,084
Final contract cost	$293,822	$431,628	$28,148,654	$31,538	$25,280
Total cost-change orders	$8,634	$38,561	$429,060	$62	$925
Starting date	7/31/67	10/2/67	11/12/68	4/25/69	5/21/71
Completion date	4/29/68	4/9/68	8/18/71	7/1/69	8/13/71
Prime contractor	Clifford C. Bong & Co.	R. L. Thibodo Construction Co.	Morrison-Knudsen Co.	Pylon, Inc.	Moulder Bros.

strated that he could complete the outlet works, the spillway past Station 23+90, the spillway stilling basin, and the dam embankment to elevation 3,300 feet by November 1, 1970 (Figure 251), he was allowed to close the flood channel and proceed with construction of stage two of the dam embankment.

In an emergency, the outlet works are capable of passing the standard project flood with a maximum water surface at elevation 3,295 feet. After November 1970, floodflows were stored behind the Dam or diverted through the outlet works.

Dam Foundation

Dewatering. Seepage of water into and along the bottom of the excavation for the Dam was controlled by using drain trenches and sumps. During stage 1 construction, the water was pumped from the sumps into either the diversion channel or the pipeline supplying Las Flores Ranch. Zone 1 and 2 embankment contact materials were placed over the dam foundation in the dry.

Excavation. Stripping for the Dam began on the left abutment knoll in December 1968. The contractor used a large tractor with a slope board. Two additional tractors were added to the left abutment stripping operation. The metamorphic rock exposed was severely weathered and friable to a depth of 10 feet. Below the severely weathered zone, the rock was moderately to slightly weathered, weak to moderately strong, and closely fractured. The stripping operation on the right abutment was accomplished by a large tractor with a double-shank parallelogram ripper.

Alluvium with an average thickness of about 30 feet and a maximum thickness of about 50 feet, including some slopewash at the base of both abutments, was excavated from the channel. The slopewash was 10 to 20 feet thick in both the left and right channel areas and generally consisted of silty sand.

Minimal excavation was required in the Harold formation under Zones 3 and 4A, since it was as fresh on the surface as at depth. Further excavation was required in the granite under Zones 1 and 2 to remove all weathered rock to a depth sufficient to ensure that the foundation would be impervious after grouting. This resulted in the formation of a core trench, the excavation of which was accomplished using conventional excavating equipment without the need for any blasting.

Ground water was encountered in the alluvium excavation from 50 feet upstream of centerline to 200 feet downstream of Station 21+75 (Figure 250). The water table was at elevation 3,167 feet. In addition, small seeps occurred as a result of cuts between elevations 3,180 and 3,185 feet in fractured rock associated with faults 1 and 5 (Figure 254).

Cleanup and Preparation. Prior to placing Zone 1 contact material in the core trench, all loose rock, soil, and waste materials were removed using hand labor and a backhoe. This was followed by a thorough cleaning using air and/or water jets immediately prior to placement of the contact material. No slush grouting was performed as it would have resulted in a rigid membrane inconsistent with the general design concept of a flexible embankment. Grout caps were placed in excavation where rock was too highly fractured or too soft to adequately anchor pipe nipples and hold the required grouting pressure.

Foundation cleanup prior to placing Zone 2 material was the same as described for Zone 1. Embankment for Zone 3 was placed upon either Harold formation or granitic rock. After excavating overlying sands and gravel, loose material was either recompacted or removed prior to placing Zone 3. Boulders larger than 18 inches excavated from the dam foundation were utilized in Zones 4A and 5 of the embankment. No further cleanup was required. Foundation cleanup prior to placing Zone 4 material was the same as described for Zone 3.

Little foundation cleanup was required for Zone 4A because most of it rests upon the upstream Zone 3 embankment. In those areas where Zone 4A was placed upon the dam foundation, all overlying topsoil, slopewash, and alluvial materials were removed prior to placement. Foundation preparation prior to placing Zone 5 was the same as described for Zone 4A.

Handling of Borrow Materials

Description. Materials for construction of the embankment were obtained from mandatory excavations and designated borrow areas. All materials from the mandatory excavations, meeting the requirements of the specifications, were used in the embankment in lieu of wasting.

Impervious. Borrow Area A (Figure 245) was the source of impervious material for Zone 1 of the Dam. A total of 1,182,000 cubic yards of material was excavated to supply material for the core. The borrow source was an unnamed dry lake just north of State Highway 18 and approximately 20 miles from the Dam site. The contractor built his own 45-foot-wide haul road from the borrow area to the specified stockpile area in the reservoir. The dry lake materials were prewetted before removal from the borrow area for dust control purposes.

Elevating belt loaders towed by tractors were used to excavate the clay to a depth of 3 to 5 feet in each pass. Maximum depth of excavation in the borrow area was 29 feet. At the Dam site, tandem bottom-dump trucks deposited the material into windrows in the reservoir area. The windrows were spread into lifts for moisture conditioning, using tractors. A sprinkler system was used to apply water to the dry material. Discs and scarifiers mixed the material to assure complete and uniform distribution of the water. After moisture conditioning to optimum moisture content, the material was picked up and hauled to the Zone 1 embankment in scrapers.

Pervious and Slope Protection. Silty sands and gravelly sands for Zone 2 material were obtained in part from areas located in the reservoir. Of approximately 483,000 cubic yards of Zone 2 material, 108,300 cubic yards were excavated from the west side of the valley floor and the remainder from required excavation for the spillway return channel and inlet works excavation.

The bulk of Zone 3 materials came from streambed sand and gravel on the dam foundation and in the upstream channel area.

Excavation in the borrow areas was accomplished by cutting across deposits of sand, gravel, and cobble and through lenses and layers of differing gradation to produce a uniform mixture of materials within specified grading limits. Scrapers, push-loaded by dozers, were used to haul the material to the embankment. Boulders too large to be picked up by the scrapers were loaded by end loaders into rear-dump trucks for hauling to the rock embankment area of the Dam. Excavation from the four borrow areas was completed in March 1971. A total of 1,348,000 cubic yards of Zone 3 material was excavated.

About 85% of the rockfill portion of the Dam for Zones 4, 4A, and 5 was excavated from a quarry located in the reservoir area about 1 mile south of the Dam. The remainder was obtained from stockpiles and mandatory excavations.

By October 1969, overburden in the rock quarry area had been stripped to such an extent that large areas of hard rock were exposed. The basic equipment used to drill the rock consisted of air tracs, a drill, and compressors. In November, a rotary drill was brought to the job site and put on this work.

Suitable rock was hauled in 75-ton rear-dump trucks, either directly to the dam embankment or to a rock-separation and processing plant. Waste material was hauled either to mandatory waste areas or used in haul roads. Shot rock was loaded by either a 15-cubic-yard shovel or a loader.

Rock processed through the plant was rear-dumped into the plant hopper bins onto a hydraulically operated grizzly scalper which removed oversize rock for Zone 5 material. The remaining rock passed onto a conveyor belt which fed the separation plant. Vibrating screens in the separation plant divided the material into the required sizes for Zones 3, 4, and 4A. Rock for the various zones was collected in bins which fed dump trucks used to haul the material to the Dam. The oversize rock scalped off for Zone 5 material was loaded into end dumps by an end loader equipped with a tined bucket. The trucks hauled the rock directly to the Zone 5 embankment. Excavation in the rock quarry was completed in March 1971. A total of 4,629,000 cubic yards of material was excavated.

Waste Areas. More than 7,500,000 cubic yards of material, stripped as overburden from borrow area sites and unsuitable embankment materials excavated from mandatory excavations, was wasted in five mandatory waste areas and two optional waste areas.

An estimated 2,300,000 cubic yards of waste material was placed in a mandatory waste area located at the upstream toe of the Dam. Another waste area was developed adjacent to the rock quarry to serve as a recreation beach.

Materials placed in spoil areas did not require any compaction other than that derived by routing construction equipment over the fill. Dozers were used to level the material after it was dumped on the fill.

Embankment Construction

Impervious. The first Zone 1 contact material was placed in the core trench in October 1969. The contact material was compacted at optimum moisture content plus 2 to 3%. The moisture-conditioned material was hauled from Zone 1 stockpiles to the core trench in both scrapers and end-dump trucks. A front-end loader was used to place the material. Then it was compacted in nominal 4-inch lifts by a rubber-tired dozer. At least two passes were made on each lift: one parallel

Figure 272. Compacting Zone 1 Material

Figure 273. Performing Field Density Test on Zone 3 Material

Figure 274. Zone 3 and 4 Material

Figure 275. Zone 5 Embankment Shell Material

to the dam axis and the other perpendicular to it. This method of placement, with minor modifications, was used to place all of the Zone 1 contact materials.

Placement of contact material to a depth of 12 inches was followed by the placement of Zone 1 embankment in the conventional manner using sheepsfoot rollers (Figure 272). The material was spread and leveled by a rubber-tired dozer and compacted by self-propelled, four-drum, sheepsfoot rollers. Twelve passes with the sheepsfoot roller were required to compact the material to a maximum depth of 6 inches and to a required compaction of 95%. The top layer of foundation contact material, compacted by a wheel-roller, was scarified to a depth of 2 inches prior to placing Zone 1 embankment. A total of 1,090,000 cubic yards of Zone 1 material was placed in the dam embankment.

Pervious. Filter blankets lying on each side of the impervious Zone 1 core consist of Zone 2 materials of the Harold formation.

Placement of Zone 2 contact material commenced in the core trench near the center of the east portion of the Dam. The material was hauled from stockpiles in the spillway return channel to the core trench where it was dumped. Then it was leveled with dozers and compacted in 3- to 4-inch lifts by a rubber-tired dozer in the same manner as described for the Zone 1 contact material. This method of placement, except for special problem areas, was used to place all Zone 2 contact material.

Zone 2 embankment placed over Zone 2 contact material was hauled in rear-dump trucks, leveled by dozers, and compacted by self-propelled, four-drum, sheepsfoot rollers to a required minimum compaction of 95%. The material was compacted in 12 passes to a maximum thickness of 6 inches. A total of 483,000 cubic yards of material was placed in the Zone 2 embankment.

The gradation of Zone 3 material varies in size from No. 200 screen to 18 inches. The upstream Zone 3 material was extended in a thick blanket along the foundation and under the Zone 4A embankment all the way to the upstream toe of the Dam. The materials were leveled by dozers and moisture-conditioned on the fill by water tankers. The material was compacted by two passes of a vibratory roller. Lifts were limited to a depth of 24 inches. Compaction tests were performed for every 10,000 cubic yards of material placed (Figure 273).

The minimum compaction required was 95%. A total of 1,949,000 cubic yards of Zone 3 embankment material was placed.

Materials for Zone 4 embankment, lying downstream of Zone 3, varied in size from 3 to 30 inches. The rock for Zone 4 was hauled in rear-dump trucks and spread on the fill by dozers. No moisture conditioning was required. Lifts were limited to 36 inches after compacting (Figure 274) with two passes of a vibratory roller. A total of 776,000 cubic yards of Zone 4 material was placed in the dam embankment.

Placements for Zone 4A embankment commenced on June 24, 1969. A total of 2,460,000 cubic yards of Zone 4A material, varying from sandy gravel to 30-inch rock, was placed in the embankment.

Little foundation cleanup was required for Zone 4A since most of this material rests upon the upstream Zone 3 embankment. In those areas where Zone 4A was placed upon the dam foundation, all overlying topsoil, slopewash, and alluvial material were removed prior to placement.

Rock was leveled on the fill by dozers, moisture-conditioned by water tankers, and placed in 36-inch lifts. Compaction was accomplished with two passes of a vibratory roller.

Slope Protection. No moisture conditioning or compaction was required on Zone 5 embankment rock. Dressing of the $2\frac{1}{4}$:1 downstream slope of the Dam was done by dozers. A total of 784,000 cubic yards of Zone 5 material was placed in the dam embankment (Figure 275).

Riprap. Riprap for the upstream face of the Dam from elevation 3,305 feet to the dam crest varied in size from 3 inches in diameter to 2 cubic yards in volume. Lifts were limited to a depth of 5 feet.

Spillway

Open-Cut Excavation. Spillway excavation was started in December 1968 in the upper reach of the spillway and in the return channel. The material excavated from the upper reach of the spillway that was unsuitable for embankment was wasted at the dam toe, while the material excavated from the return channel was either wasted in the downstream waste area or used to build the construction facility earth pad. The initial construction equipment used to strip the vegetation and overburden consisted of dozers equipped with rippers and slope boards, push dozers, and scrapers.

After stripping the vegetation and overburden, the initial excavation was in weathered granite. As the excavation progressed deeper, the material ranged from decomposed granite to hard rock. The rock was moderately to highly fractured. Air drills were used where rock was too hard to be ripped. In some isolated areas, rock had to be blasted. In addition, some boulders were encountered that had to be shot and pushed over the side of the cut into the reservoir area where they were later reshot for use in the dam embankment. All materials suitable for dam embankment were placed in the various rockfills of the Dam.

By the end of July 1969, the excavation of the upper approach channel, crest section, and chute to the intersection of the outlet works had been completed except for final foundation cleanup and structural excavation. Open-cut excavation for the spillway from the intersection of the outlet works to the end of the stilling basin essentially was completed by December 1969, except for fine grading and structural excavation. A total of 2,254,750 cubic yards of material was excavated for the spillway.

Structural Excavation. Structural excavation for the spillway included the shear keys and drain ditches below the bottom surface of the spillway floor slab. Some of the hard granitic rock in the spillway chute was difficult to blast to grade which resulted in considerable overexcavation.

Drain trenches were excavated using a backhoe with a blade and by laborers using jackhammers. Rock too hard to be excavated with the backhoe was drilled and shot prior to excavation. Final cleanup of the foundation prior to placing concrete was done with air-water jets.

Concrete Placement. The first floor concrete placement for the spillway was on the 2% slope section of the chute. The concrete was placed using a conveyor system which discharged the concrete into the form ahead of a screed. The concrete was screeded by a steel slip form pulled up the slope by winches.

Floor concrete placement on the 22% slope section of the chute was somewhat slower than those placements made on the 2% slope and caused more segregation problems.

For concrete placements on the lower reach of the 50% slope section, a 60-ton truck crane and 1-cubic-yard buckets were used to handle the concrete. A conveyor belt system was used as the placement progressed uphill and out of reach of the crane.

Floor concrete placements for the stilling basin were made using either a crane and bucket method or a conveyor belt system. The conveyor belt system was supplied by transit mix trucks. This method also was used to place the floor concrete for the approach channel and the ogee crest section. All forming was done with wood. All floor placements were completed by the time the contractor placed the cover slab over the outlet works tunnel portal.

The first concrete placement for the walls was made on the right side of the spillway at the vertical curve transition between the 2 and 22% slopes of the chute. The wall form consisted of steel panels faced with plywood. Prior to placing the concrete, considerable effort was spent anchoring the form to prevent grout leaks.

A 30-foot conveyor with rubber wheels on the bottom and track-guided wheels at the top was used to elevate the concrete to the top of the form where it was discharged through hoppers and tremie pipes. The concrete was vibrated by working off a platform at the top of the form. This method of placement, with minor modifications, was used to place all wall concrete except in the stilling basin walls, warped approach walls, and ogee crest walls.

Concrete in the stilling basin walls was placed using concrete buckets handled by a 60-ton truck crane. The 45-foot-high walls were placed in three lifts, which varied from 11 feet to about 19 feet in height.

Concrete for the warped approach walls and for the ogee crest walls was placed using conventional wood forms. The warped walls were placed in six lifts using holding forms while the crest walls were placed in three lifts. These placements were made using concrete buckets and a truck crane.

Mojave Siphon Inlet Works

Excavation. Excavation for the inlet works was all open cut. The cut was a maximum of 150 feet deep in the granitics and varied from 25 to 60 feet in Harold formation on the lake side of the ridge. Cut slopes of 1:1, with 15-foot-wide berms every 40 feet in elevation, were used in the granitics. Cut slopes of 1½:1 with a similar berm arrangement were used in the Harold formation where they were subject to submergence by the reservoir.

Excavation commenced in the vicinity of Station 15+00. The excavation work for the first nine months was, for the most part, confined to the reach between Stations 12+00 and 18+00 and was on an intermittent basis, depending upon the availability of scraper units from other phases of the work. The work during this period basically was a dozer and scraper operation.

When excavation had progressed to elevation 3,380 feet, a small slide on the fault plane was observed in the approximate area of Station 17+00, centerline right, at about elevation 3,450 feet. The contractor was directed to perform additional excavation from Station 14+00 to Station 19+00 to correct this problem. Approximately 77,000 cubic yards of material was removed above the fault plane. Major equipment used to remove the slide consisted of dozers with rippers, scrapers, track drills, end-dump trucks, and an end loader.

Excavation was completed in September 1970. Practically all of the excavated material was wasted at the upstream toe of the Dam. Exclusive of the slide, a total of 727,100 cubic yards of material was excavated for the inlet works structure.

Structural excavation for the concrete inlet works key and drain trenches under the chute floor slabs was performed using a backhoe or jackhammers, depending on the nature of the rock. Considerable care was exercised to prevent shattering the rock beyond the excavation neat lines.

Concrete Placement. Floor placements for the upper reach of the chute and the transition structure were made by chuting concrete directly into the plywood-faced forms. The walls were placed using concrete buckets handled by a 60-ton truck crane. Floor and wall placements for the 41% slope of the chute and for the dissipator structure also were made using the crane and bucket method. The contractor used a job-built slip form screed to place concrete in the invert of the 41% slope of the chute. Two cables powered by an electric winch were used to pull the slip form up the slope at a slow rate of speed while vibrating the concrete just ahead of the screed. A total of 2,634 cubic yards of concrete was placed for the inlet works structure.

Concrete Piles. Holes for the 24-inch-diameter, 30-foot-long, cast-in-place, concrete piles under the inlet works dissipator structure were drilled using a crane-mounted auger drill rig. No particular problems were encountered in placing a total of 374 feet of piling.

Mojave Siphon Extension. Open-cut excavation for the Mojave Siphon pipeline extension commenced just north of relocated State Highway 173. Excavation progressed uphill (southerly) toward the inlet works structure, and then the remaining reach of the excavation under State Highway 173 was performed. Dozers and scrapers were used to excavate a total of 33,400 cubic yards of material for the pipeline.

Pipe Installation. The first section of the 126-inch-diameter Mojave Siphon pipeline extension was placed in the open-cut trench 250 feet south of the existing Mojave Siphon. Pipelaying, like the open-cut excavation, proceeded uphill (southerly) toward the inlet works structure, except for the first 250 feet south of the existing Mojave Siphon which was installed last. The pipe installation sequence consisted of laying the pipe to grade, testing the rubber gasket ring for sealing, grouting the outside portion of the joint, installing bond cable at each joint, grouting the inside portion of the joint, and backfilling the pipe. A total of 1,177 linear feet of pipe was installed.

Debris Barriers

Two debris barriers were constructed under this contract. One barrier is located in Miller Canyon in the southeast fork of the reservoir and the other in Cleghorn Canyon in the southwest fork of the reservoir.

These barriers were constructed across the streambeds of the two canyons to catch debris coming down the canyons after closure of the diversion channel through Cedar Springs Dam. Consequently, construction of these barrier dams was prohibited prior to closure of the diversion channel.

Immediately following construction of the debris barriers and prior to reservoir filling, a large-magnitude flood overtopped the Miller Canyon dam and washed out a section containing the spillway. Just before reservoir filling, this section was replaced along with a 48-inch corrugated pipe to bypass flow during reconstruction.

Drainage Gallery

The drainage gallery, Station 0+00 to Station 8+22, was driven initially for exploration purposes, as part of the exploration program discussed previously. Except for mucking out the caved-in materials of the exploration adit, no excavation was required in the drainage gallery. One extensive cave-in had to be mucked out at the beginning of the tunnel in the vicinity of Station 1+50 (Figure 260). The remaining work in the tunnel, prior to placing concrete, consisted of laying track, bringing in utility lines, providing for drainage, realigning existing steel sets, reinstalling fallen lagging, and installing additional lagging where required.

The tunnel section between Stations 8+22 and 8+64 (Figure 260) was driven northward from the dam foundation under the main dam contract.

Concrete Placement. The first concrete placement in the invert of the drainage gallery tunnel was between Station 8+00 (approximately 22 feet from the drainage gallery extension) and Station 6+70. Subsequent invert placements in the drainage gallery proceeded toward its intersection with the access tunnel. From Station 8+00 to Station 4+60, the concrete placements were made using a concrete pump located outside at the drainage gallery extension portal. A 6-inch slickline was used to convey the concrete directly into the invert forms. Usually, additional cement over the structure's design requirements was added to the concrete to reduce friction in the slickline.

The remaining reach of the drainage gallery tunnel invert from Station 4+60, and its intersection with the access tunnel, was placed with the concrete pump located inside the tunnel. The concrete was pumped from the pump hopper directly into the invert forms using a 6-inch slickline. Length of placements varied from about 60 to 110 feet.

The first arch placement in the drainage gallery was made at its intersection with the drainage gallery extension, and subsequent placements proceeded toward its intersection with the access tunnel. A total of 14 arch placements, varying in length from 48 to 68 feet, were made using a concrete pump and slickline from inside the tunnel. Collapsible steel forms were used for these placements.

Drainage Gallery Extension. The drainage gallery extension was driven using a crawler-mounted drill. Steel supports, heavily cribbed, were required only at the portal, as sound rock was encountered as the excavation progressed. Most of the blasted rock was mucked out through the portal using an end loader except muck from the last round, which was removed through the drainage gallery using a mucker and train.

Concrete for the 6-foot-diameter drainage gallery extension was monolithically placed using a concrete pump and a slickline. The concrete was transported from the batch plant in transit mix trucks which discharged their loads directly into the pump hopper. The upstream 8-foot-long plug was placed in a similar manner.

Access Tunnel

Completion. Work began with mucking out sand and drainage water from the exploration adit tunnel and extending the track into the existing timber-supported tunnel. Mucking out the invert; laying track; and installing air, water, and electric lines were continued until the intersection of the access tunnel and drainage gallery was reached. At the intersection, the crown of the tunnel had caved in from Station 25+65 to Station 25+88 in the access tunnel and to Station 0+90 in the drainage gallery tunnel. The cave-in brought down all the previously installed supports. The cave-in was mucked out and the crown was resupported with several 4-inch and 6-inch-wide steel sets and timber lagging.

While one mining crew continued work in the previously driven part of the access tunnel and in the drainage gallery tunnel, another crew started driving the access tunnel from Station 25+88 toward the outlet works gate chamber. Fairly good rock usually was encountered in the access tunnel as it was driven toward the gate chamber. In the vicinity of Station 27+87, three steel sets, W4X13, on 5-foot centers were used in a faulted area. In the remaining reaches, the tunnel was either driven bald-headed or rock-bolted (bolts were ⅝ of an inch by 6 feet long), depending upon the nature of the granitic rock. The tunnel was advanced in 4- to 7-foot rounds.

Concrete Placement. The first concrete invert placement for the access tunnel was between Stations 28+97 and 29+62 near the outlet works gate chamber, and subsequent invert placements proceeded toward the portal. The concrete was transported by rail from the batch plant. A concrete pump with a 6-inch slickline was used to place the concrete. Placements varied from about 55 to 135 feet in length.

Concrete arch placements for the access tunnel commenced after the drainage gallery arch was completed. Placements started at the gate chamber, Station 29+78, and progressed to the portal at Station 19+90. A total of 16 arch placements, averaging 68 feet each, were required. The method used to place the concrete was the same as described for the drainage gallery arch placements.

Outlet Works Tunnel

Excavation. The initial excavation of the gate chamber was a continuation of the access tunnel excavation. A tunnel with a width of 10 feet and a height of 11 feet - 6 inches was driven to the centerline of the gate chamber and then to the air tunnel portal, thus completing the first phase of the outlet works excavation.

After completing the excavation of the air tunnel, the contractor resumed excavation in the gate chamber by driving an 8-foot-diameter shaft from the floor of the gate chamber (elevation 3,196.30 feet) to the top of the dome. The dome at the top of the shaft was rock-bolted with 10-foot-long rock bolts on 3-foot cen-

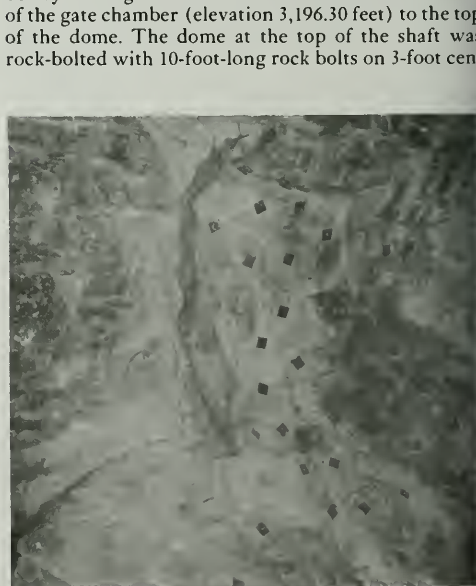

Figure 276. Rock Bolts in Dome of Outlet Works Gate Chamber

ters. A rod then was hung at the centerline of the gate chamber from the top of the dome to the springline of the dome from which a platform was attached. This enabled the miners to drill 17 feet - 8 inches in any direction above the springline from the lower end of the rod. The entire top 120-degree peripheral portion of the dome above the springline at elevation 3,212.30 feet then was drilled out and shot in one blast. The 30-degree slope bench formed by the blast served as a funnel, in effect, since the muck rolled down into the 8-foot-diameter shaft to elevation 3,213.50 feet where it was removed. The rest of the dome then was rock-bolted on approximately 3-foot centers with rock bolts varying from 6 to 10 feet in length. The center shaft was enlarged to a diameter of 15 feet, and the remaining rock above the springline was drilled, shot, and rock-bolted (Figure 276). The chamber below the springline was excavated in three separate rounds working outward from the center of the 15-foot shaft on a 17-foot -8- inch radius. The walls of the gate chamber between the springline and the floor (elevation 3,196 feet) also were rock-bolted. The contractor, throughout the tunneling, used the proper amount of tunnel supports without arguing that more support was needed for various reasons. This was due in large part to the lump-sum bid item for tunnel work which, in effect, made the contractor responsible for the amount of tunnel support used.

The major equipment used to excavate the gate chamber included jackleg drills, stoppers, a mucker on rails, a mucker on tracks, five to seven 4-cubic-yard muck cars on rails, and an 8-ton motor-driven locomotive.

The outlet works tunnel was driven upstream from

Figure 277. Placing Concrete in Gate Chamber

the downstream (north) portal where the tunnel discharges onto the 50% slope of the spillway floor. As soon as the fractured rock over the north portal was stabilized with the installation of crown bars into the overburden, a 3-set steel umbrella was installed at the portal and drilling commenced. The first shot opened cracks in the rock above the portal, and additional cribbing had to be installed along the sides and above the portal. The fractured rock at the portal extended into the tunnel for about 75 feet to Station 27+21 and was supported by W6X15 steel sets on 4- to 5-foot centers, with considerable lagging. From Station 27+21 to Station 26+79, the granitic rock was fairly good, and no supports were required.

The rock from Station 27+21 to Station 23+50, a point of contact with a shear zone, varied from slightly fractured to sound rock. As a result, this reach of the tunnel was either supported by W6X15 steel sets on 5- to 6-foot centers or rock-bolted with rock bolts varying from 6 to 10 feet long, depending upon the nature of the rock.

In the faulted area reaching from Station 23+50 to approximate Station 22+96, fault 1 was highly fractured and contained clay gouges. This area was supported by W6X15 steel sets on 3- to 4-foot centers and required extensive lagging. Between Stations 22+96 and 18+65, where the excavation broke into the previously excavated gate chamber dome, the quality of the rock varied considerably.

In November 1969, driving of the 13-foot-diameter circular section of tunnel was commenced, leaving 3 feet of material in the invert to be excavated later. From Station 17+81 to Station 13+68, the lightly fractured granite required rock bolts only in isolated areas to pin loose rock. Between Station 13+68 and the upstream portal, several 6-inch steel sets were required to support the rock.

Major equipment used to excavate the outlet works included a drill jumbo, two end loaders, and five air compressors.

The open-cut excavation for the outlet works intake structure was performed in conjunction with the open-cut excavation required for the spillway approach channel. Nine steel sets, with considerable lagging, were used to support the portal.

Concrete Placement. The first two placements for the gate chamber were made to level the foundation to allow setting of jack plates and anchor bolts for the steel-plate liner supports. The concrete was transported from the batch plant to the gate chamber in transit mix trucks (Figure 277).

For three months, the work in the gate chamber was limited to placing, fitting, and welding the steel-plate liner sections. Fit-up of the liner sections took longer than expected due to plate distortion and warping.

Embedment of the upstream steel-plate liner was done in two lifts and the downstream liner in three lifts. All placements were made with a concrete pump and slickline.

Three concrete placements were required in the sidewalls between the upstream and downstream liner to elevation 3,194 feet. Placements were made using conveyor belts to span the gate chamber.

Conventional wood forms were used for these placements. The floor, remaining walls, and dome placements were made in five segments using a pump and slickline and conventional wood forms. Concrete was transported by rail through the access tunnel.

Invert placements for the 13-foot-diameter portion of the outlet works tunnel upstream of the steel-plate liner in the gate chamber were made in seven sections. The first five placements were made using a 60-foot conveyor belt which discharged the concrete directly into the form. The last two placements were made using a concrete pump and an 8-inch slickline. The concrete pump and slickline method of placement proved to be the most effective, reducing the placing time by approximately 50%.

Ten arch placements, starting from the gate chamber and varying in length from 48 to 56 feet, were required to reach the upstream portal at Station 12+71. All placements were made using a concrete pump and an 8-inch slickline.

The first invert placement for the downstream horseshoe section of the outlet works was made for the downstream transition from Station 18+84 to Station 19+17. This placement and the next six placements from Station 18+84 to Station 24+36 were made using a concrete pump with an 8-inch slickline. The remaining placements, except for the last one at the downstream portal, were made using a conveyor belt. The last invert placement was made using a concrete pump, set at the portal, and an 8-inch slickline.

Prior to placing concrete in the arch of the downstream horseshoe section, 1-foot-high stub (curb) walls were placed on each side of the centerline. Transit mix trucks discharging directly into the steel forms were used for these placements, which were made well ahead of the arch placements. The arch placements were made using a concrete pump with an 8-inch slickline.

The outlet works intake structure was built using wood forms faced with plywood. The concrete was placed using a truck crane and 2-cubic-yard concrete buckets.

Air Shaft Tunnel

Excavation. Excavation for the air tunnel was a continuation of the outlet works gate chamber. The tunnel was driven through sound granitic rock and no supports were required. The method, equipment, and labor used to drive the tunnel essentially were the same as for the access tunnel excavation.

Excavation of the air shaft was started on July 2, 1969. By July 7, 1969, the shaft had been driven 46 feet from the invert of the air tunnel by miners working off a platform pinned to the shaft. Work was suspended at that time until a cable and air hole could be drilled from the ground surface to the heading in the shaft. Excavation of the air shaft resumed with drilling from the ground surface of an 8-inch-diameter hole in the center of the shaft. This hole was to serve as a cableway for raising a man-cage used as a work platform by the miners in excavating the shaft at higher elevations (Figure 261). Work continued using the platform until the shaft had been raised 87 feet from the invert. Then the miners began using the man-cage to excavate the shaft. The cage was hoisted from the ground surface using a 30-ton truck crane. The man-cage was used to excavate all but the top 25 feet of the raise. This reach was excavated from the top of the shaft.

Concrete Placement. The first placement consisted of 15 feet of invert and stub wall in the air tunnel to the centerline of the air shaft. The concrete was delivered to the top of the air shaft and lowered 180 feet down the air shaft in a 1-cubic-yard bucket where it was dumped and vibrated into the forms.

The first air-shaft placement was made from the invert to elevation 3,230 feet and included 8 feet of the tunnel walls and arch. Six subsequent placements were made to elevation 3,376.5 feet or to 2 feet below the finished floor of the air-shaft louver house. Two shaft placements were made monolithically using two 6-inch slicklines extending from the top of the shaft to within 3 feet of the bottom of the placement. This method of placement for the air shaft produced good workability without segregation.

The first air-tunnel invert and stub walls placement incorporated the invert of the air shaft. The remaining 78 feet of the invert and stub walls were placed using a truck-mounted pump located downstream of the gate chamber in the outlet works tunnel. Two concrete placements were required to complete the air-tunnel arch. Placements were made using a pump located at the upstream end of the tunnel, adjacent to the outlet works gate chamber. A 6-inch slickline was used to place the concrete in collapsible steel forms.

Concrete Production

Three principal concrete mixes were used in the construction of the spillway, inlet works, and tunnels. A mix with 3-inch maximum size aggregate, 306 pounds of cement, and 70 pounds of pozzolan was used for the spillway and inlet works floor slabs and large walls, and for the entire intake structure. Thin walls of the spillway and inlet works have concrete with a mix of 1½-inch maximum size aggregate, 400 pounds of cement, and 70 pounds of pozzolan. All concrete placed in the underground works consisted of a mix of 1½-inch maximum size aggregate, 447 pounds of cement, and 100 pounds of pozzolan. Using waterwashed and shaded aggregate, together with 100% ice, concrete was held below 53 degrees Fahrenheit during summer placement. This placement temperature was only 3 degrees above optimum for this type of work.

A fully automatic 4-cubic-yard plant was used to batch the concrete for Cedar Springs Dam structures. The plant included a 4-cubic-yard tilting-drum mixer and individual batches of 6,000-pound capacity for the four sizes of aggregate. Cement and pozzolan were batched cumulatively in a 6,000-pound batcher. Ice was batched into a 2,000-pound batcher with adjustable feed discharge directly onto a belt. Water was metered through a 3-inch meter. Based on a 2½-minute mixing cycle, the capacity of the plant was 80 cubic yards of concrete per hour.

Grouting

Dam Foundation. The curtain grouting program for the dam foundation involved a high-pressure, 150-foot-deep, main curtain and a low-pressure, 25-foot-deep, secondary curtain. One-and-one-half-inch-diameter pipe nipples were installed in all grout holes. Grouting was done by the split-spacing method. Primary holes were 150 feet deep on 20-foot centers with 50-foot-deep intermediate holes between the primary holes. The main curtain crosses the entire length of the Dam in a mid-position under Zone 1 and was combined with a line of holes in the secondary curtain.

Grouting of the main curtain was accomplished by a combination of stage and packer grouting in the following sequence: first stage was drilled to a depth of 25 feet and stage-grouted from the nipple; second stage was drilled the next 25 feet, a packer was set at the top of this stage, and grouted; primary holes were completed by drilling the third stage the remaining depth of hole (150 feet); packer grouting from 100 to 150 feet; and subsequently setting a packer at a depth of 50 feet and packer grouting from 50 to 100 feet.

Blanket grouting was done in fractured areas and where grouting of the first stage in the main and secondary curtains did not appear to be adequate to seal the upper 25 feet. Blanket grouting consisted of single lines of shallow holes (up to 25 feet deep) on 10-foot spacings, upstream and downstream of the curtain. Pressures of 10 to 15 pounds per square inch (psi) were used, although up to 25 psi was used occasionally.

Spillway. Curtain grouting for the spillway foundation was done by a procedure similar to that for the dam foundation except a secondary curtain was not included; maximum depth of drill holes was 125 feet in the bottom and 50 feet on the side slopes; and for the third stage, the packer was set at 90 feet, and the maximum pressure was 125 psi instead of 150 psi.

Tunnel. Contact grouting was performed in the tunnels to fill any voids between the concrete tunnel lining and the surrounding rock. Contact grout was injected through 1½-inch-diameter pipe set in the tunnel lining near the crown. Grouting of a hole was considered to be complete when 30 psi could be maintained for 15 minutes with no significant grout take.

Consolidation grouting consisted of low-pressure shallow-hole grouting of the fractured rock surrounding the tunnel. This was accomplished by drilling and grouting a series of holes in a ring pattern equally spaced around the tunnel perimeter and perpendicular to the tunnel centerline. Grout pipe, 1½ inches in diameter, was installed prior to placement of the concrete lining. Grouting of each ring usually was started at the tunnel invert, and every other hole was grouted progressing upward to the crown. Grouting was accomplished by setting a packer inside the pipe and grouting with a pressure of 50 psi. The intermediate holes on each ring then were drilled and grouted to fill any voids between the primary holes.

Gate Chamber. Contact grouting was conducted in the gate chamber dome by using seven consolidation grout pipes which were located evenly throughout the dome.

Consolidation grouting consisted of 61 holes included in six rings of 10 each, plus one hole at the top of the crown. Holes were drilled 20 feet into rock and grouting was accomplished in one stage. The rock surrounding the dome was hard to drill with diamond bits, and low grout takes were recorded.

After reservoir filling, moisture was noted on about 50% of the gate chamber walls and ceiling. To maintain a dry condition for the electrical equipment, the Department contracted for additional contact grouting using a chemical grout. Seeping plugged grout holes, construction joints, and fine-line cracks were drilled, caulked, and pressure-grouted with acrylomide grout. The treatment proved to be about 95% effective. Additional grouting is planned in the gate chamber air shaft where moisture continues to be a problem.

Air Shaft. Consolidation grouting was conducted in the air shaft for a vertical distance of 203 feet above the air intake tunnel. The grouting consisted of rings of four holes each on 15-foot centers. Grout take was low in most areas and 22 holes were tight. The only area with appreciable grout take was the south side of the shaft between elevation 3,201 feet and elevation 3,225 feet. Additional chemical grouting also is planned for the air shaft where moisture from seeping construction joints now drips from the walls.

Reservoir Clearing

Clearing in the reservoir to elevation 3,354 feet consisted of the removal and disposal of all trees, down timber, brush, rubbish, fences, floatable material, and buildings. Cesspools and septic tanks were pumped out and filled with sand. Any trees between elevation 3,354 feet and the spillway weir crest at elevation 3,355 feet were left standing at the request of the U.S. Forest Service. It was believed that many of these trees would live because of infrequent inundation, and those that die can be used by campers for firewood.

Reservoir clearing commenced in December 1968 and was completed in June 1971. A spread of four bulldozers was used to clear the reservoir area, except for inaccessible areas which had to be cleared by laborers using hand and power tools. Combustible material, except for commercial timber, was disposed of by burying or burning. All burning was performed in accordance with the project fire plan approved by the U.S. Forest Service. Merchantable pine timber in the Miller Canyon area was logged by a subcontractor.

BIBLIOGRAPHY

Lanning, C. C., "Cedar Springs Dam", USCOLD Issue No. 32, U. S. Committee on Large Dams Newsletter, May 1970.

Sherrard, J. L., Cluff, L. S., and Allen, C. R., "Potentially Active Faults in Dam Foundations", Volume XXIV, No. 3, The Institution of Civil Engineers Geotechnique, September 1974.

Stroppini, E. W., Babbitt, D. H., and Struckmeyer, H. E., "Foundation Treatment for Embankment Dams on Rock", ASCE Journal of Soil Mechanics and Foundation Division, October 1972.

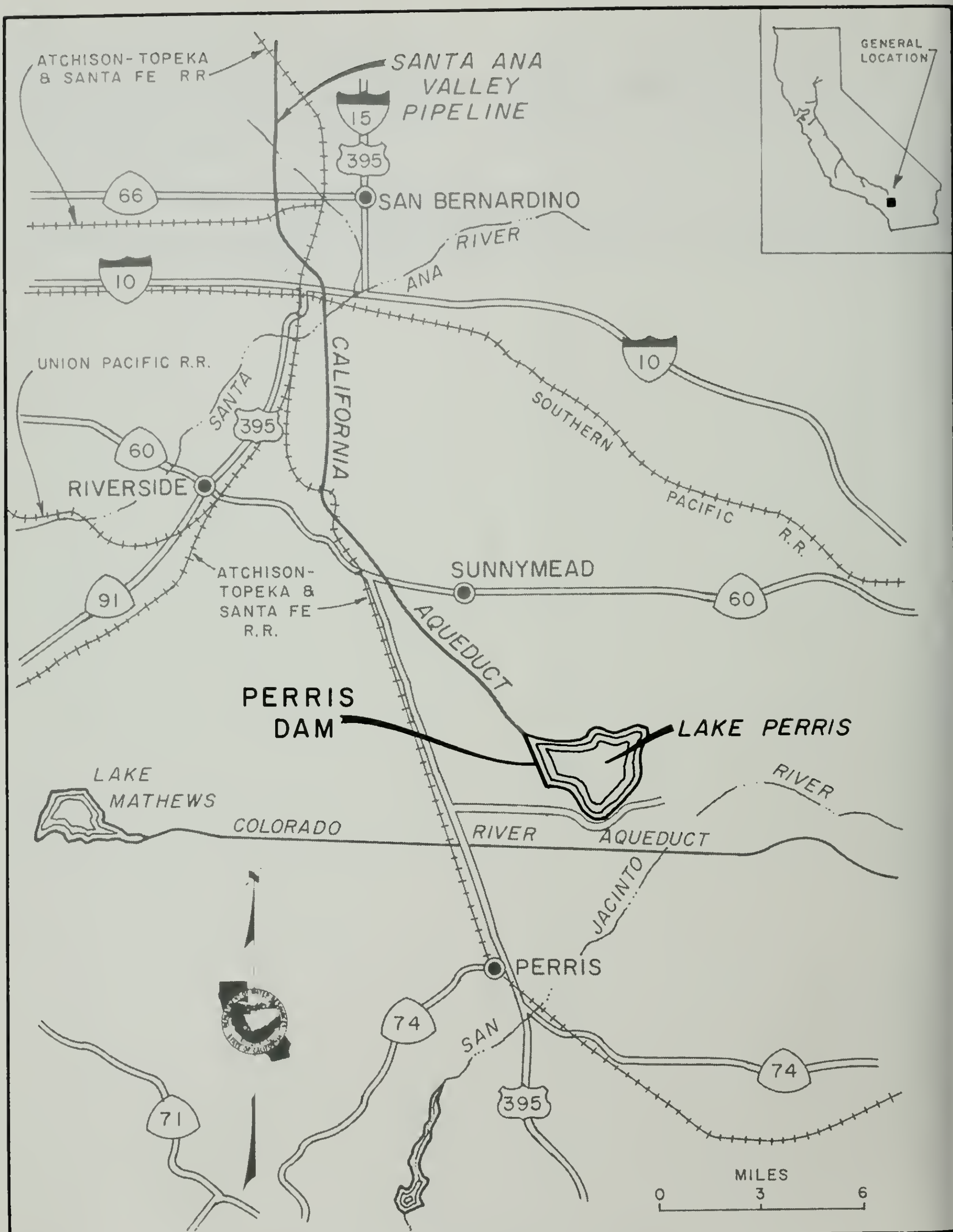

Figure 278. Location Map—Perris Dam and Lake Perris

CHAPTER XIII. PERRIS DAM AND LAKE PERRIS

General

Description and Location

Perris Dam is a zoned earthfill structure with a maximum height of 128 feet. The Dam is over 2 miles in length and impounds a reservoir with a gross storage capacity of 131,452 acre-feet within a horseshoe ring of rocky hills. Approximately 20 million cubic yards of fill material comprise the embankment.

Structures appurtenant to the Dam include a tunneled outlet works connecting to distribution facilities of The Metropolitan Water District of Southern California (MWD) and an open-channel ungated spillway. Water is discharged into Lake Perris by an extension of the Santa Ana Valley Pipeline, termed the "inlet works".

Perris Dam and Lake Perris, the terminal storage facility on the California Aqueduct, are located in northwestern Riverside County, approximately 13 miles southeast of the City of Riverside and about 65 miles east of Los Angeles. Perris, the nearest town, is about 5 miles to the southwest (Figures 278 and 279). The reach of the State Water Project terminating at the Lake is designated the Santa Ana Division.

A statistical summary of Perris Dam and Lake Perris is shown in Table 35, and the area-capacity curves are shown on Figure 280.

Figure 279. Aerial View—Perris Dam and Lake Perris

Purpose

Lake Perris, a major feature of the State Water Project, is a multiple-purpose facility with provisions for water supply, recreation, and fish and wildlife enhancement.

Chronology

Prior to 1965, the capacity of Lake Perris was set at 100,000 acre-feet. At that time, this storage volume satisfied the requirements of MWD, the only water contractor taking deliveries from this facility.

The Metropolitan Water District studies in 1965 of water service expansion concluded that increasing the capacity of Lake Perris would increase the security of service and improve flow regulation. The Department of Water Resources then was requested by MWD to study reservoir enlargement, and two amendments to the water service contract between the Department and MWD followed. Amendment No. 4, dated November 19, 1965, provided that the Department would acquire all lands necessary for a reservoir with a capacity of 500,000 acre-feet and would perform planning and design work which would enable MWD to select the appropriate ultimate reservoir size. Amendment No. 5, dated October 10, 1966, stated that the Dam and appurtenant facilities initially would be constructed to impound 100,000 acre-feet and provisions would be made so that the Dam could be raised, in any number of stages, to impound a maximum of 500,000 acre-feet of water. Final design of the facilities was completed in accordance with Amendment No. 5, and construction of the Dam commenced in October 1970.

Increased recreation potential and the adverse impact of reservoir enlargement on the onshore recreation development, along with reevaluation of MWD future demands, resulted in the decision to construct the facilities in one stage with a capacity somewhat above 100,000 acre-feet. Studies immediately following showed that a dam 10 feet higher, providing a storage capacity of 120,000 acre-feet, could be constructed with the funds available. Redesign of the Dam took place in April 1971 and the contractor, who had begun construction but had been limited to working on items unaffected by the enlargement, was given immediate notice to proceed with construction in accordance with the new plans.

All construction was completed by February 1974, except for some minor project modifications and initial recreation facilities.

Regional Geology and Seismicity

The Dam site is located on the Perris block, a large down-dropped block of Cretaceous granitic rock which contains some schist and gneiss. During earlier times, these crystalline rocks were sculptured into ridges and valleys by erosional processes. Later, the Perris block dropped down, changing the drainage pattern and filling the valleys with alluvium. Today, only the higher former ridges protrude through the alluvial surface forming a landscape of flat valleys and low ridges.

TABLE 35. Statistical Summary of Perris Dam and Lake Perris

PERRIS DAM	
Type: Zoned earthfill	
Crest elevation	1,600 feet
Crest width	40 feet
Crest length	11,600 feet
Streambed elevation at dam axis	1,480 feet
Lowest foundation elevation	1,472 feet
Structural height above foundation	128 feet
Embankment volume	20,000,000 cubic yards
Freeboard above spillway crest	10 feet
Freeboard, maximum operating surface	12 feet
Freeboard, maximum probable flood	5.8 feet
LAKE PERRIS	
Storage at spillway crest elevation	131,452 acre-feet
Maximum operating storage	126,841 acre-feet
Minimum operating storage	37,013 acre-feet
Dead pool storage	4,100 acre-feet
Maximum operating surface elevation	1,588 feet
Minimum operating surface elevation	1,540 feet
Dead pool surface elevation	1,500 feet
Shoreline, spillway crest elevation	10 miles
Surface area, spillway crest elevation	2,318 acres
Surface area, maximum operating elevation	2,292 acres
Surface area, minimum operating elevation	1,540 acres

SPILLWAY	
Type: Ungated ogee crest with concrete baffled chute and riprapped channel	
Crest elevation	1,590 feet
Crest length	16 feet
Maximum probable flood inflow	17,500 cubic feet per second
Peak routed outflow	320 cubic feet per second
Maximum surface elevation	1,594.4 feet
INLET WORKS	
Buried 8-foot - 6-inch concrete pipeline from terminus of Santa Ana Valley Pipeline above right abutment—energy dissipated by hydraulic jump inside pipeline	
Capacity	469 cubic feet per second
OUTLET WORKS	
Type: 12-foot - 6-inch-diameter lined tunnel under left abutment, with a steel delivery manifold	
Intake structure: Five-level vertical tower with 72-inch shutoff butterfly valves	
Control: Regulation of flow at delivery manifold by water users	
Design delivery	1,000 cubic feet per second
Blowoff structure: 6-foot-wide by 12-foot-high slide gate downstream of delivery manifold with bolted bulkhead at downstream terminus	
Capacity	3,800 cubic feet per second

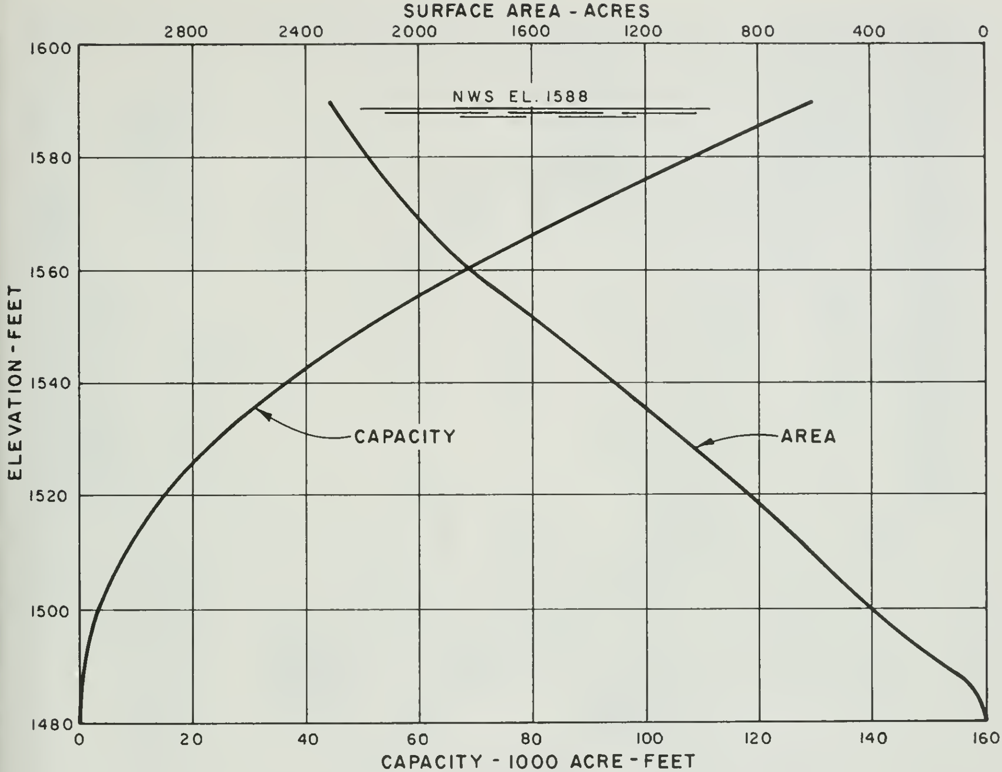

Figure 280. Area-Capacity Curves

The Dam is located in one of the most active seismic areas in Southern California. There are seven faults within 20 miles of the site which have been classified as active. These faults are: San Andreas; San Jacinto; Elsinore; Agua Caliente; and the Casa Loma, Loma Linda, and Hot Springs branches of the San Jacinto. The southwest margin of the San Jacinto fault zone is the Casa Loma fault, which is within 6 miles of the Dam site and 1 mile from the edge of Lake Perris. The Elsinore fault zone is located approximately 17 miles southwest of the Dam site.

During the 33 years of available seismograph records, 31 earthquakes with Richter magnitudes of 3.0 to 3.9 were recorded within a 10-mile radius of the Dam site, and 15 were recorded with magnitudes over 4.0 within a 30-mile radius of the site. Two of the earthquakes, within a 20-mile radius, were of major proportions, with magnitudes of 6.0 and 6.9. One of these occurred in 1918; its epicenter was only 16 miles southeast of the Dam site. Buildings and water mains in the town of San Jacinto were destroyed.

Design

Dam

Description. Perris Dam has a zoned earthfill section with a sloping clay core (Figure 281). Except for short reaches on the left abutment and at the rock outcrops, the Dam is founded on an alluvial foundation.

Necessary freeboard to prevent overtopping of the Dam during maximum flood was computed assuming a maximum wave height plus run-up caused by a wind of 100 miles per hour. The dam crest thus was established at elevation 1,600 feet. This provides a freeboard of 12 feet above normal pool. The maximum section (Figure 282) is 128 feet high.

The dam alignment was initially predicated on the ultimate development (500,000-acre-foot lake). Left abutment location was selected to yield the most favorable rock contact for the large dam, even though considerable shaping was required for the lower portion. The rock outcrops at the valley center were used

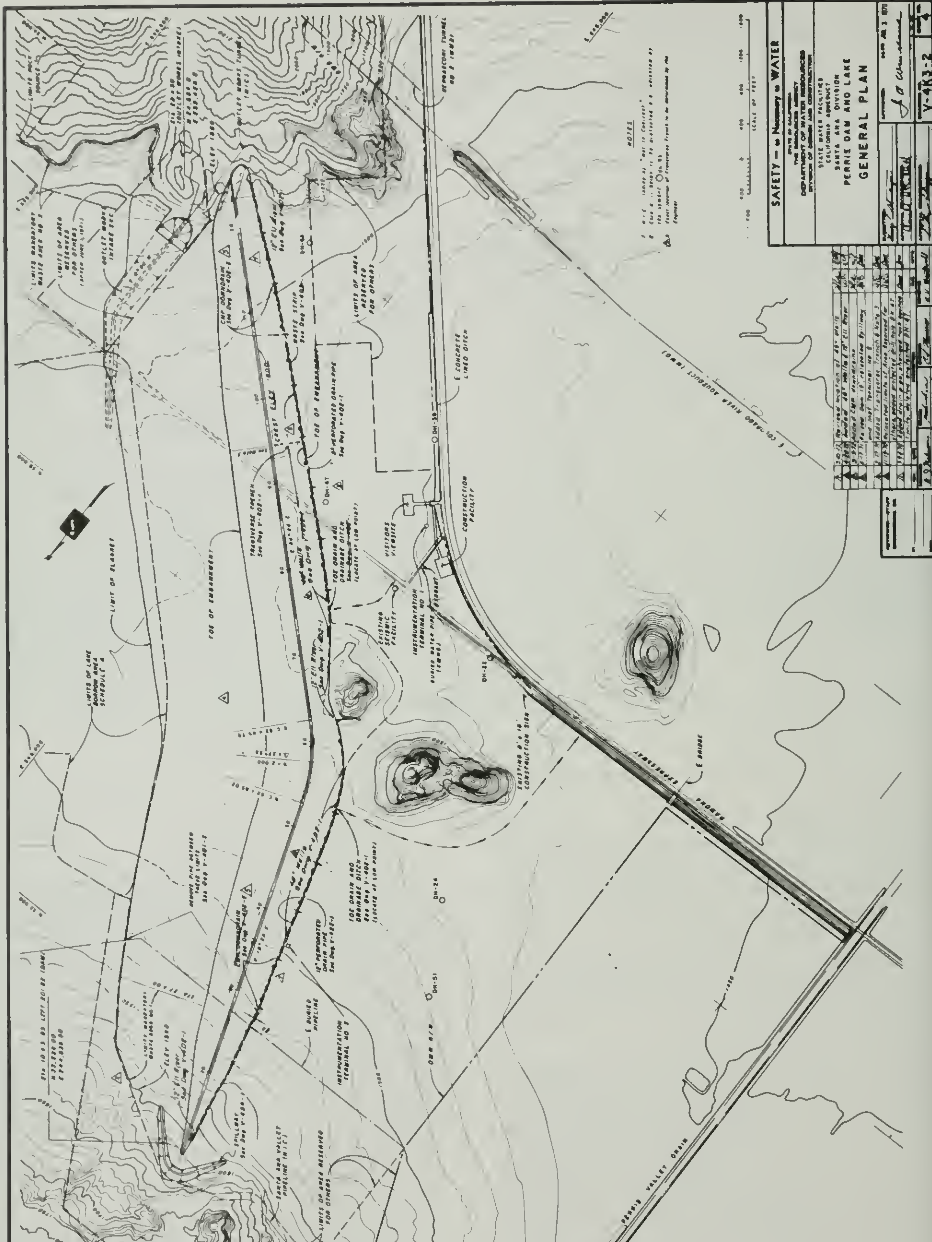

Figure 281. Embankment Plan

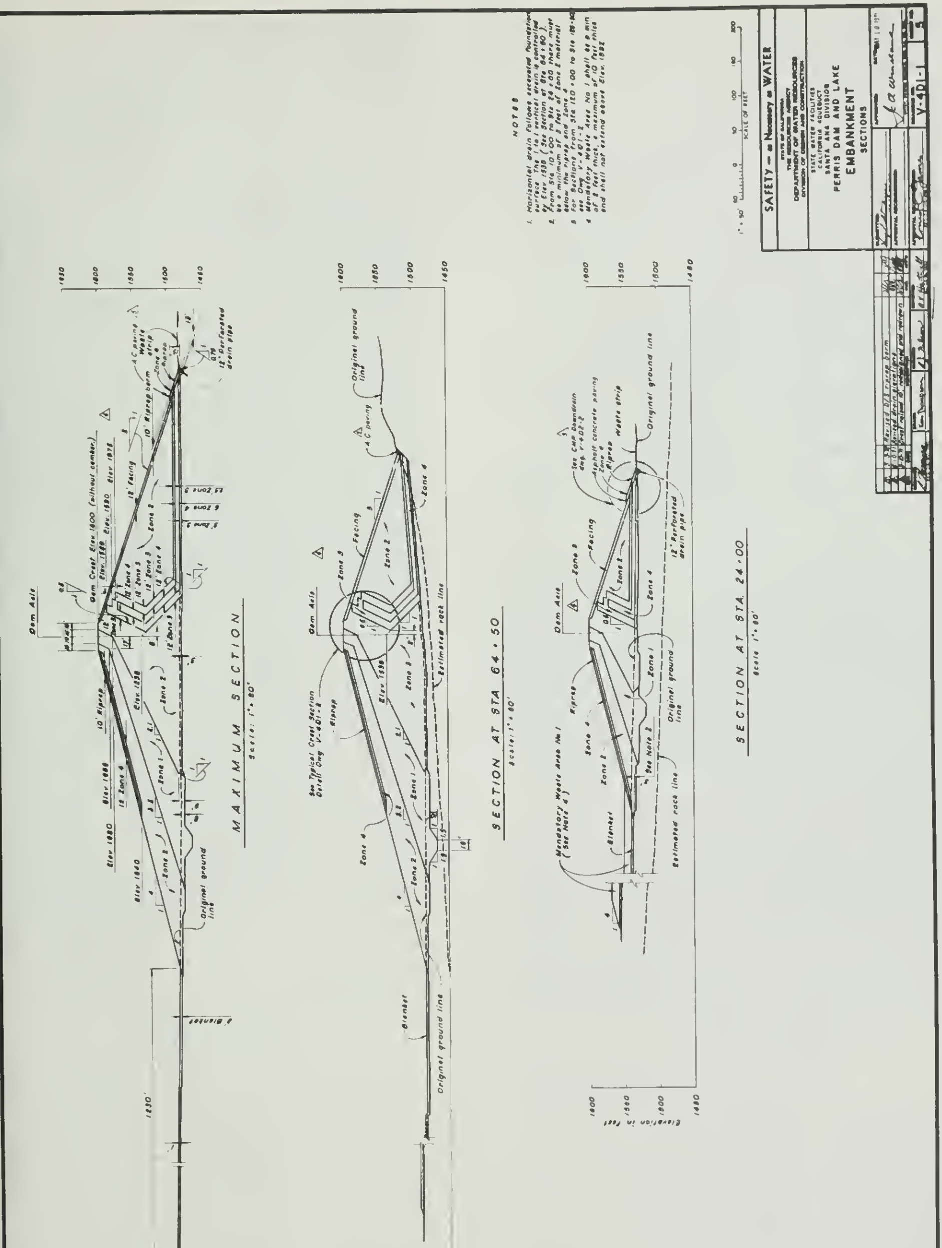

Figure 282. Embankment Sections

as a turning point, and the right-leg alignment followed the shortest line to topography of sufficient elevation. Because embankment construction was underway when the decision was made to change the reservoir capacity, the alignment described earlier in this chapter was not changed.

The core of the Dam is constructed of a plastic clay, and the outer zones are composed of silty sand. The downstream silty sand zone contains a vertical crushed-rock drain, and a horizontal drain underlies the downstream shell. A filter zone of silty sand was placed between the core and the vertical drain. Embankment zones were made a minimum of 12 feet in width to allow efficient placement and compaction.

Riprap protects the upstream face of the Dam from wave erosion. Riprap at the downstream toe also protects the exposed face of the horizontal drain. The remainder of the downstream slope was planted with a stand of grass for surface runoff protection.

Stability Analysis. Slip circle and sliding wedge analyses were made to determine the stability of the embankment under all anticipated loading conditions. Satisfactory safety factors were attained for the section as designed. Critical loading cases were: upstream slope, reservoir at critical level and earthquake load; and downstream slope, full reservoir with earthquake load. Earthquake loading utilized in these analyses was assumed to be a horizontal acceleration of the foundation in the direction of instability of the soil mass being analyzed. The assumed acceleration was 0.15g. Material strengths utilized in these analyses were derived as follows: Zones 1 and 2 and foundation, soil testing by the Department; and Zones 3 and 4, estimates based on material strengths determined for similar materials (Table 36).

Dynamic finite element analyses were made by the University of California and a private engineering firm. These analyses showed that the Dam was stable under severe earthquake shaking and that substantial increased stability could be gained by achieving high density in Zone 2 material. Therefore, compaction near the maximum department laboratory standard was required for this material.

Settlement. Analyses were made for the Dam and alluvial foundation to estimate the additional amount of embankment required to compensate for settlement during construction and to establish the crest camber necessary to compensate for postconstruction settlement.

Anticipated settlement of the embankment during construction was calculated by summing consolidation as horizontal layers were placed successively. This was found to be 2.4 feet. Postconstruction settlement occurs mostly in the foundation when saturation takes place. The settlement was estimated to be 3.6 feet. The 4-foot camber supplied at maximum section was a summation of the anticipated postconstruction settlement of the embankment and the foundation.

Construction Materials. The sloping impervious core (Zone 1) is a plastic clay material obtained from a borrow area 5 miles northeast of the Dam site. Investigation was made of ways to furnish water to this borrow area after construction to provide a fishing and waterfowl observation site. Lack of funding by potential operating agencies and possible conflicts with the main recreation area resulted in abandonment of this concept. Therefore, the borrow area was designed as a free-draining excavation with flat slopes so the land could be returned to its original use of grain farming.

Zone 2, the semipervious upstream and downstream shells and downstream dam facing, was obtained from a borrow area within the lake area and from required structural excavations. Two alternative borrow areas were considered. The first was located as close as possible to the Dam and was formed by simple excavation lines. The second contemplated excavations along the north shore for recreational enhancement. Slopes for beaches and boating facilities along with peninsulas formed of waste materials were provided in the final configuration of the borrow area. This alternative was requested by the Department of Parks and Recreation and general layout criteria were furnished by them. As the second borrow alternative

TABLE 36. Material Design Parameters—Perris Dam

Material	Specific Gravity	Unit Weight in Pounds Per Cubic Foot			Static Shear Strengths θ Angles in Degrees Cohesion in Tons Per Square Foot					
					Effective		Total		Construction	
		Dry	Moist	Saturated	θ	C	θ	C	θ	C
Zone 1	2.76	112	128	134	23	0.4	13	0.6	3	1.3
Zone 2	2.75	121	134	139	33	0	13	1.0	33	0
Zone 3	2.70	127	--	143	40	0	40	0	*	*
Zone 4	2.70	125	--	140	40	0	40	0	*	*
Foundation	2.74	116	--	136	33	0	15	1.0	--	--

* Free-draining material, use effective stress values.

obviously would be more costly, each was included in the specifications on separate schedules, and potential contractors were required to bid on each. The Department then could award the contract on the basis of either schedule, depending upon the differential cost and availability of funds.

The horizontal and vertical embankment drains are composed of a clean drain rock (Zone 4) enveloped by sandy gravel transitions (Zone 3). Originally, it was planned that Zone 3 and 4 material and riprap would either be quarried and crushed granitic rock obtained from the ridge near the left abutment of the Dam or hauled from designated sources outside the project area. The materials investigation revealed insufficient quantities of clean gravels within reasonable haul distance of the site and demonstrated that crushed rock would be the most economical alternative. The gradation of Zone 3 material was established on the basis of the Terzagi filter criteria with respect to the gradation of Zone 2 (silty sand) and Zone 4 materials.

An on-site quarry was established as the source for riprap. Average rock size required for riprap was 170 pounds, ranging from 20 to 2,000 pounds. The horizontal width of the riprap layer was established at 10 feet. Crushed rock bedding was provided beneath the riprap for a transition to the fine-grained underlying embankment material. A width of 12 feet satisfied the minimum thickness requirements and allowed room for efficient placement and compaction.

Foundation. The main portion of the foundation for Perris Dam is alluvium within a broad valley between two granitic ranges, Bernasconi Hills and Russell Mountains. The alluvium, consisting of a mixture of silty sand and clayey sand, is deposited over granitic bedrock. A layer of decomposed granite, up to 60 feet in thickness, overlies much of the intact bedrock. The alluvial foundation is divided by granite outcrops near the central region of the site. North of the outcrops, the alluvium is relatively shallow, averaging less than 40 feet. South of the outcrops, the bedrock dips forming a subsurface canyon, and the maximum alluvium depth is over 250 feet.

The left abutment of the Dam is granitic rock of the Bernasconi Hills, whereas the right is formed by rising alluvial ground surface. The embankment is founded on rock in the vicinity of the outcrops.

Instrumentation. Instrumentation is provided at Perris Dam to monitor embankment and foundation settlement, internal pore pressures, and response of the embankment and foundation to ground motions from earthquakes (Figure 283). Facilities at the site are consistent with those provided at other project dams; however, portions of the instrumentation system are somewhat more elaborate because of the unusual foundation conditions.

Control panels for monitoring instrumentation are located in two terminal buildings. Instrumentation and other features of the performance monitoring system for the Dam and Lake are described in Table 37.

TABLE 37. Features of Performance Monitoring System—Perris Dam and Lake Perris

1. Lake horizontal movement net:
   Total of 13 instrument stands.
2. Level line:
   25 miles of bench marks around Lake; 1 kilometer spacing.
3. Lake-level gauge.
4. Embankment and foundation instrumentation:
   24 piezometers.
   27 crest settlement monuments.
   5 accelerometers; crest of dam, on foundation below crest, near the rock-alluvium contact 77 feet below the embankment, in alluvium downstream of the Dam, and in rock outcrop downstream of the Dam.
   2 cross-arm settlement devices.
5. Seismographs:
   1 instrument located at Instrumentation Terminal No. 1.
   1 instrument installed on the dam crest.
6. Open-tube piezometers:
   13 cased holes downstream of Dam.

Inlet Works

Description. An 8½-foot-inside-diameter inlet pipeline joins the Santa Ana Valley Pipeline at vent structure No. 1 on the crest of the hill near the right abutment of Perris Dam (Figure 284). The inlet pipeline, located in a cut section, descends 190 feet in elevation. Ninety feet of drop occurs in the first 400 feet of length. It then rises slightly over the next 670 feet before descending at vent structure No. 2 over the final 2,630 feet to the terminus at the outfall basin.

The outfall basin structure is located beneath the lake surface and is protected by a 30-foot-wide concrete apron extending beyond the end of pipe on a 2:1 slope. A terminal stilling basin, 70 feet wide, 150 feet long, and 35 feet deep, was provided to minimize churning of silt into the Lake by water jet.

Because the inlet works is located near a recreation area, the selected design was based on maximum safety and minimum adverse environmental effects on the surrounding area. During the preliminary design stage, three alternatives were investigated. Two open-chute alternatives with high-velocity flow were eliminated for public safety reasons. The high-velocity buried pipeline alternative best satisfied the above considerations at a feasible cost.

Hydraulics. The inlet works was designed to satisfy two hydraulic requirements for conveying project water. The first requirement was a summit invert elevation of 1,698 feet. This elevation provides sufficient static head to achieve the required flow at upstream turnouts on the Santa Ana Valley Pipeline. The second requirement was sizing the inlet works to convey a maximum flow of 560 cubic feet per second (cfs) into the reservoir.

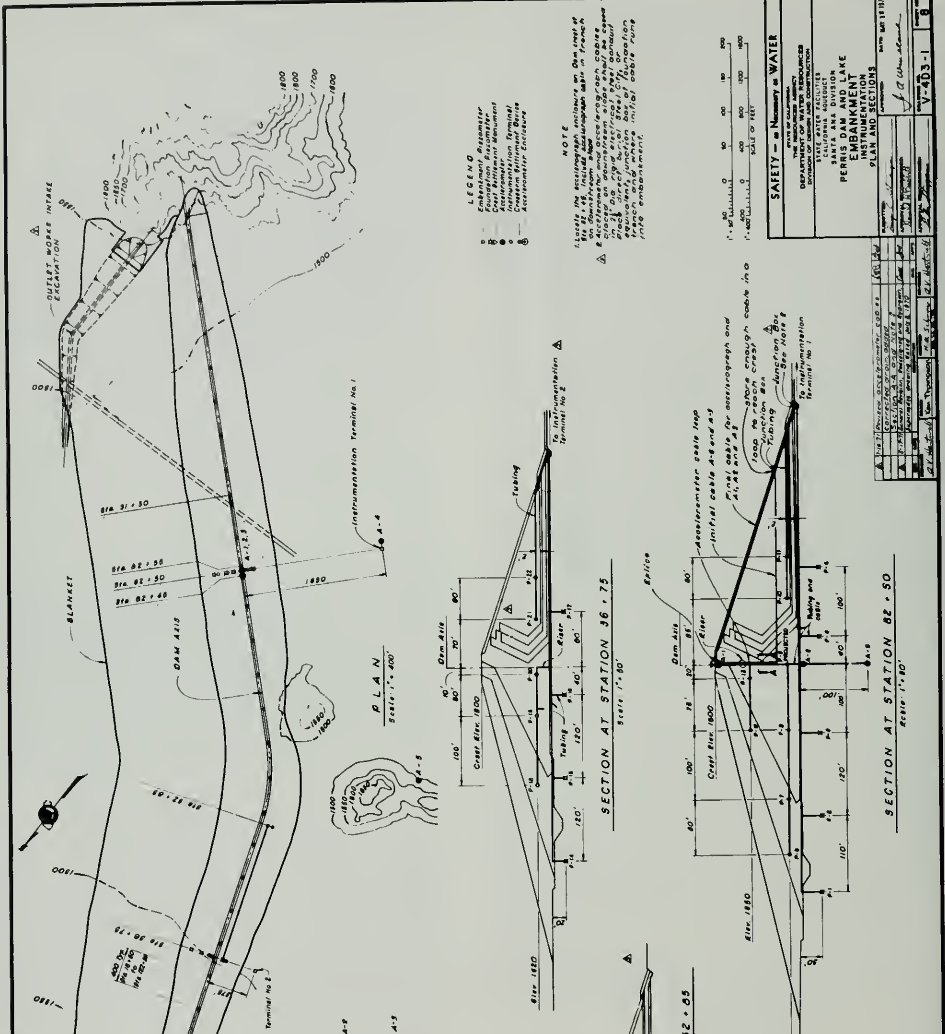

Figure 283. Embankment Instrumentation

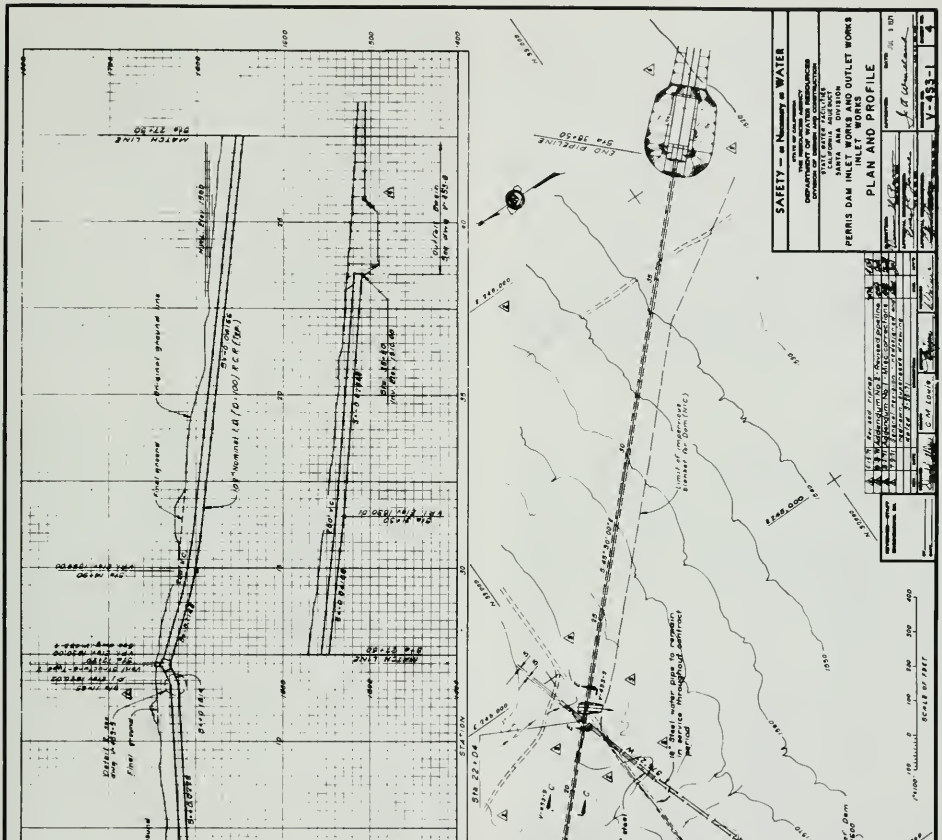

Figure 284. Inlet Works—Plan and Profile

The energy of the water downstream of the Pipeline summit partially is dissipated by hydraulic jumps inside the Pipeline. The profile grade was selected to prevent the jump from being swept out. The Pipeline was sized to prevent blowback due to air entrainment induced at the jump. Vents are provided at locations where pipe flow changes from full to partially full.

Because the maximum velocity is in excess of 60 feet per second, 2 inches of concrete cover over the inside reinforcement was selected for protection against abrasion. The pipe joints were beveled to minimize cavitation.

Pipe Structural Design. A low-pressure (43 pounds per square inch) reinforced-concrete pipe was selected to resist internal hydrostatic pressure due to the hydraulic jump and to support 20 feet of earth cover.

Outlet Works

Description. The outlet works is located at the left abutment of the Dam and consists of a multiple-level, vertical, intake tower; concrete and steel-lined tunnel; and concrete-encased, steel-pipe, delivery facilities (Figure 285). The delivery facilities connect to the water user's distribution facilities and provide a gated outlet to the downstream channel. Control and monitoring of the outlet works take place primarily from a control building jointly used by the Department and MWD.

Intake Channel. The purpose of the intake channel is to convey reservoir water to the outlet works tower when the reservoir water surface elevation drops below the original ground surface at the tower site. The channel has a trapezoidal cross section 40 feet wide at the bottom with 4:1 and 3:1 side slopes and is partially lined with impervious earth blanket.

Outlet Works Tower. The cylindrical, reinforced-concrete, outlet works tower with a 26-foot inside diameter is 105 feet high above foundation elevation and is capped by a deck supporting a 20-ton gantry crane (Figure 286). The tower contains 10, hydraulically operated, 72-inch, butterfly valves which release water to the outlet works tunnel from five selected levels within the reservoir, two valves per level. The quality of water released can be selected. Two tiers of valves (four valves) at the selected withdrawal depth are opened for delivery at the maximum rate (1,000 cfs). Steel cylindrical hoods on the discharge end of the eight upper valves direct the flow downward. Directing the flow downward minimizes the possibility of air entrainment and damage to downstream components of the outlet works. Hoods are not provided for the lower valves as no change in flow direction occurs between the valve and tunnel. A movable fish screen outside of the operating intake valves excludes small debris and fish from the system.

The tower was designed to resist stresses resulting from the vertical dead load and the effect of seismic forces with full reservoir conditions. For the dynamic analyses, the tower was assumed to act as a vertical cantilever fixed at the base.

The seismic forces on the concrete shell of the tower include lateral inertial forces due to dead load and lateral dynamic forces due to two cylinders of water with diameters equal to the inside and outside diameters of the tower. The earthquake input was 60% of the San Andreas design earthquake acceleration spectrum, as recommended by the Department's Consulting Board for Earthquake Analysis. This design spectrum suggests a horizontal acceleration of one-half gravity for rigid structures. The input was reduced because foundation conditions, including fairly intact rock, were considered better than those for which the design earthquake was developed. Two percent of critical damping was used.

The moments and shears used for design were calculated by means of an elastic model analysis. Final design moments and shears were determined by computing the root-mean-square values of the moments and shears generated in the first five modes of vibration. The tower, because of its moderate height, is a relatively rigid structure with a first mode period of 0.33 of a second. For this reason, the higher modes do not contribute significantly to the seismic loading. Inelastic yielding, in the event that the design earthquake is exceeded, probably would be confined to one hinge area at the base of the tower.

The outlet works tower footing was designed to transfer all superstructure loads into the foundation rock by means of a spread footing anchored to the rock. It was assumed that the footing would receive the shears from the superstructure shell above and transfer them to the rock mass below, primarily by its shear resistance. The compressive stresses are transferred to the rock in direct bearing since the footing concrete was placed against an undisturbed rock surface. The tensile stresses due to overturning moment are transferred by the anchor bars which are grouted into the rock mass.

Access to the tower is provided by a 109½-foot-long 16-foot-wide bridge from the outlet works access road. This road follows the south lake shore to Bernasconi Pass and then connects to Ramona Expressway. The bridge was designed for HS 20-44 loading.

Outlet Works Tower Mechanical Installation. Ten 72-inch-diameter, rubber-seated, butterfly valves were installed in the intake tower in two vertical rows. The port valves can be operated locally from the control cabinet on the tower operating deck and remotely from the joint MWD-Department control building.

The valves are intended to operate in the fully open or closed positions. Each of the valves in one vertical row, however, is capable of operating partially opened at differential heads of up to 93 feet and discharging free flows up to 60 cfs for filling the tower.

Each valve is actuated by its own hydraulic motor and screw drive-type operator. The valve operators are capable of opening or closing the valves under a maximum differential head of 30 feet and closing under full reservoir head. The hydraulic system for operating the valves consists of two vane-type oil pumps; an oil reservoir; solenoid-operated, 4-way, control valves; flow control valves; strainers; piping; valves; and appurtenances. The system was designed to operate with a maximum hydraulic oil pressure of 2,000 psi and operating time of 5 minutes per valve stroke. Two valves can be operated simultaneously.

A structural-steel maintenance platform stored in the tower above normal reservoir level is provided for servicing the upper four tiers of valves. The lower valve tier at elevation 1,503 feet is serviced from the tower base at elevation 1,495 feet.

One trolley of the 20-ton gantry crane raises and lowers the maintenance platform on stainless-steel guides embedded in the tower wall.

The platform was not designed to support the weight of the valves, operators, or hoods. These items must be removed from the tower during disassembly by the gantry crane.

A bulkhead gate is provided for closure of the butterfly valve intake openings to facilitate repair or removal of the butterfly valves. It is 9 feet - 4 inches high by 9 feet - 5¼ inches wide and weighs approximately 9,000 pounds. The bulkhead is stored off the tower in the on-site maintenance facility.

The gate, with lifting beam attached, is lowered vertically into slots on the outside of the tower by one of the 10-ton trolleys on the 20-ton gantry crane and dogged in front of the valve opening to be closed.

A 20-ton-capacity, electric, cab-operated, outdoor, traveling, gantry crane was installed on the tower deck. The crane operates on rails anchored to the tower deck and is equipped with two 10-ton-capacity trolleys which service both the inside and outside of the tower. The crane also is used to provide hoist service as necessary on the tower deck and inside the tower and to position the fish screens.

Capacities and speeds are as follows:

Rated capacity of crane, tons	20
Number of trolleys	2
Rated capacity each trolley, tons	10
Length of lift, feet	125
Length of travel, feet	13
Hoist speed, feet per minute (fpm)	14–20
Trolley travel speed, fpm	4–6
Gantry travel speed, fpm	4–6

Master control devices for the functions of the crane were installed on the console in the operator's cab.

Stainless-steel wire rope is used on the crane hoists since operation of the fish screens requires submerged service for extended periods of time.

Screens to cover operating intake ports are provided to prevent passage of fish and debris into the delivery system. The screens are ½-inch-square mesh covering structural-steel frames. One frame is suspended by the gantry crane over two operating valve tiers for each of the vertical valve rows. Rails are provided on the outside of the tower for guidance during up and down movement and for holding the screens horizontally in place.

A washing system designed to remove minor debris and algae from the fish screens is provided in the intake tower. Two washing stations, one for each fish screen, are located at elevation 1,591 feet.

A variable-spray-pattern wash nozzle is mounted on a flexible ball joint at each platform. Water to the washing stations is supplied by a six-stage, deep-well, vertical, turbine pump mounted at elevation 1,600 feet. The pump has a rated capacity of 90 gallons per minute at a total head of 265 feet and a speed of 3,450 rpm.

Outlet Works Tunnel. The outlet works tunnel is a 12-foot – 6-inch-inside-diameter pressure conduit about 2,100 feet long, located in the left abutment of the Dam (Figure 285). It conveys water from the outlet works tower to the delivery facilities which serve MWD (Figure 287). The tunnel was provided with a reinforced-concrete lining upstream of the axis of the dam impervious core. From the core axis to the downstream portal, steel liner with concrete backfill was used.

The steel liner was designed to withstand external pressure due to ground water equal to the depth of cover over the tunnel or to elevation 1,590 feet, whichever was greater. A study of types of liner plate indicated that, due to the greater stiffener spacing possible, the use of ASTM A572 steel was the most economical. Liner thickness was set at ½ of an inch so as to adequately withstand the entire internal hydrostatic pressure to elevation 1,590 feet and to provide rigidity for handling.

The reinforced-concrete lining upstream of the axis of the impervious core was designed to withstand external hydrostatic head due to reservoir water surface at elevation 1,590 feet. The section reinforcement was designed to withstand the entire internal pressure, with the hydraulic gradeline at the same elevation. An overstress of 25% was allowed for transient overpressure loading. A nominal lining thickness of 18 inches was used for most of the tunnel.

Outlet Works Delivery Facility. The delivery facility is located adjacent to and below the left abutment and delivers water to MWD's Perris control facility (Figure 287).

The delivery facility consists of a 12-foot - 6-inch-diameter steel conduit extending from the west portal of the outlet works tunnel to a 12-foot - 6-inch-inside-diameter (ID) service outlet manifold, a service outlet manifold, three branch lines extending from the mani-

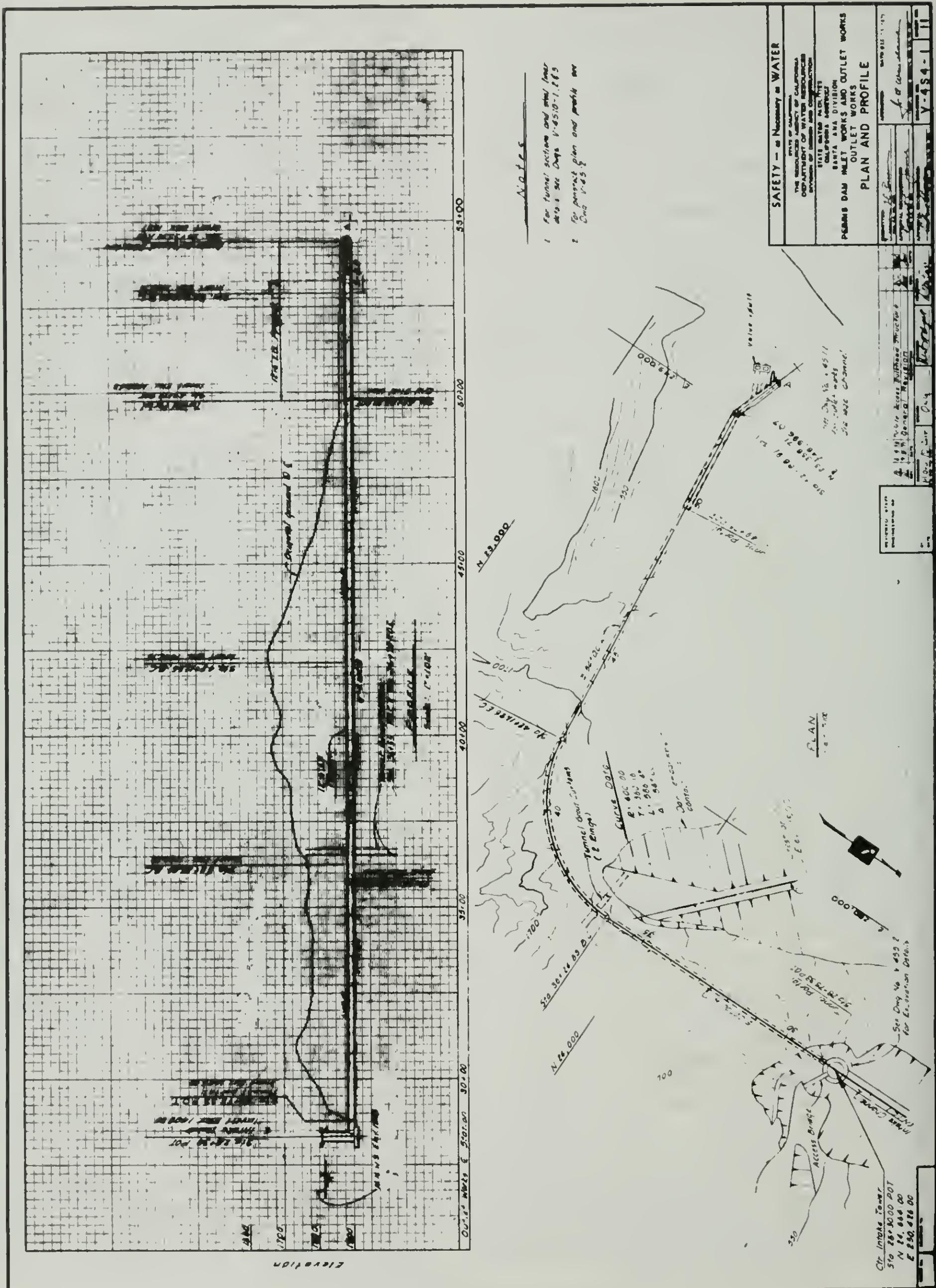

Figure 285. Outlet Works—Plan and Profile

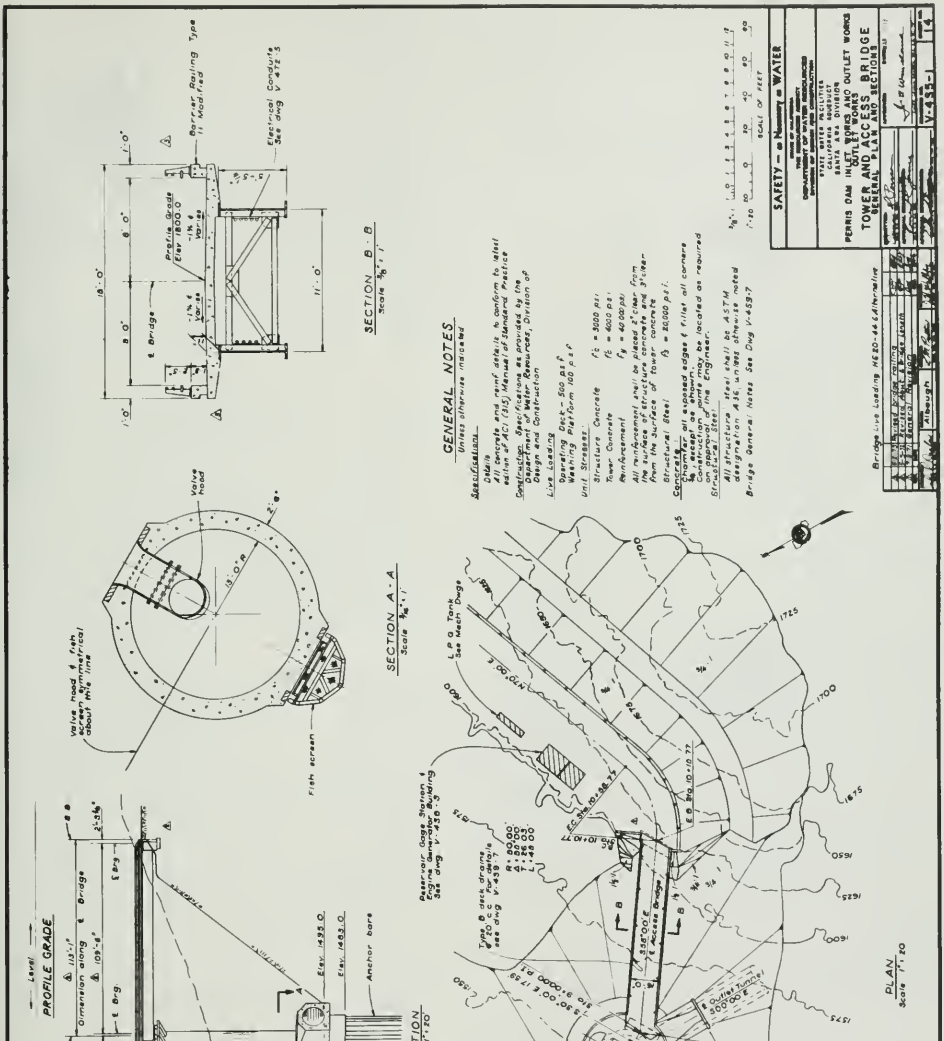

Figure 286. Outlet Works Tower

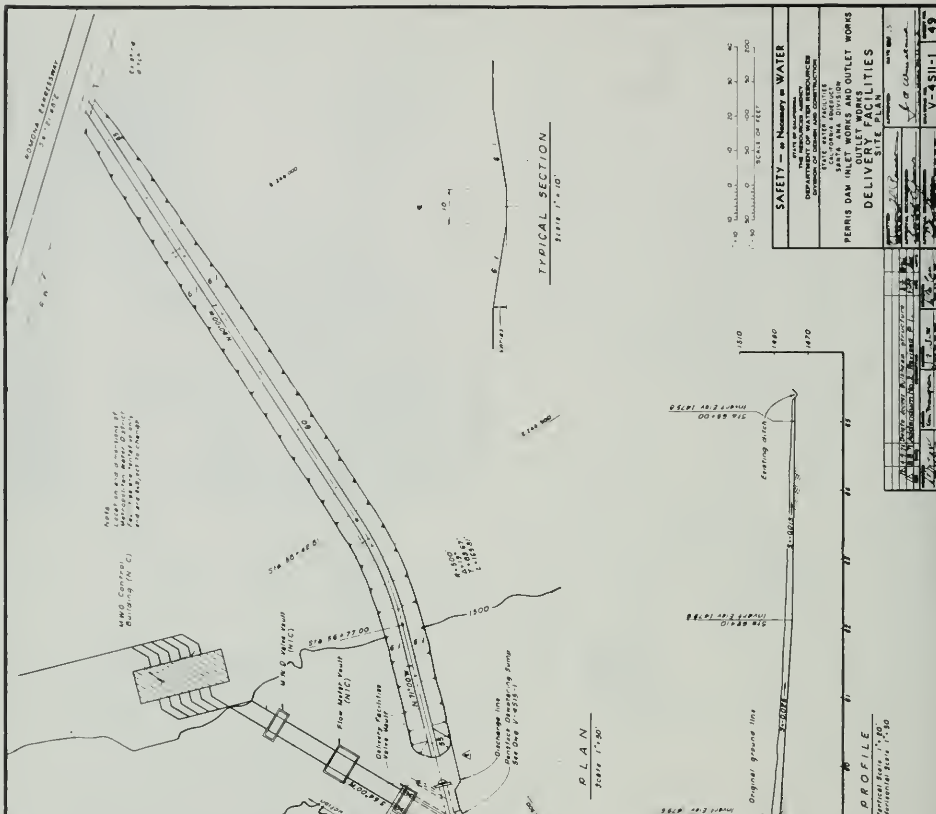

Figure 287. Outlet Works Delivery Facilities

fold, and a terminus release gate. The conduit, manifold, and branch lines are all concrete-encased with the approximate length of conduit and manifold 353 and 141 feet, respectively. The approximate overall lengths of the 4-foot - 6-inch ID branch line and the 6-foot - 6-inch ID branch line are approximately 110 and 88 feet, respectively, measured from the centerline of the service outlet manifold. Each of these lines near the downstream end is equipped with a butterfly valve. The two valves are located in a single valve vault and are accessible through hatch covers. The valves are operated in the fully open or fully closed position only and thus are not used to regulate flow. Regulation is accomplished by the MWD control facility. The third branch line, a provision for future expansion, has a 7-foot - 6-inch ID and terminates with a dished bulkhead.

A blowoff structure was provided at the service manifold terminus to permit emergency evacuation of the Lake. The original design involved a spherical head at the outlet manifold terminus. To remove the head with the pipe full of water would require the use of explosives. Subsequent to the construction period, this evacuation concept was reconsidered and a 6-foot by 12-foot slide gate was provided to increase the flexibility of the system. The maximum discharge through the slide gate is approximately 3,800 cfs. A bolted bulkhead is installed downstream of the gate to ensure that large releases cannot be made inadvertently and to allow for exercising the gate.

To avoid differential settlement, the conduit and manifold were designed to be supported on sound rock. Bedrock of an irregular nature was overexcavated and backfilled with concrete.

The 6-foot - 6-inch branch line has a capacity of 325 cfs, and the 4-foot - 6-inch line has a capacity of 175 cfs. The additional stubbed-off branch line provides for a total ultimate capacity of 1,000 cfs.

The 12-foot - 6-inch conduit provides a velocity of approximately 8 feet per second (fps) at a discharge of 1,000 cfs. The conduit also provides for a maximum release of 3,800 cfs at 31 fps through the slide gate at reservoir water surface elevation 1,588 feet.

Although the tunnel liner and the delivery facility conduits are encased in concrete, the total internal pressure is resisted by the steel conduit. Allowable stress for the maximum load case is 25,000 psi. Maximum internal pressure occurs with hydraulic gradeline at elevation 1,590 feet, plus 25% of the total static head for transient overpressure. Welded joint efficiency was considered to be 100% and all joints were required to be radiographed. The required minimum 28-day strength of the concrete was 3,000 psi.

Delivery Facility Mechanical Installation. One 78-inch-diameter and one 54-inch-diameter valve are located in branch conduits near the interface with facilities of the water contractor (MWD). The valves are housed in a common vault. Each valve is metal-seated with a hydraulic cylinder operator and a common hydraulic control system. Each delivery facility valve was designed for a working water pressure of 50 psi and to withstand a 50-psi differential water pressure across the closed valve disc from both the upstream and downstream direction.

Each delivery butterfly valve was designed to withstand opening and closing under the following conditions:

Valve Size (inches)	*Constant Head Upstream of Valve (feet)*	*Maximum Flow During Opening of Valve (cfs)*	*Maximum Flow During Closing of Valve (cfs)*
54	100	175	610
78	100	325	1,280

Both valves and their operators primarily were designed for fully open or fully closed operation, but the valves can be closed under emergency conditions, either independently or simultaneously, in approximately 5 minutes.

The hydraulic control system was designed for an operating pressure of 2,000 psi and, in order to minimize the cycle time for the oil pumps, a bladder-type hydropneumatic accumulator is used to maintain pressure on the system to keep the valves fully open.

Since Perris Dam is located in an active seismic area, a seismic acceleration of 0.5g was used in the design of the valves and their appurtenances.

One large dewatering and two small drainage pumps are located in a pump-house structure adjacent to the main outlet works conduit. All three pumps are complete with automatic controls, discharge lines, and appurtenances. The dewatering pump is used for dewatering the main outlet works conduit, and the drainage pumps are used to remove any extraneous water from the valve vault.

The dewatering pump is a vertical-shaft single-suction type with a rated capacity of 500 gallons per minute (gpm) at a total head of 30 feet. The drainage pumps are the submersible type with a rated capacity of 10 gpm, each at a rated head of 40 feet.

The outlet works release facility slide gate is located downstream of the delivery branches at the end of the delivery conduit. The gate provides a waterway 6 feet by 10 feet. The gate leaf is cast steel, 7.25 feet wide by 12.5 feet high, and weighs approximately 20,000 pounds.

The gate is operated by a hydraulic cylinder located on the concrete structure above the gate.

Preliminary design considered two gate-leaf alternatives: (1) a welded-steel gate, and (2) the cast-steel gate which finally was selected. Historically, high-pressure slide gates of this type have been cast. The basic configuration lends itself to sound casting practices and was found to be less expensive due to the extensive weldments in the alternative design.

A hydraulic operator is provided for opening and closing the slide gate. The hydraulic system consists of a hydraulic cylinder and piston rod, pumps, ac-

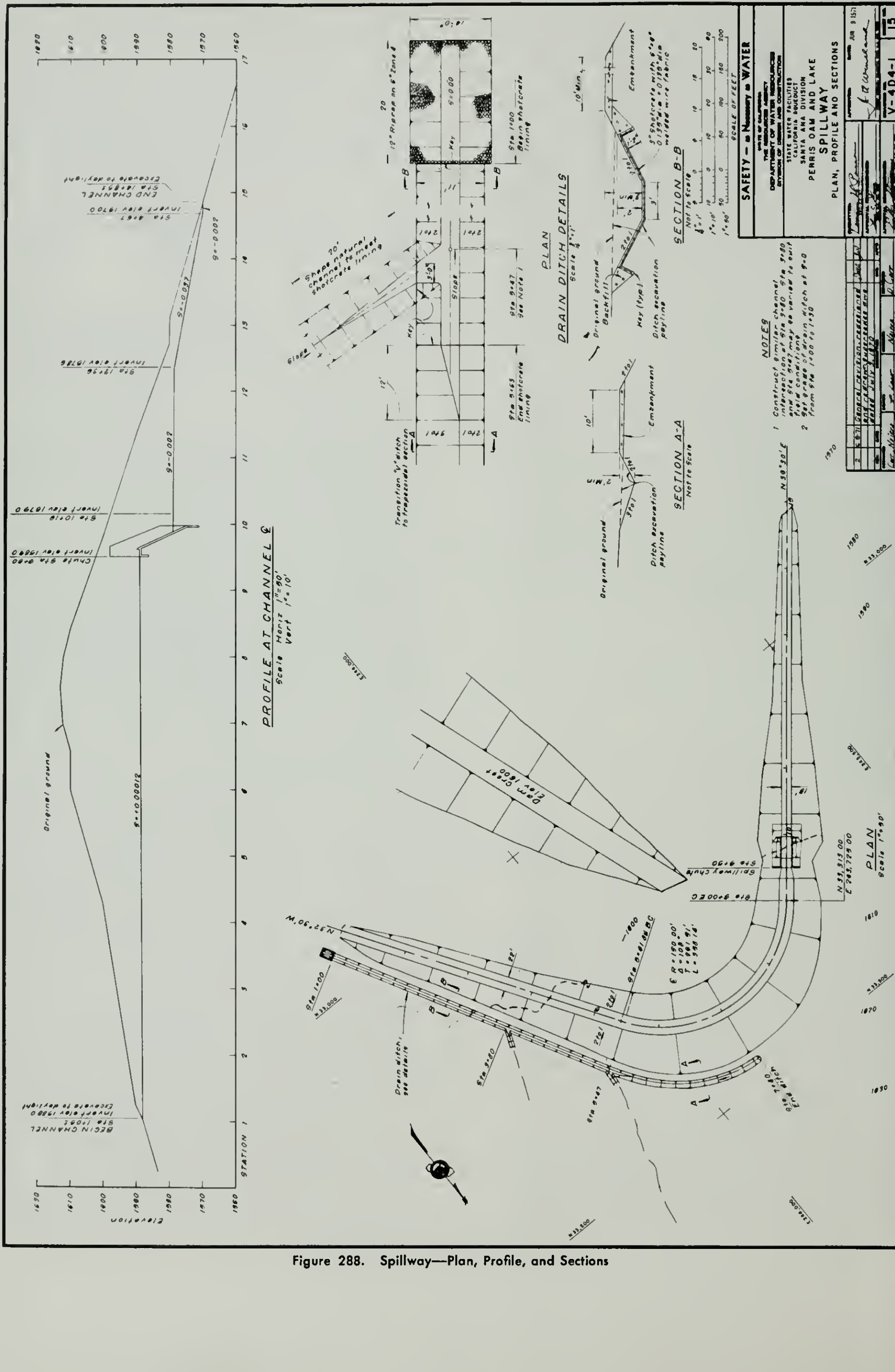

Figure 288. Spillway—Plan, Profile, and Sections

cumulators, and a control cabinet. The cylinder has a 31-inch bore and 146.5-inch stroke. The operator was designed to open the gate under an unbalanced head of 92 feet. The system operates at 1,500 psi and utilizes two pumps to obtain an opening and closing rate of approximately 1 foot per minute. If electrical power to the system is interrupted and emergency operation of the system is required, one pump is designed to receive power from the emergency engine-generator set and operate the hydraulic system. Two oil accumulators precharged with nitrogen are connected to the system to provide holding capacity in the up position. The system was designed for local-manual operation for opening and closing cycles.

The outlet works control room, located in the joint control building, contains facilities for controlling and monitoring contract water deliveries to MWD. Southern California Edison Company supplies 480-volt 3-phase power to the building which is located in close proximity to the delivery facility. The control room houses:

1. Controls for operating the tower and delivery butterfly valves.
2. Position indicators for MWD butterfly delivery valves and department valves.
3. Reservoir and tower water surface elevation indicators.
4. Venturi metering readout equipment.
5. Emergency power supply.

Spillway

Description. The spillway is located beyond the right dam abutment (Figure 288). It consists of an 850-foot-long, unlined, trapezoidal, approach channel 22 feet wide; a reinforced-concrete control structure; a concrete baffled chute; a short section of riprapped channel; and an unlined channel terminating far enough downstream to eliminate erosion adjacent to the toe of the Dam. The spillway crest elevation is 1,590 feet, a nominal 2 feet above the maximum operating water surface to prevent it from being overtopped by waves. The crest length is 16 feet. When the lake level was raised 10 feet, as discussed earlier in this chapter, no major changes were made in the channel or concrete structure, except for being moved vertically to accommodate the revised pool level and horizontally to best suit the topography.

Hydraulics. The spillway was designed for emergency use only because the probability of spilling is extremely remote. Normally, the reservoir will be drawn down during the winter season sufficiently so that the maximum probable flood volume (8,340 acre-feet) can be stored without spilling.

If the reservoir is full at the time of a flood, releases can be made to MWD through the outlet works. The sizing of the spillway was based on the requirement that a sustained inflow from the Santa Ana Valley Pipeline of 560 cfs can be discharged without excessive encroachment on the embankment freeboard. This discharge is greater than the routed outflow resulting from an occurrence of the maximum probable flood with full reservoir. The spillway rating curve is shown on Figure 289.

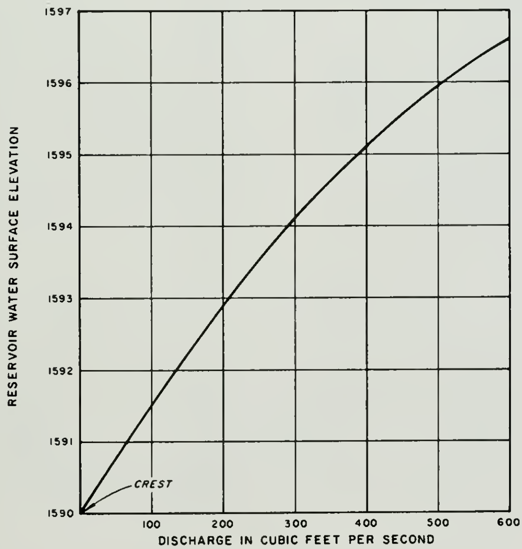

Figure 289. Spillway Rating Curve

Construction

Contract Administration

General information about the major contracts relating to Perris Dam is shown in Table 38. Perris Dam and its appurtenant structures were constructed under two main contracts.

The zoned earth embankment, spillway, and outlet works approach channel were constructed under the first contract (Specification No. 70-25). Most of the cost increase shown in Table 38 for this first contract resulted from the decision to increase the reservoir capacity to 120,000 acre-feet as described earlier in this chapter.

The inlet works and outlet works were constructed under the second contract (Specification No. 71-11).

Dam Foundation

Excavation. Four 6-foot-deep trenches were excavated approximately along the centerline of the Zone 1 embankment at Stations 25+00, 40+00, 88+00, and 105+00 to determine if low-density materials or other objectionable materials occurred within the designated excavation limits. Samples were taken from these pits to determine in-place densities and moisture contents and for relative compaction tests. By this process, it was determined that a minimum depth of excavation of 1 foot for the impervious reservoir blanket upstream of the Dam, 3 feet for Zone 2 embankment, and 6 feet for Zone 1 embankment would be necessary.

Excavation began in the blanket area near the right abutment. The first materials excavated were stockpiled in mandatory waste area No. 1 for later use in the blanket.

Excavation proceeded generally in a southerly direction from the right abutment toward the left, with the excavated materials used directly for construction of the reservoir blanket and Zone 2. During these operations, the waste strip was constructed at the downstream dam toe, as called for on the plans.

A trench was excavated along the center of the Zone 1 foundation, primarily for further exploratory purposes. This trench extended below the general Zone 1 foundation elevation, had a bottom width of 15 feet, and had a minimum depth of 10 feet, except where rock was encountered. At Station 90+00, sand lenses were encountered, and the depth of the trench was increased to 25 feet. Between Stations 24+00 and 27+00, an ancient streambed consisting of permeable clean sand was encountered and the trench was deepened about 20 feet. This trench, backfilled with Zone 1 material, forms a partial cutoff beneath the Dam.

During the geologic exploration for Perris Dam, three 40-foot-deep trenches were excavated by dragline beneath the dam foundation and later filled with loose material. These had to be reexcavated, and the material was replaced with compacted Zone 2 material.

The left abutment, which is on a granitic rock formation, required shaping and grouting. About 70,000 cubic yards of solid rock was excavated between Station 120+00 and the extreme left end of the Dam. Drilling and blasting were required for foundation excavation. Material removed beyond Station 122+00 was hauled from the embankment foundation to mandatory waste area No. 2. Rock excavation beyond Station 122+00 extended as much as 40 feet below original ground. Most of the excavation was required for shaping the foundation surface of the impervious Zone 1.

A small volume of rock also was excavated along the downstream toe of the dam embankment in the vicinity of Station 64+00. After it was decided to increase the reservoir capacity, it was determined that further rock excavation would not be necessary in this location.

Grouting. The grouting plan for Perris Dam consisted of a single line of curtain grout holes on an alignment starting 260 feet upstream of Station 120+00 and extending approximately 750 feet to the crest of the Dam at the left abutment. Due to the steeply dipping joints in the bedrock, the holes were drilled at an angle of 60 degrees from the horizontal. At Station 127+00, the curtain alignment angled to the east and extended up the rock face of the left abutment foundation to elevation 1,600 feet. Split-spacing

TABLE 38. Major Contracts—Perris Dam

	Perris Dam and Lake Perris	Perris Dam Inlet and Outlet Works	Completion of Perris Dam and Lake Perris	Completion Contract No. 2
Specification	70-25	71-11	73-01	74-39
Low bid amount	$27,394,995	$6,974,810	$2,051,552	$231,169
Final contract cost	$31,362,749	$7,197,504	$2,608,637[2]	$254,000 (Est.)
Total cost-change orders	$22,537,778[1]	$176,919	$260,847	--*
Starting date	10/10/70	10/13/71	3/29/73	9/5/74
Completion date	11/15/72	10/6/73	2/1/74	12/74 (Est.)
Prime contractor	Perris Dam Constructors	Perris Dam Constructors	Perris Dam Constructors	Jesse Hubbs & Sons

[1] Reflects bid-price adjustments.
[2] Includes $2,260,925 for recreational facilities.
* As of November 1974.

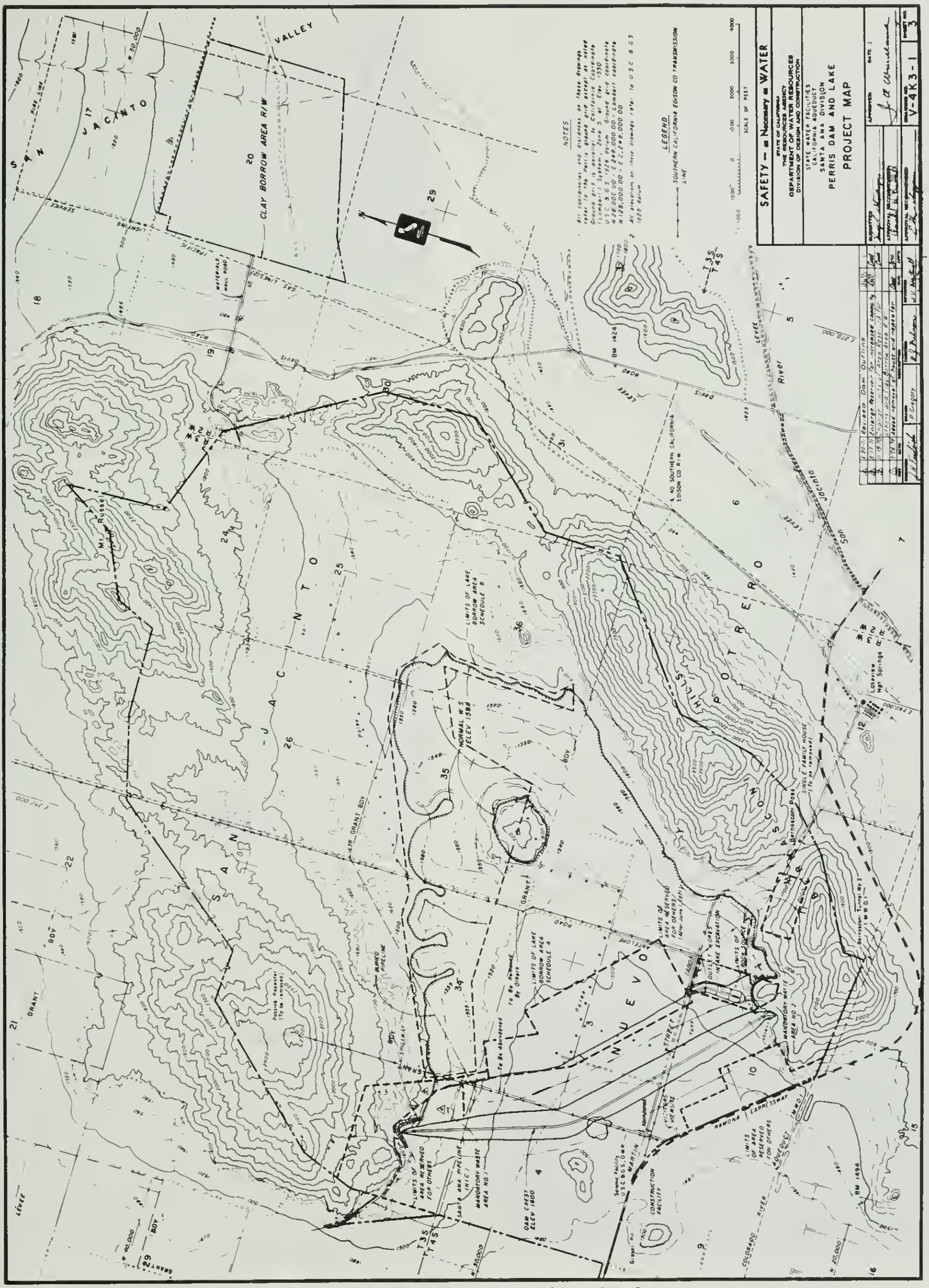

Figure 290. Location of Borrow Areas and Perris Dam Site

grouting techniques were used, and the final hole spacing was 10 feet. A grout cap was not employed, and the amount of grout injected was very small because of the tightness of the rock. Most of the grout take was due to shallow fissures in Zone 1 foundation; however, two intervals of high take were encountered at Station 120+00 and from Stations 124+80 to 125+60. It was estimated that 8,600 cubic feet of grout would be needed, but only 1,429 cubic feet were actually required.

Along the extreme left abutment, a rather deep fissure was encountered in the rock extending up the abutment to the full height of the embankment. This fissure contained broken rock and rubble that would have allowed the passage of surface water from the slopes above the left abutment down through the rock drain materials into the dam foundation. To prevent this, a concrete plug was placed against the rock contact line in the fissure at approximate elevation 1,595 feet, and a gunited surface ditch was constructed across the crest of the Dam at the extreme left abutment.

Handling of Borrow Materials

Clay Borrow Area. To obtain the estimated 8,670,000 cubic yards of Zone 1 material required for the Dam as originally designed, an excavation pattern was established having a maximum excavated depth of about 40 feet with provisions for draining the entire area. By contract change order, the estimated amount of required Zone 1 material was reduced to 4,690,000 cubic yards. Because the original surface area of the site available to the contractor was not changed, the required depth of excavation was reduced drastically.

The clay borrow area (Figure 290) was located in a lake area subject to inundation during periods of heavy rainfall, but rainfall during the construction period was small and did not interfere with borrow operations. Advance moisture conditioning, when necessary, was accomplished by a sprinkler system.

Clay borrow was excavated with a tractor-mounted belt loader (Figure 291) and hauled directly to the Dam using 100- to 150-ton-capacity bottom-dump trucks. Due to the presence of material in the clay borrow area which contained excessive amounts of silt and sand unsuitable for Zone 1 embankment, the final configuration of this borrow area was somewhat irregular in shape. It was shaped, however, to drain with maximum slopes of 10:1. Topsoil material was stripped from the borrow, stockpiled around the outer perimeter, and replaced on the excavated surfaces after completion of construction.

Figure 291. Excavation in Clay Borrow Area

Figure 292. Excavation in Lake Borrow Area

Lake Borrow Area. Two alternative lake borrow plans (within the reservoir area) were included in the contract specifications and each required separate bids. One alternative utilized a simple geometric pattern for excavation in the reservoir close to the Dam and the other required a more complex excavation configuration for the recreation development along the north reservoir margin. Although the borrow associated with the recreation plan was bid slightly higher, the amount was well within the facility allocation and therefore the contract was awarded on the basis of that alternative.

Zone 2 material was obtained from the lake borrow area which extends along the northerly shoreline a distance of approximately 4 miles. The shoreline was excavated in a series of coves and fingers of land to provide maximum potential for recreation. Excavation was accomplished with the same type of equipment as was used in the clay borrow area (Figure 292).

The original contract provided for stripping of all vegetative matter from the borrow area and placing it upon the fingers of land which were mandatory waste areas. No provisions were made for ultimate development of these waste areas, and the strippings as placed did not bring them above the originally contemplated high water line. When the reservoir capacity was increased from 100,000 to 120,000 acre-feet, provisions were made to relocate this shoreline to conform to the corresponding higher water surface. At the request of the Department of Parks and Recreation, sufficient material was excavated from the lake borrow area to

Figure 293. Rock Production

place the mandatory waste areas above the normal lake level and provide for maximum recreational use.

The embankment drain system consists of 1.3 million cubic yards of coarse material (Zone 4) of 6-inch maximum size and 1.5 million cubic yards of transition material (Zone 3) of 1½-inch maximum size. Approximately 240,000 cubic yards of riprap was used for slope protection, upstream and downstream. All of the above materials, with the exception of approximately 14,000 cubic yards of Zone 3 material, were obtained from the designated rock source located upstream of the left abutment of the Dam.

The rock source was laid out on benched faces extending from a maximum elevation of approximately 2,070 feet to the lowest level at approximate elevation 1,665 feet. The height of the faces varied from 50 feet to 30 feet. The rock was blasted from the quarry and hauled to the nearby rock-crushing plant located east of the quarry (Figure 293).

Initially, an excessive amount of fines were produced, due mainly to overburden contamination and excessive blasting. Plant operations were revised to separate these fines.

Coordination of the placement of Zones 1 and 2 with Zones 3 and 4 initially was deficient because the loading and hauling equipment used was capable of placing Zone 1 and 2 material faster than the rock-crushing plant could produce the material for Zones 3 and 4. This lack of coordination resulted in nonuniform elevations of the various embankment zones transverse to the dam axis. In an attempt to reestablish uniform zone elevations, the contractor ceased placement of Zones 1 and 2 and purchased supplemental Zone 3 material from an alternate source. However, only 14,000 cubic yards of supplemental Zone 3 material was placed before heavy rains caused that effort to be terminated.

The quarry and crusher plant operations were later revamped, and the rate of production of Zone 3 and 4 material increased sufficiently to permit the zone elevations to be regained, and the embankment construction proceeded to completion.

Embankment Construction

The Dam was constructed in a series of three reaches. Embankment construction began at the lowest point along the foundation in the vicinity of Station 85+00. When the grouting was completed at the left abutment, efforts were concentrated in that area. Embankment construction then proceeded with a slight slope toward the north to enable loaded trucks to climb to the higher area where embankment was being placed. General embankment construction activity is shown on Figure 294.

The upstream face of the Dam from the crest down to elevation 1,540 feet was faced with a layer of riprap 10 feet in horizontal width underlain by a layer of Zone 4 bedding material 12 feet in horizontal width. Riprap and bedding were placed continuously as the other embankment zones were placed. To reduce congestion, ramps were constructed down the upstream face of the Dam for use by empty trucks. These ramps consisted of Zone 4 material placed upon the riprap blanket and overlain with Zone 3 material. Zone 3 material was removed upon completion of each section but Zone 4 material was allowed to remain. A layer of riprap also was placed along the downstream toe of the Dam above the toe drain.

A layer of topsoil, having a horizontal thickness of 12 feet, was to have been placed on the downstream face of the Dam above the riprap. It was anticipated that this material would be stripped from the dam foundation and stockpiled. Because there was little vegetative matter in this material, it was used in the lower layer of the upstream blanket, and no stockpiling was done. Zone 2 material was used in place of topsoil for the downstream facing.

The original design of the embankment provided for a layer of blanket material 3 feet thick extending 1,000 feet upstream from the toe of the embankment. Blanket material was obtained primarily from excavation of the dam foundation and was the first work undertaken.

When the decision was made to forego any future

Figure 294. Embankment Construction Activity

enlargement of the Dam above crest elevation 1,600 feet, the 8:1 embankment fillet at the upstream toe no longer was needed for ultimate stability. Elimination of this fillet would have resulted in movement of the toe downstream for a considerable distance. By then, however, the work on the blanket had progressed for about four months; thus, no change was made and the blanket was constructed to the original upstream limits. The area between the original and revised upstream toes was covered with Zone 2 material for at least a 3-foot depth.

The initial problem in producing Zone 3 and 4 material for the horizontal drain in sufficient quantity (previously discussed) created the necessity of constructing Zone 1 and 2 material in lifts approximately 3 feet in height and wide enough to allow for placement and compaction. When the horizontal drain was brought to its maximum required depth, the placement of Zone 1 and 2 material proceeded in a more efficient manner.

Zone 1 material was placed in layers not exceeding 6 inches in compacted thickness. Compaction was achieved by 12 passes of a sheepsfoot roller. The initial 12-inch depth of Zone 1 material next to rock foundation was compacted in 4-inch lifts with rubber-tired rollers. The optimum moisture content of this material averaged 14% with an average field dry density of 114 pounds per cubic foot. The relative compaction obtained was approximately 98%.

Zone 2 material also was placed in layers and compacted to a 6-inch thickness by four passes of a sheepsfoot roller, followed by four passes of a pneumatic roller (Figure 295). The facing and blanket materials were placed and compacted in the same manner as the Zone 2 material.

Two test fills were constructed in Zone 2 to determine the compaction characteristics of the materials for the zone. One fill was compacted with 75-ton rubber-tired rollers and the other with 50-ton rollers. Average densities obtained with either roller were within specification requirements. After a further trial period, the 50-ton rollers continued to prove satisfactory and they were used throughout construction.

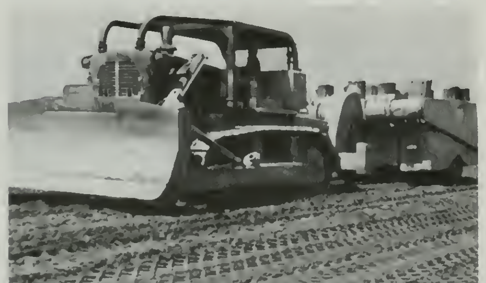

Figure 295. Pneumatic Roller on Embankment Zone 2

The average compacted dry density of this material was 125.9 pounds per cubic foot, and the average optimum moisture was 11.2%. The relative compaction averaged 101%.

Zones 3 and 4 comprise the horizontal and vertical drains. The horizontal drain consists of a lower 3-foot layer of Zone 3 overlain by a 6-foot layer of Zone 4 and topped with another 2½-foot layer of Zone 3 material. The vertical drain has a reverse slope configuration extending from the upstream end of the horizontal drain to an elevation approximately 12 feet below the dam crest (Figure 281). The drain consists of two 12-foot-wide Zone 3 strips enclosing a 12-foot-wide Zone 4.

Zone 3 was placed in layers not exceeding 18 inches in compacted thickness with a moisture content not exceeding 5% by weight. It then was rolled with two passes of a vibratory roller weighing over 20,000 pounds, operated at a frequency from 1,100 to 1,500 vibrations per minute. Zone 4 was placed in layers not exceeding 24 inches in compacted thickness and rolled with one pass of the same roller used for Zone 3.

The riprap was hauled to the site from the quarry in end-dump trucks. It was dumped directly upon the slopes and placed in final position with a grader.

The design provided for a longitudinal drain trench at the downstream toe of the Dam. The rock drain zones join this toe drain, which consists of a perforated, concrete, drain pipe 12 inches in diameter enveloped with drain rock.

The downstream face of the Dam was hydromulched immediately after construction to prevent surface erosion; however, the stand of grass obtained by the first winter was insufficient. Fine material eroded from the embankment was deposited in both the toe drain material and the drain pipe, necessitating extensive cleanup. To prevent any recurrence, major revisions were made along the toe of the embankment. Layers of Zone 4 and Zone 3, topped by a paved surface, were placed along the top of the riprap blanket to intercept surface runoff from the slope above. Downdrains carry the water over the toe drain to the original ground beyond. Cleanouts for the perforate toe-drain pipe are provided by 48-inch access wells at 400-foot intervals.

Better erosion protection than originally provided by the hydromulching process was supplied by applying straw at the rate of 4 tons per acre and incorporating it into the soil with a roller.

Instrumentation previously described (Table 37) was observed from the time of installation to completion of construction. Operations and maintenance personnel continue monitoring all instruments. The accelerometers had to be set at a higher than normal triggering threshold during completion of embankment construction because construction equipment set off recording devices.

Mandatory Waste Area No. 2

Mandatory waste area No. 2, located upstream from the left abutment of the Dam, was intended primarily to provide access to the Perris Dam outlet tower bridge. After it was determined that there would be insufficient material wasted in this area to provide the required access, revisions were made to require minimum limits of the waste area, which then was filled completely, thus supplementing the waste with material from the lake borrow area.

Inlet Works

Trench excavation for the inlet pipeline required blasting in areas between Station 1+20 and Station 14+90 (Figure 281). A bulldozer, equipped with a ripper, and a dragline removed the rock from the excavation. For pipe bedding and backfill, the contractor was permitted the option of consolidating (by saturation and vibration) imported free-draining material, consolidating excavated rock after processing for specified gradation, or compacting excavated semi-impervious alluvial materials. The alluvial material was used and backfill was compacted with small, self-propelled, vibrating compactors for the first 6 feet of cover. The remaining backfill was compacted with a sheepsfoot roller.

Outlet Works

The channel invert was excavated with scrapers on a level grade for a length of approximately 2,700 feet. The maximum depth of excavation was 40 feet. Three feet of Zone 2 blanket material was placed over the entire excavation surface. Excavated material was placed in the reservoir blanket area. Only a negligible amount of rock excavation was necessary.

Two steel girders were set into place with two cranes after being sandblasted and coated with inorganic zinc silicate at the job site. The bridge deck was formed, and three gradelines were set to guide the finishing machine during concrete placement.

The entire tower basin between elevation 1,495 feet and 1,482 feet was excavated in sound rock by drilling and blasting. The shattered rock was loaded on dump trucks with rock beds and hauled to a mandatory waste area. The 50-foot-diameter tower base was excavated by drilling and blasting with material being removed by a crawler crane equipped with a 5-cubic-yard dragline bucket. Material was loaded and hauled away as described in previous paragraphs.

The 9-inch holes for the foundation anchor bars were drilled, holes were filled with grout, and No. 18 anchor bars were vibrated into place with a form vibrator. Concrete for the tower base then was placed.

The interior form for the tower barrel was erected to full height with the aid of an interior scaffold. The interior form consisted of a double thickness of plywood backed by wooden strongbacks and metal walers. After the reinforcement steel was erected, port thimbles and valves were securely set in place. The thimbles were supported on pipes set in the previous concrete lift of the barrel.

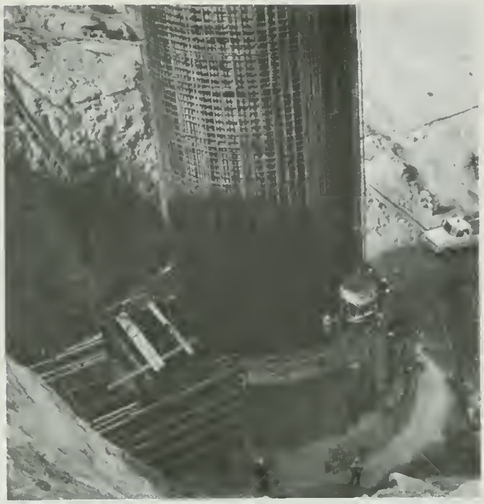

Figure 296. Tower Concrete Placement

The exterior forms, comprised of a metal shell with Finn forms attached, permitted placements of concrete in 16- and 17-foot lifts (Figure 296). Six lifts were cast (excluding the base and the tower deck) using 1½-inch maximum size aggregate concrete except around the thimbles where ¾-inch aggregate was used.

Forms for the last 5 feet of the tower barrel and the tower deck were prefabricated in the contractor's yard and set on the tower using a crawler crane.

Concrete for the intake tower and other structures was produced in a 150-cubic-yard-per-hour-capacity batch plant located near the outlet works. Aggregates were obtained from a commercial source in Riverside. Concrete was moved to the work site in 7-cubic-yard transporters. Water used to cure the concrete was applied by a sprinkling system.

Most of the outlet works tunnel was driven through hard, fresh, granitic rock, but some weathered and fractured rock was encountered in the vicinity of the portals. The tunnel was unsupported except for a few steel sets at each portal and a few rock bolts at several places in the tunnel (Figure 297). Ground water did not constitute any problem and only minor seepage was encountered.

After holing through, about five weeks were required to remove protruding ribs of rock within the tunnel excavation control line. A concrete subinvert was placed tangent to the control line to facilitate the setting of steel-liner sections and concrete-lining forms.

The 40-foot-long steel-liner sections of ½-inch plate

Figure 297. Tower Outlet Portal

Figure 298. Outlet Works Delivery Manifold

Figure 299. Concrete Placement in Spillway

were longitudinally placed and seam-welded. A concrete pump was used to place the concrete backfill. A wire-mesh bulkhead was placed at the end of each section to contain the concrete. All tunnel concrete was mixed at the on-site batch plant.

Tunnel grouting consisted of contact grouting throughout with additional skin grouting next to the steel liner. Grout was mixed outside of the tunnel. After mixing, the grout was pumped to a hopper inside the tunnel where it was remixed. From the remix hopper, the grout was pumped into the grout lines.

Skin grouting was accomplished through ½-inch holes drilled in the steel liner. These holes were plugged and welded upon completion of grouting.

Due to exceptionally good rock, no consolidation grouting was necessary. Grout curtains were located at Station 36+55 and Station 36+70.

Outlet Works Delivery Facility

The construction of this facility and associated appurtenances was performed in conjunction with other outlet works construction (Figure 298). No unusual construction methods were employed and no difficulties were encountered.

Spillway

Excavation for the spillway was made initially with a crawler crane equipped with a 5-cubic-yard dragline bucket and later with a small scraper. As excavation progressed, it became apparent that the original location of the foundation for the concrete-slab crest structure would be unsatisfactory. The crest structure was relocated 100 feet upstream where sound rock had been encountered. Approximately 100 cubic yards of concrete was placed in the spillway chute and weir (Figure 299).

Clearing and Grubbing

Clearing and grubbing consisted of the removal of all trees, concrete, and other debris in the contract area, as well as the removal of all vegetation over 1 foot in height within the lake area. Because of the concern for Russian thistle control, the reservoir area below elevation 1,578 feet had to be kept continually clear of all vegetation over 1 foot in height during the entire construction period.

Tumbleweed control in the entire project area was necessary to adhere to provisions of the California Agricultural Code. Tumbleweeds were cut and windrowed. Spring tooth harrows then were pulled through the windrows. Burning the tumbleweeds before harrowing yielded excellent results.

In addition to the work required within the reservoir area, a house, garage, and passive radio-repeater station within the project area were removed. All trees, broken concrete, rubble, and other debris within the reservoir area were buried in a pit located 3,200 feet upstream from the Dam. The major portion of the initial clearing and grubbing was completed in November 1970.

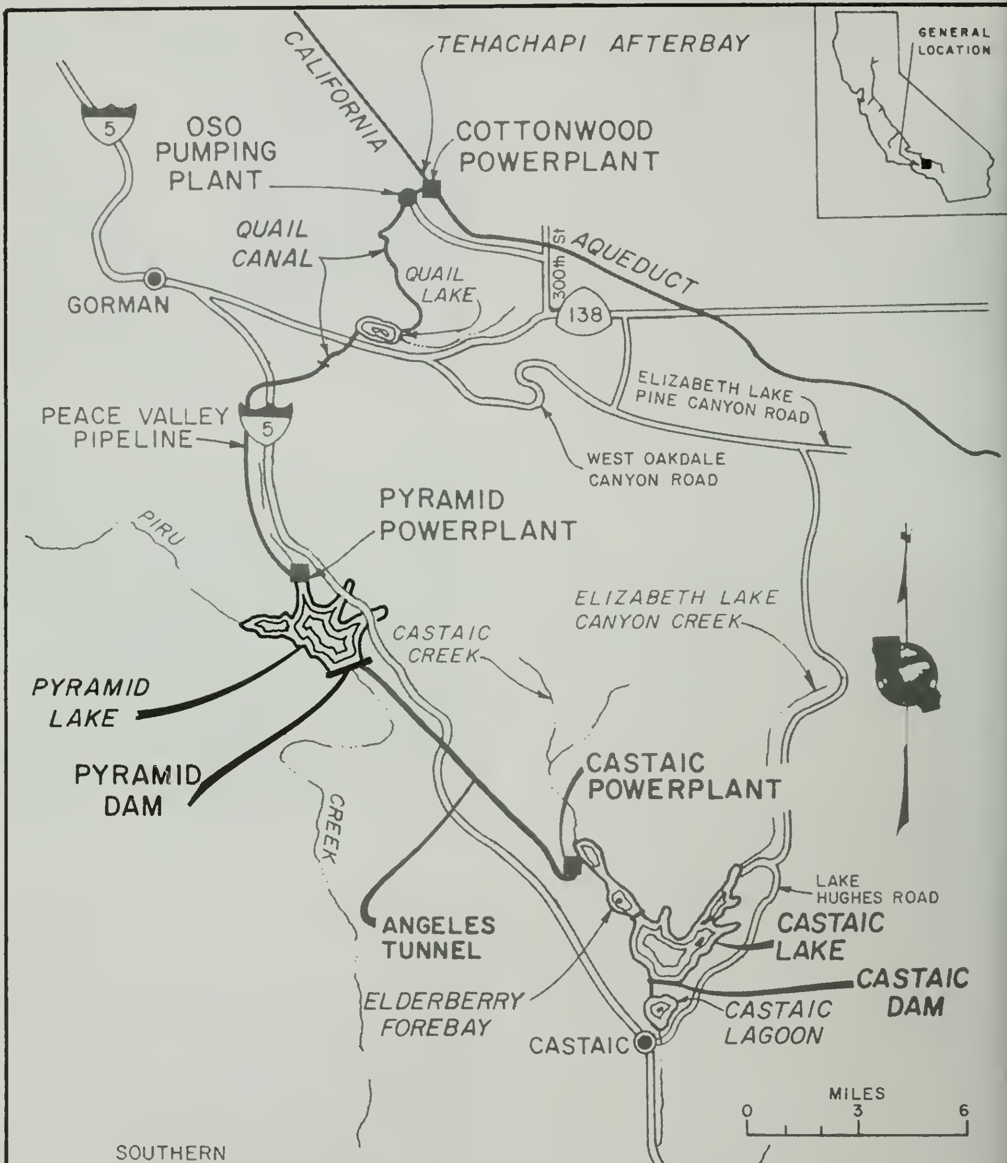

Figure 300. Location Map—Pyramid Dam and Lake

CHAPTER XIV. PYRAMID DAM AND LAKE

General

Description and Location

Pyramid Dam is a 6,860,000-cubic-yard earth and rockfill embankment with a height of 400 feet and a crest length of 1,090 feet.

The spillway for Pyramid Lake is located on the right abutment of the Dam in a deep excavation that furnished much of the rockfill for the embankment. The spillway is comprised of two elements: a controlled or gated spillway and an emergency spillway. The controlled spillway consists of an unlined approach channel, gated headworks, and lined chute. The emergency spillway consists of a 365-foot-long, concrete, overpour section with its crest set 1 foot above normal maximum storage level, discharging into an unlined chute.

There are two outlets from the Lake. The 30-foot-diameter 37,775-foot-long Angeles Tunnel starts in the left abutment just upstream of the toe of the Dam. This tunnel conveys generating flows to, and pumping flows from, Castaic Powerplant. Tunnel features at the Dam are the submerged intake and the emergency closure gate located in a vertical shaft excavated in the abutment 600 feet upstream of the dam crest. The capacities of the Tunnel are 18,400 cubic feet per second (cfs) generating and 17,300 cfs pumping. The other outlet is the stream release facility which consists of valving in the former diversion tunnel. The capacity of this outlet, with water surface elevation 2,500 feet, is 1,000 cfs.

Figure 301. Aerial View—Pyramid Dam and Lake

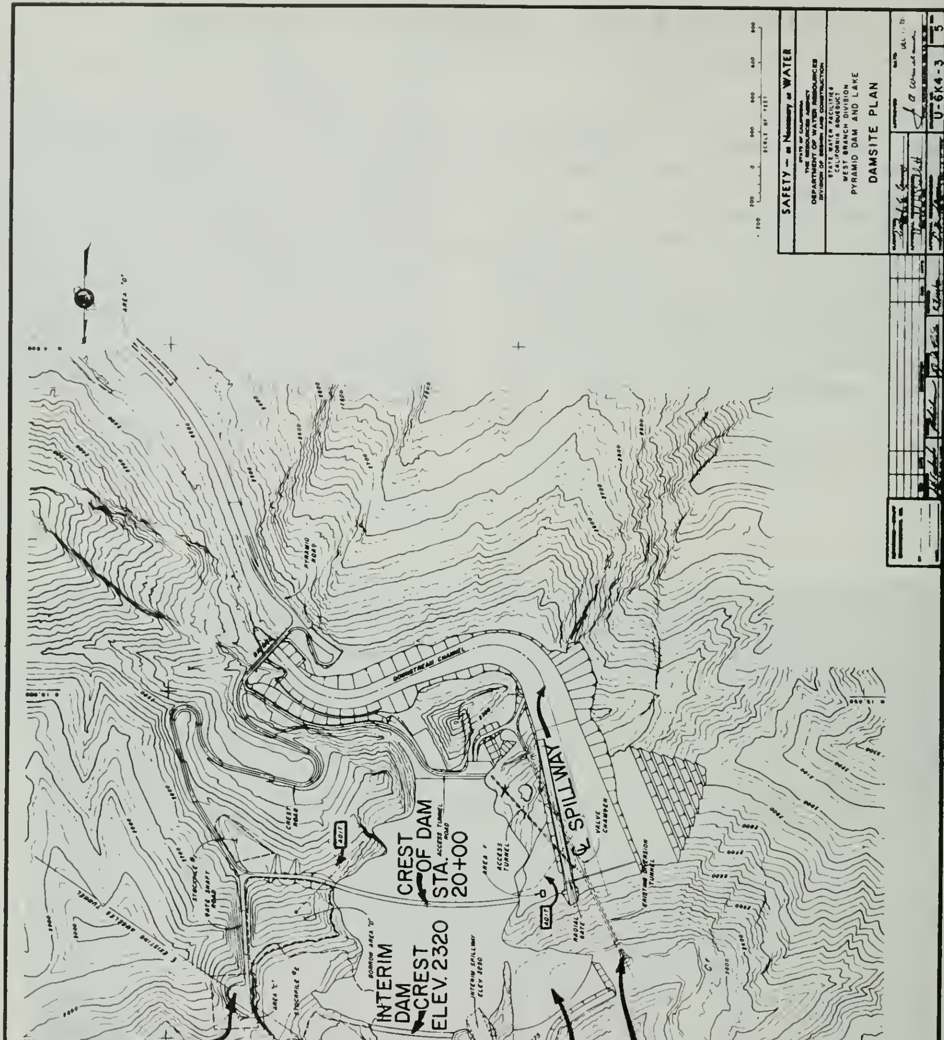

Figure 302. Dam Site Plan

The Lake inundates the lower part of three embankments on Interstate Highway 5. Culverts under the embankments had to be reinforced and extended, and ballast fills had to be constructed to counteract the effects of saturation caused by the Lake. The required excavations and embankments were designed so they could accommodate recreation developments.

A statistical summary of Pyramid Dam and Lake is shown in Table 39. Figure 300 is the location map, Figure 301 an aerial view of the Dam, and Figure 302 the Dam site plan. The area-capacity curves are shown on Figure 303.

Pyramid Dam and Lake are located on Piru Creek near Pyramid Rock in a narrow gorge which was traversed by abandoned U. S. Highway 99, approximately 14 miles north of the town of Castaic (Figure 300). The nearest major highway is Interstate 5, which crosses the two easterly arms of the Lake.

Purpose

Pyramid Dam and Lake comprise an essential feature of the West Branch of the California Aqueduct. They provide (1) en route regulatory storage for Castaic (pumped-storage) Powerplant, (2) an afterbay for Pyramid Powerplant to be operational about 1982, (3) regulatory storage for a possible future pumped-storage plant in the Piru Creek arm, (4) emergency storage for water deliveries from the West Branch, (5) recreational opportunities, and (6) incidental flood protection.

Chronology and Alternative Dam Considerations

Studies for routing project water via Piru Creek were started by the Department of Water Resources in 1953. In December 1959, Department Bulletin No. 79 recommended that a reservoir, located approximately at the Pyramid site, be sized to impound 56,000 acre-feet. In 1960, the Department signed a contract for the delivery of project water from the West Branch to three contracting agencies. Two years later, it was determined that the West Branch should have delivery and storage capacities as large as economically feasible. Subsequent studies in 1964–65 concluded that the

TABLE 39. Statistical Summary of Pyramid Dam and Lake

PYRAMID DAM	
Type: Zoned earth and rockfill	
Crest elevation	2,606 feet
Crest width	35 feet
Crest length	1,090 feet
Streambed elevation at dam axis	2,224 feet
Lowest foundation elevation	2,206 feet
Structural height above foundation	400 feet
Embankment volume	6,860,000 cubic yards
Freeboard above spillway crest	27 feet
Freeboard, maximum operating surface	28 feet
Freeboard, maximum probable flood	4 feet
PYRAMID LAKE	
Storage at spillway crest elevation	171,196 acre-feet
Maximum operating storage	169,902 acre-feet
Minimum operating storage	4,798 acre-feet
Dead pool storage	4,798 acre-feet
Maximum operating surface elevation	2,578 feet
Minimum operating surface elevation	2,340 feet
Dead pool surface elevation	2,340 feet
Shoreline, spillway crest elevation	21 miles
Surface area, spillway crest elevation	1,297 acres
Surface area, maximum operating elevation	1,291 acres
Surface area, minimum operating elevation	176 acres

SPILLWAY	
Emergency: Ungated ogee crest with unlined channel	
Crest elevation	2,579 feet
Crest length	375 feet
Flood control: Gated broad crest with lined channel and flip bucket—one radial gate 40 feet wide by 31 feet high	
Top elevation of gate	2,579 feet
Sill elevation	2,548 feet
Sill width	40 feet
Combined spillways: Gate open	
Maximum probable flood inflow	180,000 cubic feet per second
Peak routed outflow	150,000 cubic feet per second
Maximum surface elevation	2,602 feet
Standard project flood inflow	69,000 cubic feet per second
Peak routed outflow	52,300 cubic feet per second
Maximum surface elevation	2,596.7 feet
INLET	
Improved Gorman Creek (interim)	
Design flow	850 cubic feet per second
Pyramid Powerplant (future)	
Maximum generating flow	3,128 cubic feet per second
INLET-OUTLET	
Angeles Tunnel: A 30-foot-diameter lined tunnel to Castaic Powerplant—uncontrolled inlet tower—upstream shutoff, 25-foot by 32-foot coaster gate	
Maximum generating release	18,400 cubic feet per second
Pumping capacity	17,300 cubic feet per second
OUTLET WORKS	
Stream release: 15-foot-diameter lined tunnel under the right abutment with valve chamber at midpoint—intake, uncontrolled tower with steel plug emergency bulkhead—control, series of rated valves in conduits through tunnel plug—discharge from fixed-cone dispersion valves into tunnel downstream of valve chamber	
Capacity, stream maintenance	1,000 cubic feet per second
Capacity, reservoir drainage	3,200 cubic feet per second

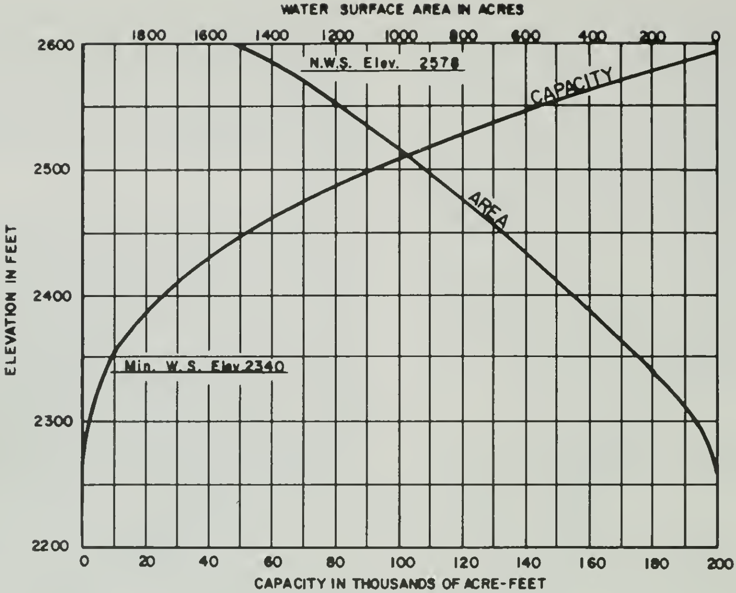

Figure 303. Area-Capacity Curves

maximum, feasible, operating, water elevation at the Pyramid site was 2,578 feet.

During the preliminary design period, various types of dams were investigated for this site. These included concrete gravity, concrete thin arch, concrete arch gravity, and earth and rockfill. The concrete thin arch had the lowest cost; however, it was rejected because the thin right abutment ridge appeared inadequate to support a structure of the height being considered, and spillage during floods would cause adverse vibrations of the foundation rock. The earth-rockfill section was chosen over the remaining alternatives because of lower cost and better resistance to damage due to earthquake-associated movements.

Construction started in the fall of 1969. Included in the initial contract were: the diversion tunnel, access road and bridge, and site development excavation for Angeles Tunnel intake. This work was identified as essential in a construction sequence that would allow initial project deliveries to pass down the West Branch on schedule. The remaining contracts for the Dam and other works were deferred for several months pending resolution of a temporary cash-flow problem existing at the time.

A subsequent contract for Angeles Tunnel intake civil work was awarded in September 1970, and the contract for the Dam and spillway was awarded in June 1971. The latter contract included provisions for an interim dam to be constructed to divert project flows into Angeles Tunnel in October 1971. The interim dam was later incorporated into the upstream toe of Pyramid Dam.

The dam embankment was topped out in March 1973 and storage commenced that spring. The reservoir filled a year later.

Regional Geology and Seismicity

The Dam site is at the southern edge of the Ridge Basin. The north boundary of the Ridge Basin is formed by the San Andreas fault, the southwest and southeast boundaries are formed by the San Gabriel fault and by nonmarine Pliocene sediments, and the northeast and east boundaries are formed by granitic rocks and by a metamorphic complex. Prominent faults in and near the Ridge Basin are the Liebre and Clearwater faults in the northern portion of the Basin and the Bee Canyon and San Francisquito faults to the east.

In the general region, which includes Pyramid Dam and Lake, several major earthquakes have been recorded. The 1857 Tejon earthquake was estimated to be among the largest in California history. It is thought to have had its epicenter on the San Andreas fault in the Carrizo Plain, approximately 50 miles northwest of the Dam site. The 1952 Kern County earthquake had its epicenter on the White Wolf fault, about 25 miles north of the site. Estimates are that it had a Richter scale intensity of 7 in the dam area. Between

1915 and 1965, approximately four other earthquakes occurred which could have produced an estimated intensity of 6 or greater at the Dam site.

Design

Dam

Description. The zoned earth-rockfill embankment (Figures 304 and 305) consists of a central clay core with flanking shells of quarried rock. Transition drain zones are included between the core and shell. A blanket drain is extended from the transition drain zones to the toe of the Dam to complete the downstream drain system. Random zones were designated in the downstream shell and in isolated parts of the upstream shell for quarried rock of lower permeability than the remainder of the shell. Thin zones were included on the faces of the Dam for dumping the uncompactable plus 30-inch rock. Very durable riprap available from processing the streambed deposit for transition drain zones was specified for use within the anticipated range of normal reservoir fluctuation.

A 90-foot-high interim dam (Figures 306 and 307) was located at the upstream toe of Pyramid Dam, the bulk of its mass being included in the upstream shell. It was a central core structure with a single transition zone located downstream of the core, a downstream shell constructed to Pyramid Dam shell zone specifications, and an upstream shell of semipervious rockfill.

Extensive waste areas were designated at the upstream toe up to elevation 2,400 feet, and a buttress of random fill and select spoil was placed on the upstream right abutment.

A single-line grout curtain with a maximum depth of 150 feet was injected into the foundation under the core. Abutment drainage is provided by two former exploration adits.

Foundation. The deeply incised gorge through the Dam site contained Piru Creek and old U.S. Highway 99. The Highway was abandoned in the summer of 1970 upon completion of Interstate 5 adjacent to the area. The overall width occupied by the stream channel and highway at the Dam site ranged from 200 to 350 feet.

The abutments are narrow, the right abutment being just thick enough to contain the Dam with the help of the buttress. Natural abutment slopes along the axis of the Dam generally were from 1:1 to 2:1 with some steep cliffs and overhangs and a major highway cut at 0.4:1. These slopes were flattened by controlled blasting to 1:1 under the core and transitions and 1/4:1 in other areas.

Sands and gravels deposited in the stream channel ranged up to 25 feet thick, while fills associated with Highway 99 ranged up to 50 feet thick. Channel and fill materials were excavated to provide a bedrock foundation under the embankment.

The rock consists of thin beds of argillite. The beds strike nearly parallel to the dam axis and dip favorably upstream at 40 to 50 degrees. Pyramid argillite is intermediate in hardness between shale and slate and is a unique unit within the Ridge Basin group, being considerably more competent and resistant than the remainder of the group. It has a stratigraphic thickness of approximately 1,800 feet at the Dam and grades on three sides into softer, characteristic, ridge basin sediments. On the fourth side, the argillite grades into conglomerate sandstone and sedimentary breccia that is adjacent to the San Gabriel fault zone.

Construction Materials. The impervious core of Pyramid Dam is clayey shale material found in the reservoir area 1 mile upstream from the Dam site. This material is divided into Zones 1A and 1B. Zone 1A was taken from the lower slopes of the borrow area and consists of slopewash material that washed down from the in-place shales above. Slopewash material was specified for Zone 1A because it was believed that the weathered in-place shales used in Zone 1B might not break down fine enough for the tight core desired.

The bulk of the material for the filter and blanket drain zones was processed from the Piru Creek sand and gravel deposits in the reservoir area. Zone 2A (minus ¾-inch material) flanks both sides of the core, acting as the fine half of the transition between the core and the shells. Zone 2B (¾- to 6-inch material) was the coarse half of the downstream transition and also acts as a chimney and blanket drain system for the downstream shell. An intermediate transition Zone 2D (minus 6-inch material) was added during construction after segregation occurred in Zone 2B. Fine Zone 3B replaced Zone 2B upstream at that time because Zone 2D required the stream gravels. Zone 2C (minus 3-inch streambed material) is the transition in the interim dam. Zone 3A is the rockfill shell of fresh to slightly weathered rock. It is compacted except on the outer slopes where oversized rock was dumped. Zone 3B is finer material than 3A and was used both upstream and downstream in the random zones and as noted above in the upstream transition between Zones 2A and 3A.

The primary design problem was to determine if available sources could produce suitable rockfill materials. This problem at Pyramid Dam was more complex than usual because there was some question that the excavated argillite would retain its rockfill characteristics through chemical changes or other breakdown processes after it was placed in the embankment. Test quarries, test fills, and special tests were used to make the determination.

Two rocks were quarried: argillite that would be representative of the spillway and other required excavations, and sandstone found 1,000 feet downstream of the Dam.

After careful examination of both the sandstone and argillite quarry materials, and after considering the economies of the various material sources in the re-

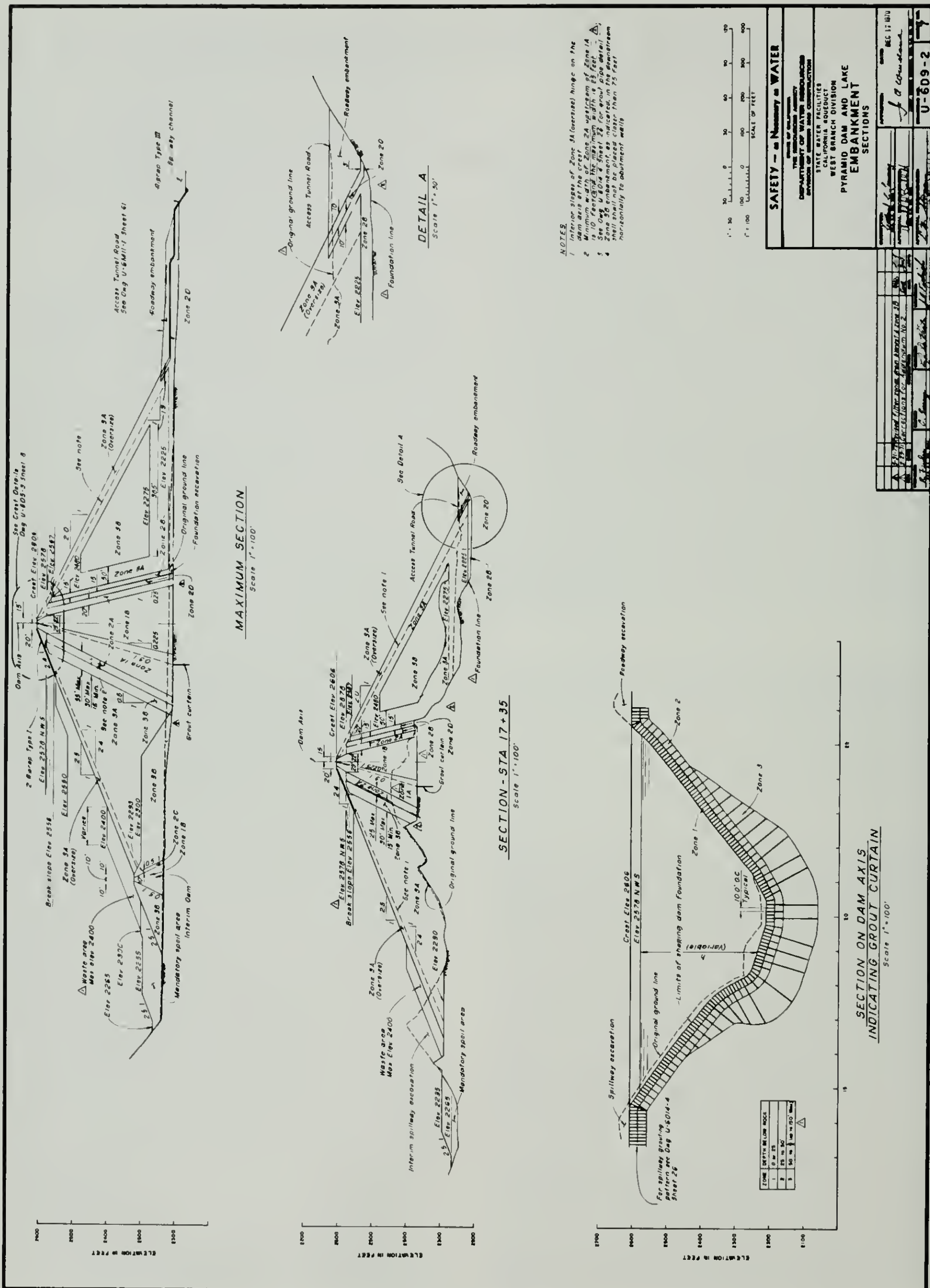

Figure 304. Embankment Sections

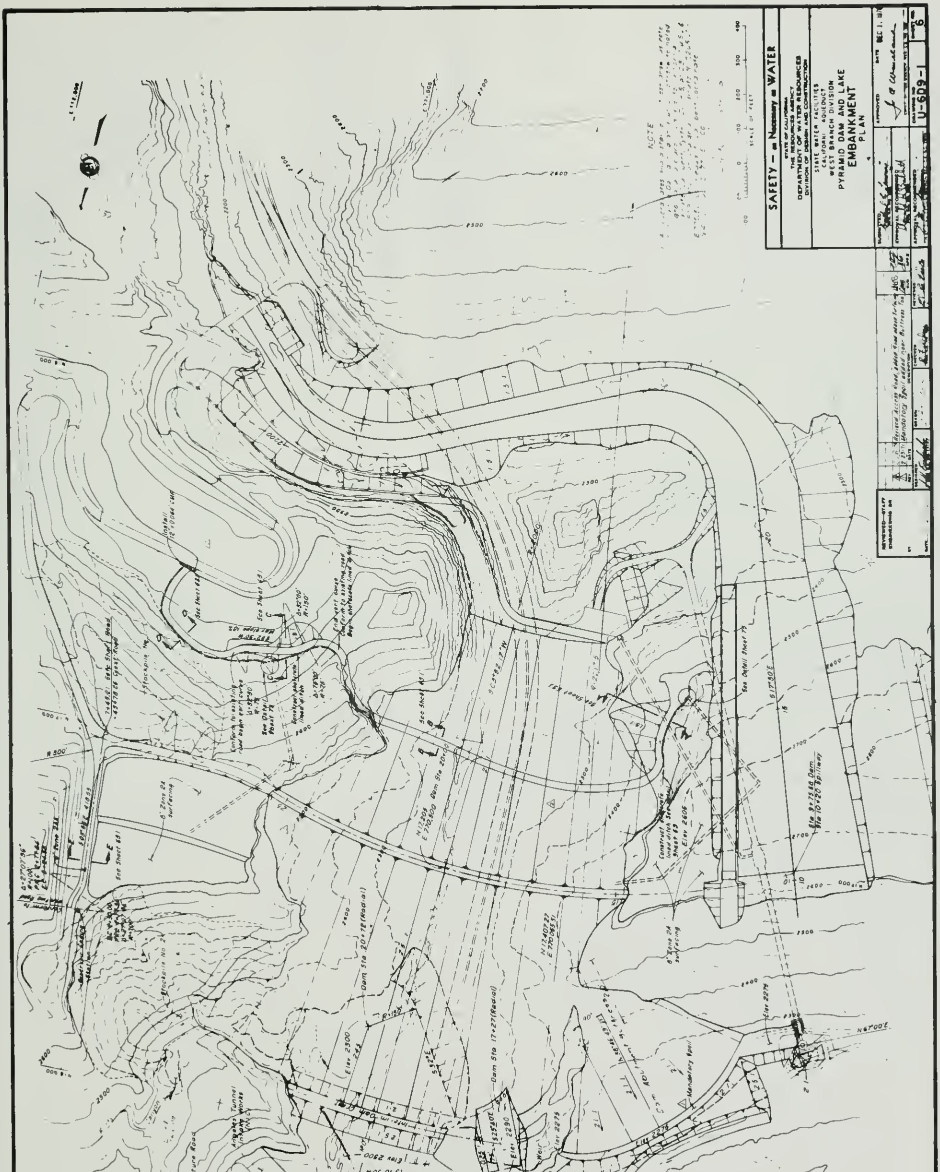

Figure 305. Embankment Plan

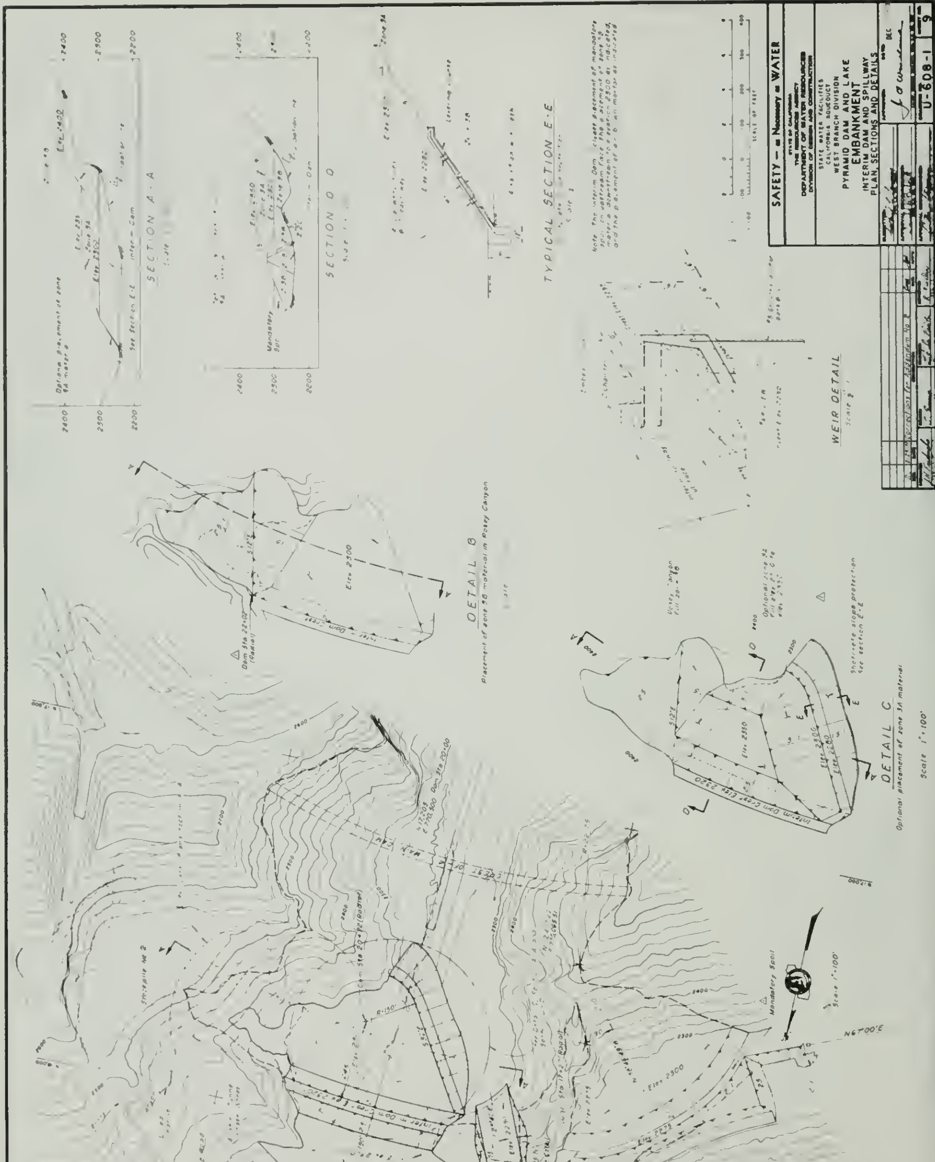

Figure 306. Interim Dam—Plan, Sections, and Details

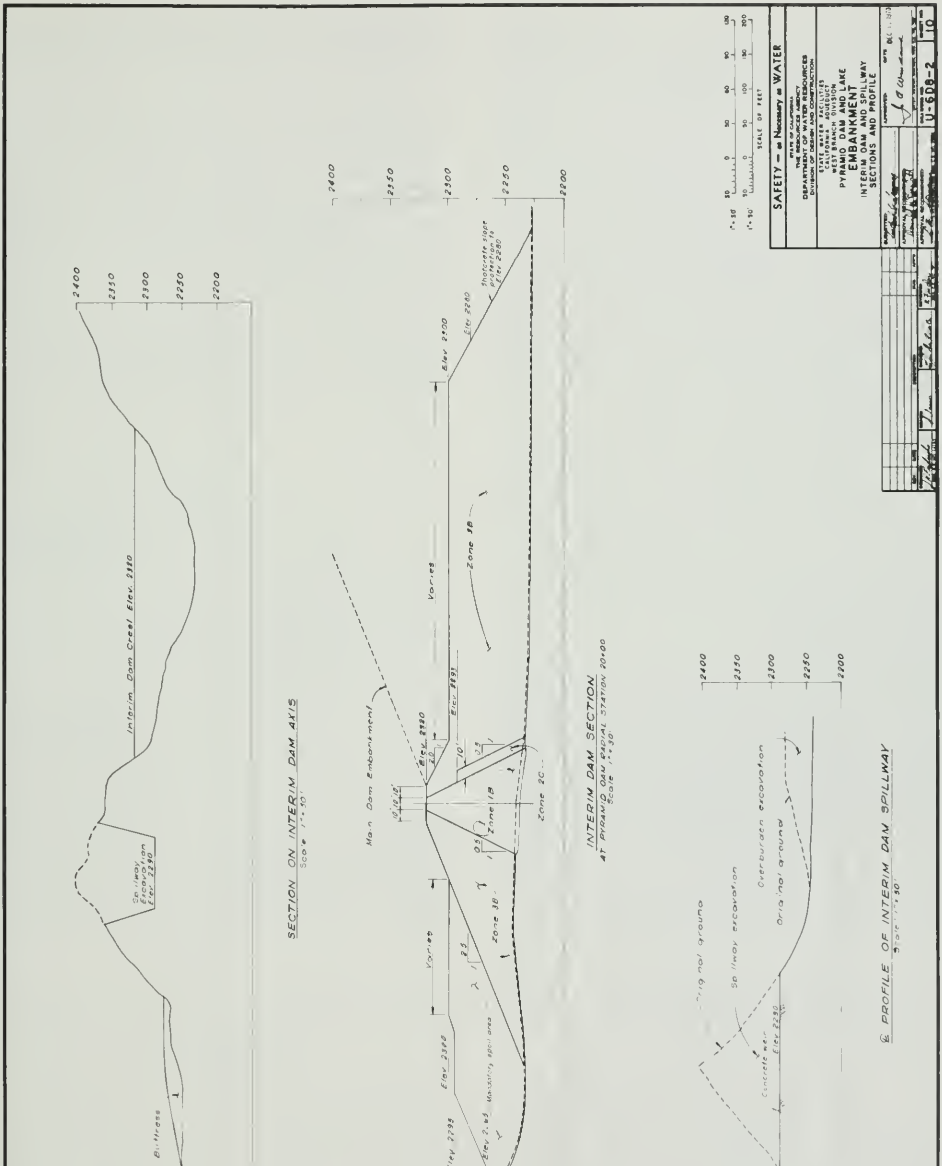

Figure 307. Interim Dam—Sections and Profile

quired excavations, the sandstone was eliminated from further consideration as a borrow source for the following reasons: (1) much of the material is poorly cemented and would easily break down to sand-size particles upon rolling; (2) separation of the softer rock from the harder portion would have been difficult; (3) if the material did become fresher and harder with depth, an excessive amount of material would be wasted in reaching the better material underneath; and (4) large amounts of argillite material had to be removed from mandatory excavations such as the spillway and core shaping site development, and these materials were considered suitable for rockfill. Therefore, the test fills were confined to the argillite. They were 90 feet by 150 feet in plan. Various lift thicknesses were compacted by 5- and 10-ton vibrating rollers. The rock showed favorable characteristics. Design properties and specification requirements for placement and compaction were obtained.

Special tests used to resolve the question of whether the rock would break down in the dam embankment were: (1) standard solubility and wetting-drying tests, (2) a petrographic analysis, and (3) excavation into a 20-year-old highway fill. The first two tests provided technical reasons why the rock exposed in the third test was found to be sound after the surface material was removed. It was concluded that the surface of the rock weathers because it is free to relax and crack. Some weathering is therefore expected on the downstream face of the Dam.

Stability Analysis. Embankment stability was analyzed by the infinite slope, sliding wedge, and Swedish Slip Circle methods of analysis. The rock (argillite) foundation is significantly stronger than the embankment materials. Consequently, the rock surface is treated as a limiting boundary for failure planes after a few trial analyses. Material properties used in the analyses are given in Table 40. These values were based on the results of a detailed exploration program, the test quarry and fill previously discussed, and an extensive soil- and rock-testing program.

Core materials were explored and laboratory-tested by conventional methods. Special attention was given to the gradation tests to assure that nonrepresentative breakdown of the residual soil was avoided in the testing process. Gentle hand-sieving was required.

Sand and gravel explorations and testing were routine except for handling of shales that were mixed naturally into the alluvial deposit. The exploration program did not disclose the true quantity of shales present. They broke down during processing of the material during construction and reduced the permeability of the fine transition zones, thus requiring additional tests and reevaluation of the embankment design during construction. The lower permeabilities were determined to be acceptable. Shell materials from the quarry were tested in large samples at the Richmond Laboratory discussed under Chapter V of this volume. These tests provided the design parameters and, as part of the testing program, proved that much smaller samples would have provided acceptable results.

Seismic forces for the stability analyses were approximated by a steady horizontal force acting in the direction of instability. The force was taken as 0.15w, where "w" is the moist or saturated embankment weight, whichever was applicable. The procedures for applying the seismic forces to the Swedish Slip Circle analysis, given in the U. S. Army Corps of Engineers' design manuals, were modified to expedite a computer solution, and forces were applied at the center of mass for each infinitesimal slice rather than at the base.

This conventional earthquake analysis was considered at the time of the design to be an approximate and somewhat empirical approach. Consideration was given to using the then-developing, two-dimensional, finite element analysis which had been tried on dams with crest lengths several times longer than their heights. However, the shortness of the crest and shape of the canyon made this analysis inapplicable to Pyramid Dam. The 0.15g factor provided conservative slopes, and the following features were included in the embankment to emphasize earthquake-damage-resistant considerations: providing a plastic core thicker

TABLE 40. Material Design Parameters—Pyramid Dam

Material	Specific Gravity	Unit Weight in Pounds Per Cubic Foot			Static Shear Strengths θ Angles in Degrees Cohesion in Tons Per Square Foot					
					Effective		Total		Construction	
		Dry	Moist	Saturated	θ	C	θ	C	θ	C
*Rockfill	2.54	113	115	135	40	--	40	--	40	--
Core	2.75	104	124	129	26	--	14	0.2	12	1.2
Foundation	2.54	--	165	165	60	1.0	60	1.0	60	1.0

* The same parameters were used in transition and drain zones.

than would otherwise be necessary, providing additional freeboard, and compacting the rockfill as much as possible without adversely affecting the permeability.

Settlement. A crest camber of approximately one-half of 1% of the fill height was provided to compensate for long-term embankment settlement. The adequacy of the assigned camber was checked with data, which were analyzed from one-dimensional consolidation tests using samples initially compacted to 95% of relative compaction. These samples were consolidated under incremental loads without the addition of moisture until maximum test loads were reached. Then, they were immersed in water under maximum test load to induce further consolidation. Various maximum test loadings were used to simulate various embankment heights. These procedures verified that the allowance for camber was reasonable.

Instrumentation. Following is a summary of structural performance instrumentation initially installed in the Dam:

Type of Instrument	*Number Installed*	*Data Obtained*
Piezometers (Hydraulic)	21 *	Pore Pressure
Slope-Indicator Installations	3	Internal Deflection Internal Settlement Water Surface Elevation
Embankment Monuments	5	Surface Settlement Surface Deflection
Crest Monuments	13	Surface Settlement Surface Deflection
Drainage, Right Adit		Flow Rate
Drainage, Left Adit		Flow Rate
Seepage, Access Tunnel		Flow Rate
Drainage, Dam Toe		Flow Rate

* Four in the foundation and 17 in the embankment

Drainage Adits

Two 9-foot by 7-foot former exploration adits have been retained and improved to provide drainage of the abutments. One 900-foot adit is in the right abutment with portal at elevation 2,450 feet and the other 650-foot-long adit is in the left abutment with portal at elevation 2,450 feet (Figure 308). Structural steel support was used in portals and areas of weak rock, while the remaining portions of the tunnels were either rock-bolted or left unsupported. (Construction of the adits was accomplished by a lump-sum bid contract.) The structural-steel-supported sections were lined with reinforced concrete and contact-grouted while the remaining sections were lined with shotcrete. Drain holes initially were drilled in all reinforced-concrete-lined reaches farther than 100 feet from the portal to protect the concrete. Later, drain holes were drilled in shotcreted sections to increase the area of influence of the abutment drainage system. Rings of four holes, 25 feet in length, were drilled at 20-foot intervals.

The concrete-lined section is of uniform dimensions and was designed to resist the worst of the following loading cases:

1. A rock load of one bore diameter in height.
2. A rock load of one-half bore diameter in height plus a hydrostatic head of 25% of the overburden depth.
3. A uniform grouting pressure of 30 pounds per square inch.

Dead load is included in all the above loading cases.

Diversion Tunnel

The diversion facility is a 15-foot-diameter concrete-lined tunnel approximately 1,350 feet long through the right abutment (Figure 309). This tunnel was used to divert streamflow during construction of the Dam and now is used to carry controlled downstream releases.

During the flood season of 1972–73, the embankment was required to be above elevation 2,445 feet. The tunnel was sized to pass the standard project flood with maximum water surface elevation at 2,441 feet. The inflow of 69,000 cfs would have been reduced to an outflow of 11,300 cfs. Space requirements for future stream release facilities also were considered in sizing the tunnel.

For design, the tunnel was divided into a reach upstream and a reach downstream of the grout curtain. The loading on the concrete lining for the upstream portion is due to an external hydrostatic head resulting from the normal reservoir water surface at elevation 2,578 feet with tunnel dewatered and dead load. Loading on the downstream reach is dead load plus external hydrostatic head to ground surface or 25% of reservoir head, whichever is greater.

The concrete lining also was checked for internal pressures to the hydraulic gradeline for maximum diversion discharge. The internal load was distributed between hoop reinforcement and the surrounding rock by equating the deformations.

Intake Tower

The intake to the former diversion tunnel is a 119-foot-high reinforced-concrete tower (Figure 310) which has an internal diameter of 15 feet. The 119-foot height includes an 18-foot-high trashrack. The base of the tower contains a 30-foot-radius elbow which connects to a 40-foot-long, cut-and-cover, conduit connection to the tunnel. The elbow and conduit also are 15 feet in diameter. The tower rests on a 10.25-foot-thick by 70-foot, horizontal, hexagonal, slab base. The tower and conduit are founded on fresh bedrock.

There were two openings in the tower. The lip of the upper opening is at elevation 2,340 feet which provides 100 years of silt storage below this intake. The reinforced-concrete trashrack frame which extends above this opening was included in the initial tower construction. The other 15-foot-diameter opening was at the base of the elbow at streambed elevation.

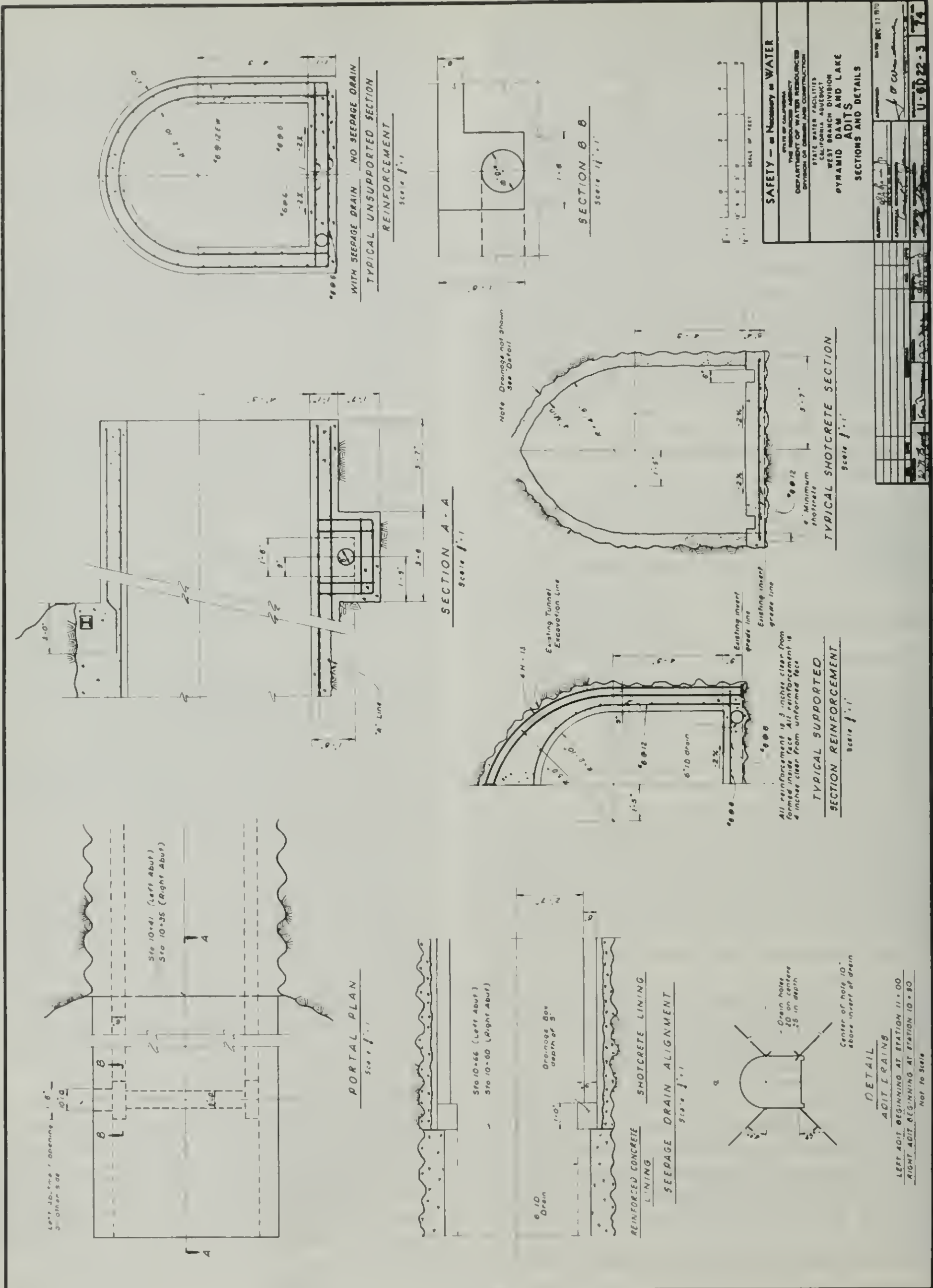

Figure 308. Exploration Adits—Section and Details

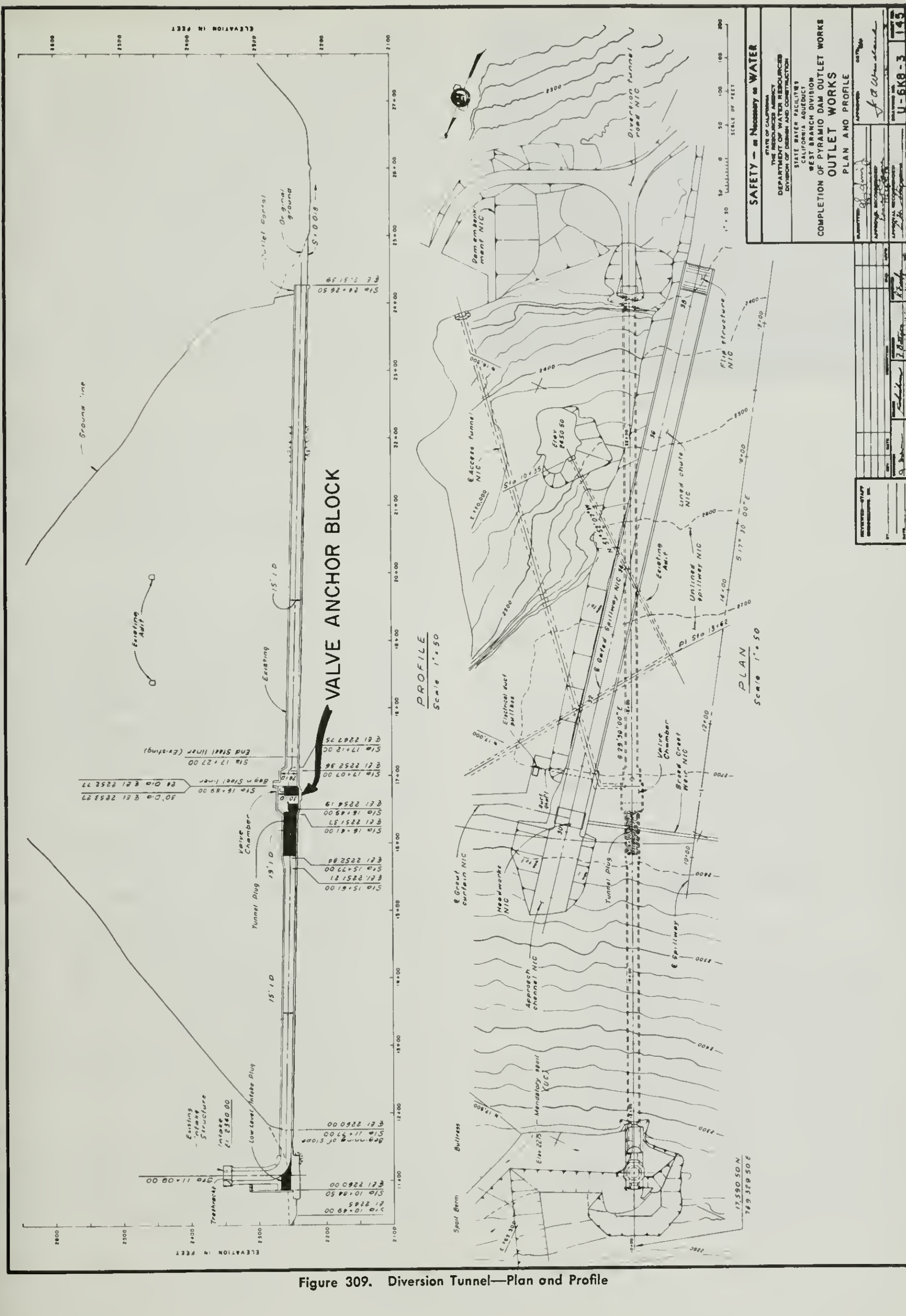

Figure 309. Diversion Tunnel—Plan and Profile

Figure 310. Intake Tower

The low-level intake provided for initial diversion flows. It was blocked when the interim dam was completed and later plugged with concrete except for two 30-inch-diameter and one 10-inch-diameter temporary conduits. This allowed water to pond behind the Dam to elevation 2,293 feet. Stream releases during construction were controlled through the conduits by sluice gates (30-inch) and a butterfly valve (10-inch). Floodflows passed over the interim spillway the first winter of dam construction and through the upper tower opening the second.

After the second winter, the trashracks were added and the temporary conduits were plugged. The trashrack on the top of the structure can be removed so a bulkhead can be placed on the intake opening to dewater the upstream part of the tunnel. Such a bulkhead was used to provide a dry working area during construction of the stream release facility. The bulkhead also can be used on the Castaic Dam outlet works low-level intake.

The intake tower is considered a free-standing structure and was analyzed for the following loading cases:

1. Tower submerged and discharging 1,000 cfs (standard operation).
2. Tower submerged and discharging 11,300 cfs (diversion condition).
3. Reservoir at normal water elevation 2,578 feet, with seismic load.
4. Tower bulkhead closed and empty with external hydrostatic head to elevation 2,578 feet.

Normal working stresses were applied to Cases 1 and 4. Stresses resulting from Case 2 and 3 loadings were allowed one-third overstress.

The trashracks were sized to limit the velocities to 2 feet per second during a discharge of 1,000 cfs, and the trash bars were designed to fail at stresses resulting from a differential head of 20 feet through the trashrack.

Stream Release Facility

The stream release facility at Pyramid Dam utilizes the former diversion tunnel as a conduit and releases all natural reservoir inflows on an equal-rate basis to a maximum of 1,000 cfs. Inflows above 1,000 cfs are stored temporarily until reservoir inflows drop below 1,000 cfs. If temporary storage capacity is not sufficient, inflows will be routed through the spillway.

A valve chamber (Figure 311) is located in the diversion tunnel near its midpoint and contains five steel conduits: one emergency, 78-inch with a butterfly valve; two 42-inch; one 24-inch; and one 12-inch. Each of the last four conduits is equipped with a fixed-cone dispersion valve and a shutoff valve. Access to the chamber is gained through a tunnel from the downstream portal area.

Pressure flow is maintained in the tunnel from the intake to a tunnel plug at the upstream end of the chamber and in the conduits, which extend through the plug to the valves. The flow from the valve system discharges into a steel-lined energy-dissipating chamber. From the dissipating chamber, the water flows as open-channel flow in the tunnel to the downstream portal. The tunnel is sloped to maintain supercritical flow.

As a safety measure for the Dam, it is possible to draw down the reservoir 100 feet below the normal water surface in approximately 12 days. This can be accomplished by opening the 78-inch butterfly valve in the emergency conduit and opening the 40-foot-wide by 31-foot-high, spillway, radial gate. This results in a maximum discharge of approximately 3,200 cfs through the butterfly valve, which would operate during the entire drawdown period, and 20,000 cfs through the spillway gate, which operates only during the first 30 feet of drawdown.

The shutoff valves and the 78-inch butterfly valve are capable of being opened and closed with the reservoir at elevation 2,578 feet.

The series of rated fixed-cone dispersion valves is capable of accurately releasing discharges from near zero to 1,200 cfs. This system was designed to release a minimum of 1,000 cfs at water surface elevation 2,500 feet. Rating curves for the outlet works are shown on Figure 312.

The tunnel plug was designed to resist hydrostatic loads equivalent to the reservoir at elevation 2,602 feet (maximum water surface) on the full face, and the valve anchor block was designed to resist the thrust on the block from the conduits.

The valve-chamber concrete lining is undrained and was designed to resist the full external hydrostatic pressure of the reservoir. The steel conduits through the valve chamber were designed to resist an internal hydrostatic pressure equivalent to the reservoir at nor-

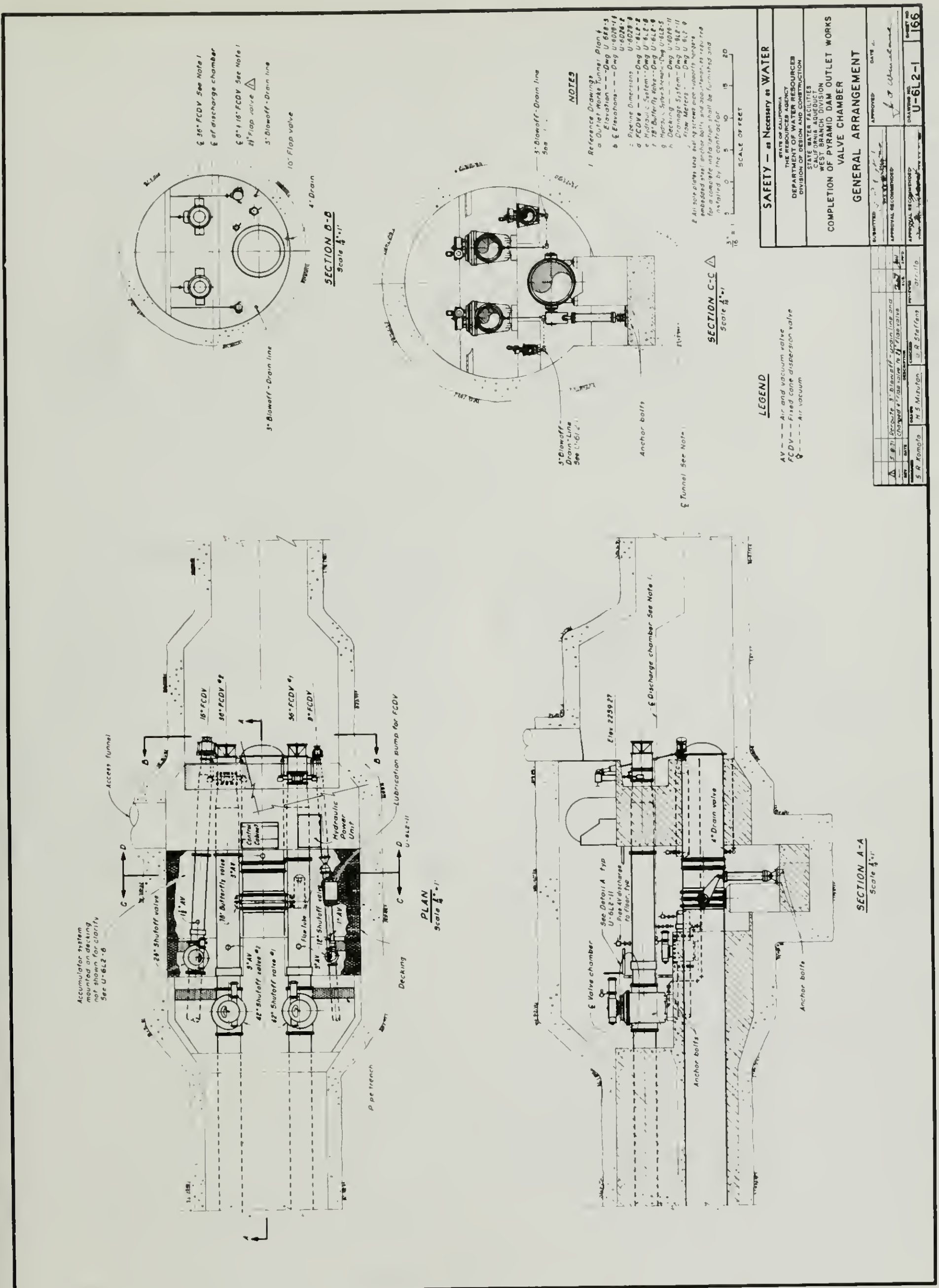

Figure 311. Valve Chamber

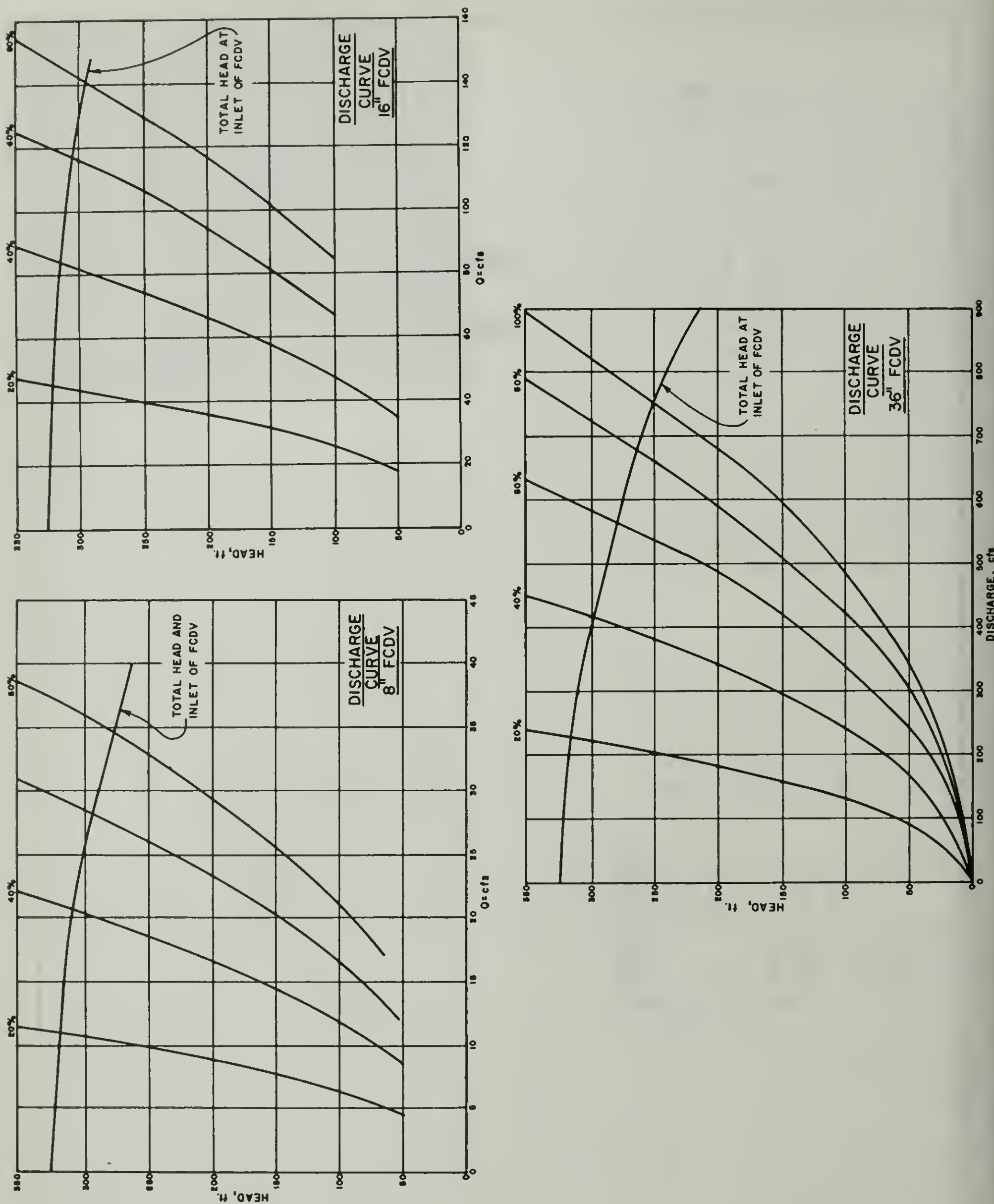

Figure 312. Outlet Works Rating Curves

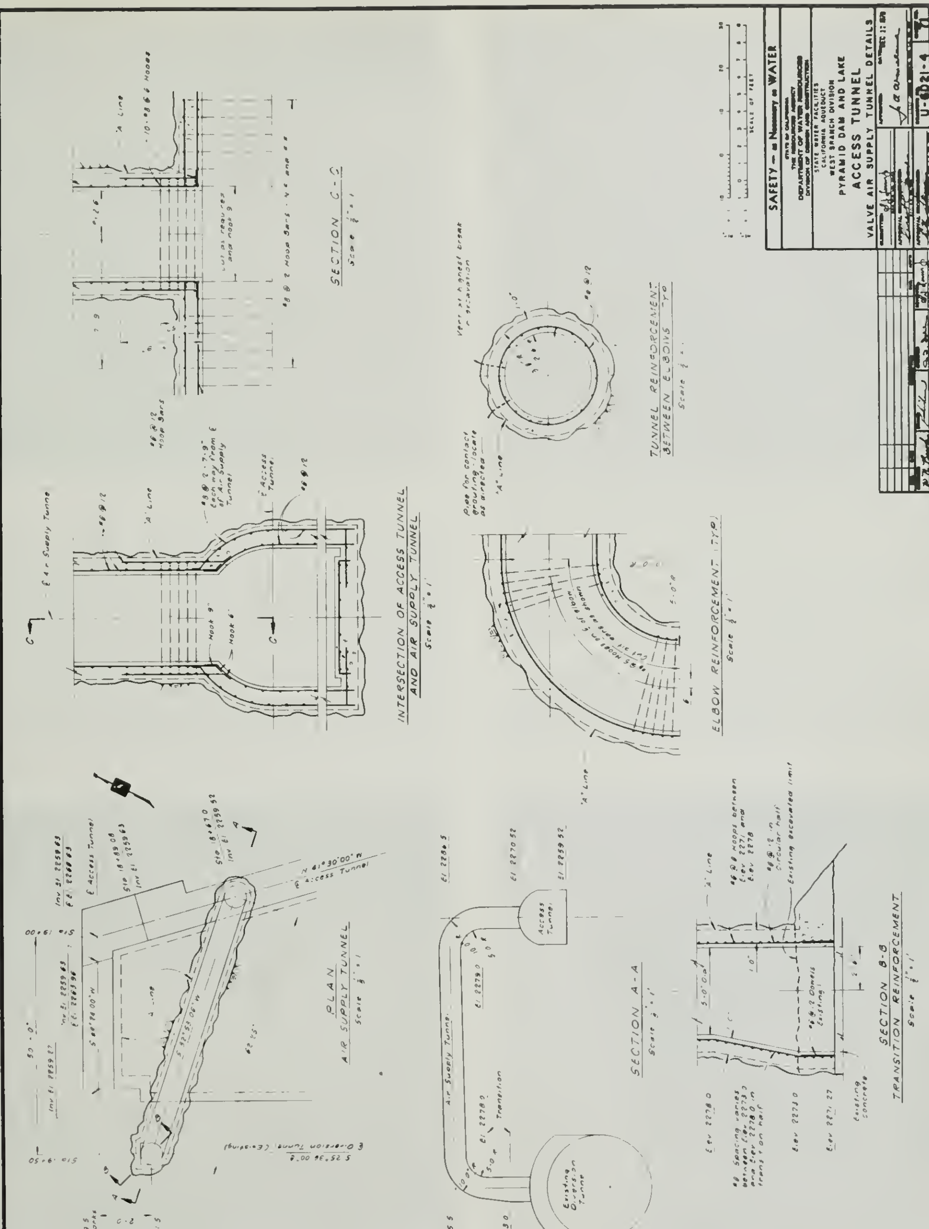

Figure 313. Air Shaft

mal water surface elevation 2,578 feet and a water-hammer load of 50% larger head.

The energy-dissipating chamber was designed to resist external hydrostatic pressure prorated between 100% of full reservoir at the grout curtain at Station 16+20 (Figure 309) and 25% at the drained downstream tunnel reach at Station 17+38. Steel plates anchored to the concrete lining are used to protect the inside surface of the concrete.

The concrete lining of the valve-chamber access tunnel and air-supply tunnel was designed to resist an external hydrostatic head equal to one-half the height of overburden. The finished dimensions of the access tunnel are sufficient to allow passage of foot traffic and small maintenance carts. The air-supply tunnel (Figure 313) was designed to limit air velocities to 150 feet per second (fps) and the access tunnel was designed to limit the air velocity to 37 fps (25 mph), assuming the air demand to be two and one-half times the design valve discharge.

Fixed-Cone Dispersion Valves. Two 36-inch-diameter, one 16-inch-diameter, and one 8-inch-diameter, cylindrical-type, free-discharge, regulating, cone valves (commonly known either as hollow-cone, Howell-Bunger, or fixed-cone dispersion valves) were installed to control stream release and downstream water deliveries. Each valve installation consists of a valve body, valve gate, operating power screws and gear units, interconnecting shafting, electric motor-driven operator, and companion flange.

A centralized lubrication system is provided to lubricate the power screws on all valves. The system provides a forced continuous supply of grease to the valve power screws during valve operation. This system has sufficient capacity to lubricate the two 36-inch-diameter valves if operated simultaneously.

The valves may be operated from the motor control center in the access tunnel or a hydraulic cabinet in the valve chamber. The valves also may be controlled from the Pyramid Dam control building and area control center at Castaic. Each valve operator is provided with a handwheel for manual operation.

Design criteria employed was as follows:

1. Valves were designed for a maximum head of 350 feet.
2. All valves were designed to withstand a seismic load of 0.5g.
3. The 8-inch-diameter valve will operate approximately 75% of the time at flow rates from 0.5 to 29 cfs.
4. The 16-inch-diameter valve will operate approximately 20% of the time at flow rates from 21 to 120 cfs.
5. The 36-inch-diameter valves will operate approximately 5% of the time at flow rates from 120 to 600 cfs, each.

Research data obtained from the Oroville river outlet fixed-cone dispersion valves indicated that flow through these valves was not increased when valve openings exceeded 80%. At openings of greater than 80%, the discharge area at the cone exceeded the valve inlet area and excessive vibrations occurred in the valve body. As a result of this investigation, design criteria for the Pyramid valves required that the maximum valve stroke be limited to the 80% value.

In order to reduce cavitation effects at small valve openings, the valve gate seats and downstream edge of

Figure 314. 8-Inch-Diameter Fixed-Cone Dispersion Valve

Figure 315. 36-Inch-Diameter Fixed-Cone Dispersion Valve

Figure 316. 42-Inch-Diameter Shutoff Valve

Figure 317. Shutoff Valves and Butterfly Valve—Accumulator System

the deflector cone of the 8-inch- and 16-inch-diameter valves are overlayed with Stellite since they are in service most of the time. The deflector cone on the 8-inch valve (Figure 314) is fabricated of stainless steel. The valve gate seat and downstream edge of the deflector cone of the 36-inch-diameter valves (Figure 315) are overlayed with stainless steel.

Flow through the valves is measured by means of flowmeters installed upstream of the valves.

Valve tests under flow conditions were conducted after completion of construction. The purpose of the tests was to measure the performance of the valves in order to evaluate original design concepts. Measurements and data taken were as follows:

1. Valve vibration
2. Valve displacement
3. Tangential and circumferential strain in the valve bodies
4. Flow rate through the valves
5. Valve-operator voltage and current measurements

The test results correlated extremely well with the expected data.

Shutoff Valves. Two 42-inch-diameter, one 24-inch-diameter, and one 12-inch-diameter, conical, plug-type, shutoff valves are located in the valve chamber (Figure 316). They serve as guard valves to the two 36-inch-diameter, one 16-inch-diameter, and one 8-inch-diameter, stream-release, fixed-cone dispersion valves. Each plug valve is equipped with an oil hydraulic-cylinder operator and dual seats for both opened and closed valve positions.

The valves were designed for a 15-pound-per-square-inch (psi) operating pressure and to open and close under the following conditions:

Valve Size (inches)	*Shutoff head (feet)*	*Flow rate (cfs)*
42	325	870
24	325	184
12	325	42

Each valve-operator hydraulic cylinder was designed to operate with an oil pressure between 1,500 psi minimum to 2,000 psi maximum under the operating conditions described. However, for normal operation, the valve control system is required only to operate the valves under balanced head conditions. The operating mechanism is designed to be self-locking so that hydraulic cylinder pressure is not required to hold the plug in the open or closed positions.

The valve control system incorporates an accumulator bank of sufficient capacity to operate the shutoff valves in case of power failure (Figure 317).

Each shutoff valve is capable of opening and closing in three minutes under the maximum flow condition; however, for normal operation, the control is designed so that the valve will operate only when its corresponding fixed-cone dispersion valve is closed. The shutoff valves may be operated from the motor control center in the access tunnel or the hydraulic cabinet in the valve chamber. They also can be controlled from the Pyramid Dam control building.

Spillway

The spillway (Figures 318, 319, and 320) consists of a depressed concrete-lined channel controlled by a 40-foot-wide by 31-foot-high radial gate for passing inflows through the Lake and a 365-foot-long overpour weir with crest at 1 foot above the maximum operating water surface elevation of 2,578 feet, with an unlined channel to discharge very large inflows.

Routing the standard project flood through the spillway reduced the inflow of 69,000 cfs to a 52,300-cfs release. This flood is expected to occur only once in 200 years. The maximum probable flood inflow of 180,000 cfs is reduced to a 150,000-cfs release. The gated passage has the capacity of 20,000 cfs to pass a flood of 10-year recurrence interval without overtopping the weir.

Freeboard requirements for the Dam during an occurrence of the maximum probable flood are as follows:

1. With releases through the gated passage equaling inflow, up to the gate capacity, 4 feet above peak stage.
2. With spillway gate and all outlet works inoperable, 2 feet above peak stage.

Grouting. The embankment grout curtain extends across the abutment between the Dam and spillway, under the controlled channel, and 50 feet under the overpour weir. All of the grout holes were 50 feet deep.

Overpour Weir. The overpour weir is a concrete gravity block with rounded upstream and downstream crest edges. This block extends below channel elevation a sufficient depth for stability.

Stability of the block was checked for all discharges up to the maximum probable flood peak with an earthquake acceleration of 0.2g and reservoir water surface at elevation 2,579 feet.

Headworks Structure. The headworks structure (Figure 321) consists of the concrete floor and walls, gate, bridge, and machinery deck which act as a unit and were analyzed as such.

This structure is founded on competent rock, and the sliding factor was not a problem. Therefore, design was based on a shear friction safety factor of 4 and an overturning resultant falling within the middle one-third of the base under normal operating conditions. For seismic loading with the reservoir at flood stage or the bulkhead gate in place, the resultant was required to fall within the middle one-half of the base.

The entire foundation slab has underdrains downstream of the grout curtain. For design, uplift pressure was applied over the entire base of the structure with full head applied at the upstream edge, reducing linearly to zero at the downstream end. The headworks carry the water load and seismic loads transmitted from the gate to the foundation through prestressed rods from the trunnion anchorage. The bridge deck live and dead loads are transmitted axially through the walls. The foundation slab was designed to transmit forces on the base of the wall to the underlying rock for any loading systems on the walls.

Approach Walls. The approach walls to the headworks structure give a smooth transition into the gate opening. These walls were placed against cut slopes. Weep holes were installed in the walls to reduce hydrostatic loading and anchor bars were installed to maintain stability. Wall loading includes hydrostatic head to elevation 2,578 feet. Weep holes were assumed 50% effective, and the horizontal load is carried by the anchor bars.

Radial Gate. One 40-foot-wide by 31-foot-wide gate was installed in the structure. Under regular operating conditions, normal stresses were allowed in the structural design of the gate and trunnion anchorages. Overstresses were allowed for certain conditions, such as seismic loading and flood conditions with the gate closed.

Bulkhead Gate. The bulkhead gate consists of several noninterchangeable leaves whose size was limited by the crane lifting capacity. Slots are provided in the headworks to accept the gate which is used when it becomes necessary to dewater and then service the radial gate. Steel bearing and seal plates are provided in the slots and floor. The design load on the gate consists of hydrostatic head to elevation 2,578 feet.

Concrete Chute. A concrete chute conveys discharge from the headworks structure to the downstream slope of the abutment ridge. Concrete for the walls and invert slab was placed directly against the rock, and grouted anchor bars and drains were used under the entire chute. The chute terminates with a gravity flip bucket.

Earthquake Criteria

Because Pyramid Dam was constructed in an area of high seismic activity, particular attention was paid to the inclusion of seismic loads in design of structures.

In a structural complex, such as a dam and appurtenant structures, arrangement is of prime importance. Situations were avoided where failure of one component would impair the operation or cause the failure of another component. Design loads that were applied to each structure were conservatively scaled as to the importance of that structure and the consequence of its failure.

Design earthquake accelerations were based on the San Andreas design earthquake developed by the Consulting Board for Earthquake Analysis. This design earthquake is applicable to structures on foundations of alluvium; it can be reduced when dealing with foundations of more competent material. A reduction factor was derived assuming 100% of the San Andreas design earthquake for alluvium foundations and 50% for foundations of hard rock. A straight-line variation between design intensity and seismic velocity in the foundation material was assumed.

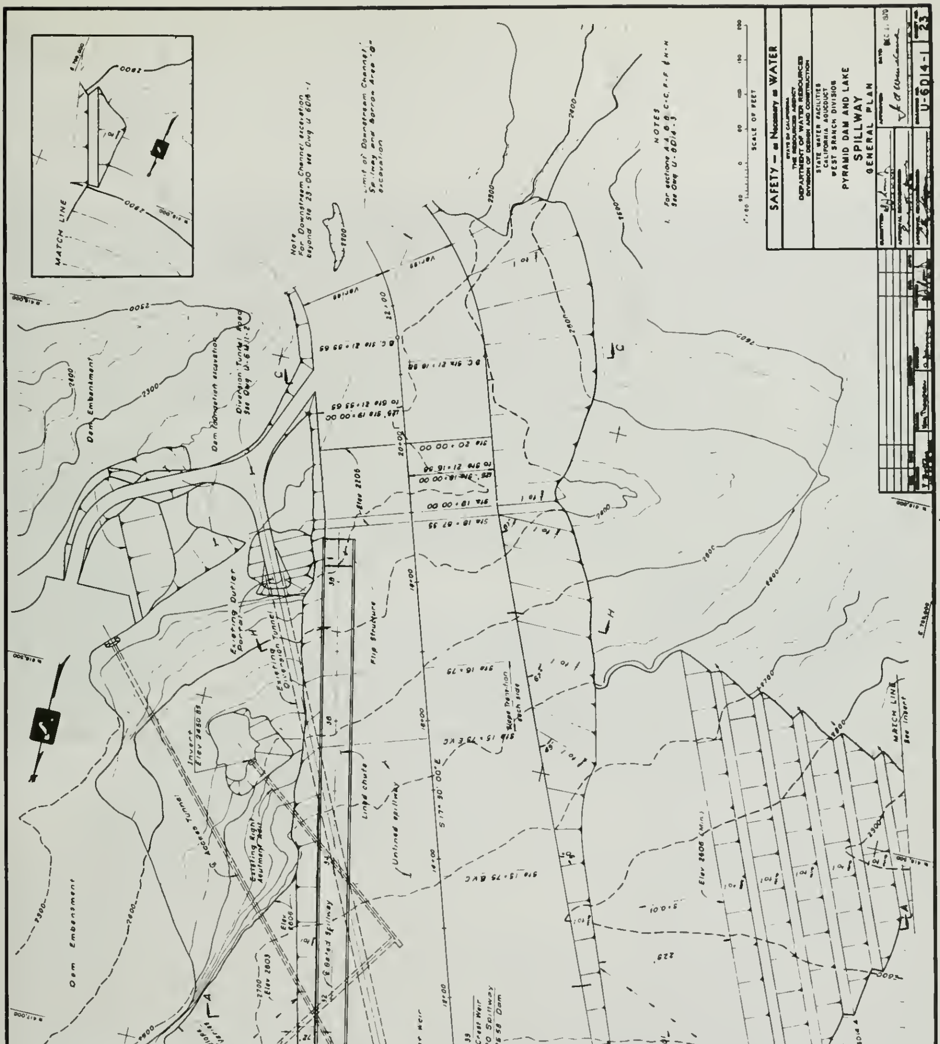

Figure 318. General Plan of Spillway

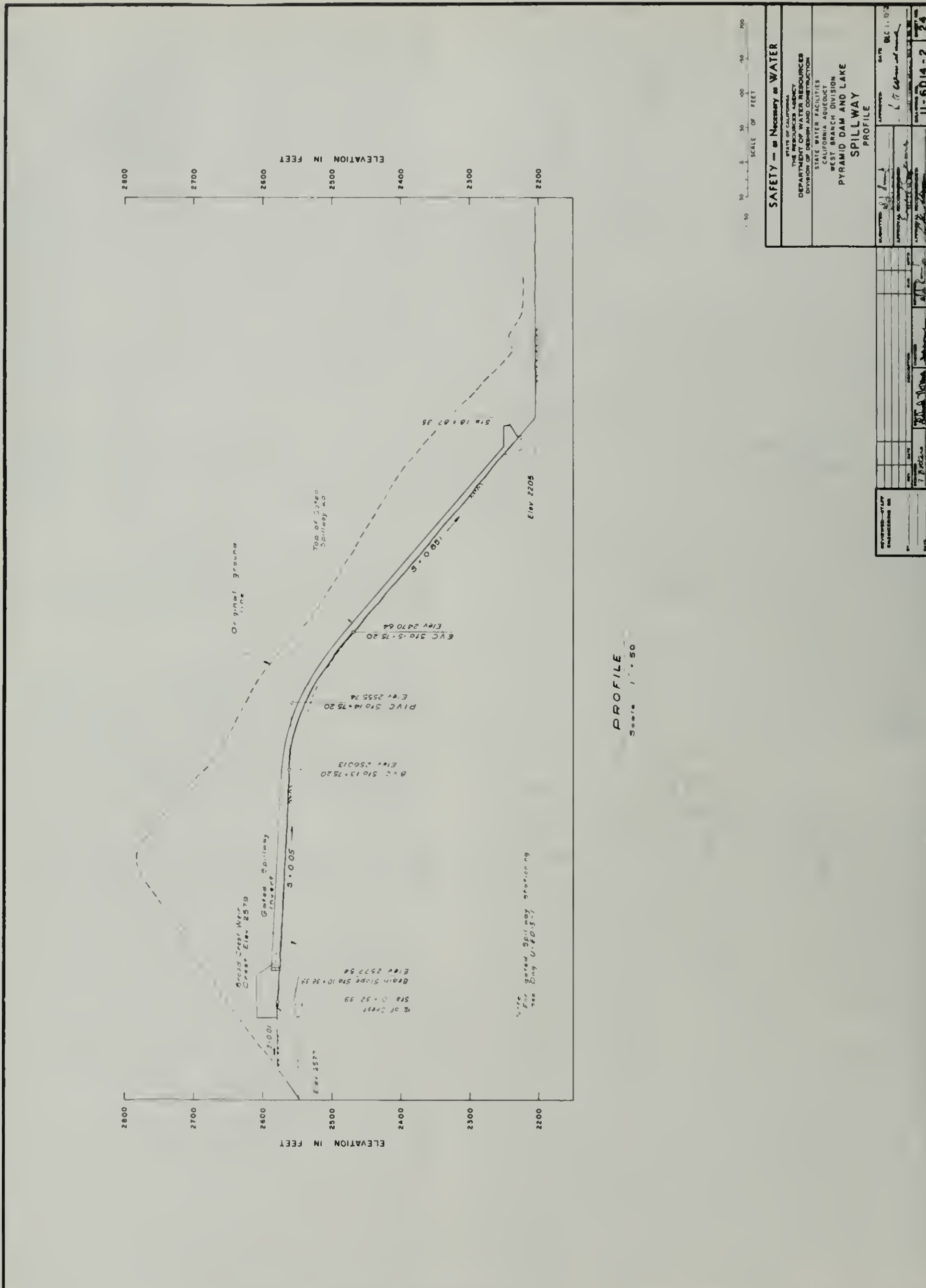

Figure 319. Spillway Profile

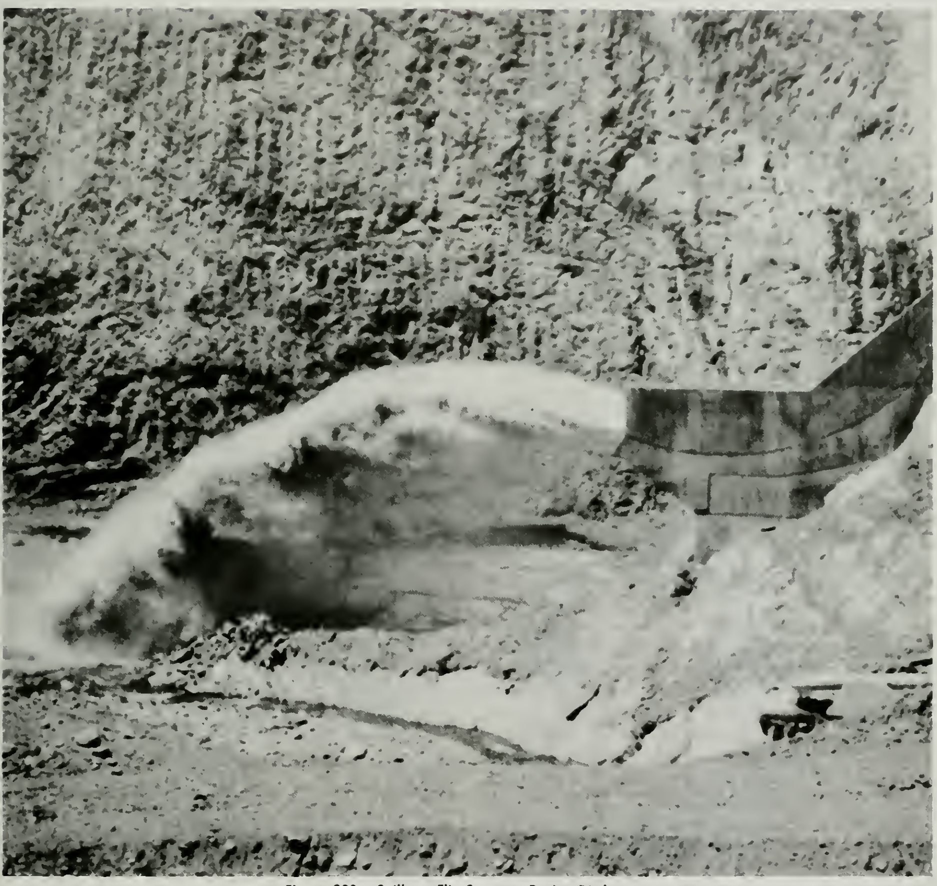

Figure 320. Spillway Flip Structure During Discharge

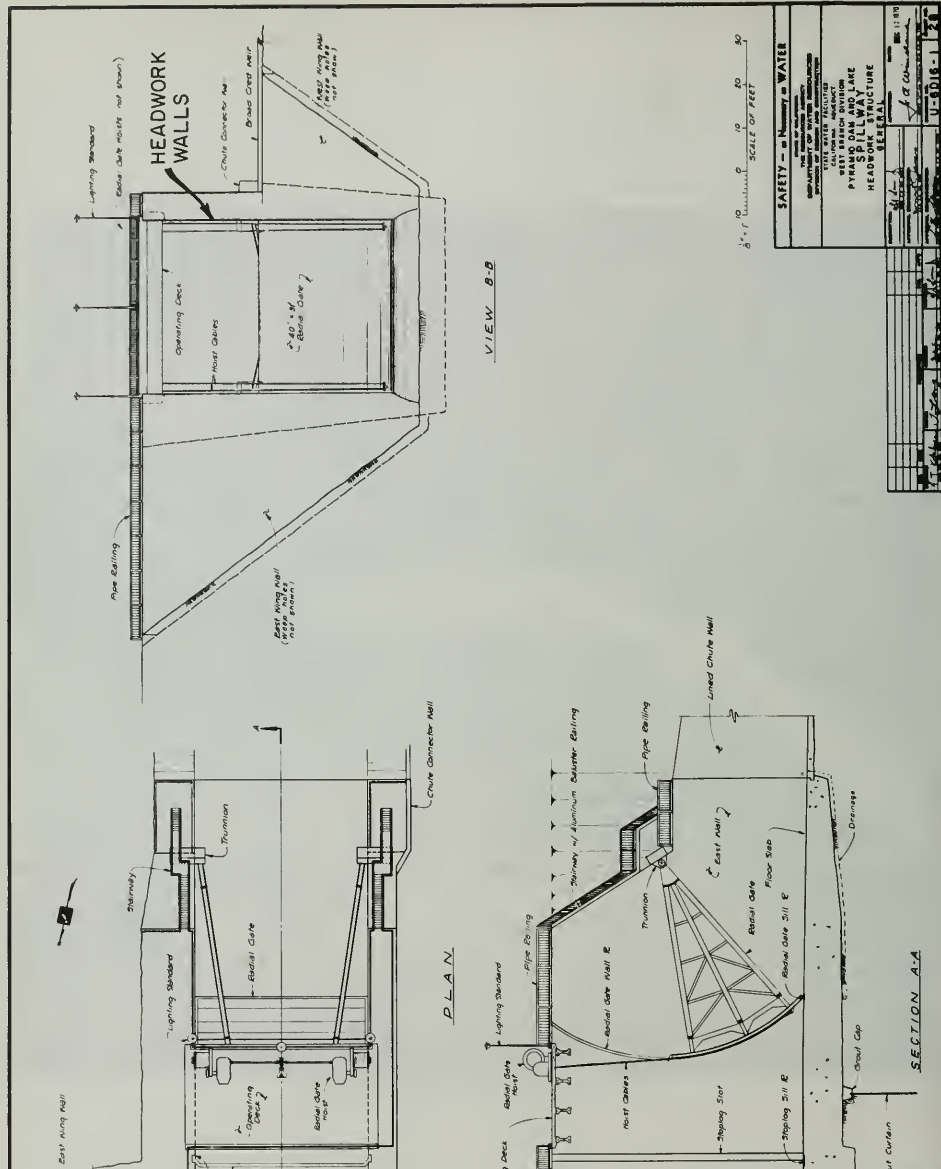

Figure 321. Spillway Headworks Structure

The structures at Pyramid Dam were founded on a hard argillite with an in-place modulus of deformation near 1.5 million psi. To determine the maximum response of structures on this foundation, an interpolation was made between the given response of structures on alluvium and an arbitrary value of that on rock, with a modulus equal to that of concrete. The seismic velocity characteristic of the material was used as the scale of comparison. An earthquake with a magnitude of 60% of San Andreas design earthquake was chosen as the maximum to be experienced by structures at this site.

Design earthquake loadings used for specific structures were as follows:

1. Spillway weir and gated passage—Failure of the gate passage will not affect the Dam nor result in a progressive loss of the reservoir. A design earthquake of 40% of the San Andreas design earthquake was used.
2. Outlet tunnel intake tower—Failure of this structure will not result in a loss of water from the reservoir nor impair the safety of another structure. Its failure, however, would require draining the reservoir for reconstruction; therefore, a design value of 50% of the San Andreas design earthquake was used.
3. Outlet tunnel downstream portal—The same criteria exist for this structure as for the outlet tunnel intake tower, except that draining the reservoir would not be required. A design value of 40% of the San Andreas design earthquake was used.
4. Features inside the outlet tunnel—History has shown that earthquake damage to tunnels resulting from vibration alone appears near the portals and little or no damage occurs inside. No specific design criteria were therefore set. Features, however, have an adequate safety factor applied to static loads.

Concrete and Steel Structures. For structures having relatively long natural periods of vibration, various modes of vibration were investigated. The natural period of each mode was calculated and the corresponding acceleration from the design earthquake was applied. For structures with short natural periods, less than 0.1 of a second, a uniform acceleration was assumed.

The design assumed that components of earthquake acceleration occur in all directions and must be considered acting simultaneously. Vertical accelerations were taken as approximately two-thirds of horizontal accelerations. As the components probably will not maximize at the same instant, full value was used on one component and a reduced value applied to the others.

Hydrodynamic Pressures. Hydrodynamic pressures on the face of structures were determined from Westergaard's formula. These pressures on submerged structures were based on an equivalent cylinder of water, modified where appropriate.

Earth Pressures. Earth pressures on structures due to an earthquake were derived by assuming Coulomb's wedge was accelerated into the structure. A critical wedge of fill was determined for various values of lateral acceleration and the angle of internal friction of material.

Materials and Design Stresses. Temporary stresses in structure due to design earthquake loads were allowed to approach the yield stress of the material. Steel used for structural members and concrete reinforcement had a relatively high elongation between yield and ultimate strength to provide a large capacity for energy absorption.

Electrical Installation

Equipment for remote monitoring and control of the Pyramid Dam and Lake facilities is located in the Pyramid control building on the left abutment. Local controls for the spillway and stream release facility are located at the spillway and in the valve chamber, respectively. Normal electrical services are supplied by a utility company, whereas standby electrical power is provided by engine-generator sets. One set is in the control building while the other is on the right abutment near the spillway.

Interstate 5 Embankments

Description. Parts of three embankments for Interstate 5 were inundated by Pyramid Lake (Figure 322). These fills cross Liebre Gulch, West Fork Liebre Gulch, and an unnamed canyon in the vicinity of the old highway maintenance station. Stabilization of the embankments for this condition involved reinforcing and extending culverts beneath the fills and placing additional ballast fill and slope-protection material. The improvements discussed here were designed, and the construction supervised, by the Department of Water Resources with review by the California Department of Transportation.

The ballast fills at Liebre Gulch and West Fork Liebre Gulch consisted of widening the existing highway fill berm at elevation 2,600 feet by a minimum construction width and sloping to the streambed at $4\frac{1}{4}$:1. Stability of the fill at the maintenance station site required raising the berm to elevation 2,620 feet and increasing its width to a total of 200 feet. The same fill slope was used here.

The level berms at the top of the ballast fills formed a portion of the relocation corridor for utility lines which traversed the lake area.

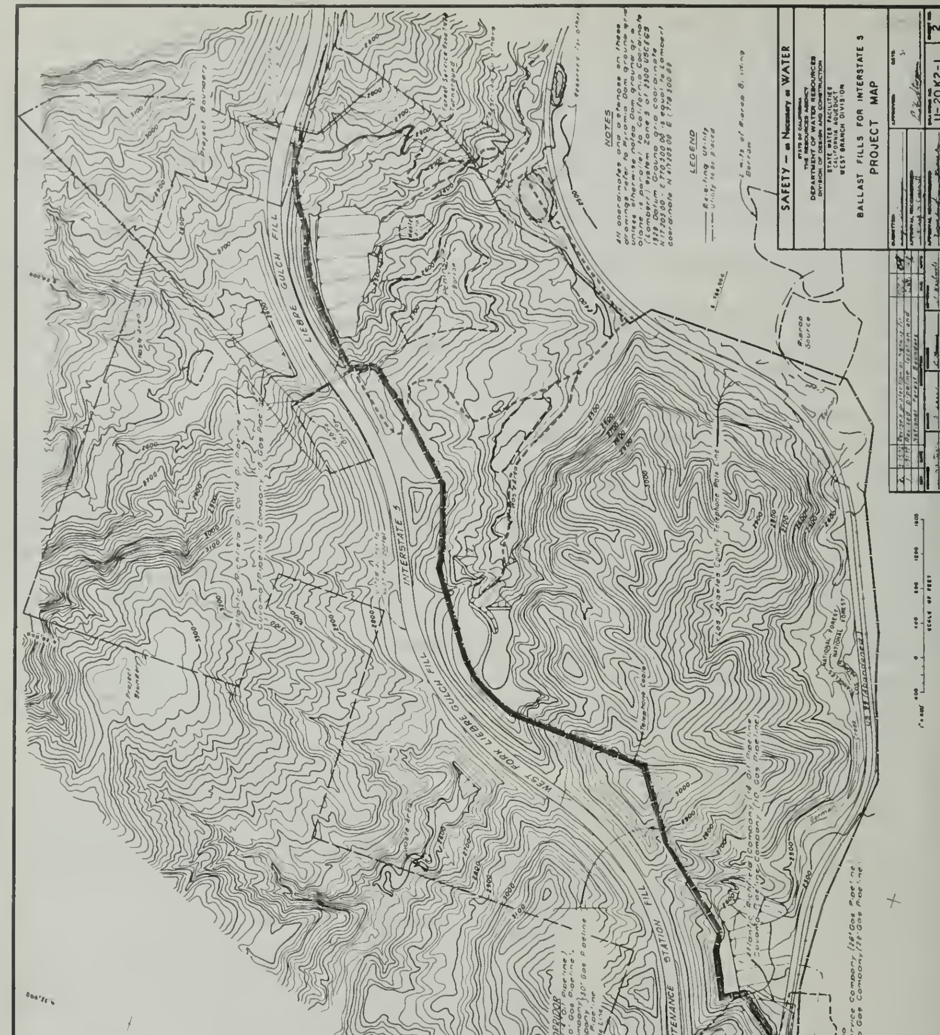

Figure 322. Interstate 5 Embankments

Shear Strength Properties. Shear strength properties were determined from saturated, consolidated-undrained, triaxial shear tests with pore pressure measurements.

Effective Stress	*Foundation Landslide Material*	*Embankment Material (Fill)*
Cohesion (C's), psf	200	200
Angle of internal friction (θ), degrees	30	34
Total Stress		
Cohesion (C), psf	500	500
Angle of internal friction (θ), degrees	15	23

Stability Analyses. Embankment stability was analyzed by sliding wedge and Swedish Slip Circle methods for the following cases:

Case I Critical pool elevation, horizontal phreatic line

Case II Pool elevation 2,570 feet, phreatic line at elevation 2,578 feet

Case III Sudden drawdown from water surface elevation 2,578 feet to toe of fill

The safety factor of the existing highway fill under dry conditions was determined first. Ballast fill was added to return the embankment safety factor to this value under reservoir conditions described under Cases I and II. The safety factor under Case III conditions was required to be above unity.

Borrow Areas. Borrow areas for embankment materials were chosen for ease of access between the area and fills, to blend best into the existing surroundings while enhancing recreation potential, and to help solve an existing slope instability problem on the highway.

One borrow area was located on Liebre peninsula, a finger of land extending westerly between Liebre Gulch and West Fork Liebre Gulch. This borrow area was excavated in such a manner as to provide a use area above normal water surface and a beach slope into the reservoir. Access berms and recreation areas, including grading for a boat ramp, also were provided by the borrow at maintenance station fill. Excavation for materials for the easterly fill slopes involved flattening the side slope of an unstable existing highway cut.

Culvert Reinforcement. The concrete arch culverts at Liebre Gulch and West Fork Liebre Gulch, approximately 16 feet in height and width, previously had shown signs of higher than expected strains. To reinforce these culverts to handle the combined loads of the existing fill (which would be saturated by the reservoir) and the ballast fill, structural-steel arches of the culvert shape were placed inside. A gunite lining was added to fill the voids between the arches and to provide a smooth inside surface.

The culvert at the maintenance station fill is a corrugated-metal pipe. To reinforce the culvert in reaches of added load, a smaller corrugated-metal pipe was placed concentrically inside and the annular space filled with grout.

As culvert reinforcement reduced the flow areas, flood hydraulics were investigated. The contribution to spillway design flood from each drainage area was determined and routed through the culvert into the main reservoir. It was found that none of the subreservoirs formed behind the fills overtopped the highway during this event.

Culvert Extensions. Prestressed-concrete cylinder pipe with a 12-foot – 6-inch inside diameter was used for the Liebre Gulch and West Fork Liebre Gulch extensions. Historically, areas around the inlets and outlets to these culverts have been subject to extensive landslides. There is a predominant bedding in the soft sedimentary rock of the area and the landslides are largely dip slope. Since saturation by the reservoir might reactivate the sliding, the inlets and outlets were located in the safest possible sites with the primary inlets placed above the streambeds to allow for slides and silt accumulation below them. Thirty-inch-diameter concrete pipe was used to provide lower secondary inlets to the culverts. The secondary inlet at Liebre Gulch assured nearly balanced water levels on both sides of the embankments during initial filling and balanced water levels during subsequent drawdowns. In addition to these functions, the secondary inlet at West Fork Liebre Gulch conveys water into and out of the small subreservoir (water behind the fill) at that site as the primary intake is set above the maximum operating surface elevation.

Construction

Contract Administration

General information about the major contracts for the construction of Pyramid Dam is shown in Table 41. The principal construction contract, Specification No. 71-03, included construction of the dam embankment and excavation and concrete construction for the spillway. Work done under other contracts included excavation of the diversion tunnel, excavation of exploratory adits in each abutment, construction of an access road and bridge, construction of ballast fills to stabilize the slopes of Interstate 5 embankments which were inundated by Pyramid Lake, mechanical and electrical work required for completion of Pyramid Dam outlet works, and a seepage weir at the downstream toe of Pyramid Dam. Features of the Angeles Tunnel, which extends from Pyramid Lake to Castaic Powerplant, also were constructed under other contracts.

Diversion Tunnel

The diversion tunnel was one of several tunnels constructed for the Department on a lump-sum bid item. This approach was tried after major overruns occurred in support system costs on other tunnels. Safe working conditions were obtained even though less tunnel support was required.

Excavation. Excavation of the 1,233-foot-long diversion tunnel was started on January 8, 1970. Excavation was performed by conventional tunneling methods using a job-built rubber-tired jumbo on which were mounted one burn drill and four stopers. This jumbo, which had been used on the Angeles Tunnel, was modifed to fit into the diversion tunnel bore. Drilling and blasting operations were started from the south portal. The jumbo was moved out of the tunnel after each drilling cycle. Blasted material was removed by a front-end loader which backed out of the tunnel and deposited the muck in a temporary disposal area just outside the portal. Front-end loaders removed muck from the temporary location and hauled it to the designated spoil location just east of the portal. The tunnel was excavated full section to a nominal 17-foot diameter, except the valve transition area which was a nominal 21-foot diameter and the valve chamber, a nominal 33-foot diameter. Few difficulties were encountered.

Horizontal and vertical survey control for driving the tunnel was maintained by the use of a laser system.

Limited use was made of steel tunnel supports since the surrounding rock was competent. Five-inch, wide-flange, horseshoe-shaped ribs were used at the portal areas and in a short zone of less competent rock within the tunnel.

Ventilation for the tunnel during driving was provided by an in-line fan mounted above the south portal. Air was conducted to the heading by a 40-inch-diameter fan line salvaged from the Angeles Tunnel. The fan line was suspended from the tunnel roof by short rock bolts.

Concrete. Tunnel concrete placement started at the upstream portal and was divided into six segments. Invert and arch placements were made upstream of the valve chamber, in the chamber, and downstream of the chamber. Concrete was produced at the Angeles Tunnel south adit batch plant and hauled to the job site by transit mix trucks, which backed into the tunnel to discharge into hoppers. Concrete was pumped from the hoppers through a 6-inch-diameter slickline onto a conveyor belt and then to the placement where it was vibrated into place.

The 119-foot-high, reinforced-concrete, intake structure and the reinforced-concrete outlet structure were founded on bedrock. A total of 3,990 cubic yards

TABLE 41. Major Contracts—Pyramid Dam

	Specification	Low bid amount	Final contract cost	Total cost-change orders	Starting date	Completion date	Prime contractor
Pyramid Dam Adits	67-30	$239,250	$259,018	--	10/18/67	3/25/68	Bill Anderson Co. and Bill Anderson Co., Inc.
Pyramid Dam Initial Facilities	69-21	2,498,484	2,558,028	$-48,838	10/13/69	2/ 8/71	Shea-Kaiser-Lockheed-Healy
Pyramid Dam and Lake	71-03	22,036,875	26,533,214	760,662	5/26/71	2/ 1/74	Shea-Healy
Completion of Angeles Tunnel Intake Works and Pyramid Dam Outlet Works	71-10	4,552,630	4,902,094	347,981	7/ 6/71	4/18/74	Wismer & Becker Contracting Engineers
Ballast Fills for Interstate 5	71-27	4,222,222	4,479,582	870,189	1/19/72	7/10/73	Kasler Corp., Gordon H. Ball, Inc., & Robert E. Fulton Co.
Pyramid Lake Gauging Stations	73-43	47,216	57,797	1,522	11/14/73	6/ 7/74	Ted Watkins
Completion of Pyramid Dam	74-40	418,796	460,000 (Est.)	--	9/ 6/74	1/ 6/75 (Est.)	Ray N. Bertelson Co., Inc. and Ray N. Bertelson Co.

of concrete was required for these facilities.

Forms for these structures were built in place or as close as possible to the site in order to minimize handling and transporting. Concrete was placed by using a mobile crane and a 2-cubic-yard bucket.

Diversion and Care of Stream

In June 1971, Piru Creek was diverted into Pryamid Dam diversion tunnel through the low-level intake controlled at that time by one 30-inch slide gate at the base of the diversion tunnel intake tower. The diversion was accomplished by construction of a 6-foot-high dike across the stream channel. Dewatering of the dam foundation was accomplished by the use of several small pumps situated in low areas. During the fall of 1971, the interim dam was constructed to elevation 2,320 feet, and an interim spillway with a crest elevation of 2,293 feet was cut through the right abutment ridge. The interim dam permitted diversion of project flows through Angeles Tunnel and natural streamflow though two low-level, 30-inch, slide gates in the diversion tunnel intake tower. All natural inflows during the 1971–72 runoff season were passed through the slide gates except those resulting from a Christmas-week storm. During this storm, a flow of approximately 300 cfs flowed over the crest of the interim spillway.

Foundation Preparation

Overburden. Overburden in the foundation area, comprised of streambed material, old highway fill, and weathered shale, was excavated with rubber-tired front-end loaders with 7-cubic-yard buckets assisted by bulldozers. The material was hauled by dump trucks and rock wagons. Overburden was disposed of in the mandatory waste area at the upstream toe of the Dam, in the buttress fill, and the upstream waste area.

Removal of the highway fill in the downstream area of the foundation revealed four old highway bridge bents of reinforced concrete (Figure 323). These were located in the pervious shell section of the Dam, and they were left in place as they did not hinder placement or compaction of the embankment.

Shaping. The right abutment of Pyramid Dam was very steep and irregular. Extensive shaping excavation was performed to provide a uniform surface for placement of impervious embankment and to flatten the slope in the area of the old Highway 99 cut slope (Figure 324). Drilling for this excavation started at the end of the Dam in January 1971, and excavation to the stream channel was completed in March 1971. The slopes of the excavation were presplit with an average hole spacing of 30 inches. This produced a neat line excavation with deviations limited, in general, to within 12 inches of the drilled line (Figure 325). Lifts normally were 30 feet deep.

Five areas on the dam abutments had overhangs. These were designated as shaping areas in the plans and were required to be laid back to a slope of 1/4:1

Figure 323. Old Highway 99 Bridge Bents Uncovered in Downstream Shell Area of Dam

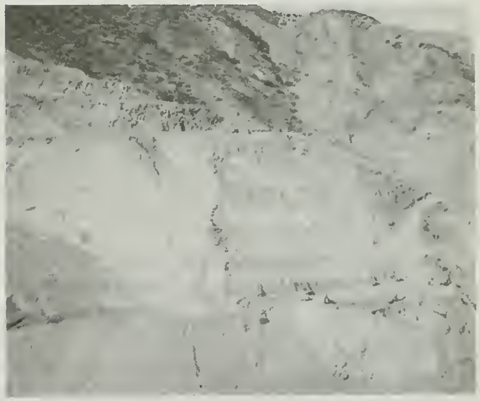

Figure 324. Shaping Excavation on Right Abutment

Figure 325. Presplit Face of Shaping Area on Right Abutment

in shell areas and 1/2:1 in the core zone. It was required that these areas be removed by the presplit method. Presplit hole spacing was varied from 18 inches to 36 inches, and it was found that a spacing of 30 inches gave excellent results.

Cleanup. After overburden material was removed, the final cleanup prior to placement of embankment was accomplished. In the rock shell zones, this consisted of machine removal of loose material. Backhoes and graders were used for this operation. In the core zone, great care was taken to remove all loose and weathered material to expose fresh solid rock. Final cleanup was by air and water jets (Figure 326). Several deep holes encountered in the stream channel in the core zone were backfilled with concrete to provide a uniform surface which facilitated placement and compaction of the impervious material.

Grouting. The Pyramid Dam grout curtain consists of a singe line of holes with 10-foot maximum spacing. Grouting was accomplished in three zones by the split-spacing stage-grouting method, usually with a primary 40-foot spacing. Zone depths were 0 to 25 feet, 25 to 50 feet, and 50 to 100 feet. The third zone was extended to 200 feet in three holes in the channel. The holes were drilled normal to the slope. In the three zones, the pressures at the grout nipple (surface) were 15 psi, 35 psi, and 75 psi. The foundation was tight, and the average grout take was only 0.13 of a bag per foot of hole.

Channel Excavation

Alluvium in Piru Creek downstream from the Dam was excavated to form the downstream spillway channel. Excavation and disposal of this material was performed in a manner identical to that used for overburden excavation.

Embankment Materials

Impervious. The borrow area for the impervious material was located in the lake area about 1 mile upstream from the Dam. It was divided into two subareas: one lying along the east side of the stream channel contained slopewash material and the second was higher on the slopes immediately east of the first and was comprised of weathered in-place shales. While both materials were very clayey, slopewash material was somewhat finer than the weathered shale. It was used in the upstream portion of the core, while weathered shale went into the downstream portion of the core.

Clearing and grubbing of the impervious borrow area, which had a brush and grass cover, were done with bulldozers and a small labor crew. The original intent was to strip up to 18 inches of surface material prior to borrowing but, because of the light vegetative cover, this was not necessary and stripping depth averaged less than 6 inches.

Following clearing and stripping, the area was sprinkled by a portable water system to bring the

Figure 326. Air-Water Jet Cleanup of Foundation

material to the desired moisture content. Ripping was used to enhance the water penetration, and loading operations were shifted to ensure that properly conditioned materials were delivered to the Dam. In general, this system worked very well, and only minor supplemental sprinkling was done at the Dam site. It was somewhat more difficult to moisture-condition the weathered shale than the slopewash material. This was because the weathered shale was situated on steeper slopes and water did not penetrate the shale fragments as easily as it did the more uniform slopewash.

Bulldozers pushed the impervious material to front-end loaders which, in turn, loaded the hauling units.

Two types of haul units were used: dump trucks (Figure 327), which could haul approximately 20 cubic yards bank measure, and tractors with rear-dump wagons that had a capacity of approximately 15 cubic yards (Figure 328). The haul route was downslope on old Highway 99 to the upstream toe of the Dam and then up a 15% grade on a 40-foot-wide haul road traversing the face of the Dam.

Rock Shell. Rock for the upstream and downstream shells of the Dam is a hard shale (argillite) obtained primarily from the spillway excavation. Other sources were: the abutment shaping areas, overburden removal, channel excavation, material that had been stockpiled under the initial facilities contract which included the access roads and the Angeles Tunnel gate-shaft bench excavation, and an auxiliary borrow area above the left abutment.

Drilling for excavation of the spillway began in July 1971. Initial drilling was accomplished with crawler-mounted, air-powered, self-propelled, percussion drills. The air supply was a compressor plant at the downstream toe of the Dam. An 8-inch air-supply pipeline extended from the compressor plant up the west limit of the excavation to the top of the spillway

Figure 327. Dump Truck Used for Embankment Material Hauling

Figure 328. Rear-Dump Rock Wagon Used for Embankment Hauling

cut. After adequate access and working areas had been developed, truck-mounted, diesel-powered, rotary drills were used for drilling production holes. A typical drill pattern used with these units was 6¾-inch holes on a 22-foot by 22-foot grid. The maximum lift thickness was 50 feet. The holes were loaded with ammonium nitrate and fuel oil (ANFO) with dynamite primers. Detonation was by electric blasting caps. The ANFO was delivered by bulk trucks and loaded into the holes with the fuel oil being added as the ammonium nitrate was fed into the hole. The largest blast of the operation comprised 67,225 pounds of explosive. The average powder factor for the spillway excavation was 0.6 of a pound of powder per cubic yard of rock. The cut slopes of the spillway were presplit. Although this was not required by the specifications, the contractor considered this to be economical as it saved barring down and cleanup of the slopes that would have been required with conventional blasting.

The specifications were written to preclude the bulldozing of rock so that breakdown during handling would be minimized. Therefore, it was necessary for the contractor to build haul roads from the excavation site, i.e., the spillway ridge, to the dam embankment. This involved roads traversing a maximum difference in elevation of approximately 800 feet and required the use of a 15% haul-road gradient. Two roads were constructed: one to the downstream part of the unlined spillway, and the other from the upstream side of the Dam to the top of the spillway excavation. The downstream road involved a fill of almost 900,000 cubic yards with material from stripping approximately 25 feet of weathered rock from the spillway chute area. As the chute excavation was brought down, this road material was removed and placed in the Dam, if suitable, or otherwise in the waste area. The upstream haul road was cut into the steeply dipping slope of the right abutment until, at a point about 150 feet below the dam crest, it swung out onto the upstream buttress fill.

The haul units used for transporting rock from the spillway excavation were the same types as used for the impervious material. They were loaded by rubber-tired front-end loaders, some of which had a special steel tread to protect the tires in the rocky material. A spread of one loader and five haul units could excavate and haul about 5,000 cubic yards in a 10-hour shift. Maximum daily production for two 10-hour shifts was approximately 30,000 cubic yards.

Transition and Drain. Materials for the filter and drain zones of the Dam were obtained from the stream channel of Piru Creek upstream from the Dam. The sands and gravels were dozed into piles and then loaded into the dump trucks and rear-dump wagons by rubber-tired front-end loaders. They were hauled to a stockpile in the lake area about 1½ miles upstream from the Dam site. From the stockpile, they were pushed by bulldozer to a processing plant which separated them into desired fractions.

The specifications provided for production of three types of material from the stream-channel borrow area. These were: (1) minus ¾-inch material for the filter zone, Zone 2A; (2) ¾-inch to 6-inch material for the drain zone, Zone 2B; and (3) plus 6-inch rock for riprap. After placement of the first drain zone material, it was noted that undesirable segregation occurred at the interface between drain and transition zones due to the tendency for the larger rocks in the drain to roll to the outside when a lift was placed. Because of concern that this segregation would permit migration of fines from the filter zone into the drain, a change order was issued to provide for an additional transition zone. This added zone comprised a mixture of the drain and filter zone materials and was designated Zone 2D.

The processing plant consisted of a grizzly for removal of plus 6-inch rock and two sets of vibrating screen decks for separation of the minus 6-inch material into the two desired fractions. The added transition zone, which was essentially a pit-run material with plus 6-inch rock removed, was produced by blanking off the lower screens of the vibrating decks. Production rates varied from 230 cubic yards per hour when producing Zones 2A and 2B to 300 cubic yards per hour for Zone 2D.

After production was started, it was found that the dry screening contemplated by the specifications did not remove enough fines to produce filter material with the specified 10% maximum of material passing the No. 200 mesh screen. Wet screening was considered but, after study and additional permeability testing, it was concluded that the allowable amount of material passing the No. 200 screen could be increased to 15% without detriment to the Dam. A change order was issued to cover this modification.

The proportion of ¾-inch by 6-inch drain zone rock in the borrow pit turned out to be less than contemplated, so a shortage of this material developed. This problem was alleviated by lowering the top elevation of the downstream drain blanket and replacing the upstream drain zone with a zone of weathered rock obtained from dam overburden and spillway excavation. It was still necessary, however, to produce about 100,000 cubic yards of minus ¾-inch material in excess of what was needed in order to generate the necessary quantity of drain rock.

The riprap, drain rock, and transition materials were hauled to the Dam site by the same equipment and over the same haul route that was used for the impervious fill.

Embankment Construction

Pyramid Dam embankment was constructed in two stages. A low interim dam at the upstream toe of the main dam was built in the fall of 1971 to divert project flows through Angeles Tunnel. Construction of the interim dam also provided an excellent opportunity to check out the specified placing and compaction procedures before construction of the main dam. It was found that the specified methods worked well, and no problems were experienced in obtaining desired compaction of the fill when these methods were properly followed.

Impervious. Placement of impervious fill in the interim dam commenced in August 1971. The first step was to cover the foundation area with contact material which was placed about 2% above optimum moisture and wheel-rolled with a rubber-tired front-end loader with a loaded bucket (Figure 329). Throughout placement of the core sections, the contact material was placed about 2% wetter than the remainder of the impervious fill so that it would easily conform to irregularities in the rock surface. As the fill progressed, a layer of contact material 10 to 15 feet wide was brought up ahead of, and compacted prior to, the rest of the embankment. The abutment contact line generally was maintained 1 to 2 feet higher than the plane of the embankment, with the surface sloping gently away from the abutment. These measures were taken to ensure coverage of the foundation with the wetter material and to preclude the possibility of loose material rolling into a low area against the abutment and being poorly compacted.

After compaction of contact material to an approved elevation, the impervious embankment was

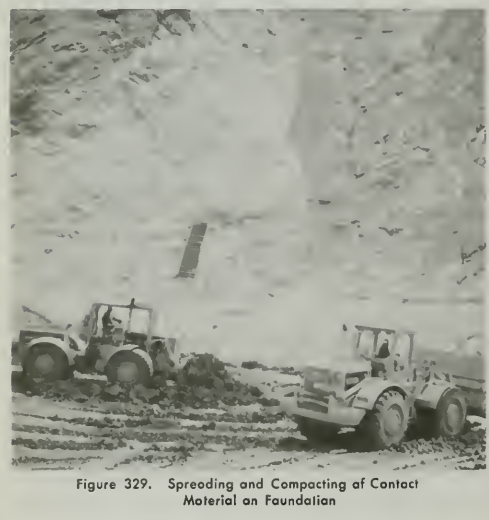

Figure 329. Spreading and Compacting of Contact Material on Foundation

Figure 330. Rolling of Impervious Fill

raised to that elevation. After being dumped by the hauling units, the material was spread with a bulldozer into 8-inch loose lifts, then processed by at least two passes of a disc. At this time, if any supplemental moisture was required, it was added by a water wagon. After disking, the lift was compacted by 12 passes of a self-propelled, four-drum, sheepsfoot roller (Figure 330). Compaction of the fill was closely monitored with an average of one relative compaction test being taken for each 2,500 cubic yards of fill placed.The average relative compaction for the impervious embankment was 98.5%.

Shell. Rock shell material was dumped and spread in 3-foot-thick layers and then rolled with two passes of a vibrating drum roller (Figure 331). Both self-propelled and towed rollers were used. The compacting and grading of this material were closely monitored by testing. These tests involved considerable effort as a field density test required the excavation and weighing of 1,000 to 1,500 pounds of rock, and mechanical analyses required the screening of several thousand pounds of material.

Transition and Drain. Material for the transition and drain zones was spread in 18-inch lifts. Transition material was compacted by two passes of a vibrating roller and drain material by one pass. Here again, compacting and grading were closely monitored by daily field testing.

Spillway

After the spillway area was excavated down to the elevation of the unlined chute, the cut for the 40-foot-wide concrete-lined chute was made using presplitting to shape the cut slopes against which concrete lining was placed.

Figure 331. Compacting Pervious Material With 10-Ton Vibratory Roller

Drainage. The spillway underdrain system plans called for horizontal, triangular-shaped, gravel-filled, outlet drains to be cut into the east slope of the spillway excavation at 10-foot vertical intervals. This would have been extremely difficult to do, and the contractor proposed an alternate system which was adopted. This consisted of embedding 4-inch plastic pipes in the reinforced-concrete wall leading from gravel drains under the invert slab and discharging into the chute. The originally detailed gravel-filled drains were intended to provide drainage for the wall lining as well as for the invert slab. This function was provided by drilling horizontal drainage holes into the rock at points where the plastic drains returned into the chute.

Drain holes in the 2% slope portion of the chute invert were made by installing 4-inch-long pieces of 4-inch-diameter plastic pipe in the invert concrete as the concrete was finished. Later, a drill rig was brought in to drill the drain holes. This method gave a neat hole in the invert as spalling of concrete by the drills was eliminated. For the horizontal drain holes in the spillway wall, plastic pipe sleeves were substituted for the originally detailed steel to eliminate rust and consequent staining.

Reinforcing Steel. Installation of reinforcing steel was more or less routine except that this operation, like all others involved in the spillway work, was complicated by the steep slope of the chute. Inconvenience was caused by the fact that carpenters, ironworkers, and laborers had to work in close quarters on the steep slope.

Concrete Production. All concrete for the work was produced by a job-site, automated, batching and mixing plant with an 8-cubic-yard tilting-drum mixer with a theoretical capacity of 120 cubic yards per hour. This was larger than needed but was on hand from the Angeles Tunnel contract. Ice was added to the mix to hold the temperature as near 50 degrees Fahrenheit as possible. It was brought to the site in 300-pound blocks and chipped and blown into the mixer as needed. Bulk cement and pozzolan were stored in silos at the plant, and aggregates were stockpiled by the delivery units and then loaded into the batching bins by a front-end loader via a rinsing screen. The specifications required that all aggregate be shaded on the day of use. In lieu of this, the Department approved the contractor's proposal that the aggregate piles be sprinkled for cooling.

Concrete for the headworks walls, broad-crested weir, and all chute invert sections contained 3-inch maximum size aggregate. Concrete for the chute walls and operating deck contained 1½-inch maximum size aggregate. Slumps ranged from 1 to 2 inches for the inverts and massive wall placements. An average slump of 3 inches was used for the chute wall placement.

Forms. The basic form used for the wall placements was a 32-foot-long, prefabricated, steel form complete with struts which spanned the spillway width and formed both side walls at one setup (Figure 332). Form heights were adjusted by addition or removal of panels. A system of jacks and ratchets enabled adjustment of the form for sloping walls. This unit was pulled up the slope by a double-drum hoist which was anchored at the top of the 85% slope. The form assembly was supplemented by wooden panels where necessary, and transverse joints in the walls were formed with wood.

The invert slab of the spillway was formed with a 10-foot-long by 40¼-foot-wide, steel, slip form with a weight of approximately 60 tons. This was comprised of six 27-inch WF beams spanning the chute slab. These beams were tied together by four 6-inch WF beams at the top and a ¼-inch-thick skinplate on the bottom. The spaces between the beams were filled with concrete. The form was suspended by steel wheels riding on 75-pound rails which were supported by a system of 3-inch pipe posts and No. 11 reinforcing steel braces grouted into the rock in the wall sections of the spillway. The rails were set 2½ feet above spillway invert grade. The slip form was pushed forward (upslope) during concrete placement by two 6-inch hydraulic rams which were clamped to the rails. The cylinder stroke was 6.5 feet so several strokes were required to complete a 30-foot slab. While the cylinders were being retracted and reclamped, the slip form was anchored by cable to the double-drum hoist at the top of the slope.

The headworks structure walls were formed with a combination of steel and wood forms and were placed in 14½-foot lifts.

Concrete Placement. The concrete was hauled to the placement site in transit mix trucks. Two five-man labor crews, one for concrete placement and one for cleanup, worked a normal day shift; an additional labor crew worked a night shift to sandblast, clean up, and prepare for placements. Consolidation of the concrete primarily was done with 6-inch immersion-type vibrators. In some instances, 3½-inch vibrators were used to supplement the larger ones and to consolidate in congested areas.

The first portion of the lined spillway to be placed was the invert slab of the upper chute, which was on a 2% slope. Concrete for the ten 30-foot-long sections was placed by crane and bucket, with the crane being situated on the invert of the unlined spillway. The concrete was struck off by a rail-mounted screed, 3 feet long and 40¼ feet wide. This screed was a segment of the slip form to be used on the steeper slopes and was moved by hydraulic cylinders clamped to the rails. A platform from which the finishers worked was pulled behind the screed. A steel trowel finish was specified for the invert, and cold weather, which prevailed at that time, retarded the concrete set and caused long finishing hours. In general, these placements proceeded well and the contractor was able to make one 30-foot section per day.

The headworks structure was the second spillway feature to be placed. The major problem encountered in this work was in connection with placement of the invert slab. An attempt was made to place the entire invert in one placement (2,000 cubic yards) by utilizing "drop pipes" in conjunction with one crane and 4-cubic-yard-capacity buckets. The drop pipes consisted of five 10-inch-diameter steel pipes along each side of the headworks (Figure 333). Concrete was dumped directly from buckets into the center of the invert and dropped through the pipes along the sides. Small hoppers about 2 feet square were mounted on top of the pipes. Concrete was conveyed from the transit mix trucks to the hoppers by chutes. Discharge of the concrete from the trucks through the gently sloping chutes was very slow. The large aggregate tended to roll down the chutes, jump over the hoppers, and fall among the workmen below. Concrete also plugged the pipes several times; it tended to stack at the pipe outlets and had to be flattened and moved with vibrators. Surprisingly, little segregation was noted at the pipe outlets. After ten hours of placing, and with only about one-third of the invert done, the contractor elected to stop the placement and make a construction joint. Prior to placing the second lift of the invert, the inlets of the drop pipes were lowered, chutes steepened, and an additional crane brought in. These changes increased the placement rate significantly, and the remainder of the invert was completed without any major problems.

After completion of the headworks concrete, the chute walls in the 2% slope section were placed. Both walls were placed simultaneously and were formed by a steel-form strut assembly. Except for tight clearances due to the narrow wall, no major problems were encountered and the contractor was able to place a section every other day.

After completion of the walls on the 2% slope, the invert of the 85% slope section was placed. The slip form performed well and produced a good surface that required little additional hand finishing. Movement of the slip form was slowed by trouble in attaching the ram clamps to the rail. Attaching and releasing these clamps was a major factor in delaying many of the placements. The concrete was spotted in front of the slip form by a 120-ton crane with up to 220 feet of boom. This crane was able to reach the lower 3 sections from the toe of the unlined spillway slope, the next 11 sections from the right adit bench, and the remaining section from the top of the 85% slope.

When operating with the maximum length of boom, placements were slow due to the flexibility and resultant "bounce" of the boom. Also, in several areas, the operator was operating "blind" and had to spot the bucket by telephone, radio, or hand signals. Despite these conditions, one invert section per day was placed (Figure 334).

In placing the wall sections on the steep chute, the same problems with operation of the crane were encountered as for the invert. Also, due to the steep slope, the tops of the walls had to be formed as placement progressed.

The final feature of the spillway to be placed was the flip structure at the end. It was placed in four lifts, and no major problems were encountered except for some difficulty in slip forming the invert, which was on a 40-foot radius. A 3-foot-long slip form on curved pipe rails was used for this.

In general, the concrete for the lined spillway turned out well despite the narrow battered walls and steep slope. Considerable repair work was required along the tops of the walls. The concrete was well consolidated and reasonably well finished.

Radial Gate

The 31-foot-high, 40-foot-wide, radial gate was fabricated by Hopper, Inc. in Bakersfield, California. Prior to shipment, the entire gate was fabricated with two 8-inch pipe spreaders, assembled in the shop, and checked for correct spacing and alignments. Temporary installation of the same two spreaders in the field facilitated correct positioning of the trunnion girders. On October 30, 1975, the gate was lowered to contact the sill plate and was found to have a ⅝-inch bow in the bottom edge. Under the direction of the manufacturer, the gate was straightened by the alternate application of controlled heating and cooling to within a tolerance of plus or minus ¼ inch. Final adjustment of the gate sill plate was made by lowering the gate and adjusting the plate to meet the bottom edge. The gate installation was quite good as attested by the fact that, when the reservoir was filled, leakage past the side seals was nil and only a couple of damp spots showed up downstream of the bottom seal.

Access and Air-Supply Tunnel

Driving of the access tunnel was started on July 1, 1971 and completed on July 23, 1971. The contractor worked 27 ten-hour shifts in driving the tunnel an average of 28 feet per shift.

The air-supply tunnel is a circular tunnel 5 feet in diameter and 90 feet in length connecting the crown of the access tunnel with the crown of the valve chamber of the Pyramid Dam diversion tunnel. This tunnel was completed in six 10-hour shifts with an average advance of 15 feet per shift.

The equipment used by the contractor was left over from the Angeles Tunnel job. The jumbo was contractor-built and modified to adapt it to the access tunnel size and shape. The 8-foot by 11-foot access tunnel was driven about 1 foot oversize at the contractor's request as he elected not to reduce the basic size of the jumbo. The contractor was not paid for the extra concrete lining in the overexcavation. The jumbo was on rub-

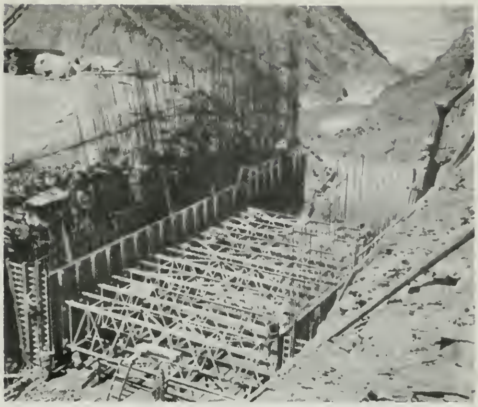

Figure 332. Prefabricated Steel Form Used for Spillway Walls

Figure 333. Concrete Placement in Spillway Headworks Invert

Figure 334. Concrete Placement in Spillway Chute Slab

ber tires, diesel-powered, and mounted with four pneumatic drills—two upper and two lower. These drilled 45 holes, 1½ inches in diameter, and a 3-inch "burn" hole for each round. Compressed air for the drilling initially was supplied by three 600-cubic-feet-per-minute (cfm), portable, air compressors. Later, two 900-cfm, portable, air compressors were used. At the end of tunnel driving, compressed air was supplied by two 2,000-cfm and two 1,200-cfm stationary air compressors. These last compressors also supplied air to other drilling operations on this contract and to the Angeles Tunnel intake works contract.

After drilling of a round was completed, the holes were loaded (using the jumbo as the loading platform) with caps being used to control the sequence of the blast.

After a shot was set off, the tunnel was ventilated using a 7½-horsepower ventilating fan with a 35,000-cfm capacity.

The muck pile was removed from the face and hauled out of the tunnel by a front-end loader. After the muck pile was completely hauled out, the jumbo returned to the face to resume drilling.

There were no problems or delays during tunnel driving. No water was encountered. The tunnel was monitored for explosive and hazardous gases, but there was never any indication of their presence.

No steel supports were used in the tunnel. After the driving operation was completed, the contractor elected to place shotcrete, approximately ½-inch thick, in two areas intersected by beds of Pyramid shale which were much softer and more fractured than the Pyramid argillite.

Preparations for lining the access tunnel and the air-supply tunnel were begun on June 26, 1972. Work included scaling loose material from the walls and crown and removing loose muck from the invert. The first concrete placement of the tunnel lining was made on July 11, 1972, and the concreting was completed with the placing of the portal structure on October 26, 1972.

Six concrete placements were made for the tunnel invert. Sixteen placements were required for the walls and crown, two for the air-supply tunnel, and one for the portal structure.

All concrete for the tunnels was produced at the job-site, central, batch plant and transported into the tunnel in transit mix trucks. Concrete for the invert section was a standard design five-sack mix with 1½-inch maximum size aggregate. Wall and crown section concrete was the same mix except that sand content was increased to approximately 40% to facilitate pumping.

The initial invert placement was made using manually operated buggies. Due to the excessive amount of labor required for this method of placement, the remainder of the invert was placed using a belt conveyor. Use of the over 200-foot-long, self-propelled, conveyor belt enabled invert placements to proceed with a minimum of labor and with no major problems. The concrete was consolidated with 6-inch and 3-inch, pneumatic, immersion vibrators. Inconsistent slump of the concrete and lack of competent finishers were the primary problems encountered in the invert placements. Scheduling of reinforcing steel installation to prevent congestion in the tunnel was another persistent problem.

A 6-inch hydraulically operated pump was used for the first placement of the walls and crown. Plugging of the 8-inch slickline occurred several times during this placement, so this pump was replaced with an 8-inch mechanically operated pump for the remainder of the wall and crown sections. This pump performed satisfactorily and could handle the desired 4 inches or less slump concrete. Up to 450 feet of slickline was used. Air sluggers were used along the slickline to aid in moving the concrete.

A 45-foot-long steel form was used for the normal section of the walls and crown. This form was mounted on dolly-type wheels and was hinged at the crown. A system of ratchets and jacks folded and lowered the form, which enabled easy and fairly rapid spotting and removal of the forms for each placement. The form was towed along the previously placed invert sections by an air tugger mounted on the form. Although the form had access and inspection windows, the large amount of overbreak in the tunnel enabled access inside the forms for concrete placement and consolidation.

Consolidation of the wall and crown areas was done with 6-inch and 3-inch, pneumatic, immersion vibrators as well as external form vibrators.

Adits

The adits in the abutments were excavated under an earlier contract. Invert paving and shotcrete lining were added under the dam contract.

Concrete in the left adit invert was placed by laborers pushing buggies from the portal. This was so difficult that the contractor changed the method for the right abutment. Concrete in the right abutment was placed by a 6-inch pump through a 4-inch slickline. It had ¾-inch maximum size aggregate. The left adit required 167 cubic yards and the right adit 290 cubic yards of concrete.

After placement of the inverts, shotcrete was applied to the walls and crown. A dry mix was used with water being added at the nozzle. The pot was positioned at the portal and was fed by transit mix trucks. As much as 900 feet of hose was required to reach the ends of the adits.

Completion of Outlet Works Intake Structure

Contract work for the completion of the outlet works intake structure was started on July 26, 1972.

In order to place concrete in the intake tower plug, maintain the required water level upstream of the interim dam, and provide water release facilities during the winter of 1972–73, it was necessary to install 30-

inch pipe extensions on the two low-level, 30-inch, slide gates and a 10-inch pipe extension on the low-level, 10-inch, butterfly valve.

One interim slide gate was not provided with sealing surfaces and a 1-inch bolt had been left under the gate during a previous contract. As the gate was under approximately 30 feet of head, partial sealing was accomplished from the outside with gravel and cottonseed hulls. The remaining leakage was collected in a 6-inch pipe, which was grouted full after completion of the plug.

During the period when the contractor was placing concrete for the intake plug, he completed the installation of the vertical trashracks, checked and prepared the horizontal trashrack for proper fit, and installed the seals for an 18-foot-diameter dished head.

The other 30-inch slide gates remained in operation until May 8, 1973 and then were grouted full. The 10-inch pipe was extended through the diversion tunnel with a 12-inch pipe to allow the contractor to complete valve chamber work while maintaining a minimum flow of 10 cfs in Piru Creek. The 18-foot-diameter dished head was installed on the intake tower on May 8, 1973 to permit the reservoir level to rise above the intake tower.

During the summer and fall of 1973, the 10-inch butterfly valve was operated to maintain minimum flows in Piru Creek. On December 11, 1973, the 10-inch butterfly valve was found inoperative and could not be closed. An attempt was made to seal the valve from the outside with a steel cap fitted with a rubber sealing surface. This was done by divers at a depth of 223 feet. Due to poor visibility and extreme depth, the sealing was unsuccessful. It was essential that the pipe be sealed and the dished head removed from the tower to provide discharge capabilities through the outlet works.

On January 7, 1974, equipment and materials were moved in for grouting the 10-inch pipe. The next day, the grouting pipe was installed and the 10-inch pipe grouted with an expansive cement-pozzolan slurry. On January 9, 1974, the inside 10-inch valve was opened and the pipe was found to have only minor leakage. The valve was removed, a blind flange installed, and the concrete blockout and cleanup completed on January 11, 1974.

On January 17, 1974, the equalizer plug on the 18-foot-diameter dished head was pulled and the dished head removed from the intake tower. On January 18, 1974, the horizontal trashrack was lowered onto the intake tower. During lowering of the trashrack, the cable sling became entangled under the trashrack, preventing seating. Divers completed the seating of the trashrack on January 19, 1974.

Diversion Tunnel Plug

On June 6, 1973, the contractor installed a concrete chipping machine (scabbler) in the diversion tunnel plug area to remove the required 1 inch of surface concrete. This machine was equipped with a rotating arm, a scabbler unit on each end of the arm, and seven vibrating scabbler heads in each unit. The machine was self-centering on supporting wheels and was air- and hydraulically operated. Considerable adjustment and changes were required before the scabbler became fully operational. The machine did an extremely effective job of providing a rough irregular surface for the plug concrete. Areas in the valve chamber and plug that could not be reached with the scabbler were bush-hammered by hand. On completion of the concrete chipping, the piping and reinforcing steel were installed in the plug.

The first concrete was placed in the tunnel plug on August 7, 1973. The diversion tunnel was too small for concrete trucks so permission was given to pump concrete from the diversion tunnel entrance to the plug and valve chamber area. The maximum pumping distance was 830 feet with a vertical lift of 39 feet. The pumping was done by a concrete pump through a 6-inch slickline. The mix used had 1½-inch maximum size aggregate and contained 400 pounds of cement and 70 pounds of pozzolan. This mix was pumped satisfactorily with a slump of 4 to 5 inches at the pump. The slump loss in the slickline was 1 inch. Vibration of the concrete was effectively handled with 6-inch vibrators. Concrete was obtained from a plant located near the work site until November 15, 1973. After that, concrete was obtained from a supplier at Castaic. Concrete was mixed and transported to the job site by transit mix trucks.

Total concrete placement in the tunnel plug and valve chamber was 898 cubic yards.

Mechanical and Electrical Installations

The contractor moved the westerly 42-inch plug valve into the valve chamber on October 27, 1973. Transporting of the valve through the diversion tunnel was accomplished with a four-wheeled cart with wheels sloped to conform to the curvature of the tunnel. The cart was pulled into the chamber by an electric winch. Rock bolts with lifting eyes were installed at the center of the tunnel just ahead of the valve deck, and four 10-ton chain hoists were used to lift the valve above the deck elevation. Steel I-beams were installed under the valve, four sets of wheels were placed under the valve, and the valve was transported along the beams to the deck. The valves then were jacked into position over the anchor plates and lowered. All valves were transported in a similar manner.

Installation of the 78-inch valve was delayed because the contractor's proposed anchorage was not satisfactory, and considerable revision for the valve and operator was required to prevent uplift during valve stroking. The 78-inch valve was moved into the tunnel on November 10, 1973, and final positioning was completed on December 5, 1973.

The piping for the tunnel plug and valve chamber was fabricated and hydrotested to 275 psi in position or outside the tunnel, depending on the pipe size, length, and installation problems. This was accomplished in two phases: the upstream inlet sections ahead of the valves and the downstream discharge sections. This procedure tested all the field welds except the nipple weld to the pipe flanges at the valves. The Department accepted radiographing of these welds in lieu of the specified hydrotest. All welds were ultrasonically tested prior to hydrotesting or radiographing. All welding was satisfactory except about 6 inches of longitudinal factory weld just upstream of the 78-inch valve which was satisfactorily repaired.

Installation of the fixed-cone dispersion valves started as soon as the downstream sections of pipe were installed and aligned. Piezometer piping was installed as main piping was completed.

In the Pyramid Dam outlet works, epoxy coating was applied to all metalwork and valves in the dispersion chamber, to the interior of all piping 4 inches and larger, to the sump pumps and appurtenances, and to all exterior ferrous and galvanized surfaces exposed to water. Exposed ferrous metalwork and galvanized piping within the valve chamber which was not exposed to water was primed with a red-lead alkyd and finished with machinery enamel.

Coatings generally were not applied until all major work items had been completed. This procedure provided a better-appearing finished product and eliminated considerable touchup work although it did require additional surface preparation where metalwork had become rusted.

Miscellaneous mechanical work included the air ventilation fan and duct systems, and sump pumps in the valve chamber. With the exception of the sump pumps, the work was performed without difficulty.

The sump pumps are operated by water-level pressure switches and were found to be extremely difficult to adjust and maintain. The pump transfer relay did not function correctly and was replaced.

Electrical installation for the Pyramid outlet works included the embedded conduit in the valve chamber, surface runs in the valve chamber and diversion tunnel, electrical-duct shaft, completion of the conduit runs to the spillway motor control center, control cabinets, and all devices required to complete the electrical installation. Electrical work was completed as equipment was installed and made ready for operation.

Instrumentation

All 21 piezometers functioned properly for a period after installation but, by July 1973, all those above elevation 2,300 feet had failed. The plastic tubing to all of these tips contained vertical runs, and it is surmised that the riser tubing failed in the vicinity of elevation 2,260 feet due to high consolidation of the impervious material in this area. The slope-indicator data showed that embankment settlement in this area was approximately 6 feet.

Open-tube piezometers were installed in holes drilled from the crest of the Dam to the inoperative piezometers. The drilling fluid in these holes evidently caused temporary hydraulic fracturing of the core, and some of the drilling fluid was lost. The fracturing was determined to be possible because the effective overburden pressure in the core had been reduced by the stiffer transition and shell zones carrying the weight of the core. Extensive exploration and supplemental analyses showed that there was no permanent affect on the Dam. Performance of the Dam since then has verified this conclusion. A finite element stress analysis is being conducted to check the details of the estimated stress conditions.

The slope-indicator installations were comprised of 5-foot sections of extruded aluminum tubing approximately 3 inches in diameter. The tubing has four tracking grooves which guide the slope-indicating device. It was found that the grooves in the tubing had a slight twist which caused a rotation in the grooves as the individual sections of tubing were added. When this was discovered, slope indicator No. 3 had rotated over 8 degrees. At this time, measures were taken to correct this and eliminate future rotation. This was done by using a spanner wrench to twist the added section of tubing in the desired direction while it was riveted to the previously installed section. This proved successful and, thereafter, installations were held within the prescribed tolerance of plus or minus 3 degrees of the specified orientation. Another problem was that the settlement of the embankment tended to move slope indicators Nos. 2 and 4 away from the dam abutment. This required correction as each section was added. A great deal of survey crew time was required to monitor the orientation and location of the slope indicators.

BIBLIOGRAPHY

Marachi, N. D., Chan, C.K., and Seed, H. B., "Evaluation of Rockfill Materials", ASCE Journal of the Soil Mechanics and Foundation Division, January 1972.

Stroppini, E. W., Babbitt, D. H., and Struckmeyer, H. E., "Foundation Treatment for Embankment Dams on Rock", ASCE Journal of the Soil Mechanics and Foundation Division, October 1972.

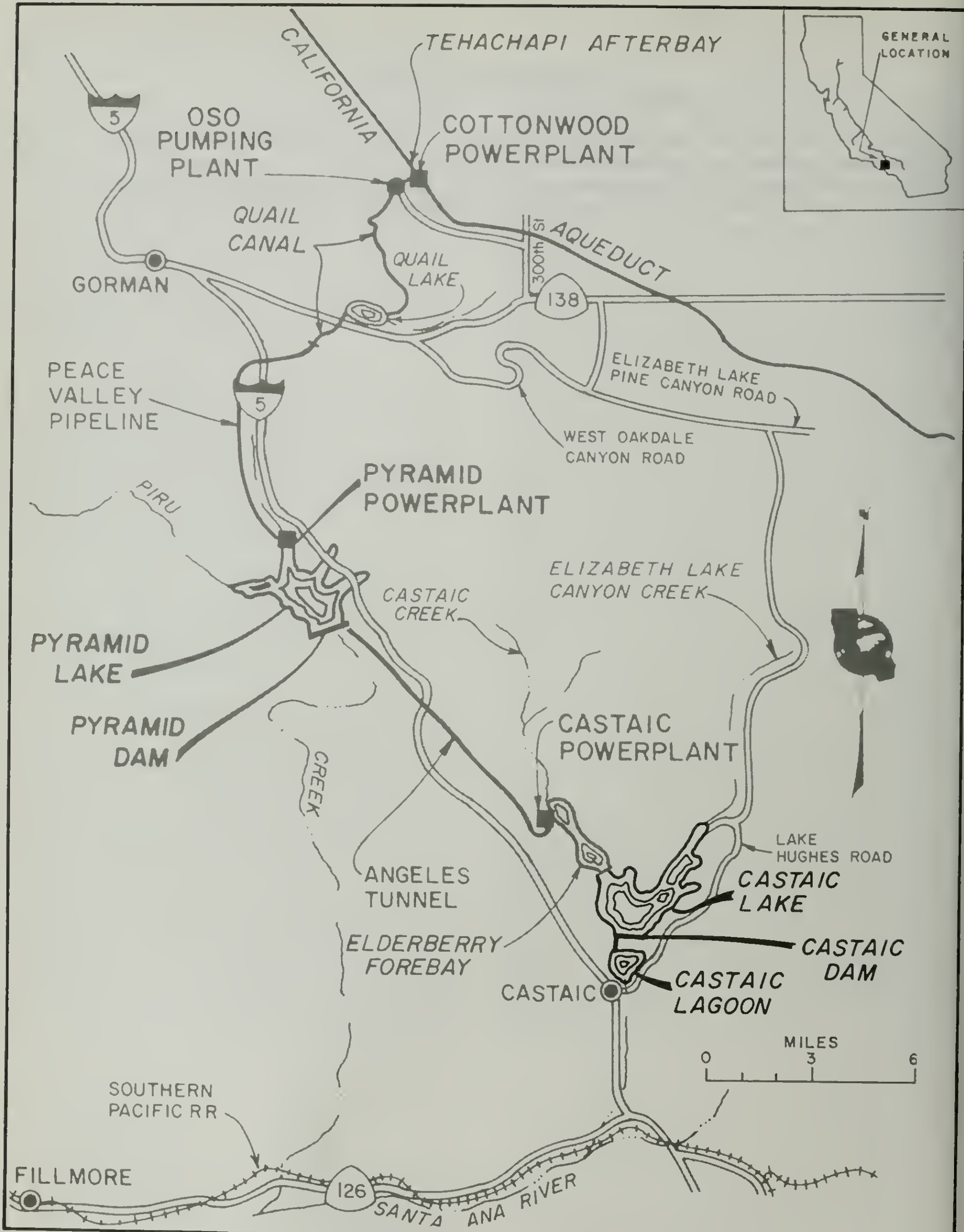

Figure 335. Location Map—Castaic Dam and Lake

CHAPTER XV. CASTAIC DAM AND LAKE

General

Description and Location

Castaic Dam rises 425 feet above streambed excavation and spans 4,900 feet between abutments at its crest. The 46,000,000-cubic-yard embankment is made up of a central impervious core flanked by pervious shells with appropriate transition zones.

The spillway is located at the right abutment of the Dam and consists of an unlined approach channel; a 360-foot-wide, ungated, ogee weir; and a 5,300-foot-long lined chute with energy dissipator.

The outlet works provides delivery of water through a 19-foot-diameter penstock installed inside the 27-foot-diameter, former, diversion tunnel. Downstream control is provided through a valving complex for stream releases up to 6,000 cubic feet per second (cfs), delivery to water users up to 3,788 cfs, and reservoir emergency releases. Upstream control is provided by a multiple-level, high, intake tower equipped with 72-inch butterfly valves.

Elderberry Forebay located at the upper end of, and separated from the right arm of, Castaic Lake provides regulatory storage for Castaic Powerplant.

Downstream of the Dam, Castaic Lagoon, a former borrow area, now serves as a recreation area and a recharge basin.

Castaic Dam and Lake are located about 45 miles northwest of Los Angeles and about 2 miles north of the community of Castaic at the confluence of Castaic Creek and Elizabeth Lake Canyon Creek (Figures 335, 336, and 337). Elderberry Forebay Dam (owned and operated by Los Angeles Department of Water and Power, LADWP) is located 3½ miles upstream of Castaic Dam on Castaic Creek. The weir that contains Castaic Lagoon is located 1½ miles downstream of Castaic Dam where Lake Hughes Road crosses Castaic Creek. The nearest major highway is Interstate Highway 5, about 2 miles to the west.

Figure 336. Aerial View—Castaic Dam and Lake

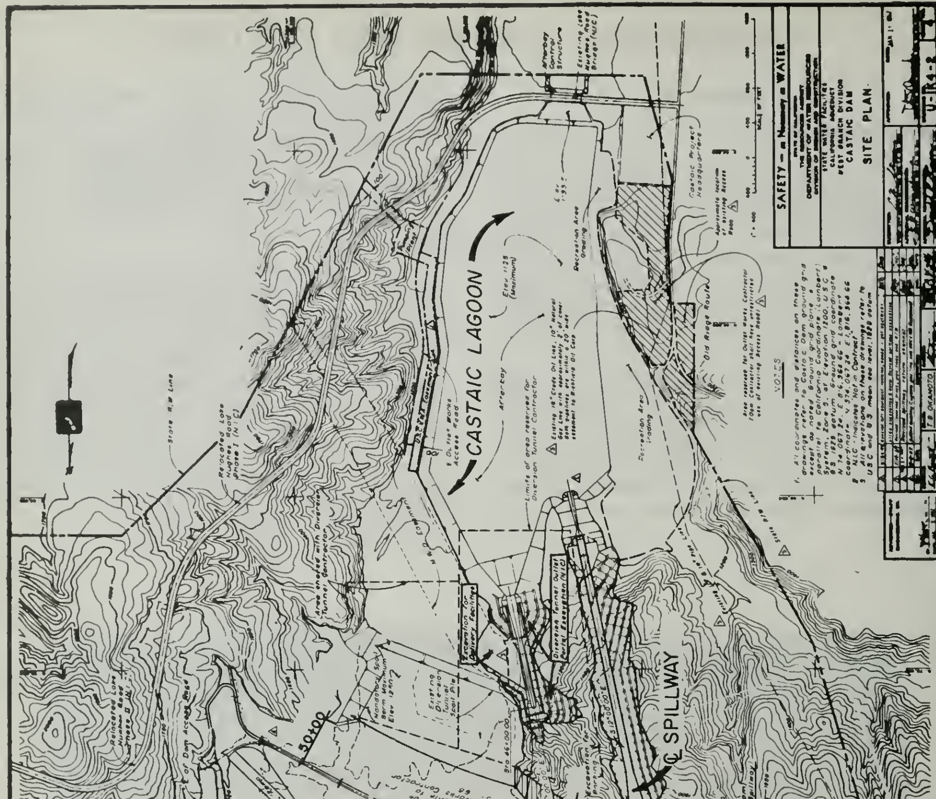

Figure 337. Site Plan

Purpose

Castaic Lake was built to accomplish the following: (1) provide emergency storage in the event of a shutdown of the California Aqueduct to the north, assuring water deliveries to the West Branch water users; (2) act as regulatory storage for deliveries during normal operation; and (3) provide a setting for recreational development by state and local agencies for the Southern California area. Although flood control is not a primary purpose, inflows of up to 61,000 cfs will be reduced to the capacity of the downstream channel.

Elderberry Forebay serves three purposes: (1) provides 18,000 acre-feet of live storage which can be utilized by Castaic Powerplant during off-peak hours for pumpback into Pyramid Lake, (2) provides submergence for the pump-generator when Castaic Lake is at its lower operating levels, and (3) reduces daily and weekly fluctuations in Castaic Lake.

Castaic Lagoon originally was a borrow area for the construction of Castaic Dam. Now, its purposes are (1) to provide a recreation pool with a water surface at a constant elevation of 1,134 feet, and (2) to function as a recharge basin for the downstream ground water basin.

Chronology

Studies by the Department of Water Resources indicated that storage of any appreciable amount at the terminus of the West Branch Aqueduct would be provided most logically and economically at the Castaic Dam site. Planned storage capacity of Castaic Lake has varied from 150,000 acre-feet in 1959, to 370,000 acre-feet in 1960, to 100,000 acre-feet in 1961, and to 350,000 acre-feet in 1963. The final decision to increase the capacity of Castaic Lake from 100,000 acre-feet to 350,000 acre-feet was made to utilize the optimum capability of the site in providing terminal storage for regulatory and emergency requirements. With the construction of Elderberry Forebay by LADWP, storage in Castaic Lake was reduced to 323,702 acre-feet.

Final design was started in January 1964, excavation was started in August 1965 in a foundation trench, and the completion contract was finished in June 1974. Statistical summaries of Elderberry Forebay Dam and Forebay and of Castaic Dam and Lake are shown in Tables 42 and 43, respectively. The area-capacity curves are shown on Figure 338.

TABLE 42. Statistical Summary of Elderberry Forebay Dam and Forebay

ELDERBERRY FOREBAY DAM

Type: Zoned earthfill

Crest elevation	1,550 feet
Crest width	25 feet
Crest length	1,990 feet
Streambed elevation at dam axis	1,370 feet
Lowest foundation elevation	1,350 feet
Structural height above foundation	200 feet
Embankment volume	6,000,000 cubic yards
Freeboard above spillway crest	20 feet
Freeboard, maximum operating surface	10 feet

ELDERBERRY FOREBAY

Maximum operating storage*	33,004 acre-feet
Normal maximum operating storage	28,231 acre-feet
Minimum operating storage	19,041 acre-feet
Dead pool storage	811 acre-feet
Maximum operating surface elevation	1,540 feet
Normal maximum operating surface elevation	1,530 feet
Minimum operating surface elevation	1,480 feet
Dead pool surface elevation	1,412 feet
Shoreline, spillway crest elevation	7 miles
Surface area, maximum operating elevation	492 acres
Surface area, spillway crest elevation	460 acres
Surface area, minimum operating elevation	379 acres

* Storage above elevation 1,530 feet to be utilized only during the months of May and June when additional storage on the California Aqueduct may be required.

SPILLWAY

Emergency: Ungated ogee crest with lined channel, discharge into draw

Crest elevation	1,540 feet
Crest length	420 feet

Service: Glory hole with reinforced-concrete conduit and stilling basin—10-foot-high stoplogs provided at crest

Top of stoplogs	1,540 feet
Crest elevation	1,530 feet
Crest length, elevation 1,540 feet	173 feet
Crest length, elevation 1,530 feet	126 feet
Crest diameter	54.9 feet
Conduit diameter	21 feet

Combined spillways: No stoplogs in service spillway

One-in-1,000-year-flood inflow	28,747 cubic feet per second
Outflow with 5 feet of freeboard	28,747 cubic feet per second

INLET-OUTLET

Castaic Powerplant tailrace

Maximum generating release	18,400 cubic feet per second
Pumping capacity	17,300 cubic feet per second

OUTLET WORKS

Type: High-level, spillway conduit beneath dam along base of right abutment; low-level, reinforced-concrete conduit with valve chamber adjacent to glory-hole spillway—discharge into spillway conduit downstream of elbow

Diameter: High-level, 21 feet—low-level, 7 feet

Intake structures: High-level, slide gates on spillway shaft; low-level, uncontrolled box with stoplog emergency bulkhead

Control: High-level, two 8-foot-wide by 9-foot-high slide gates at elevation 1,498 and six 8-foot-wide by 12-foot-high slide gates at elevation 1,477 on spillway shaft; low-level, single set of two 5-foot-wide by 6-foot-high, high-pressure, slide gates in tandem within gate chamber

Capacity	17,000 cubic feet per second

TABLE 43. Statistical Summary of Castaic Dam and Lake

CASTAIC DAM

Type: Zoned earthfill

Crest elevation	1,535 feet
Crest width	40 feet
Crest length	4,900 feet
Streambed elevation at dam axis	1,200 feet
Lowest foundation elevation	1,110 feet
Structural height above foundation	425 feet
Embankment volume	46,000,000 cubic yards
Freeboard above spillway crest	20 feet
Freeboard, maximum operating surface	20 feet
Freeboard, maximum probable flood	5 feet

CASTAIC LAKE

Maximum operating storage	323,702 acre-feet
Minimum operating storage	18,590 acre-feet
Dead pool storage	18,590 acre-feet
Maximum operating surface elevation	1,515 feet
Minimum operating surface elevation	1,280 feet
Dead pool surface elevation	1,280 feet
Shoreline, maximum operating elevation	29 miles
Surface area, maximum operating elevation	2,235 acres
Surface area, minimum operating elevation	372 acres

SPILLWAY

Type: Ungated ogee crest with lined chute and stilling basin

Crest elevation	1,515 feet
Crest length	360 feet
Maximum probable flood inflow	120,000 cubic feet per second
Peak routed outflow	78,400 cubic feet per second
Maximum surface elevation	1,530 feet
One-in-400-year-flood inflow	61,000 cubic feet per second
Peak routed outflow	27,200 cubic feet per second
Maximum surface elevation	1,522.7 feet

INLET

Elderberry Forebay outlet Capacity	17,000 cubic feet per second

OUTLET WORKS

Type: Lined tunnel under right abutment—upstream of tunnel plug, 19-foot-diameter pressure tunnel—downstream, 19-foot-diameter steel conduit in a 27-foot-diameter tunnel to delivery manifold and stream releases

Intake structures: Low-level uncontrolled tower with provision for steel plug emergency bulkhead and 6-foot by 10-foot coaster gate shutoff at junction with high intake; high-level vertical nine-level tower with 72-inch butterfly shutoff valves

Control: Regulation by water users beyond downstream delivery manifold—stream release, series of rated valves in a structure immediately downstream of the delivery mainfold

Design deliveries	3,788 cubic feet per second
Capacity, stream maintenance	6,000 cubic feet per second
Capacity, reservoir drainage	11,000 cubic feet per second

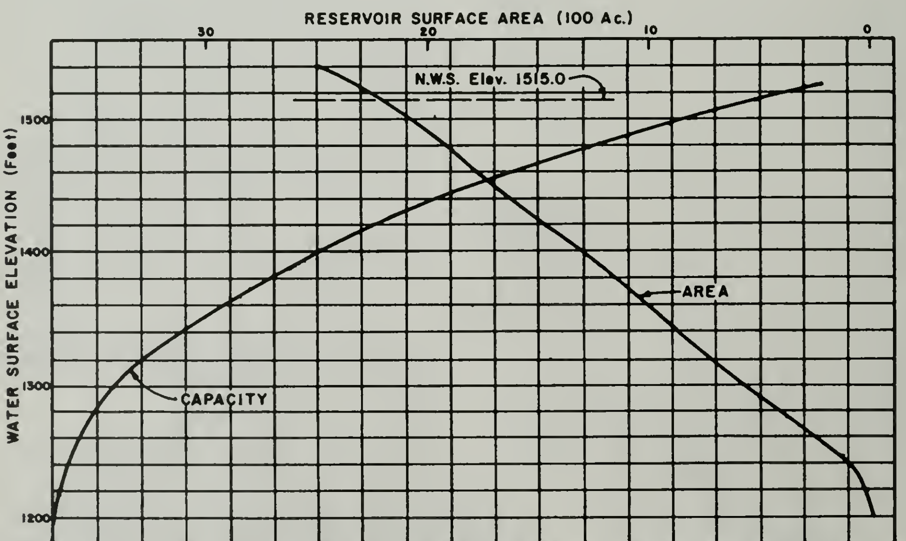

Figure 338. Area-Capacity Curves

Regional Geology and Seismicity

Castaic Dam site, approximately 10 miles southeast of Pyramid Dam, is approximately 3 miles northeast of the San Gabriel fault and 13 miles southwest of the San Andreas fault. The section on regional geology and seismology in Chapter XIV of this volume applies to Castaic Dam as well as Pyramid Dam.

Design of Elderberry Forebay

Elderberry Forebay was designed and constructed by the City of Los Angeles Department of Water and Power.

Operation

Castaic Powerplant, located between Pyramid Lake and Elderberry Forebay, generates during peak demand periods when the value of energy is high and pumps water back to the higher level of Pyramid Lake during off-peak periods when the value of energy is low. On a daily basis, approximately 18,000 acre-feet of water is used for power generation during the day, 8,000 acre-feet of which is pumped back at night. This leaves a daily flow of about 10,000 acre-feet through Elderberry Forebay to Castaic Lake. On weekends, a full 18,000 acre-feet of live storage from the Forebay (a 60-foot drawdown) can be pumped to Pyramid Lake using off-peak power. Then, during the week, this drawdown can be replenished with part of the daily 18,000-acre-foot flow from Pyramid Lake.

Under normal operating conditions, minimum water surface in Castaic Lake may be at about elevation 1,435 feet each year, which is 45 feet below the minimum operating surface of the Forebay. Under these conditions, the Forebay is necessary to supply submergence for the Castiac Powerplant tailrace.

Embankment

The 160-foot-high forebay embankment is a zoned earthfill with free-draining material in the upstream rapid drawdown area. The downstream section, where drawdown will be more gradual, is of pervious material protected from the wave wash of Castaic Lake by 2½ feet of soil-cement. The depth of Castaic Lake on the toe of the forebay embankment will be at least 55 feet under normal operation. A core trench is provided to sound rock under the impervious Zone 1 material.

Emergency Spillway

An emergency spillway discharges across a ridge east of the left abutment and into a draw that returns to Castaic Creek near the toe of the Forebay Dam. The spillway consists of an unlined approach channel with side slopes protected by riprap; a 4-foot-high, ungated, ogee crest; and a concrete-lined chute.

Outlet Works

The outlet works of the Forebay consists of a high-level and a low-level outlet.

The high-level outlet is a 64½-foot-diameter glory-hole-type intake with a 40-foot-diameter shaft. The intake has 12 stop-gate bays around its crest and slide gates at two levels below the crest. Closing of the stop gates on the crest will raise the Forebay 10 feet, to the level of the emergency spillway crest. Below the slide gates and above the elbow of the outlet, the diameter of the shaft reduces from 40 to 21 feet. At the toe of the Forebay Dam, the 21-foot pipe discharges into a transition to a flume which empties into a concrete-lined stilling basin. Backwater from Castaic Lake stands above the top of the elbow at the base of the shaft, except in case of emergency draining of the system.

The low-level slide-gate outlet consists of an intake structure with a trashrack; a 10-foot-diameter pressure conduit; a gate chamber containing two 5- by 6-foot, motor-driven, remotely operated, pressure gates; and a 7-foot-diameter outlet pipe that discharges into the 21-foot-diameter, high-level, outlet discharge line.

Design of Castaic Dam

Diversion Tunnel

The diversion tunnel for Castaic Dam served to divert floodflows during construction of the embankment and now serves as part of the outlet works. The diversion tunnel passes through the right abutment under the embankment. The finished tunnel is a circular concrete-lined section, 19 feet in diameter from intake to Station 25+90 and 27 feet in diameter from that point to the outlet. The total length of the tunnel is 3,766 feet (Figures 339 and 340).

A reinforced-concrete channel extends from the diversion tunnel outlet portal to an energy dissipator. The channel and dissipator were utilized to pass floodflows during construction. Later, the channel was modified to meet the needs for turnouts and stream release facilities. The stilling basin was designed for use with the stream release facilities as well as the diversion tunnel.

The diversion tunnel contract provided for construction of the diversion facilities plus the necessary provisions for their later incorporation with the delivery and stream release facilities. The upstream portal structure included a foundation pad for a low intake tower and a tower barrel to an elevation above the diversion tunnel soffit. At Station 20+18, provisions were made for a high-level intake. The downstream facilities consisted of a concrete channel with a semicircular to rectangular transition section, a rectangular channel, and a hydraulic-jump energy dissipator (stilling basin) at the downstream end. The downstream structure also included provisions for a delivery penstock and was designed to serve as the foundation for the delivery branch bifurcations and stream release structure.

Hydraulics. The diversion tunnel, as designed and constructed, could have passed the standard project flood, which has a peak inflow of 58,000 cfs, with a maximum reservoir water surface elevation of 1,306 feet and a peak discharge of 14,600 cfs. The diameter

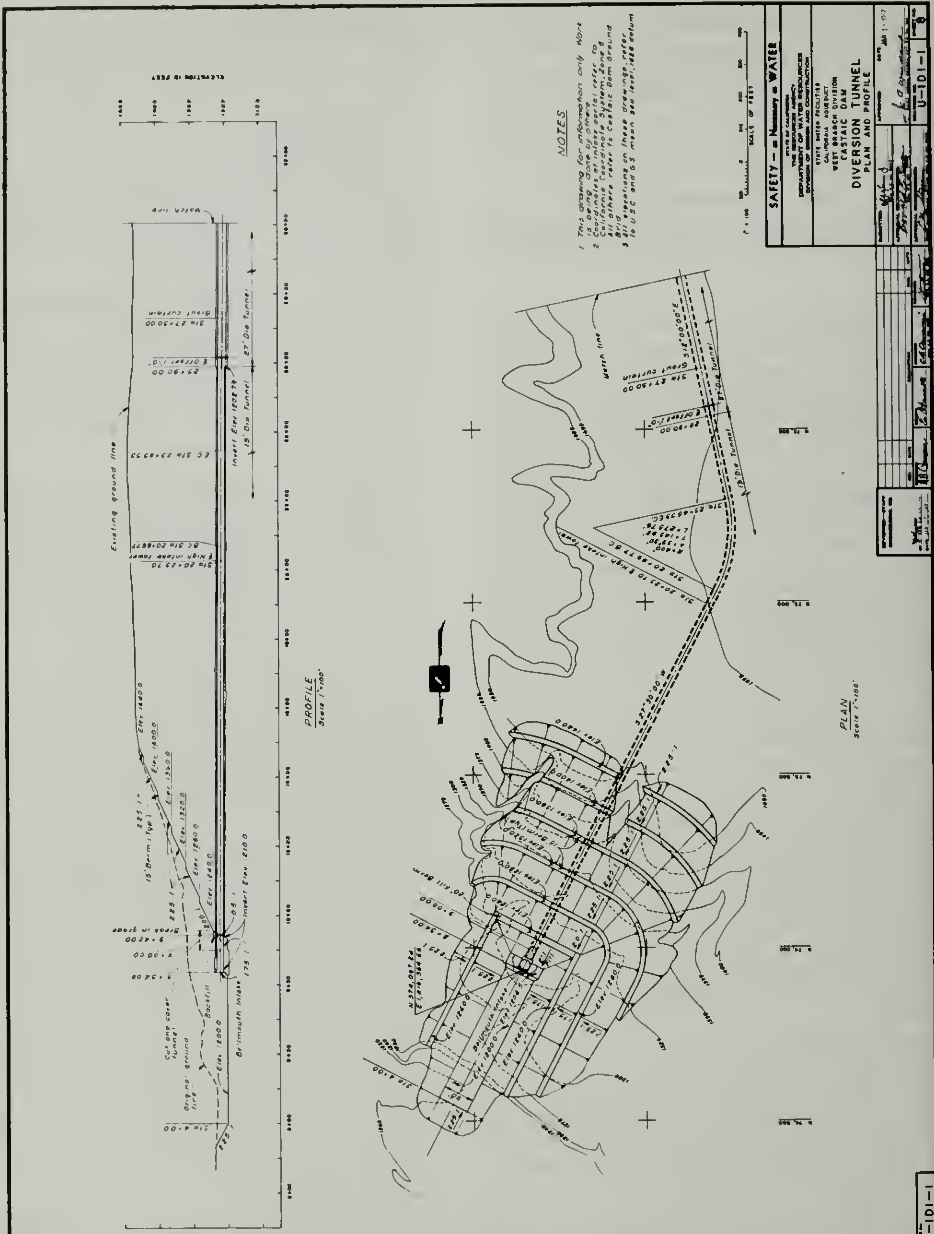

Figure 339. Diversion Tunnel—Plan and Profile

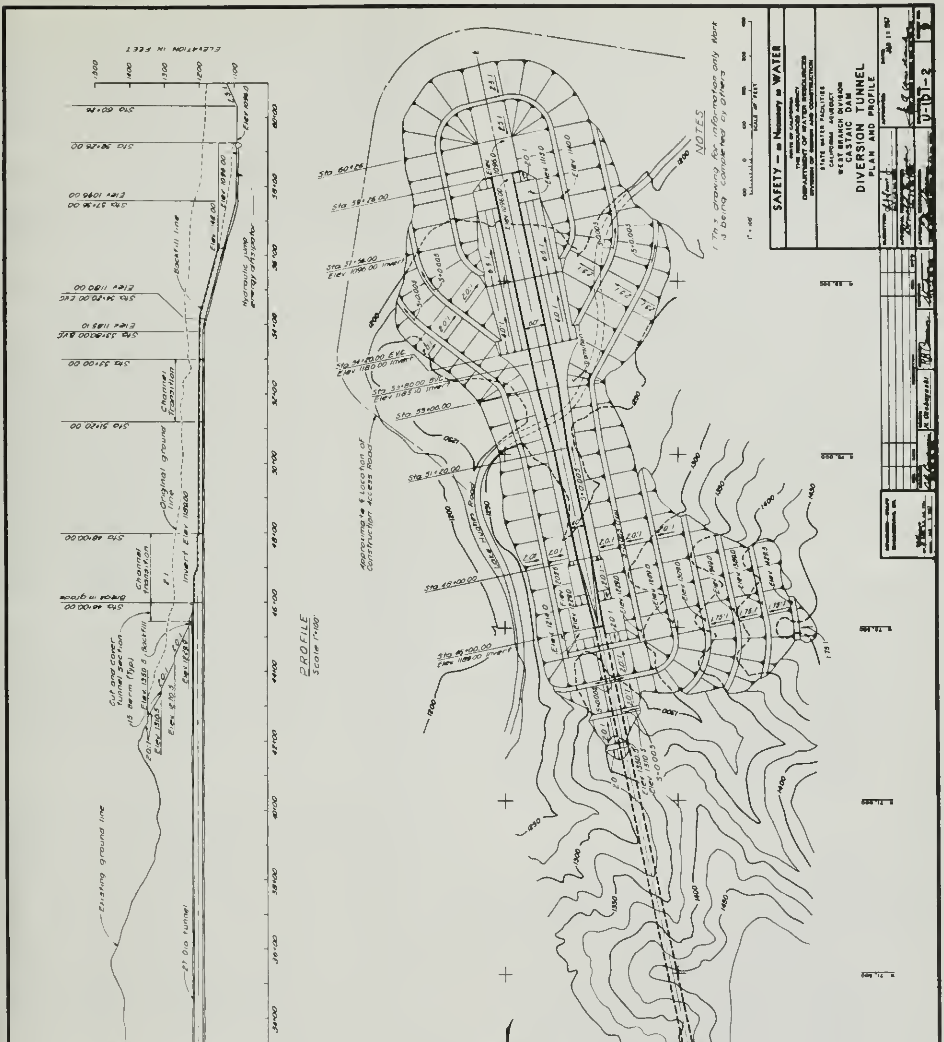

Figure 340. Diversion Tunnel—Plan and Profile (Continued)

of the tunnel was determined by the hydraulic criteria of delivering a maximum flow of 3,788 cfs to the water users with the reservoir water surface elevation at 1,421 feet and a hydraulic gradeline elevation of 1,400 feet at The Metropolitan Water District of Southern California/Department of Water Resources delivery point. With the reservoir water surface at elevation 1,380 feet, the tunnel and related system have to discharge downstream releases of up to 6,000 cfs, plus delivery to water users of 1,600 cfs. It also was necessary to design the shaft-tunnel intersection to permit 1969–70 floodwaters to pass unobstructed through a partially completed intersection structure. With the stream release facilities installed, maximum release is reduced to approximately 8,000 cfs (water surface elevation 1,515 feet). In case of emergency, two fixed-cone dispersion valves can be removed from the stream release facilities increasing the capacity to approximately 11,000 cfs.

The energy dissipator at the end of the channel chute was designed to control downstream erosion. Energy is dissipated by use of a hydraulic jump, which mainly is controlled by downstream backwater conditions. Chute blocks and a dentated sill were added as further aids to stabilize and contain the hydraulic jump. Design of these facilities was verified by model studies.

The dissipator structure was sized to operate with a discharge of 20,000 cfs. Final design of the outlet works, which was completed after the stilling basin was constructed, lowered maximum discharge to 14,600 cfs during diversion and approximately 11,000 cfs upon completion of the work. Tailwater for the jump is controlled by a downstream weir (elevation 1,136 feet) located at the new Lake Hughes Road crossing. The dissipator floor was set at elevation 1,090 feet to provide the necessary tailwater for the development of a hydraulic jump and to prevent sweep-out.

Structural Design. It was anticipated that the entire length of the diversion tunnel would require structural-steel support. Due to the varying nature of material encountered, support loading varied. Support of the tunnel, except in designated reaches, was the responsibility of the contractor. A minimum support was designated for a short reach at each portal. Special rib placement and shapes were required for the high intake-shaft intersection supports. An invert strut was designed for use through reaches of material exhibiting lateral yielding tendencies. An umbrella of structural steel was designed for both intake and outlet portals to ensure stability of the cut faces and to provide safe working areas. The design also included grouted crown bars installed around the soffit of tunnel portal excavations to tie the umbrella structure to the portal face and to provide overhead protection during initial rounds of excavation. The upstream structural-steel umbrella was incorporated into the section of cut-and-cover tunnel required between the low intake tower and tunnel portal.

Cut slopes at the portals were designed to be no steeper than 1.67:1, with 15-foot-wide berms at elevation intervals of 40 feet. The resulting minimum equivalent slope for cuts involving one or more benches was approximately 2:1. Cut slopes were determined by the following criteria:

1. Stability analyses by the circular arc and/or sliding wedge methods.
2. Comparison of design cut slopes with stable natural slopes in similar material in the immediate vicinity.
3. Design slopes parallel to or flatter than the average bedding plane of the formation in the cut.

The tunnel's concrete lining was divided into three reaches, each with different loading conditions: (1) from upstream portal to high intake-shaft intersection, loads are hydrostatic head to ground surface and dead load; (2) from high intake-shaft intersection to Station 25+90, loads are hydrostatic head to reservoir normal water surface and dead load; and (3) from Station 25+90 to downstream portal, loads are hydrostatic head to ground surface and dead load (Figure 339). Rock load was taken as a uniform pressure around the concrete lining of one bore diameter of material. Concrete lining also was designed to withstand internal pressures to hydraulic gradeline during maximum diversion flow.

The reach of diversion tunnel in the vicinity of the high intake-shaft intersection (Figure 341) was designed to constitute the initial stage of the shaft-tunnel intersection, which was completed under the outlet works contract. No reinforcement was placed in the areas where concrete was to be removed at a later time.

The tunnel intake structure (Figure 342) was designed to function both as a diversion during dam construction and later as a base for the low-level intake tower in the outlet works. The tower base was located a sufficient distance from the portal to be relatively free of slides and be a part of the portal approach floor. The critical design forces (hydrostatic and earthquake) on the tower were considered of sufficient magnitude to require the use of foundation piling around the perimeter of the tower base. Piling was designed for tension with the bearing floor serving as a pile cap and tie. A pile friction of 500 pounds per square foot was used.

Primary concern for the channel section was that it carry the standard project flood. However, to accommodate the future stream release facility and to provide room for penstock valving, channel width was increased from 40 to 60 feet downstream at the bifurcation. The cantilever walls of the transitions, channel, and chute sections were designed to withstand level backfill within 2 feet of the top. Earthquake loading of 0.1g was added. The structure was provided with a side drainage system.

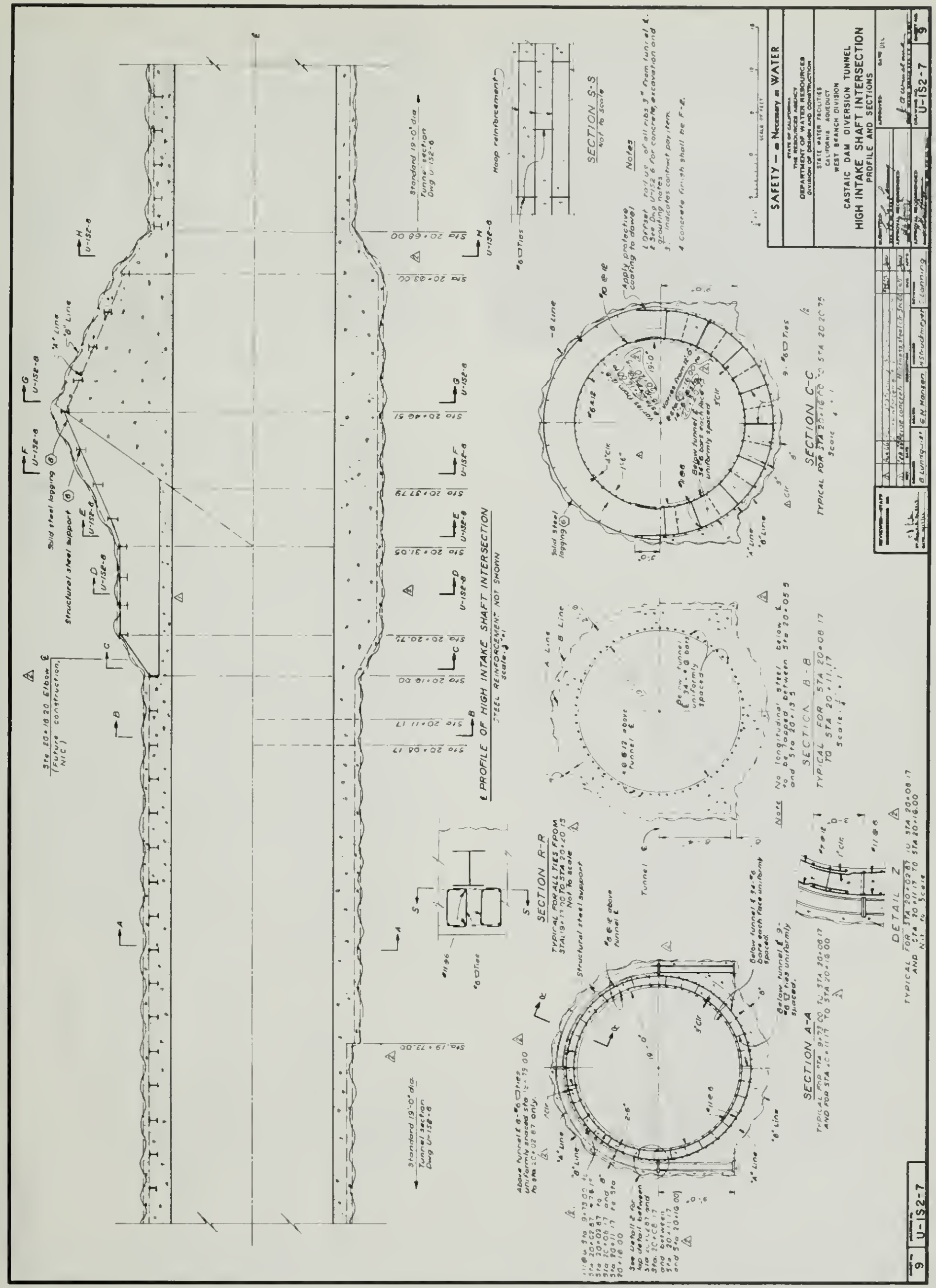

Figure 341. High Intake-Shaft Intersection

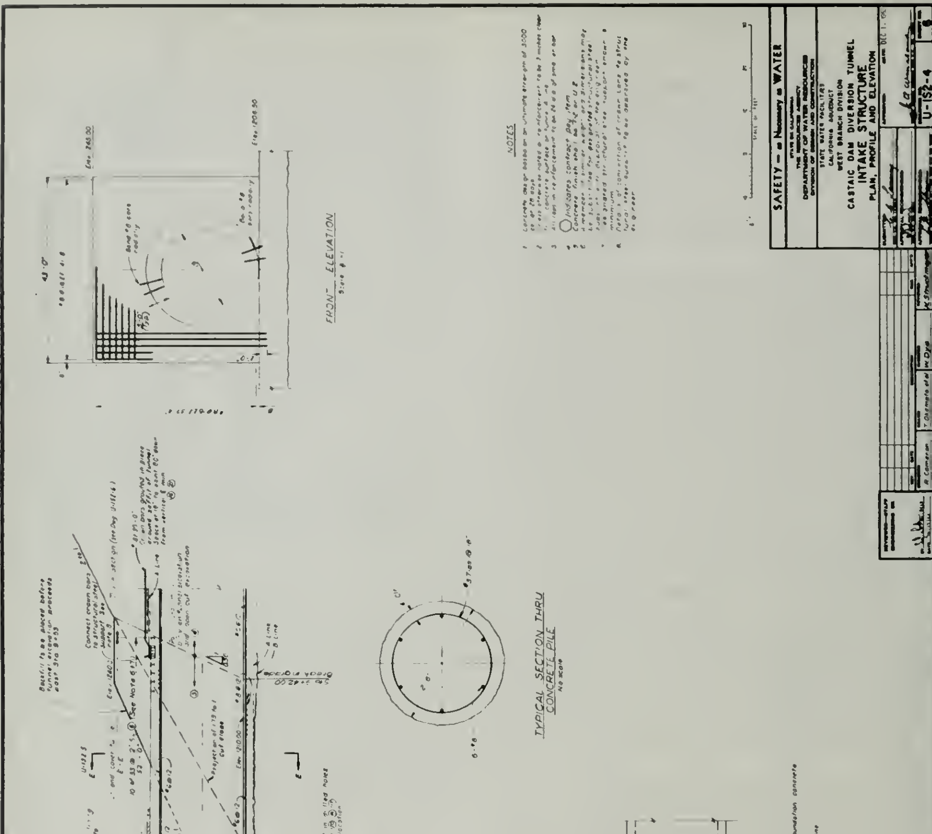

Figure 342. Diversion Tunnel Intake Structure

Although counterforts generally are more economical for walls of the height used for the energy dissipator, a gravity structure was designed to increase stability. This was achieved by designing the walls as tapered cantilevers with thickened bases. The additional concrete required for this design increased the weight of the structure and produced the desired stability (Figure 343).

Grouting. The design grouting program for the tunnel called for a grout curtain, consolidation grouting, and contact grouting. The grout curtain consisted of two rings of holes from 80 to 150 feet deep located near the dam axis so as to mesh with the dam grout curtain. Consolidation grouting consisted of rings of six holes 20 feet deep. Contact grouting was required throughout the length of the tunnel.

Figure 343. Outlet Works Energy Dissipator

Embankment

Description. The basic embankment section consists of a central impervious core flanked by pervious shells and a zone of random material contained within the downstream pervious shell. Internal embankment drainage is provided by an inclined drain, downstream of the impervious core, connected to a blanket drain in the channel and on the abutments up to elevation 1,450 feet. Protective filters are positioned between the core and inclined drain and between the core and upstream shell. Various details of the embankment are shown on Figures 344 and 345.

Stability Analysis. Embankment stability was analyzed by the infinite slope, sliding wedge, and Swedish Slip Circle methods of analysis. Seismic forces used in stability analyses were approximated by applying a steady horizontal force acting in the direction of instability. The steady force was assumed at 0.15 times the moist or saturated embankment weight, whichever was applicable. The design properties of the compacted embankment materials are shown in Table 44.

Design features providing protection against failure from seismically induced displacements included filters and an inclined drain at least twice as thick as would be specified for the same zoned embankment in a nonearthquake area, a substantially increased impervious core width, and plasticity requirements for the core materials. These features were intended to prevent concentrated leakage and piping along any plane of differential transverse movement. A minimum plasticity index of 7% was specified for impervious core materials, resulting in an average plasticity index of core materials in the range of 10 to 15%. It is generally accepted that the higher the plasticity index of a fine-grained soil, the greater the ability to deform without cracking and thus the higher the resistance to concentrated leakage.

An extensive drainage system in the downstream shell and in the natural sand and gravel downstream of the Dam was provided to collect core and foundation seepage and transport it to Castaic Lagoon. A weir to determine seepage quantities was provided at the seepage outfall facility adjacent to the Lagoon.

Construction Materials. The impervious core was selected from unweathered (Zone 1B) and weathered (Zone 1A) Castaic formation materials obtained from required excavation. Additional Zone 1A material was

TABLE 44. Material Design Parameters—Castaic Dam

Material	Specific Gravity	Unit Weight in Pounds Per Cubic Foot			Static Shear Strengths θ Angles in Degrees Cohesion in Tons Per Square Foot					
					Effective		Total		Construction	
		Dry	Moist	Saturated	θ	C	θ	C	θ	C
Zone 1A	2.72	113	130	135	30	0	18	0.15	20	1.5
Zone 1B	2.72	113	130	135	30	0	18	0.15	20	1.5
Zone 2A	2.70	131	140	145	38	0	*	*	*	*
Zone 2B	2.70	131	140	145	38	0	*	*	*	*
Zone 3	2.70	131	140	145	38	0	*	*	*	*
Zone 4	2.70	120	135	--	35	0	35	0	35	0

* Free-draining material, use effective stress values.

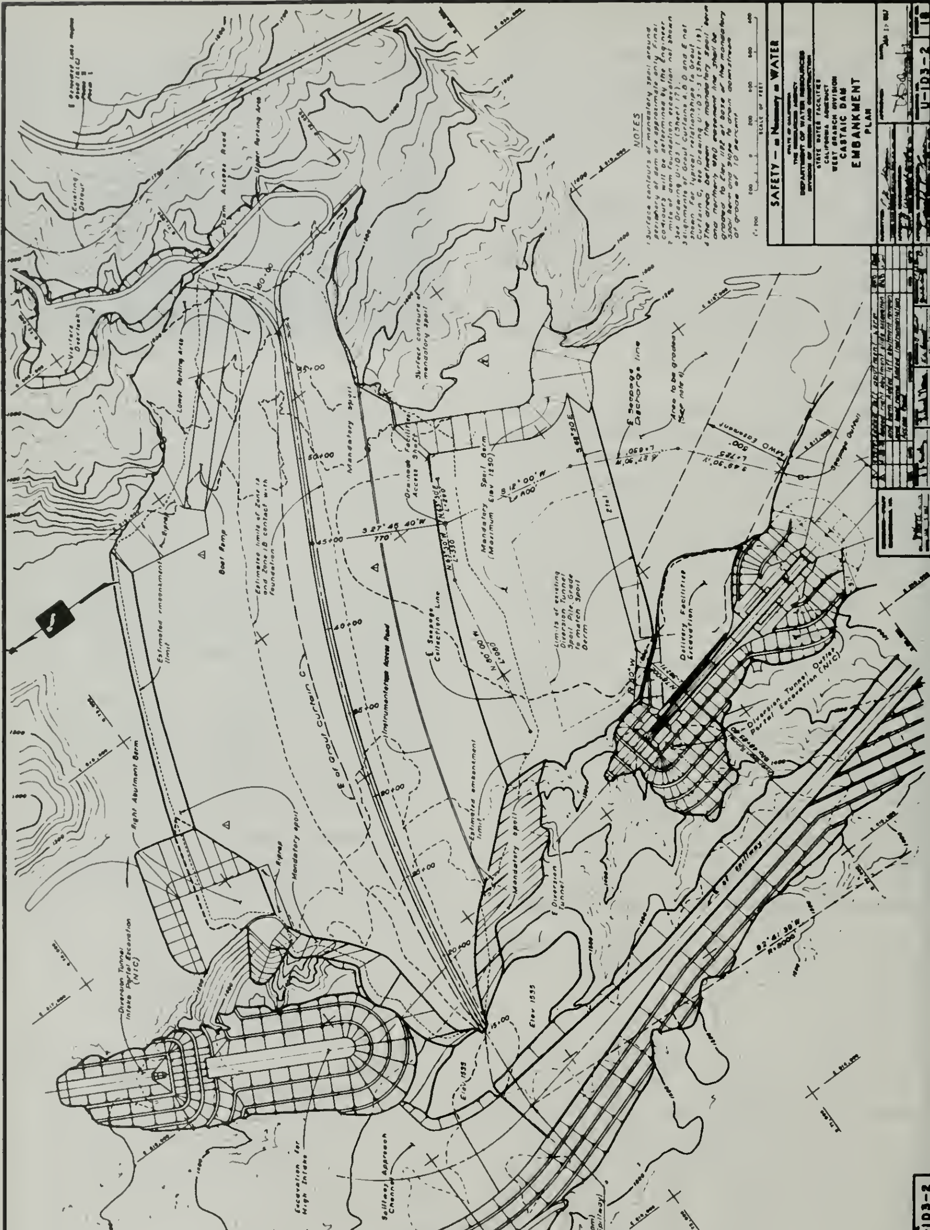

Figure 344. Embankment Plan

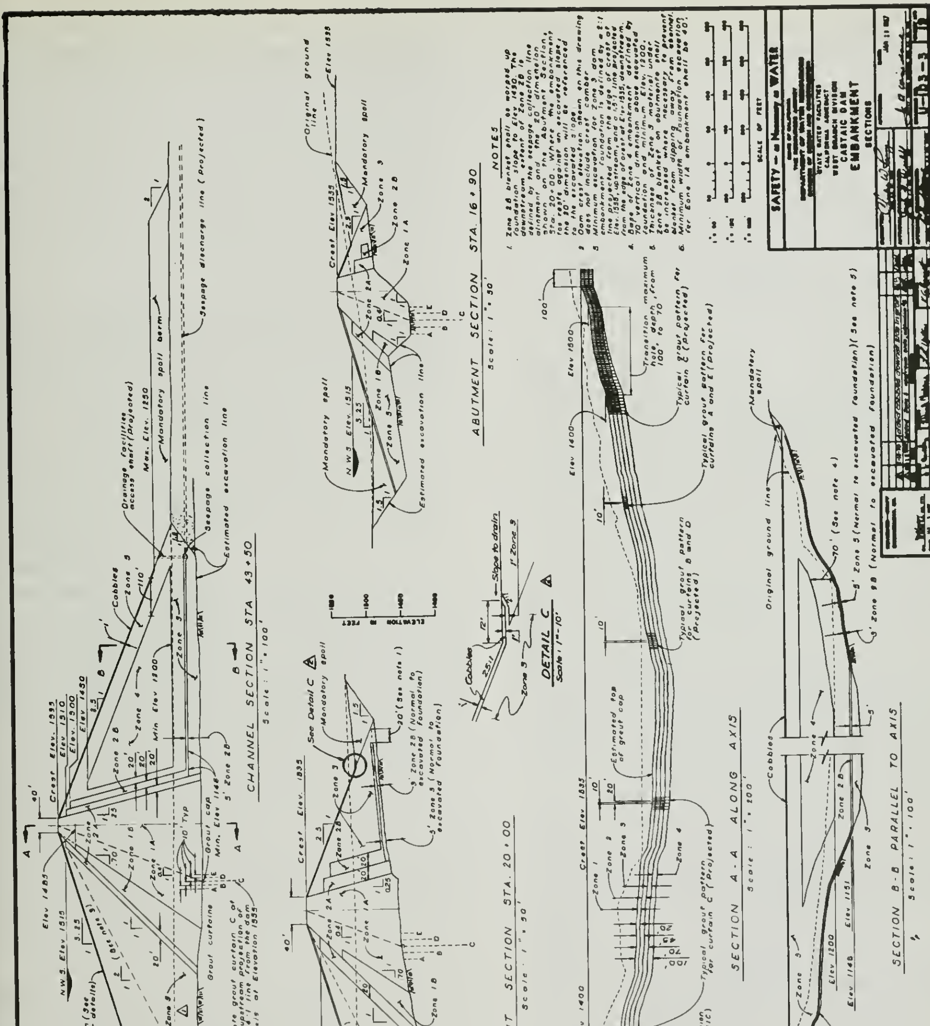

Figure 345. Embankment Sections

obtained from selected borrow areas. Transition material (Zone 2A) and drain material (Zone 2B) are processed streambed sands and gravels from required excavation and pervious borrow areas. Pervious shell material (Zone 3) is streambed sands and gravels from required excavation and pervious borrow areas. Random material (Zone 4) consists of terrace sands and gravels from required excavation and sandstones with some shale of the Castaic formation.

Soil-cement for upstream slope protection was produced by mixing cement with excess Zone 2A material. Cobbles for downstream slope protection were obtained during the processing of streambed deposits for Zones 2A and 2B.

Test Fill. A test fill was constructed at the Dam site in October 1966 to determine more adequately the reaction of Castaic formation materials to procedures being considered for construction of the impervious cores. Extensive field and laboratory testing confirmed that weathered and unweathered plastic materials from the Castaic formation would break down during excavation and compaction to form an impervious mass with a shear strength at least equal to that used in design.

Settlement. Settlement of the zoned embankment is caused primarily by consolidation of the central impervious cores. Consequently, laboratory consolidation testing and analysis concentrated on materials to be used in constructing the core. Because there is no proven analytical settlement approach known to determine that portion of core consolidation which occurs after completion of an embankment, a camber of approximately 1% of the fill height was provided to compensate for long-term embankment settlement.

Seepage Analysis. Seepage under the central impervious core is controlled by excavation for the core foundation to sound unweathered bedrock, by grout curtain construction, and by blanket grouting of fractured or sheared bedrock zones outside the area of the grout curtains. The design called for a main grout curtain 150 feet in depth, flanked by two curtains with depths of 70 feet, and two outside curtains with depths of 40 feet. Some of this grouting was deleted, as is discussed later. Seepage that does occur under the core should be concentrated in sandstone layers contained in the bedrock.

Seepage through the impervious core, based on flow-net analysis, was estimated to be less than 0.25 cfs with the reservoir at normal pool. The analysis assumed a horizontal to vertical permeability ratio of 9 for the core and infinite permeability for Zones 2A, 2B, and 3. A vertical permeability of 0.01 of a foot per day was used for the core.

Transition and Drain. The transition zones prevent movement of fine-grained core material into the surrounding pervious zones due to seepage forces, and the drain ensures drainage of the downstream shell. Materials used for the transition and drains were processed by separating sands and gravels at approximately the ½-inch particle size and using oversize and undersize materials for drain and transition zones, respectively. To satisfy recommendations of the Department's Earth Dams Consulting Board, some smaller grain sizes were blended with the oversize material to prevent segregation when placing and compacting the drain zone. Specification limits for the transition (2A) and drain (2B) materials were derived to fit the anticipated processing scheme previously described.

Upstream Slope Protection. Due to the relatively short period of wind-velocity recording and the topographic differences at the Dam site, a wind velocity of 60 miles per hour (mph) in any direction was used for a conservative determination of maximum wave height. The reservoir configuration results in a maximum fetch (length of reservoir over which wind can blow) of approximately 5 miles and an effective fetch (an equivalent length that a wave could traverse without being dampened) of 1.6 miles.

Wave run-up was investigated for a 60-mph wind, an effective fetch of 1.6 miles, and a 3¼:1 embankment slope. The vertical wave run-up on riprap and smooth (soil-cement) facings was found to be 2.4 feet and 6.1 feet, respectively. Under conditions of wave run-up and the standard project flood maximum water surface, the riprap and soil-cement-protected slopes would have a freeboard of 9.7 feet and 6.0 feet, respectively. The soil-cement facing was assumed smooth to determine an extreme wave run-up condition; actually, the stairstep method of soil-cement construction results in a corrugated surface.

A cement content of 8% by weight and a total soil-cement thickness of 2 feet was provided. To aid drainage immediately behind the facing during drawdown of the reservoir, Zone 3 material containing less than 3% by weight passing the No. 200 sieve was placed within 10 feet horizontally of the soil-cement.

The riprap alternative included in the bid documents apparently was not economically feasible and no contractor bid on that alternative.

Dam Axis Alignment Changes. Slides occurring in the east (left) abutment during foundation excavation raised concern that the scheduled deep excavations for the dam foundation at the easterly extremity of the Dam would undermine the stability of Lake Hughes Road and the ridge downstream of the Dam (Figure 344). To avoid this, the alignment of the dam axis was moved upstream by changing it from the 8,000-foot-radius curve to a line tangent to the curve running easterly at Station 45+20 (Figure 345). To further eliminate excavation in the vicinity of Lake Hughes Road and to take advantage of the more competent foundation, a second change was made along a 400-foot-radius curve to the northeast from Station 56+50 to Station 60+97 and then along a tangent line to the south end of the visitor's construction overlook.

The alignment change to the tangent line at Station 45+20 departed from the original axis location at a very small rate. The locations of the various zones of embankment were brought into proper relationship with the new axis by varying outer and contact slopes slightly through a small height differential. The alignment change from the tangent to the 400-foot-radius curve at Station 56+50 would have been simple, except that there was insufficient room to extend the downstream Zone 2A and the 2B chimney. At the time of this change, Zone 1A at the east abutment was being built upward along a 1:1 foundation cut slope at its southern limit. The existing alignment also located Zones 2A and 2B on the 1:1 slope. To correct this situation, Zone 1A material was carried to higher elevations in this area until there was sufficient room to extend Zones 2A and 2B and also accommodate a section of Zone 3 adjacent to the slope.

Earth buttresses were added during construction to stabilize this area and an old landslide area on the right abutment. This construction included the earthwork for a boat ramp and parking lot on the left abutment as discussed later in this chapter.

Foundation

Site Geology. The entire project area is underlain by the Castaic formation. Bedrock under the embankment is composed of approximately two-thirds shale interbedded with one-third sandstone. Silty shales and sandy shales comprise approximately 90% of the shale areas; clay shales comprise approximately 10%. Generally, the bedrock is not exposed at the ground surface, being covered by sand and gravel alluvium up to 85 feet thick in the channel, by terrace deposits up to 200 feet thick on the right abutment, and by landslides over 100 feet thick on the left abutment.

Fresh shale is moderately hard. Bedding thickness ranges from ½ to 1 foot. Some areas contain soft clay seams varying from a thin coating to ¼ of an inch thick along bedding planes. Weathered shale is soft to moderately hard and contains gypsum in joints and along bedding planes.

Fresh sandstone is soft to moderately hard. Bedding thickness ranges from ¼ inch to 15 feet with most beds between 1 and 4 feet. Grain size ranges from fine to coarse, with fine grains predominating. The grains range from very weakly cemented to moderately well cemented with clay. Some beds contain hard sandstone concretions up to 5 feet in diameter.

Tight, soft, clay gouges caused by shearing or faulting both across and parallel to bedding planes occur in zones that vary from a fraction of an inch to 2 feet in thickness.

Excavation. Removal of the overburden materials and highly weathered bedrock under the embankment was specified in order to provide a foundation at least as strong as the embankment materials to be placed on it. Another requirement was that Zone 1A and 1B materials be placed on fresh, undisturbed, Castaic formation. This requirement necessitated the design of a cutoff trench.

Spillway

The spillway extends along the ridge line that comprises the right abutment of the Dam. It is approximately 5,300 feet long and consists of an approach channel, weir, transition, chute terminating in a stilling basin, and return channel (Figures 346 and 347). During the early design phase, several spillway locations were studied, including an alignment directly through the ridge into Grasshopper Canyon with a return channel down Grasshopper Canyon to Castaic Creek. Other alternatives included a dual-purpose stilling basin for the spillway and outlet works. Final design selection was based on economy and safety.

Flood Routing. The spillway protects the Dam against the maximum probable flood (spillway design flood) which has a peak discharge of 120,000 cfs and a three-day volume of 86,600 acre-feet. There is 5 feet of freeboard above the resulting water surface. Design of the structures based on this flood is allowed to have stresses above normal. This conforms to the U.S. Army Corps of Engineers' design criteria.

The 1-in-400-year flood is the greatest flood for which design is based on normal stresses. This flood has a peak discharge of 61,000 cfs and a three-day volume of 57,300 acre-feet.

Approach Channel. The approach channel conveys floodflows from the reservoir to the weir, limiting the velocities in the immediate vicinity of the upstream slope of the Dam. It was designed to minimize excavation costs and limit approach velocities to 5 feet per second in the unlined portions of the channel and 15 feet per second in the area where riprap is used.

Hydraulic model tests of the original configuration showed unsatisfactory flow conditions. Other wall alignments were tested for the approach walls. Final wall alignments were based on the results of the model studies.

Weir. The control structure of the spillway consists of an ungated, concrete, ogee weir with a crest elevation of 1,515 feet and net length of 360 feet. The weir length is based on economic studies which compare length of weir to height of dam embankment. The spillway during the maximum probable flood has a discharge of 78,400 cfs and a surcharge above weir crest of 15 feet. The spillway rating curve is shown on Figure 348.

The weir structure was analyzed as a gravity structure. Stability against sliding and overturning was analyzed, and the magnitude and distribution of the foundation reaction resulting from the weight of the weir and the applied loads were determined. The weir structure is anchored to the rock foundation to increase its effective weight. For computing the stabil-

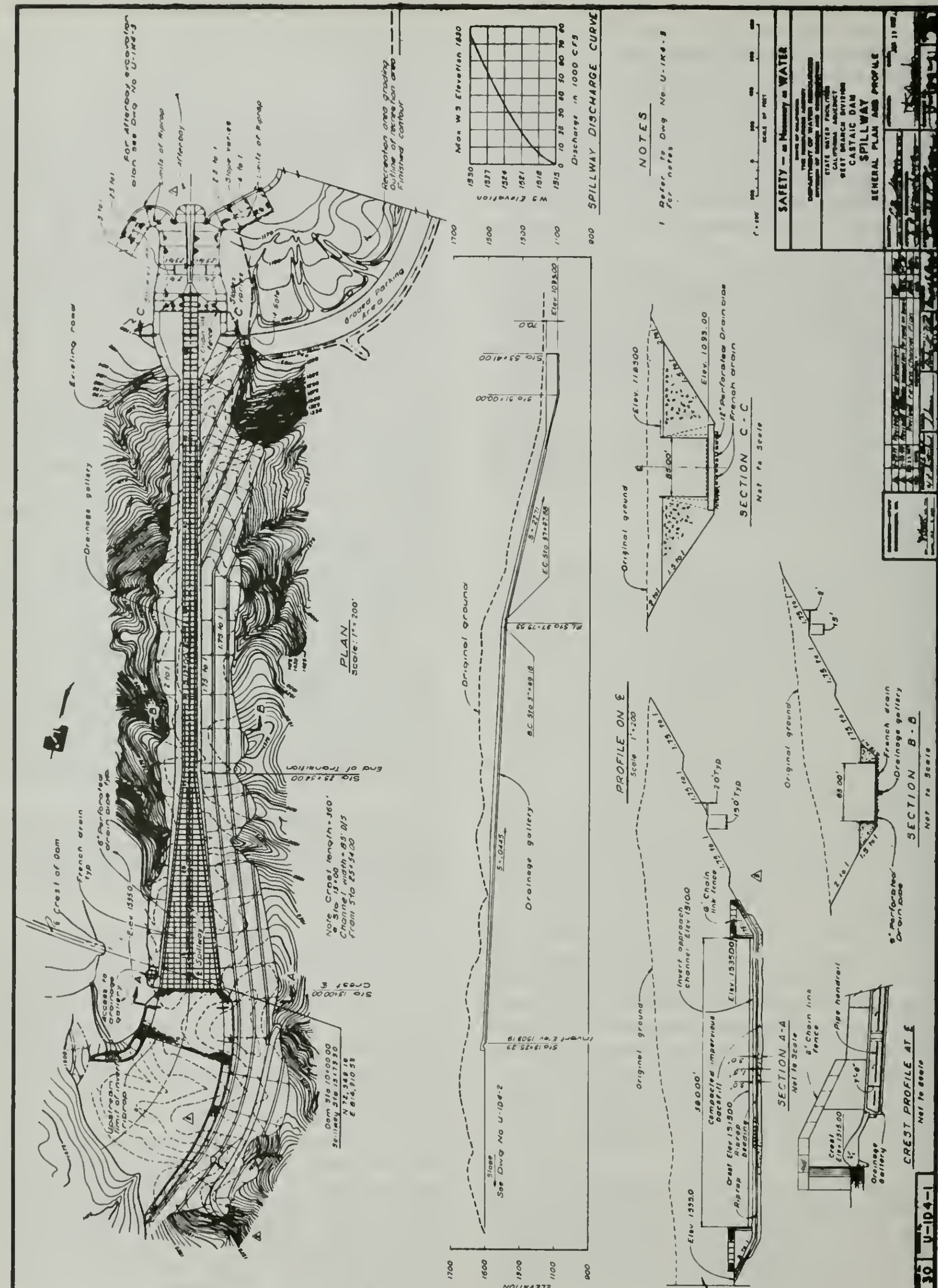

Figure 346. General Plan and Profile of Spillway

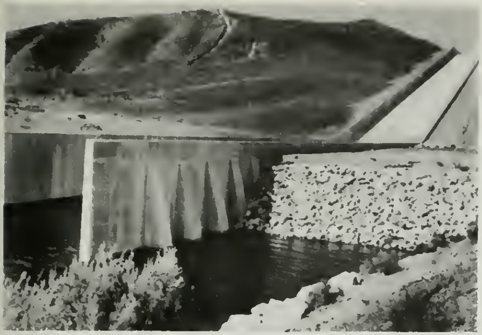

Figure 347. Spillway and Stilling Basin

ity, the critical sliding plane was assumed as the surface passing from the bottom of the upstream shear key to the bottom of the toe curve slab at the downstream end.

Transition. The transition, immediately downstream of the weir, directs the flow into the 85-foot-wide chute with a minimum of turbulence. The flare angle for the transition was checked for contractions in supercritical flow. Various flare angles were tested in the hydraulic model. The design angle used gave the most satisfactory model study results.

Chute. The chute extends between the end of the transition at Station 25+54 and the beginning of the stilling basin at Station 51+00 (Figure 346).

Hydraulic model tests showed the flow down the transition and the chute to be satisfactory for the maximum discharge of 78,400 cfs, except for a short area where wave action approached the top of the wall. Wall heights were increased in this area to maintain freeboard.

Stilling Basin. The spillway stilling basin was designed to still all flows up to the 400-year flood. It will traject all the discharges that exceed the 400-year flood to a point not less than 100 feet downstream.

Freeboard to contain the hydraulic jump was the controlling factor in determining the wall height required. The length and radius of the flip bucket were chosen to yield a trajectory length of greater than 100 feet and to resist all dynamic, hydrostatic, and soil loads applied to it during spillway operation and during the design earthquake. Following standard practice, it is assumed for design purposes that there will be no seismic action during maximum probable flood conditions.

Return Channel. Immediately downstream of the flip bucket, a channel approximately 400 feet long returns all flows to Castaic Lagoon. Channel side slopes and floor are protected from erosion by riprap sized to resist return flows that occur when the flip bucket trajects the flow.

Retaining Wall Design. The active and passive states of earth pressure in the pervious backfill were

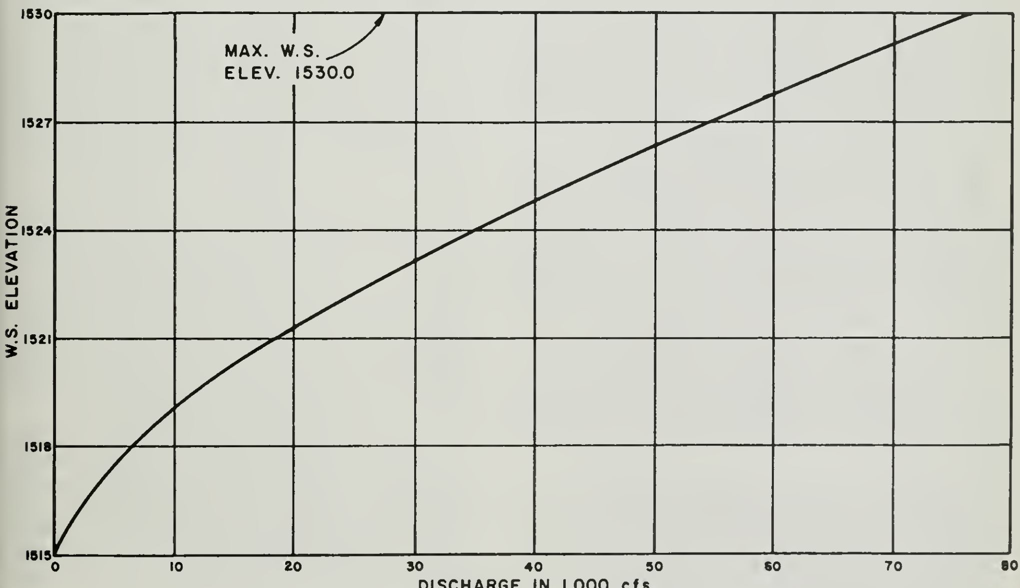

Figure 348. Spillway Rating Curve

computed. Soil load was determined under the effect of earthquake loading.

For various retaining wall design conditions where normal stresses were allowed, the resultant of the applied forces is within the middle one-third of the base. For improbable conditions (such as water in the spillway at the time of an earthquake) where the allowable stresses are increased by one-third over normal stresses, the resultant of the applied forces is within the middle one-half of the base.

The approach, transition, and chute walls are of the cantilever type and vary in height from 7 to 26.5 feet. Near the end of the chute, at Station 49+08, the wall changes from cantilever to counterfort. Counterfort walls vary in height from 26.5 feet at Station 49+08 to 70 feet at the start of the stilling basin (Station 51+00).

The use of gravity, cantilever, and counterfort walls for the stilling basin was investigated. Counterfort walls were selected on the basis of economy and ease of construction.

Floor Design. The impervious blanket in the aproach channel extends upstream from the spillway crest structure approximately 300 feet to create a percolation barrier to control seepage below and around the spillway crest structure. The blanket is a 3-foot-thick, compacted, impervious material. Three feet of riprap was placed over the impervious blanket in the approach channel.

The floor slabs in the transition and chute were designed in approximately 30- by 35-foot panels. Stability without the use of anchor bars and ease of construction were considered when the size and thickness of the floor slabs were selected. All transverse contraction joints were provided with a cutoff and drains. The floor slabs were designed to resist uplift when the downstream transverse drain in a panel is plugged and no water is flowing in the spillway channel.

Open-Cut Excavation. All construction slopes, except in the area of the stilling basin, are ¾:1. Near the stilling basin, the maximum construction slopes are 1½:1, which provides more stability. The extra slope stability was required because the excavation in the area of the stilling basin is deep and was left exposed for a considerable period of time. Permanent slopes do not exceed 2:1 on the eastern side of the spillway excavation nor 1½:1 on the western side.

Berms, in general, are spaced 40 feet vertically. The berms are 15 feet wide and slope at 7½:1 from the toe to the heel of the berm. Where necessary, a pneumatically applied mortar was provided for erosion protection.

Drainage. To reduce uplift, drainage was provided under the downstream end of the crest toe, floor slabs of the transition chute, and stilling basin, thus adding to the stability of the structure. The underdrain system consists of a drainage gallery, cross drains, wall heel drains, and French drains.

Foundation. The final selection of alignment and grade of the spillway weir, chute, and stilling basin achieved acceptable foundation conditions and satisfied all the hydraulic requirements. The upstream starting point placed the foundation of the weir crest structure in fresh Castaic formation that was not faulted nor otherwise disturbed. The location of the stilling basin was governed by locating the flip bucket in fresh Castaic formation. The stilling basin floor is 10 feet deeper than required for hydraulics to ensure that the entire foundation of the structure is in fresh Castaic formation.

Outlet Works

During the early design phases, several configurations were considered for the outlet works. These involved principally the intake tower and the downstream facilities. These studies included (1) a free-standing tower with its base at elevation 1,200 feet, (2) a sloping intake similar to the one at Lake Oroville for Edward Hyatt Powerplant, and (3) the final design that was built. The first two plans were eliminated because of topographic and geologic conditions. All plans considered multiple intakes to satisfy water quality requirements. The outlet works (Figures 349 and 350) utilizes the existing diversion tunnel to convey water under the Dam. Several modifications and additions to the diversion tunnel were required.

A low intake tower (Figure 351) is located on the diversion tunnel intake structure. The tower is 15 feet in diameter, 100 feet high, and draws water from the reservoir above elevation 1,280 feet, minimum reservoir pool.

A multiple-level, high, intake tower (Figure 352) is located above the diversion tunnel and is connected to it by a vertical shaft. An access bridge (Figures 353 and 354) spans from the right abutment of the Dam to the high intake tower. A fixed-wheel gate was placed upstream of the shaft and diversion tunnel intersection to shut off discharges through the low intake.

A 19-foot-diameter steel penstock, located in the 27-foot-diameter diversion tunnel, terminates at a bifurcation at Station 50+50. This bifurcation reduces and separates into two 9-foot - 6-inch-diameter penstocks. The stream release facility (Figures 355 and 356) is composed of these two penstocks and three smaller lines that branch from the left 9-foot - 6-inch branch. Fixed-cone dispersion valves and a pressure-reducing valve are used to regulate stream releases. The control valves are protected by butterfly guard valves.

Turnouts just upstream from the bifurcation (Figures 357 and 358) deliver water to the various water users. These turnouts enter vaults containing guard valves which are operated either fully open or fully closed. This permits each water user to regulate the flow rate with his own downstream valving. Meter vaults are located downstream of the guard valve vaults.

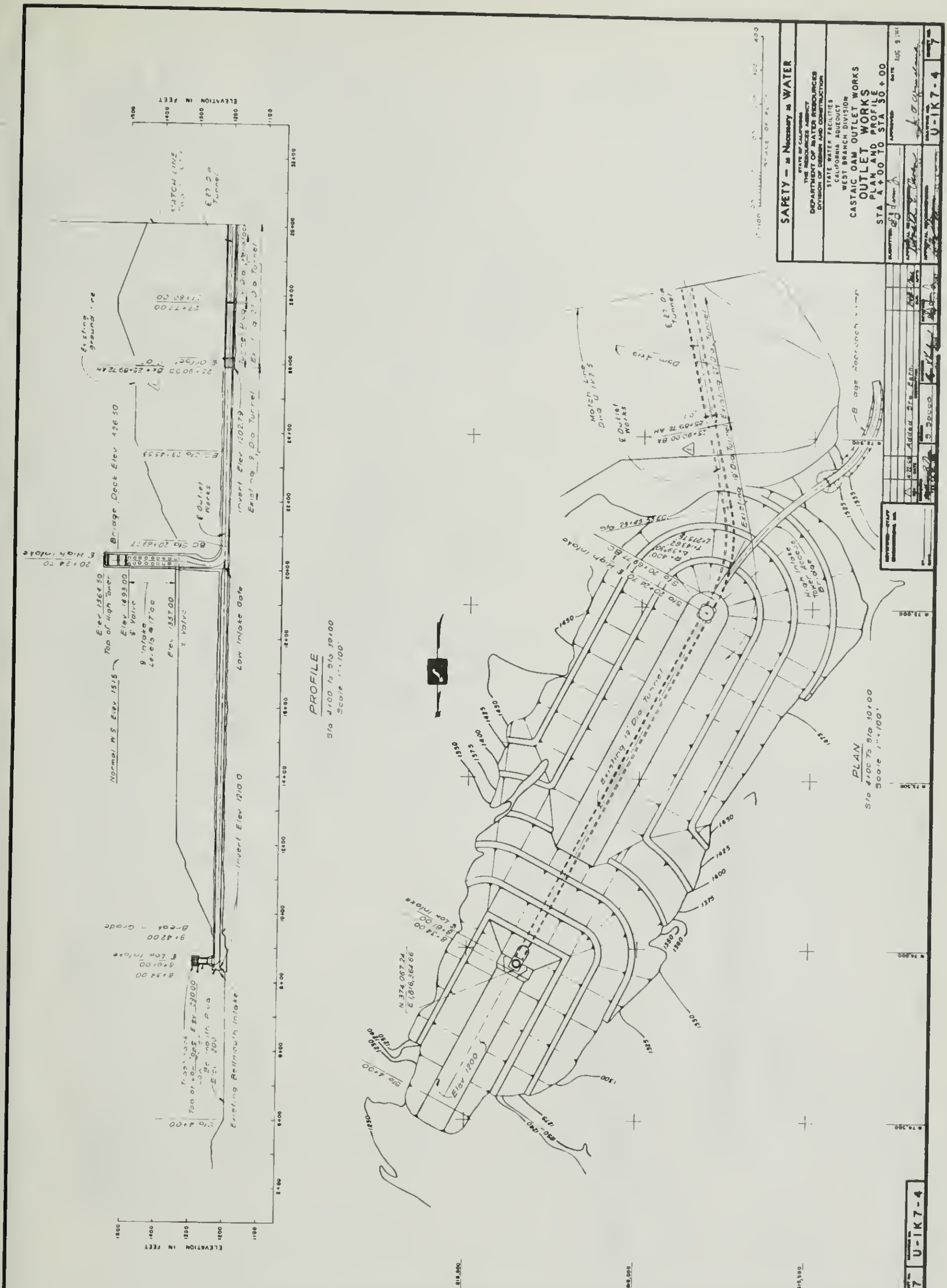

Figure 349. Outlet Works—Plan and Profile

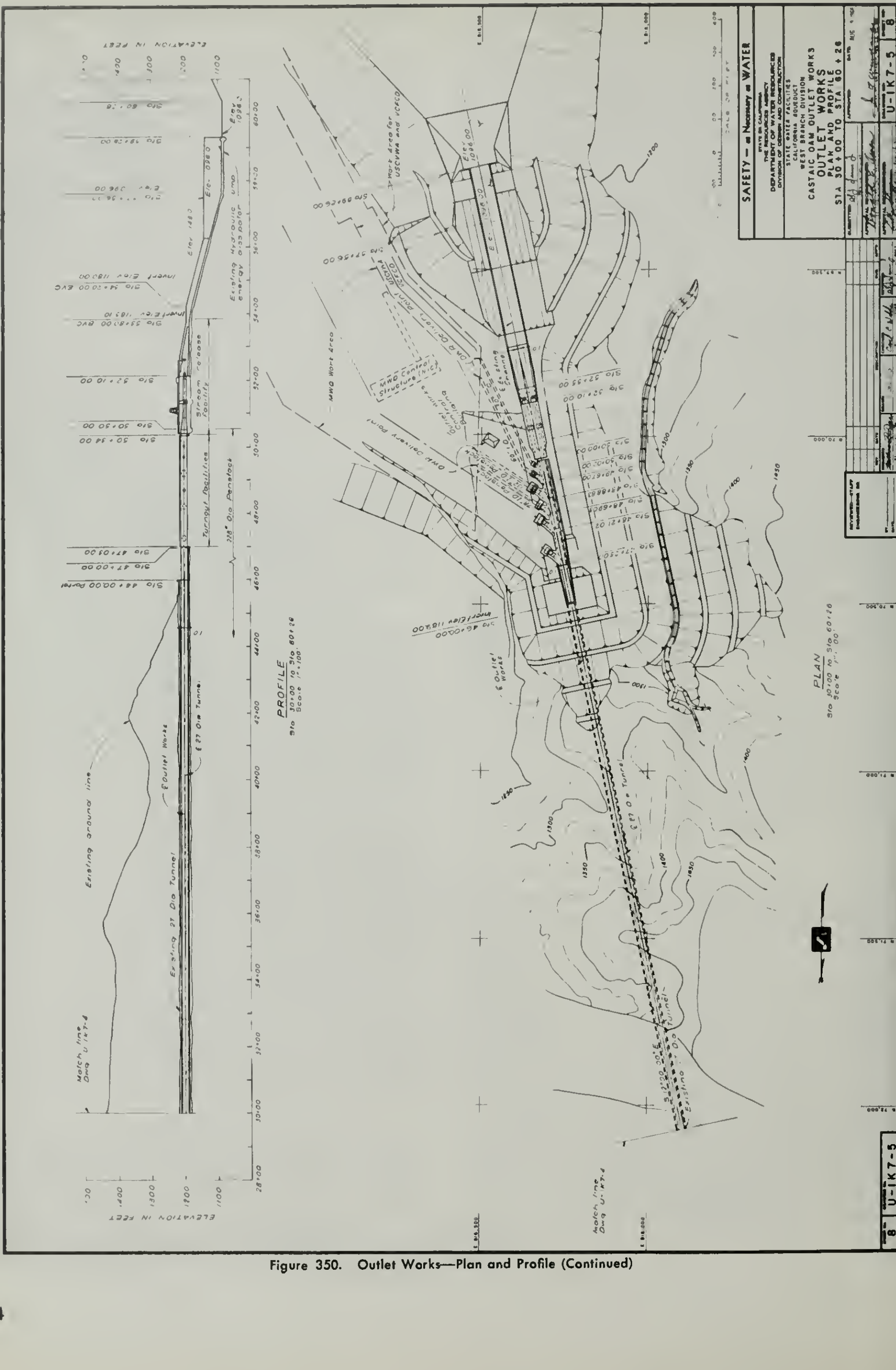

Figure 350. Outlet Works—Plan and Profile (Continued)

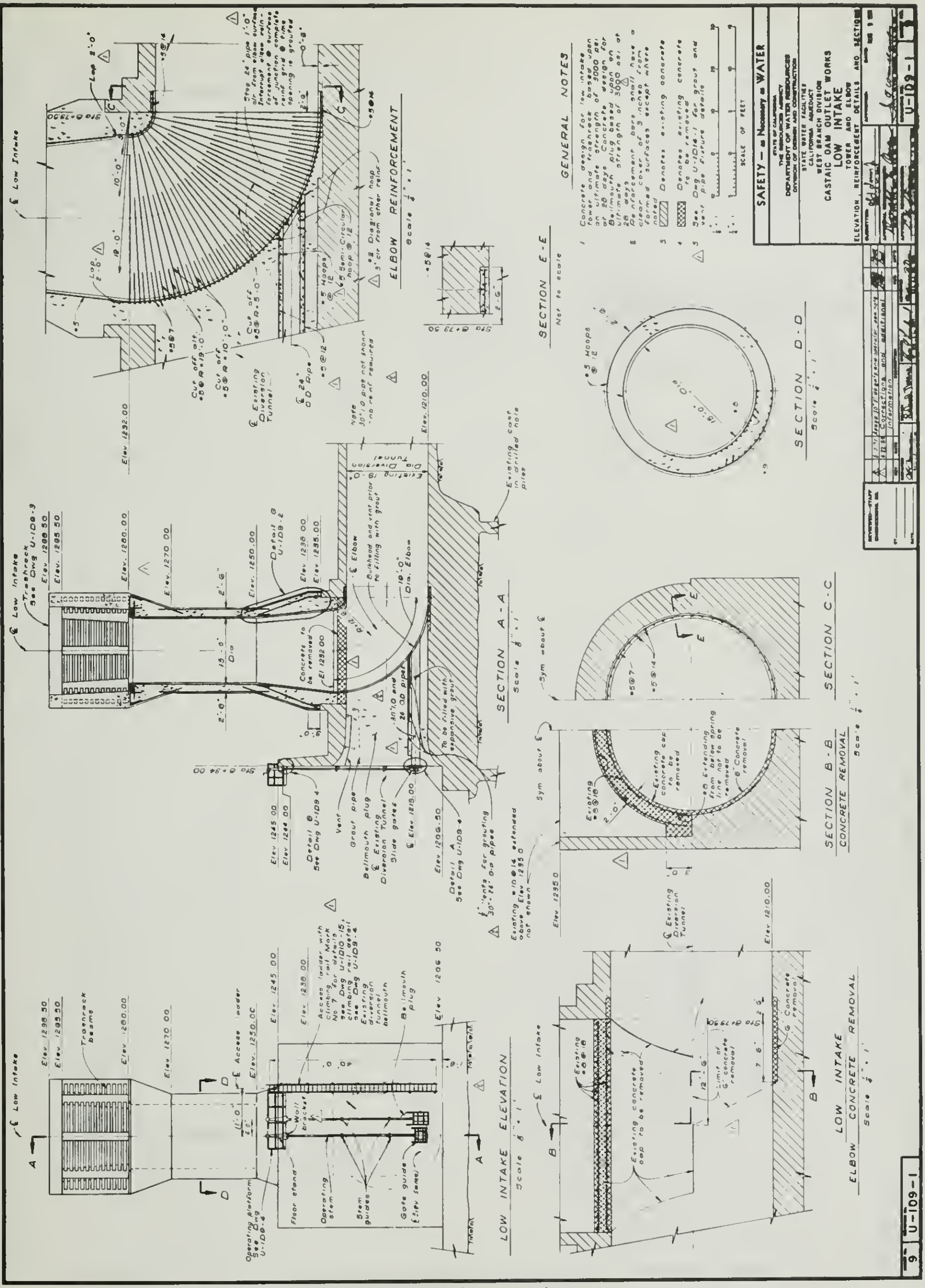

Figure 351. Outlet Works—Low Intake Tower

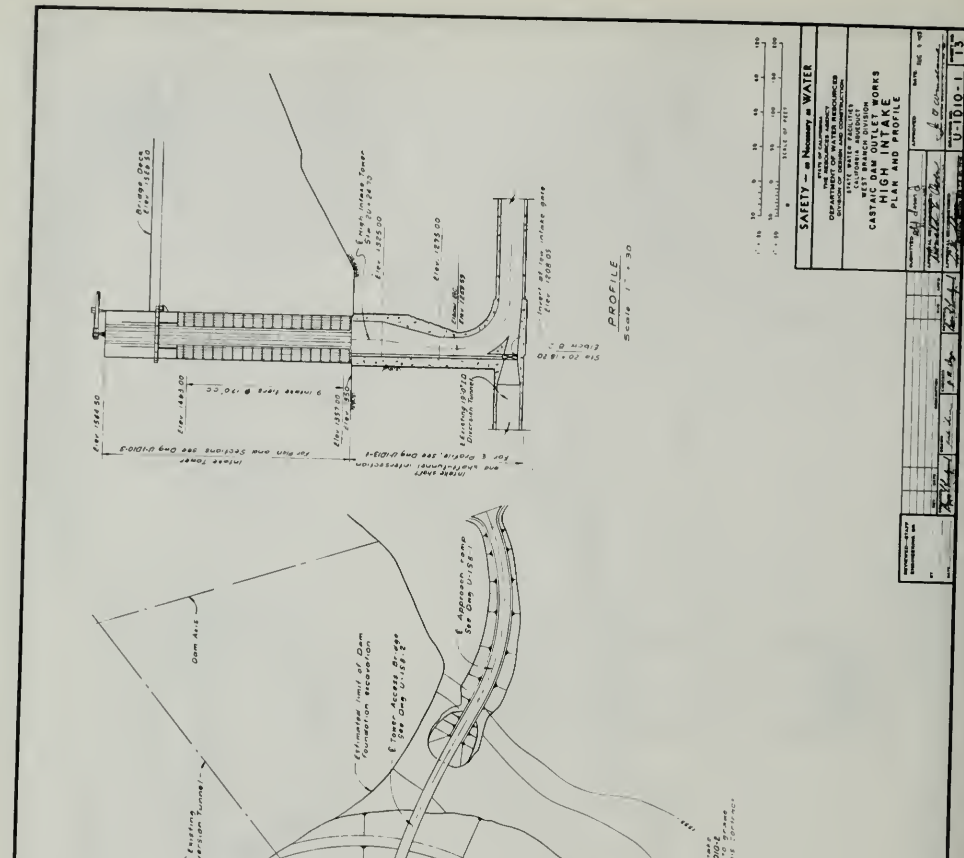

Figure 352. Outlet Works—High Intake Tower

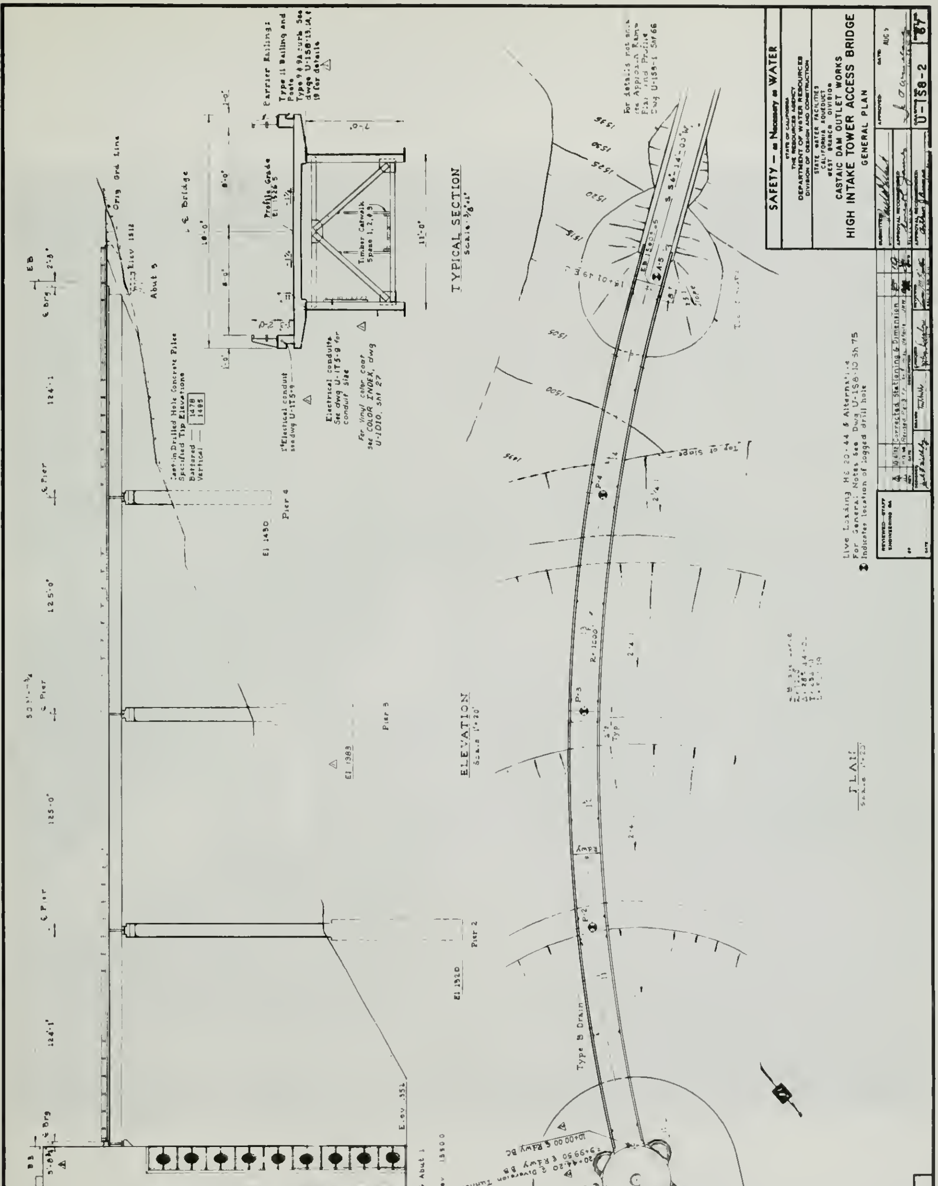

Figure 353. Access Bridge

Figure 354. High Intake Tower and Access Bridge

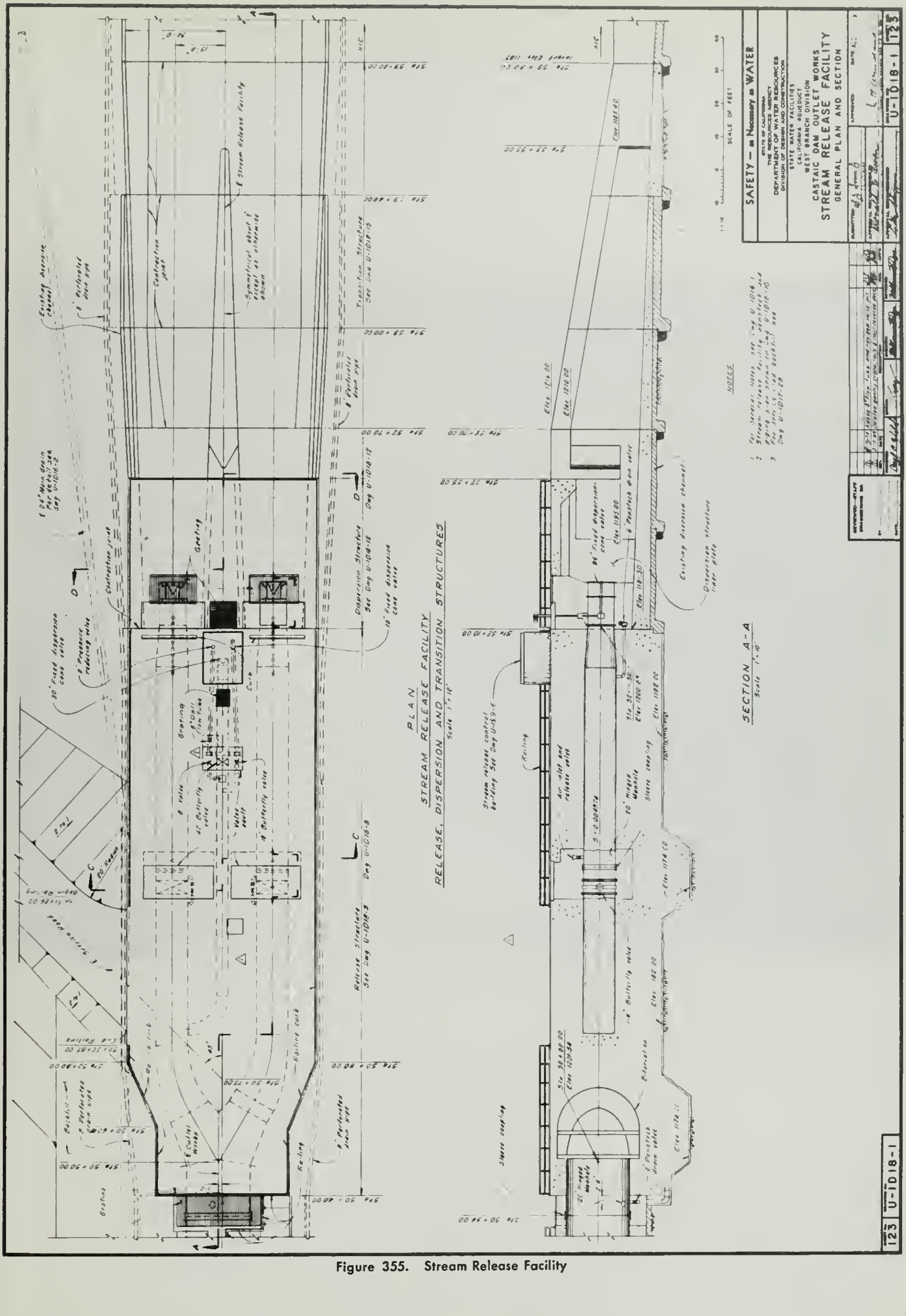

Figure 355. Stream Release Facility

Figure 356. Outlet Works Stream Release

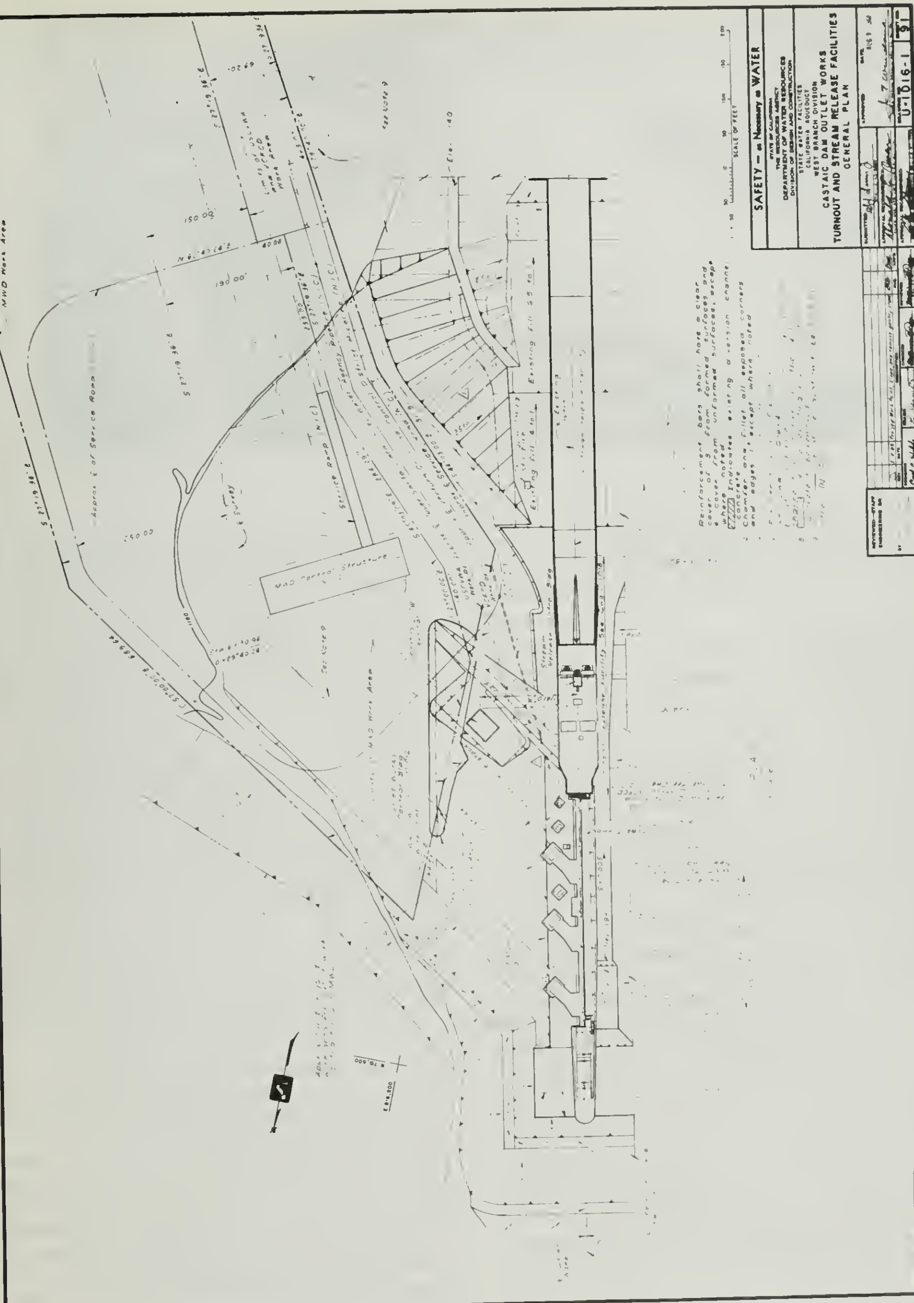

Figure 357. General Plan of Turnouts

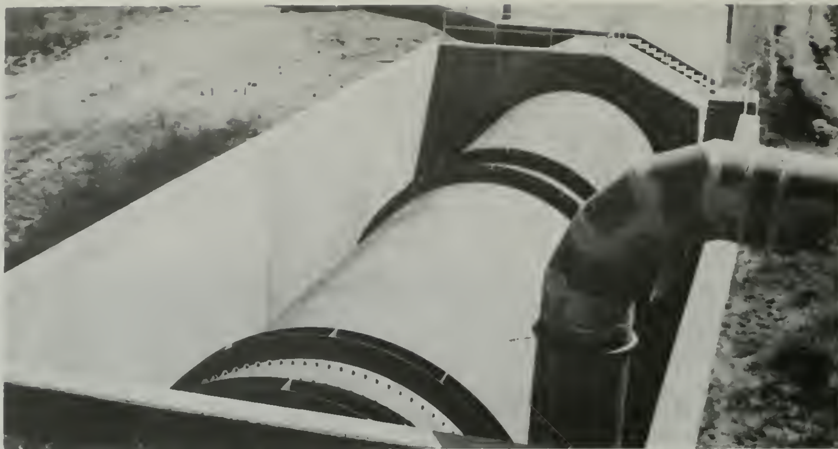

Figure 358. Outlet Works Turnouts

Hydraulics. The hydraulic criteria used to design the delivery system were based on contract commitments and maximum deliveries as follows:

	Yearly Contract	*Maximum Delivery*
The Metropolitan Water District of Southern California (MWD)	2,000,000 acre-feet	3,650 cfs
Castaic Lake Water Agency (CLWA)	41,500 acre-feet	103 cfs
Ventura County Flood Control District (VCFCD)	20,000 acre-feet	35 cfs
Totals	2,061,500 acre-feet	3,788 cfs*

* Design flow with reservoir surface at elevation 1,421 feet.

The head available to make these deliveries was established at 21 feet (reservoir water surface elevation 1,421 feet—hydraulic gradeline elevation 1,400 feet at the Department/MWD delivery point). From the just-mentioned criteria, tunnel, pipe, and turnout sizes were selected. The sizes of the turnouts are as follows: two 150-inch, one 132-inch, and one 78-inch for MWD; one 60-inch for CLWA; and one 30-inch for VCFCD. The required port area per level at the high intake tower was 88 square feet, based on a velocity through the ports of approximately 25 feet per second. This was based on operating two levels of ports for maximum delivery.

All natural inflow is to be measured for release to the downstream water rights owners. Flows up to 6,000 cfs are released at the same rate as the inflow. Flows above the maximum outflow rate of 6,000 cfs are stored until this excess inflow can be safely released. Downstream flow releases are through two 96-inch, one 30-inch, and one 10-inch fixed-cone dispersion valves, and one 8-inch pressure-reducing valve.

To minimize cavitation problems, the fixed-cone dispersion valves will not be operated less than 10% open. The 8-inch pressure-reducing valve can discharge 0.5 to 8 cfs between minimum total net head of 156 feet and 320 feet of maximum static head.

Reservoir drawdown time from elevation 1,515 feet (maximum water surface) to elevation 1,400 feet is about 14 days. Maximum discharge with the fixed-cone dispersion valves removed is approximately 11,000 cfs.

A fish barrier was required to prevent the passing of fish from the reservoir into the system. The fish screens are movable and cover any two adjacent ports in a vertical row. The units travel inside the trashrack on rails with a clear distance of at least 6 inches between the trashrack system and the screen. Openings with movable covers are provided to allow the screen unit to pass protruding hardware on the outside of the tower.

The fish screens consist of stainless-steel mesh with $\frac{3}{8}$-inch clear opening attached to a frame of stainless-steel tubing. The mesh was designed to withstand 5 feet of differential head, and the frame was designed to withstand 10 feet of differential head.

Structural Design of High Intake Tower. The superstructure arrangement was chosen to accommodate mechanical and electrical equipment and to satisfy structure requirements.

In design of the operating deck, five live load sources were considered. In design of the intake tower, six loading cases were considered, including seismic forces with tower dry and with water both inside and outside the tower. San Andreas design earthquake spectra, as developed by the Department's Consulting Board for Earthquake Analysis, was used as the basis for determining design accelerations for the tower. For determination of design base moment and shear, 60% of the San Andreas spectral acceleration was used. This reduction was based on consideration of the seismic response of the foundation material and on the relative importance of the high intake system and the consequences of its failure. The calculated modal periods of the tower were 0.78 of a second, 0.12 of a second, and 0.045 of a second for the first three modes. The 8% damping factor used in the seismic response analysis resulted in acceleration factors of 0.25, 0.33, and 0.30 for the first three modes.

The tower was treated as a moment-deflecting cantilever. For the controlling seismic case, the base moment and shear are a result of the dead-weight inertial load of the tower and superstructure, internal hydrodynamic load, and external hydrodynamic load. Dynamic and hydraulic model studies, conducted at the University of California at Davis, verified the design assumptions (see Bibliography).

The intake shaft was designed to transfer base moment and shear to the foundation rock. A joint was designed for the shaft at elevation 1,262 feet to prevent transfer of the moments to the tunnel. Tower and shaft dead weight were included with the maximum moments from flexural design. Hoop reinforcement was designed to provide tensile reinforcement to resist diagonal tensile stresses associated with moment and shear in horizontal planes during an earthquake.

Placement of shotcrete was specified after the completion of each 6 feet of shaft excavation, thus preventing air slaking and minimizing the need for lagging. Excessive lagging would reduce the effectiveness of the shaft concrete contact with the rock. A consolidation grouting program also was specified to fill voids and structural defects in the surrounding rock and between the rock and shaft concrete.

Trashracks protect all the tower intake ports and the traveling fish screens from logs and other debris. Maximum spacing of the trashrack tubes was set at 4 inches, based on the maximum-size debris that can pass through downstream facilities. Stainless steel was chosen because of its low maintenance requirement and long service life.

High Intake Tower Access Bridge

The high intake tower access bridge, in combination with an approach ramp, provides access to the Castaic outlet works high intake tower from the nearby crest of the Dam. A curved bridge alignment was required to permit an efficient arrangement of equipment in the tower. This alignment also located the abutment a safe distance from the top of an adjacent cut slope and allowed for an economical design of the abutment.

The bridge is 504 feet long with a superstructure consisting of four simple spans of welded-plate girders acting compositely with a lightweight concrete deck. The girders are supported by the high intake tower, reinforced-concrete piers which are socketed into rock, and a reinforced-concrete abutment. The superstructure is highly articulated to accommodate extreme earthquake movements. It provides a clear roadway width of 16 feet between barrier railings.

Design was in accordance with the 1965 AASHO specifications and with the State of California "Bridge Planning and Design Manual". The bridge was designed for a live loading of HS20-44 and an alternative loading of two 24,000-pound axles, 4 feet apart.

Instrumentation

Castaic Dam instrumentation consists of 15 pneumatic foundation piezometers, 16 pneumatic embankment piezometers, 18 hydraulic piezometers, 13 open-tube piezometers, 8 embankment slope indicators, 9 abutment slope indicators, 69 embankment surface monuments, 15 embankment soil stress cells, 7 embankment accelerometers, 1 foundation accelerometer, and 2 accelerographs. Monuments also are located along the top of the spillway walls and on other concrete structures. The instrumentation was designed to monitor pore pressures and vertical and horizontal movements, as well as acceleration response of earth motion and dynamic stresses resulting from seismic activity (Figure 359). Seepage is measured at the end of the embankment drainage system and in the spillway drainage gallery.

Castaic Lagoon

As previously mentioned, the Lagoon was initially a borrow area immediately downstream from the toe of Castaic Dam. Maximum elevation of the floor of the Lagoon is 1,125 feet. A minimum of 2 feet of fine-grained, mandatory, spoil blanket was required over most of the Lagoon to control seepage. Where Castaic formation was exposed, no cover was required. The west side of the Lagoon was excavated to enhance the recreation development.

The control structure for the Lagoon is a 170-foot-long by 350-foot-wide concrete apron under the new Lake Hughes Road Bridge. A rectangular discharge channel near the east abutment of the Bridge carries streamflows up to approximately 12 cubic feet per second. A standard Parshall flume with a throat width of 2 feet and a depth of 1 foot - 6 inches was installed near the upstream edge of the apron for flow measurement. Downstream releases are recorded by a flume-recording well. Flows greater than the capacity of the Parshall flume pass over the entire 350-foot width of the apron. The control structure is designed to pass the standard project flood discharge from the spillway.

Details and sections of the lining of the approach channel are shown on Figure 360. Downstream erosion control consists of riprap as shown on Figure 361.

The bridge abutments form a portion of the control structure walls. Cantilever walls of varying height flank these abutments upstream and downstream to complete the control structure walls (Figure 362).

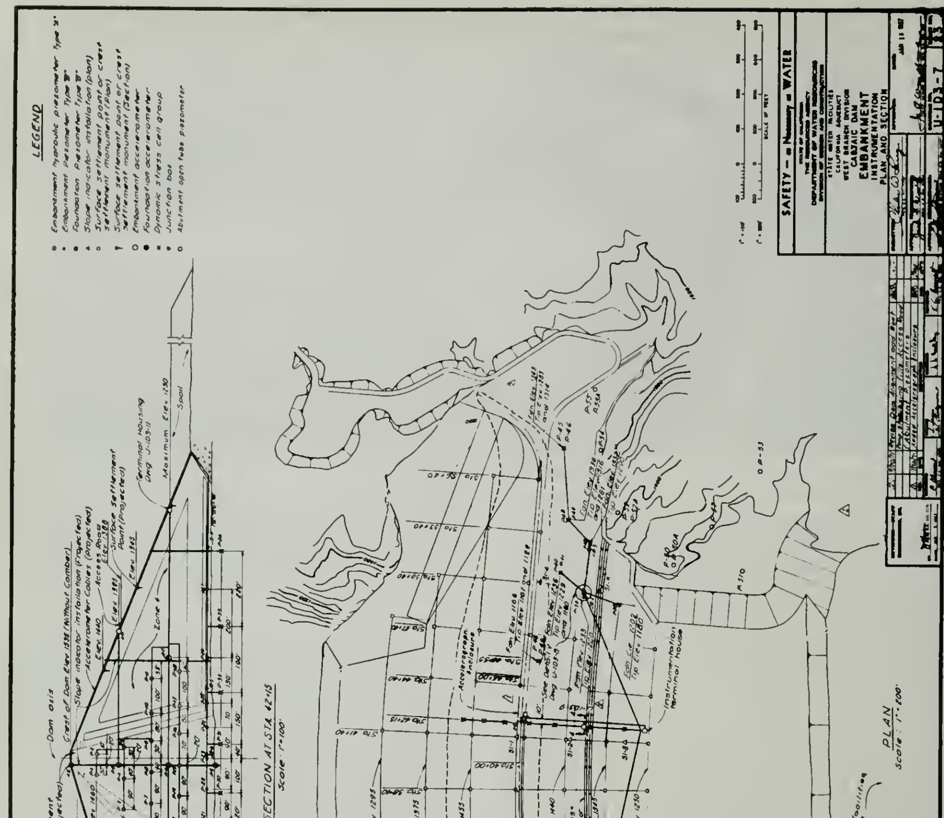

Figure 359. Instrumentation—Plan and Section

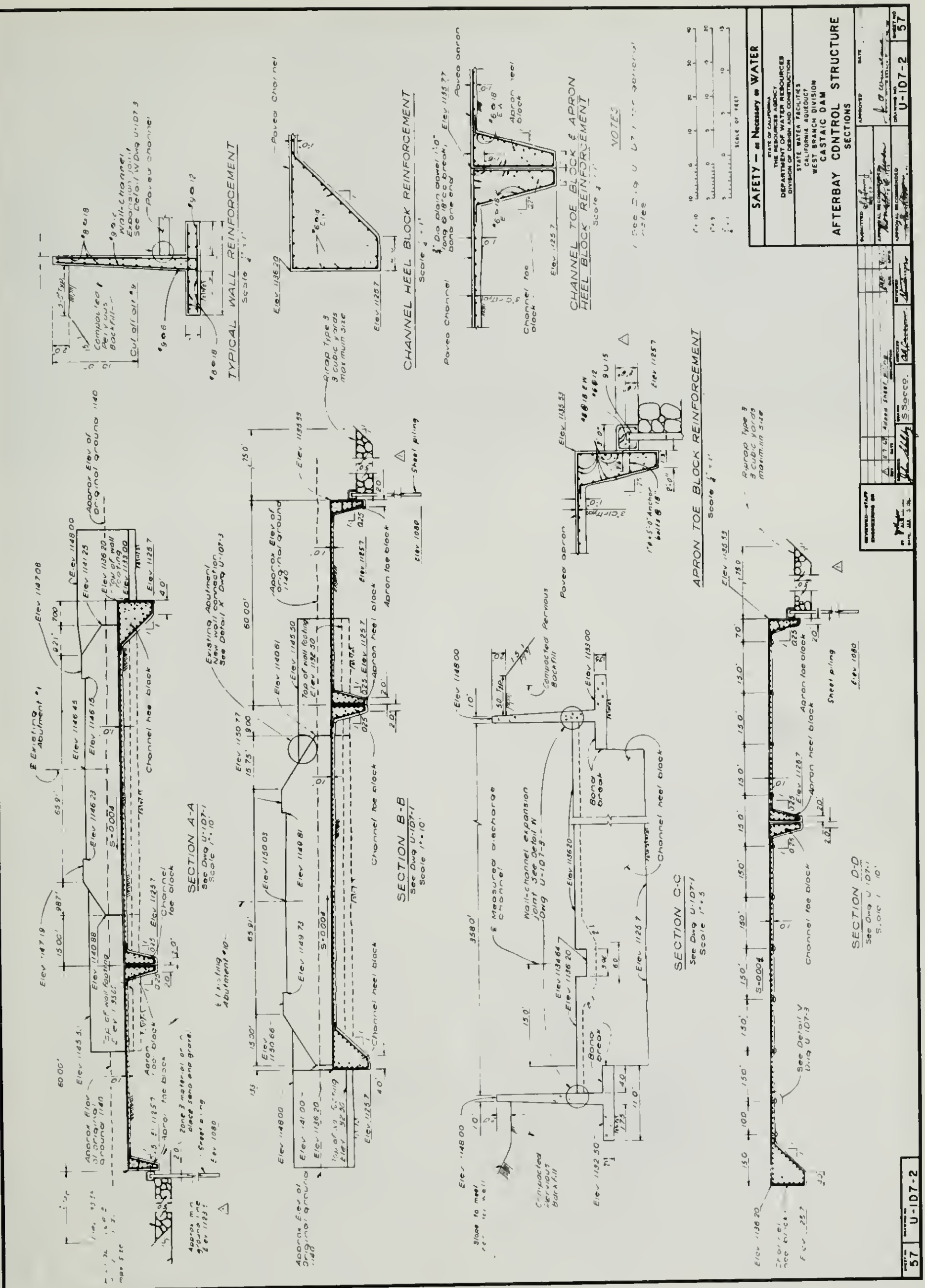

Figure 360. Castaic Lagoon Control Structure Sections

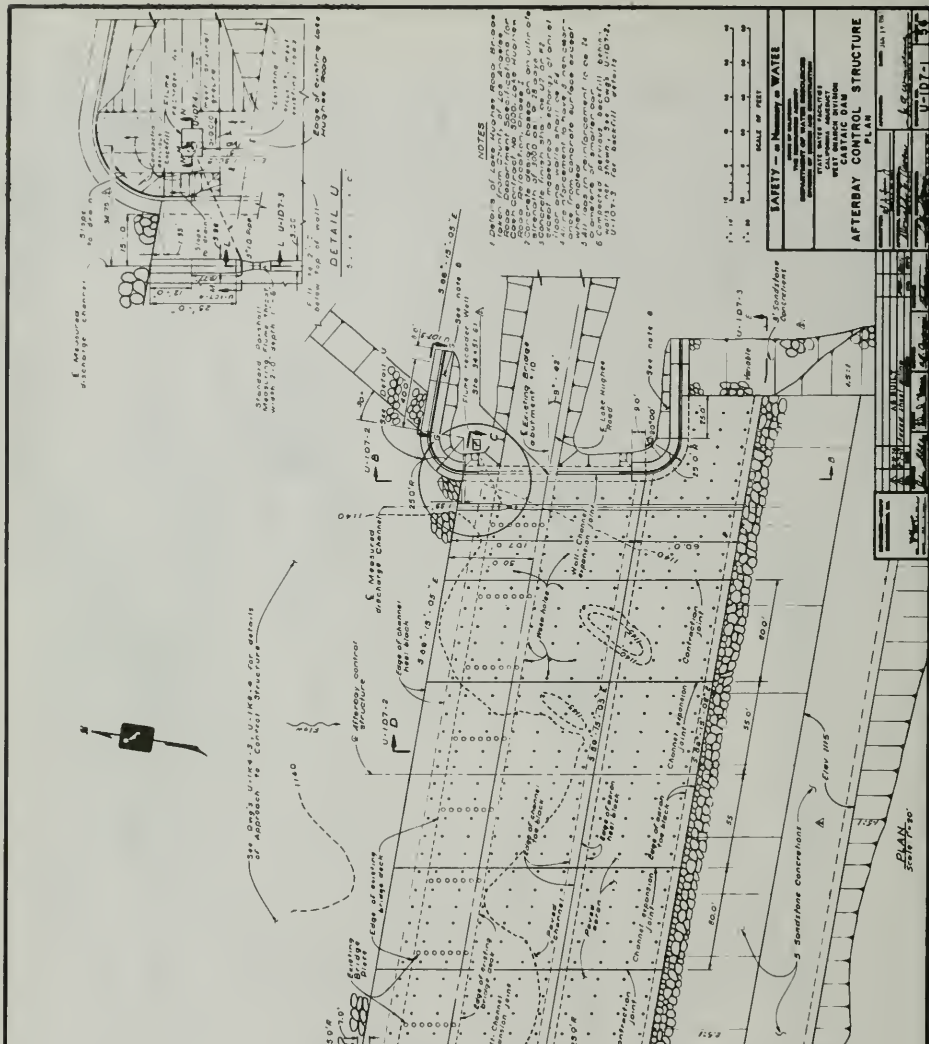

Figure 361. Castaic Lagoon Control Structure—Erosion Control

Figure 362. Castaic Lagoon Control Structure

Mechanical and Electrical Installations

The mechanical features of the outlet works consist of three major systems: (1) the high intake valves, operating system, and auxiliary equipment; (2) the turnout guard valves and operating systems; and (3) stream release regulating system.

The flow of water into the outlet works is controlled by the high intake tower port valves for normal reservoir operating levels and the low intake gate for low reservoir levels. The high intake auxiliary equipment provides emergency power, handling, maintenance, and repair capabilities. The turnout guard valves and support equipment provide closure capability for the performance of maintenance work on water user pipelines and meet the needs of any outlet works emergency. The stream release regulating system provides for release of normal streamflow, storm inflow, and emergency drainage of the reservoir.

Separate normal electrical service is supplied by a utility company, and standby electrical power is provided by engine-generator sets for the intake tower and stream release facilities. A single-line diagram for the intake tower is shown on Figure 363 and the stream release facilities on Figure 364.

All equipment necessary for remote monitoring and control of the intake tower is contained in the tower, while the outlet works control building contains all the equipment for the stream release facilities. Data, status, and alarm information is transmitted to Castaic Powerplant and Castaic Area Control Center. Emergency closure of the intake tower valves and the fixed-wheel gate may be accomplished from the Castaic Area Control Center.

Local controls and annunciations for the intake tower are located in the tower. Local controls and annunciations for the stream release facilities are located in the outlet works control building with duplication at the valves.

High Intake Port Valves. The high intake port valves consist of 22 hydraulic motor-operated, 72-inch-diameter, rubber-seated, butterfly valves with provisions for future installation of 14 additional 72-inch-diameter valves.

The valves are located within the tower on four vertical rows with nine tiers. The tiers are on 17-foot centers from elevation 1,357 to elevation 1,493 feet. Only the bottom two tiers contain valves in all four rows. The remaining valves are on the two opposing rows. Two rows of port thimbles without valves are blocked with a blind flange until future use is required.

A hydraulic power and control unit was installed on the operating deck (elevation 1,526 feet) to provide power to the hydraulic motors. The valves are set to open or close in a five-minute operating time, and controls are provided to operate any combination of valves on a given tier. The valves are designed for a normal operating condition to pass 765 cfs in free discharge with a static head of 30 feet.

Each valve in any one row of the tower is capable of operating partially opened to provide a free discharge flow of 80 cfs at a differential head of 30 feet. The partially open condition is used to fill the tower. Except for filling the tower, the valves are used only in the fully opened or fully closed position.

The port valve thimbles were designed with an elliptically shaped bell-mouth-type entrance based on the equation $\frac{x^2}{1296} + \frac{y^2}{100} = 1$ where constants are selected to suit the space available for the port.

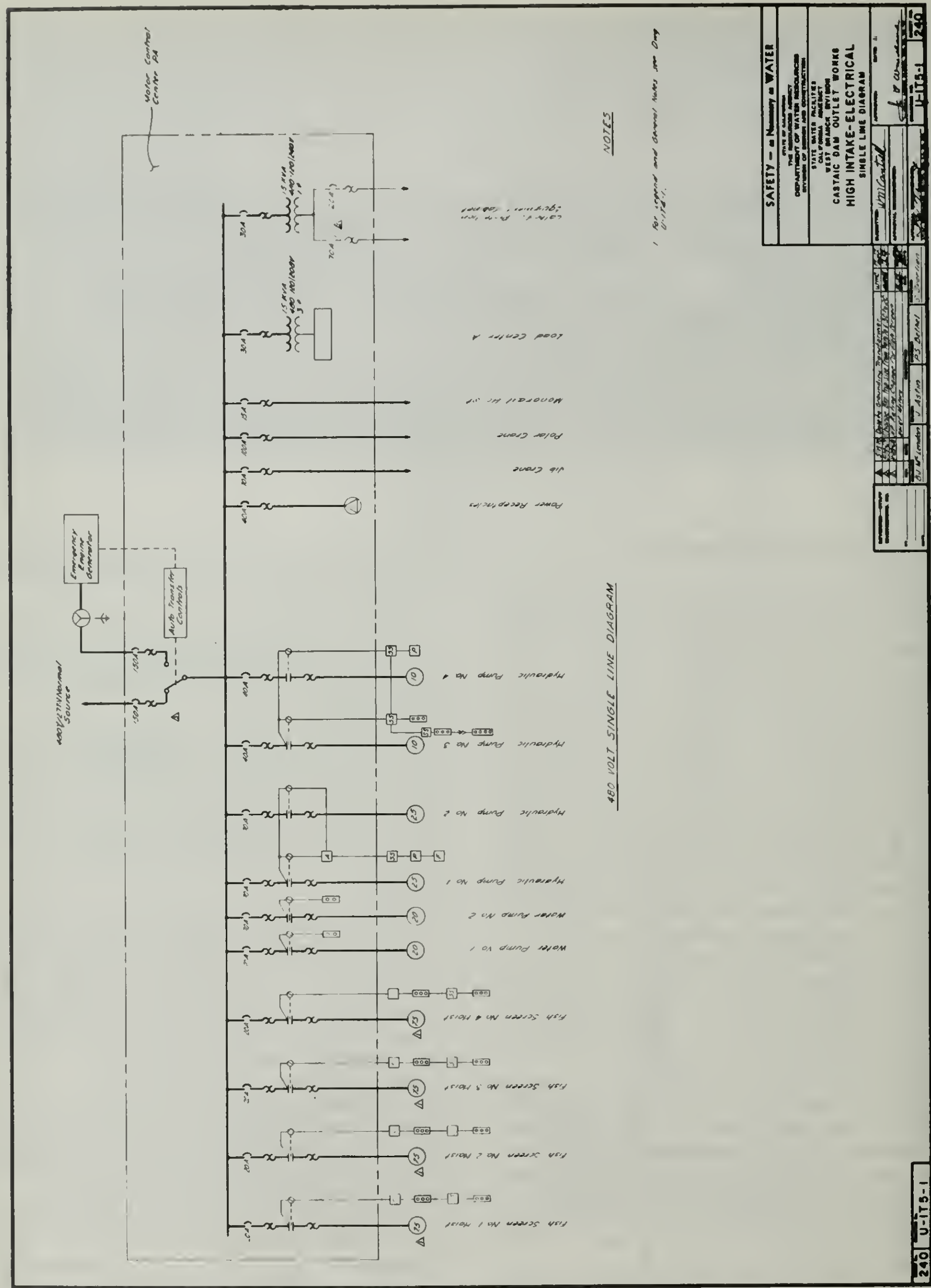

Figure 363. Single-Line Diagram—Intake Tower

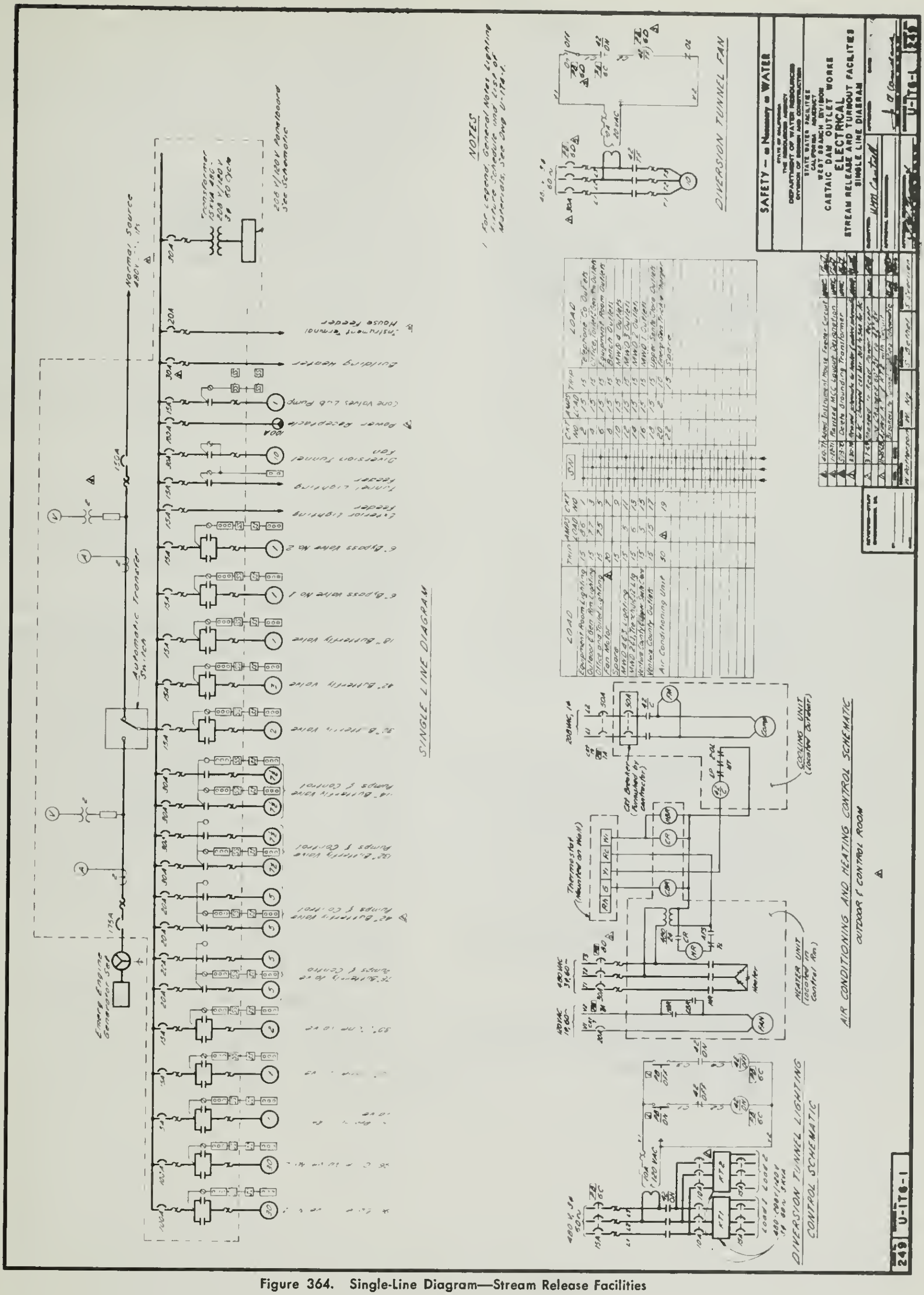

Figure 364. Single-Line Diagram—Stream Release Facilities

The port valves were designed to meet AWWA C504 Class 75B or better rating (Figure 365). The valve disc is offset from the stub shafts with disc position set to create a closing torque imbalance. Should a failure occur in the operator that allows the disc to rotate freely, the valve will be closed automatically by hydraulic forces caused by the flow of water. The valves were installed with the shafts in a vertical position to provide the most suitable location of the operator and to minimize disc vibration. Mounting holes were made on the downstream valve body flange to permit future installation of hoods when four valves per tier are installed. The hoods are to direct the flow in a downward direction. Model studies indicate that the four-valve-per-tier configuration could cause pockets of air to be trapped if no hoods are used.

Three different models of the worm gear-type valve operator are used to match the emergency torque requirements of each tier. The hydraulic motors used on the operators are a constant displacement, multiple axial-piston, rotating type. The valve shaft is attached to the operator by a splined adapter which fits into the operator drive sleeve.

A 10-gallon-capacity lube oil reservoir was installed on the operating deck. It is connected by tubing to each operator housing. This oil lubricates the operator and provides a positive head inside each limit switch and operator housing to prevent water intrusion.

The hydraulic power system is rated 2,000 pounds per square inch (psi) at 20 gallons per minute (gpm). Power is provided by an electric motor-driven pump. The system contains directional control valves, relief valves, check valves, and other auxiliary equipment as shown on Figure 366.

Low Intake Gate System. The low intake gate is located at the upstream interior wall of the high intake tower. The gate is of the vertical-lift fixed-wheel type, with upstream seals and skinplate (Figure 367).

The gate normally is in the closed position and will be used only for delivery of water when the reservoir level is below the level of the tower outlet valves or for emergency drainage of the reservoir. The gate is approximately 11½ feet wide by 11 feet high and is designed for a maximum head of 322 feet. The gate is connected by a 306-foot guided stem extending vertically to a hydraulic gate operator cylinder located on the top deck of the tower.

Preliminary design of the gate-stem system indicated the possibility of a resonating condition produced by flow under the partially open gate.

Model studies (see Bibliography) showed that pressure fluctuations in the flow at the gate lip were near the resonant frequency of the gate-stem system and that the magnitude of the load fluctuations were such that failure of the gate stem was possible. The original design of the sluiceway and gate was modified to reduce the vibrations. The modifications are shown on Figures 367 and 368.

A hydraulic cylinder operator is provided for opening and closing of the low intake gate. The operator has a 14-inch bore and a 192-inch stroke. The operator was designed to open the gate under an unbalanced head of 213 feet and close the gate under an unbalanced head of 307 feet. The hydraulic system is a dual pressure system incorporating separate pumps for opening and closing. Opening pressure is 1,500 psi and closing pressure 750 psi. The dual system was used due to the great difference in force required between raising and lowering the gate. The operator was designed for local-manual operation for both opening and closing the gate and for remote-manual opening of the gate.

High Intake Auxiliary Equipment. The high intake auxiliary equipment (Figures 369, 370, and 371) consists of a 6-ton tower jib crane, 20-ton polar bridge crane, maintenance platform, fish-screen hoists, bulkhead gates, emergency engine-generators, and a fish-screen washing system.

A 6-ton-capacity, cab-operated, outdoor, full-revolving, leg-braced, jib crane was installed at the top of the outlet works high intake tower (Figure 372). The crane is trunnion-mounted and revolves on a rail anchored to the edge of the tower. The crane's purpose is to install and remove the valve maintenance bulkhead gates, trashrack panels, and fish screens. The motors for the hoist, jib motion, and trolley travel controls are regulated stepless adjustable voltage, direct-current, regenerative-braking, and static-reversing. Separate operation is provided for each control. The controls are interlocked so that the jib and trolley drives will not operate while the hoist is being raised or lowered. Capacities and speeds are as follows:

Rated capacity of crane, tons	6
Rated capacity of hoist, tons	6
Rated hoisting speed, feet per minute (fpm)	25–30
Jib travel speed, fpm at centerline of rail	4–6
Trolley travel speed, fpm	4–6
Maximum lift, feet	210
Operating reach, feet	0–12

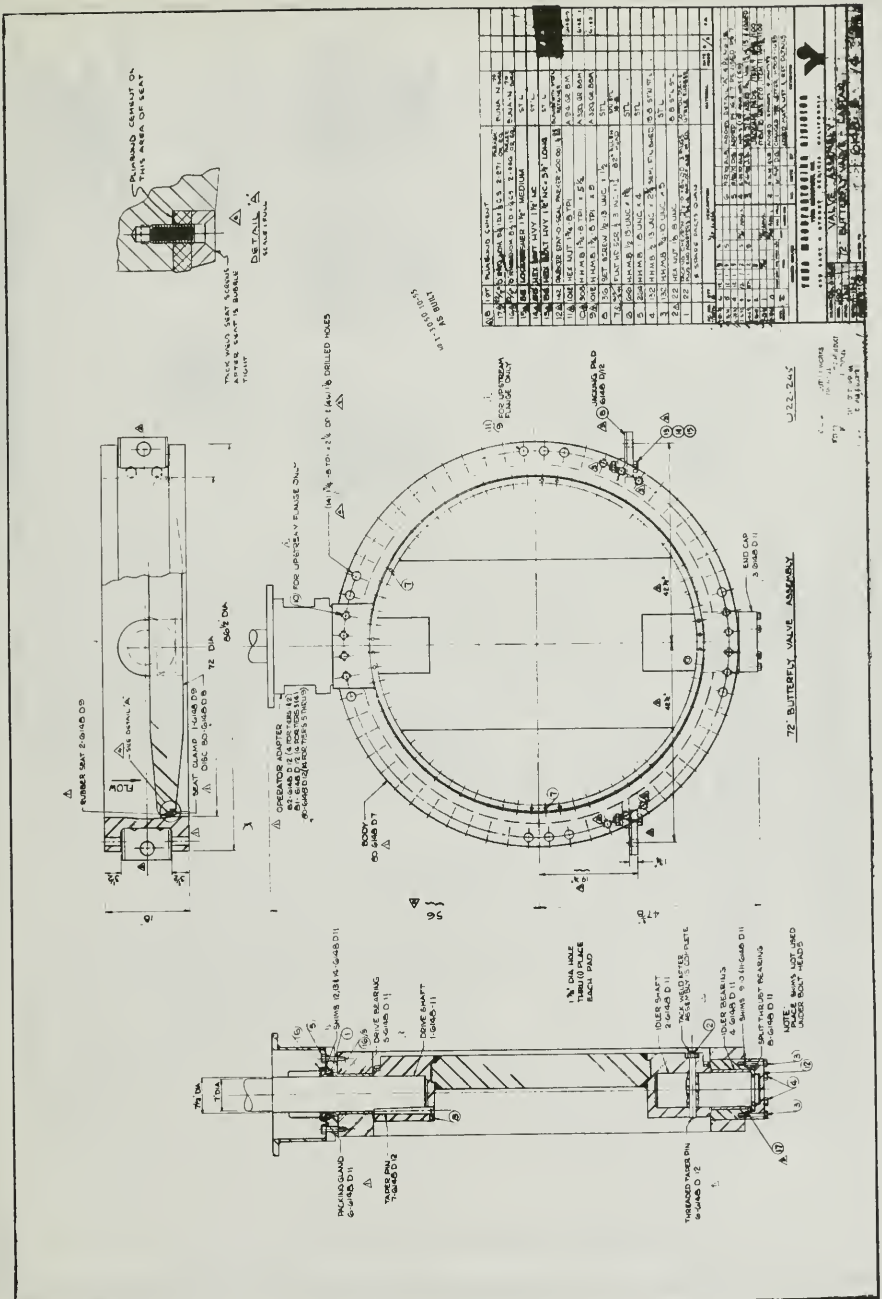

Figure 365. Butterfly Valve Assembly

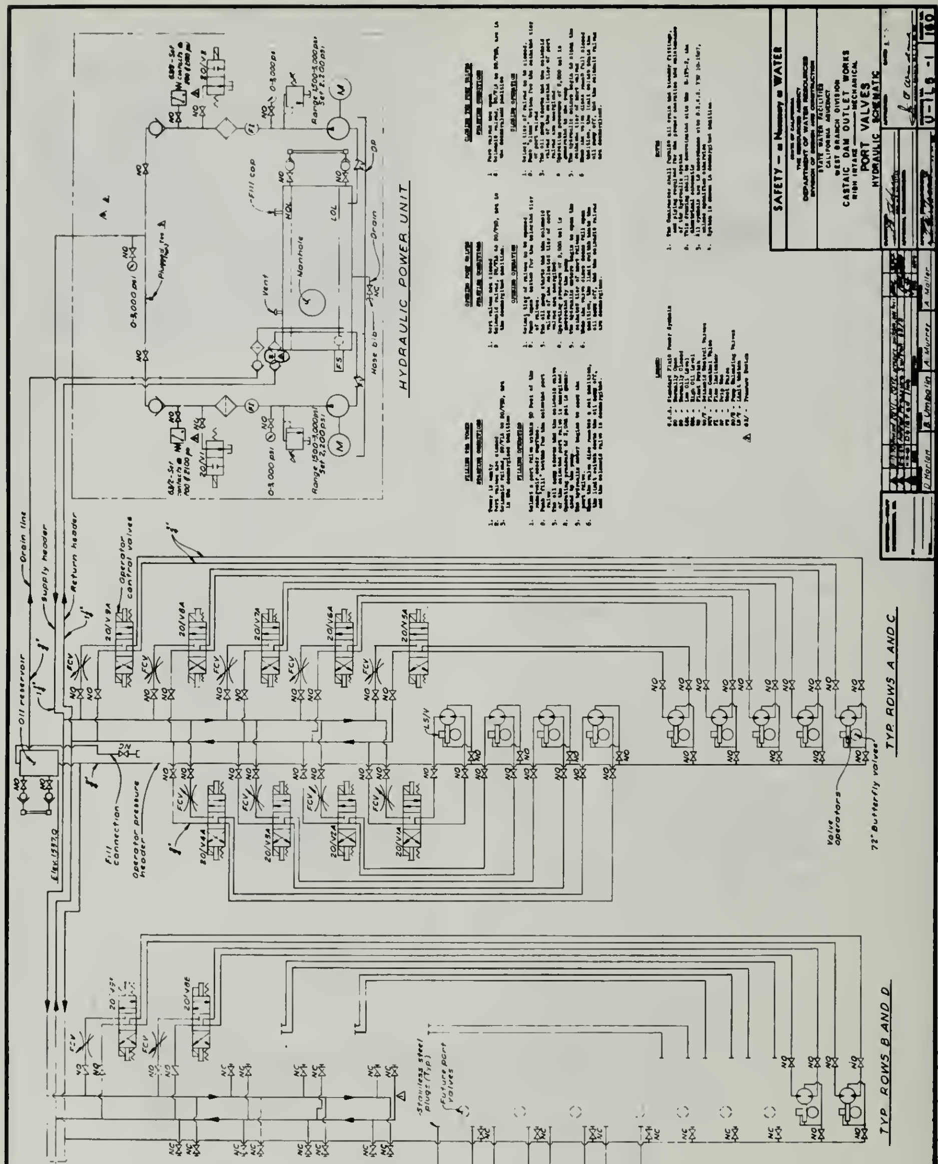

Figure 366. Part Valves—Hydraulic Scheme

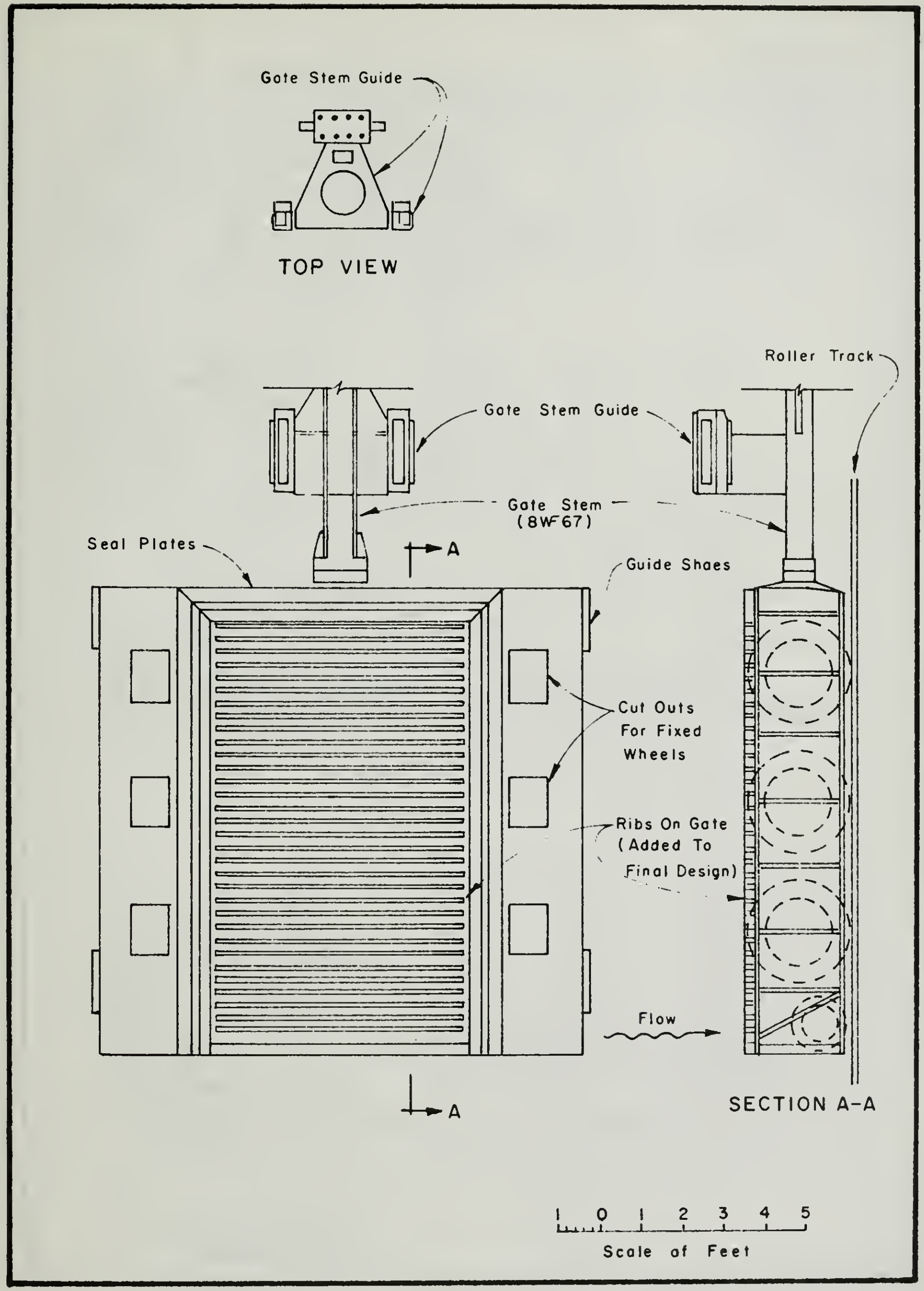

Figure 367. Low Intake Gate

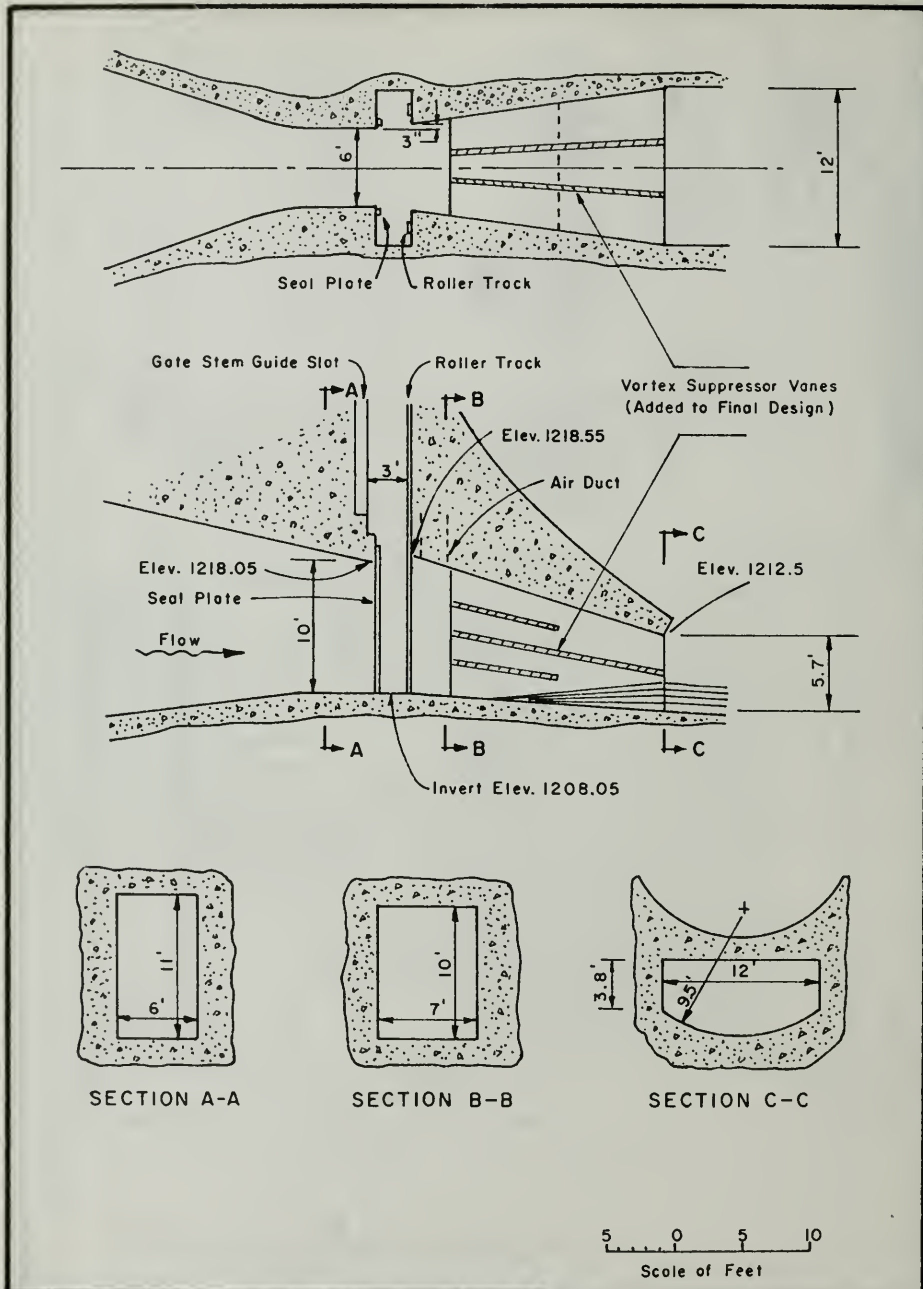

Figure 368. Low Intake Sluiceway

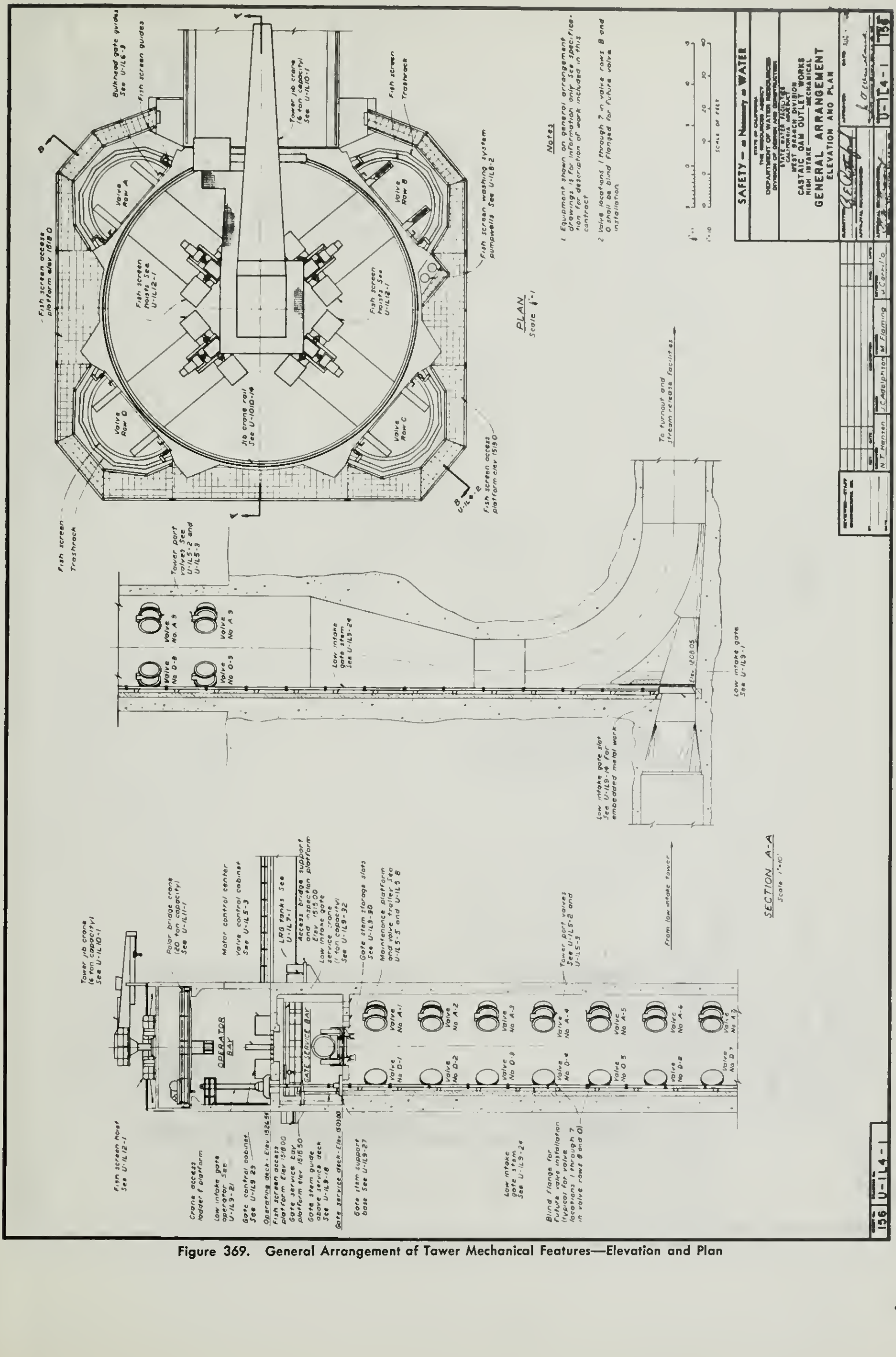

Figure 369. General Arrangement of Tower Mechanical Features—Elevation and Plan

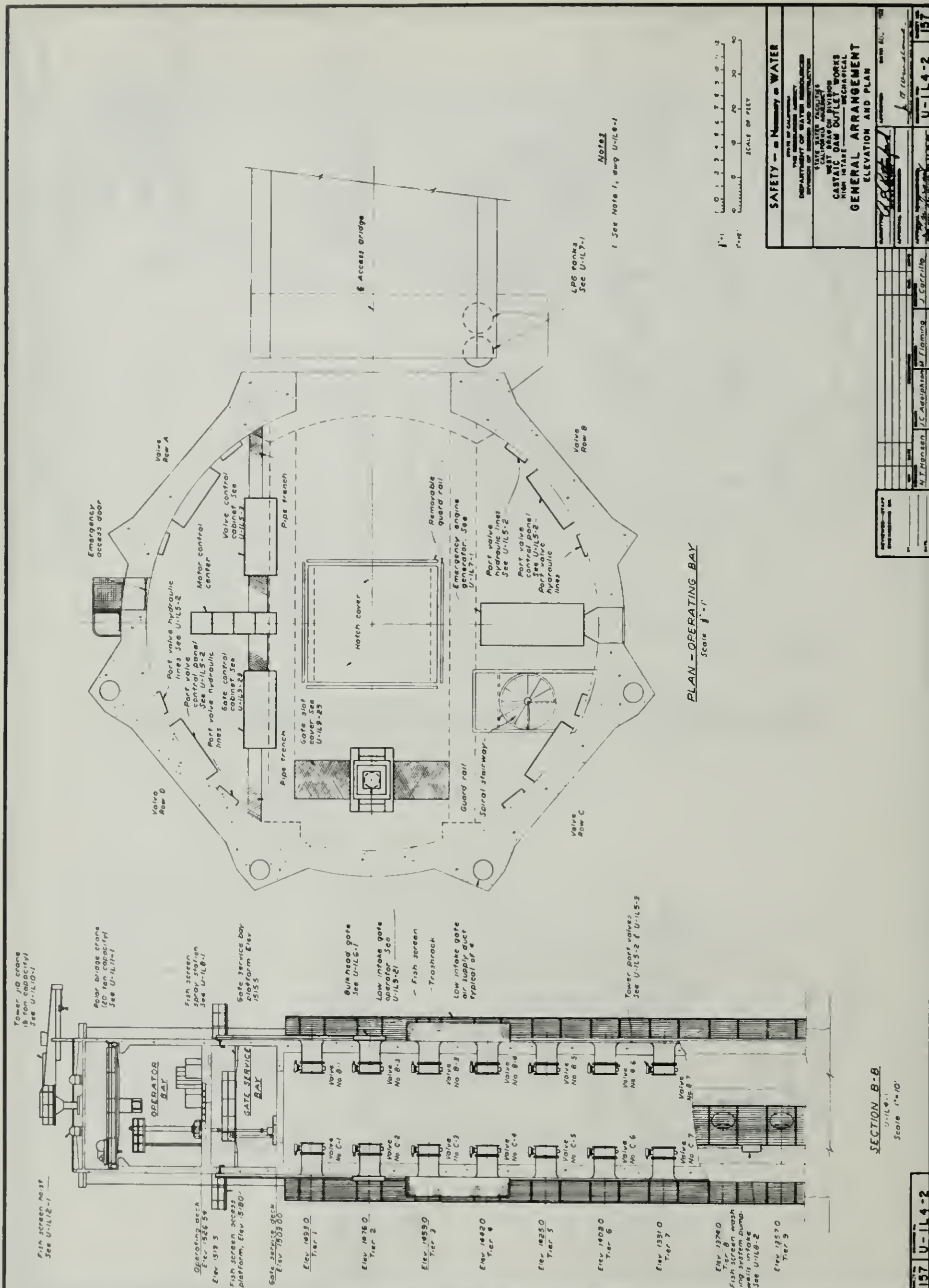

Figure 370. General Arrangement of Tower Mechanical Features—Elevation and Plan (Continued)

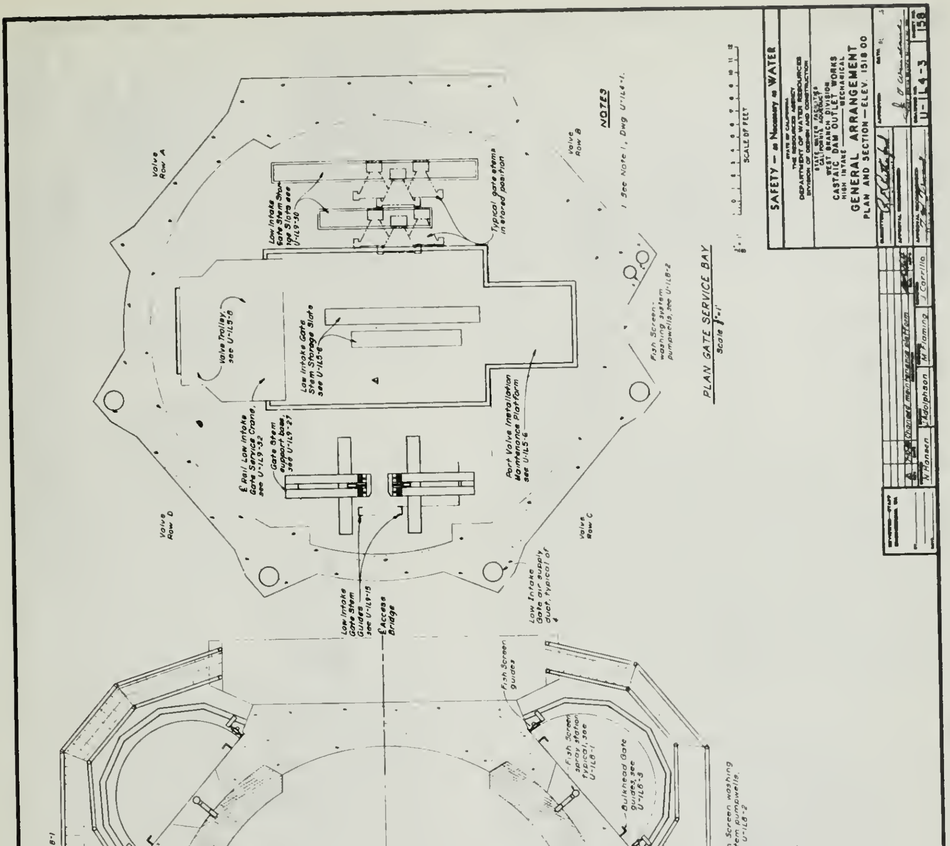

Figure 371. General Arrangement of Tower Mechanical Features—Plan and Section

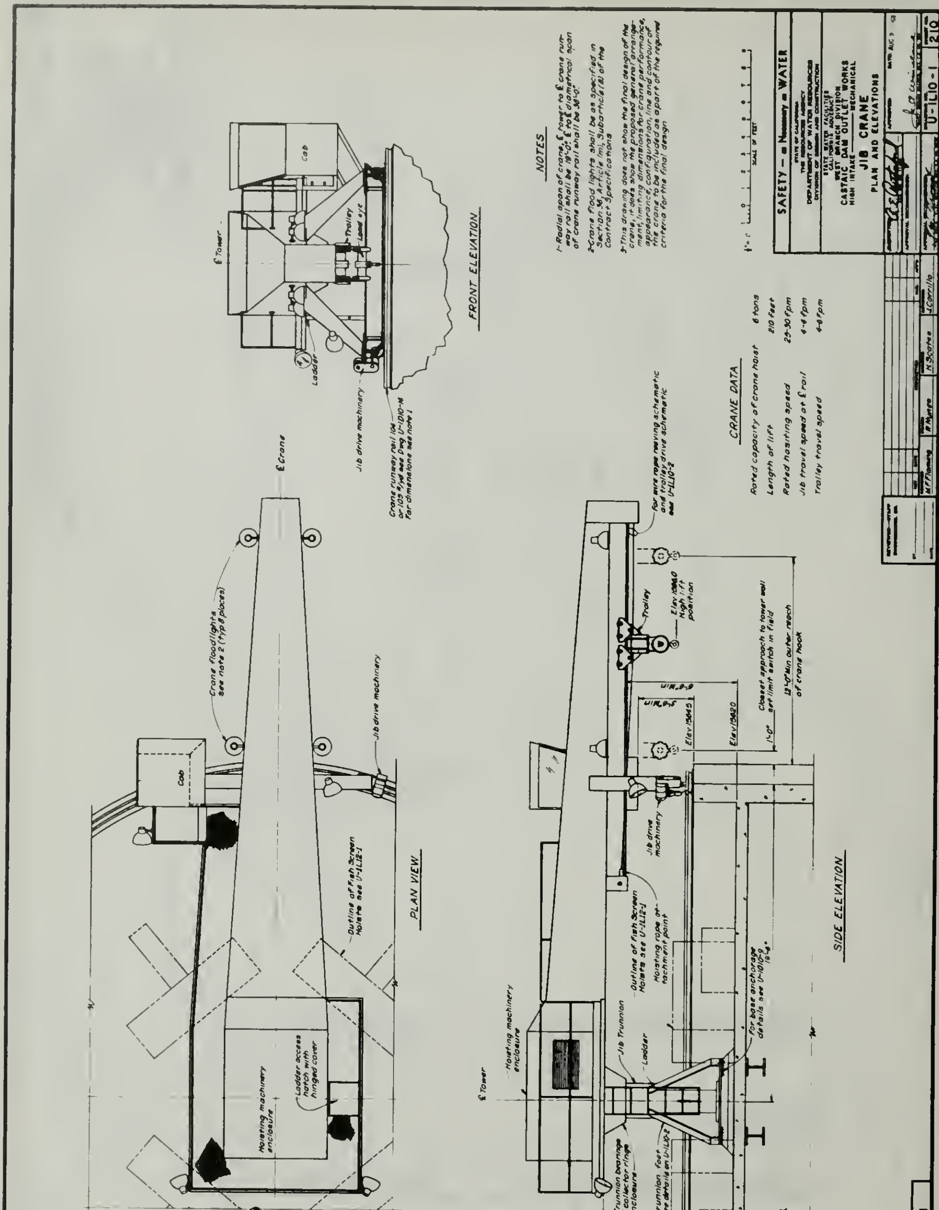

Figure 372. Tower Jib Crane

A 20-ton-capacity, electric, cab-operated, indoor, overhead, revolving, Polar bridge crane was installed in the outlet works high intake tower operating bay (Figure 373). The crane revolves on a rail anchored to the inside tower wall. The primary use of the crane is to install, remove, or perform maintenance on the tower port valves and valve operators, and to position the valve maintenance platform. The crane also is used for low-level gate removal, low-level gate hydraulic-cylinder servicing, and handling of miscellaneous equipment in the operating bay. The motors for the hoist, bridge, and trolley travel controls are of the same type as previously mentioned for the jib crane and were designed so that the hoist may be operated at full load simultaneously with the operation of the trolley and bridge drives. Capacities and speeds are as follows.

Rated capacity of crane, tons	20
Number of trolleys	1
Rated capacity of hoist, tons	20
Rated hoist speed, fpm, with maximum working load	1–10
Bridge travel speed with maximum working load:	
Bridge, rpm	0.47–0.70
Truck speed at wheels, fpm	50–75
Trolley travel speed, fpm, with maximum working load	20–30
Diametric span, feet	35
Length of lift, feet	200

The maintenance platform consists of a portable maintenance platform and trolley. Its main function is to aid in the servicing of the tower port valves and any maintenance work in the tower below the gate service deck. Its secondary function is to aid in the storage of the low intake gate stems when the platform is stored.

Four 7½-ton-capacity fixed hoists are provided to position the fish screens in front of the tower port valves. The hoists are located on the top of the high intake tower adjacent to the jib crane. Each hoist has a lifting speed of 9 to 10 feet per minute and a maximum lift of 175 feet. Each hoist consists of a parallel shaft gear reducer, gear motor, magnetic brake, and two hoist drums with stainless-steel wire ropes. The control for the fish-screen hoist was designed for local-manual or remote-automatic operation.

Two structural-steel bulkhead gates are provided to facilitate removal or repair of the high intake tower port valves. The gate system is composed of two bulkhead gates, a lifting beam, four gate guides, and gate seal plates mounted around each valve port (Figures 374, 375, and 376). The gate body was constructed of six W12X106 (12WF106) beams welded to a skinplate. The gate was designed to seal at all depths up to 168 feet by the use of a counterweighted linkage system which extends or retracts dogging bars. The lifting beam also has a counterweighted linkage system to operate its hooks so that the lifting beam can be automatically attached or detached from a gate in its parked condition. The gate must be installed or removed from the tower under a balanced head condition.

The two emergency engine-generator sets are liquefied petroleum gas-fueled. One set provides emergency power for equipment in the high intake tower, and the other set provides emergency power to the stream release and turnout facilities. The engine-generator set for the stream release and turnout facilities is a continuous-duty rated unit of 100 kW. The high intake engine-generator set is a continuous-duty rated unit of 75 kW.

Generator	*Tower*	*Stream Release and Turnout*
Standby Power	85 kW	115 kW
Continuous Power	75 kW	100 kW
rpm	1,800	1,800
Voltage Regulation	± 1%	± 2%

The fish-screen washing system uses reservoir water to clear away debris attached to the four fish screens on the high intake tower. The system consists of two 165-gpm, 400-foot total dynamic head, submersible pumps and a piping manifold system supplying water to four spray stations and four hose valves. The washing system manifold was installed just below the operating deck. The pumps were installed in two 12-inch wells in the high intake tower walls at a distance of approximately 150 feet below the supply manifold. The spray nozzles were selected by testing several types. The type chosen provided a flat spray with a 15-degree spray angle.

Turnout Guard Valves. All turnout guard valves are adjustable, steel-seated, butterfly valves. The 42-inch, 78-inch, and 132-inch valves are operated by hydraulic cylinders. The 30-inch valve is electric motor-operated. Control for each valve is provided remotely at the Castaic Area Control Center and locally at the valve hydraulic control cabinet. These valves are designed to operate under the following conditions:

Valve Size (inches)	*Maximum Static Head (feet)*	*Emergency Maximum Flow (cfs)*	*Operating Time (minutes)*	*Total Downstream Head at Maximum Flow (feet)*
132	322.9	6,600	10	271
78	322.7	2,300	5	301
42	323.1	1,050	5	188
30	323.2	340	5	306

The valves normally are in the fully open or fully closed position. The 132-, 78-, and 42-inch valves are opened under a balanced head condition by means of a bypass system.

The operators used on the valves were based on torque required during emergency closure. The higher torques required by the 132-, 78-, and 42-inch valves were suitable for hydraulic cylinder operators. The motor operator for the 30-inch valve is a worm gear type with handwheel.

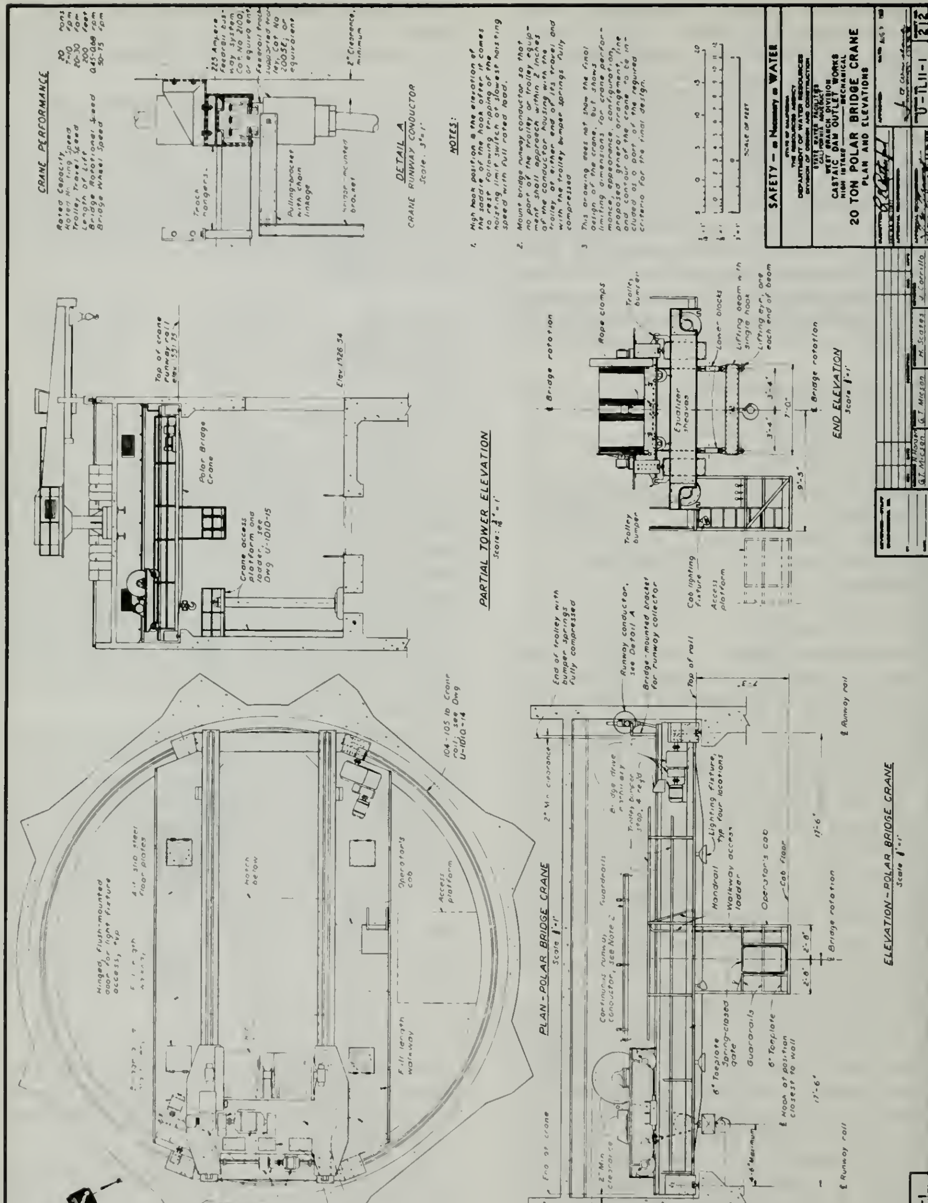

Figure 373. 20-Ton Polar Bridge Crane

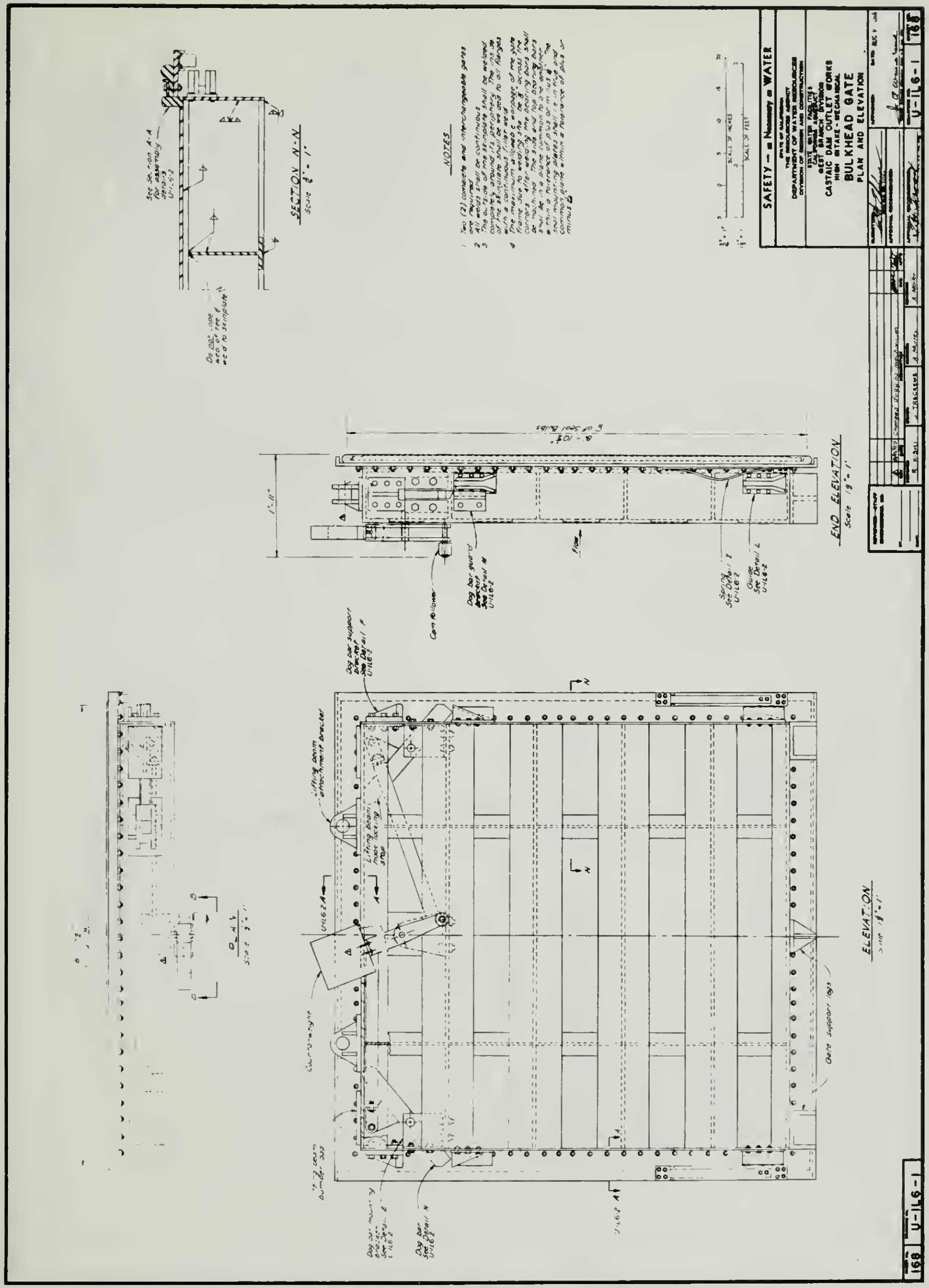

Figure 374. General Arrangement of Bulkhead Gate

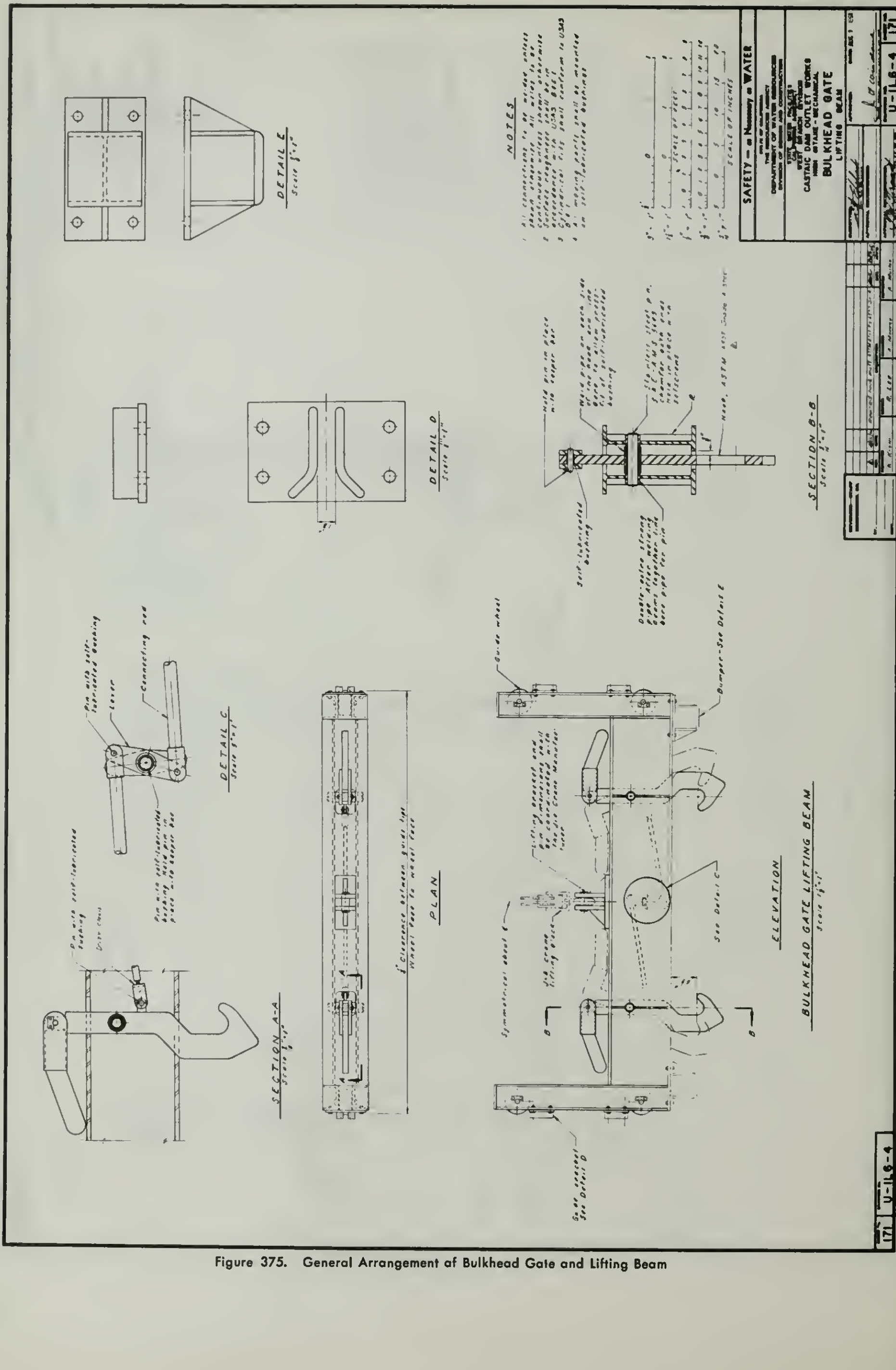

Figure 375. General Arrangement of Bulkhead Gate and Lifting Beam

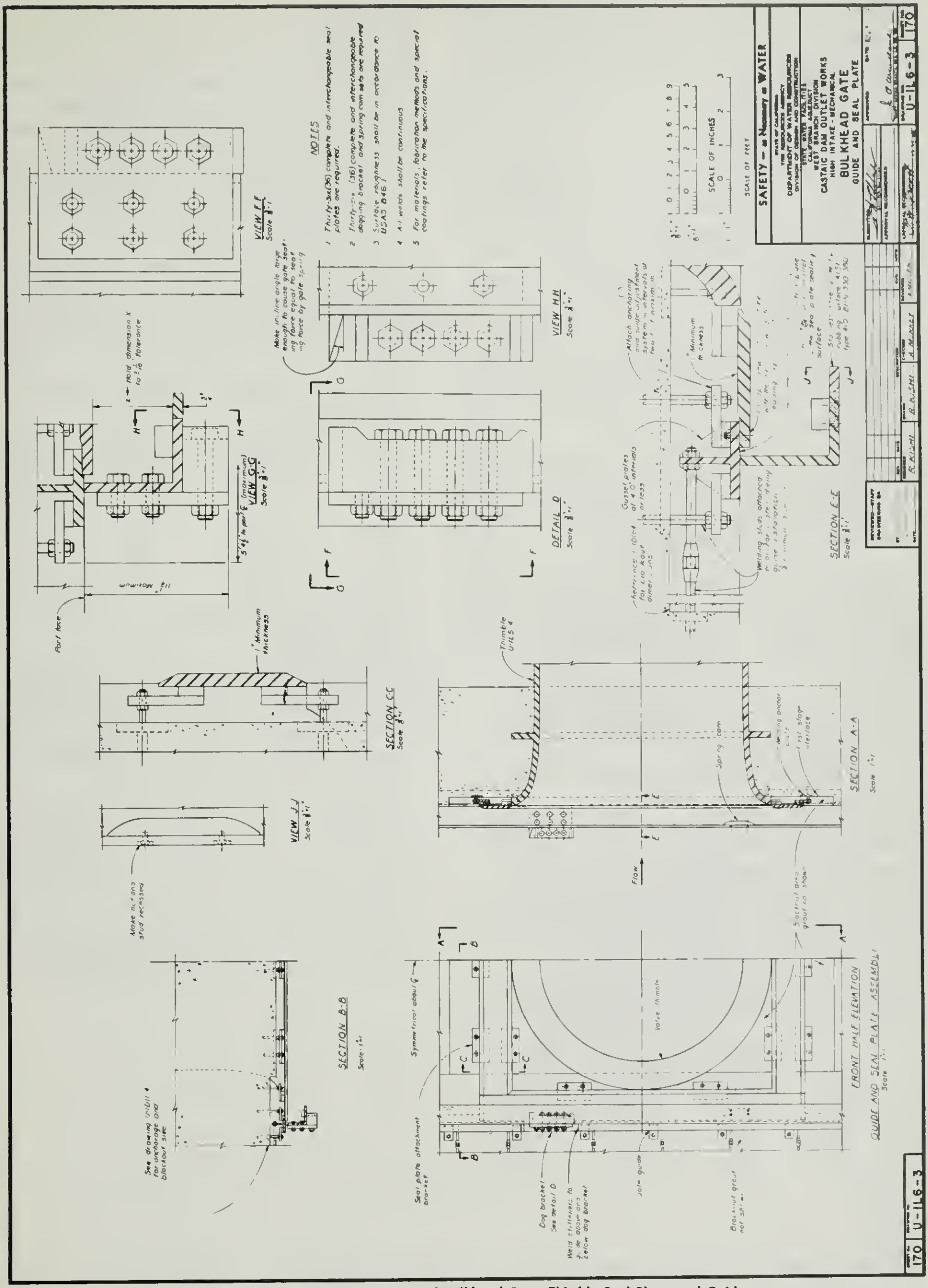

Figure 376. General Arrangement of Bulkhead Gate, Thimble Seal Plate, and Guide

The hydraulic power for each cylinder-operated valve is provided by its own system. Each system is housed in a control cabinet installed in its own valve vault. The hydraulic power system for each of the 132-, 78-, and 42-inch valves are pump-operated at a pressure of 2,000 psi (Figure 377).

The turnout valves are to be locked whenever they are in the open or closed position. In the open position, the 132-, 78-, and 42-inch valves are locked to overcome the closing tendency of butterfly-type valves by interference, fitted, clamping sleeves which are released by each valve's hydraulic power system prior to closing the valve. The holding power is equal to 25% of the maximum force of the hydraulic cylinder. In the closed position for the 132-, 78-, and 42-inch valves, a hand-operated mechanical lock is used to effect a holding power greater than the maximum force of the hydraulic cylinder. The 30-inch valve is locked in the open or closed position as a consequence of the worm gear-type operator.

Stream Release Facilities. The stream release facilities (Figure 378) consist of five discharge regulating systems installed on or within vaults in the concrete facility structure. The structure is located at the end of the 228-inch penstock as shown on Figure 379.

Two 96-inch, one 30-inch, and one 10-inch fixed-cone dispersion valves and one 8-inch pressure-reducing valve are provided as discharge regulating valves.

The guard valves provided for the stream release system are:

1. A 114-inch valve installed upstream of each 96-inch fixed-cone dispersion valve.
2. A 42-inch valve installed upstream of the 30-inch fixed-cone dispersion valve.
3. An 18-inch valve installed upstream of the 10-inch fixed-cone dispersion valve.
4. An 8-inch manually operated plug valve installed upstream of the 8-inch pressure-reducing valve.

Stream Release Regulating Valves. Each fixed-cone dispersion valve is controlled by its electric motor operator and gear reducer. Each operator can be controlled locally in the gate chamber and remotely from the Castaic Area Control Center.

The fixed-cone dispersion valves were designed in accordance with the following requirements and design criteria:

Valve Diameter (inches)	*Maximum Static Head (feet)*	*Minimum Total Discharge Head (feet)*	*Minimum Discharge @ Minimum Total Dynamic Head (cfs)*	*Opening and Closing Time (minutes)*
96	350	111	3,000	10.9
30	350	149	337	4.0
10	350	162	39	1.0

The critical dimensions for the net discharge areas of the valves are based on a discharge coefficient of 0.8. Special features were included in the design of these valves to provide additional stiffness in the valve body and ribs. These features were incorporated to prevent vibration and progressive failure of ribs which have been previously experienced at other facilities.

An 8-inch pressure-reducing valve was installed in the stream release facility, which discharges 0.5 cfs at a minimum total head of 156 feet and 8 cfs at a maximum total head of 320 feet.

Stream Release Guard Valves. The 114-, 42-, and 18-inch guard valves are butterfly valves. The 114-inch valve is hydraulic cylinder-operated and the 42- and 18-inch valves are electric motor-operated. The valves were installed in vaults inside the stream release structure.

The valves are required to operate under the following conditions:

Valve Size (inches)	*Maximum Static Head (feet)*	*Maximum Flow (cfs)*	*Operating Time (minutes)*
114	322.9	5,500	10
42	323.6	490	5
18	314.5	58	5
8	320.0	8	—

The function of the valves is to shut off flow to the fixed-cone dispersion valves to allow their maintenance and repair.

The stream release valves are locked whenever they are in the open or closed position. The locking system for the 114-inch valves is the same as that for the cylinder-operated turnout valves. The 18- and 42-inch valves are locked in the open or closed position due to the self-locking worm gear operator.

The operators used on the valves were based on torque required during emergency closure. The higher torques required by the 114-inch valves were suitable for the use of a hydraulic cylinder operator. Motor operators for the 18-inch and 42-inch valves are a worm gear type with handwheel.

The hydraulic power for both 114-inch valves is provided by a single pump-operated system with an operating pressure of 2,000 psi (Figure 380).

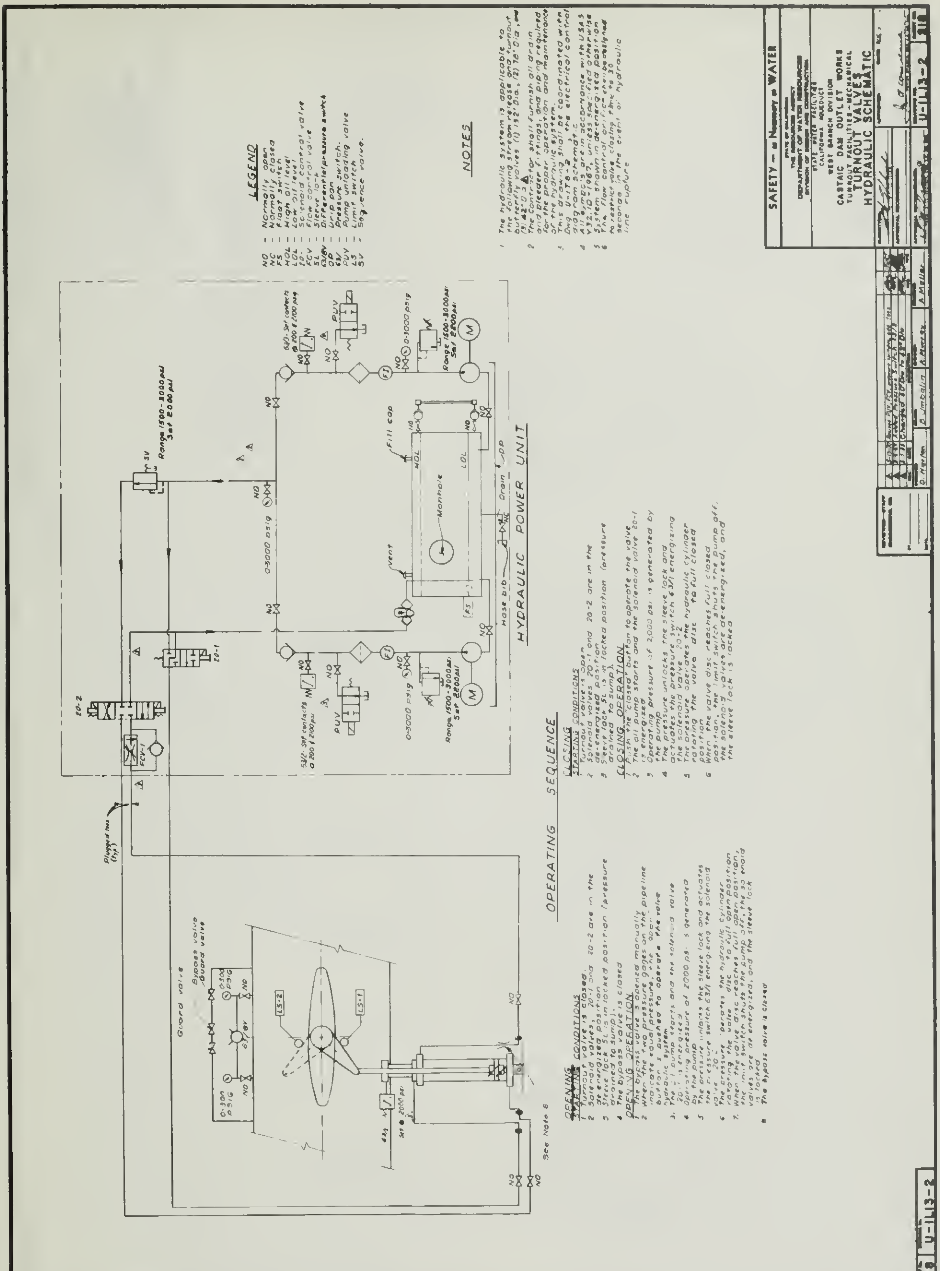

Figure 377. Turnout Valve—Hydraulic Schematic

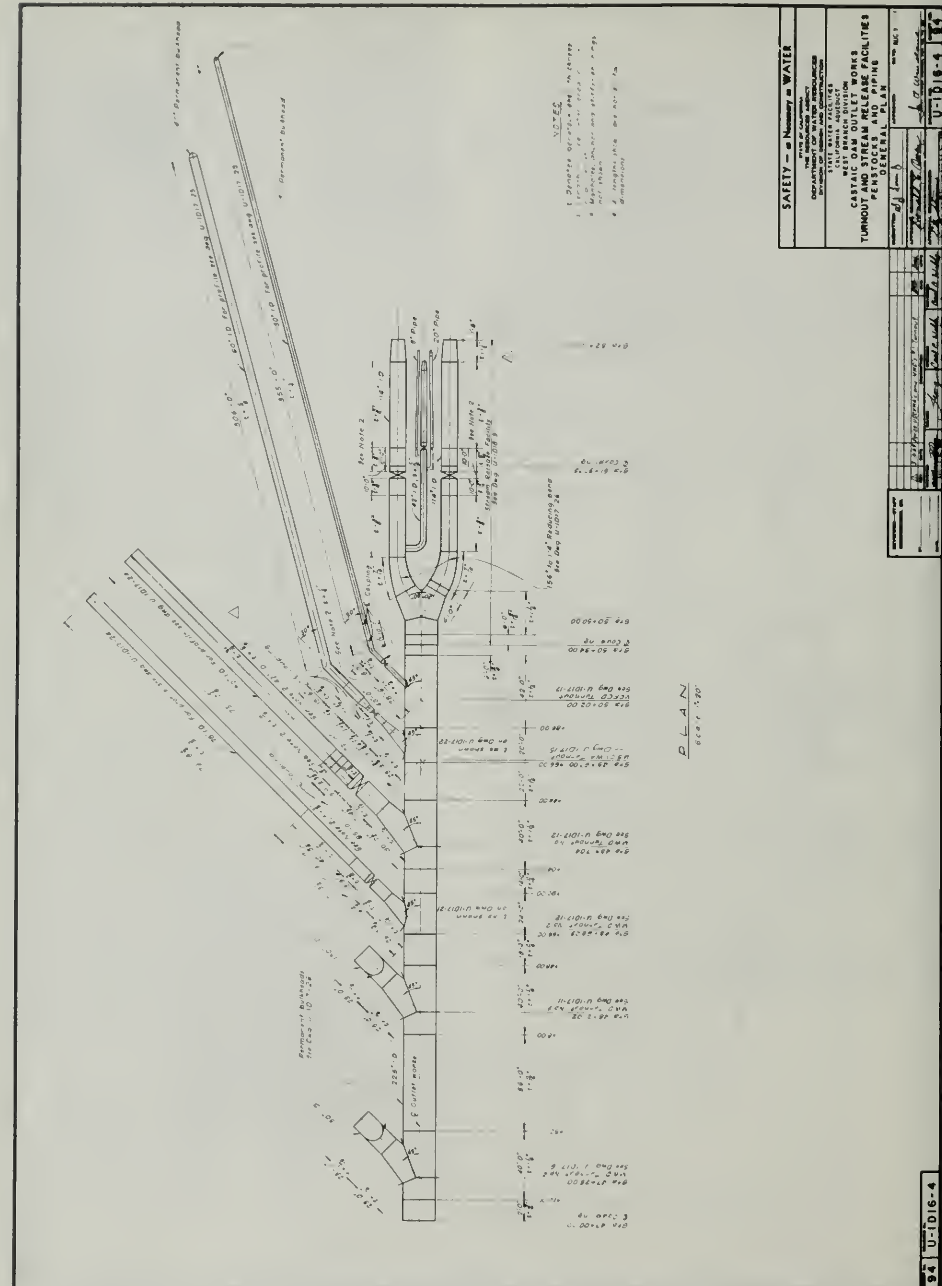

Figure 378. General Plan of Stream Release and Turnouts

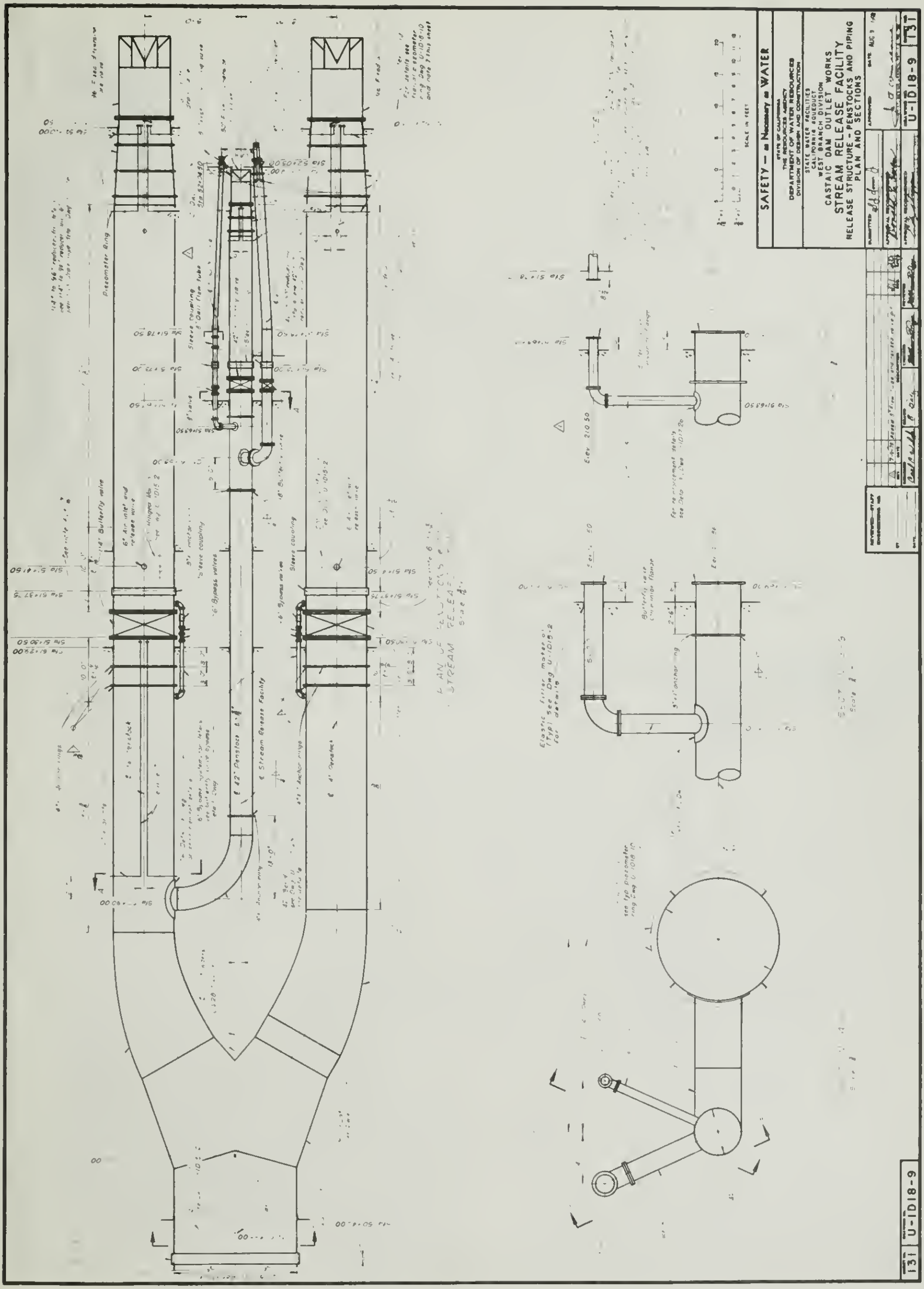

Figure 379. Stream Release Facility—Plan and Sections

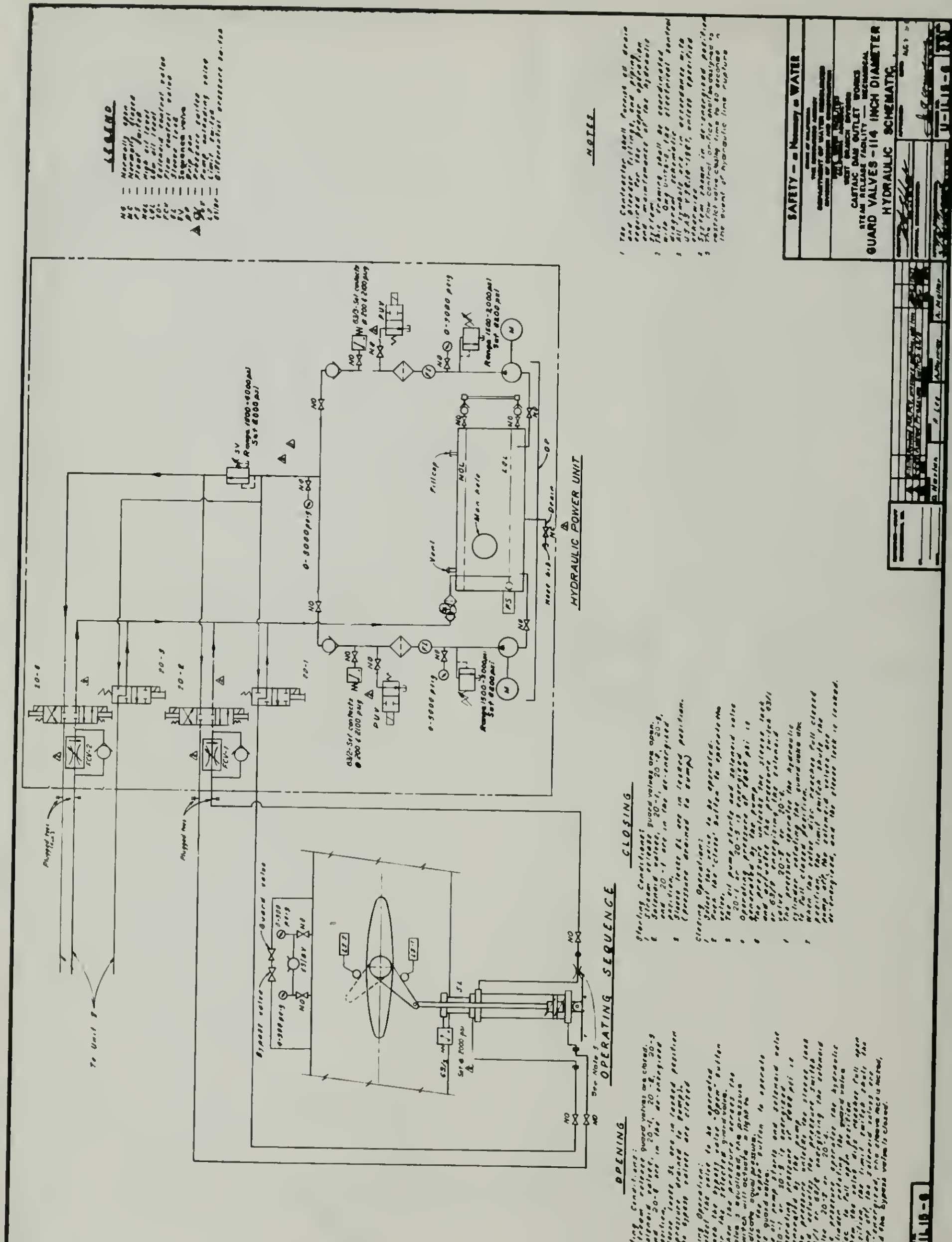

Figure 380. Hydraulic Schematic

Construction

Contract Administration

General information about the major contracts for construction of Castaic Dam and appurtenances is shown in Table 45. Most of the contract work was included in three large contracts. All contract work was administered by the Castaic Project Office located near the Dam.

Foundation Trench

Between August and December 1965, in advance of the large contracts, a trench was excavated to bedrock in the vicinity of the dam axis. Approximately 475,000 cubic yards of material was removed in forming a combination foundation excavation and exploration trench.

Diversion Tunnel

Open-Cut Excavation. Excavation for the outlet channel and the stilling basin for the diversion tunnel began in April 1966. It involved the excavation of a bowl-like basin in which the south portal, outlet channel, and stilling basin were situated. Materials from ripping and scraping operations were disposed of in spoil areas with the exception of selected materials, which were stockpiled near the Dam site. Small landslides were encountered in the east-slope excavation where aquifers or shears were intercepted.

The upstream portal approach was excavated to elevation 1,200 feet and to a point where the tunnel cover was well within the Castaic formation. The excavation was 50 feet below original ground with a floor width of 90 feet.

Miscellaneous excavations were performed for footings for the portal protective structures, keyways beneath the sloping outlet channel footings, and a cutoff trench that was dug around the stilling basin and backfilled with impervious fill. A dragline and scraper were used to excavate 85,000 cubic yards of material.

Underground Excavation. The north (upstream) tunnel heading was excavated to a 23-foot-diameter and finished to a 19-foot-diameter circular section. The south (downstream) heading was excavated to a 30-foot diameter and has a finished section with a 27-foot diameter. In the vicinity of the transition section between the north and south headings, trenches for nine cutoff collars were excavated.

North heading tunnel excavation was done by the top heading method after completion of the portal protective structure. Drilling was accomplished from a jumbo by four pneumatic auger drills with carbide-insert dragline bits. An imbalance in the loading of the holes, lack of relief cut in the center of the face, and concentration along the lower rib sections contributed to larger than normal overbreak and perimeter shattered areas.

Construction of a reinforced-concrete junction section for the high intake structure was provided by enlarging the tunnel diameter by 2½ feet from Station 20+68 to Station 20+16. From Station 20+31 to Station 20+46, the roof was enlarged an additional 4 feet (Figure 341).

Final excavation operations provided for a semicircular invert section to final rough grade. It left cut rock benches along both sides as foundation for support posts. A 4-inch-thick, concrete, mud slab was

TABLE 45. Major Contracts—Castaic Dam and Appurtenances

	Specification	Low bid amount	Final contract cost	Total cost-change orders	Starting date	Completion date	Prime contractor
Foundation Trench	65-42	$398,620	$412,269	$11,383	8/11/65	12/22/65	Tomei Construction Co.
Castaic Dam Diversion Tunnel	66-16	8,580,940	12,896,281	465,826	4/ 9/66	10/ 4/68	Peter Kiewit Sons' Co.
Castaic Dam and Reservoir	67-20	43,389,724	63,681,552	7,576,510	5/19/67	11/ 8/71	Western Contracting Corp.
Castaic Project Facilities	67-28	1,080,406	1,121,834	42,945	6/22/67	5/ 4/68	Montgomery Ross Fisher, Inc.
Castaic Dam Outlet Works	68-31	13,725,362	15,457,729	1,750,360	2/17/69	6/ 9/73	Polich-Benedict Constructors
Soil Stress Measuring and Recording System for Castaic Dam	68-37	61,275	63,316	--	10/ 8/68	8/ 5/69	Aerojet-General Corp.
Strong Motion Acceleration Monitoring Systems (Incl. Wheeler Ridge and A.D. Edmonston Pumping Plants)	68-38	106,382	144,673	23,985	10/ 8/68	3/ 8/73	Slope Indicator Co.
Drainage and Paving for Castaic Dam	71-21	335,415	443,065	56,892	9/27/71	4/20/72	F. W. Richter Assoc.
Completion of Castaic Dam	73-15	666,186	796,336	37,558	6/25/73	5/24/74	Altfillisch Construction Co.

placed over the invert to protect the rock from slaking and softening.

In the course of excavating the top heading, structural support distortion and one major cave-in occurred as a result of improper blocking and cribbing, softening of blast-shattered foundation rocks by water seepage, and weight of the ground above.

Pervious Backfill and Riprap Placement. Pervious backfill consisted of carefully selected rock, 100% passing No. 8 screen and 1% passing No. 4 screen. It was spread into striplike lifts adjacent to the walls of the stilling basin with a crawler tractor followed by at least five passes of the same equipment. A total of 2,600 cubic yards of quarried riprap was placed over the top of the compacted backfill.

Structural and Tunnel Lining Concrete. A protective coating was needed on excavated surfaces which were to receive concrete to prevent breakdown and sloughing of the structure foundation material when it became exposed to air and water. A 3-inch-thick, concrete, protective slab was placed over excavated foundation, except for the steeper slope above the stilling basin where pneumatically applied mortar was employed. The actual thickness of the protective slab varied considerably because of problems in excavating a smooth surface in the dipping and folding planes of shale and sandstone. Consequently, there was a significant overrun in the quantity of concrete used.

Most of the structural concrete was used in construction of the outlet channel structure. It is basically a rectangular channel with a transition from the circular tunnel to a rectangular cross section, a transition in the size of the channel, and a vertical curve to a steep slope which terminates at a large rectangular energy dissipator. Floor slabs are as much as 6 feet thick, and the 52-foot-high walls are over 8 feet thick at the base.

The tunnel, including portal structure, consists of a 1,756-foot section having a 19-foot inside diameter and a 2,010-foot section having a 27-foot inside diameter. The entire tunnel lining contains steel reinforcement. Spacing was sufficient to permit the use of a standard, 1½-inch maximum size aggregate, lining mix except in the area of the high intake shaft intersection, where spacing of bundled bars required a mix containing ¾-inch maximum size aggregate. A total of 64,868 cubic yards of concrete was placed.

Diversion and Care of Stream

Dewatering operations started May 25, 1967, with pumping from the dam foundation exploration trench. This trench, excavated to bedrock on a previous contract, crossed the stream channel in the vicinity of the dam axis (Figure 381). As the water table was lowered, dam foundation excavation was extended outward from the exploration trench. Ditches at the bases of cut slopes diverted ground flows to sump holes where the water was pumped and discharged downstream. The low summer flows of 1967 from Castaic and Elizabeth Lake Canyon Creeks were intercepted upstream of the Dam site and piped across the excavation areas.

In August 1967, the creek flow was diverted into a diked section built along the west side of the canyon floor. A ditch and dike were provided just upstream of the Dam to divert Elizabeth Lake Canyon flows to the right abutment. Construction of an intermediate channel, with a bottom width of approximately 40 feet, along the right abutment was started in early November 1967. Runoff from rains in the last part of November overtopped this channel and flooded the lower parts of the dam foundation excavation. The system was adequate for the remainder of the 1967–68 wet season.

Early in 1968, as the upstream Zone 3 foundation in the channel excavation area was exposed, a ditch to intercept ground water was dug along the base of the upstream toe of the dam excavation slope.

Water was pumped and piped from sumps within the ditch. As the embankment was built, the ditch and a section immediately above were backfilled with cobbles encased in a well-graded drain material to create an aquifer. Riser pipes with deep well pumps were installed in the cobbles. This arrangement kept upstream seepage from reaching most excavation areas and provided a reliable water source for construction purposes.

In October 1968, the diversion tunnel was available for the contractor's use.

Ditching was done to dewater material in the Lagoon. A channel section also was excavated at the Lake Hughes Road Bridge to facilitate the flow of water by that area.

Figure 381. Foundation Exploration Trench

Castaic Canyon pervious borrow was dewatered by ditching and sump pumping. A double-barreled culvert of open-ended, railroad, tank cars was installed between the diverion tunnel inlet and Castaic Canyon borrow pit operations to handle large flows of water.

Between January 18, 1969 and January 29, 1969, 15.17 inches of rain fell in the Castaic area. The peak floodflow through the project area was estimated to be 18,000 cfs. Erosion and silting occurred at the Dam site and locations in the Lagoon. Landslides occurred in various areas, and heavy erosion occurred under the Lake Hughes Road Bridge at the south end of the Lagoon.

Rainfall continued to delay the work during February 1969. An additional 11.87 inches of rainfall was recorded during that month. The floodflow through the project area at the end of February peaked at 32,000 cfs. The damages sustained during the January flooding were increased. A section of the Lake Hughes Road Bridge was destroyed, and extensive repairs were necessary (Figure 382).

The dam embankment was restored to prestorm conditions by May 1969, and the minimum embankment elevation of 1,315 feet was reached by November 1, 1969 in accordance with the specifications for flood protection.

During the 1969–70 wet season (December through March), the contractor took precautions in the left abutment area by diking, sandbagging, and ditching to minimize aggravation of landslide conditions by storm runoff and to minimize interference with the work being conducted there. Dewatering operations of the upstream borrow areas continued by ditching and sump pumping to the diversion tunnel. Late rains of the 1969–70 season ponded in the excavation areas of Castaic Canyon below the tunnel invert. This water remained while operations continued in Elizabeth Lake Canyon.

The lagoon control structure was completed, and the low-level tunnel inlet was bulkheaded by the outlet works contractor prior to the 1970–71 rainy season. Storms at the end of November 1970 flooded borrow operations upstream of the Dam. Continued releases from runoff and impounded water in the reservoir threatened a long-term inundation of the lagoon area. The gates to the diversion tunnel were closed, and no further releases were made from the reservoir during the contract since the embankment was high enough to contain the inflow.

The pervious borrow operations were confined to the only remaining borrow area, the Lagoon. This necessitated deeper excavations in this area than originally planned. Ditches and the sump at the south end were deepened and higher head pumps installed.

Dam Foundation

Excavation. Excavation for dam foundation included removal of all streambed sands and gravels, landslide material, terrace deposits, soil deposits, and weathered rock required to reach competent foundation material (Figure 383). The extreme upstream portion of the Dam within the stream channel was to rest upon streambed sands and gravels excavated to elevation 1,190 feet leaving about 60 feet of sand and gravel but, when excavation reached this elevation, large silt deposits were discovered. These deposits were removed under a change order, exposing bedrock.

Foundation excavation began at the easterly side of the channel section immediately north of the explora-

Figure 382. Flood Damage to Lake Hughes Road Bridge

Figure 383. Dam Site at Beginning of Work

tion trench excavated under the contract for Castaic Dam foundation trench. Scrapers were used for most of this excavation, aided by a dragline in areas inaccessible to the scrapers. The dragline also was used to construct sump holes for dewatering the foundation. Streambed sands and gravels were stockpiled for later use in embankment construction and, as embankment areas became available, the excavated streambed sands and gravels went directly to Zone 3 dam construction. Other material went to waste piles and haul-road construction.

As previously discussed, storms and inadequate stream diversion at the end of November 1967 caused flooding of the low portion of Zone 1 excavation. The area required pumping and truck removal of muck (thick, saturated, shale, foundation tailings) for several days. Final removal of muck was done by pumper trucks of the type used in oil fields. The interceptor system at the upstream toe of the Dam, completed during January 1968, eliminated the need for the dewatering sumps within the channel area and excavation of the channel section was advanced toward the right abutment.

Above streambed level, the excavation procedure established by the contractor was to start successive cuts within the abutment areas which would intercept the dam foundation at predetermined elevations to coincide with his embankment construction scheduling. Major problems encountered during the foundation excavation were (1) the uncovering of an existing slide mass not previously detected at the base of the upstream right abutment, (2) the easterly sliding movement of a triangular-shaped block of Castaic formation in the right abutment above the aforementioned slide mass (Figure 384), and (3) the massive slides at the left abutment (Figures 385 and 386).

It was determined that the slide mass at the base of the upstream right abutment required removal. This was initially accomplished with a scraper operation. As excavation continued, the area at the base of the excavation became limited, and water seepage became a major problem because the slide mass extended below streambed level. The contractor utilized a dragline to excavate and load the remaining material and to progressively lower sump holes for dewatering. The lowest elevation reached in the slide removal area was about 120 feet below streambed. The base of the hole was cleaned, and filling with Zone 3 compacted embankment was started on January 13, 1969. About 50 feet of the excavation had been backfilled to approximate elevation 1,150 feet by January 20, 1969, when heavy rains started. Early attempts to dewater the slide excavation were futile. Rain continued periodically, completely inundating the dam embankment area and heavily silting the slide excavation area, burying the dragline and pumping equipment in the hole. Between March 21, 1969 and April 16, 1969, a dredge pumped silt to an area upstream of the Dam in the lower reach of Castaic Canyon. Dewatering of the hole and removal of remaining sediment was started April 16, 1969 and was finished May 23, 1969. On May 29, 1969, backfilling of the slide excavation area was completed.

The triangular-shaped block of Castaic formation, between Stations 25+00 and 26+80, on the right (west) abutment was sliding easterly along a horizontal clay seam at elevation 1,200 feet. It was decided to excavate at higher elevations to relieve the driving force and to remove a portion of the block.

Excavation of a portion of the sliding block was accomplished between July 17, 1969 and August 7, 1969. Much of the material excavated was clean hard sands and gravels and was utilized directly in the adjacent Zone 3 embankment construction. Zone 3 embankment then was rapidly constructed to elevation 1,250 feet. Grouting was performed to fill large voids

Figure 384. Block Slide at Right Abutment

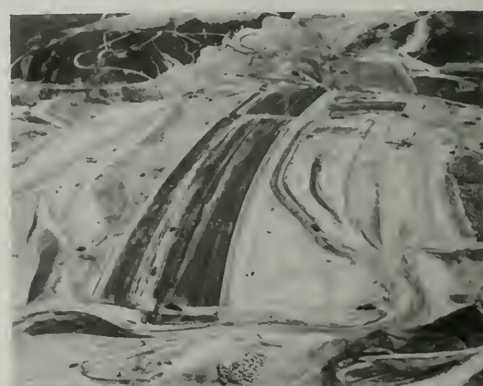

Figure 385. Dam Under Construction—View Across Dam Toward Left Abutment

Figure 386. Excavating Slide Area at Left Abutment

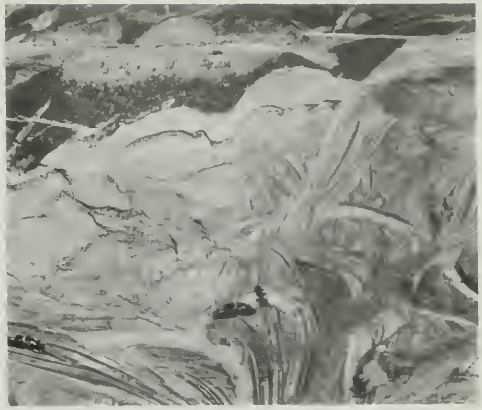

Figure 387. Grouting Through Embankment at Left Abutment—White Lines Show Limits of Core Zone

and stabilize the area around the slide block. A berm was built on the upstream toe of the Dam at the right abutment to elevation 1,300 feet to permanently stabilize this area.

After the first major slide occurred at the left abutment on April 26, 1969, slides became a continuing problem during excavation above and within the dam foundation of the left abutment. Slide scarps were sloped back, and overburden above the cuts was removed to stabilize the area. At times, it was evident that any work performed to advance the dam foundation at the embankment contact would precipitate sliding. In this situation, embankment was built to a predetermined elevation. Narrow sections of this material then were excavated to good foundation with a resulting buttressing on either side of the exposed foundation. The foundation was quickly prepared and permanent embankment placed. This process was repeated as many times as necessary to complete the embankment.

The equipment used to perform the major portion of excavation for dam foundation were scrapers and tractors for support and pioneering. Shaping of cut slopes was done primarily with a large tractor equipped with a slope board. Loaders with a 12-cubic-yard capacity were used during excavation of the channel area. A dragline with a 14-cubic-yard capacity was used extensively for this work from the time of arrival on the site through August 1968. The material from the loaders and dragline operations was transported by off-highway end-dump units.

Grouting. This work consisted of the construction of grout curtains beneath the Dam and spillway and for blanket grouting as necessary in the exposed core zone dam foundation.

Curtain grouting was used to provide a continuous barrier against leakage and to reduce hydrostatic pressures in the downstream foundation. Originally, it was planned to grout the main center curtain through a concrete grout cap. The cap was placed and used between dam Stations 39+65 and 46+00. However, grout leakage, uplift, and cracking of the grout cap occurred during grouting of the upper 10 feet of the foundation, even though very low pressures were used. Construction of the grout cap and grouting of the upper 10 feet of the foundation therefore were discontinued. Instead, a core trench 10 feet deep along the curtains was excavated for removal of the overburden after grouting operations were completed. The bottom width of the trench varied from 20 to 50 feet, depending on the number of grout curtains used and the geologic conditions. The trench was backfilled with Zone 1A embankment material.

Most of the grout cap was removed due to requirements of foundation shaping. Excavation for this shaping also made it necessary to deepen the outer grout curtains and sections of the center curtain between Stations 42+80 and 45+00.

Excavation of the core trench was discontinued in the east abutment at Station 52+20 because of slide conditions. Also, due to the need for buttressing excavated areas against sliding in the east abutment, as discussed previously, grouting was done through the embankment from Station 53+60 to Station 61+00.

Grouting of the spillway section was delayed to allow for rebound resulting from excavation in the area immediately adjacent to the approach channel. This grouting was done through pipe installed in the concrete at the spillway crest section.

Shallow grouting was performed in the core zone channel sections to restrict seepage from the dam foundation. Blanket grouting also was done through the embankment at the east abutment to consolidate a small area of foundation that had been disturbed by sliding (Figure 387).

An anticline at the easterly side of the channel section was treated with deep blanket grouting upstream and downstream of the grout curtain along the anticlinal axis. Additional holes on a closer spacing and angled holes were grouted at the intersection of the anticline and the grout curtain.

Foundation Preparation. Initial excavation for dam embankment foundation was cut to within 12 inches of final grade. Removal of this final 12 inches and cleaning of surfaces was limited to a period of two days, within which time embankment placement was started (Figure 388). This was necessary due to the rapid deterioration of the Castaic formation, especially the shales, when exposed to air.

In the filter and shell zones, final cleanup was often accomplished with motor graders and tractors with slope boards. At times, the cleanup for these zones was accomplished more conveniently by a grader.

Foundation cleanup for the core zone was more critical. A clean undisturbed surface was necessary to obtain a tight seal with the embankment material placed in direct contact. Gradalls produced the desired results. Slush grouting was used in several areas to fill voids in shallow, jointed, and fractured rock. Occasionally, a tremie pipe was used to assure penetration of grout into narrow deep (10 to 20 feet) voids. Upon completion of this grout treatment, the area was covered immediately with embankment contact material.

Handling of Borrow Materials

Impervious. Soils designated foundation contact for the core zone were selected from the high intake excavation, the spillway basin, portions of impervious borrow areas, selected portions of previously built embankment, and Borrow Area D. Borrow Area D was established for this purpose after dam construction had started and was located immediately north of the spillway approach channel.

Soils for the construction of Zones 1A and 1B came from required excavations mainly in the spillway, dam abutments, and designated borrow areas adjacent to the Dam and spillway. The areas known to have large deposits of acceptable material were prewetted by irrigation sprinklers. Additional wetting of impervious material was done in the stockpile by sprinklers and tanker trucks.

Excavation of the lower channel section of the spillway, starting in 1967, yielded large quantities of Zone 1 material lying above the Castaic formation. Approximately 700,000 cubic yards of this material was stockpiled in the lagoon area.

Borrow Area "T-South" immediately downstream of the Dam on the left abutment was left intact and deleted from the contract as a source of material, after the landslides had occurred. This was done because it was believed excavation in this area could jeopardize the stability of the ridge downstream of the left abutment and possibly portions of Lake Hughes Road. Castaic Ridge borrow area, Borrow Area "T-East" near the upstream left abutment, and excavations above the spillway cut designated spillway shaping west and spillway shaping east, were established to replace the material from "T-South". These areas were prewetted in the same manner previously discussed.

Filter and Drain. Material for Zones 2A and 2B was obtained from the lagoon and upstream Castaic Creek pervious borrow areas. Hauling to the separation plant was done primarily with off-road belly dumps powered by tractors. Other hauling units were used as needed.

Problems with grading of Zone 2B material were evident from the start of stockpiling at the plant and the first placements in the embankment. Huge removals, in relation to the amount placed in the embankment, were required because of improper grading. Actions taken to bring the grading within the specified range were as follows: decreasing the spacing on the grizzly to allow initial removal of oversize material; revising the screen sizings at the plant; installing a crusher to reduce oversize rock, thereby increasing the amount of intermediate rock sizes available; providing washing facilities; and reducing handling of the material to a minimum. The most satisfactory results were obtained when material was loaded from the plant hopper and taken directly to the embankment.

Pervious. The first sections of Zone 3 were placed using material that had been taken directly from foundation excavation and foundation excavation material that had been stockpiled. Sands and gravels excavated for placement of the seepage discharge line also were stockpiled for use in Zone 3 embankment areas. Excavation of pervious borrow areas started in the Lagoon during August 1968 and in Castaic Creek Canyon in

Figure 388. Scraper Spreading First Load of Embankment in First Approved Dam Foundation Area

November 1968, and continued from Castaic Canyon through November 1970.

Excavation of the pervious borrow areas was performed mainly with the large-capacity dragline and shovel loader. Off-highway end- and bottom-dump units were used for hauling material to the embankment. Belt loaders fed by dozers and other large loaders were used intermittently to load hauling equipment. Large scrapers also were used extensively at times for hauling to the Zone 3 embankment.

Random. The random section of embankment was constructed of material obtained from required excavations. Sandstones and terrace sands and gravels were used, the latter being preferred. Required excavation from the Lagoon which did not meet Zone 3 requirements, but was of good Zone 4 quality, also was utilized. Sands and gravels that underlaid the impervious material from Borrow Area D were allowed because it gave the contractor the benefit of a shorter haul than from stockpiled terrace sands and gravels.

Soil-Cement. Material for the production of soil-cement for the upstream-face slope protection was designated to be terrace sands and gravels from required excavation meeting specified grading standards. The contractor was permitted to use surplus Zone 2A material with the addition of fines; however, the addition of fines was discontinued at an early date, thus producing a stronger material.

The contractor's plant was a twin-pugmill continuous-mix plant. Material was stockpiled near the plant and fed into the hopper with a front-end loader. Material then was moved into the pugmill, where the correct amount of cement and water was added and the mixing took place. After mixing, material was lifted by a belt to a loading hopper, where it was transferred into bottom-dump trucks to be hauled to the upstream face of the Dam.

Figure 389. Loading of Pervious Borrow at Elizabeth Canyon

Downstream Cobbles. The material was obtained from the separation plant and from the stockpile of oversize material removed from the embankment areas. The contractor elected to place cobbles in a 2-foot-thick course rather than the specified 1 foot to facilitate placement, alleviate a greater selection process, and make greater use of the material available.

Embankment Construction

Impervious. Contact material was placed in a slightly wetter condition than the soils in the general core zone embankment. The purpose was to cause it to mold into irregularities in the foundation and thus seal effectively. The contact material was pushed into place by rubber-tired dozers from adjacent embankment or uncompleted foundation areas, leaving the prepared foundation undisturbed. Compaction was by wheel rolling with heavy equipment.

Hauling to Zone 1A and Zone 1B areas of the embankment was done by scrapers. Immediately following placement, front-end loaders equipped with rock buckets removed oversize material. The embankment was disked, and final moisture adjustments were made with tankers equipped with spray bars. Compaction was performed with self-propelled sheepsfoot rollers and a vibratory roller. The lift thickness was 6 inches after compaction. Twelve coverages by the rollers were required. Leveling between embankment lifts was done with motor graders and dozers.

Filter and Drain. Material for Zone 2A, the transition zone between the dam core and the other zones, was hauled to the Dam and placed from single- and twin-bowled power scrapers. Each 12-inch lift was leveled by a motor patrol, watered, and then compacted with two passes of a three-drum vibrating roller. The material from the plant consistently met grading requirements, and satisfactory compaction was never a problem.

Inspection trenching of Zone 2B material, the chimney and blanket drain, revealed that the finer fractions on the grading scale tended to migrate to the bottom of the lifts. Testing showed that in-place densities were far exceeding what had been expected. To alleviate this condition, wetting for particle lubrication was reduced to a minimum, only that traffic necessary for placement was allowed on the zone, and compaction was reduced to one pass of the vibrating roller on the 24-inch lift. Otherwise, the hauling and placing was similar to that of Zone 2A material. The blanket drain was increased to a depth of 10 feet in a faulted area to the right of the dam axis.

Pervious. End-dumped material in the Zone 3 embankment was leveled into 15-inch lifts and pushed into place by track and rubber-tired dozers. Oversize material was removed with rock buckets having properly sized openings and mounted on front-end loaders (Figure 389). The embankment was wetted with tanker trucks and compacted by two coverages of the three-drum vibratory rollers.

Figure 390. Soil-Cement Batch Plant

Figure 391. Spreading Soil-Cement

Figure 392. Compacting Soil-Cement

Random. Hauling and placing of Zone 4 embankment materials was done primarily with the power scrapers. Compaction was accomplished in four coverages by pneumatic rollers with wheel loadings of 30,000 pounds. The 10-inch embankment lifts were watered and scarified as necessary, depending on the types of materials being utilized. No major problems were encountered with this zone, other than maintaining area and slope control.

Soil-Cement. Bottom dumps hauled and windrowed the material at the work site (Figure 390). The material was graded into position with a spreader box attached to a tractor (Figure 391). The soil-cement was compacted with a 15-ton pneumatic roller. The roller was unable to propel itself through the loose lifts and had to be towed to make the initial passes. This problem was solved by attaching a grid-type roller to a tractor for a breakdown pass as it was spreading the soil-cement. The pneumatic roller was equipped with a side-mounted hydraulically operated wheel to contain the outer lift edge during rolling operations (Figure 392). This wheel was modified as to angle of mount and tire size to increase its efficiency during the work. Required densities were achieved easily and usually exceeded.

The soil-cement was cured by water trucks during working hours and by irrigation sprinklers at other times.

Downstream Cobbles. The downstream dam slope was overbuilt and then brought to finished grade with a crane-operated dragline or grader. Placement of cobble was done with a crane equipped with an orange-peel bucket (Figure 393).

Boat Ramp and Parking Area. A boat ramp and parking area were built at the east abutment to increase the stability of the area due to slides and to make use of materials that otherwise would have gone to waste (Figure 394).

The boat ramp and parking area were constructed primarily of sandstones from spillway excavation and excavation from borrow areas where removal was necessary to reach Zone 1 material deposits. These areas of embankment were compacted to Zone 4 embankment standards. The embankment slope below the ramp surface was protected with concretions obtained from excavation and designated Type IV riprap. Pervious material meeting dam embankment specifications was used for loose material, and the ramp and parking area were paved with asphalt in a completion contract.

Outlet Works

High Intake Tower—Excavation. A 4-foot-diameter pilot hole was excavated from ground level to the intersection with the diversion tunnel, and a tractor with backhoe was used to enlarge the hole to the required dimensions. The excavated material fell into the diversion tunnel and was removed to a designated spoil area by a loader. Along with conventional backhoe methods, drilling, shooting, and hand labor were used in the excavation.

The sequence of operation was to excavate a maximum of 6 feet to the required dimension; install the required welded wire fabric, rock bolts, and structural support steel; and apply 4 inches of shotcrete. After excavating to elevation 1,346.6 feet, a shaft support collar with support beams was placed to hold the structural support members as excavation progressed.

For reasons of safety and to expedite the work, a second arch over the tunnel intersection was omitted; an 8-inch channel collar bracing and flanges were welded to the in-place tunnel arch.

The in-place concrete in the tunnel crown was successfully removed by controlled blasting. Thin-wall tubing had been installed previously by the diversion tunnel contractor, making the blasting operation efficient. After blasting, large pieces of concrete were broken up with pneumatic hammers and removed from the tunnel.

Excavation for the low intake gate slot was narrowed ½ of a foot downstream to avoid removing one of the in-place tunnel support posts and invert struts. The invert and sides of the slot were overexcavated and protected with concrete to prevent air slaking of the Castaic formation material.

High Intake Tower—Concrete Operations. Reinforcing steel was installed in three stages: (1) shaft-tunnel intersection to the shaft control joint, (2) shaft control joint to the welded splice connection just below the shaft collar, and (3) welded joint to the intake tower.

The No. 14 and No. 18 reinforcing bars required butt welding at splice points. All welding was 100% X-rayed. The variable-diameter horizontal steel that required butt welding was set up in a jig in the reverse order in which it was to be used. A hydraulic bender was used to eliminate tangent ends and properly align bars prior to welding. The bars then were hoisted and lowered into position using a crane. Hydraulic jacks were used for final alignment. Back-up strips were used to weld the vertical No. 18 bars that were not accessible for conventional welding as well as the No. 11 bars that had to be removed in the shaft-tunnel intersection for the installation of a sluiceway liner.

Tower reinforcement was erected by first setting a skeleton template and anchoring it to concrete anchors on the ground to maintain alignment. Removable scaffolding then was set at the base of the template. Vertical bars were hoisted into place in pairs using a truck or tower crane. Horizontal bars were loaded on the scaffold and hand-hoisted into position.

Concrete construction for the high intake tower and shaft was by conventional methods. Wooden forms were used to shape and confine the concrete in the shaft, and metal forms were used for the tower above elevation 1,350 feet. Concrete was placed using buckets and tremie pipes (Figure 395).

Figure 393. Downstream Cobble Slope Protection

Figure 394. East Abutment Boat Ramp

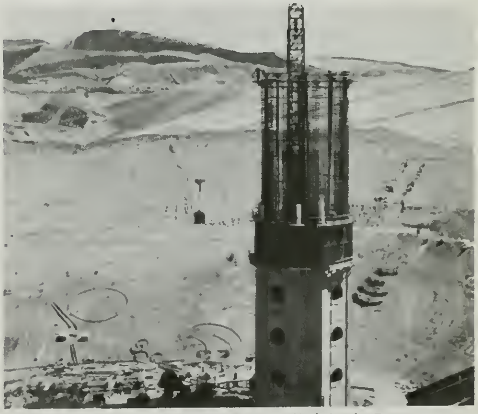

Figure 395. Concrete Placement—High Intake Tower

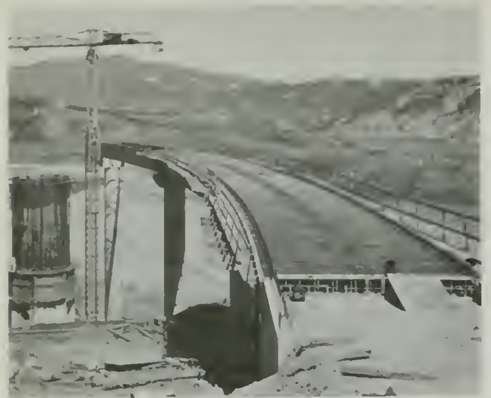

Figure 396. High Intake Tower Bridge Under Construction

Figure 397. Low Intake Tower

Access Bridge Piers. A crane-mounted hole digger equipped with a telescoping Kelly bar, a 4-foot rotary drilling bucket, and a 10-foot-diameter auger and reamer were used to excavate the required holes (Figure 396). Initially, a 10-foot-diameter hole was excavated using the 10-foot-diameter auger. The hole was then reamed to 11 feet - 4 inches using a drill extension reamer. An 11-foot-diameter casing constructed of ¾-inch steel plate was lowered into the hole to support the shaft walls.

Hoop reinforcing steel, having a minimum amount of vertical reinforcement, was fabricated aboveground. After the cage was lowered into the pier excavation and supported, longitudinals were lowered into place and secured.

Concrete was placed by crane and bucket using a hopper aboveground and tremie pipes. The steel casing was removed in sections as concrete placement progressed. At no time was it necessary to place the concrete above the bottom of the casing. Steel forms were used to confine the concrete aboveground.

Access Bridge Girders and Deck. Upon completion of the bridge abutment and piers, preassembled girders were delivered to the job site and set in place using one track-mounted and one truck-mounted crane.

Crane and buckets were used to place spans Nos. 3 and 4 of the bridge deck, with spans Nos. 1 and 2 placed by conveyor belts. High winds and the crew's lack of familiarity with lightweight concrete caused placing and finishing difficulties. It was necessary to refinish parts of the span with a coat of epoxy-sand compound.

Low Intake Tower. Concrete was first removed from the tunnel section at the intersection of the tower (Figure 397). Reinforcement then was welded to the vertical reinforcement stubbed out of the tun-

Figure 398. General View of Penstock Under Construction

Figure 399. Tunnel Penstock

nel for the low intake structure. Concrete was placed in stages up to the elevation for the precast-concrete trashrack beams. The beams were set, the ends encased in concrete, and the tower cap and seal plate placed.

Initially, a 24-inch pipe and slide-gate unit was to be installed to control the flow of water through the diversion tunnel plug during construction of the high intake tower. However, due to delays in completion, an additional 30-inch, corrugated-metal-pipe, slide-gate unit was installed through the diversion tunnel plug, and a temporary bulkhead gate was installed on top of the tower. These were to control flow through the tunnel prior to installation of the low intake gate located at the high intake tower.

After installation of the slide gates, access platform, and ladder, storm runoff raised the water elevation in Castaic Lake enough to cause silting in and around the gates. This necessitated building an earthfill dam out to the tower so that grouting of the bypass piping, removal of the temporary bulkhead gate, and installation of the metal trashracks could take place.

Penstock. The penstock (Figure 398) includes the wye branches for the turnouts and the bifurcation for the dispersion structure. Some difficulty was experienced when installing the penstock (Figure 399) sections because of flat spots, out of roundness, and misalignment in the sleeve-type coupling areas.

Modification of the downstream side of the penstock was necessary in most of the valve vaults because of misalignment of the upstream and downstream sections in the areas of the valve couplings. The 78-inch penstock-to-turnout coupling outside the valve vault was not installed correctly and later started to leak. This section eventually had to be welded together.

Concrete removal consisted of roughening the existing concrete in the area to receive the tunnel plug liner, removal of concrete for keying the penstock support piers, and removal of various areas of the diversion channel invert and walls. Surface roughening and pier concrete removal were accomplished by use of a tractor-mounted hoe ram and men working from a high scaffold using hand-held pneumatic tools. The outline of the area for concrete removal was first sawed to a depth of 2 inches. The contractor was permitted to remove and replace some of the reinforcement steel in the pier areas to facilitate and expedite the work.

Figure 400. Spillway Stilling Basin and Chute Excavation

After removal of the required concrete in a given area, a backhoe was used to excavate and clean up to the required lines and grade. Immediately following excavation, the area was cleaned of all loose and semidetached material, and 3 inches of foundation protection shotcrete or concrete was applied.

Placing of concrete was by conventional methods: crane and buckets for the outside work, pumps for the tunnel plug concrete, and conveyor belts for the penstock piers.

Because of delays, it was necessary to construct a bypass channel through and around the work area to carry the runoff during the 1970–71 rainy season. This was accomplished by building a 4-foot retaining wall across the turnout area at turnout Station 49+17 down to the stream release area. The wall normally was 1 foot inside the vertical reinforcement. In the stream release and dispersion area, the excavated slope was steepened and the face shotcreted to provide a bypass channel outside the work area. Water reentered the paved channel at Station 53+70.

Major difficulty was encountered in maintaining specified tolerances on the penstock diameters and installing the couplings according to the manufacturer's instructions. During filling and hydrotesting of the penstock, a crew was maintained to inspect and adjust the penstock coupling to a point where no appreciable leakage occurred. After the hydrotesting was completed, the penstock was drained, the test head and other testing equipment removed, and the necessary protective-coating repair made. After minimal cure time for the coating elapsed, the penstock was refilled to within 20 feet of the surface elevation of Castaic Lake.

Spillway

Excavation. Initial excavation (Figure 400) was carried to within 2 feet of final grade. Rebound gauges were installed and monitored. Depending upon the rate of rebound and the contractor's schedule for starting structural concrete, sections of the spillway were released for final excavation prior to expiration of the stipulated five months. Where the foundation had been cut and cleaned to final grade, protection requirements called for concrete placement or application of an asphaltic emulsion covering within 24 hours. Structural concrete was placed within 30 days in the areas protected with the asphaltic emulsion. Much of the early excavation material was stockpiled for later use in the dam embankment. However, large quantities of unweathered shales were wasted.

Slides occurred between spillway Stations 53+00 and 54+00 on both sides of the excavation in the vicinity of the main dewatering sump hole. The slides probably were due to the fluctuating action of the water table. Other sliding took place along a bedding plane in the vicinity of Station 50+00 at the easterly side of the excavation. Slopes were flattened to control the sliding. Ground water inflow was intercepted on the slopes and directed to sump holes for pumping.

Investigation disclosed an unstable condition existing in the left foundation at the flip bucket. A block of Castaic formation was subject to slippage along a shear plane. Because of area limitation and economy, the block was stabilized by pinning it in place with grouted rock bolts.

The heavy rains at the end of January and again in February 1969 flooded and silted the stilling basin. In March 1969, a dredge pumped silted material from the basin to portions of the Lagoon. Dredging was completed in March 1969, and dewatering of the basin was started immediately. Well points were established at the right side of the excavation in the vicinity of Station 53+50 at the end of April 1969.

Excavation for the remaining structural section was worked back from Station 37+00 to the crest section. The major portion of the approach channel section was left in place to facilitate its use in dam embankment.

Sliding occurred on the easterly side of the spillway excavation between Stations 12+70 and 17+00. This area was unloaded and buttressed. However, additional sliding seemed imminent at Station 12+50 when keyways were cut for the wall upstream of the crest in the approach channel. Rock bolts were placed at this location to ensure its stability.

Excavation for the drainage and access gallery was begun by sawing into the foundation material to the appropriate lines and depth and then excavating with a track-mounted backhoe. The wall-footing excavation was made using a tractor with rippers, scrapers, and motor graders. Excavation for the transverse drains was made with a backhoe, or a backhoe in conjunction with sawing, as was done in the gallery. Final cleanup was by hand. Considerable overbreak occurred during these operations, and large quantities of backfill concrete were used.

Concrete. Standard wooden forms were used to confine the concrete for the spillway invert and a weighted steel slip form was used to strike the concrete to grade. Forms for the spillway walls were made from 1-inch resin-treated plywood and were supported with appropriate horizontal and vertical members. Forms for the ogee-crest section and the flip-bucket section were constructed to shape the concrete to the desired configurations. These forms were removed board by board, starting from the bottom, to allow for finishing of the concrete prior to its final set.

Invert and footing steel was supported by concrete and steel chairs. The major portion of the spillway wall steel was secured in place with galvanized metal chairs.

Construction of the spillway (Figure 401) progressed from Station 37+00, down the chute section to the flip bucket, and then from Station 37+00 back to the crest. The order of concrete placement, not including that placed for foundation protection, was (1) keyways, (2) wall footings, (3) gallery and invert slabs, and (4) walls. Placements were performed concurrently as conditions permitted.

Concrete was batched and mixed in a central batch plant. Concrete was hauled to the job site in 8-cubic-yard agitator transports and discharged into 2- or 4-cubic-yard buckets. The buckets were hoisted into placing position by motor cranes. All permanently exposed concrete surfaces were water-cured. Membrane curing was allowed for surfaces that were to be backfilled.

Flip-Bucket Piles. Design studies after the start of construction indicated that additional support for the flip-bucket section in the stilling basin would be required during periods of heavy discharges. A design change was issued calling for the installation of vertical and battered cast-in-place piles beneath the section.

Structural Backfill. The designated backfill behind the spillway walls was Zone 3 material meeting gradation requirements below the 6-inch size. The contractor was permitted to use surplus Zone 2A material for this operation since it met gradation requirements in this range.

Originally, no provisions had been made to protect the backfill from erosion in the steep chute area of the spillway. Intense rains late in the 1970–71 season washed and heavily eroded backfill in this area. The backfill surface was the ultimate collection area for

Figure 401. Spillway Construction

runoff. The damaged backfill was repaired and a 6-inch layer of riprap bedding was placed on a lowered backfill surface. Cobble slope protection, as specified for the downstream slope of the Dam, then was placed from just above the start of the chute section to the stilling basin area.

Castaic Lagoon Control Structure (Figure 402)

Foundation excavation and concrete placement for this structure started in November 1968. Wingwalls were constructed first. Reinforcing steel for the wingwalls was doweled and anchored into the Lake Hughes Road Bridge abutments.

Storms in January 1969 caused erosion around some of the bridge pilings (Figure 382). The contractor attempted to prevent further erosion by placing riprap lining on the diversion channel at the Bridge. However, the extremely heavy flows at this location from the February 1969 storms undercut and displaced the pilings and the westerly section of the bridge collapsed. Cleanup of storm debris, structural repair, bridge repair by Los Angeles County, and dam closure of the diversion channel caused delays in construction of the major portion of the structure until the following year.

The flooding that occurred in 1969 indicated the need for additional protection to prevent erosion downstream of the lagoon control structure during periods of heavy water flows. It was determined that much of the damaging erosion was because of the degradation of the streambed downstream and immediately adjacent to the State's right of way, resulting from the removal of streambed gravels by another contractor. An extended riprap design was provided, and performance of the work was directed by a contract change order. The work consisted of grading and shaping the area immediately downstream of the structure, laying a blanket for riprap consisting of Zone 2B drain material, and placing Type IV riprap (native sandstone concretions). The riprap extended well below normal streamflow depth; therefore, the change order provided for dewatering of the area during placement of the riprap. However, due to the time of year and the contractor's upstream pumping diversions, no water was encountered. This work was performed during October and November 1970.

Figure 402. Castaic Lagoon Control Structure

Clearing, Grubbing, and Erosion Control

Reservoir Clearing. The reservoir area to be cleared extended 4 miles up Elizabeth Lake Canyon and 2½ miles into Castaic Canyon from the Dam. Elizabeth Lake Canyon was heavily wooded and brush-covered. Castaic Canyon had relatively few trees or brush-covered areas. Two areas in Elizabeth Lake Canyon about 1 mile from the Dam were designated for vegetation retention to approximate a more normal habitat for some fish species.

The clearing subcontractor moved onto the job in early November 1969 and began clearing operations with the removal of floatable debris from the stream channel areas. Reservoir clearing work was restricted because of unfavorable fire load indexes. However, some tree felling, brushing, channel debris removal, stacking, and burning in Elizabeth Lake Canyon was done during the spring of 1970. At the end of the first season of reservoir clearing in June 1970, it was estimated that this work was 60% complete. The reservoir clearing was completed on April 24, 1971.

The clearing operations consisted of felling and bucking trees for burning or hauling to approved disposal sites, dozing of brush-covered areas, hand clearing of slopes where equipment could not operate, and removal of channel debris. Downed trees and floatable debris were removed from between elevations 1,515 feet and 1,525 feet and in the vegetation retention areas. Anchorage of large downfalls was allowed in remote and inaccessible parts of the vegetation retention areas. Unburned debris and fire residue were buried as specified. Some trees and shrubs were removed by private concerns to landscape areas under development.

Clearing and Grubbing of Other Areas. The prime contractor performed the remainder of the work under the contract item "clearing and grubbing". Clearing and grubbing of the dam foundation started on July 5, 1967. This work was extended to borrow areas, excavation areas, and structure and roadway locations throughout the contract period as the scheduled work was performed.

Other items of work included removal of buildings, water tanks, water lines, property improvements, and the county road bridge across the Dam site area. The bridge was dismantled in July 1967, with the contractor salvaging much of the timber. The bridge pilings were removed during excavation operations. Disposal of the material was by burning, burying, or removal from the job site. No major difficulties were encountered.

Erosion Control. Erosion control for designated areas called for seeding on slopes less than 15% and for seeding and mulching on slopes 15% or steeper. No seeding was required within the reservoir area. All seeding was done toward the conclusion of the contract as areas were completed and dressed. Work was started in September 1971.

Mulching was done with straw which was applied with an air-blowing spreader. Coverage was governed by the number of bales to the acre. The mulch then was worked into the ground with rollers equipped with protruding studs. The less steep areas were disked prior to seeding.

Seeding and fertilizing were accomplished in one operation. Batches of 1,500 pounds of fertilizer and 225 pounds of seed were suspended in water in a 1,500-gallon mixing tank. Agitation maintained the suspension, and the mixture was evenly distributed over a 3-acre area. Following seeding, mulched areas were rerolled. The seed was raked into the soil in the flatter areas. The erosion control work was completed in November 1971.

BIBLIOGRAPHY

Amorocho, J., Babb, A. F., and Ross, J. A., "Hydraulic Model Investigations of the Castaic Dam High Intake Tower", Water Science and Engineering Papers 1046, Department of Water Science and Engineering, University of California, Davis, June 1971.

Devries, J. J., Amorocho, J., and Ross, J. A., "Hydraulic Model Study of the Castaic Low Intake Gate", Water Science and Engineering Papers 1049, Department of Water Science and Engineering, University of California, Davis, July 1971.

Godden, W. G., Tuncag, M., Wasley, D. L., "Model Analysis of the Castaic Reservoir Intake Tower", College of Engineering Office of Research Services, University of California, Berkeley, California, September 1970.

Gordon, B. B., and Wulff, J. G., "The Effects of Changes in the State of the Art on Heights of Earth and Rock Dams—Oroville and Castaic Dams", Proceeding of Western Water and Power Symposium, Los Angeles, California, April 1968.

Perry, C. W., and Kruse, G. H., "Instrumentation for Pore Pressures and Seismic Effects—State Water Project Dams", Proceeding of Western Water and Power Symposium, Los Angeles, California, April 1968.

Ross, J. A., Amorocho, J., and Babb, A. F., "The Castaic Dam Diversion Facilities, A Hydraulic Investigation", Water Science and Engineering Papers 1022, Department of Water Science and Engineering, University of California, Davis, August 1968.

Ross, J. A., Jones, W. C., Amorocho, J., and Babb, A. F., "Hydraulic Model Investigations of the Castaic Dam Release Facilities", Water Science and Engineering Papers 1042, Department of Water Science and Engineering, University of California, Davis, November 1970.

———, "Hydraulics of the Castaic Dam Spillway (Laboratory Model Studies)", Water Science and Engineering Papers 1023, Department of Water Science and Engineering, University of California, Davis, August 1968.

APPENDIX A

CONSULTANTS

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APPENDIX A
CONSULTANTS

Several major consulting boards were employed by the Department to advise on the dams covered in this volume. These boards were composed of men, acknowledged to be experts in their fields. who could bring to the Project many years of experience gained from their work on other major water projects around the world.

Earth Dams Consulting Board

Mr. Wallace L. Chadwick
Mr. Julian Hinds
Mr. Roger Rhoades
Dr. Philip C. Rutledge
Mr. B. E. Torpen

The Board advised the Department's engineers with regard to design and construction of Del Valle, Castaic, Pyramid, Cedar Springs, and Perris Dams and other proposed dams.

Earthquake Analysis Board

Dr. Clarence Allen
Dr. Hugo Benioff
Dr. John Blume
Dr. Bruce Bolt
Dr. George Housner
Dr. H. Bolton Seed
Dr. James L. Sherard
Mr. Nathan D. Whitman

The Board advised the Department concerning the evaluation of seismic effects to be anticipated at any given site or area and on the development of rational procedures for seismic design in regard to hydraulic structures.

Oroville Dam Consulting Board

Mr. A. H. Ayers
Mr. John Hammond
Mr. Raymond A. Hill
Mr. J. Donovan Jacobs
Mr. Thomas A. Lang
Mr. Roger Rhoades
Dr. Philip C. Rutledge
Mr. Byram W. Steele
Mr. B. E. Torpen

The Board advised the Department's engineers on design and construction for Oroville Dam first when a concrete dam was being considered and later on the earth dam. They also advised on the other dams and appurtenances in the Oroville Division.

Many other individuals furnished consulting services and several universities, especially the Berkeley and Davis Campuses of the University of California, furnished technical support as did the U.S. Bureau of Reclamation and several engineering firms.

APPENDIX B

ENGLISH TO METRIC CONVERSIONS

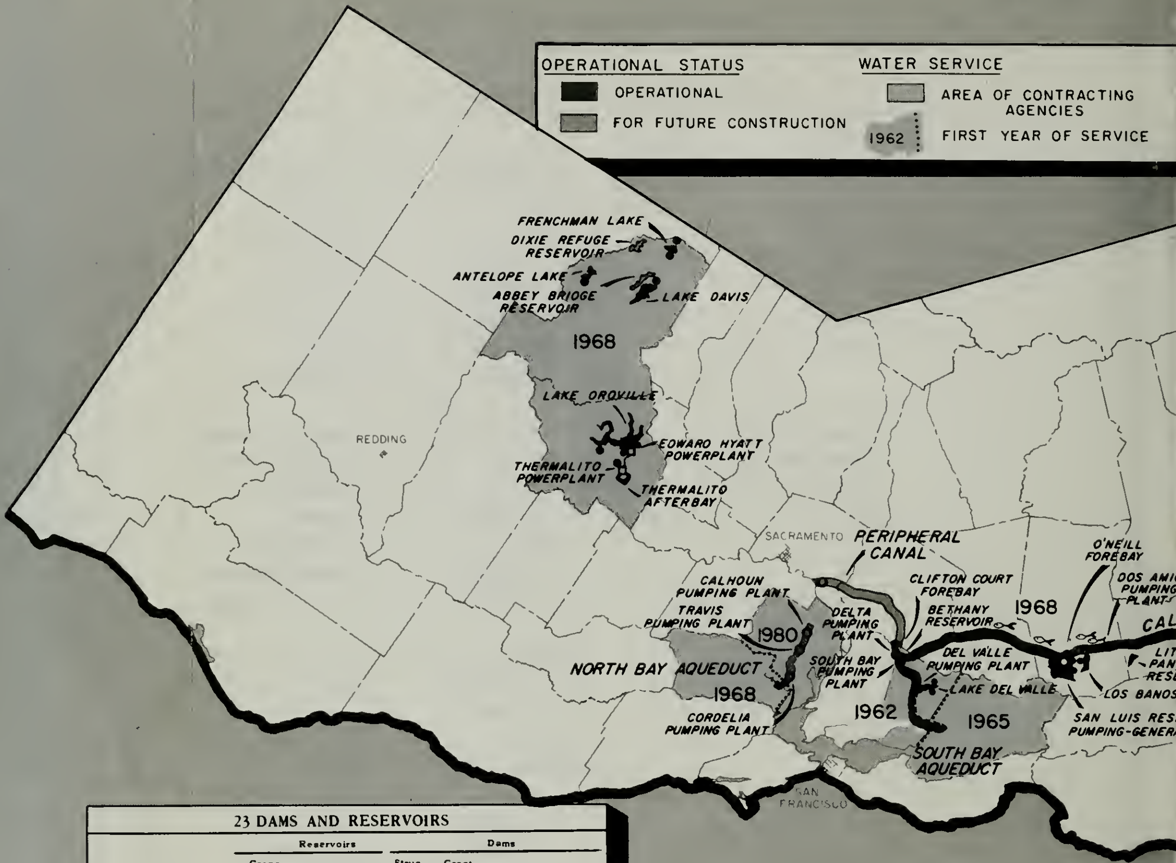
AND PROJECT STATISTICS

CONVERSION FACTORS

English to Metric System of Measurement

Quantity	English unit	Multiply by	To get metric equivalent
Length	inches	2.54	centimeters
	feet	30.48	centimeters
		0.3048	meters
		0.0003048	kilometers
	yards	0.9144	meters
	miles	1,609.3	meters
		1.6093	kilometers
Area	square inches	6.4516	square centimeters
	square feet	929.03	square centimeters
	square yards	0.83613	square meters
	acres	0.40469	hectares
		4,046.9	square meters
		0.0040469	square kilometers
	square miles	2.5898	square kilometers
Volume	gallons	3,785.4	cubic centimeters
		0.0037854	cubic meters
		3.7854	liters
	acre-feet	1,233.5	cubic meters
		1,233,500.0	liters
	cubic inches	16.387	cubic centimeters
	cubic feet	0.028317	cubic meters
	cubic yards	0.76455	cubic meters
		764.55	liters
Velocity	feet per second	0.3048	meters per second
	miles per hour	1.6093	kilometers per hour
Discharge	cubic feet per second or second-feet	0.028317	cubic meters per second
Weight	pounds	0.45359	kilograms
	tons (2,000 pounds)	0.90718	tons (metric)
Power	horsepower	0.7460	kilowatts

PROJECT



23 DAMS AND RESERVOIRS

Name of Reservoir	Reservoirs: Gross Capacity(1 (Acre-feet)	Reservoirs: Surface Area (acres)	Reservoirs: Shore-line (miles)	Dams: Structural Height (feet)	Dams: Crest Elevation(2 (feet)	Dams: Crest Length (feet)	Dams: Volume (cubic yards)
Frenchman Lake	55,477	1,580	21	139	5,607	720	537,000
Antelope Lake	22,566	931	15	120	5,025	1,320	380,000
Lake Davis	84,371	4,026	32	132	5,785	800	253,000
Abbey Bridge	45,000	1,950	21	117	5,4[illegible]	1,150	500,000
Dixie Refuge	16,000	900	15	100	5,754	1,050	400,000
Lake Oroville	3,537,577	15,805	167	770	922	6,920	80,000,000
Thermalito Diversion Pool	13,328	323	10	143	233	1,300	154,000
Fish Barrier Pool	580	52	1	91	181	600	10,500
Thermalito Forebay	11,768	630	10	91	231	15,900	1,840,000
Thermalito Afterbay	57,041	4,302	26	39	142	42,000	5,020,000
Clifton Court Forebay	28,653	2,109	8	30	14	36,500	2,440,000
Bethany	4,804	181	6	121	250	3,940	1,400,000
Lake Del Valle	77,106	1,060	16	235	773	880	4,150,000
San Luis	2,038,771	12,700	65	385	554	18,600	77,645,000
O'Neill Forebay	56,426	2,700	12	88	233	14,350	3,000,000
Los Banos	34,562	623	12	167	384	1,370	2,100,000
Little Panoche	13,236	354	10	152	676	1,440	1,210,000
Buttes	21,800	580	6	190	2,790	2,230	3,130,000
Silverwood Lake	74,970	976	13	249	3,378	2,230	7,600,000
Lake Perris	131,452	2,318	10	128	1,600	11,600	20,000,000
Pyramid Lake	171,196	1,297	21	400	2,606	1,090	6,860,000
Elderberry Forebay	28,231	460	7	200	1,550	1,990	6,000,000
Castaic Lake	323,702	2,235	29	425	1,535	4,900	46,000,000
Totals	6,848,617	58,072	533			172,880	270,629,500

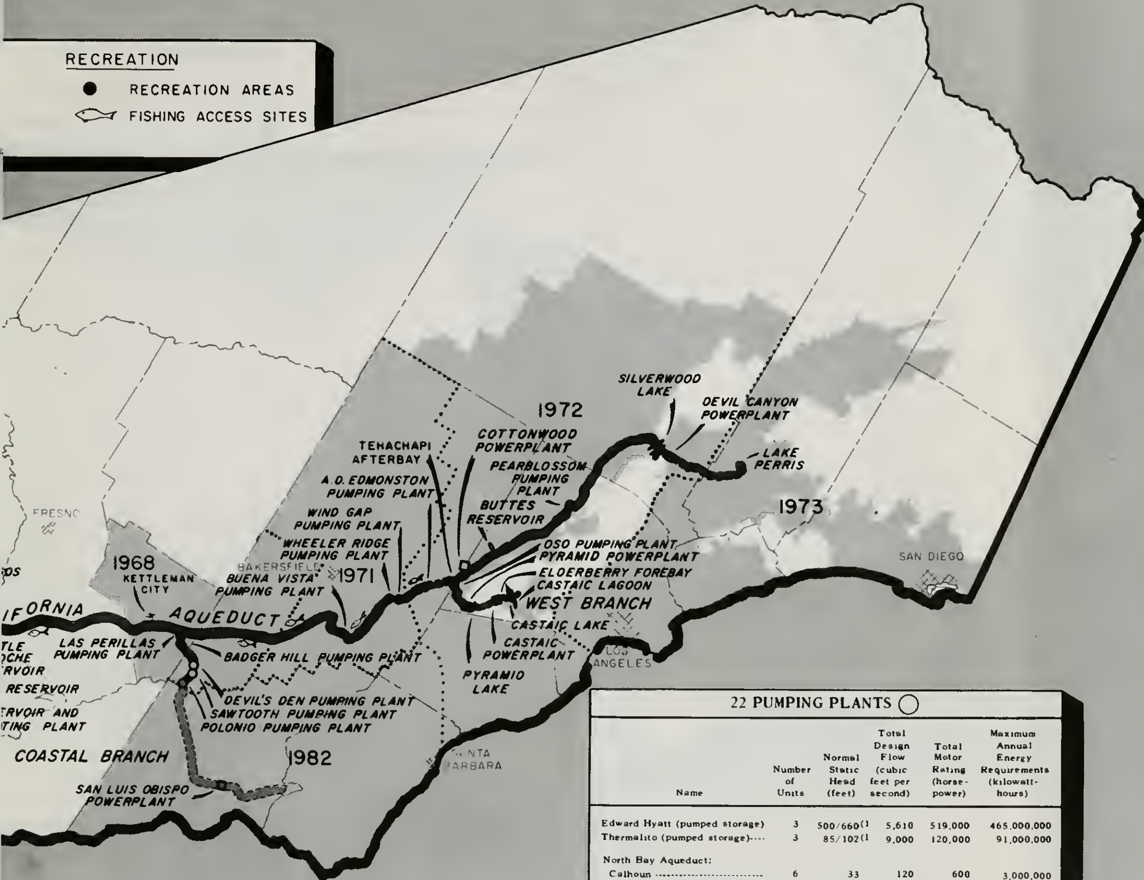
1) At maximum normal operating level

2) Above sea level.

AQUEDUCTS

Name	Length (miles): Total	Canal	Pipeline	Tunnel	Channel and Reservoir
North Bay Aqueduct	26.5	14.3	12.2	0	0
South Bay Aqueduct	42.9	8.4	32.9	1.6	0
Peripheral Canal	43.0	42.0	1.0	0	0
Subtotal	112.4	64.7	46.1	1.6	0
California Aqueduct (main line):					
Delta to O'Neill Forebay	68.4	67.0	0	0	1.4
O'Neill Forebay to Kettleman City	105.7	103.5	0	0	2.2
Kettleman City to A. D. Edmonston Pumping Plant	120.9	120.9	0	0	0
A. D. Edmonston Pumping Plant thru Tehachapi Afterbay	10.6	0.2	2.5	7.9	0
Tehachapi Afterbay thru Lake Perris	138.4	93.4	38.3	3.8	2.9
Subtotal, main line	444.0	385.0	40.8	11.7	6.5
California Aqueduct (branches):					
West Branch	31.9	9.1	6.4	7.2	9.2
Coastal Branch	96.2	14.8	81.4	0	0
Subtotal, branches	128.1	23.9	87.8	7.2	9.2
Totals	684.5	473.6	174.7	20.5	15.7

STATISTICS



8 POWERPLANTS

Name	Number of Units	Normal Static Head (feet)	Total Design Flow (cubic feet per second)	Power Generator Output (kilowatts)	Maximum Annual Energy Output (kilowatt-hours)
Edward Hyatt	6	410/676[1]	14,550	678,750	2,475,000,000
Thermalito	4	85/100[1]	16,900	119,600	383,000,000
San Luis					
Total	8	99/327[1]	13,120	424,000	
State Share			6,872	222,100	170,000,000
Cottonwood	1	140	1 637	15,000	115,000 000
Devil Canyon	2	1,418	1 200	119 700	1,003,000,000
Pyramid	2	740	3,100	157,000	1,001,000,000
Castaic					
Total	7	1,063	18,400	1,250,000	
State Share[2]			3,092	214,000	1,457,000,000
San Luis Obispo	1	730	111	5,900	41,000,000
Total, State Share					6,645,000,000

1) Minimum and maximum static heads

2) The City of Los Angeles Department of Water and Power will construct and operate a 1,250,000-kilowatt Castaic Powerplant and will supply the Project with electrical power and energy equivalent to the generation from a 213,984-kilowatt powerplant the State originally planned to construct

22 PUMPING PLANTS

Name	Number of Units	Normal Static Head (feet)	Total Design Flow (cubic feet per second)	Total Motor Rating (horse-power)	Maximum Annual Energy Requirements (kilowatt-hours)
Edward Hyatt (pumped storage)	3	500/660[1]	5,610	519,000	465,000,000
Thermalito (pumped storage)	3	85/102[1]	9,000	120,000	91,000,000
North Bay Aqueduct:					
Calhoun	6	33	120	600	3,000,000
Travis	6	0	120	900	5,000,000
Cordelia	3	448	48	3,100	14,000,000
South Bay Aqueduct:					
South Bay	9	545	330	27,750	166,000,000
Del Valle	4	0/38[2]	120	1,000	2,000,000
California Aqueduct (main line):					
Delta	11	244	10,303	333,000	1,355,000,000
San Luis					
Total	8	99/327[2]	11,000	504,000	
State Share			5,762	264,000	313,000,000
Dos Amigos					
Total	6	113	13,200	240,000	
State Share			7,100	130,000	607,000,000
Buena Vista	10[3]	205	5,049	136,000	746,000,000
Wheeler Ridge	9[3]	233	4,598	140,000	797,000,000
Wind Gap	9[3]	518	4,410	308,000	1,761,000,000
A. D. Edmonston	14[3]	1,926	4,095	1,040,000	5,916,000,000
Pearblossom	6	540	1,380	113,200	647,000,000
California Aqueduct (branches):					
Oso	8	231	3,128	93,800	446,000,000
Las Perillas	6	55	450	4,050	20,000,000
Badger Hill	6	151	450	10,500	56,000,000
Devil's Den	4	409	126	8,000	51,000,000
Sawtooth	4	331	126	6,500	41,000,000
Polonio	4	810	126	16,000	101,000,000
Peripheral Canal					
Total	9[3]	10	21,800	35,200	
State Share			10,900	17,440	88,000,000
Total, State Share					13,691,000,000

1) Minimum and maximum total pumping heads.
2) Minimum and maximum static heads.
3) Includes one spare unit.

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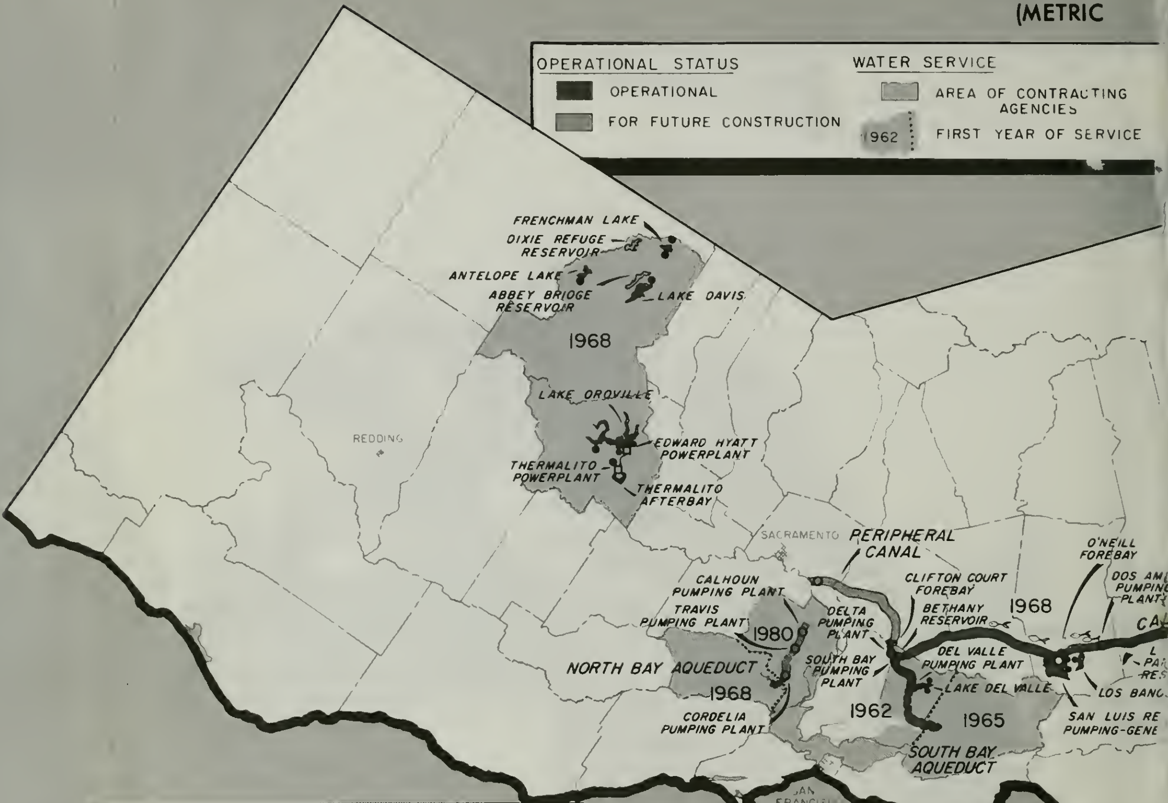


23 DAMS AND RESERVOIRS

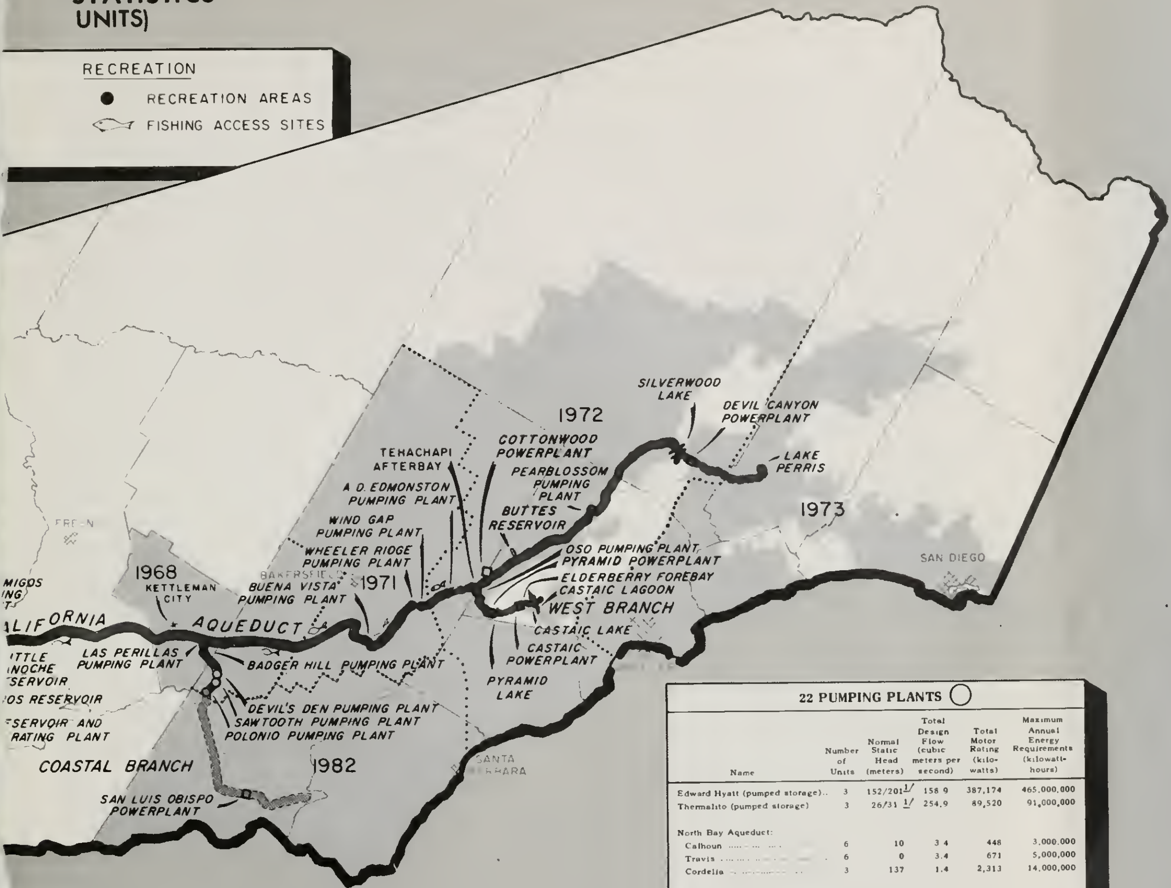
Name of Reservoir	Reservoirs: Gross Capacity 1/ (millions of cubic meters)	Reservoirs: Surface Area (hectares)	Reservoirs: Shore-line (kilo-meters)	Dams: Struc-tural Height (meters)	Dams: Crest Eleva-tion 2/ (meters)	Dams: Crest Length (meters)	Dams: Volume (cubic meters)
Frenchman Lake	68.43	639	33.8	42	1709	219	410,600
Antelope Lake	27.84	377	24.1	37	1532	402	290,500
Lake Davis	104.07	1,629	51.5	40	1763	244	193,400
Abbey Bridge	55.51	789	33.8	36	1669	351	382,300
Dixie Refuge	19.74	364	24.1	30	1754	320	305,800
Lake Oroville	4,363.60	6,396	268.8	235	281	2,109	61,164,000
Thermalito Diversion Pool	16 44	131	16 1	44	71	396	117,700
Fish Barrier Pool	0.72	21	1.6	28	55	183	8,000
Thermalito Forebay	14.52	255	16.1	28	70	4,846	1,406,800
Thermalito Afterbay	70.36	1,741	41.8	12	43	12,802	3,838,000
Clifton Court Forebay	35.34	853	12.9	9	4	11,125	1,865,500
Bethany	5.93	65	9.7	37	76	1,201	1,070,300
Lake Del Valle	95.11	429	25.6	72	236	266	3,172,900
San Luis	2,514.82	5,140	104.6	117	169	5,669	59,363,500
O'Neill Forebay	69.60	1,093	19.3	27	71	4,374	2,293,700
Los Banos	42 63	252	19.3	51	117	416	1,605,600
Little Panoche	16.33	143	16.1	46	206	439	925,100
Buttes	26.89	235	9.7	58	850	680	2,393,000
Silverwood Lake	92.46	395	20.9	76	1,030	680	5,810,600
Lake Perris	162.15	938	16.1	39	488	3,536	15,291,000
Pyramid Lake	211.17	525	33.8	122	794	332	5,244,800
Elderberry Forebay	34 62	186	11.3	61	472	607	4,587,300
Castaic Lake	399.29	904	46.7	130	468	1,494	35,169,300
Totals	8,447.79	23,500	857.9			52,695	206,909,700

1/ At maximum normal operating level.
2/ Above sea level.

AQUEDUCTS

Name	Length (kilometers): Total	Canal	Pipeline	Tunnel	Channel and Reservoir
North Bay Aqueduct	42.6	23 0	19 6	0	0
South Bay Aqueduct	69.1	13.5	53.0	2.6	0
Peripheral Canal	69.2	67 6	1.6	0	0
Subtotal	160.9	104.1	74.2	2.6	0
California Aqueduct (main line):					
Delta to O'Neill Forebay	110.1	107.6	0	0	2.3
O'Neill Forebay to Kettleman City	170.1	166.6	0	0	3.5
Kettleman City to A.D. Edmonston Pumping Plant	194.6	194.6	0	0	0
A.D. Edmonston Pumping Plant thru Tehachapi Afterbay	17.0	0 3	4 0	12 7	0
Tehachapi Afterbay thru Lake Perris	222 7	150 3	61.6	6 1	4 7
Subtotal, main line	714.5	619.6	65.6	18.8	10.5
California Aqueduct (branches):					
West Branch	51.3	14.6	10.3	11.6	14.6
Coastal Branch	154.6	23.6	131.0	0	0
Subtotal, branches	206.1	38 4	141.3	11.6	14 6
TOTALS	1,101.5	762.1	281.1	33.0	25.3

STATISTICS
UNITS)



8 POWERPLANTS

Name	Number of Units	Normal Static Head (meters)	Total Design Flow (cubic meters per second)	Power Generator Output (kilowatts)	Maximum Annual Energy Requirements (kilowatt-hours)
Edward Hyatt	6	125/206 1/	412 0	678,750	2,475,000,000
Thermalito	4	26/30 1/	478.6	119,600	383,000,000
San Luis					
Total	8	30/100 1/	371.6	424,000	
State Share			194.6	222,100	170,000,000
Cottonwood	1	43	46.4	15,000	115,000,000
Devil Canyon	2	432	34.0	119,700	1,003,000,000
Pyramid	2	226	87 8	157,000	1,001,000,000
Castaic					
Total	7	324	521.0	1,250,000	
State Share 2/			87.6	214,000	1,457,000,000
San Luis Obispo	1	223	3 1	5,900	41,000,000
Total, State Share					6,645,000,000

1/ Minimum and maximum static heads.

2/ The City of Los Angeles Department of Water and Power will construct and operate a 1,250,000-kilowatt Castaic Powerplant and will supply the Project with electrical power and energy equivalent to the generation from a 213,984-kilowatt powerplant the State originally planned to construct.

22 PUMPING PLANTS

Name	Number of Units	Normal Static Head (meters)	Total Design Flow (cubic meters per second)	Total Motor Rating (kilowatts)	Maximum Annual Energy Requirements (kilowatt-hours)
Edward Hyatt (pumped storage)	3	152/201 1/	158 9	387,174	465,000,000
Thermalito (pumped storage)	3	26/31 1/	254.9	89,520	91,000,000
North Bay Aqueduct:					
Calhoun	6	10	3 4	448	3,000,000
Travis	6	0	3.4	671	5,000,000
Cordelia	3	137	1.4	2,313	14,000,000
South Bay Aqueduct:					
South Bay	9	166	9.3	20,702	166,000,000
Del Valle	4	0/12 2/	3.4	746	2,000,000
California Aqueduct (main line):					
Delta	11	74	291.8	248,418	1,355,000,000
San Luis					
Total	8	30/100 2/	311.5	375,984	
State Share			163.2	196,944	313,000,000
Dos Amigos					
Total	6	34	373 8	179,040	
State Share			201 1	96,980	607,000,000
Buena Vista	10 3/	62	143.0	101,456	746,000,000
Wheeler Ridge	9 3/	71	130 2	104,440	797,000,000
Wind Gap	9 3/	158	124.9	229,766	1,761,000,000
A D. Edmonston	14 3/	567	116.0	775,840	5,916,000,000
Pearblossom	6	165	39 1	84,447	647,000,000
California Aqueduct (branches):					
Oso	8	70	88 6	69,975	446,000,000
Las Perillas	6	17	12 7	3,021	20,000,000
Badger Hill	6	46	12 7	7,833	56,000,000
Devil's Den	4	125	3.6	5,968	51,000,000
Sawtooth	4	101	3.6	4,849	41,000,000
Polonio	4	247	3 6	11,936	101,000,000
Peripheral Canal					
Total	9	3	617.3	26,259	
State Share			308 7	13,010	88,000,000
Total, State Share					13,691,000,000

1/ Minimum and maximum total pumping heads.
2/ Minimum and maximum static heads.
3/ Includes one spare unit.