UNITED STATES
DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

Ellis L. Armstrong, Commissioner

TECHNICAL RECORD
OF
DESIGN AND CONSTRUCTION

GLEN CANYON DAM AND POWERPLANT

Constructed 1957-1966

Colorado River Storage Project
Arizona-Utah

Denver, Colorado
December 1970

Price $8.25

As the Nation's principal conservation agency, the Department of the Interior has basic responsibilities for water, fish, wildlife, mineral, land, park and recreational resources. Indian and Territorial affairs are other major concerns of America's "Department of Natural Resources."

The Department works to assure the wisest choice in managing all our resources so each will make its full contribution to a better United States—now and in the future.

GLEN CANYON DAM AND POWERPLANT

Glen Canyon Unit
Colorado River Storage Project—Arizona and Utah

General Information

Glen Canyon Dam, Powerplant, and Reservoir (Lake Powell) comprise one and the largest of the four units authorized by the Congress in 1956 for initial development under the Colorado River Storage project. The reservoir will contribute to the longtime regulatory storage needed to permit the upper basin States (Utah, Wyoming, Colorado, and New Mexico) to utilize their apportioned water under the terms of the 1922 Compact of the Colorado River. In addition to regulatory storage, the Colorado River Storage project and participating projects will provide electrical energy to a large area where it is urgently needed. Flood protection, sediment, retention, fish and wildlife conservation, and recreational opportunities are also provided.

Approximately 95 percent of the costs of the Colorado River Storage project and participating projects are reimbursable and will be repaid by the water and power users. About 60 percent of such costs will be repaid with interest.

Glen Canyon Dam

Location: On the Colorado River in north-central Arizona, about 15 miles (24.1 kilometers) upstream from Lees Ferry and 12 river miles (19.3 kilometers) downstream from the Arizona-Utah State line.

Type: Concrete arch

Reservoir: Lake Powell

- Total capacity to top of active conservation level, elevation 3700.0 feet (1127.76 meters): 27,000,000 acre-feet (33,304,500,000 cubic meters)
- Active capacity: 20,876,000 acre-feet (25,750,546,000 cubic meters)
- Surface area: 161,390 acres (65,313 hectares)
- Maximum reservoir elevation: 3711 feet (1131.11 meters)

Dam dimensions:

- Structural height: 710 feet (216.4 meters)
- Top width: 25 feet (7.62 meters)
- Maximum base width: 300 feet (91.44 meters)
- Crest length: 1,560 feet (457.49 meters)
- Crest elevation: 3715 feet (1323.32 meters)
- Volume: 4,901,000 cubic yards (3,747,059 cubic meters)
- Lowest point in dam foundation excavation: 3005 feet (915.9 meters)

Spillways:

- One spillway in each dam abutment. Each spillway consists of an approach channel, intake structure, combination inclined and horizontal spillway tunnel, and deflection bucket. Each spillway has a concrete crest and concrete-lined circular tunnel through the dam abutments, controlled by two 40- by 52.5-foot (12.19- by 16.00-meter) radial gates.
- The spillway tunnels for the greater part of their length are 41 feet (12.5 meters) in diameter. The transition section downstream from the intake structure changes from a flat-arch-roof section 89 feet wide by 52 feet high (27.13 by 15.85 meters) to a circular section 48 feet 3 inches (14.71 meters) in diameter. From this point there is a further transition of the circular section to the 41-foot-diameter tunnel. The tunnels are designed to flow partially full, never more than 0.7 of the height.
- Length of left spillway tunnel: 1,870 feet (570 meters)
- Length of right spillway tunnel: 1,696 feet (517 meters)
- Elevation of top of all gates: 3700.0 feet (1127.76 meters)
- Crest elevation: 3648.0 feet (1111.91 meters)
- Capacity of each spillway: 138,000 cubic feet per second (3,907.75 cubic meters per second) with water surface at elevation 3711 feet (1131.11 meters)

Penstocks:

- Eight 15-foot (4.57-meter) inside-diameter steel penstocks, each reducing to 14 feet (4.26 meters) inside-diameter within the dam, convey water to the turbines at centerline elevation 3140 feet (957.07 meters). Centerline elevation of the concrete bellmouth intake to each penstock is 3470 feet (1057.65 meters). A fixed-wheel gate upstream from each bellmouth controls the flow from the reservoir to the penstocks.

Outlet works:

Four 96-inch (2.44-meter) diameter steel pipes through the dam, terminating at the downstream end in a valve structure. Each outlet has a cast iron bellmouth intake, a 96-inch (2.44-meter) ring-follower gate for emergency closure, and a 96-inch (2.44-meter) hollow-jet valve at the outlet end for regulation.

Elevation centerline all inlets: 3374.00 feet (1028.40 meters)

Elevation centerline all outlets: 3175.00 feet (967.74 meters)

Maximum capacity of all four pipes: 15,000 cubic feet per second (424.76 cubic meters per second) at reservoir elevation 3490 feet (1063.75 meters).

Diversion tunnels:

Left: 41-foot (12.50-meter) diameter, concrete lined, 3,011 feet (917.75 meters) long, invert elevation 3170.67 feet (966.42 meters); temporary outlet works consisting of three 7- by 10.5-foot (2.13- by 3.20-meter) outlets controlled by 7- by 10.5-foot (2.13 by 3.20-meter) slide gates in tandem.

Right: 41-foot (12.50-meter) diameter, concrete lined, 2,749 feet (837.89 meters) long, invert elevation 3137.37 feet (956.27 meters). Used for riverflows from 0 to 15,000 cubic feet per second (424.76 cubic meters per second).

Glen Canyon Powerplant

Location: 400 feet (121.9 meters) downstream from the axis of the dam.

Structure: Indoor type; structural-steel superstructure enclosed with concrete curtain walls; intermediate structure and substructure are reinforced concrete. The powerplant is L shaped with the longitudinal centerline of units perpendicular to the canyon walls. The service bay and machine shop bay form the bottom of the L and are parallel to the left canyon wall. That portion of the powerplant housing the 8 generating units is about 546 feet (166.42-meters) long and 127 feet 6 inches (38.86 meters) wide. That portion of the plant housing the service bay and machine shop bay is about 103 feet (31.39 meters) wide and about 230 feet (70.10 meters) long.

Number of units: 8

Total installed capacity: 1,000,000 kv.-a. at *90-percent power factor.

Generators: Vertical-shaft type; 125,000 kv.-a. at *90-percent power factor, 3 phase, 60 cycles, 13,800 volts, and 150 r.p.m.

Turbines: Francis type, 155,500 h.p., 150 r.p.m., headof 450 feet (137.16 meters)

Glen Canyon Bridge

The bridge was constructed to serve as a vital link in the new highway to the remote damsite, extending between Flagstaff, Ariz., and Kanab, Utah, a distance of about 200 miles (320 kilometers). The bridge was also essential to the transportation of construction materials and equipment by truck to the damsite as there were no rail facilities near the dam.

Location: On the Colorado River in Arizona near the Utah border (approximately 17 miles (27 kilometers) upstream from Lee Ferry and 12 river miles (19 kilometers) downstream from the Arizona-Utah State line).

Name Change: In the authorization, appropriations, specifications, original drawings, etc., this bridge was identified as the Colorado River Bridge. However, in 1959 the bridge was officially named the Glen Canyon Bridge as an aid in obtaining a more precise location identification.

Technical Information:

Bridge type: Steel-arch type with a single span

Height above river: Approximately 700 feet (213 meters)

Length of bridge: 1,271 feet (387.4 meters) including abutments with an arch span of 1,028 feet (313.3 meters)

Location: 865 feet (263.6 meters) downstream from axis of Glen Canyon Dam

Elevation of bridge deck: 3828 feet (1166.8 meters)

Bridge roadway: The concrete roadway is 30 feet (9.14 meters) wide with 4-foot (1.22-meter) sidewalks on each side.

Approximate construction quantities:

Structural steel: 7,837,000 pounds (3,555,000 kilograms)

Reinforcing steel: 371,000 pounds (168,000 kilograms)

Handrailings: 110,000 pounds (50,000 kilograms)

Concrete for bridge abutments, skewbacks and deck: 2,550 cubic yards (1,950 cubic meters)

*Rerated 95 percent, see page 296.

Frontispiece—Glen Canyon Dam and related features. P557-400-988A

FOREWORD

Technical records of design and construction are a series of publications which record the planning, design, construction, and initial operation of Bureau of Reclamation structures.

This technical record of design and construction for Glen Çanyon Dam and Powerplant is divided into three parts. Part I is devoted to general planning, historical information, a description of the features, a summary of costs, foundation treatment, and geology. Part II contains eight chapters covering the design of the various features and their components. Part III contains one chapter on contract administration and five chapters comprising a concise narration of construction operations and equipment installations.

This technical record was prepared by Warren E. Foote of the Technical Services Branch, Engineering and Research Center in Denver, Colorado, from final design reports submitted by the design branches, final construction reports and cost information submitted by the field offices, and various planning reports. Acknowledgment is gratefully made to the designers and field personnel for their contributions to this work.

There are occasional references to proprietary materials or products in this technical record. These must not be construed in any way as an endorsement, as the Bureau cannot endorse proprietary products or processes of manufacturers or the services of commercial firms for advertising, publicity, sales, or other purposes.

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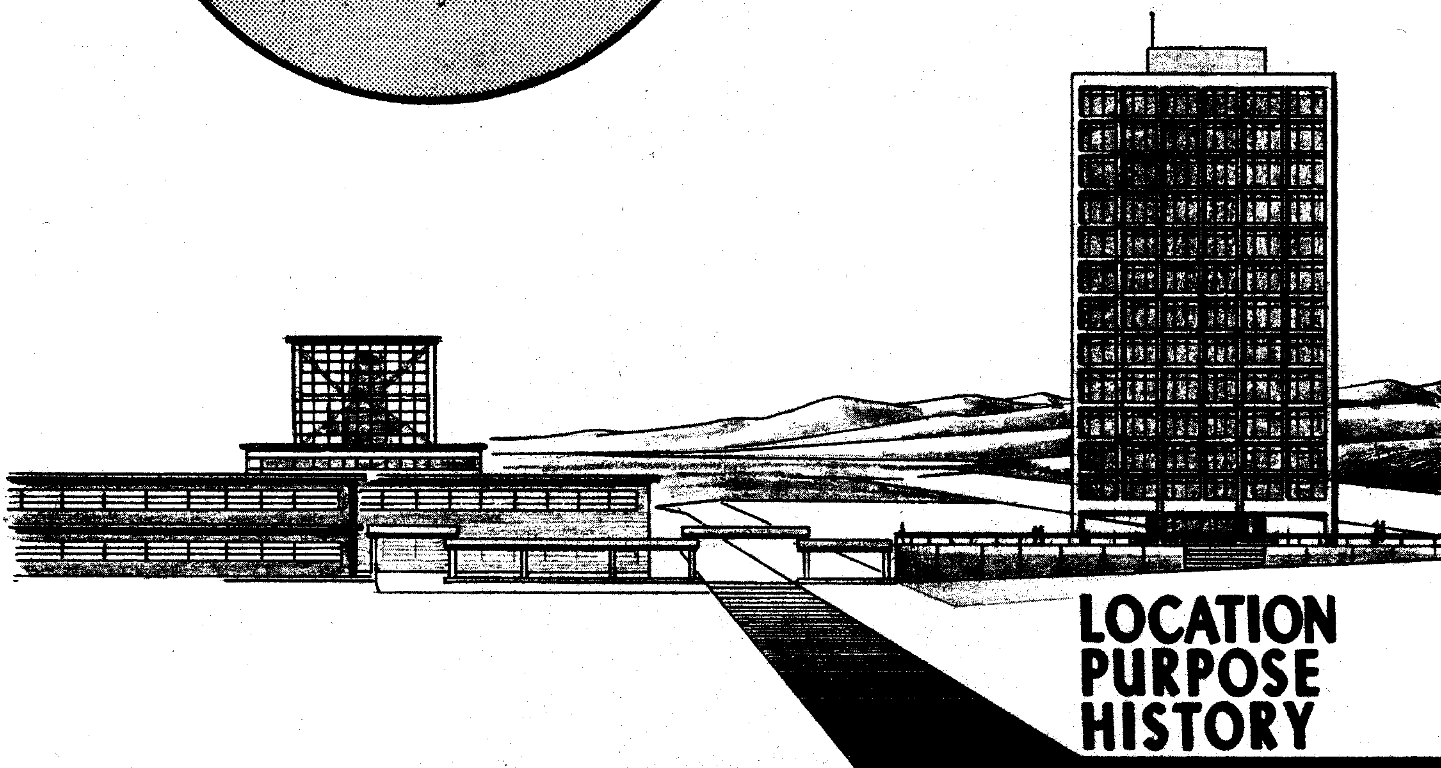
PART I



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PART I-INTRODUCTION

CHAPTER I. HISTORY AND DESCRIPTION

1. COLORADO RIVER STORAGE PROJECT. The Colorado River Storage project provides for the comprehensive development of the Upper Colorado River Basin. The project furnishes the long-time regulatory storage needed to permit States in the upper basin to meet their flow obligation at Lee Ferry, as defined in the Colorado River Compact, and still utilize their apportioned water.

Water stored by the project will provide a portion for direct use in the upper basin and, in addition, will control sediment, control flooding, facilitate recreational development, and aid in fish and wildlife conservation. A significant amount of electrical energy is created through project development to meet the needs of the upper basin and adjacent areas.

The project includes four storage units as follows: Glen Canyon (the subject of this publication) on the Colorado River in Arizona near the Utah border, Flaming Gorge on the Green River in Utah near the Wyoming border, Navajo on the San Juan River in New Mexico near the Colorado border, and Curecanti on the Gunnison River in west-central Colorado. Authorized with and linked to the Colorado River Storage project, but not part of it, are a number of participating projects which will share in the power revenues of the larger project to help pay for irrigation construction costs. These participating projects are listed in Subsection (c). Figure 1 is a location map of the Colorado River Storage project while the frontispiece shows the completed Glen Canyon Dam.

(a) *Plan.*—The reservoirs formed by the four units of the Colorado River Storage project have a total capacity of nearly 34 million acre-feet. During periods of low streamflow, the stored water in the upper basin is released to meet the Lee Ferry obligation and, in exchange, upstream flow is diverted for use in the upper basin. Powerplants and other pertinent facilities are provided at each dam except Navajo, and a complex transmission system has also been provided. This transmission system will carry Colorado River Storage project (CRSP) power to key load points in the marketing area. The system is integrated with preference-user and private-company transmission lines to form the CRSP Interconnected Transmission system. CRSP hydropower is delivered to the preference-user organizations for distribution to their consumers as required by Federal Reclamation law.

2. UPPER DRAINAGE BASIN DEVELOPMENT. (a) *Early History.*—Settlement of the upper drainage basin began in 1854 when the early pioneers established Fort Supply in Wyoming on the Emigrant Trail and diverted water from Blacks Fork to the adjacent lands. Breckenridge, Colo., on the basin's eastern rim, was settled in 1859 by miners and prospectors pushing over the mountains from older mining districts on the eastern slope of the Continental Divide. Within the next decade, other mining camps were established nearby. Unsuccessful miners turned to farming and supplied agricultural products to the mining communities. Settlements grew downward from the mountains to the valleys, the advance being slowed somewhat by conflicts with the Indians who occupied the territory. Grand Junction, Colo., now the largest community in the upper drainage basin, was not settled until 1882. The greater part of the Uinta Basin in northeastern Utah was established as an Indian reservation in 1861, and lands unoccupied by Indians were not open to settlement until 1905. Most lands of agricultural importance in the San Juan River Basin in Colorado, New Mexico, and Arizona were once included in Indian reservations, and substantial areas are still under Indian control. Numerous tributary streams in the upper drainage basin have been diverted to irrigate meadows and mountain valleys and farmlands and broader valleys at the base of the mountains.

(b) *Investigations.*—Investigations of means to develop the waters of the Upper Colorado River system were started by the Reclamation Service (predecessor of the Bureau of Reclamation) in 1902, the year of its organization. Since that year, many of the larger irrigation projects within the basin have been undertaken with Federal assistance, and the Bureau of Reclamation has constructed, or is now constructing, 25 projects to utilize water in the upper basin. The need for the Colorado River Storage project was envisioned at the time of the Colorado River Compact of 1922. In dividing Colorado River water between the Upper and Lower Colorado River Basins, the compact set aside for consumption in the upper basin 7,500,000 acre-feet of water each year. However, this allocation is contingent upon the upper basin's delivering to the lower basin not less than 75 million acre-feet of water in any period of 10 consecutive years and delivering additional water for use in Mexico under certain circumstances. The dividing point between the two basins is at Lee Ferry, near the northern border of Arizona. Water allocated to the upper basin was further apportioned to the individual States of Arizona, Colorado, New Mexico, Utah, and Wyoming by the Upper Colorado River Basin Compact of 1948.

This compact also created the Upper Colorado River Commission, consisting of representatives of the Federal Government and each contracting State except Arizona.

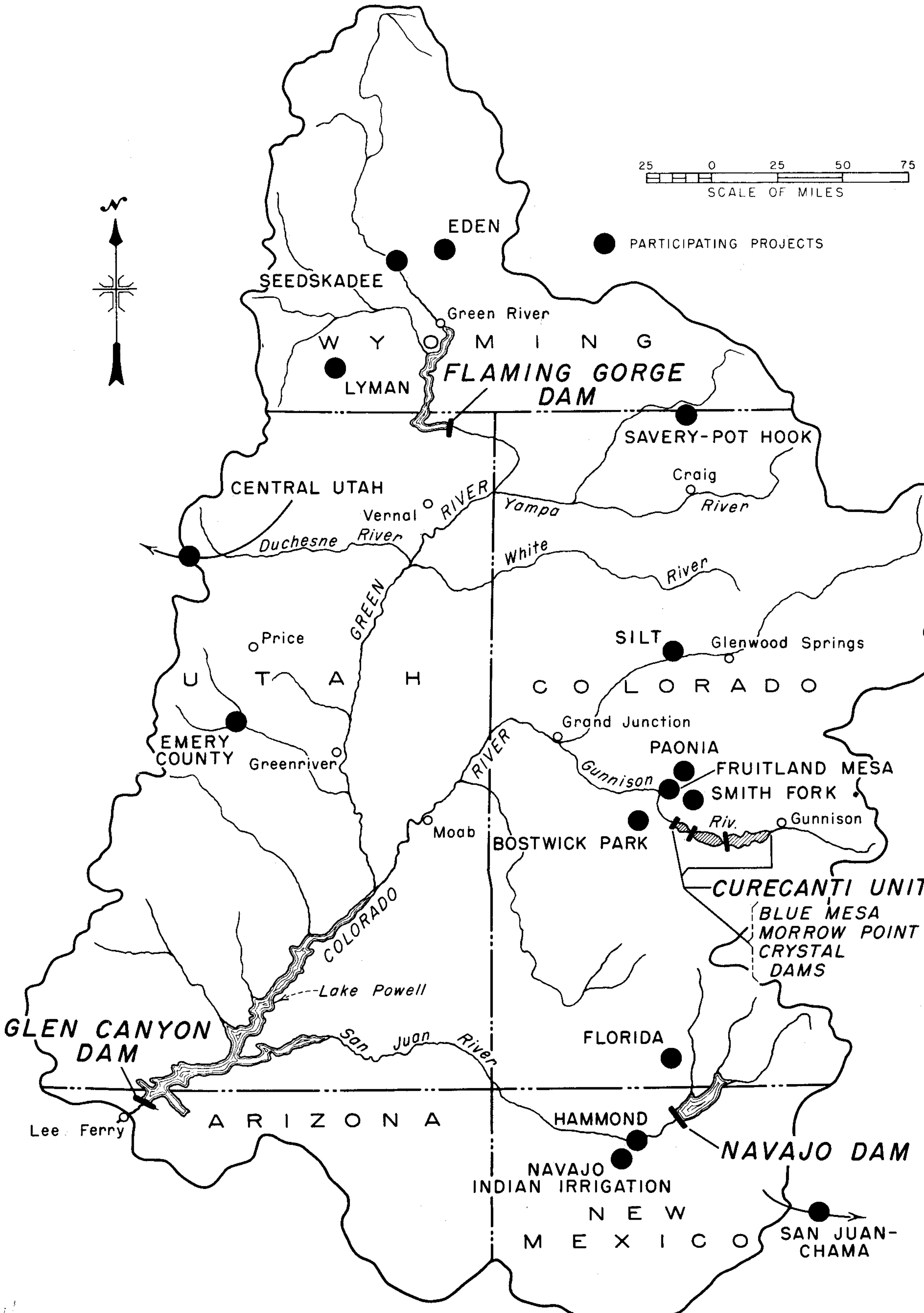

Figure 1.–Colorado River Storage project location map.

The flow of the Colorado River is extremely erratic, ranging from 4 to 22 million acre-feet annually at Lee Ferry. There is a tendency for the high years or the low years to be grouped, thus accentuating problems of river regulation and use. In prolonged dry periods, there is not enough water to permit the upper basin to increase its use of water under the 1922 compact and, at the same time, make the required deliveries to the lower basin. In wetter periods, however, flows are more than sufficient for these purposes. Large storage reservoirs, that can be filled when flows are high and that can provide additional water when needed for compact fulfillment, are required. Favorable sites for such reservoirs are found in the deep canyons of the Colorado River and its principal tributaries in the upper basin. A plan for the Colorado River Storage project, including a series of dams and reservoirs to provide storage capacity in combination with power development and other services, was presented in a Bureau of Reclamation report in 1950, which was subsequently printed as House Document 364, 83d Congress, 2d session. The report was formulated in cooperation with other Federal agencies and with the Upper Colorado River Commission. An initial group of participating projects that would develop water for irrigation and other purposes in the upper basin and that would be linked financially with the Storage project was also described in the 1950 report. Following several years of congressional deliberation, the project was authorized in 1956.

(c) *Authorization*.—Construction of 4 storage units of the Colorado River Storage project and 11 participating projects was authorized by the act of April 11, 1956 (Public Law 485, 84th Cong., 70 Stat. 105). Additional projects have been added since the original legislation was adopted. Authorized developments are:

- Glen Canyon unit on the Colorado River in Arizona and Utah,
- Flaming Gorge unit on the Green River in Utah and Wyoming,
- Navajo unit on the San Juan River in New Mexico and Colorado, and
- Curecanti unit, consisting of three dams on the Gunnison River in Colorado.

Participating projects originally authorized are:

- Central Utah (initial phase), Utah,
- Emery County, Utah,
- Florida, Colorado,
- Hammond, New Mexico,
- La Barge, Wyoming,[1]
- Lyman, Wyoming and Utah,
- Paonia, Colorado (works additional to existing project),
- Pine River extension, Colorado and New Mexico,[1]
- Seedskadee, Wyoming,
- Silt, Colorado, and
- Smith Fork, Colorado.

The Eden project in Wyoming, by terms of its authorizing act of June 28, 1949, became financially related to the Colorado River Storage project as a participating project. In 1962, their authorizing legislation named the following two as participating projects:

- San Juan-Chama, Colorado and New Mexico, and
- Navajo Indian Irrigation (being constructed for the Bureau of Indian Affairs by the Bureau of Reclamation).

And in 1964, the following three projects were also named:

- Bostwick Park, Colorado,
- Fruitland Mesa, Colorado, and
- Savery-Pot Hook, Colorado and Wyoming.

(d) *Benefits*.—The Upper Colorado River Basin has a scarcely tapped potential of agricultural, industrial, and recreational assets. It contains tremendous quantities of uranium, coal, and other minerals. Realization of the potential in economic growth and contribution to the national welfare is dependent on maximum utilization of limited water supplies. The Colorado River Storage project and participating projects conserve the very limited precipitation which falls principally in the form of snow in the high mountains and utilize it for municipal, industrial, and agricultural growth. Project development provides municipal and industrial water supplies, flood control, extensive recreation, and fish and wildlife preservation.

3. LOCATION AND PURPOSE OF GLEN CANYON UNIT. Glen Canyon Dam was constructed on the Colorado River in north-central Arizona, about 15 miles upstream from Lee Ferry and 12 river miles downstream from the Arizona-Utah State line. The dam is a concrete-arch structure, 710 feet high above foundation and has a volume of 4,901,000 cubic yards. At the time of construction (1956-64) it was the second highest dam in the Western Hemisphere, exceeded in height only by the 726-foot-high Hoover Dam. The location of Glen Canyon Dam and the Glen Canyon Powerplant is shown on figure 2

The reservoir (fig. 3) impounded by the dam, named Lake Powell in honor of Major John Wesley Powell,

[1] Later found to be infeasible and deleted from the plan.

renowned explorer of the Colorado River and its tributaries, has a total storage capacity of 27,000,000 acre-feet and will extend 186 miles up the Colorado River and 71 miles upstream on the San Juan River, with 1,900 miles of shoreline. The reservoir is a major storage feature to provide the longtime regulatory storage needed to permit the States of the Upper Colorado River Basin to utilize their apportioned water and still meet their flow obligations at Lee Ferry, Ariz.,[2] under the terms of the 1922 Compact of the Colorado River (see sec. 2).

The 900,000-kilowatt Glen Canyon Powerplant will provide the principal portion of the electrical energy generated by the Colorado River Storage project. Surplus revenue from sale of this energy will assist irrigators in the Upper Basin to repay costs of constructing the participating projects which were authorized by the Congress in 1956 to be developed with the Colorado River Storage project.

4. GENERAL DESCRIPTION OF GLEN CANYON FEATURES. (a) *Dam.*—The dam is a constant-radius concrete arch with fillets. It has a structural height of 710 feet and a crest length of approximately 1,560 feet. The crest of the dam is at elevation 3715, 583 feet above the riverbed, and accommodates a 35-foot-wide roadway which is a service road for the dam and provides access between the spillways.

A general plan of Glen Canyon Dam and Powerplant is shown on figure 4 and elevation and sections are shown on figure 5. Figures 68 and 69 show the general arrangement of the gallery system of the dam and the access to mechanical equipment in the dam.

(b) *River Outlets.*—Four 96-inch-diameter steel-lined river outlets are installed near the left abutment, extending from the upstream face of the dam to a point approximately 150 feet downstream from the machine shop. Each outlet is provided with a 96-inch hollow-jet valve at the downstream end, for regulation, and a 96-inch ring-follower gate located in the dam at elevation 3374 for shutdown emergency closure. One bulkhead gate is provided to close off one outlet at a time for inspection and maintenance of the four ring-follower gates and the outlet pipes upstream from the ring-follower gates. The intakes for each pair of outlets are protected by a trashrack structure on the upstream face of the dam. General plan and profiles of the river outlets are shown on figures 109 and 110.

(c) *Spillways.*—One spillway is provided on each abutment. Each spillway consists of an approach channel, intake structure, spillway tunnel, and deflector bucket. Discharges are controlled by two 40- by 52.5-foot radial gates in each intake structure. The deflector bucket flips the water downstream from the spillway and away from the canyon to prevent undercutting of the canyon wall. General plans and profiles of the spillways are shown on figures 100 and 101.

(d) *Power Waterways.*—Water is conveyed from the reservoir to the hydraulic turbines by eight 15-foot-diameter penstocks, extending from the upstream face of the dam to the powerplant. The intake to each penstock is protected by a trashrack. A 13.96- by 22.45-foot fixed-wheel gate is provided at the face of the dam for emergency closure and for inspection and maintenance of each penstock. Water from the turbines is collected by the draft tubes and carried to the tailrace. Bulkhead gates are provided at the downstream end of the draft tubes for unwatering the draft tubes. General plan and profiles of the penstocks are shown on figures 129 and 130.

(e) *Powerplant.*—The powerplant is an indoor-type structure with eight generating units, a service bay, a machine shop bay, and a control area located on top of the powerplant over unit bays 1 and 2. Each generating unit has a capacity of 125,000 kilovolt-amperes at 0.9 power factor. The turbines, of the Francis type, are rated at 155,500 horsepower at full gate opening when operating at 150 revolutions per minute under an effective head of 450 feet. The effective head on the turbines may range between 341 and 560 feet.

The control area consists of a control room, cable spreading area, and office space. The control area is designed to provide fallout protection for the operating personnel. Because of the remoteness of assumed targets relative to Glen Canyon, residual radiation from fallout should be very low; therefore, full-time occupancy by the operating personnel of the powerhouse for the usual 14-day period should be unnecessary in the predictable future.

The powerplant structure is of reinforced concrete construction in the substructure and intermediate structure, with a superstructure of exposed structural-steel columns and reinforced concrete wall panels. The general arrangement of the

[2] The Colorado River Compact provides principally for a division of the available water of the Colorado River system between the "Upper Basin" and the "Lower Basin" at Lee Ferry, which is defined as a point on the Colorado River 1 mile below the mouth of Paria River. The nearest stream gage to this point on the Colorado River is at Lees Ferry, which is above the mouth of the Paria River. Lee Ferry, a few miles below the Arizona-Utah boundary, is a natural point of demarcation.

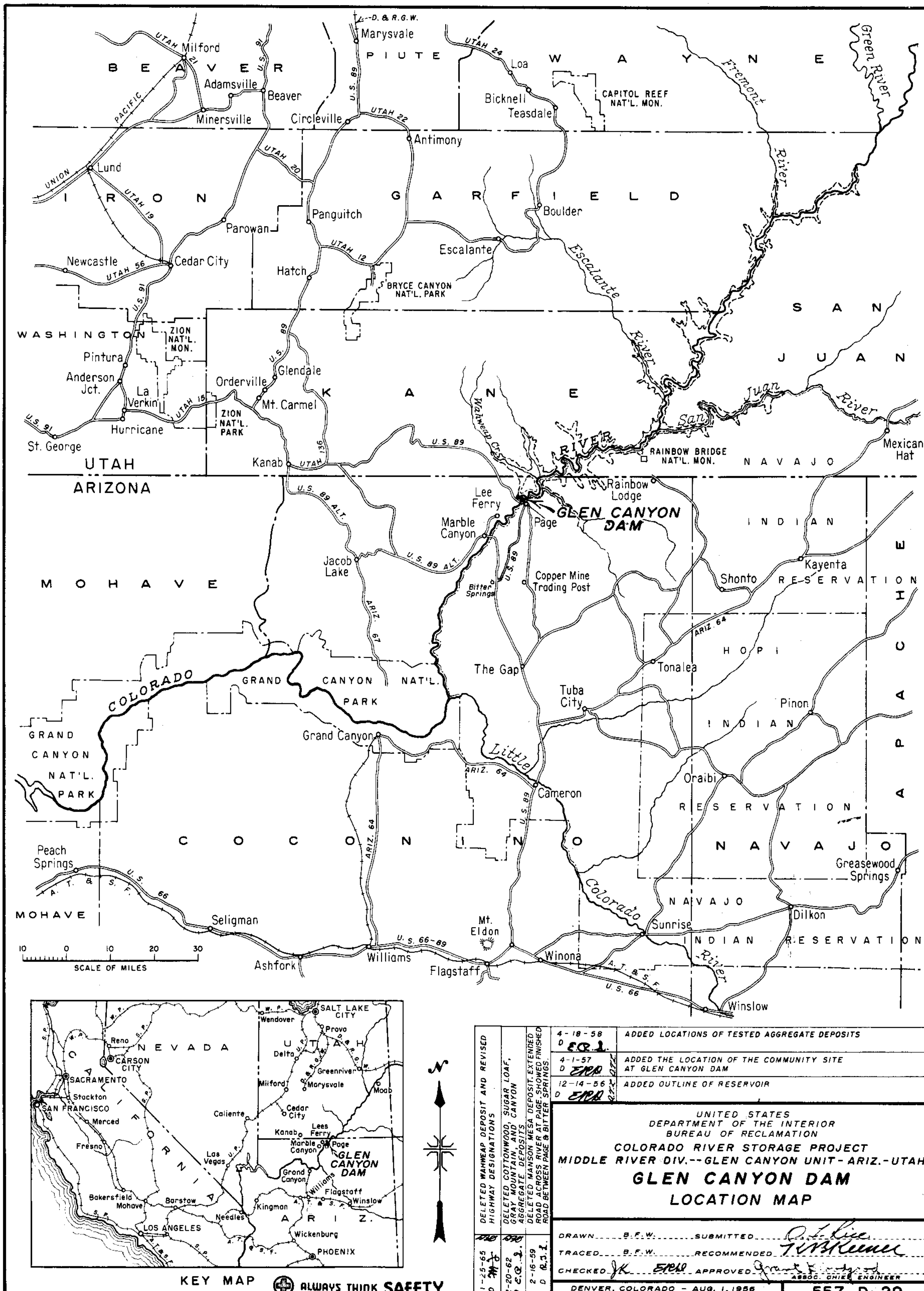

Figure 2.—Glen Canyon Dam location map.

powerplant is shown on figures 150 through 155 and figure 158.

(f) *Switchyard.*—The switchyard is located within a fenced area 1,190 by 473 feet, approximately 850 feet southwest of the right abutment of the dam. The elevation of the switchyard ranges from 3886.00 to 3883.62 feet for drainage purposes. The switchyard is designed to serve loads at 345, 230, and 25 kilovolts initially. The initial equipment includes a 345- to 230- to 25-kilovolt, 300,000-kv.-a. bank of three single-phase autotransformers; a 25-kilovolt regulator; a 25-kilovolt series reactor; a 25-kilovolt grounding transformer; three bays of 345-kilovolt breaker-and-one-half bus; three bays of 230-kilovolt breaker-and-one-half bus; and five bays of 25-kilovolt main and transfer bus. The general arrangement of the switchyard is shown on figure 216.

(g) *Glen Canyon Bridge.*—The bridge across the Colorado River at Glen Canyon Dam is the subject of a prior technical record of design and construction and is available as indicated on the inside of the back cover of this technical record. As an aid to the reader, a portion of that publication has been reproduced and is presented as appendix E.

(h) *Access Roads.*—Owing to the isolation of the Glen Canyon damsite it was necessary to construct an access highway from U.S. Highway No. 89 in the vicinity of Echo Cliffs (Bitter Springs on fig. 2) to the damsite. The nearest railhead was at Flagstaff, Ariz., on U.S. Highway No. 89. The construction of Page, Ariz., as a construction camp and permanent community necessitated building connecting roads to the access highway and to the dam. Access to the dam crest and both spillways was provided by service roads on each abutment which interconnected with the dam access highway. The service road to the powerplant was constructed on the left abutment from a point on the access highway downstream from the Glen Canyon Bridge to the powerplant at the toe of the dam. Owing to the sheer canyon walls, the last approximately 2 miles of service road was constructed in tunnel section. The State of Arizona constructed a road from the vicinity of the west abutment of the Glen Canyon Bridge to the Arizona-Utah State line to provide access from Utah. This road was later reshaped and surfaced by the Bureau and connected to the access highway at Glen Canyon Bridge, providing an all weather road to communities in Utah. A detailed description of each road is given in Chapter X.

5. CLIMATE. The climate at Glen Canyon Dam is typical of the arid plateaus in the western deserts. Summers are hot and dry, with occasional thunderstorms. Winters are dry, with frost at night the general rule. The percentage of sunshine is very high, averaging about 80 percent of the total possible. The relative dryness of the air modifies the effect of the summer heat and winter cold so that the temperature extremes are not too noticeable. High winds and sandstorms occasionally occur during the spring and summer months.

Prior to the construction of the dam, the nearest U.S. Weather Bureau station was at Lee Ferry, Ariz., about 15 miles downstream from Glen Canyon Dam. At Lee Ferry, the rainfall over the 1916-1952 period ranged from 3 to 10 inches per year, with an average of about 6 inches per year. A large part of this rainfall occurred in thunderstorms during the months of July, August, and September. A total annual snowfall of between 3 and 5 inches normally occurs in the area in December and January, although spring snows in March are not uncommon.

The mean annual air temperature at Lee Ferry for the period 1931-1952 was 62.5° F. This compares with the 6-year record at the dam of 61.6° F. at the canyon rim and 64.6° F. at the bottom of the canyon. During the hottest part of the year (usually July), daily temperatures normally range from 73° F. at night to 101° F. during the day at the canyon rim, and from 76° to 105° F. in the bottom of the canyon. During the coldest part of the year (December and January), the daily temperatures normally range from 26° F. at night to 46° F. during the day at the canyon rim, and from 28° to 47° F. in the bottom of the canyon. The average date for the first killing frost is October 29 and for the last killing frost is April 11, making the length of the average growing season 201 days.

6. HISTORY AND SETTLEMENT OF THE AREA. The first white man known to have visited the Colorado River was the Spanish explorer, Hernando de Alarcon. In 1540, while exploring the Gulf of California, he found the mouth of the then unknown river, and ventured up the reddish-brown stream some 150 miles. Two years later the Grand Canyon of the Colorado was discovered by Cardenas. Cardenas never succeeded in descending the sheer walls of the canyon, and other explorers and missionaries who followed him were also discouraged by the seemingly hopeless task of penetrating this section of the canyon. Two centuries passed before a passage was discovered permitting a crossing.

By the treaty concluding the Mexican War in 1848, and by the Gadsden Purchase of 1853, the United States acquired the territories of New Mexico, Arizona, and California. Discovery of gold in California in 1849

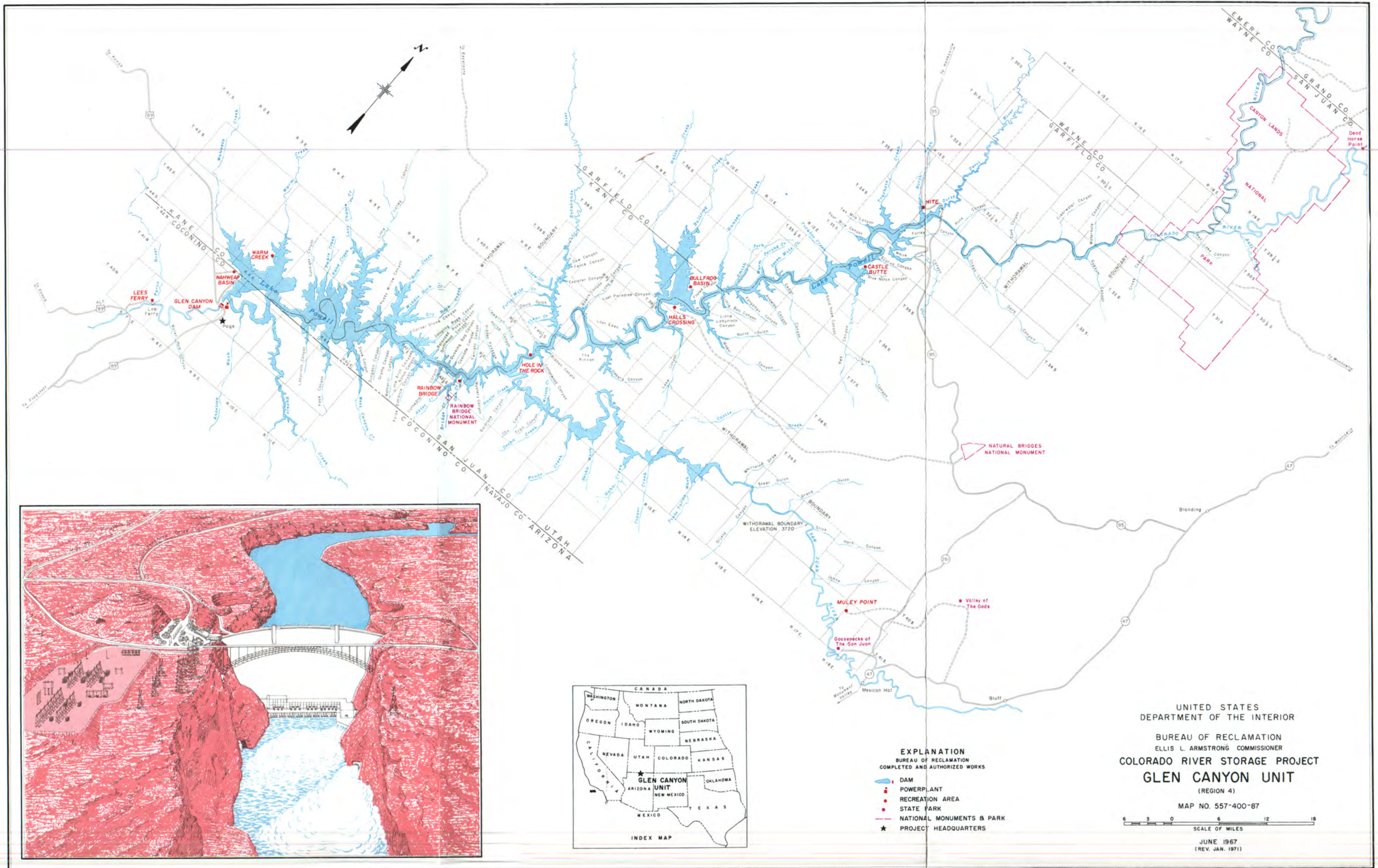

Figure 3.—Glen Canyon Unit location map.

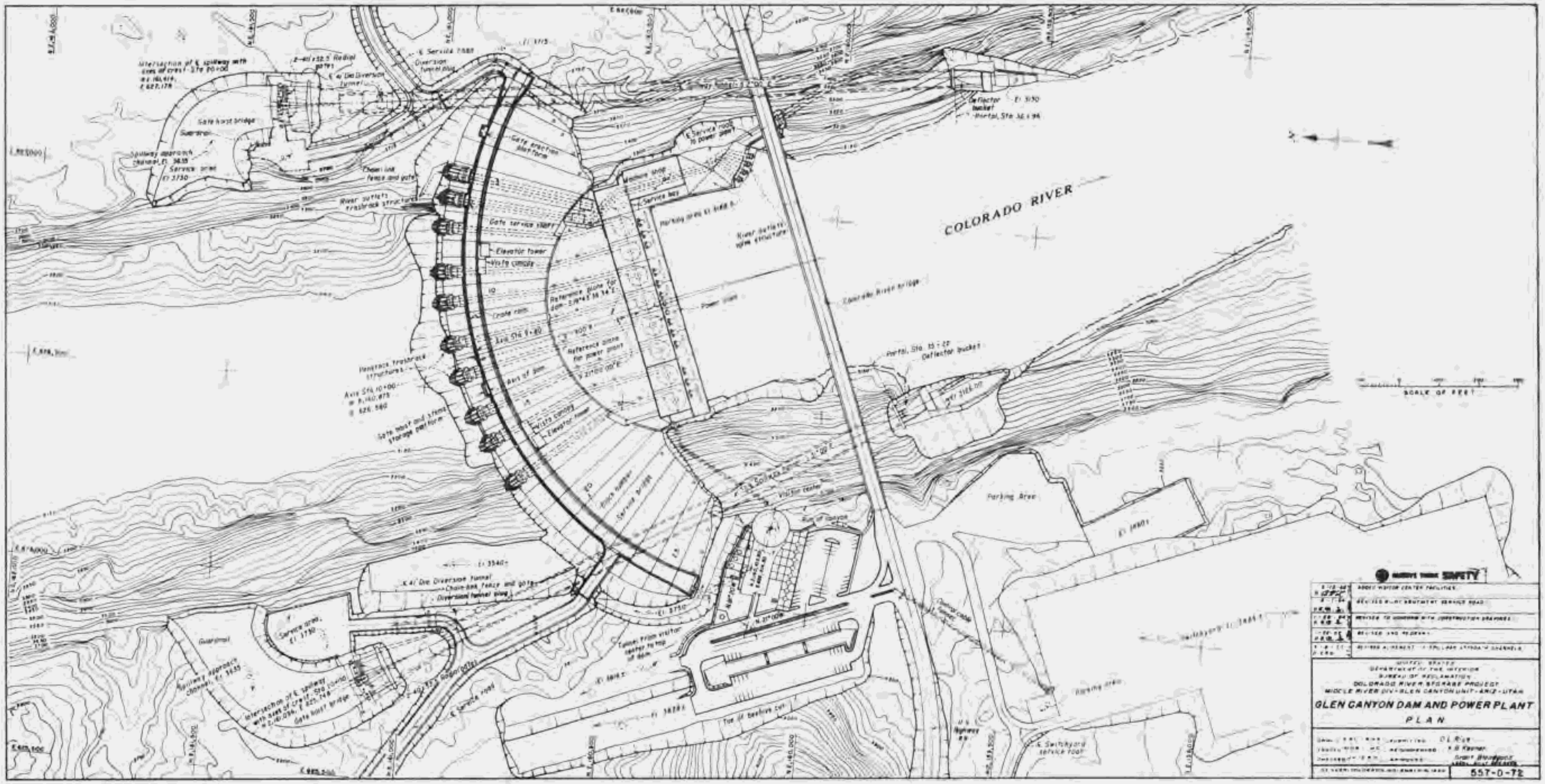

Figure 4.—Glen Canyon Dam and Powerplant—Plan.

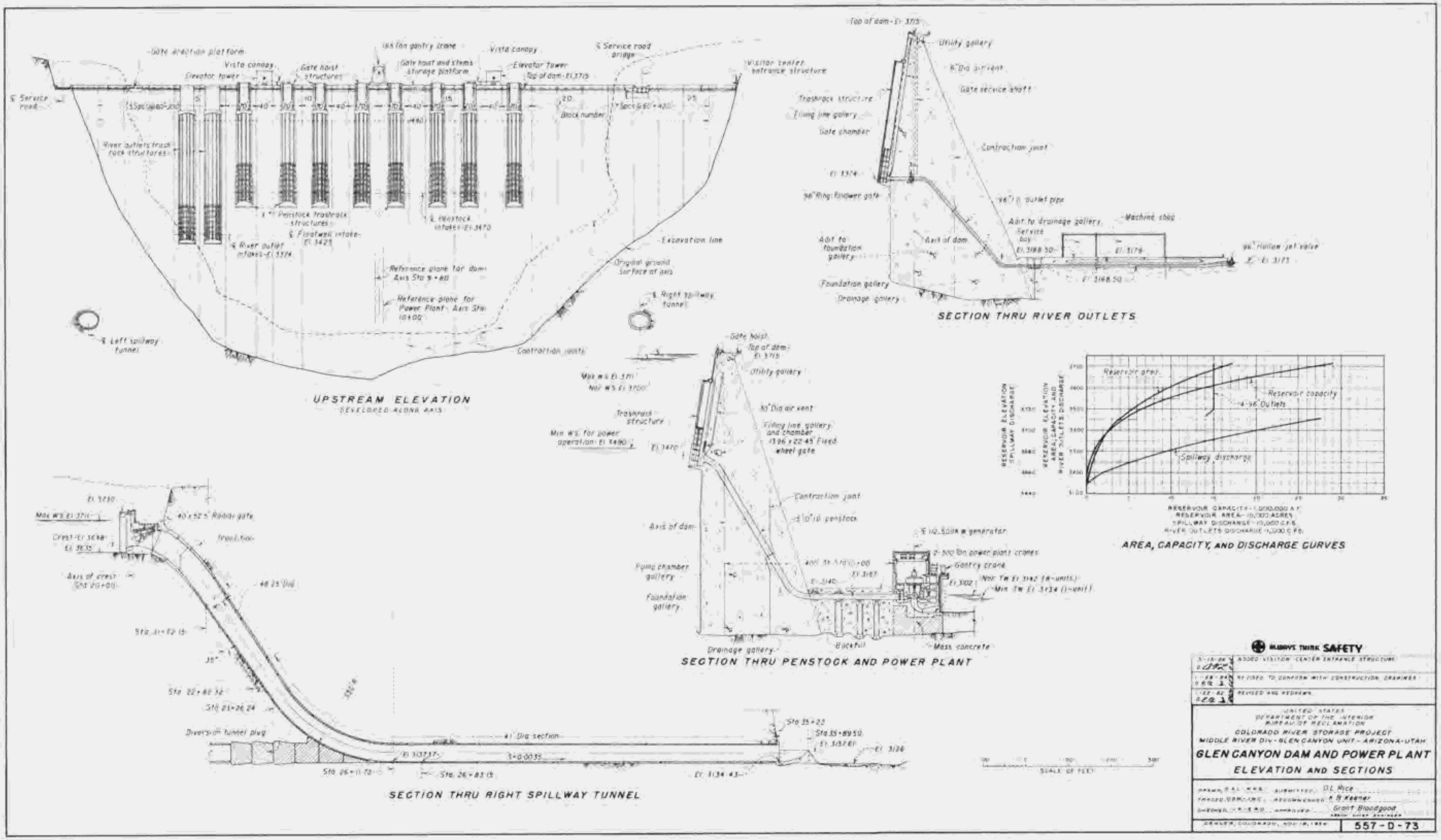

Figure 5.—Glen Canyon Dam and Powerplant—Elevations and sections.

brought hordes of adventurers westward. They poured across the Colorado River at two points. One crossing was near Yuma, Ariz., and the other at Needles, Calif.

With the acquiring of the territory, sentiment grew in the United States that the area should be explored at all costs. In 1857 the War Department dispatched Lieutenant J. C. Ives to the task, instructing him to proceed up the river by boat as far as practicable. He succeeded in going 400 miles up the river to the Black Canyon, present site of Hoover Dam. He reported the region to be valueless.

In 1869 Major J. W. Powell of the Geological Survey succeeded in leading a river expedition down the canyon of the Colorado. His expedition traveled from Green River in Utah to the Virgin River in Nevada, a few miles above where Lieutenant Ives stopped. Powell covered a thousand miles of unknown rapids and treacherous canyons, and became the first white man to gaze up at the sheer walls of the Grand Canyon and live to describe the adventure. Further discussion of Major Powell's investigations is given in section 9.

Following this, efforts were made to investigate methods whereby the river might be used beneficially. The river, annually fed by melting snows in the Rockies, swelled to a raging flood in the spring, then dried to a trickle in the late summer and fall, so that crops were frequently destroyed. Farmers built levees to keep out the river. Even when the levees held, however, crops withered and died during the months when the river ran too low to be diverted into the canals.

The flow of the Colorado River is extremely erratic, ranging from 4 to 22 million acre-feet annually at Lee Ferry. There is a tendency for the high years or the low years to be grouped, thus accentuating problems of river regulation and use. Faced with constantly recurring cycles of flood and drought, the people of the Southwest appealed to the Federal Government to solve the problem. With the establishment in 1902 of the Reclamation Service, engineers began extensive studies of the river in search of a feasible plan for its control. The completion of Hoover Dam in 1936 undoubtedly did more to regulate the erratic flow pattern of the mighty Colorado than any other single item of construction. However, the upper reaches of this river were still more or less uncontrolled. The Glen Canyon Dam was one of the key structures depended upon for additional regulation of this river system.

7. COST SUMMARY. The following tabulation summarizes the total estimated cost of construction of Glen Canyon Dam and Powerplant and appurtenant structures based on information contained in the Final Construction Report. The costs are as of June 30, 1967. A listing of contracts and purchase orders is contained in appendix A.

COST SUMMARY

Land rights	$ 421,876	
Labor by Government forces	394,904	
Construction facilities	905,768	
Construction by contract	178,087,355	
Materials furnished by Government	35,192,407	
Subtotal		$215,002,310
Investigations, engineering, and other costs:		
Investigations	$ 1,560,384	
Engineering and supervision	8,634,019	
Design and specifications	9,669,859	
General services	7,797,749	
Service facilities	2,348,664	
Subtotal		$ 30,010,675
Total		$245,012,985
Less transfers and credits		142,435
Total estimated cost of Glen Canyon Dam and Powerplant and appurtenant structures		$244,870,550

CHAPTER II. GEOLOGY

8. REGIONAL GEOLOGY. (a) *Physiography*.—Glen Canyon Dam is situated in the Colorado Plateau province, a vast area of nearly horizontal beds which have been elevated without materially disturbing the component layers. It is characterized by broad, cliff-edged mesas cut by narrow, steep-walled canyons. The general elevation of the province ranges from 4,000 to 11,000 feet above sea level. Between these two elevations are plateaus at various altitudes.

Glen Canyon embraces a 200-mile section of the Colorado River from the lower end of Cataract Canyon, 14 miles above Hite, Utah, to Lee Ferry, Ariz. This canyon is one of the more spectacular canyons cut by the Colorado River. Throughout most of its length, Glen Canyon is a narrow river gorge confined by near-vertical, massive, red sandstone walls with heights up to a maximum of 1,200 feet above the river.

Formations range from Permian to Tertiary, with the bulk of the sedimentary rocks being Mesozoic. Sandstone is the predominant rock type, but numerous shale formations are present. The Glen Canyon damsite is in the Jurassic Navajo sandstone. The following is a generalized stratigraphic section applicable to a broad area:

	(Unconformity)
Mesozoic Upper Jurassic San Rafael Group	Summerville Formation Entrada Sandstone Carmel Formation—pink to red and bluish sandy shale; white and buff sandstone
	(Unconformity)
Mesozoic Lower Jurassic Glen Canyon Group	Navajo Sandstone—(Thickness 1,200 to 1,800 feet) Massive, cross-bedded sandstone; few very thin limestone lenses (Local Unconformity) Kayenta Formation—Maroon, cross-bedded sandstone; conglomerate; blue-gray, hard, dense limestone; maroon and brown shale. All in thin irregular beds Wingate Sandstone
	(Unconformity)

(b) *Structure*.—The Glen Canyon Area is a part of the Kaiparowits region which Gregory[1] describes as follows: "As suggested by the wide, sensibly flat plateaus and long, even crested escarpments, the rock beds throughout most of the Kaiparowits region are gently inclined or nearly horizontal. This simple general attitude is interrupted in places by sharp monoclinal flexures, which trend in a general northerly direction and subdivide the region into large gently tilted blocks. In places minor undulations interrupt the otherwise regularly inclined beds between the monoclines. West and northwest of the Kaiparowits region northward-trending faults divide the plateaus into blocks not unlike those produced by the monoclines. In each of the three monoclinal folds that traverse central southern Utah the dip of the beds is eastward and the rocks on the west are elevated and those on the east depressed. Along each of the faults the movement is in the opposite direction, the rocks on the east being elevated and those on the west dropped. The monoclinal folds affect all the rocks from the uppermost Cretaceous downward but do not involve the Tertiary, whereas the faults displace the Tertiary beds as well. The displacements of the two types are thus of different geologic age.

"Aside from the deflections that are due to the monoclinal folds, the general inclination of the beds in the southern part of the Kaiparowits region is northward, for the rocks here constitute the north flank of the broad Grand Canyon upwarp in Arizona."

Local uplifts, due to igneous intrusions, are represented in the Glen Canyon area by the Navajo and Henry Mountains. Evidence of other igneous activity in the area is found only in the recent basalt flows which cover parts of the higher plateaus.

All of the large faults and monoclinal folds are located a considerable distance from Glen Canyon Dam, so the beds in the latter area have been only slightly disturbed. At the damsite, the massive Navajo sandstone beds lie essentially horizontal with only a slight dip (1° to 2°) upstream and into the left abutment. This is in harmony with the regional dip which is northeastward away from the Grand Canyon uplift.

There are no faults in the immediate vicinity of the damsite. The area is characterized by relatively few joints of two distinct types: (1) Steeply dipping subparallel joints restricted principally to a single set trending NE-SW diagonally across the axis of the dam,

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[1] Gregory, H. E., and More, Raymond C., U.S.G.S. Professional Paper No. 164, "The Kaiparowits Region", 1931, p 118.

and (2) stress-relief joints roughly paralleling the canyon walls. The joints of the former type resulted from regional warping. They are generally tight and relatively clean and continuous; individual joints can be traced across the extensive bedrock exposures for distances of up to 800 to 1,000 feet. The latter type of joints reflect the reduction of loading due to the erosion of the deep canyon; they are discontinuous and spaced from 1 to 5 feet apart, commonly open from 1/16 to 1/4 of an inch. The resulting "sheeted" structure parallels major natural rock faces; it diminishes with depth back of the rock face and disappears within 20 to 50 feet.

(c) *Stratigraphy.*—The Navajo sandstone forms the canyon walls at the damsite and throughout most of the reservoir basin. It has a larger outcrop than any of the other Jurassic formations in the area. At the damsite, the Navajo sandstone, over 1,400 feet thick, extends from approximately 1,000 feet above river level to more than 400 feet below river level.

The Navajo sandstone is buff to reddish, medium to fine grained, and moderately hard to soft. It is massive with pronounced crossbedding and commonly indistinct horizontal bedding. It is composed essentially of quartz grains with a minor amount of feldspar and is poorly to moderately well cemented principally by secondary quartz, chalcedony, and to a much lesser extent by calcite and hematite. The sandstone is moderately porous and highly absorptive, owing to the high capillarity created by the small size of intergrain pore spaces.

The Navajo sandstone is remarkably uniform and homogeneous over wide areas and nearly identical samples can be obtained from areas separated by many miles. Two thin, shaly layers, encountered at elevations 3065 and 3115 in the right abutment keyway excavation were the only changes in the lithology in the entire excavation area.

9. INVESTIGATIONS. (a) *Early History.*—Perhaps the first explorer to traverse the Glen Canyon area was Father Silvestre Velez de Escalante. Father Escalante and his party crossed the Colorado River on September 26, 1776, at a point which has since become known as the Crossing of the Fathers. This crossing has been inundated by Lake Powell, as it was located about 10 miles upstream of Glen Canyon Dam.

Major John Wesley Powell, working under the sponsorship of the Smithsonian Institution, made the first ecologic traverse of the unknown canyons of the Colorado River during the summer of 1869. The section of the Colorado River from the mouth of the Green River to the mouth of the Virgin River, a distance of 539 miles, was covered in 23 days. This first trip was hurried due to the loss of supplies at the beginning of the trip; as a result, Major Powell was able to make only a fraction of his intended studies.

Not satisfied with the results of his first survey, Powell determined that he would once more attempt to pass through the canyon in boats, devoting 2 to 3 years to the trip. Supplies were taken by pack train to several strategic points on the river where they would be available as the boat party progressed. The second expedition left Green River, Wyo., in May 1871. During this second trip and the years following, the parties of the Powell survey completed topographic maps of the area from the Henry Mountains to Kanab, Utah. These maps for many years constituted the sole available information regarding the topography and drainage of this region.

Valuable as were these data collected by the Powell survey, they were not sufficiently detailed for definite planning for the development of the Colorado River. Consequently, in the years 1921 and 1922, the U.S. Geological Survey undertook, in cooperation with power companies, a comprehensive survey of the potential power and water resources of the Colorado River. This survey consisted of taking strip topography of the canyon with more detailed survey of potential damsites. A hydrographic survey, with stream gaging at key locations, constituted an important part of these studies. Six potential damsites were studied from the mouth of the San Juan River to Lee Ferry. The site most favored was the Glen Canyon No. 1, located at RM4 (4 miles upstream from Lee Ferry, Ariz.). Six core holes were drilled at this site with the assistance of the Southern California Edison Co.

Concurrent with the drilling and other surveys at the Glen Canyon No. 1 site, the Bureau of Reclamation employed a Board of Consultants to examine the Glen Canyon damsite along with other sites on the lower Colorado River at Boulder and Black Canyons. This Board pointed out certain undesirable features of site No. 1 and, in particular, one set of rather closely spaced joints which crossed the proposed axis in a diagonal fashion. The Board suggested that the axis be moved downstream where the foundation would be improved without materially increasing the width of section. Work at Glen Canyon was not resumed until 1946 when the Bureau began a reconnaissance survey, thus initiating the studies that led to authorization of the project.

(b) *Choice Of Site.*—A number of damsite locations in Glen Canyon were considered, but planning investigations resulted in the elimination of all but two. These were located at RM4 and RM15, respectively, above Lee Ferry.

Reconnaissance studies of the two sites in 1947 indicated that the RM15 damsite was geologically superior to the RM4 damsite. Although the site at RM4 had serious geologic deficiencies, it had an apparent topographic advantage so it was felt that it should not be discarded without further preliminary testing. The completion of three core holes in the river revealed that soft sandstone with thin shale beds existed at an unfavorable elevation in the foundation; accordingly, it was decided to forego further exploration until some comparable drilling had been done at the RM15 site. Initial phases of the drilling revealed no geologic defects; and, before the initial drilling program was completed, comparative estimates showed that the cost of a dam at this site would be less than at the geologically inferior RM4 site. As a result, the RM4 site was eliminated and the investigation program at the RM15 site expanded to provide sufficient data on foundation conditions and concrete aggregate sources for feasibility design of a concrete dam.

(c) *Construction Materials Investigation.*—Concrete aggregate sources available in the vicinity of Glen Canyon Dam consisted of a high-level terrace deposit on both sides of the Colorado River and one alluvial deposit on Wahweap Creek. A careful search for 60 to 70 miles in all directions—involving extensive study of air photos and ground checks—did not reveal any other important aggregate deposits.

The high-level terrace on the west side of the Colorado River opposite the mouth of Wahweap Creek was the most obvious deposit of sand and gravel in the area, so it was the first discovered. Preliminary test pitting and sampling of this deposit indicated that it was far too small to supply enough aggregate for construction of the dam and appurtenant works. The search was then expanded into the surrounding area.

The Wahweap deposit, a rather unimpressive looking stream deposit, was not at first given serious consideration as a possible source of aggregate. Later, more careful investigation indicated that this deposit contained considerable gravel, and it ultimately became the prime source of aggregate for the dam.

The Manson Mesa deposit is in a high-level terrace located on the east side of the Colorado River. This deposit was covered with blow sand and was so well hidden that it was discovered only by chance and was developed as the aggregate source for the construction of the townsite. Had it not been for this deposit, all of the aggregate required for the construction of the town of Page would have had to be ferried by highline from the west side of the river. The highway bridge over the Colorado River had not been completed at that time (construction on the bridge commenced in 1957 and was completed in 1959).

(d) *Investigations For Final Design And Specifications.*—Preliminary investigations at the mile 15 site were initiated in October 1947, and a total of 28 holes and 2 test drifts was completed. All but five were vertical holes drilled in the bottom of the canyon to outline the bedrock profile and to determine condition of the rock in the foundation area. One horizontal hole and one exploration drift was completed near the base of each abutment. Three shallow holes were drilled near the base of the left abutment to secure 6-inch-diameter core for laboratory testing. Additional field investigations in 1949 included Nx (3-1/2-inch-diameter core hole) and 6-inch-diameter core holes, grouting, and load-bearing tests in and near the left abutment drift.

In the latter part of 1956, additional drilling was initiated to supply supplementary data on the bedrock profile in the river section and to provide samples of the abutment rock for laboratory testing. A total of 49 Nx-Bx (3-1/2- and 2-7/8-inch) holes and two 6-inch-diameter holes were completed. Six were vertical holes about 500 feet deep. They were drilled from the top of and a short distance back from the face of each abutment.

Twelve of the Nx-Bx holes were angle holes (DH A through J, Ln and Kn) drilled in the approximate direction of the maximum principal stresses in the abutments of the dam except DH56-J which (owing to an overhang interfering with the original location) had to be drilled in the direction of the horizontal component of the principal stresses at that elevation. The two 6-inch-diameter holes (La and Ka) were drilled parallel to and a few feet from the lowermost Nx angle holes (Ln and Kn) near the base of each abutment. Figure 6 is a drawing showing the areal geology and location of all but a few of the exploration holes. Figures 7 through 11 are typical geologic cross sections through the damsite.

The test drifts in each abutment were later utilized to perform in situ jacking tests to determine the modulus of elasticity and deformation characteristics of the sandstone. Tests were also performed at the Bureau's Denver laboratories on core samples to determine compressive strength, modulus of elasticity, Poisson's ratio, percent porosity, and percent set. A graphical portrayal of the principal elements of these tests is shown on figures 12 and 13. Geophysical

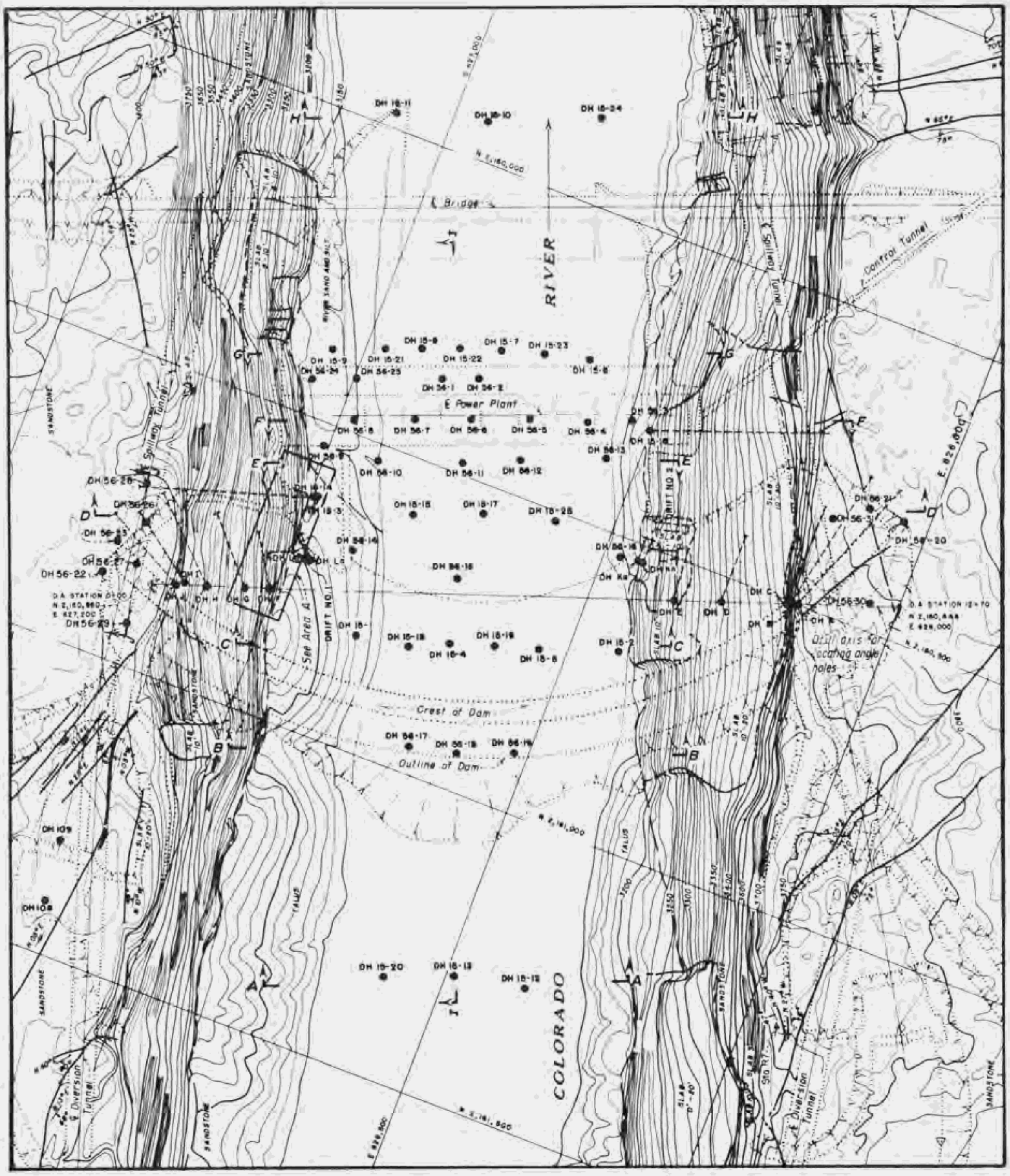


Figure 6.–Topography, areal geology, and location of exploration for dam and powerplant. From drawing No. 557-D-186.

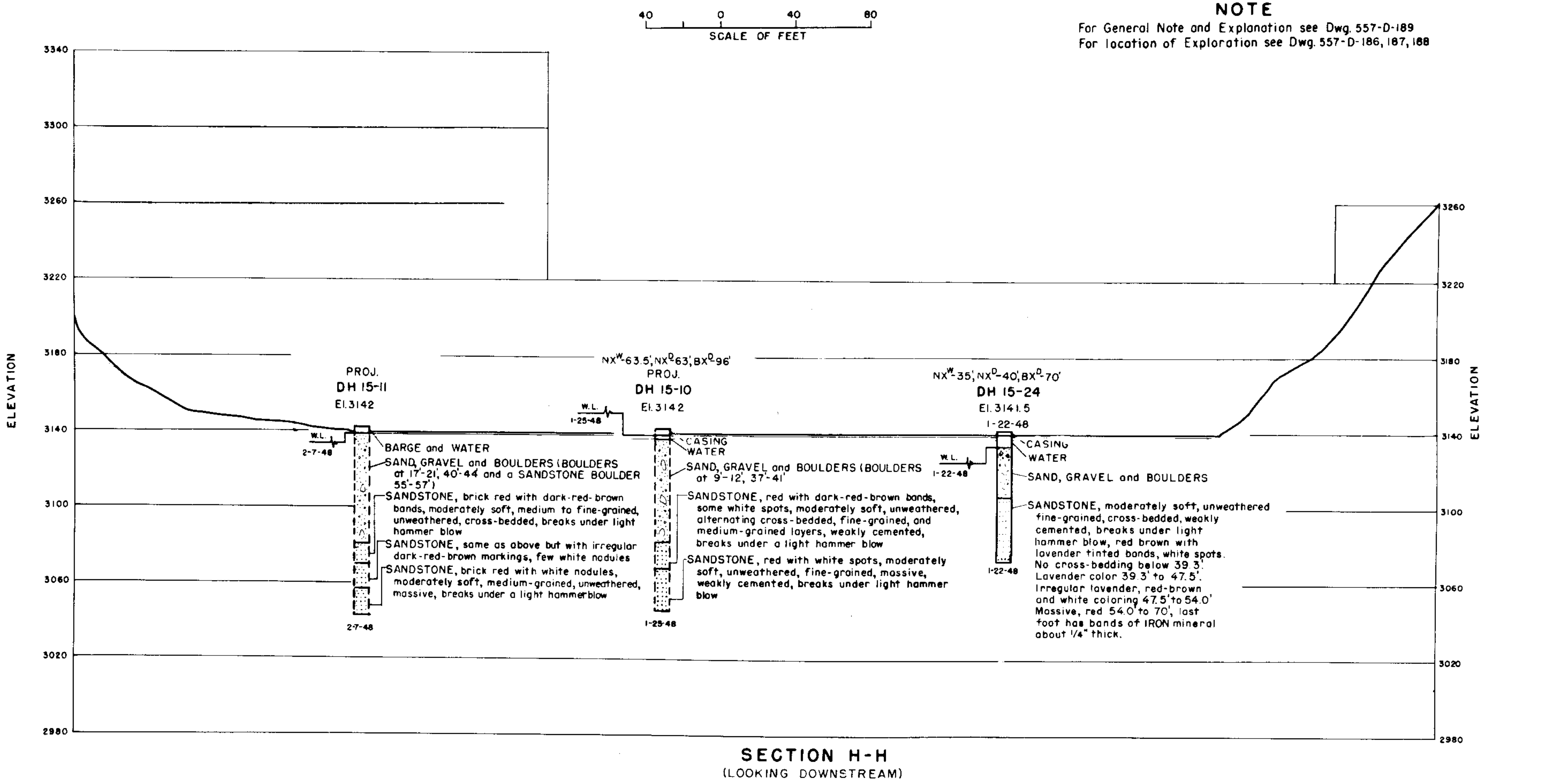


Figure 7.--Logs of exploration on section H-H. From drawing No. 557-D-193.

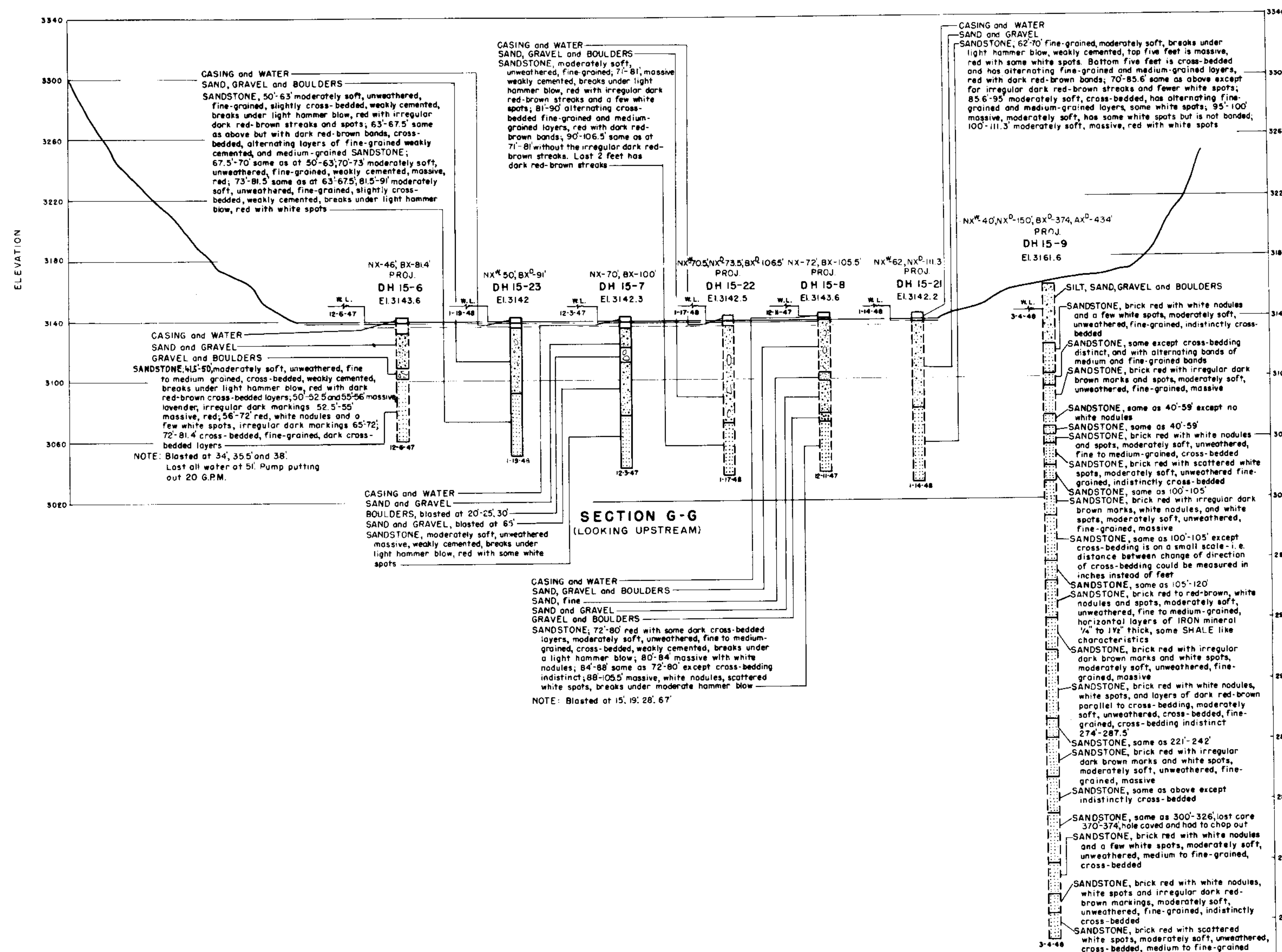


Figure 8.—Logs of exploration on section G-G. From drawing No. 557-D-193.

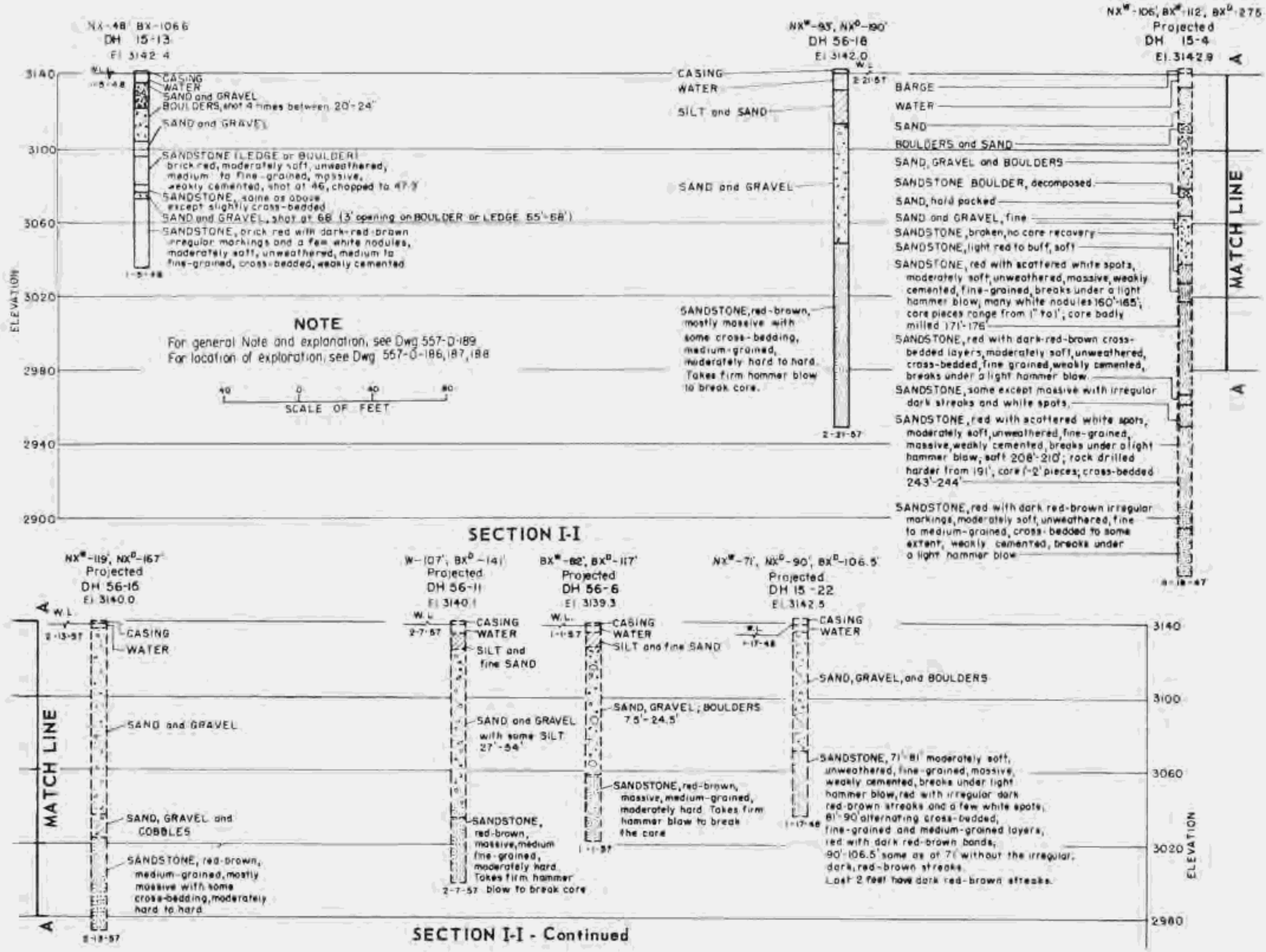
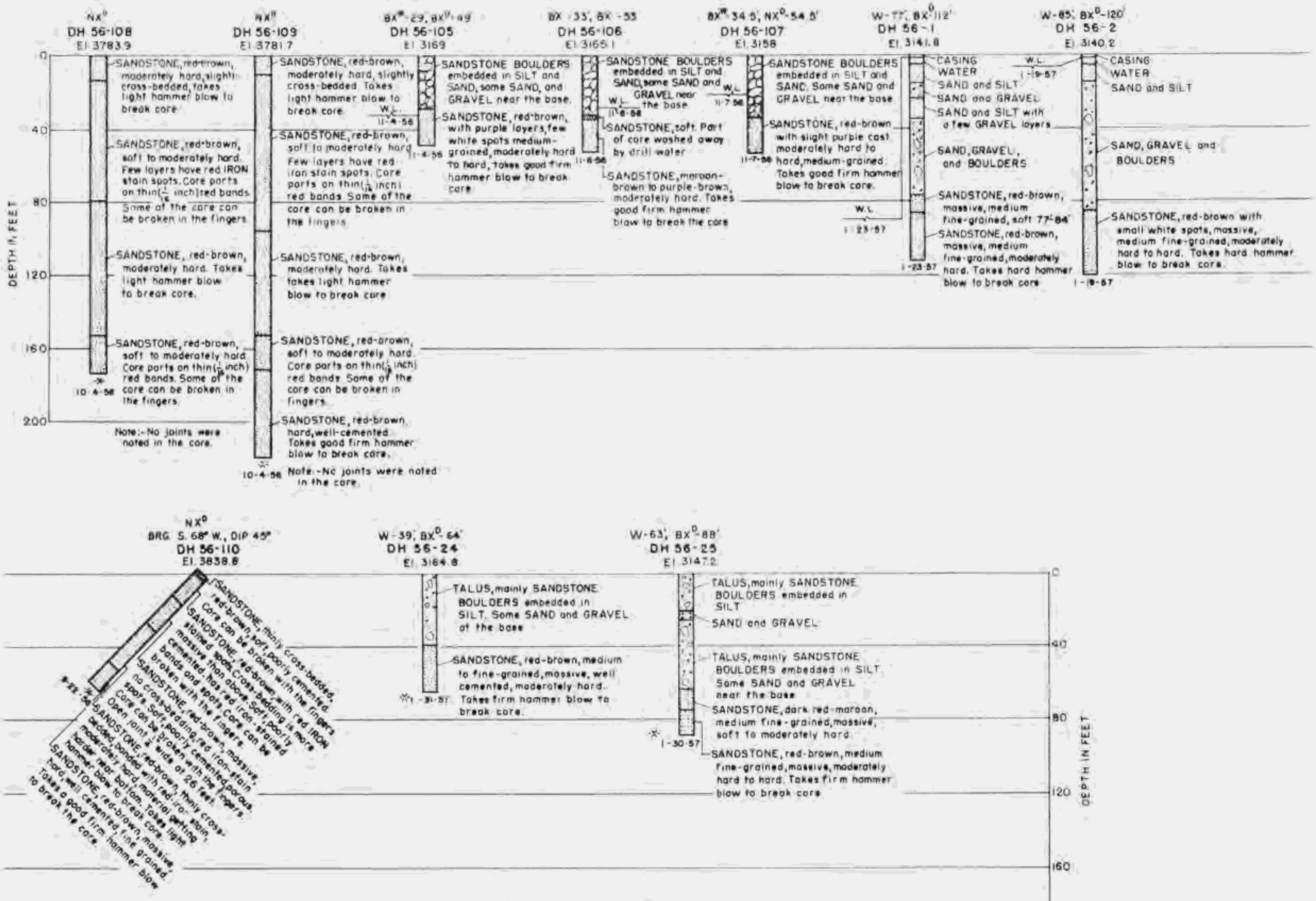


Figure 9.—Logs of exploration of section I-I. From drawing No. 557-D-194.



LOGS OF MISCELLANEOUS HOLES - Continued

Figure 10.—Logs of miscellaneous exploration holes. From drawing No. 557-D-194.

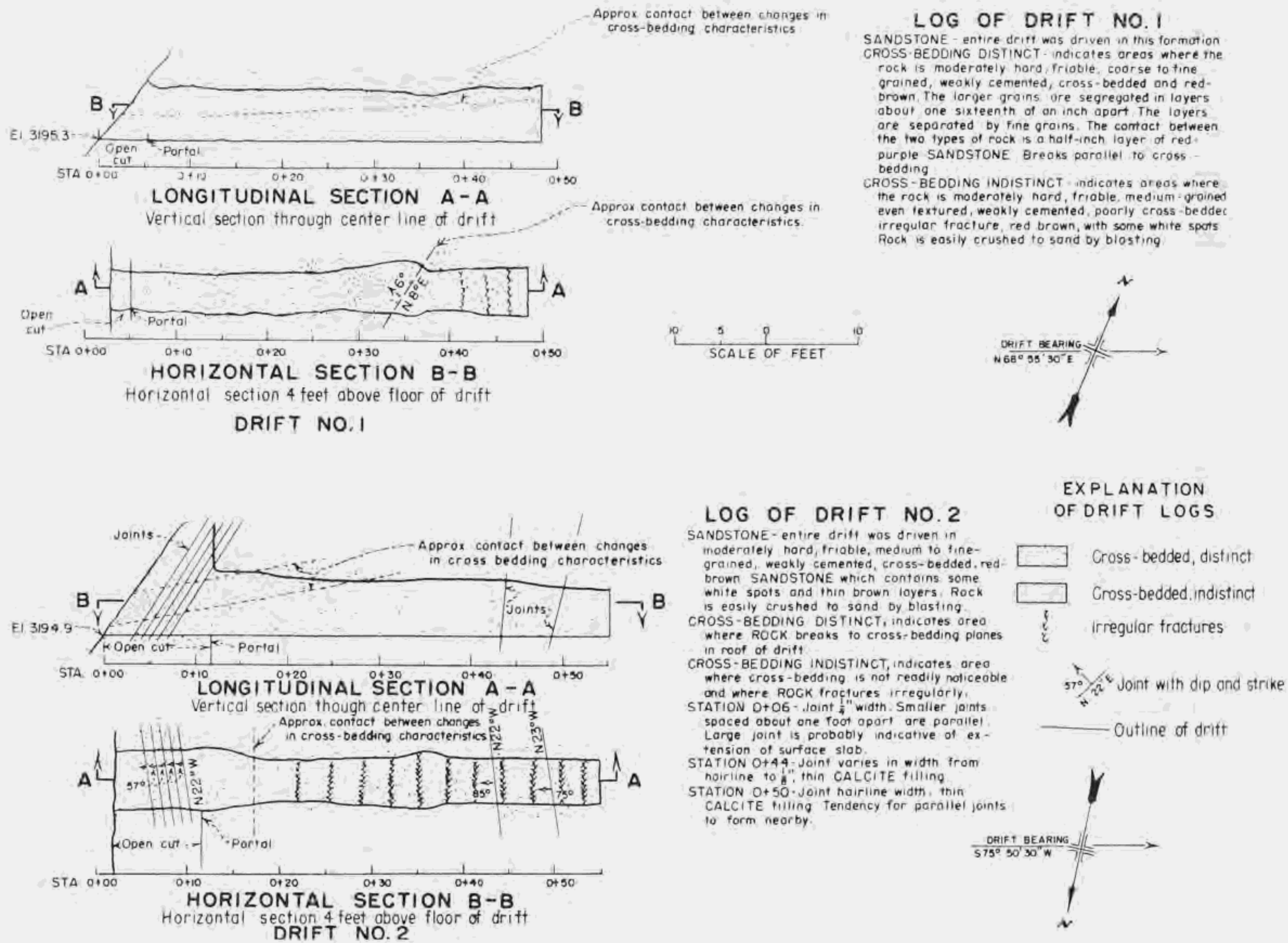


Figure 11.—Logs of drifts No. 1 and 2. From drawing No. 557-D-194.

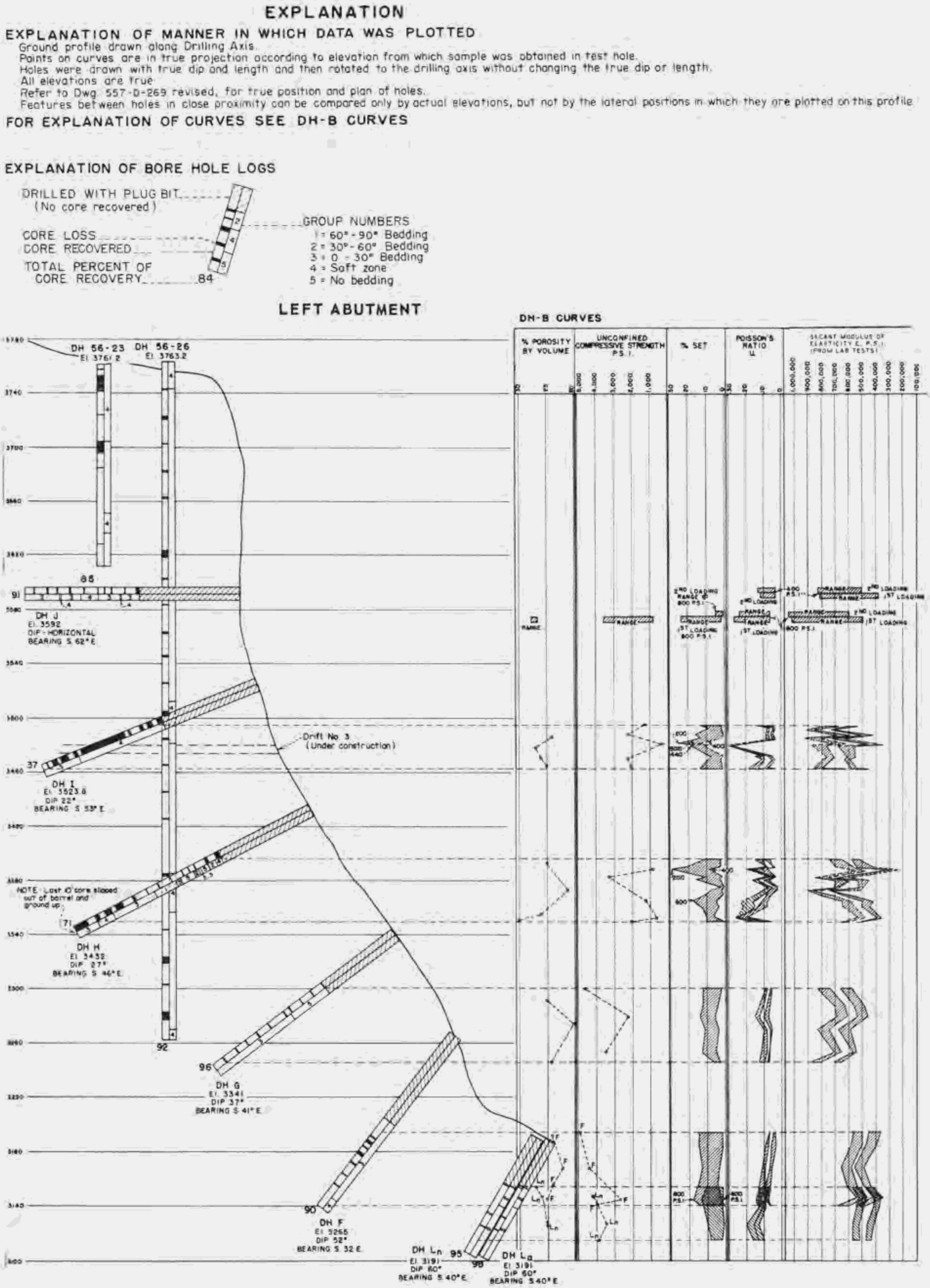


Figure 12.—Graphical portrayal of the physical property tests on foundation cores for the left abutment. From drawing No. Geol. 57-123.

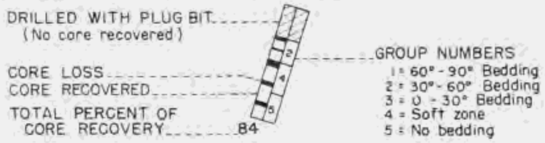
EXPLANATION

EXPLANATION OF MANNER IN WHICH DATA WAS PLOTTED

Ground profile drawn along Drilling Axis.
Points on curves are in true projection according to elevation from which sample was obtained in test hole.
Holes were drawn with true dip and length and then rotated to the drilling axis without changing the true dip or length.
All elevations are true.
Refer to Dwg. 557-D-269 revised, for true position and plan of holes.
Features between holes in close proximity can be compared only by actual elevations, but not by the lateral positions in which they are plotted on this profile.

FOR EXPLANATION OF CURVES SEE DH-B CURVES

EXPLANATION OF BORE HOLE LOGS



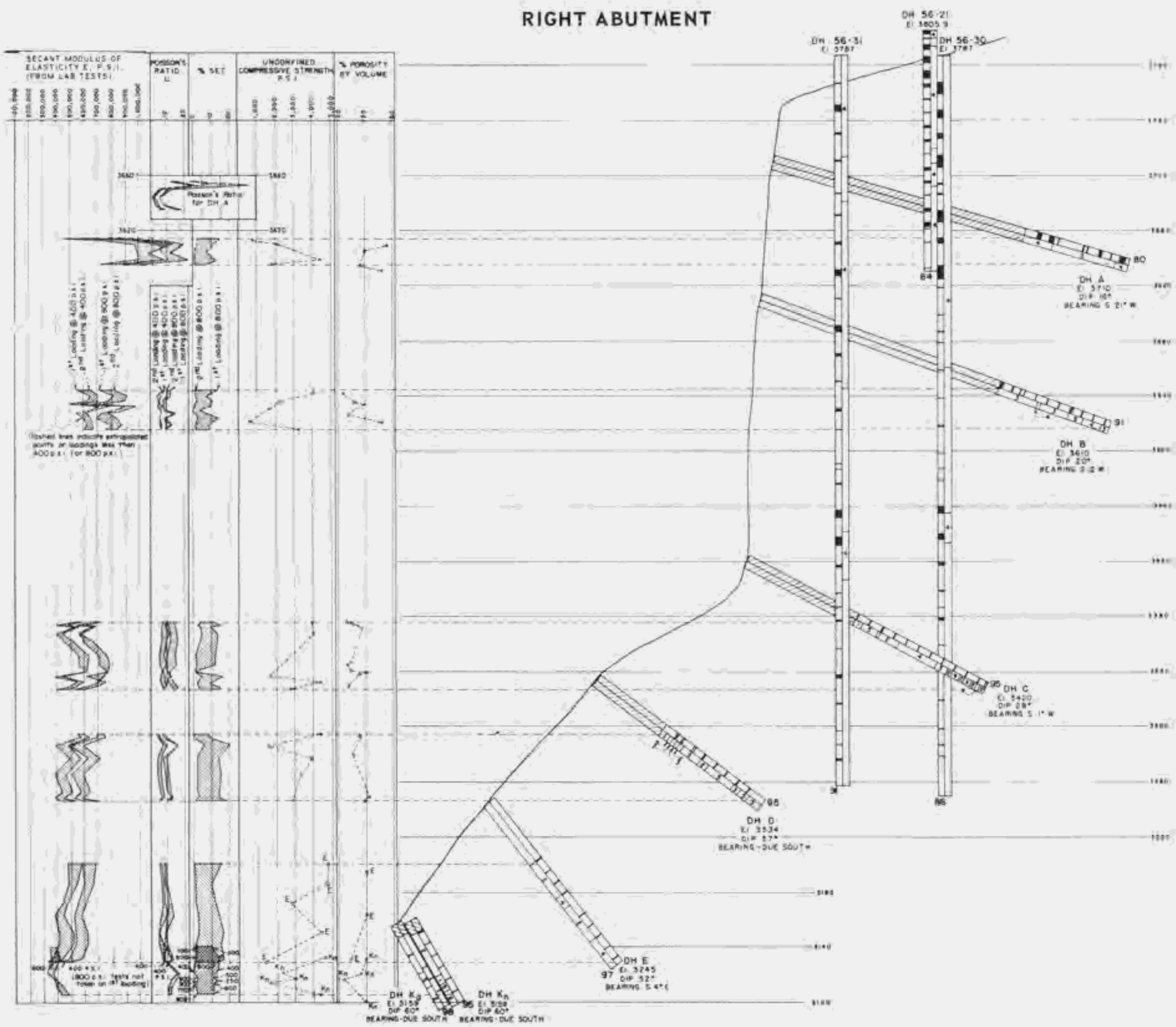


Figure 13.—Graphical portrayal of the physical property tests on foundation cores for the right abutment. From drawing No. Geol. 57-123.

(seismic) tests were also conducted at the damsite to determine modulus of elasticity of the rock mass.

10. CONSTRUCTION GEOLOGY AT DAMSITE. In view of the uniformity of the Navajo sandstone, no wide area geologic mapping of keyways, tunnels, and other excavations was necessary. From time to time, detailed mapping of local areas—principally to show position and spacing of joints—was accomplished to assist in locating structures; such as, transmission tower footings plus length and spacing of rock bolts. Presentation of the data was mostly in the form of partial cross sections and marked photographs.

Exploration during construction consisted principally of core drilling and exploration or cutoff drifts. The drill holes fell into three general categories: (1) Core holes to determine details of bedrock condition beneath structure sites; (2) holes drilled to determine depth of overburden at the toe of the dam in order to stake the overburden excavation cut; and (3) core holes drilled ahead of or near grout holes to determine the location of joints and to serve as vents during grouting.

Thin shale seams were encountered at elevations 3115 and 3065 at the base of the right abutment keyway. The seam at elevation 3115 varied from one-eighth of an inch to about 4 inches thick and had a waterflow of 2 to 3 gallons per minute. A 5- by 7-foot drift following this seam was excavated near the heel of the dam to a depth of 73 feet into the abutment to a point where the flow of water disappeared. The seam at elevation 3065 varied from a thin shale parting in the sandstone to a shale layer 1 to 2 inches thick and had a waterflow of 75 gallons per minute. A 5- by 7-foot drift following this seam was excavated near the heel of the dam to a depth of 215 feet into the abutment. The flow of water decreased with depth and at the end of the drift was just a small trickle. Both drifts were backfilled with concrete and grouted to form a barrier to seepage through the foundation.

Although the Navajo sandstone is remarkably uniform and yields remarkably smooth excavation surfaces, it has two principal characteristics which contributed to design problems. The stress-relief jointing parallel to the canyon walls showed a tendency to open slightly with time and slab or peel off onionskin fashion. The second defect is that the rock has a fairly large percentage of "set" or unrecovered strain occurring during the first loading of the sandstone. Special grouting design was developed to offset this characteristic.

Rock bolts were used extensively as shaped excavations developed; such as, tunnel portals and reentrants in the powerplant foundation adjacent to the base of the canyon walls. When excavation faces cut through sheeted-type, stress-relief joints, the free ends of slabs or sheets tended to separate and warp. The low tensile strength of the sandstone contributed to fallouts, and bolting was used for safety and to maintain the original integrity of the rock. Experience demonstrated that multiple lines of bolts installed adjacent to the edge of a proposed excavation surface prior to blasting was the most satisfactory procedure. Similarly, edges of rock slabs were "sewed" with bolts to prevent progressive warping and deterioration.

The influence of stress-relief in the rock in the canyon walls was perhaps the most important single geologic problem encountered during the construction at Glen Canyon Dam. These joints increased the estimated quantities of excavation, rock bolting, and wall scaling.

The problem of stress-relief jointing was first noticed during the excavation for skewback No. 2 for the Glen Canyon Bridge on the right canyon wall. A large slab had to be removed which extended from below the skewback at elevation 3570 to the canyon rim at elevation 3828. This overexcavation resulted in considerably more concrete under the skewback.

A second local problem area caused by stress-relief jointing occurred at the downstream portal of the left diversion tunnel. A large rock slab fell during excavation due primarily to warping along stress-relief joints. That part of the slab which did not fall was later removed and the sandstone area stabilized with rock bolts.

Stress-relief jointing, along with undercutting by water discharging from the right diversion tunnel, caused a large rock slab, estimated at 18,000 cubic yards, to fall into the outlet portal of this tunnel. This rock mass almost completely choked the outlet portal, and the tunnel had to be unwatered before the rock could be removed and repairs made to the portal area. This operation involved closing of the gates at the upstream portal and construction of a cofferdam around the downstream portal. The problem was corrected by building a concrete wall to support and protect the sandstone. In addition, the area downstream from the outlet portal of the left diversion tunnel was lined with concrete to prevent the occurrence of erosion or undercutting.

Excavations in the powerplant area near the right canyon wall and in the machine shop area near the left canyon wall were troubled with slabbing along stress-relief joints. The bench-type excavation design was modified to more or less fit the joint attitude.

Zones of less well-cemented sandstone encountered in the keyway excavations, although surrounded by normally cemented sandstone, required considerable local resloping to meet existing design concepts. A program of extensive rock bolting and drainage immediately downstream of the dam was accomplished to securely anchor large slabs of rock which were the result of jointing. This work was done for about 200 feet downstream of the toe from elevation 3190 to elevation 3450 on the right abutment and from elevation 3190 to elevation 3350 on the left abutment. The problem of stress-relief joints in the keyway was taken care of by grouting the joints, one lift at a time, as the dam rose in elevation.

Laboratory tests performed on cores of Navajo sandstone showed a high percentage of "set" or unrecovered strain during the initial loading period. This adjustment is thought to be due to a slight rearrangement of the quartz grains in the interstitial cement. With subsequent loadings, the unrecovered strain decreased and the sandstone became more elastic. The range in "set" under the first loading was 5 percent in drill hole J at a load of 800 pounds per square inch to 29 percent in drill hole H at 200 pounds per square inch. The range under the second loadings were zero percent in drill hole D at 800 pounds per square inch to 9 percent in drill hole H at 400 pounds per square inch. Figures 12 and 13 are graphic presentations of the principal variations in engineering properties of the Navajo sandstone as related to the exploration drilling and typical canyon section.

The correction or adjustment for this characteristic of the sandstone was accomplished by prestressing the dam so as to bring the first loading onto the rock before the reservoir exerted a force. This was done by cooling the blocks of the dam concrete down to 40° F. and grouting the contraction joints. As the dam warmed up, the reservoir load was transmitted to the abutment. This load will be maintained and so becomes the minimum load imposed by the dam on the rock. After this first loading, the sandstone becomes essentially an elastic substance which will deform slightly with loading and will recover this deformation when unloaded.

Transformer circuit towers are located on the canyon rim downstream from the highway bridge. The joints in the area were mapped and examined in detail prior to construction. Three test pits were excavated on joints to determine the dip of the joints and study their character underground. There was no evidence to indicate the joints extended very far below the surface. Similar joints intersecting the canyon wall are visible for only 50 to 60 feet down from the canyon rim. Most of the joints dip away from the canyon, and all of the joints exposed in the test pits tightened with depth and were well cemented.

11. GLEN CANYON RESERVOIR GEOLOGY. The main body of Glen Canyon Reservoir (Lake Powell) occupies a long, narrow canyon of the main stem of the Colorado River for a distance—at full reservoir—of about 186 miles. A number of long, slender arms extend out from the main canyon where tributary streams enter the Colorado River. The largest of these is the arm extending approximately 70 miles up the San Juan River.

The canyon as a whole is long and narrow and is characterized by smooth, steep walls and a gentle stream gradient. The average slope for the first 165 miles upstream is 2 feet per mile. The gradient increases rapidly above this point as the reservoir enters the lower end of Cataract Canyon.

Except at the Waterpocket Fold, where Chinle shale is exposed, the confining walls of the main body of the reservoir basin are the massive sandstone cliffs of the Glen Canyon Group made up of the Wingate, Kayenta, and Navajo formations. Throughout its course in the sandstone formations, the canyon is narrow and straight-sided; but, in the weaker shale, its bottom widens and the sides become gently flaring slopes. That part of the reservoir which will extend up the San Juan River will be largely in Triassic beds made up of the Moenkopi, Shinarump, and Chinle formations.

Porosity of the Navajo sandstone ranges between 20 and 25 percent, so the bank storage or waterholding potential is considerable. The reservoir started filling in 1963; and, since that time, a program has been underway to determine the volume of water in bank storage. This program is continuing, and it will be several years before an accurate quantitative figure on bank storage can be determined.

Minor problems have arisen as the reservoir rises regarding rim stability. Sand dunes and talus deposits slide into the reservoir as the materials become saturated. Most of this material goes into the zone of dead storage so has only a very small effect on the total storage capacity. Rockfalls have only occurred on a very small scale and in areas where the sandstone has been undercut.

A slight movement along an existing joint was observed on the right side of the reservoir several miles upstream of the dam. It is assumed that such slight readjustments are occurring elsewhere in the reservoir area as the reservoir level reaches successively higher levels. This movement is a normal reaction and represents readjustment along existing joints resulting from the increasing waterload of Lake Powell. Future movements will be recognized, as in the case of the other large reservoirs, only as slight shocks on seismograph instruments.

CHAPTER III. FOUNDATION TREATMENT

12. GENERAL. The foundation treatment of Glen Canyon Dam consists of three parts: Grouting, drainage, and excavation of cutoff drifts and backfilling them with concrete. The cutoff drifts and the grouting will seal the cracks and joints. The drainage hole pattern will relieve the water that may seep through the massive sandstone.

13. GROUTING. The general plan for grouting the rock foundation beneath the dam provided for low-pressure shallow grouting of the upstream 60 feet of the foundation, to be followed by high-pressure deep curtain grouting from the foundation gallery. Grout holes were also provided for in the foundation tunnels which would complete the deep-curtain grouting in the vicinity of the tunnels and which could extend the grout curtain further into the abutment if desirable at a later date. The low-pressure shallow holes were to seal any near-surface cracks and were designated as B-holes. The main cutoff or grout curtain beneath the dam was to be formed by the high-pressure deep-curtain holes known as A-holes. In addition to the above foundation treatment for the dam, radial grouting was provided for in the spillway tunnels and around the diversion tunnel plugs, and periphery and perimeter grouting was provided for around the backfill and tunnel plug concrete placed in the diversion tunnels. Provisions were also included for grouting major seams, cracks, crevices, and channels, both under the dam and under the powerplant.

Consolidation grouting of the near-surface rock by means of the B-holes was originally required to be completed prior to placing the overlying concrete. Because of the steepness of the abutments and the possibility of lifting the near-surface rock, this method was modified so that B-hole grouting above elevation 3110 was to be performed from the tops of concrete lifts in the abutment blocks. The B-holes were to be spaced at 20-foot centers in both directions, and were to be drilled and grouted to 25-foot depths. The upstream row of B-holes was located just inside the upstream face of the dam.

The A-hole deep-curtain grouting in the lower part of the dam was to be performed generally at 40-foot spacings to a depth of 250 feet, with intermediate holes at 20-foot spacing going to 150-foot depths, and with the final grout holes at 10-foot spacing going to 100-foot depths. This curtain varied uniformly from the above pattern to approximate 110-, 65-, and 45-foot depths for the 40-, 20-, and 10-foot spacings, respectively, at about elevation 3450. Above elevation 3480, the curtain decreased in depth to a 50-foot depth at the top of the dam, all holes to be at 10-foot spacings and drilled from the foundation tunnels.

Because of vertical and near-vertical joints encountered in the upper parts of the abutments, adits paralleling the abutments were constructed in the dam between elevations 3157.5 and 3630. These adits were constructed with a twofold purpose. The first was to provide a means of grouting the concrete-rock contact at any time in the future as the need required. The second purpose was to provide the means for extending the B-hole consolidation grouting so as to cover practically all of the abutment area in the steeper part of the abutments. Grout holes from these adits were to be drilled and grouted at 40-foot spacings along the profile of the abutment and at 80-foot spacings across the width of the abutment. All holes were to be 50 feet into rock.

One near vertical joint, called the A-joint, was observed in the right abutment above elevation 3120 and required special grouting. This was done through nearly horizontal holes drilled 30 feet beyond the A-joint and spaced at 20-foot centers horizontally. Pipes were calked into these holes and brought above the top of the next lift of concrete to be placed. After placement of the lift of concrete, similar holes 7 and 1/2 feet above the lower holes were drilled beyond the A-joint. The lower holes were then grouted using the upper holes as vents. After use as vents, the upper holes were washed and then used as grout holes for the next 7-1/2-foot lift. This procedure was repeated throughout the A-joint area. The A- and B-hole grouting patterns and layouts are shown on figures 14 and 15. The grouting performed from the special adits is shown on figure 16.

Two water-bearing seams were discovered in the foundation on the right abutment at elevations 3118 and 3070 and one on the left abutment at elevation 3080. The one on the left was treated by extending the B-hole grouting downstream in this area an additional 60 feet. The two on the right were treated by excavating drifts into the abutment at the upstream side of the foundation area and backfilling with concrete to increase the path of percolation (fig. 17). The lower drift was extended 160 feet and the upper drift 60 feet into the abutment. The extent of the drifts was determined by field inspection of the leakage in the water-bearing seams.

A joint opened up between the rock-contact line at the right abutment, block 26. This break in contact between the concrete and the rock extended about 30 feet down the upstream and downstream faces of the dam. Since the upper grout lift in the dam is ungrouted, no arch action was transmitted to the abutment. It was necessary to grout this joint to

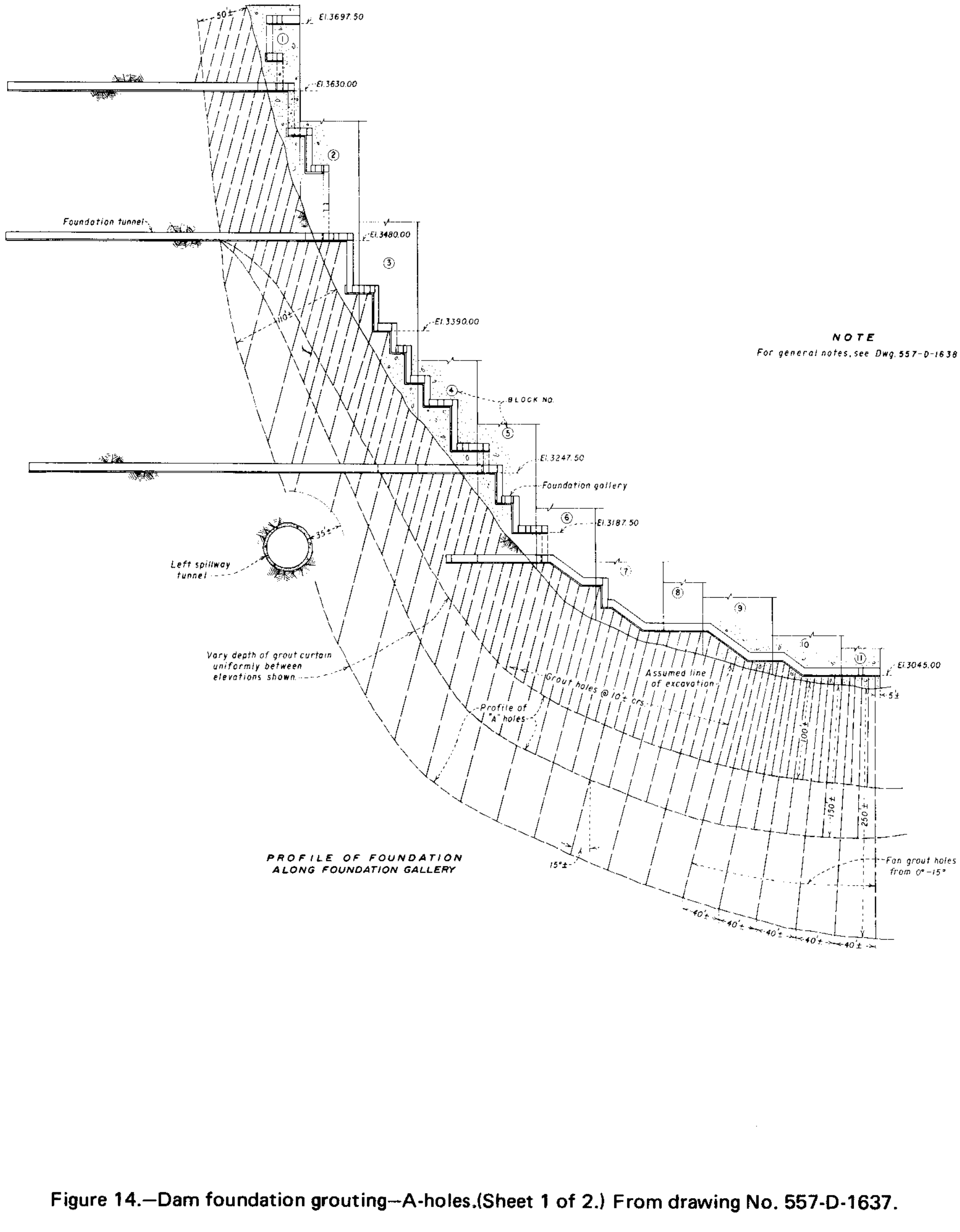


Figure 14.—Dam foundation grouting—A-holes.(Sheet 1 of 2.) From drawing No. 557-D-1637.

prevent water from leaking by the abutment. Details of contact grouting are shown on figure 18.

14. DRAINAGE. Drainage of the foundation rock was provided for through 3-inch-diameter holes drilled into the rock foundation from the foundation and drainage galleries in the dam and from the foundation tunnels in the abutments. To prevent plugging the drainage holes during grouting operations, the drilling of drainage holes was delayed until all grouting within 250 feet was completed.

The main purpose of the drainage holes drilled from the drainage gallery was to reduce uplift and percolation of water near the downstream toe of the dam and in the powerplant area. In the drainage

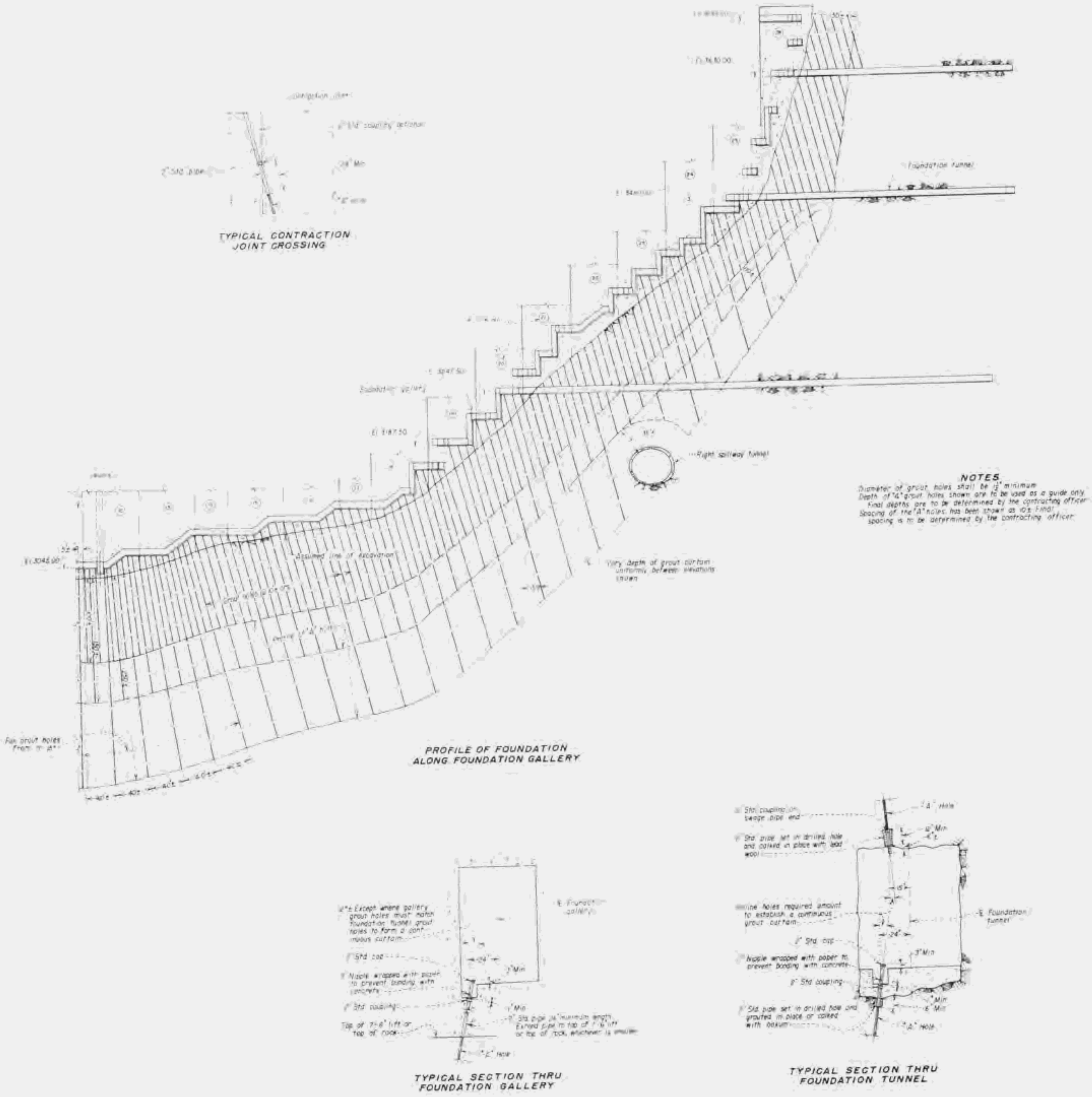


Figure 14.—Dam foundation grouting—A-holes. (Sheet 2 of 2.) From drawing No. 557-D-1638.

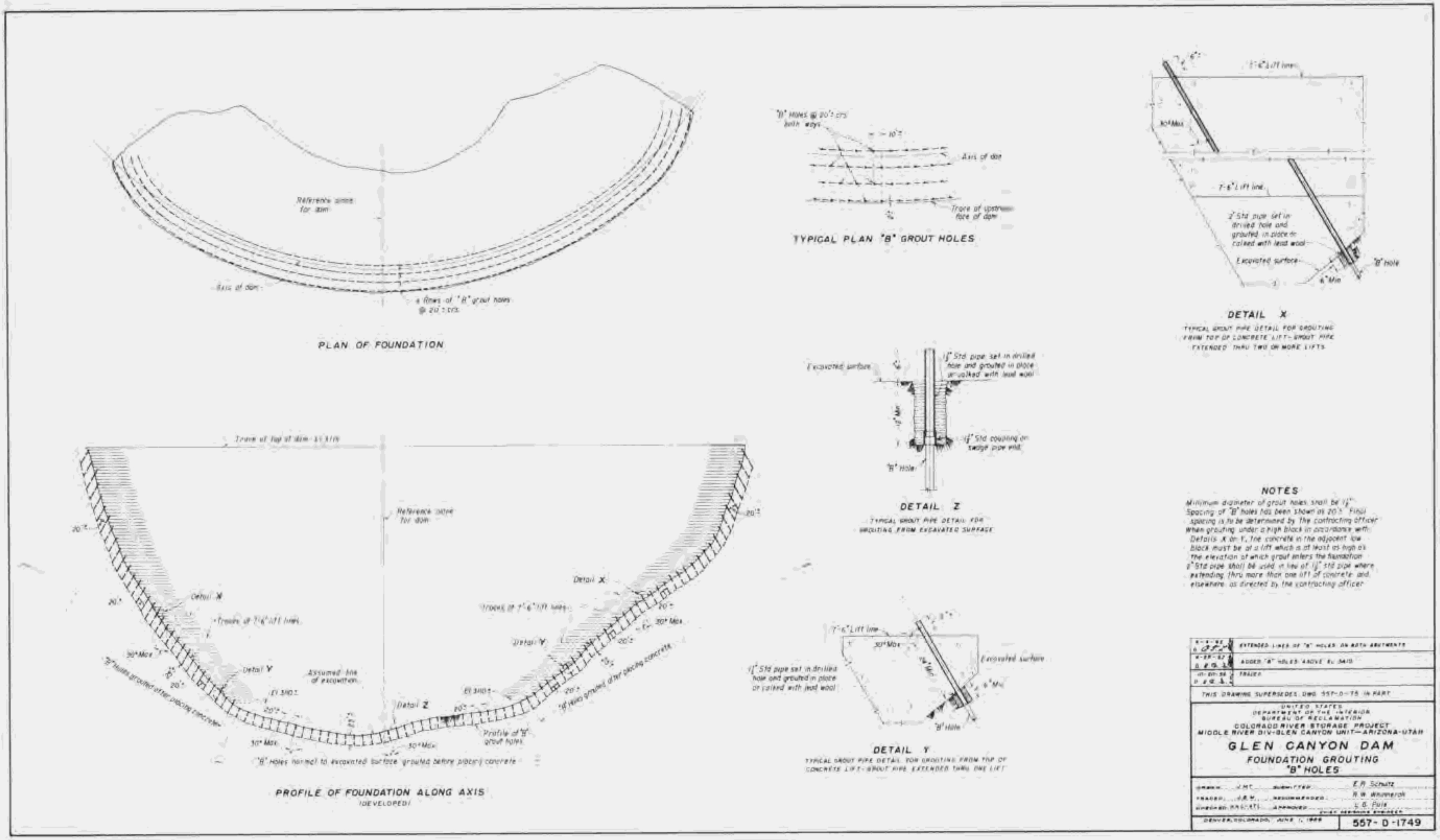


Figure 15.—Dam foundation grouting—B-holes.

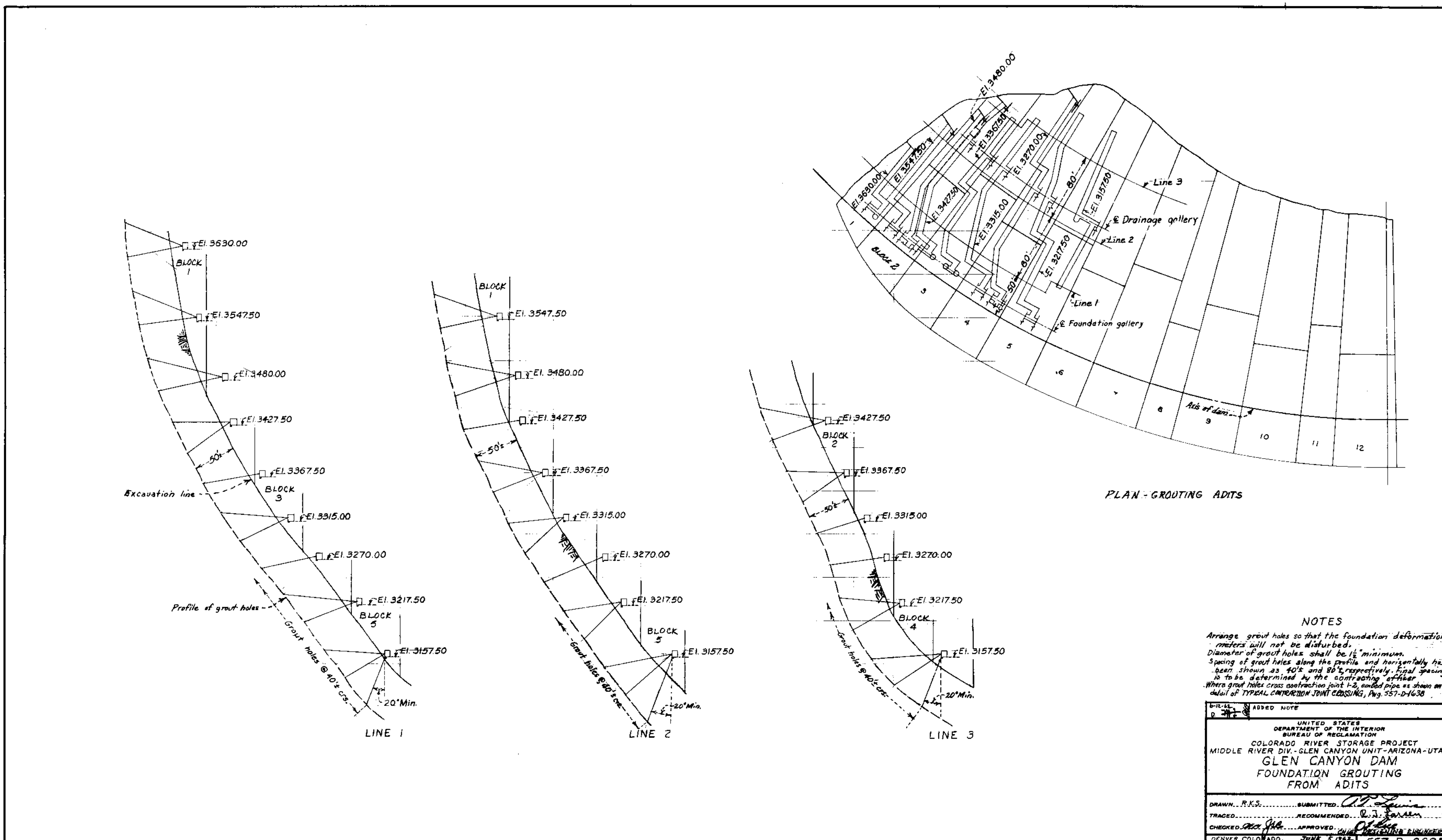


Figure 16.—Dam foundation grouting from adits. (Sheet 1 of 2.)

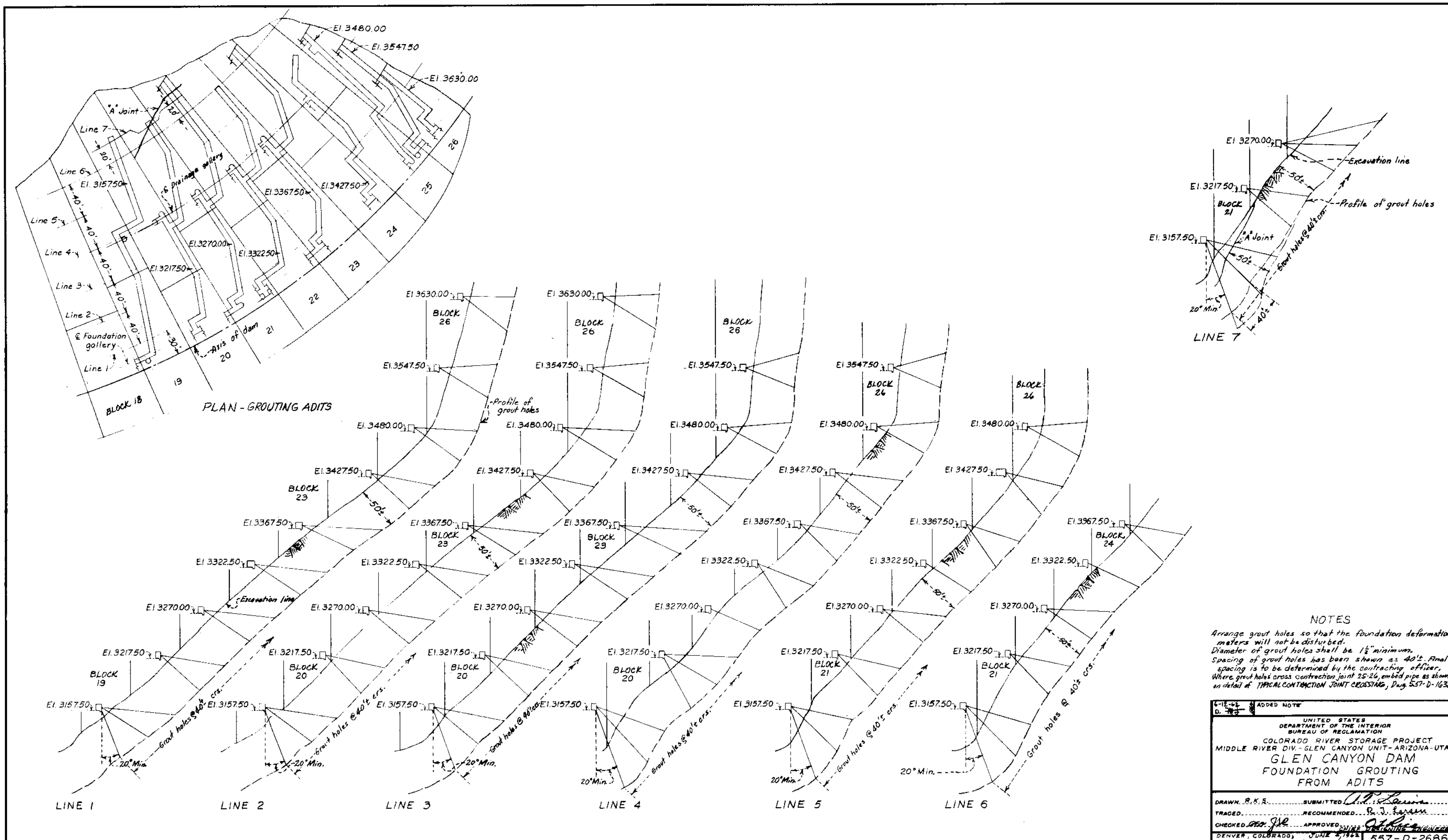


Figure 16.—Dam foundation grouting from adits. (Sheet 2 of 2.)

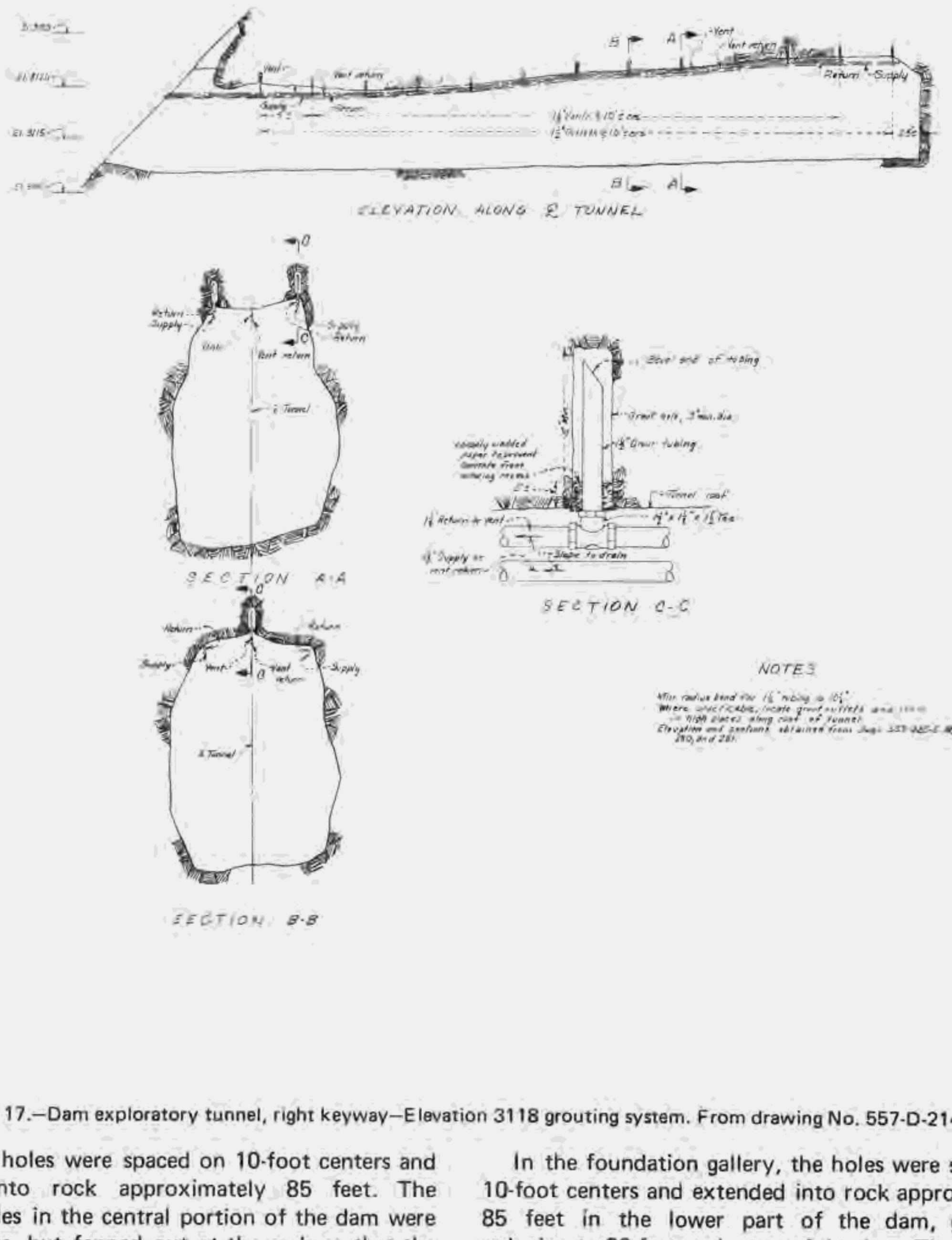


Figure 17.—Dam exploratory tunnel, right keyway—Elevation 3118 grouting system. From drawing No. 557-D-2140.

gallery, the holes were spaced on 10-foot centers and extended into rock approximately 85 feet. The drainage holes in the central portion of the dam were vertical holes, but fanned out at the ends so that the uppermost holes were free draining. The uppermost holes from the drainage gallery were at about elevation 3150. The layout of these holes is shown on figure 19.

In the foundation gallery, the holes were spaced at 10-foot centers and extended into rock approximately 85 feet in the lower part of the dam, gradually reducing to 20 feet at the top of the dam. The drainage holes in the central portion of the dam were vertical holes, but the pattern fanned out so that all drainage holes in the abutments were self-draining. Drainage

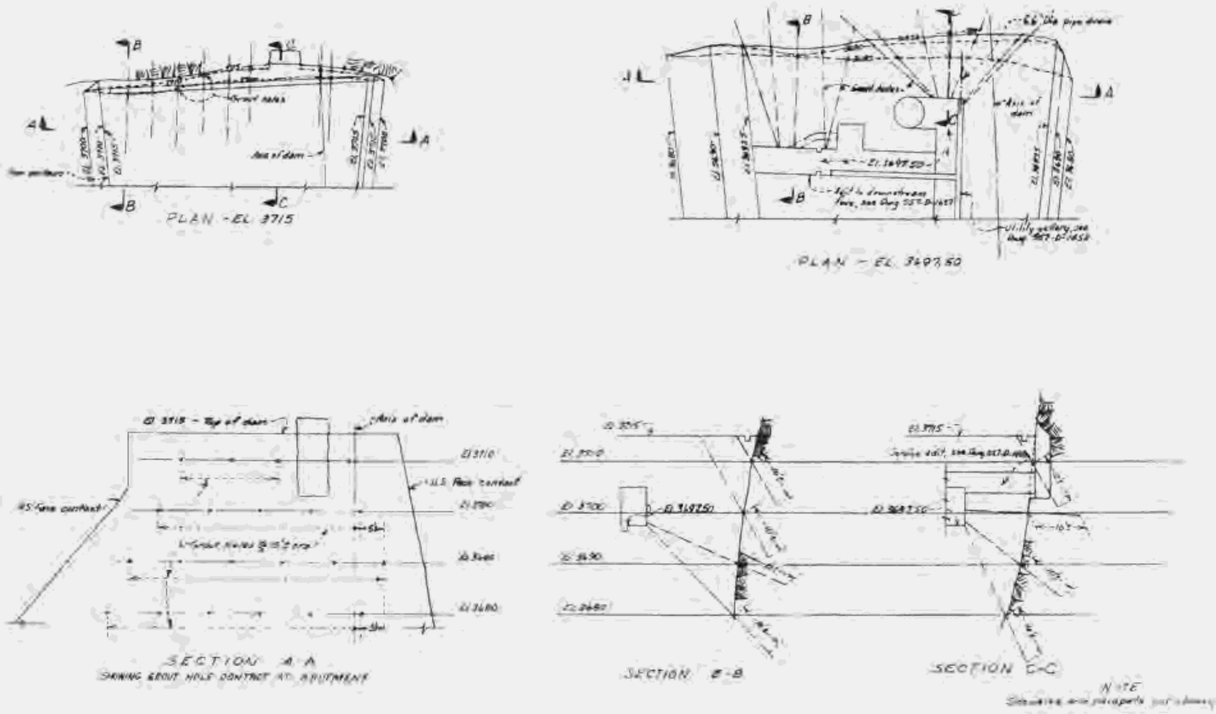


Figure 18.—Dam right abutment—Foundation contact grouting above elevation 3680. From drawing No. 557-D-2973.

holes drilled from the foundation tunnels at 10-foot centers supplemented the holes drilled from the foundation gallery in the drainage curtain. Other drainage holes from the tunnels extended the curtain in the abutment areas, and were to be drilled from both the floor and ceiling of the tunnels, the "down" holes being limited to about 75 feet. These holes were to be spaced at 30-foot centers, although intermediate drainage pipes were to be embedded in the floors of the tunnels at 10-foot centers in case additional holes were needed. Drainage holes from the foundation gallery and foundation tunnels are shown on figures 19 and 20.

The design of the drainage system for the dam foundation was based partially on the results of electric-tray analogy studies.[1]

15. CANYON WALLS. In its report to the Bureau of Reclamation dated October 12, 1961,[2] the Glen Canyon Dam Board of Consultants made the following observations and recommendations:

"In view of the stress-relief joints that have developed, or may develop, in the abutment rock downstream of the dam, particularly in the right abutment, and the increase in the rock stresses that will result from the arch thrust, the Board is of the opinion that extensive anchorage and drainage should be provided to insure the stability of these rock masses. The more critical areas of the abutment appear to be those below about elevation 3450 in the right abutment and somewhat below elevation 3350 in the left where the intrados of the arch at the abutments lies close to the surface and where the relief jointing is most noticeable. Of special importance is the rock mass lying riverward of relief joint "A" where, in the right abutment in the lower part of the keyway, the intrados is riverward of this joint. (Quote continued.)

[1] Interoffice Bureau of Reclamation memorandum of November 15, 1957, subject "Electrical Analogy Tray Study of Abutment Drainage, Glen Canyon Dam" (unpublished).

[2] "Glen Canyon Dam Design and Construction Problems," R. E. Davis, J. W. Vanderwilt, J. J. Hammond, E. B. Burwell, Jr., Julian Hinds, October 12, 1961.

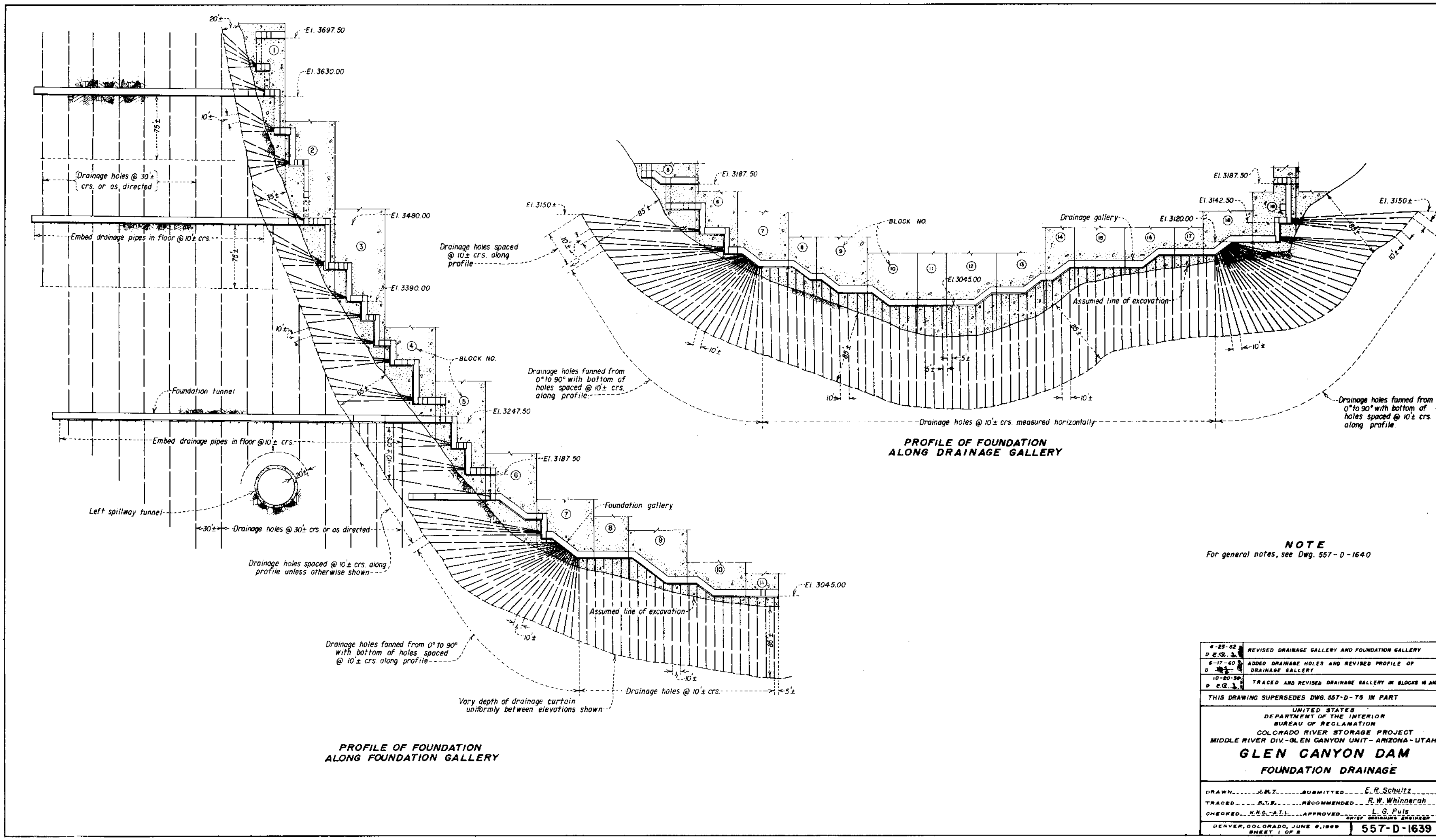


Figure 19.—Left abutment foundation drainage.

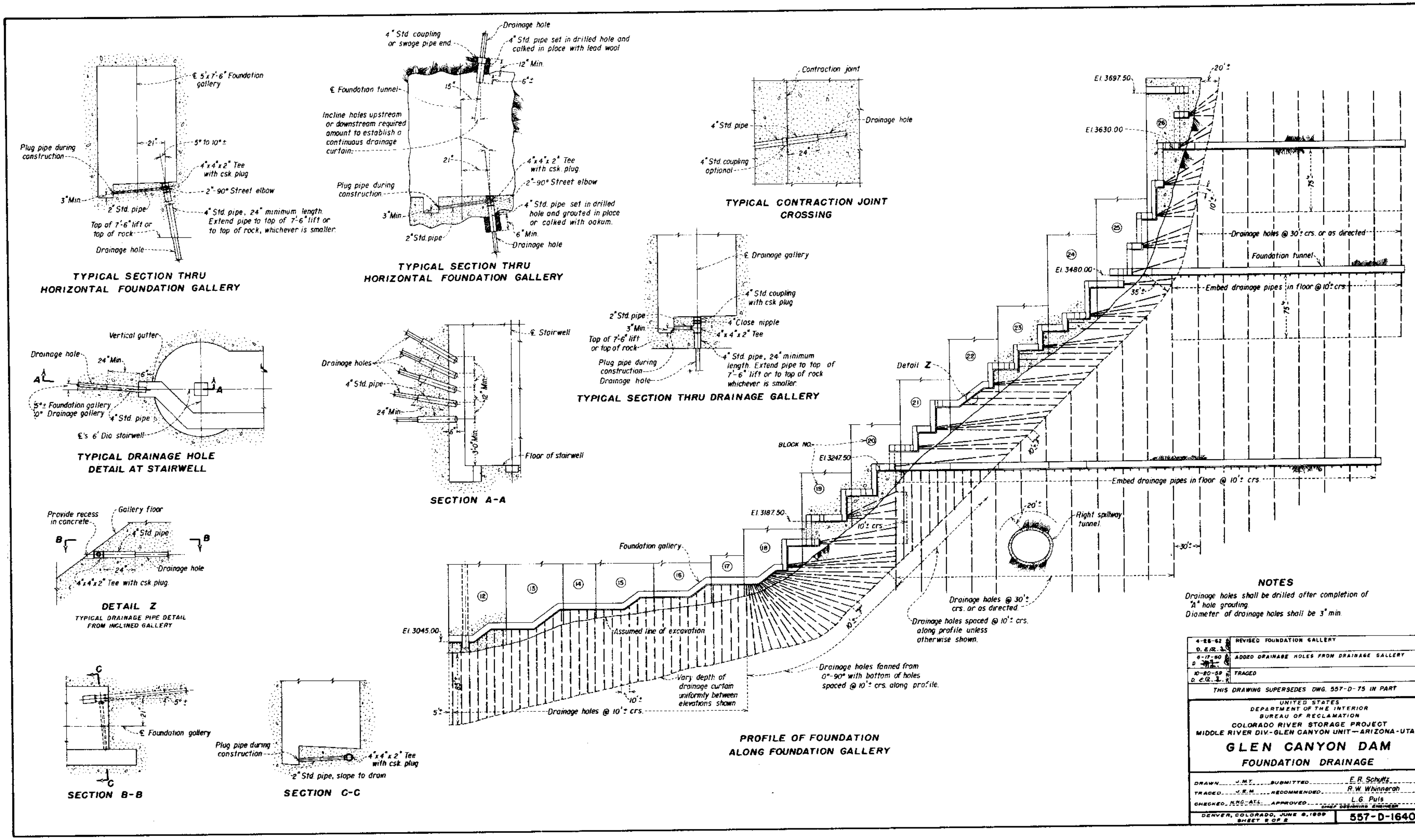


Figure 20.—Right abutment foundation drainage system.

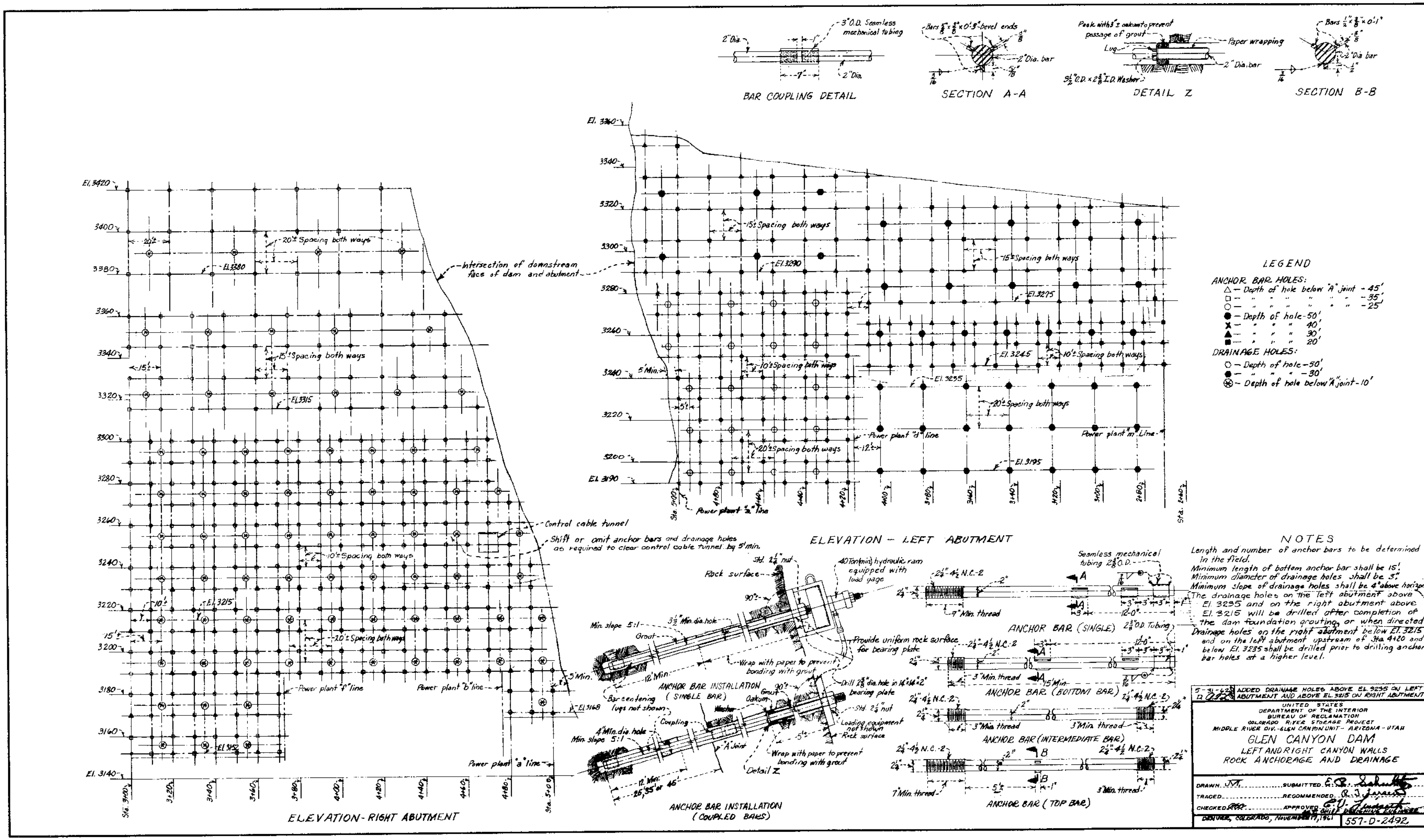


Figure 21.—Left and right canyon walls—Rock anchorage and drainage holes.

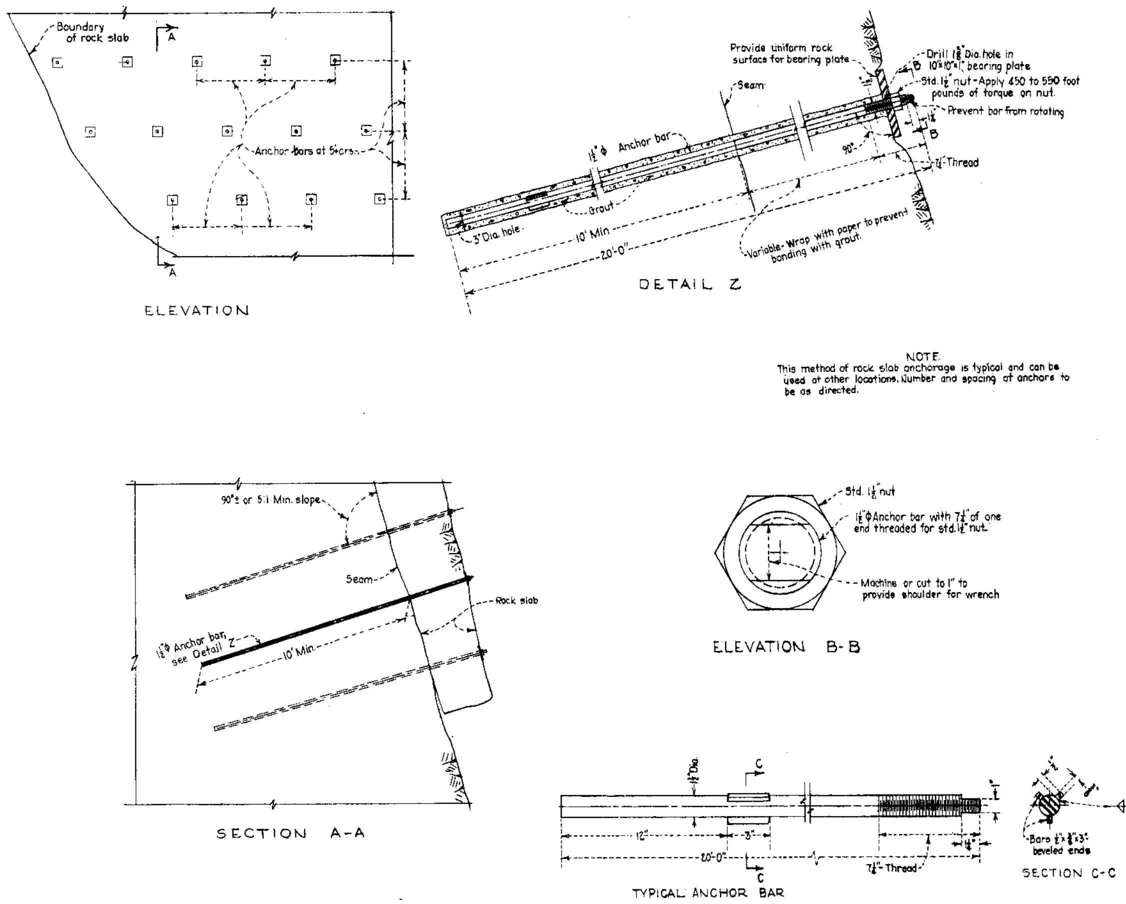


Figure 22.—Left diversion tunnel—Rock slab anchorage downstream of outlet portal. From drawing No. 557-D-1505.

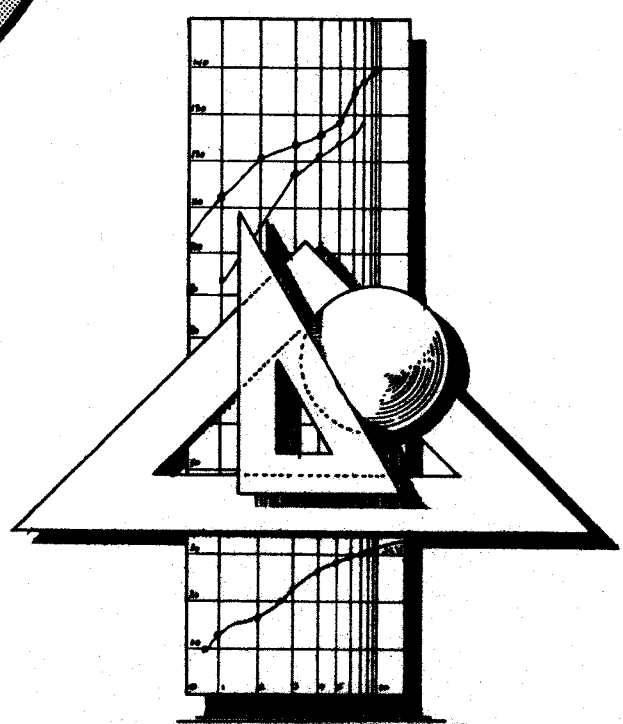
"The Board recommends that the Bureau conduct studies to determine the details of the anchorage treatment between about elevations 3190 and 3450 in the right abutment and about elevations 3190 and 3350 in the left abutment extending downstream for a distance approximately 200 feet from the keyways. The Board is of the opinion that the anchorage in the right abutment should extend back of the "A" joint for a distance of at least 25 feet. It would be desirable to use high-carbon steel bars prestressed to say one-half their yield strength and to grout them in after prestressing. It would be prudent to install the anchorage at the earliest practicable date."

To implement this recommendation, studies were made of the extent, depth, and details of anchorage and drainage necessary to allay concern for stability of the rock masses involved.

The anchorage and drainage of the slabs are shown on figure 21. Two-inch anchor bars in 3-inch-diameter holes were grouted in rock beyond the A-joint. After grouting the lower ends, the bars were tensioned with a hydraulic ram to 20,000 pounds per square inch (63,000 pounds total load). The nuts were then tightened to a torque of 3,500 inch-pounds.

Because the concreting of the powerplant was scheduled to begin, the area between the powerplant d- and m-lines and below elevation 3240 was anchored with 1-1/2-inch torque-tensioned bolts similar to those used in the spillway outlet portal area (see fig. 22). These were on hand and allowed the contractor to begin installation of the bolts immediately.

PART II



DESIGN

PART II-DESIGN

CHAPTER IV. *Design*—DAM

16. SELECTION OF TYPE. Preliminary designs and estimates were prepared for a concrete dam. Also considered was a conventional-type powerplant and an underground-type powerplant. The final unit consists of a constant-radius concrete arch dam with a tunnel-type spillway through each abutment and an outlet works through the dam near the left abutment. The powerplant is located immediately downstream of the dam and an indoor-type structure was finally selected. A layout of the dam and powerplant is shown on figure 4.

As constructed, the crest of the dam is at elevation 3715 and the crest length is l,560 feet. The top width (neglecting added width to accommodate roadway and walkways) is 25 feet. The dam is 710 feet high above the lowest point of the foundation, and the maximum base width is 300 feet. The radius at the axis of the dam is 900 feet. The upstream face is vertical below elevation 3300 and is curved slightly in a downstream direction above that elevation. The downstream slope of the central portion of the dam is constant at 0.31 to 1. A more detailed description of the dam is given in section 30 and the powerplant is described in section 66. The spillway and outlet works are described in sections 40 and 47, respectively.

A. RIVER DIVERSION

17. GENERAL. Diversion of the Colorado River during construction of Glen Canyon Dam was accomplished through two 41-foot-diameter concrete-lined tunnels, one located in each abutment.

The diversion tunnel through the right abutment is 2,749 feet long. Approximately 910 feet of the downstream portion of the diversion tunnel was later incorporated into the right spillway tunnel. The invert elevation at the entrance is 3137.37 which is essentially river level.

The diversion tunnel through the left abutment is 3,011 feet long. Approximately 1,085 feet of the downstream portion of the diversion tunnel was later incorporated into the left spillway tunnel. The left diversion tunnel invert is at 3170.67 which is 33.3 feet higher than the right diversion tunnel. The left tunnel entrance was set at the higher elevation so that a temporary outlet works could be installed in the left tunnel plug during a low flow season without the use of an entrance closure structure. Riverflows less than 15,000 cubic feet per second were confined to the right diversion tunnel.

The temporary outlet works (sec. 18), consisting of three 7- by 10.5-foot outlets controlled by 7- by 10.5-foot slide gates in tandem, were located in the left tunnel plug to control releases during reservoir filling. Access to the gate operating chamber was through a 5- by 7-foot access adit from the dam. A trashrack at tunnel entrance was constructed to protect the outlet works. The left diversion tunnel and outlet works are shown on figure 23, and the access adit from the dam is shown on figure 24.

In the initial design planning, the diversion tunnels were 50 feet in diameter, unlined, and approximately 2,500 feet in length. Tests performed to determine whether the sandstone through which the tunnels were bored could withstand the erosive force of sediment-laden, high-velocity flow indicated that the diversion tunnels should be lined. Accordingly, lined tunnels were specified and the diameter was reduced to 44 feet. Subsequent to the final diversion studies, the diameter of the tunnels was further reduced to 41 feet to match the final size requirement for spillway discharges. (See preceding discussion relative to use of a portion of the diversion tunnel for the spillway tunnel.)

Hydraulic model studies were made to check the alinement and elevation of the diversion tunnels with respect to the river channel. The hydraulic model also was used for preliminary investigations of the deflector buckets at the end of the tunnel spillway. The two discharge quantities used for the diversion studies were 30,000 and 65,000 cubic feet per second per tunnel. Model tests were made with the right tunnel operating singly and also with both tunnels operating. Since the intake portal of the left tunnel is about 33 feet higher than the right tunnel intake, the left tunnel would not operate singly.

The investigations showed that, in general, the tunnel alinement and grade were satisfactory for diversion flows. The curved channel downstream from the right tunnel caused some eddies in that vicinity, but when the curved channel was replaced by a straight channel, the eddies were eliminated and the flow was entirely satisfactory.

18. DIVERSION PLAN. During early construction of the dam, the river was diverted through two 41-foot-diameter concrete-lined diversion tunnels. One tunnel, with the inlet invert at elevation 3137.37, passed around the damsite through the right canyon wall. The other, with the inlet invert at elevation 3170.67, passed through the left wall. After construction of the dam and

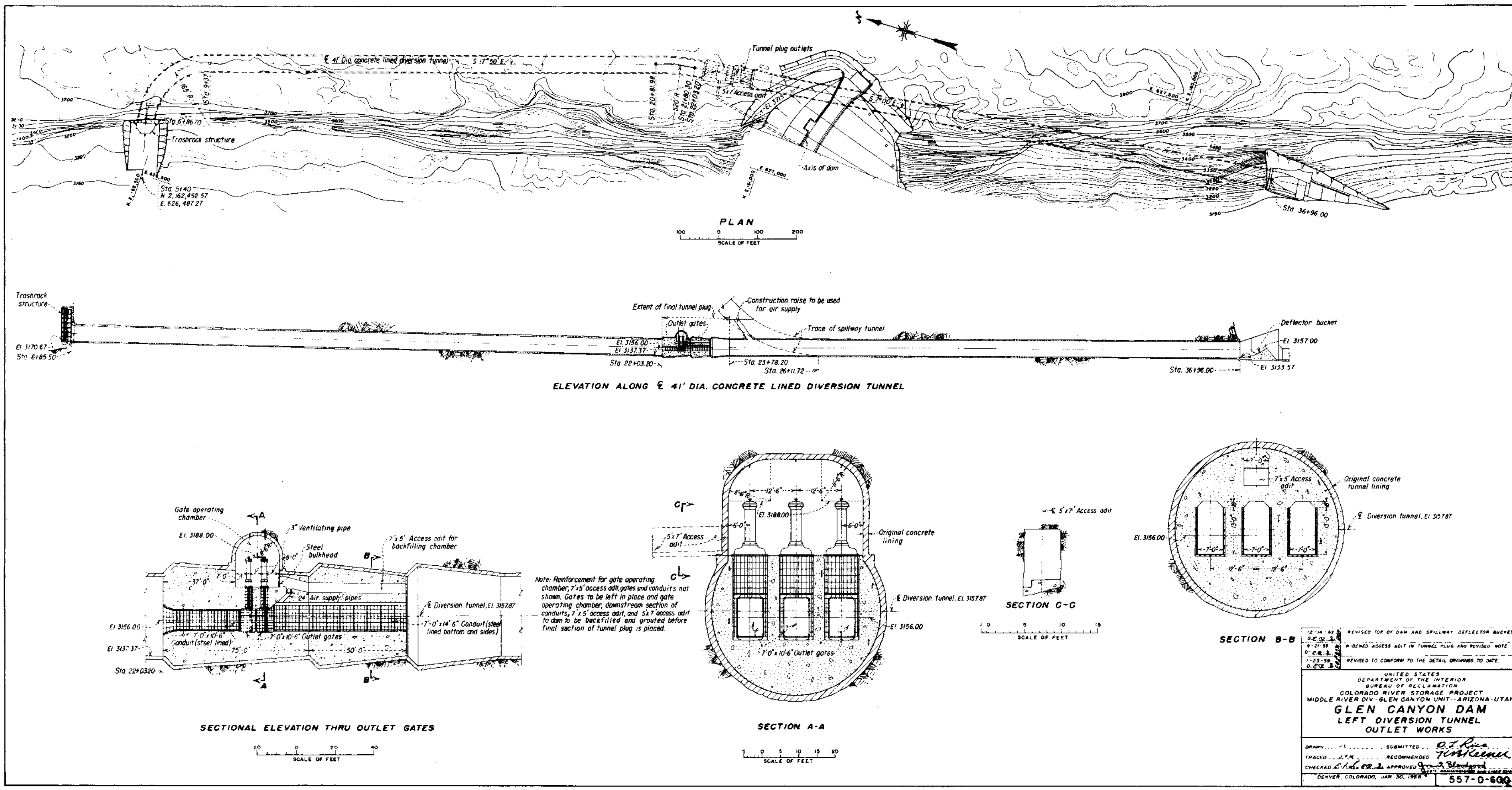


Figure 23.—Left diversion tunnel outlet works.

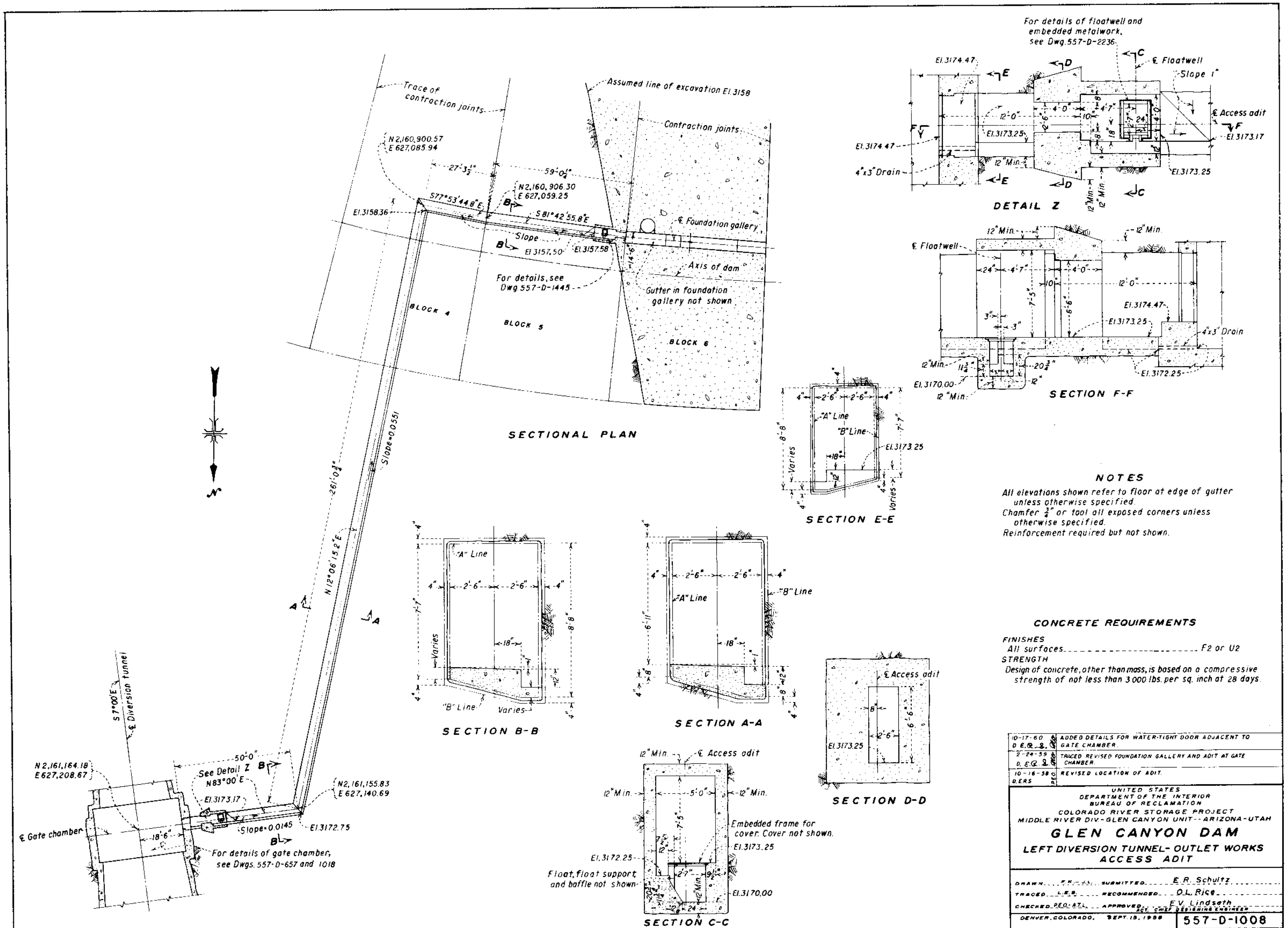


Figure 24.—Left diversion tunnel outlet works—Access adit.

power units had advanced sufficiently, the upstream portions of these diversion tunnels were abandoned and permanent seals, or plugs, were installed to close them (figs. 5 and 25). The downstream portions were then connected to the spillway tunnels that slope down to meet them, thereby completing the reservoir spillway facilities (figs. 5 and 25).

From practical and economic points of view, water storage had to start as soon as the water was available and sufficient construction had been achieved. During such storage, water had to be released through or around the dam to meet downstream commitments. To fulfill these conditions, some form of low-level outlet works had to be provided. Owing to the relative narrowness of the canyon, the appreciable space requirements of the penstocks, and predicted future silt levels, the inlets to the permanent river outlets in the dam had to be placed at elevation 3374 (fig. 5). This meant that a lake about 265 feet deep had to be impounded before the river outlets could release water. They therefore could not be used to make releases during the early reservoir storage period.

To provide the necessary river control for the interim period, an outlet structure was built into the left diversion tunnel plug (fig. 23.) It consisted of three 7-foot-wide by 10.5-foot-high rectangular passages in the plug, followed by the same size high-head slide gates and by 7-foot-wide by 14.5-foot-high conduits that discharged into the downstream tunnel. Bellmouth inlets were provided at the conduit entrances in the upstream face of the plug. An emergency gate and a service gate were provided one behind the other in each conduit. The gates are of the slide type (sec. 22) developed and used successfully at the Bureau's Palisades Dam for a number of years at various gate openings with heads up to 200 feet.

The tunnel plug outlet works was constructed during the low flow period in the fall and winter of 1961-62. During this construction, all of the riverflow was diverted past the damsite through the right diversion tunnel. After the construction was completed, massive steel slide gates were lowered in the right diversion tunnel entrance and a solid concrete plug installed to close it permanently. After this closure, and up to the time the lake had risen to service level, and overall construction had advanced enough, all flows past the dam were released through the higher, left diversion tunnel outlet works.

After the reservoir reached a service level and the permanent river outlet works and turbines could be operated, the tunnel outlet gates were closed and the conduits filled and sealed with concrete. The remaining section of tunnel plug was then installed to complete the plug and permanently seal the upstream part of the tunnel. The final connection to the left spillway tunnel was then made (fig. 25).

19. HYDRAULIC CONSIDERATIONS. (a) *Diversion Tunnels.*—Diversion floods of 5-, 10-, 25-, 50-, and 100-year frequencies are shown on figure 26. The size of the tunnels (41 feet) was determined from routing studies made using the 25-year frequency flood which has a peak flow of 196,000 cubic feet per second and a 15-day volume of 3,550,000 acre-feet. Routing of this flood through the tunnels resulted in a maximum reservoir water surface elevation of 3277 and a combined discharge of 143,000 cubic feet per second. The upstream cofferdam, designed by the contractor, had a top elevation of 3300 and the downstream cofferdam a top elevation of 3165.

(b) *Temporary Outlet Works.*—In order to meet downstream requirements during initial filling of the reservoir, an outlet works of sufficient capacity to pass all riverflows downstream with only temporary storage was necessary. It was determined, from routing studies, that a diversion tunnel outlet works with a capacity of approximately 33,000 cubic feet per second at reservoir elevation 3566 (410 feet of head), in conjunction with the permanent river outlets in the dam (invert elevation 3374), would satisfy all requirements. The outlet works were tested in a hydraulic model. The conclusions are presented below and further outlined in Hydraulic Laboratory Report No. Hyd-468.[1] Discharge curves for the outlet works are shown on figure 27.

20. CONCLUSIONS FROM MODEL STUDIES.

(1) A satisfactory diversion tunnel outlet works was obtained with three conduits spaced 12 feet 6 inches apart and provided with regulating slide gates 7 feet wide by 10.5 feet high.

(2) Rectangular bellmouth conduit entrances with elliptically curved surfaces provided good boundary surface pressures under all operating conditions.

[1] "Air and Hydraulic Model Studies of the Left Diversion Tunnel Outlet Works for Glen Canyon Dam," Hydraulic Laboratory Report No. Hyd-468, Bureau of Reclamation, September 28, 1960 (unpublished).

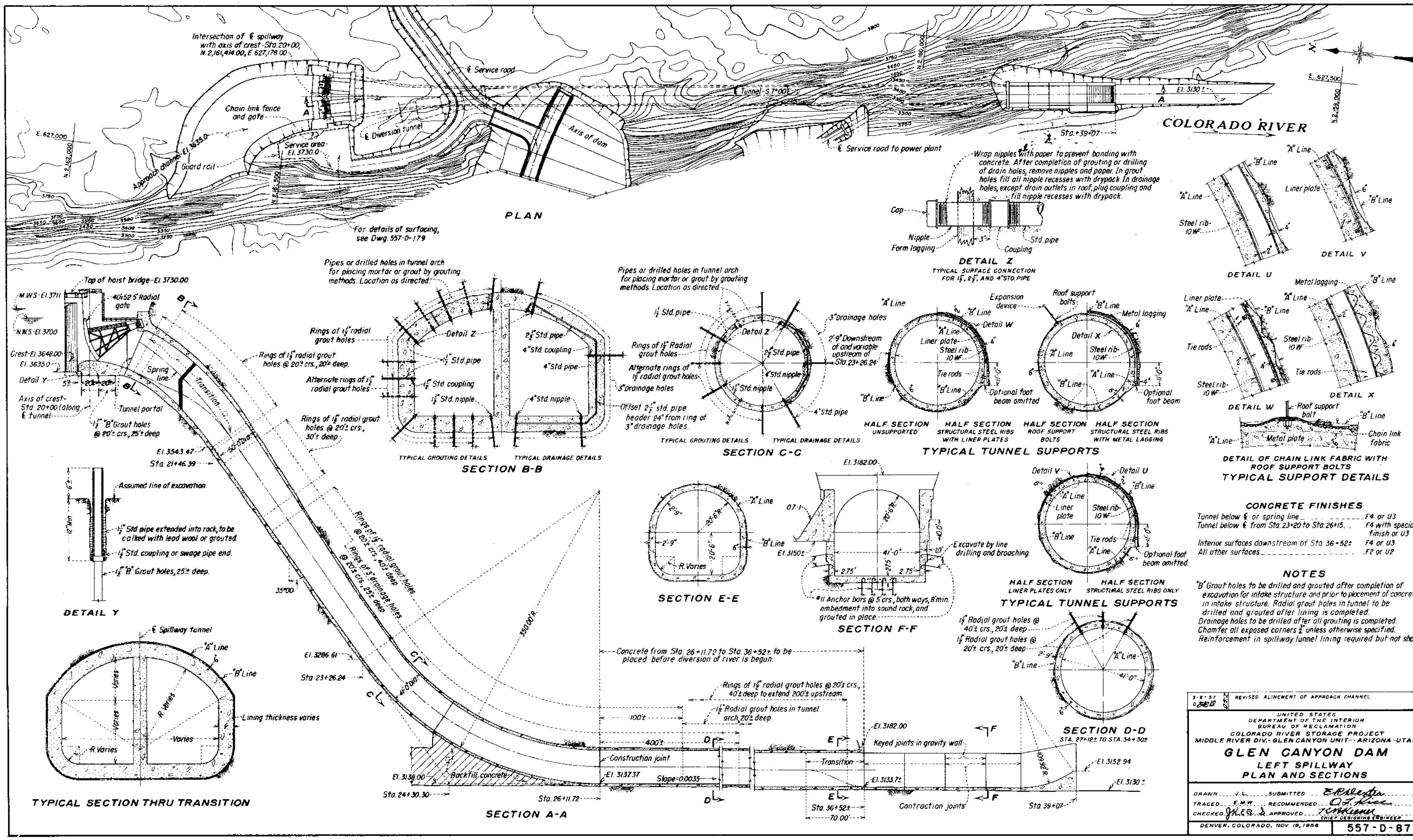
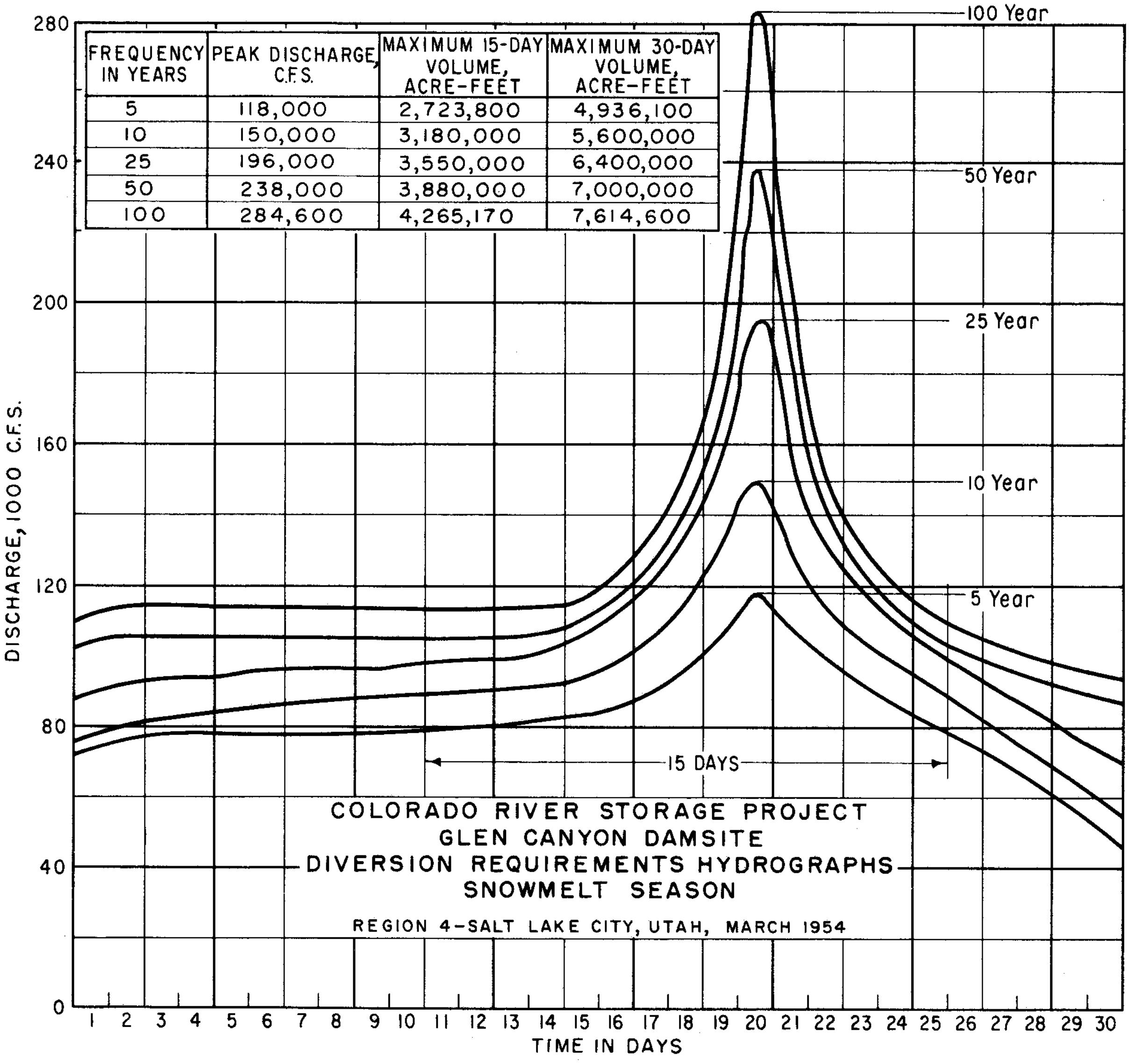


Figure 25.—Left spillway—Plan and sections.



FREQUENCY IN YEARS	PEAK DISCHARGE, C.F.S.	MAXIMUM 15-DAY VOLUME, ACRE-FEET	MAXIMUM 30-DAY VOLUME, ACRE-FEET
5	118,000	2,723,800	4,936,100
10	150,000	3,180,000	5,600,000
25	196,000	3,550,000	6,400,000
50	238,000	3,880,000	7,000,000
100	284,600	4,265,170	7,614,600

Figure 26.—Damsite diversion requirements hydrographs—Snowmelt season. From drawing No. 557-D-3220.

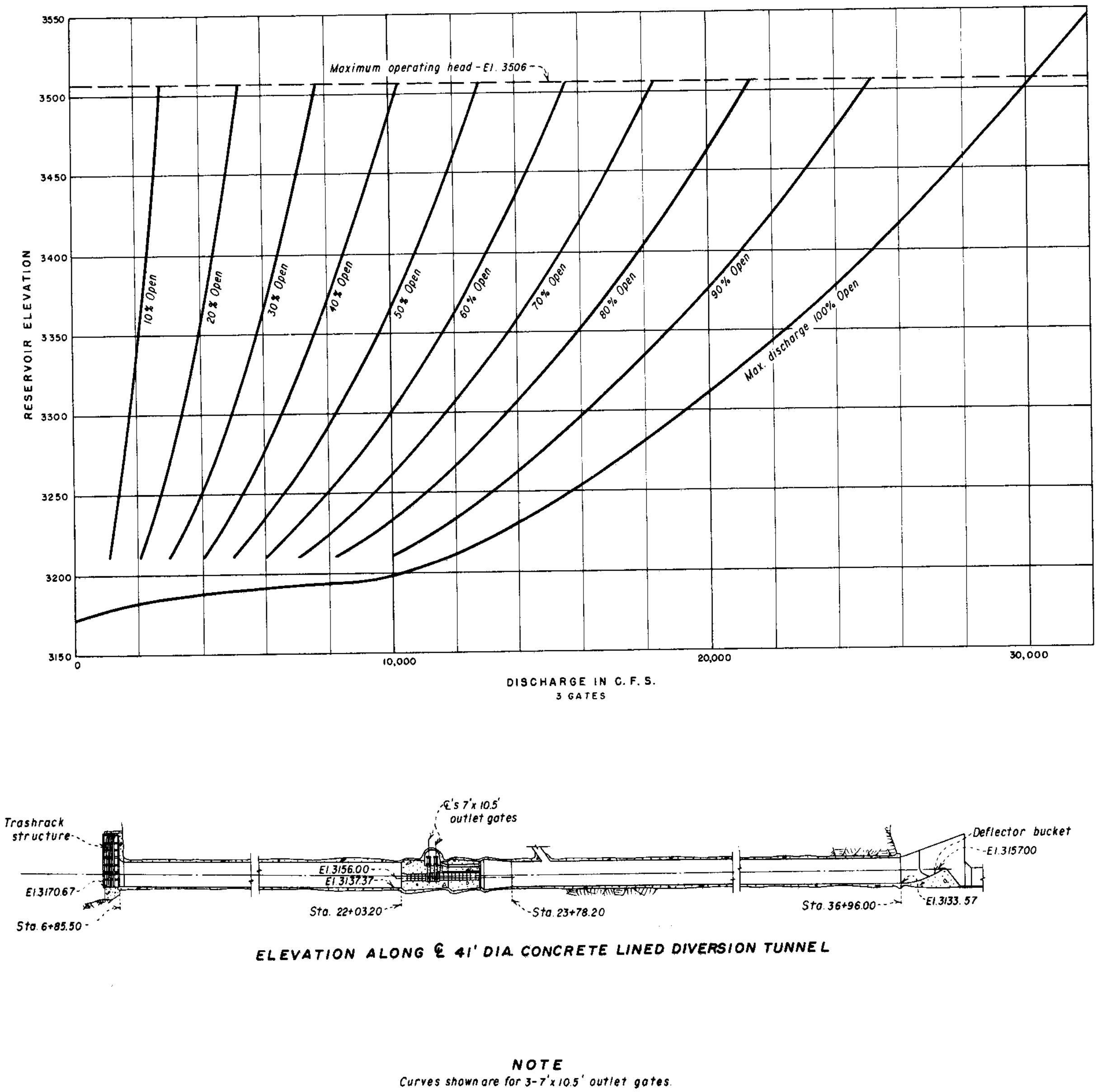


Figure 27.—Left diversion tunnel—Outlet works discharge curves. From drawing No. 557-D-2771.

(3) Steel liners were desirable in the bellmouths and in the conduits leading to the gates to insure smooth, continuous flow boundaries free from surface irregularities that could cause local cavitation.

(4) Slide gates of the type developed for the Bureau's Palisades Dam outlet works provided excellent, trouble-free regulation of flow through the outlet conduits. A guard gate and a service gate of this same basic design were placed one behind the other in each conduit.

(5) Twenty-four-inch-diameter ducts connected to a 7-foot-wide by 5-foot-high passage

leading to the downstream face of the plug, and opening into the 41-foot-diameter tunnel supplied adequate air to the top of the conduits just downstream from the control gates.

(6) The conduits downstream from the gates were parallel, horizontal, 7 feet wide, 14.5 feet high, and free from surface irregularities that could produce cavitation. Steel lining extending across the floor and 13 feet up the walls was desirable.

(7) A deflector 6 feet long and 6 inches high, on the floor at the downstream end of the center conduit, directed that outlet's flow on a longer trajectory to produce better flow conditions in the downstream tunnel. (Note: To simplify fabrication of the conduits, this was not included in the prototype construction of the outlets.)

(8) Better flow conditions occurred in the tunnel when the keyway or conic tunnel plug section was replaced by a straight 41-foot-diameter tunnel. This alternative was costly and not justified by the moderate improvement in performance. The keyway section, which was needed for strength in the final tunnel closure, remained as originally proposed.

(9) Unsymmetrical flow releases from the three outlets resulted in side-to-side swinging flow in the circular tunnel. This swinging persisted to the outlet portal. No difficulty occurred in the tunnel due to this action.

(10) Swinging flow affected the direction in which water left the flip bucket at the tunnel portal, and under some operating conditions, water struck the lower portions of the left canyon wall.

(11) Deflecting the left wall of the deflector bucket to the right tended to prevent water from striking the canyon wall. A 20.5-foot deflection worked well for all symmetrical and unsymmetrical outlet flows, but was too severe for the large spillway flows. A sloped left wall of less deflection kept almost all the water off the canyon wall, but was unsatisfactory with spillway flows. A 12.5-foot deflection with a vertical wall allowed an appreciable portion of the water to strike the lower canyon wall during certain outlet flows, but is ideal for spillway flows.

(12) Constructing the deflector bucket in two stages, with only part of the curved invert present during outlet releases, did not significantly decrease the amount or intensity of water impingement on the canyon wall with either symmetrical or unsymmetrical flows approaching the bucket.

(13) The full bucket with the left wall vertical and deflected 12.5 feet at the downstream end was used for both outlet works and spillway releases. Water will impinge on the lower canyon wall during small outlet releases. In the model the impingement was greatest with certain unsymmetrical flows, and such operation should be avoided wherever possible.

21. STRUCTURAL DESIGN OF DIVERSION TUNNELS. (a) *Linings.*—The tunnels were lined with a minimum thickness of 15 inches of concrete. The purpose of lining was twofold:

(1) To prevent destructive erosion of the sandstone; and

(2) To obtain the same capacity with the 41-foot-diameter tunnel as that of a larger unlined tunnel.

The lining of the right tunnel was not reinforced except at the bottom of the construction raise. The lining of the left tunnel was reinforced at the following locations:

(1) The first 20 feet immediately downstream from the trashrack structure for possible water pressures during operation of tunnel plug outlet works.

(2) The high-pressure gate chamber lining; this lining was both rock-bolted and reinforced for grout pressures and gate hanger loads.

(3) The tunnel invert between station 23+68 and station 26+11.72 because of impact and turbulence of discharge from high-pressure gates during operation of tunnel-plug outlet works.

(4) At the bottom of the construction raise.

The lining for the right diversion tunnel is shown on figure 28. The lining for the left diversion tunnel was similar to that shown for the right diversion tunnel.

(b) *Trashrack Structure.*—The left diversion tunnel trashrack structure was designed for a differential water load of 40 feet and temperature effects. The roof had a 2-foot 6-inch layer of gravel to

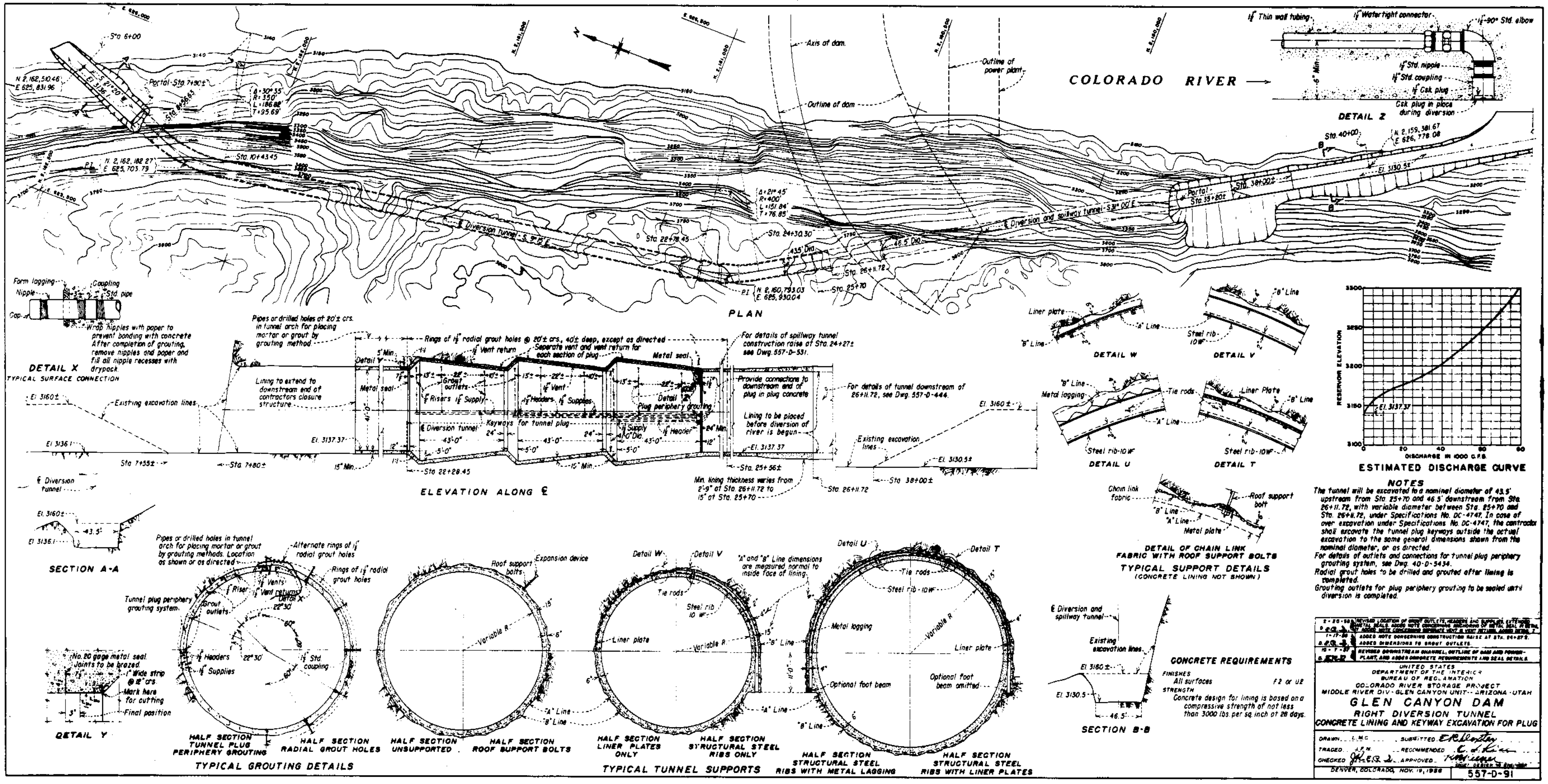


Figure 28.—Right diversion tunnel—Concrete lining and keyway excavation for plug.

cushion the effect of falling rock. The roof was designed for a rock load of 14,000 pounds. Details of the trashrack structure are shown on figure 29.

(c) *Tunnel Plugs.*—Both tunnels were permanently closed by concrete tunnel plugs. To facilitate construction of the permanent plug section for the right diversion tunnel, the entrance was provided with a closure structure consisting of three structural steel slide gates designed for a 90-foot head and a temporary concrete plug immediately downstream from the gates designed to withstand a 200-foot head. These facilities were designed by the contractor and reviewed by the Bureau.

The right tunnel plug consisted of three 50-foot sections keyed into the walls of the tunnel. The area between the plug and the spillway lining was filled with backfill concrete. Details of the right diversion tunnel plug are shown on figure 30.

The left diversion tunnel plug was similar to that for the right tunnel except that the upstream section of the plug was increased in length to 75 feet to accommodate the outlet gates. A gate operating chamber, conduits, and access adit were also formed in the first two sections of the plug (fig. 23). These voids were filled with backfill concrete and grouted after the outlets had served their purpose.

(d) *Outlet Works.*—Because of the high-velocity water, the diversion tunnel outlet conduits were lined with steel except for the top of the conduits, downstream of the gates. The conduits, downstream of the gates, were made larger to aerate the jet and provide free flow conditions.

The conduits and gates were reinforced for the following loads:

(1) Internal bursting pressure in the conduits upstream from the gates.

(2) Differential pressures between adjacent conduits upstream from the gates.

(3) Internal bursting pressure in the frames and bonnets of the gates.

(4) The grout load due to the periphery grouting.

The differential pressure between adjacent conduits upstream from the gates occurred with one gate open and an adjacent gate closed. The differential pressures were determined from the laboratory model.

22. 7.0- BY 10.5-FOOT OUTLET GATES AND CONTROLS. (a) *Description.*—Six 7.0- by 10.5-foot outlet gates were installed in the tunnel plug in the left diversion tunnel to regulate discharges through the three outlets. The gates were used from the time of closure of the right diversion tunnel until discharges could be made through the permanent outlet works at which time the outlet was taken out of service and plugged with concrete.

The gates were manufactured by Yuba Consolidated Industries, Inc., Benicia, Calif., under invitation No. DS-5216. The controls were manufactured by Kendo, Inc., Denver, Colo., under invitation No. DS-5364.

Two 7.0- by 10.5-foot outlet gates, which are identical, are installed in tandem in each of the three outlets; the upstream gate in each pair serves as a guard gate for the downstream regulating gate. Steel liners extend upstream to bellmouth entrances at the plug face, 40 feet from the guard gates, and downstream from the service gates a distance of 68 feet to the end of the tunnel plug. Installation and assembly details are shown on figures 31 and 32.

(1) *Gates.*—Each gate has a steel body and bonnet, a flat, cast steel leaf and cast steel bonnet cover. An oil-operated hydraulic hoist is mounted on the bonnet cover and has a 32-inch-inside-diameter cylinder with a 10-foot 7-inch stroke for operating the leaf. The piston is connected to the leaf by a corrosion-resisting steel stem and bronze nut. Bronze seal bars on the gate leaf bear and slide on nickel-copper alloy seat surfaces in the body and bonnet. The leaf position is shown by a full-scale, direct-reading indicator mounted on the side of each hoist cylinder. The estimated weight of the six outlet gates with hoist cylinders, liners, and anchor bolts is 1,370,000 pounds.

(2) *Controls.*—A single control cabinet in the gate chamber contains the hydraulic and electrical equipment for operating all six gates and has a single oil tank mounted on the wall of the gate chamber directly above the cabinet. The estimated weight of the controls for the six gates is 4,500 pounds.

(b) *Design.*—

(1) *Outlet gates.*—All six gates were designed to open or close with full flow under a maximum

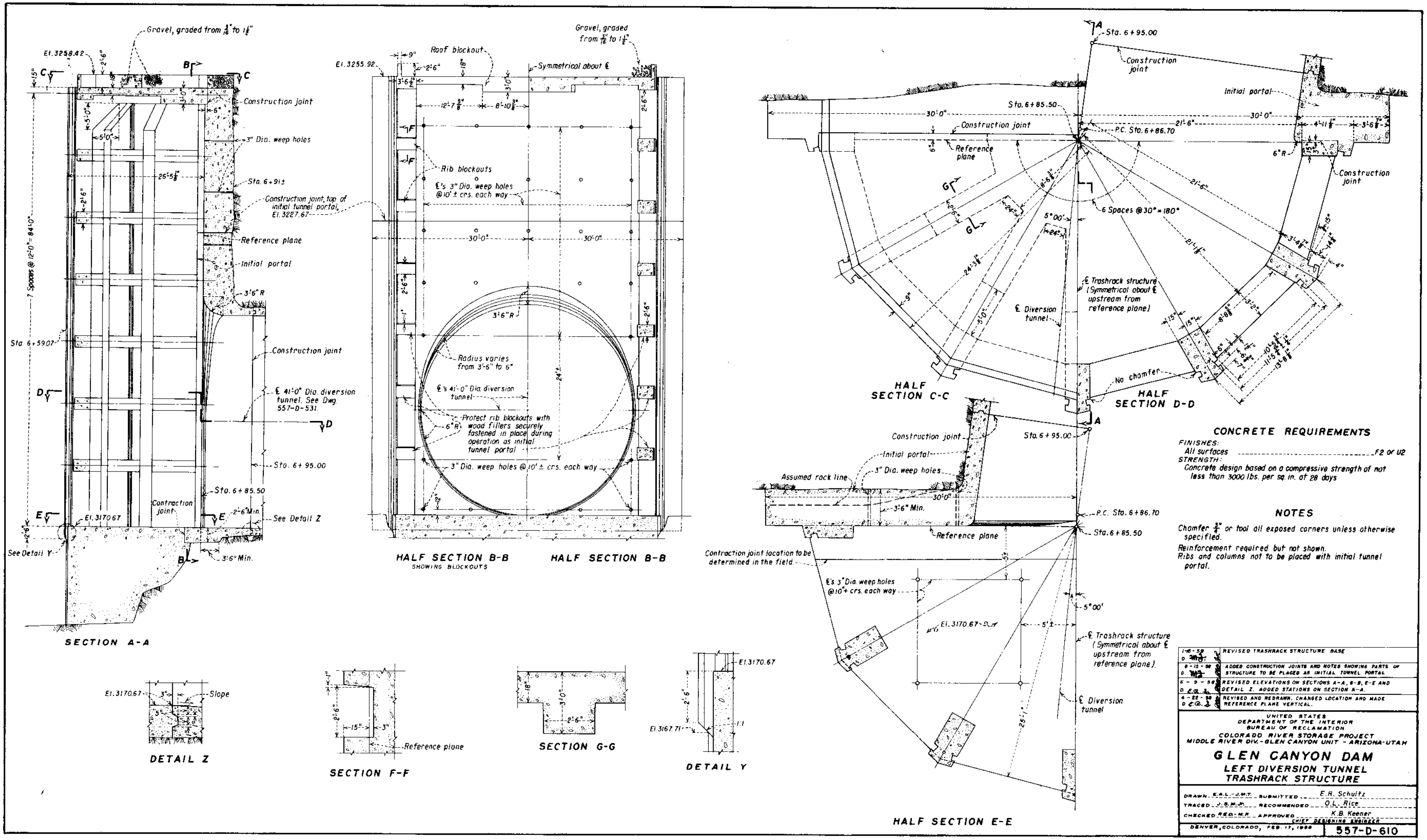


Figure 29.—Left diversion tunnel—Trashrack structure.

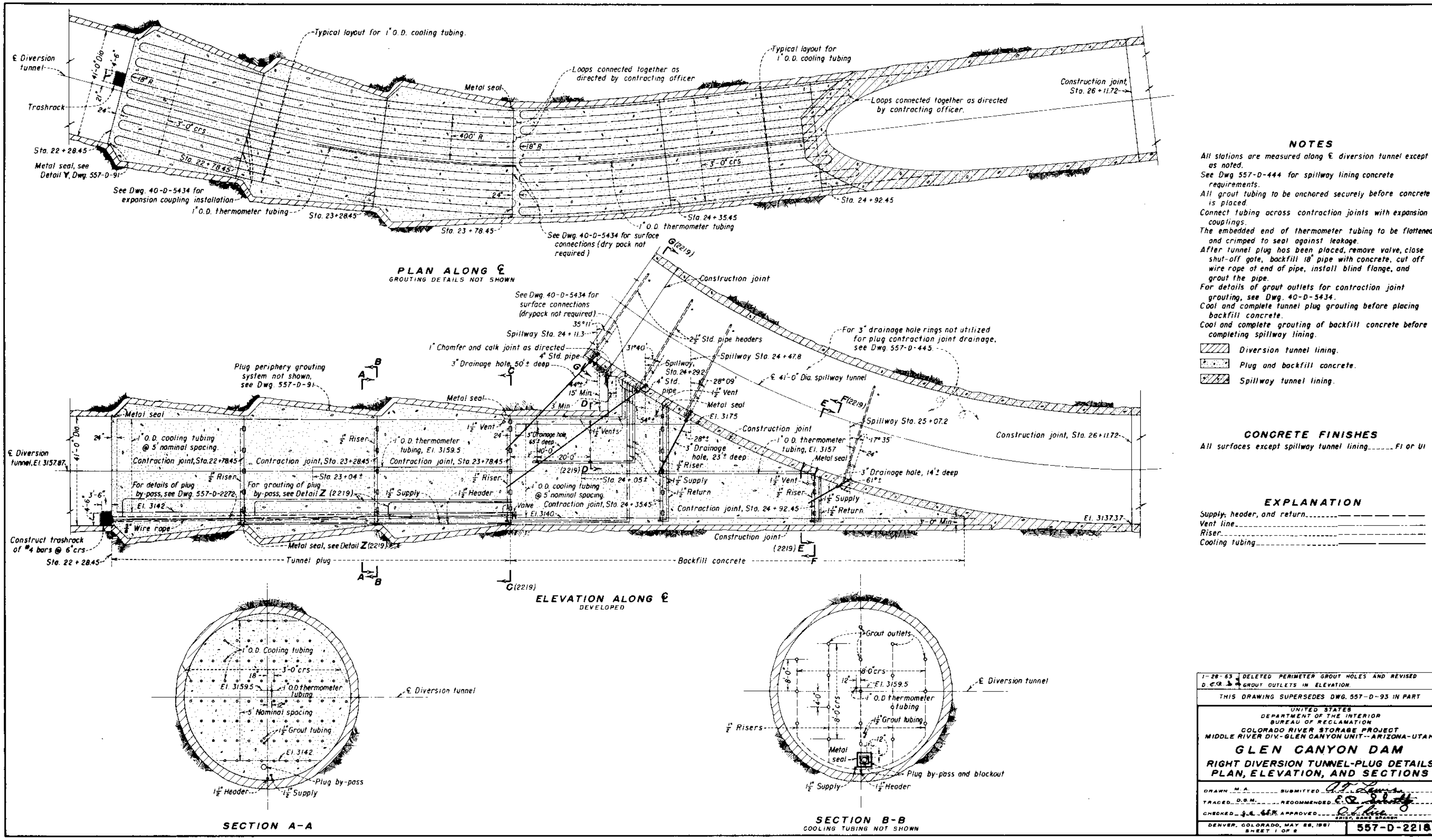


Figure 30.—Right diversion tunnel plug details—Plan, elevation, and sections. (Sheet 1 of 2.)

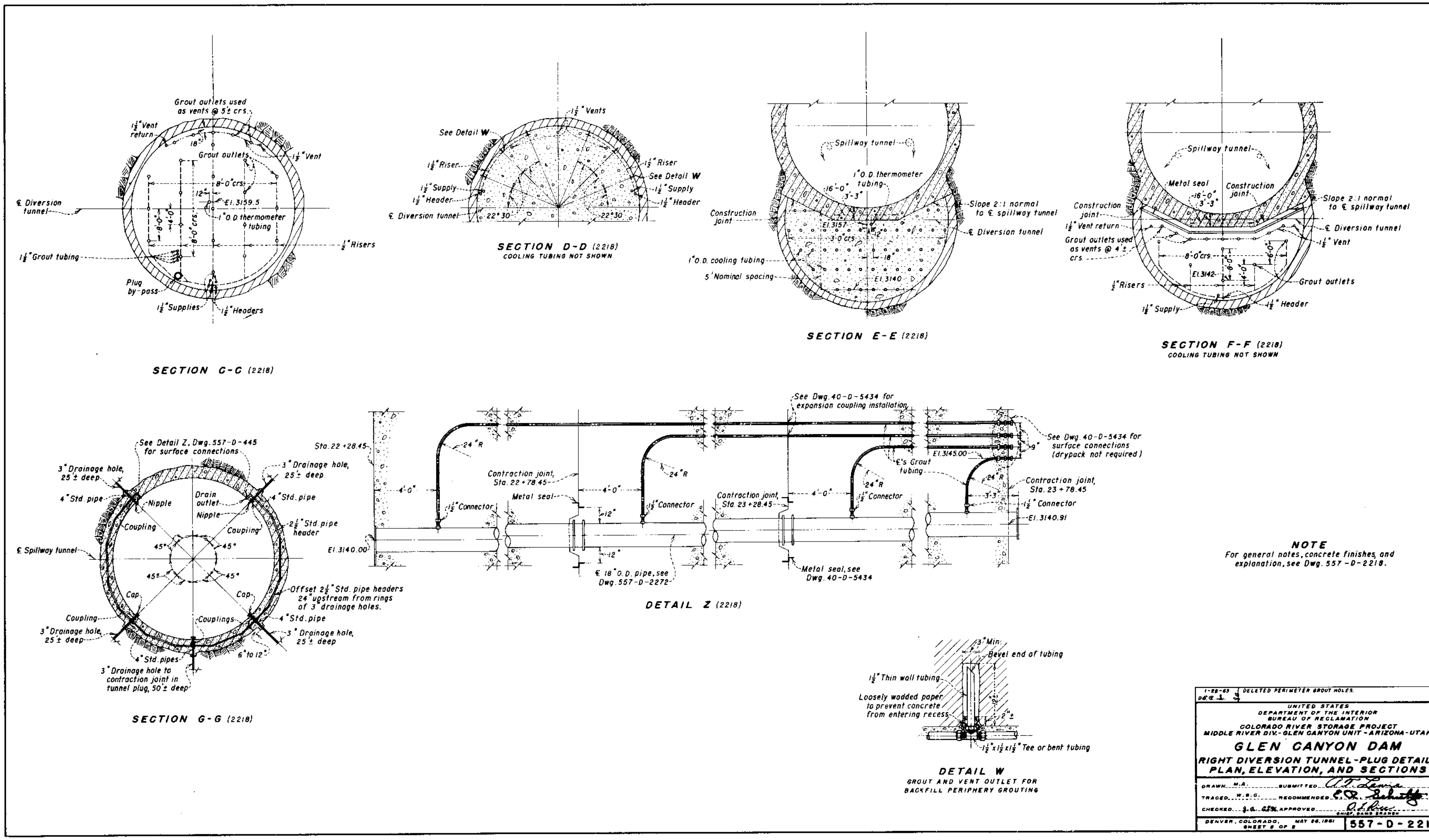


Figure 30.—Right diversion tunnel plug details—Plan, elevation, and sections. (Sheet 2 of 2.)

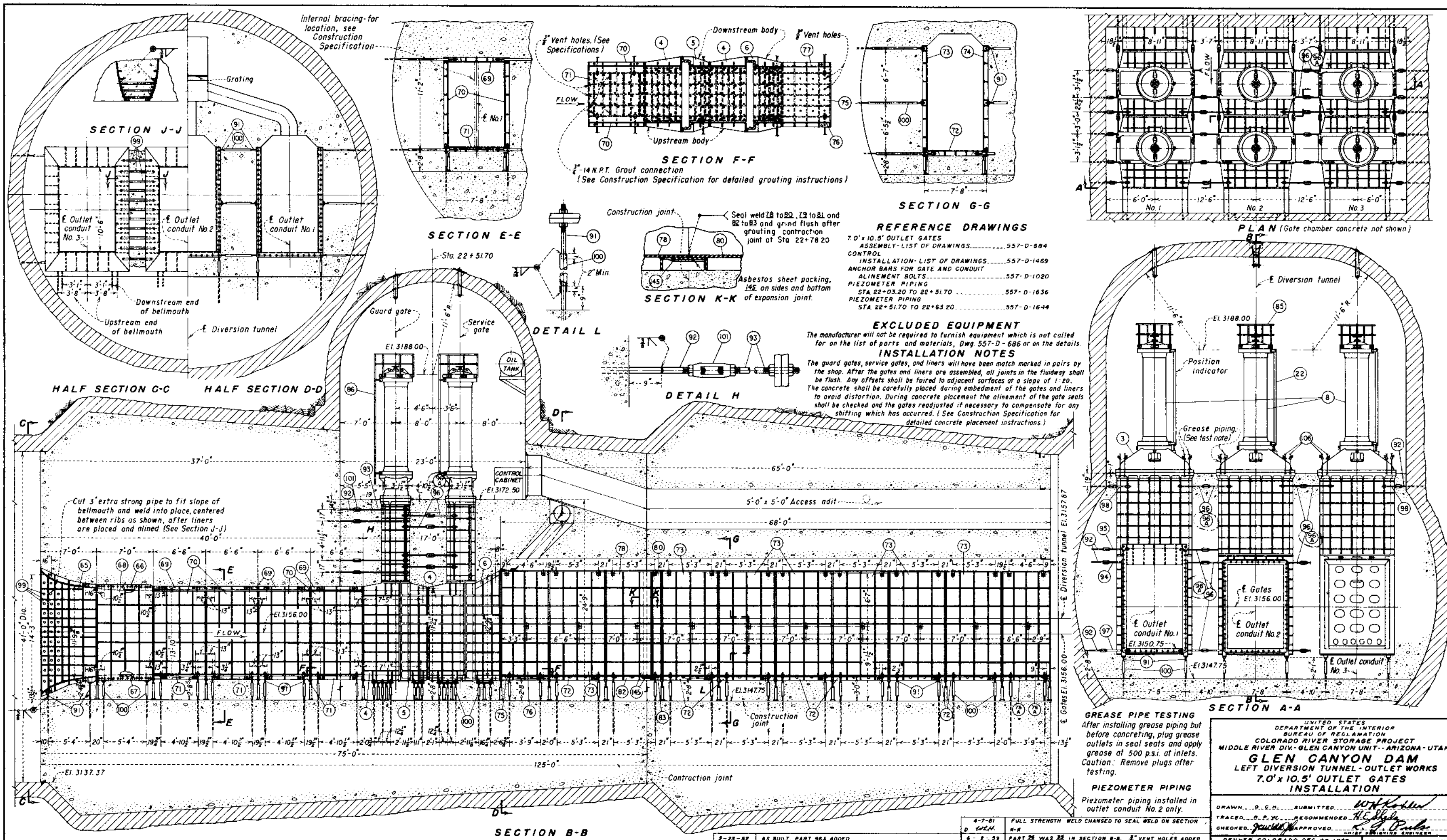


Figure 31.—Left diversion tunnel outlet works—7.0- by 10.5-foot outlet gates installation.

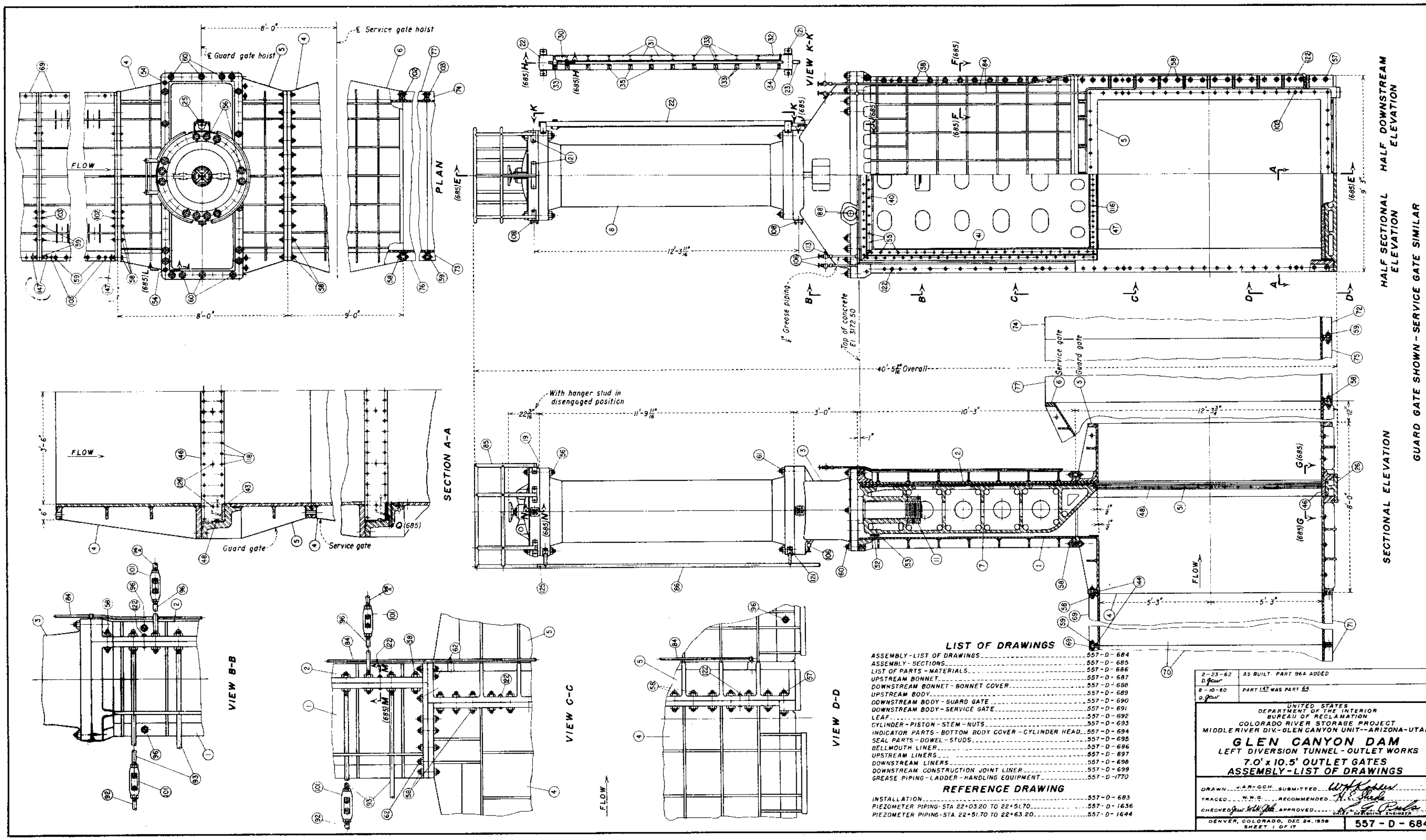


Figure 32.—Left diversion tunnel outlet works—7.0- by 10.5-foot outlet gates assembly.

reservoir head of 350 feet and to safely withstand a maximum nonoperating head of 561 feet. The gate bodies were designed for the internal pressure to be resisted by the reinforced concrete in which the gates are embedded. To reduce the flow disturbances past the slot opening, the width of the fluidway at the downstream edge of the slot was made 1 inch greater than at the upstream edge, with the sides of the downstream body converging the flow to the original width. This practice was based on a paper published in the Proceedings of the American Society of Civil Engineers.[2] The bonnet cover was designed to withstand the hydrostatic pressure plus the maximum thrust developed by the hydraulic hoist when seating the leaf.

The average bearing pressure between the leaf and body seats under the maximum reservoir head of 561 feet was limited to 1,675 pounds per square inch (static), and under the maximum operating head of 350 feet was limited to 1,040 pounds per square inch. A 1/16-inch-thick asbestos sheet packing was installed between the leaf and the leaf seals so that more nearly uniform bearing pressure across the seals could be obtained when the leaf deflects under load.

The hydraulic hoists were designed for an oil pressure of 2,000 pounds per square inch with normal factors of safety; however, the relief valve was set at 2,150 pounds per square inch to limit the maximum system pressure. The maximum pressure required to operate the gates will not exceed 1,950 pounds per square inch, at which pressure the hoists will develop a lifting force of approximately 1,400,000 pounds which exceeds the calculated force of 1,200,000 pounds required to lift the leaf and to overcome downpull and frictional resistance. The upper cylinder head of each hoist was designed with a hanger which can be manually engaged with the piston stem to hold the gate open. Each hanger stud is equipped with a break stud designed to support the leaf and piston weight of approximately 35,000 pounds and to break at approximately 92,000 pounds in case gate closure is initiated before the hanger stud is disengaged.

(2) *Controls.*—To provide the desired operating time of approximately 30 minutes per gate, a pumping capacity of 13.6 gallons per minute at 2,000 pounds per square inch was necessary. Two pumps were provided so that if one pump or motor fails, the other will operate a gate but will require twice the normal operating time. Each pump is driven by a 10-horsepower, 1,200-r.p.m., 440-volt, 3-phase, 60-cycle electric motor. The pumps have a combined capacity of 13.6 gallons per minute at a pressure of 2,000 pounds per square inch.

Adequate control for each gate required a single lever-operated four-way valve for directing the flow of oil from the pumps into two headers. The headers are connected to the bottom and top of the cylinders through distributing lines to the individual hoists, and the flow of oil is controlled by hand-operated isolating valves.

The control system required a 360-gallon tank to provide storage and expansion capacity for the hydraulic oil when the gates are operated. The control design was based on the use of hydraulic oil which has a viscosity of 153 Saybolt seconds universal at 100° F. and a viscosity index of 97. The minimum operating temperature was assumed as 50° F.

(c) *Design Stresses.*—

(1) *Tension.*—The allowable design stresses in tension for the following materials were based on the yield point or the ultimate strength of the material. The smaller of the tabulated values was used in each instance.

Material	Type	Percent of yield point	Percent of ultimate tensile strength
Steel	Rolled or forged	40	25
Bolt steel	Rolled or forged	25	16.5
Cast steel	Cast	33	20
Brass or bronze	Rolled or cast	33	16.5

(2) *Compression.*—The allowable design stresses in compression used for the materials listed above were the same as for tension.

(3) *Bearing.*—The allowable design stress used for seal bearing was limited to 1,040 pounds per

[2] Ball, J. W., "Hydraulic Characteristics of Gate Slots," Journal of the Hydraulics Division—Proceedings of the American Society of Civil Engineers, Paper No. 2224, October 1959.

square inch for operating and 1,675 pounds per square inch for nonoperating (static) conditions.

(4) *Shear.*—Allowable design stresses in shear were not more than 0.6 of the allowable design stresses in tension.

(5) *Hoist cylinder.*—Allowable design stresses for hoist cylinders were based on the recommendations of the ASME Boiler and Pressure Vessel Code—Unfired Pressure Vessels—Section VIII.

B. STRUCTURAL DESIGN OF DAM

1. Analysis of Stress and Stability Factors for Dam

23. GENERAL. The degree of safety of an arch-type dam is defined by the stresses developed in the dam. These stresses can be obtained through mathematical analyses or by model studies. The design of Glen Canyon Dam was developed using a mathematical analysis, termed "Trial-load Method of Stress Analysis." A brief description of this system is presented in the following section.

24. METHOD OF ANALYSES. The trial-load method of stress analysis assumes that the load applied to an arch dam is divided between horizontal and vertical elements in such a way as to produce equal movements in all directions, at points of intersection of these horizontal and vertical elements. Because the required agreement of all deformations can best be obtained by assuming different distributions of load and computing the resulting movements until the specified conditions are fulfilled, the procedure is logically called the trial-load method.

It may be assumed that the dam to be analyzed is divided into a series of arch and cantilever elements by passing through it a series of horizontal and vertical planes. The horizontal planes defining the arch elements are assumed to be spaced a unit distance apart, and the vertical radial planes defining the cantilever elements spaced a unit distance apart at the axis of the dam. The sum of the arch elements occupies the total volume of the dam, which is also the case with the cantilever elements. Each arch and each cantilever is assumed to move independently of all others, but at the conclusion of the analysis, geometrical continuity must be restored at all points in the structure.

Instead of investigating a large number of horizontal and vertical elements, only a relatively few representative arches and cantilevers are studied to complete the analysis within a reasonable length of time. If the dam is approximately symmetrical about the maximum cantilever section, only half of the structure needs be analyzed, and five to seven cantilevers may be sufficient. If the structure is nonsymmetrical, however, both sides must be analyzed and approximately twice as many cantilevers are necessary.

The trial-load analysis is carried out in steps, generally referred to as adjustments. Three adjustments—radial, tangential, and twist—are necessary to achieve geometrical congruence. The radial adjustment accounts for radial displacement. Tangential movement is brought into agreement by use of the tangential adjustment. The twist adjustment provides rotational congruency about the tangential and vertical axes. When equality of these linear and angular displacements of the arch with those of the cantilever has been achieved at their points of juncture, the requirements for a solution are complete.

The loads on the arch and cantilever elements that produce deformation agreement may be freely chosen with the provision that the sum of the arch and cantilever loads must equal the external load at every point. The external loads include all external forces acting on the dam. These consist of the weight of the structure, reservoir water, tailwater, temperature changes, earthquake shocks, and silt loads. These loads are divided between the arch and cantilever system until a satisfactory agreement of radial deflections is obtained.

To complete the deformation agreement, it is necessary to introduce internal tangential and twist loads. These are applied in pairs of equal and opposite loads, one acting against the arch and the other against the cantilever. By this means, arch and cantilever deflections may be brought into tangential and rotational agreement without changing the external load on the structure. The internal loads represent forces set up by the interaction between the assumed arches and cantilevers in the dam.

To facilitate the process of dividing the loads between the horizontal and vertical elements, certain patterns of loads called unit loads have been developed. In the case of the arches, these loads consist of a uniform load over the entire length of the arch and triangular loads varying from a maximum value at the abutment to zero at the quarter points. These loads may be used to represent radial forces, tangential thrusts, or moments, depending on the adjustment under consideration. The unit cantilever loads are assumed to vary from a maximum at one arch elevation to zero at the arches directly above and below. These loads are used to represent radial shears, tangential shears, or twisting moments on the cantilevers, depending on the appropriate adjustment.

In applying the unit loads, it is advantageous to compute the movements of arch and cantilever elements produced by the unit loads before attempting to divide the external load between the arches and cantilevers. The arches are statically indeterminate structures terminating at elastic abutments. Computations of deflections of arch elements are made by the elementary theory of flexure for curved beams, with the effects of rib-shortening, transverse shear, and yielding abutments included. The arch elements resist radial forces applied at the faces, tangential forces and horizontal moments applied at the centerlines, and twisting moments in vertical radial planes. The cantilevers are elastic units, seated on an elastic foundation. They resist vertical and radial forces applied to the upstream or downstream faces, and tangential forces, twisting and bending moments applied at the centerlines.

The total loads resisted by horizontal and vertical elements are determined by the trial-load adjustments. With the loads, stresses may be computed throughout the dam, provided a definite variation of stress between the upstream and downstream faces of the dam is assumed. Three of the stresses—the vertical stress normal to a horizontal plane, the horizontal stress normal to a vertical radial plane, and the horizontal tangential shear stress acting in a tangential direction on a horizontal plane—are assumed to vary linearly between the upstream and downstream faces. Arch and cantilever shearing stresses are assumed to vary parabolically from the upstream face to the downstream face of the dam. These stresses may be computed using the total arch and cantilever loads. From these stresses, principal stresses or stresses on any plane may be computed throughout the dam.

25. BASIC DESIGN DATA. The following criteria were used in the trial-load analyses of Glen Canyon Dam, except where noted:

(1) Top of dam, elevation 3715.

(2) Normal reservoir water surface, elevation 3700.

(3) Top of saturated backfill on downstream face, elevation 3158.

(4) Minimum tailwater surface, elevation 3142.

(5) Temperatures used in the analyses are changes between average arch temperatures at time of grouting contraction joints and minimum operating temperatures. Operating temperatures are assumed to vary linearly from the upstream face to the downstream face of the dam.

(6) Coefficient of thermal expansion of concrete, 0.000,005,6 per degree Fahrenheit.

(7) Effects of silt and uplift omitted.

(8) Sustained modulus of elasticity of concrete, 3,000,000 pounds per square inch.

(9) Poisson's ratio of concrete 0.20.

(10) Unit weight of concrete, 150 pounds per cubic foot.

(11) Unit weight of water, 62.5 pounds per cubic foot.

(12) Unit weight of saturated fill, 120 pounds per cubic foot. Saturated fill assumed to have an equivalent horizontal fluid pressure of 85 pounds per cubic foot.

(13) Effects of earthquake, unless omitted, are included as follows:

> Earthquake is assumed to move the dam upstream and downstream horizontally in a direction parallel to the plane of centers with an acceleration of 0.1 gravity and a period of vibration of 1.0 second. Increased water pressure assumed to act equally on all cantilevers. Effects of vertical acceleration are not included.

Because the abutment rock at the Glen Canyon damsite has low strength and high yielding characteristics, many tests were performed to determine working values for the sustained modulus of elasticity, Poisson's ratio, and strength. Evaluation of the tests resulted in numerous changes, so these values are listed with each study.

The construction, grouting, and storage programs were divided into steps to facilitate making the analyses. As the programs developed, these steps were revised to be more realistic or to meet anticipated schedules. For this reason, these steps are described with each study.

26. BASIC ASSUMPTIONS. The following assumptions were used in the trial-load analyses of Glen Canyon Dam:

(1) The horizontal elements are assumed to be symmetrical with symmetrical loading except in the studies for design A-22, where the elements are nonsymmetrical with nonsymmetrical loading.

(2) Arch abutments are assumed to be radial, except in the studies for design A-22 where the abutments are triangular, approximating the actual excavation.

(3) Arch, cantilever, and tangential shearing stresses vary linearly from the upstream face to the downstream face of the dam.

(4) Radial arch and cantilever shearing stresses have a parabolic distribution from the upstream face to the downstream face.

(5) Competent rock formations exist at the damsite and are capable of carrying the loads transmitted by the dam with stresses well below the elastic limit.

(6) Concrete in the dam is homogeneous and uniformly elastic in all directions and strong enough to carry the applied loads with stresses well below the elastic limit.

(7) The arches and cantilevers are assumed fixed relative to the foundation rock.

27. PRELIMINARY DESIGNS. The major factors influencing the design of Glen Canyon Dam are the wide U-shaped canyon and the relatively low modulus of elasticity with corresponding low strength characteristics of the abutment rock.

A dam in a U-shaped valley generally requires more concrete than one in a V-shaped valley. In a wide U-shaped valley, load in the central portion of the valley is carried almost entirely by cantilever action vertically to the foundation. To carry this load, the vertical section must be thicker than would be required in a V-shaped canyon. This additional thickness may increase the volume of concrete appreciably in an arch dam. Because of the low strength and low modulus of elasticity of the abutment rock at the Glen Canyon site, the dam will require wider abutments than normal to spread the forces over a larger area, and thus reduce the stresses. The abutments yield in an arch dam when the waterload is applied to the dam. This yielding produces increased compressive stresses in the arches at the crown extrados and abutment intrados, and decreases the compressive stresses at the crown intrados and abutment extrados. A reduction in the amount of yielding by the abutment is accomplished by increasing the area of contact between the concrete and rock.

A gravity-type dam could be designed with an overflow spillway, eliminating the necessity of tunnel spillways through the abutments. However, in a gravity dam uplift pressures due to porosity of the rock would be a design factor and an overflow spillway would make it necessary to put the powerplant underground or along the side of the canyon. An arch dam would require less concrete in the dam than the gravity design, but provision for a spillway around the abutments would be required. The powerplant could be underground, or at the toe of the dam.

With these alternatives under consideration, schemes with both straight gravity and thick arch dams were considered. The gravity dam was studied with an overflow spillway and an underground powerplant. Several thick arch dams were laid out using radii to the axis of 1,000 feet and 1,100 feet, with slopes on the downstream face of 0.55 vertical to 1.0 horizontal. An underground powerplant and a powerplant at the toe of the dam were considered with the arch-type dam. The scheme with a powerplant at the toe of an arch dam proved to be the more economical; therefore no further consideration was given the design for gravity dam or underground powerplant.

The early preliminary designs were made for a reservoir capacity of 30,000,000 acre-feet. With this capacity, the normal reservoir water surface was at elevation 3725 and the top of the dam at elevation 3740. Base of the maximum section was assumed to be at elevation 3040. Estimated costs of a dam and powerplant were based on a preliminary design.

In 1956, soon after Congressional approval of the Colorado River Storage project more information on the abutment rock was obtained and a refined reservoir capacity study was prepared. As a result, the normal reservoir water surface elevation was set at 3700, with the top of the dam at elevation 3715. The base of the maximum section was assumed to be at elevation 3010. Laboratory tests indicated a value of 750,000 pounds per square inch for the modulus of elasticity of the abutment rock and 0.15 for Poisson's ratio. At the abutments a limiting compressive stress of 750 pounds per square inch for a loading condition, including the effects of earthquake, was tentatively set. In the interior of the dam a compressive stress of 1,000 pounds per square inch was the limiting stress established by design criteria.

(a) *Design A-6.*—Design A-6, the first of the new layouts, is shown on figure 33. The shortest usable radius to the axis of the dam was found to be 900 feet. By thus reducing the radius, more load is taken in arch action and less in the vertical direction. Consequently, the vertical section could be thinned. To reduce tensile cantilever stresses on the downstream face near the top of the dam, the upstream face was curved in a vertical plane. The crown cantilever analysis in design A–6 revealed the presence of compressive stresses in excess of 750 pounds per square inch at the abutments of the top half of the dam. The need for thicker abutments in the upper portion was evident.

(b) *Design A-7.*—By reducing the intrados radii in the upper portion of the dam in design A-7, the abutment thicknesses were increased. The crown section of design A-6 was retained. A plan and maximum section for design A-7 is shown on figure 34.

A radial adjustment analysis showed excessive abutment stresses at the top of the dam. By leaving that portion of the dam above elevation 3665 ungrouted, thus assuming no load above this elevation to be carried by arch action, stresses from the complete trial-load analysis were found to be well within the limits set at the time. The arch and cantilever stresses parallel to the faces, along with the loading conditions and assumptions used in the complete trial-load analysis, are shown on figure 35.

In reevaluating the strength of the rock, a limiting compressive stress at the abutments of 600 pounds per square inch was established. To reduce the stresses of design A-7 to an acceptable limit, several steps were considered other than increasing the abutment thickness. Temperature in the dam at the time of grouting had been assumed to be 45° F. at all levels of the dam. By varying this temperature from 40° F. in the bottom to 55° F. at the top, the bottom part could support more of the load, while relieving some of the load in the top. The other measure taken was to formulate a construction program that was realistic and would force the arches and cantilevers in the lower part of the dam to support more of the load. This construction program assumed the dam constructed to elevation 3550 and grouted to elevation 3500. As construction is continued, water would be stored in the reservoir, and when the dam is topped out, the water in the reservoir would have been raised to elevation 3500. The grouting would then be completed, and the water in the reservoir would be allowed to rise to a normal level of 3710.

A comparison of stresses including the effects of the construction program and omitting them is shown on figure 36. At the abutments the arch stresses are reduced from 20 percent to 30 percent in the upper portion of the dam. Although these stresses were improved, they were not considered entirely satisfactory.

To further reduce the critical abutment stresses, the abutments had to be thickened. Since further reduction of the intrados radii did not appear to be feasible, the alternative was to add concrete on the upstream face.

28. SPECIFICATIONS DESIGN—DESIGN A-8. In design A-8, as shown on figure 37, 15 feet of concrete was added to the maximum section at the base and extending up to elevation 3300. From here the face was curved in a vertical plane to the axis at elevation 3710. Stresses listed at the bottom of figure 37 are estimated final stresses based on stresses resulting from a Crown Cantilever Analysis. Since these stresses were acceptable at the time, specifications for construction of Glen Canyon Dam were based on this layout.

29. SOME DESIGNS PREPARED SUBSEQUENT TO SPECIFICATIONS ISSUANCE. After specifications were issued, based on design A-8, a number of additional layouts were made in an effort to reduce the volume of concrete in the dam and produce a more acceptable stress distribution on the abutment rock. Additional tests of the abutment rock resulted in a lower value of the modulus of elasticity, which in turn tended to increase the compressive stresses at the abutment intrados. Stresses in other portions of the dam were conservative and well below the allowable limits. The problem of design resolved into how to increase the abutment thickness while maintaining or reducing all other thicknesses. This could be accomplished by using uniform thickness sections in the central portion of the dam, terminating in short radii fillets on the downstream face.

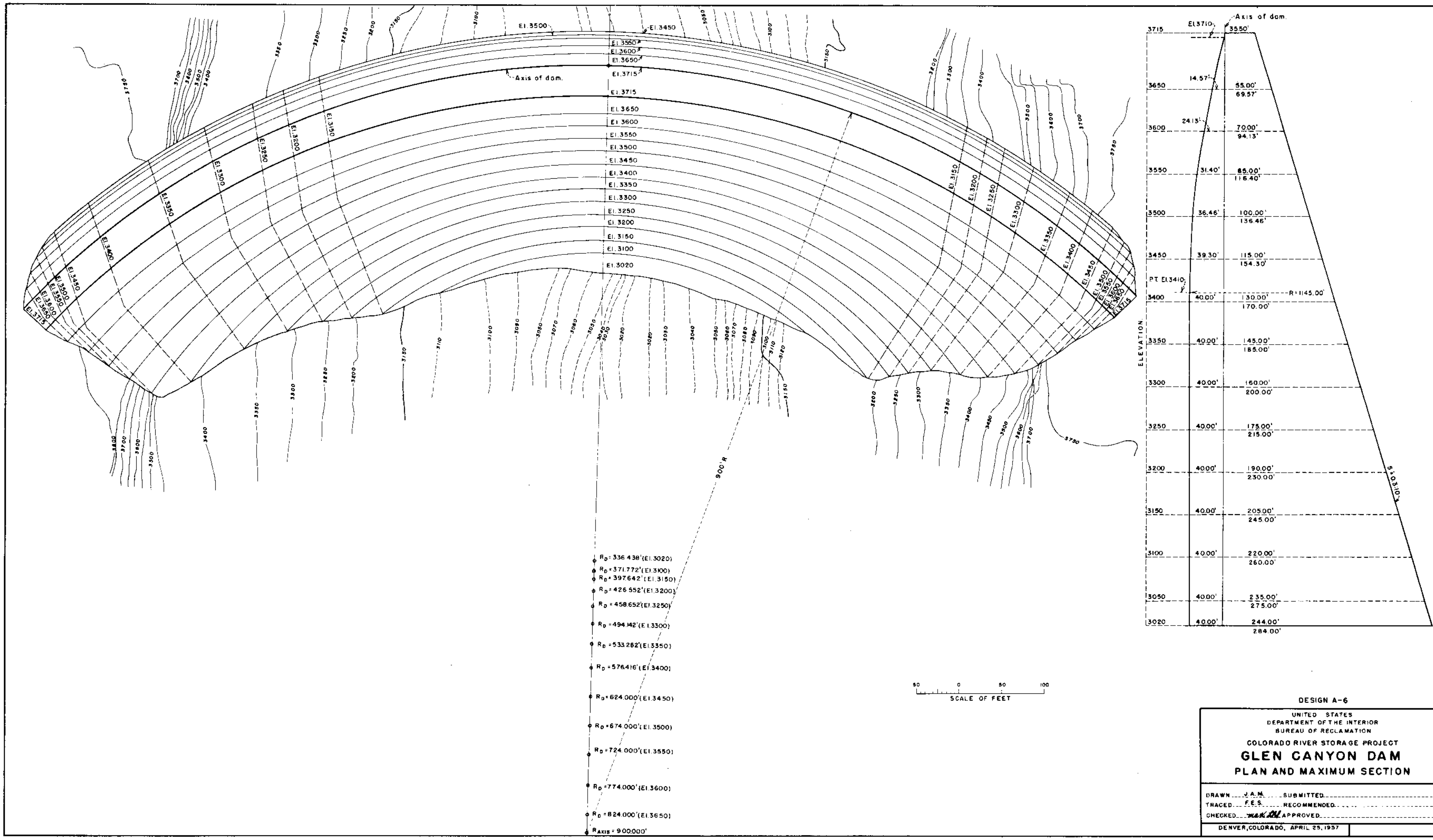
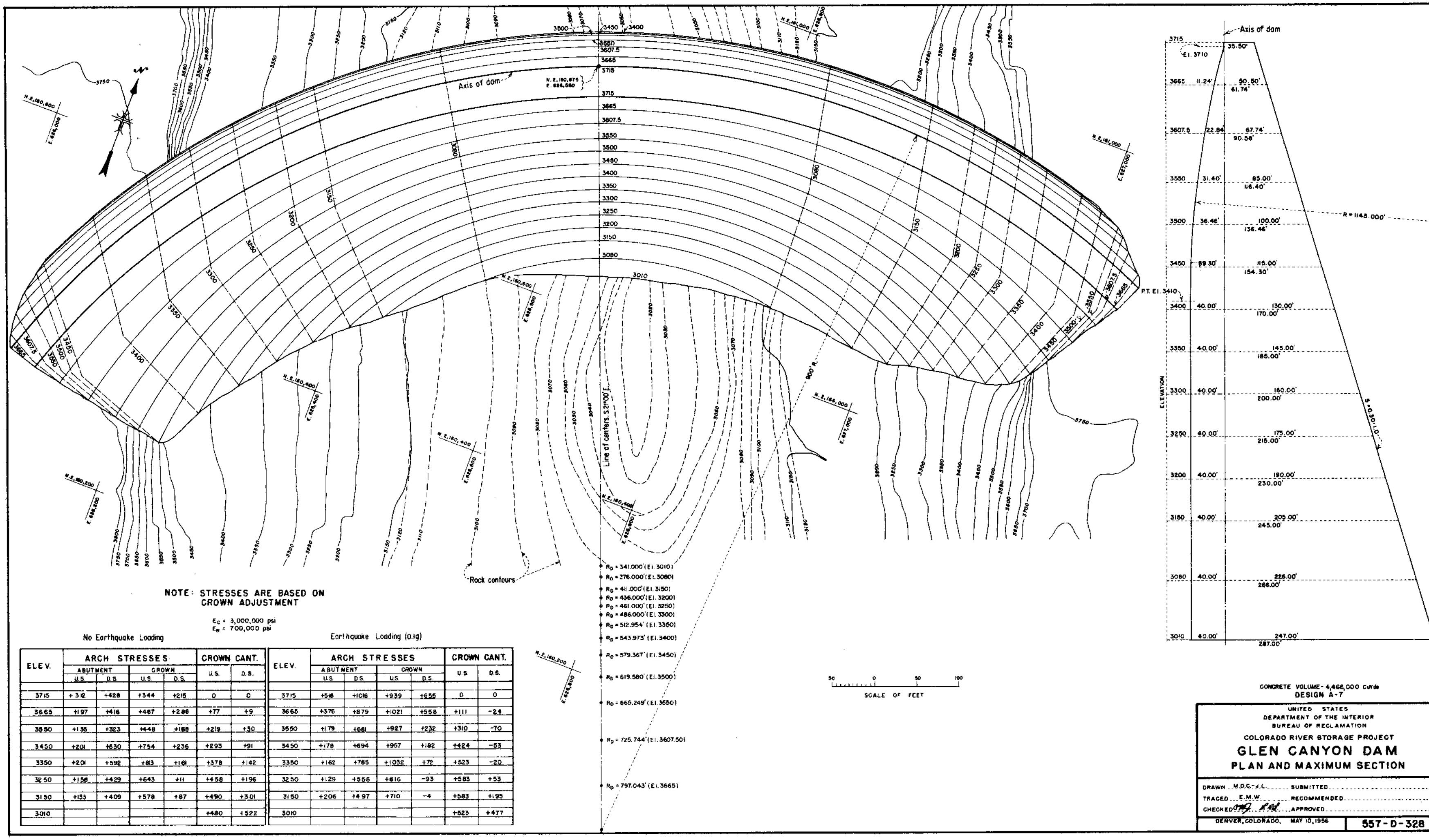


Figure 33.—Dam design A-6—Plan and maximum section.



No Earthquake Loading

ELEV.	ARCH STRESSES				CROWN CANT.	
	ABUTMENT		CROWN			
	U.S.	D.S.	U.S.	D.S.	U.S.	D.S.
3715	+312	+428	+344	+215	0	0
3665	+197	+416	+487	+286	+77	+9
3550	+135	+323	+448	+188	+219	+30
3450	+201	+630	+754	+236	+293	+91
3350	+201	+592	+813	+161	+378	+142
3250	+158	+429	+643	+11	+458	+196
3150	+133	+409	+578	+87	+490	+301
3010					+480	+522

Earthquake Loading (0.1g)

ELEV.	ARCH STRESSES				CROWN CANT.	
	ABUTMENT		CROWN			
	U.S.	D.S.	U.S.	D.S.	U.S.	D.S.
3715	+516	+1016	+939	+655	0	0
3665	+376	+879	+1021	+558	+111	-24
3550	+179	+681	+927	+232	+310	-70
3450	+178	+694	+957	+182	+424	-53
3350	+162	+785	+1032	+72	+523	-20
3250	+129	+558	+816	-93	+583	+53
3150	+206	+497	+710	-4	+583	+195
3010					+623	+477

Figure 34.—Dam design A-7—Plan and maximum section.

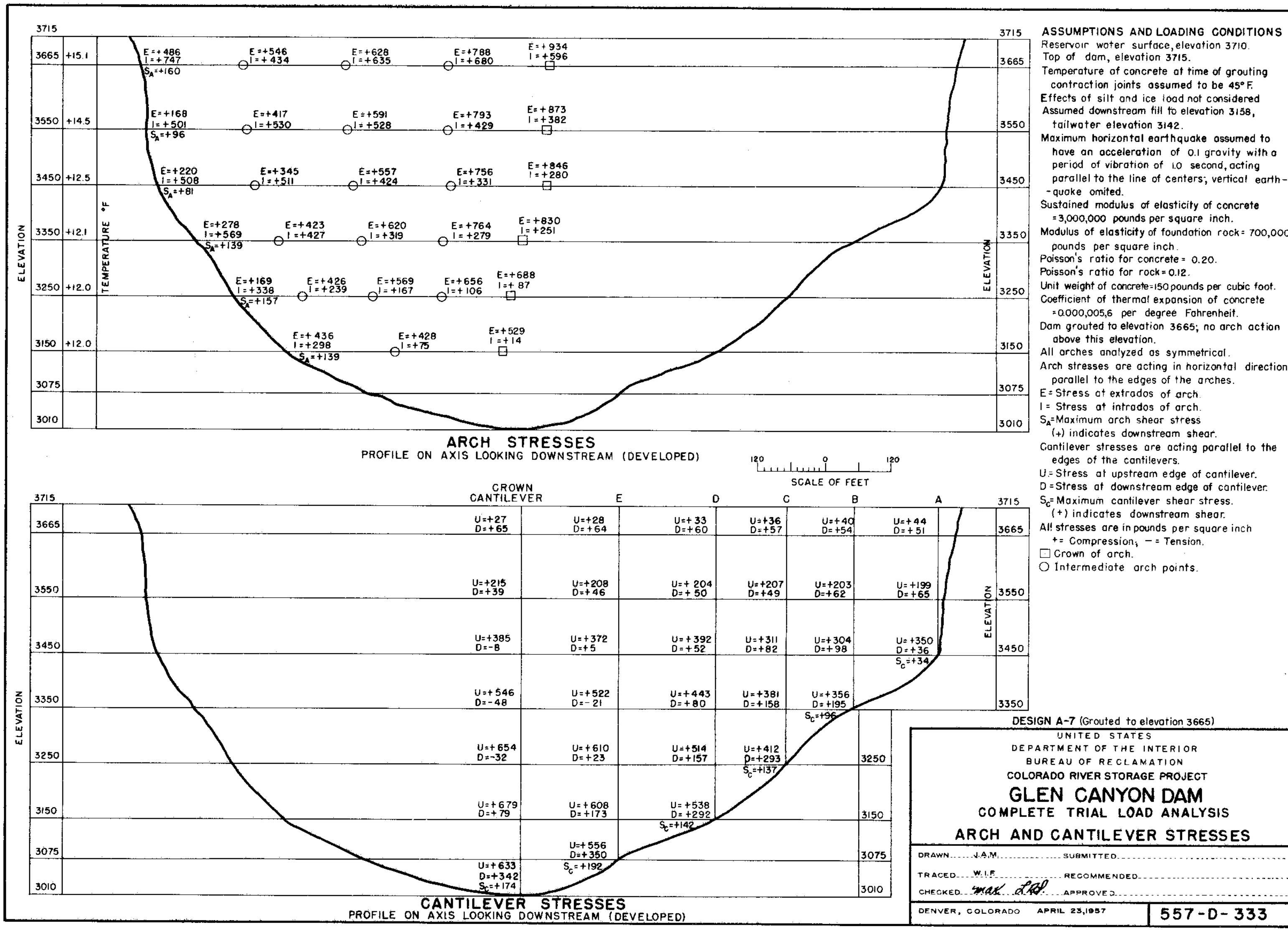


Figure 35.—Dam design A-7 (grouted to elevation 3665), complete trial-load analysis—Arch and cantiliver stresses.

ARCH AND CANTILEVER STRESSES

Construction Program included

ELEVATION		ARCH STRESSES ABUT.	ARCH STRESSES CROWN	CROWN CANT.
3715	U.S.	+413	+631	0
	D.S.	+692	+405	0
3665	U.S.	+273	+770	+83
	D.S.	+664	+410	+4
3550	U.S.	+160	+698	+221
	D.S.	+506	+211	+33
3450	U.S.	+229	+997	+281
	D.S.	+704	+254	+111
3350	U.S.	+223	+1,057	+352
	D.S.	+774	+244	+180
3250	U.S.	+173	+827	+416
	D.S.	+549	-38	+251
3150	U.S.	+161	+727	+430
	D.S.	+508	+59	+378
3010	U.S.			+394
	D.S.			+632

Construction Program not included

ELEVATION		ARCH STRESSES ABUT.	ARCH STRESSES CROWN	CROWN CANT.
3715	U.S.	+518	+939	0
	D.S.	+1016	+655	0
3665	U.S.	+376	+1021	+111
	D.S.	+879	+558	-24
3550	U.S.	+179	+927	+310
	D.S.	+681	+232	-70
3450	U.S.	+178	+957	+424
	D.S.	+694	+182	-53
3350	U.S.	+162	+1032	+523
	D.S.	+785	+72	-20
3250	U.S.	+129	+816	+583
	D.S.	+558	-93	+53
3150	U.S.	+206	+710	+583
	D.S.	+497	-4	+195
3010	U.S.			+523
	D.S.			+477

CONSTRUCTION PROGRAM:

(1) Concrete to El. 3550, contraction joints grouted to El. 3500 at 40°F with reservoir empty.

(2) Water surface at El. 3500, concrete to top of dam, El. 3715. Contraction joints grouted to El. 3600 at 45°F and to El. 3715 at 50°F.

(3) Reservoir surface at El. 3710, downstream saturated fill to El. 3158 and Earthquake effects included. All stresses are parallel to faces and are in pounds per square inch.

For other assumptions, see drawing No. 557-D.C.-94.

DESIGN A-7

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
GLEN CANYON DAM
EFFECTS OF CONSTRUCTION PROGRAM

DRAWN M.A.K. SUBMITTED
TRACED J.S.M. Jr. RECOMMENDED
CHECKED G.S.H. APPROVED

DENVER, COLORADO APRIL 17, 1957 557-DC-93

Figure 36.—Dam design A-7—Effects of construction program.

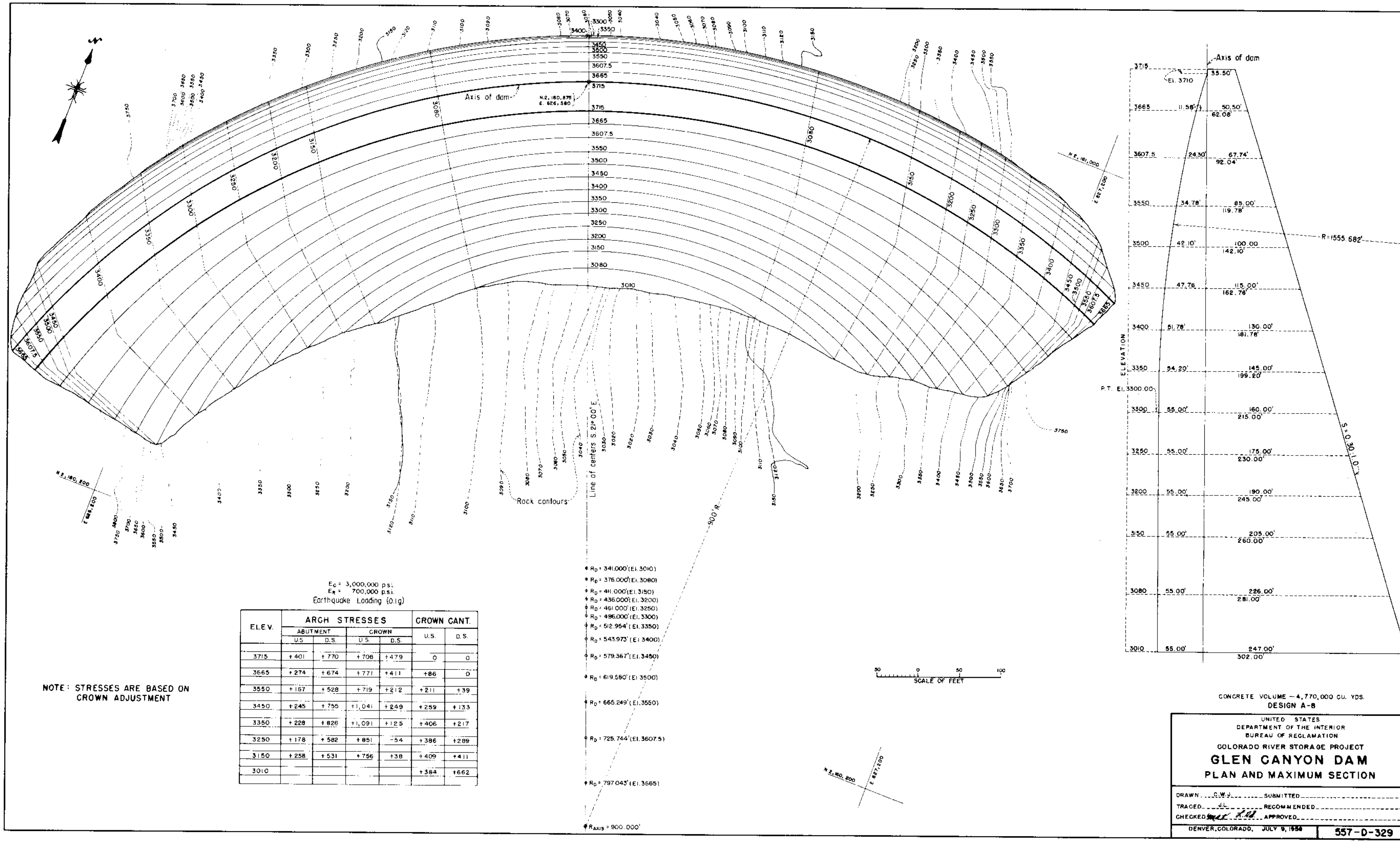
(a) *Design A-18.*—Design A-18, as shown on figure 38 was based on this premise. The maximum section has a top thickness of 25 feet and a base thickness of 295 feet. The horizontal sections are uniform in thickness in the central portion, extending to short-radii fillets on the downstream face, and terminating with tangents from the fillets. The tangents permit some flexibility to the abutment thicknesses, depending on the depth of excavation necessary. Estimated stresses for a complete adjustment indicated the design to be acceptable. However, by changing and rearranging the fillets and tangents on the downstream face of the dam, the rate of divergence of the faces near the abutments could be reduced and a better distribution of stresses effected.

(b) *Design A-19.*—This better distribution of stresses was accomplished in the layout of design A-19, shown on figure 39. The volume of concrete in this layout is about 6 percent greater than that in design A-8, the specifications design.

Figure 40 shows the arch and cantilever stresses and lists the assumptions and loading conditions used in the complete analysis of design A-19. The principal stresses at the abutments are shown on figure 41.

(c) *Design A-20.*—As excavation progressed on the dam the actual abutments became better defined. The layout with the abutments as excavated is shown on figure 42. Average radial abutments were assumed. With the refined abutments, this layout was designated as design A-20. A complete trial-load analysis was made for this design, considering five conditions of loading. While results approached those desired, refinements were made in the assumed loadings and certain minor changes were made in the dam configuration to further improve the stress distribution in the final design.

30. FINAL DESIGN—DESIGN A-22. After abutment excavations were completed a final layout of Glen Canyon Dam was made. This layout, designated design A-22, is shown on figure 43. The radius at the

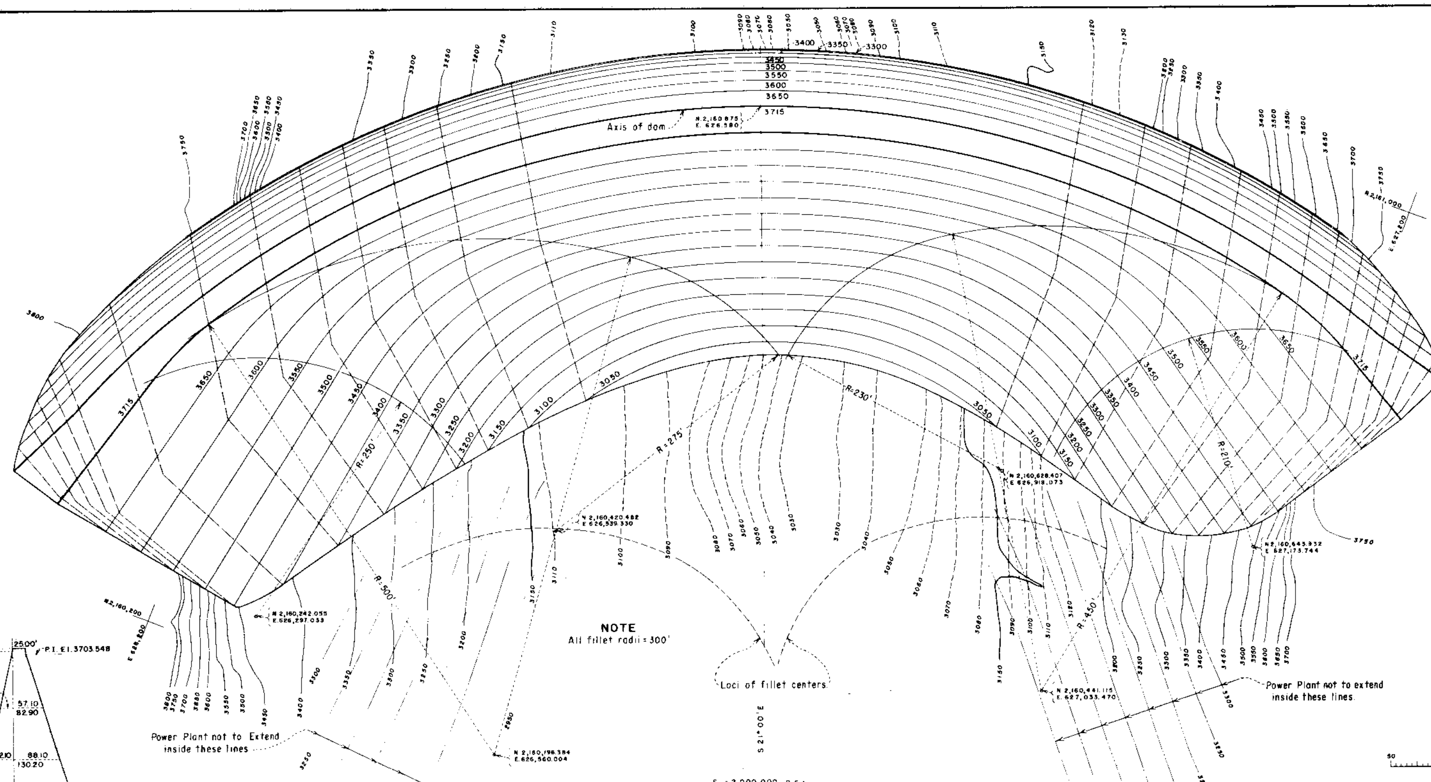


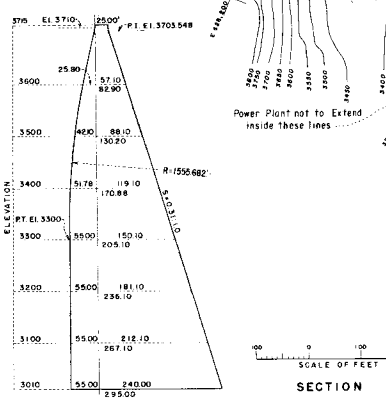
E_c = 3,000,000 p.s.i.
E_R = 700,000 p.s.i.
Earthquake Loading (0.1g)

ELEV.	ARCH STRESSES				CROWN CANT.	
	ABUTMENT		CROWN		U.S.	D.S.
	U.S.	D.S.	U.S.	D.S.		
3715	+401	+770	+708	+479	0	0
3665	+274	+674	+771	+411	+86	0
3550	+167	+528	+719	+212	+211	+39
3450	+245	+755	+1,041	+249	+259	+133
3350	+228	+826	+1,091	+125	+406	+217
3250	+178	+582	+851	-54	+386	+289
3150	+258	+531	+756	+38	+409	+411
3010					+384	+662

NOTE: STRESSES ARE BASED ON CROWN ADJUSTMENT

Figure 37.—Dam design A-8 (specifications design)—Plan and maximum section.





E_c = 2,000,000 p.s.i.
E_R = 500,000 p.s.i.

No Earthquake Loading

ELEV.	ARCH STRESSES				CROWN CANT.	
	ABUTMENT		CROWN		U.S.	D.S.
	U.S.	D.S.	U.S.	D.S.		
3715	+228	+154	+467	+310	0	0
3600	+147	+282	+609	+173	+171	-2
3500	+147	+392	+726	+151	+239	+61
3400	+188	+503	+824	+200	+310	+122
3300	+153	+463	+696	+66	+416	+145
3200	+141	+407	+556	1	+499	+195
3100	+165	+247	+385	+22	+521	+315
3010					+498	+486

Earthquake loading (0.1g)

ELEV.	ARCH STRESSES				CROWN CANT.	
	ABUTMENT		CROWN		U.S.	D.S.
	U.S.	D.S.	U.S.	D.S.		
3715	+334	+227	+688	+454	0	0
3600	+161	+396	+840	+209	+174	-4
3500	+169	+501	+923	+164	+235	+70
3400	+217	+634	+1,035	+221	+297	+144
3300	+173	+574	+865	+54	+395	+177
3200	+158	+496	+682	-27	+466	+240
3100	+171	+315	+491	-26	+471	+381
3010					+430	+575

* R axis = 900.000'

NOTE: STRESSES ARE BASED ON CROWN ADJUSTMENT

CONCRETE VOLUME – 5,073,000 CU. YDS.
DESIGN A-18

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
GLEN CANYON DAM
PLAN AND MAXIMUM SECTION

DRAWN M.D.C. – S.R. SUBMITTED
TRACED F.E.S. RECOMMENDED
CHECKED [illegible] APPROVED
DENVER, COLORADO, MARCH 12, 1957 | 557-D-330

Figure 38.—Dam design A-18—Plan and maximum section.

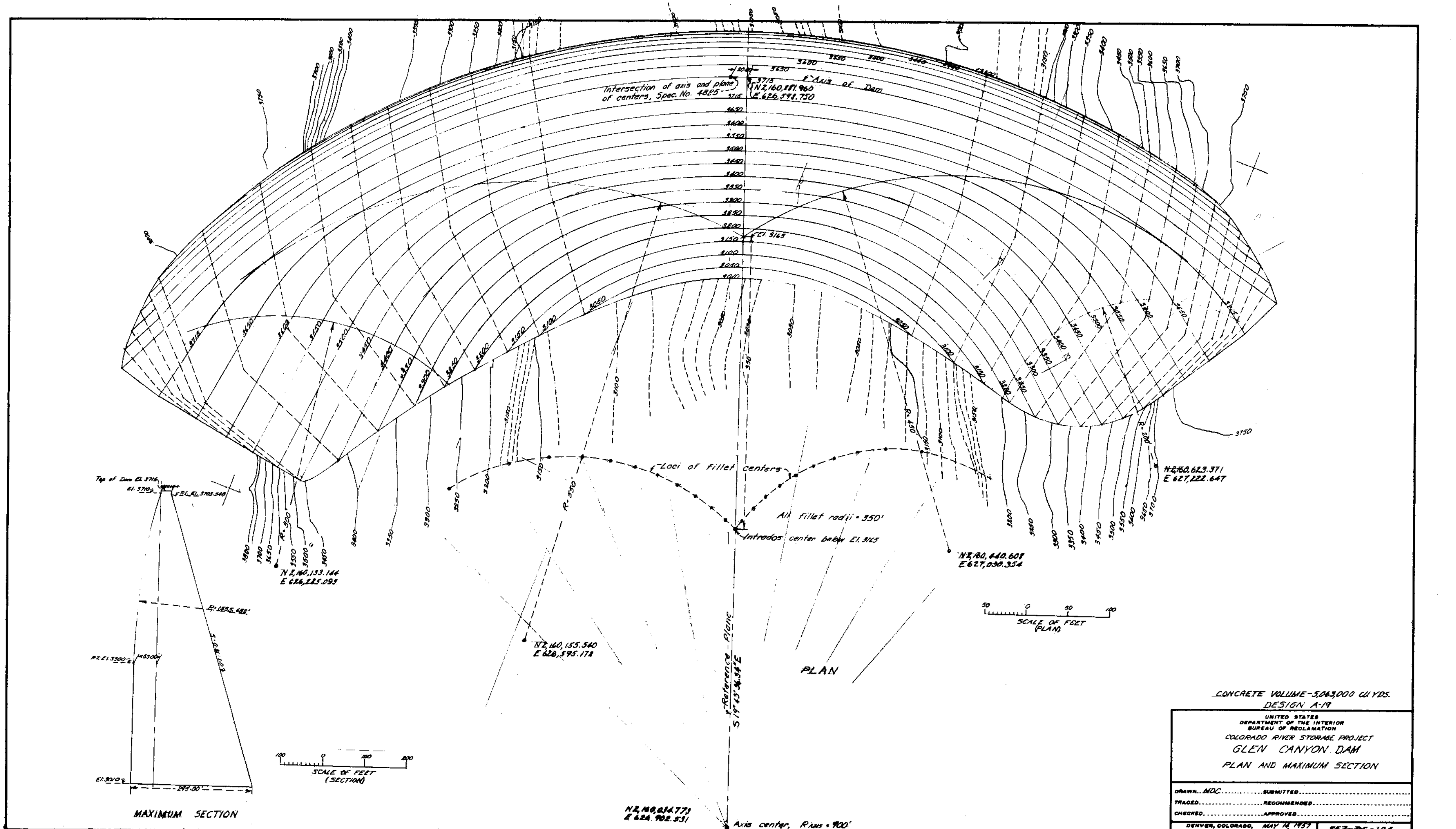
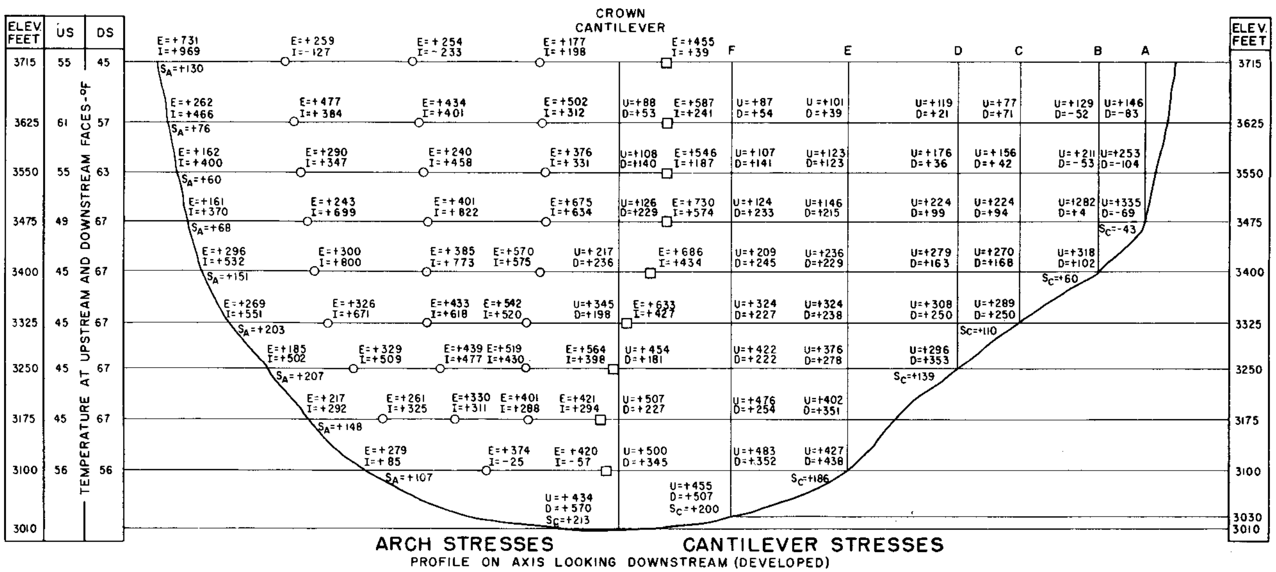


Figure 39.—Dam design A-19.—Plan and maximum section.



ASSUMPTIONS AND LOADING CONDITIONS

Top of dam, elevation 3715.
Reservoir water surface, elevation 3700.
Top of fill on downstream face, elevation 3158.
Tailwater surface, elevation 3142.
Temperatures used in analysis are changes between average arch temperatures at time of joint closure and minimum operating temperatures assumed to vary linearly from upstream to downstream faces.
Effects of construction and grouting program included as follows:

1. Concrete placed to elevation 3480; reservoir water surface, elevation 3240; joints ungrouted.
2. Concrete cooled to 40°F. and contraction joints grouted to elevation 3480; concrete placed to elevation 3715 and water surface raised to elevation 3490. Radial adjustment only.
3. Contraction joints grouted from elevation 3480 to 3715 after concrete has been cooled to temperatures varying from 40°F. at elevation 3480 to 50°F. at at elevation 3715; reservoir water surface raised to elevation 3700 and the effects of earthquake, earth embankment and tailwater included.

Earthquake assumptions:
Dam moves upstream and downstream horizontally in the direction of the plane of centers with an acceleration of 0.1 gravity and a period of vibration of 1.0 second.
Increased water pressure acts equally on all cantilevers
Effects of vertical acceleration not included.
Effects of silt and uplift not included.
Modulus of elasticity of concrete, 3,000,000 pounds per square inch.
Modulus of elasticity of foundation rock, 500,000 pounds per square inch.
Poisson's ratio of concrete, 0.20.
Poisson's ratio of foundation rock, 0.06.
Unit weight of concrete, 150 pounds per cubic foot.
Coefficient of thermal expansion of concrete, 0.0000056 per degree Fahrenheit.
All arches analyzed as symmetrical with symmetrical loading.
Arch stresses are acting in horizontal directions parallel to the edges of the arches.
Cantilever stresses are acting parallel to the edges of the cantilevers.

SYMBOLS

E = Stress at extrados of arch.
I = Stress at intrados of arch.
S_A = Maximum arch shear stress. (+) indicates downstream shear.
U = Stress at upstream edge of cantilever.
D = Stress at downstream edge of cantilever.
S_C = Maximum cantilever shear stress. (+) Indicates downstream shear.
□ Crown of arch.
○ Intermediate arch points.
All stresses are in pounds per square inch. (+) Compression; (-) Tension.

80 0 80 160
SCALE OF FEET

DESIGN A-19 (STUDY 1)

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COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV. GLEN CANYON UNIT-ARIZ-UTAH

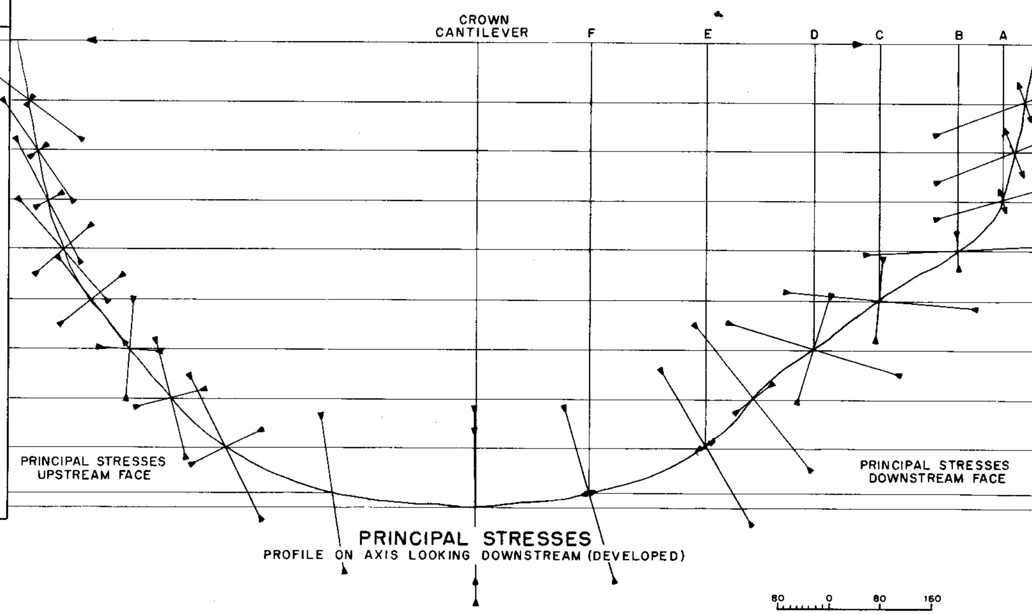
GLEN CANYON DAM
COMPLETE TRIAL LOAD ANALYSIS
ARCH AND CANTILEVER STRESSES

DRAWN P.C.V. SUBMITTED
TRACED R.D.B. RECOMMENDED
CHECKED C.W.J. APPROVED
DENVER, COLO., MAY 14, 1958 | 557-D-3226

Figure 40.—Dam design A-19 (study 1), complete trial-load analysis—Arch and cantiliver stresses.

Principal stresses σP_1	Principal stresses σP_2	Principal stresses α	Elev. feet	US	DS
0	+311	0°00'	3715	55	45
+36	+383	36°10'	3625	61	57
+66	+357	54°53'	3550	55	63
+93	+403	62°06'	3475	49	67
+220	+394	48°31'	3400	45	67
+238	+320	51°52'	3325	45	67
+296	+184	3°33'	3250	45	67
+208	+352	75°19'	3175	45	67
+233	+473	63°55'	3100	56	56
−10	+466	81°34'	3030		
0	+434	89°06'	3010		

TEMPERATURE AT UPSTREAM AND DOWNSTREAM FACES—°F



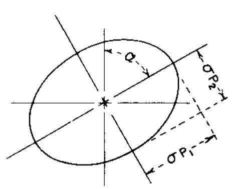
Elev. feet	Principal stresses σP_1	Principal stresses σP_2	Principal stresses α	Δ Max. shear
3715	0	+969	0°00'	+128
3625	−124	+546	20°18'	+69
3550	−153	+491	22°04'	+54
3475	−68	+406	16°02'	+67
3400	+112	+533	3°07'	+118
3325	+553	+248	85°04'	+201
3250	+518	+326	73°04'	+236
3175	+541	+136	38°27'	+255
3100	+525	−64	30°13'	+178
3030	+531	−43	15°54'	
3010	+570	0	0°23'	

PRINCIPAL STRESSES

PROFILE ON AXIS LOOKING DOWNSTREAM (DEVELOPED)

80 0 80 160

SCALE OF FEET



NOTES

α=Angle first principal stress (σP_1) makes with the vertical, positive angle measured in a clockwise direction on the left side of the dam, and in a counterclockwise direction on the right side of the dam.

σP_1=First principal stress.

σP_2=Second principal stress.

Δ=Maximum horizontal shear stress at rock abutment planes (+) Indicates downstream shear.

►—◄ Compression ; ◄—► Tension.

For constants; assumptions and loading conditions see Drawing 557-D-3226.

DESIGN A-19 (STUDY I)

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MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZ.-UTAH

GLEN CANYON DAM
COMPLETE TRIAL LOAD ANALYSIS
PRINCIPAL STRESSES

DRAWN P.C.V. SUBMITTED
TRACED R.D.B. RECOMMENDED
CHECKED C.W.J. APPROVED
DENVER, COLO., MAY 14, 1958 557-D-3225

Figure 41.—Dam design A-19 (study 1), complete trial-load analysis—Principal stresses.

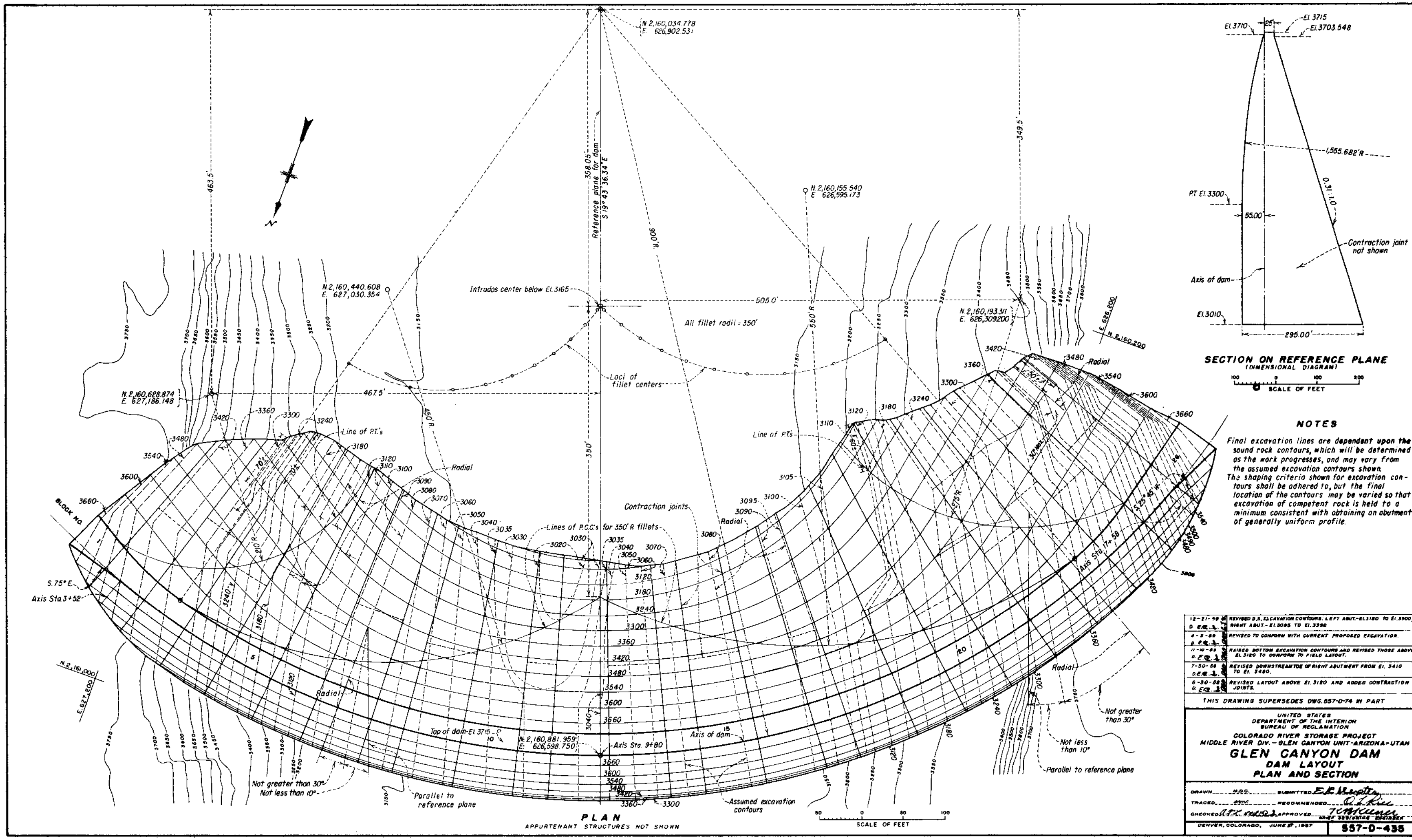


Figure 42.—Dam layout for study A-20—Plan and sections. This layout was based on final abutment excavations.

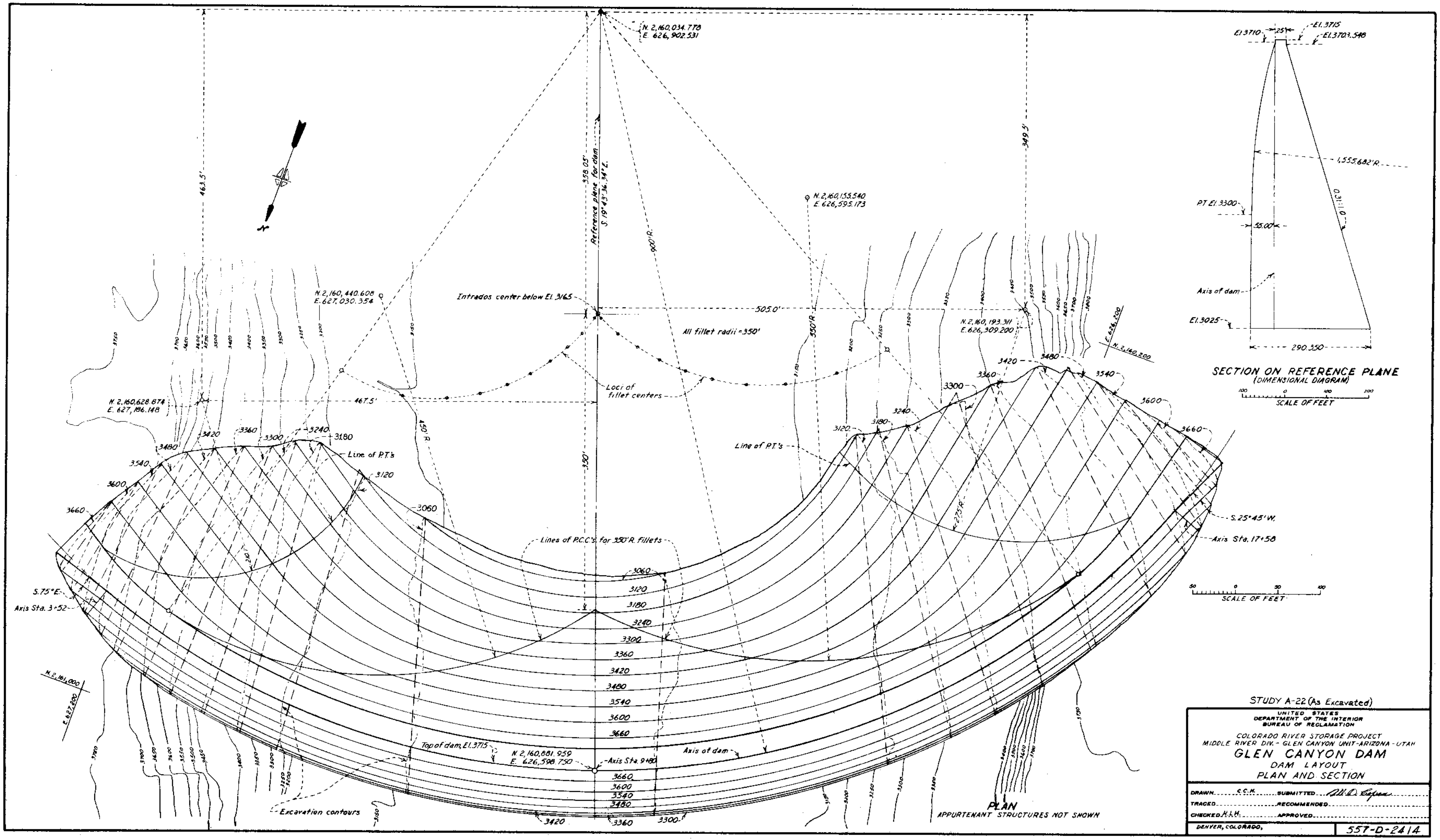


Figure 43.—Dam study A-22 (final)—Layout, plan, and section.

axis of the dam is 900 feet. Below elevation 3300, the upstream face is vertical and 55 feet upstream from the axis. Above elevation 3300, the upstream face curves in a downstream direction to meet the axis of the dam at elevation 3710. The radius of curvature, in vertical radial planes, for this portion of the upstream face is 1,555.68 feet. All horizontal arcs in the upstream face are concentric with the axis, thus forming an upstream surface with no overhangs or abruptly changing surfaces. The upper central portion of the downstream face is formed of horizontal arcs concentric with the axis of the dam and has a constant slope of 0.31 to 1.00, horizontal to vertical. To thicken the dam at the abutments, these concentric arcs are terminated at a line on the surface where short (350-foot) radius fillets begin. These fillets continue to points near the abutments, from which tangents extend to the rock. To further increase the abutment thicknesses at elevation 3540 and above, the upstream face near the abutments was directed slightly upstream. For analytical purposes and to approximate the abutment excavation as nearly as possible, triangular abutments were assumed as shown by the long dashed lines.

Four complete trial-load analyses were made with design A-22. The first study included the effects of earthquake, while the second study was made omitting the effects of earthquake. The third study was made to bring in the effects of a revised cooling program. The fourth and last study included the effects of the cooling program and a modified reservoir storage and contraction joint grouting program. These studies were made assuming the horizontal elements as being nonsymmetrical with nonsymmetrical loading.

The loading conditions and assumptions used in each study are listed on the drawing showing the resulting arch stresses. Figures 44 through 47 show the arch stresses parallel to the faces. Cantilever stresses parallel to the faces of the dam are shown on figures 48 through 51. Principal stresses at the abutments are shown on figures 52 through 59.

In the modified cooling and grouting program in study A-22c, it was assumed that water in the reservoir would rise to elevation 3490 before grouting could be performed above elevation 3480. This specification was written in anticipation of normal runoff. However, in the years 1962 and 1963 the runoff was subnormal. In April 1963, the storage forecast for Lake Powell predicted the reservoir to be near elevation 3420 by late summer 1963. Since the contractor would be ready to proceed with grouting above elevation 3480 before another runoff, the decision as to whether to stop the grouting program or to continue when the reservoir water surface rose to elevation 3420 had to be made. Study A-22d was initiated to help in making this decision. Since grouting had been completed to elevation 3480, temperatures of the concrete at the time of grouting used in the analysis are the recorded ones below elevation 3480 and the anticipated ones above that elevation.

Arch and principal stresses at the extrados and intrados of the abutments and their variations from 600 pounds per square inch are shown graphically on figures 60 and 61. As can be seen from these figures, stresses at the abutments exceeding 600 pounds per square inch are limited to local areas. The arch elements as analyzed and the stresses normal to their abutments are shown on figure 62. The average stresses at the abutments are shown by dashed lines on the stress diagrams.

Summarized in the table below are the maximum stresses resulting from the loading conditions used in the complete trial-load analyses of design A-22. Stresses developed in design study A-22d were computed for full reservoir, minimum usual temperatures in the dam, and the effects of horizontal earthquake. Since these stresses are reasonable, the design is considered adequate.

Type of stress	Study a	Study b	Study c	Study d
Principal stress at abutments	622	570	623	645
Arch stress	975	859	868	777
Cantilever stress	562	516	598	564
Rock plane shear	373	242	298	287
Arch shear	316	255	273	268
Cantilever shear	333	169	237	231

2. Pertinent Design Details

31. COOLING OF MASS CONCRETE. The principal temperature control measures adopted for Glen Canyon Dam were: (1) Precooling of the concrete mix materials to obtain a maximum 50° F. placing temperature; and (2) postcooling of the concrete in the dam to minimize the temperature rise after placement and to obtain the desired concrete temperatures prior to grouting the contraction joints. Other measures included use of type II cement, reduction of cement content, and use of a pozzolan.

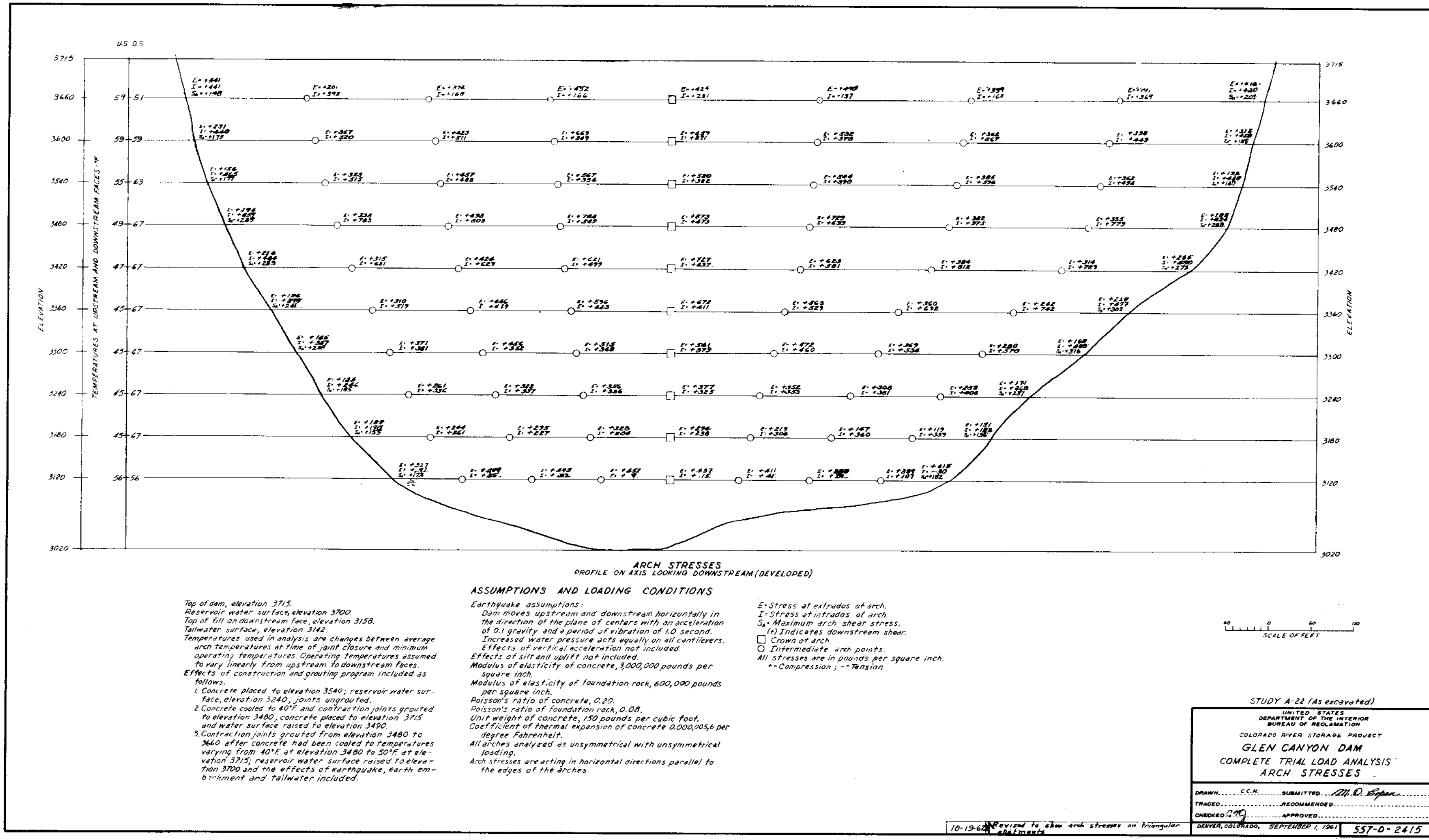


Figure 44.—Dam study A-22a (final), complete trial-load analysis—Arch stresses.

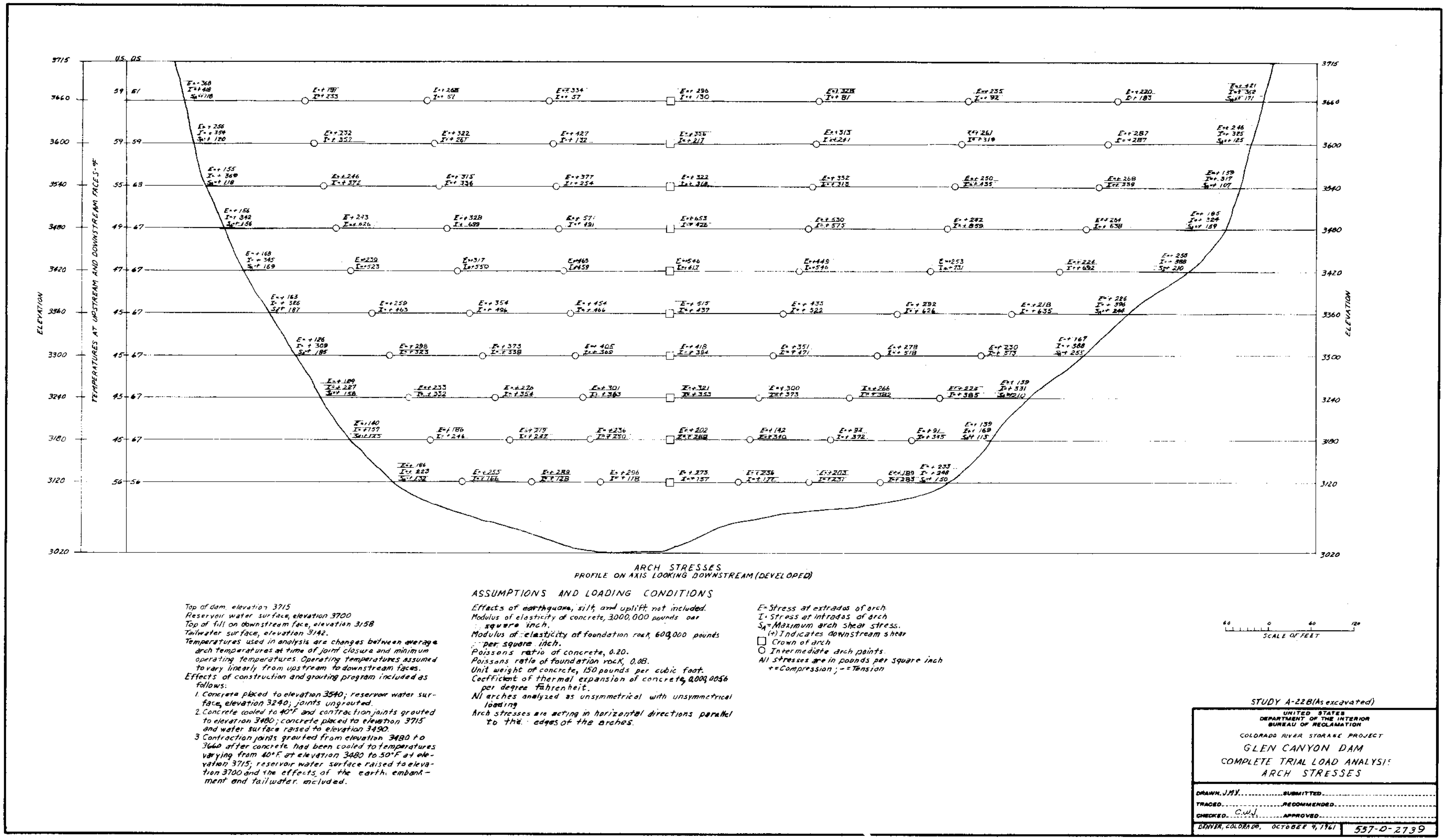


Figure 45.—Dam study A-22b (final), complete trial-load analysis—Arch stresses.

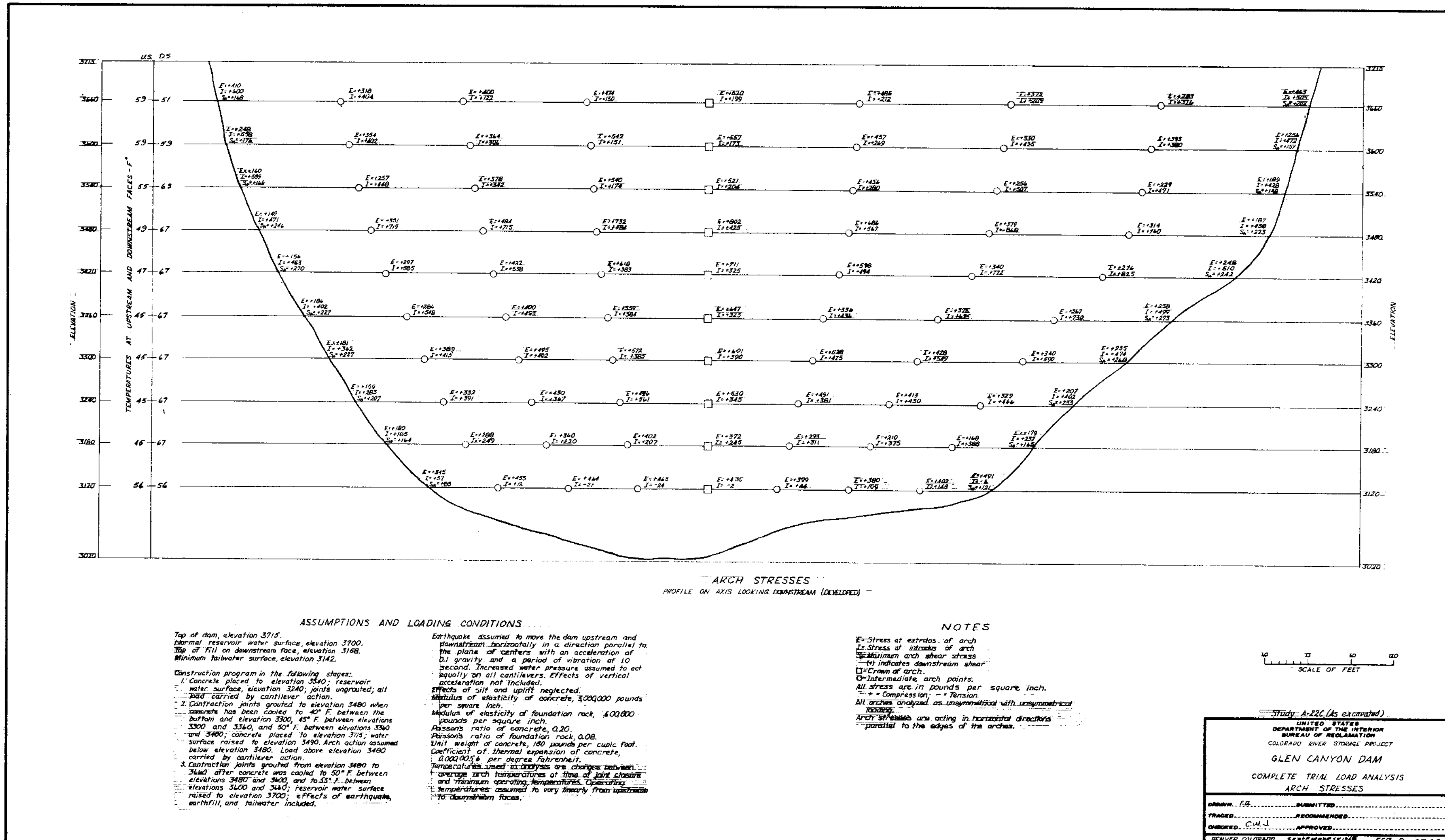
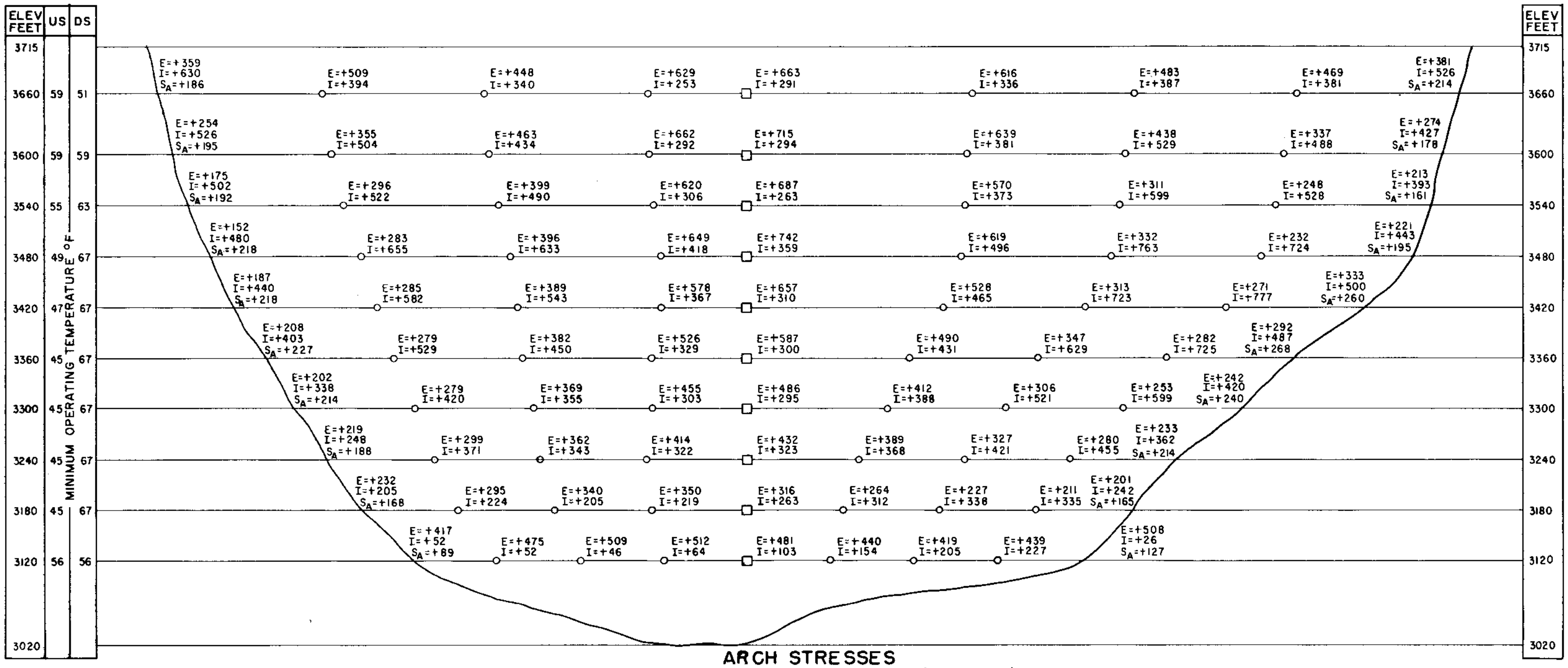


Figure 46.—Dam study A-22c (final), complete trial-load analysis—Arch stresses.



ARCH STRESSES

PROFILE ON AXIS LOOKING DOWNSTREAM (DEVELOPED)

CONSTANTS, ASSUMPTIONS AND LOADING CONDITIONS

Top of dam, elevation 3715.
Normal reservoir water surface, elevation 3700.
Top of fill on downstream face, elevation 3158.
Minimum tailwater surface, elevation 3142.
Earthquake assumed to move the dam upstream and downstream horizontally in a direction parallel to the plane of centers with an acceleration of 0.1 gravity and a period of vibration of 1.0 second. Increased water pressure assumed to act equally on all cantilevers. Effects of vertical acceleration not included.
Effects of silt and uplift neglected.
Sustained modulus of elasticity of concrete, 3,000,000 pounds per square inch.
Sustained modulus of elasticity of foundation rock, 600,000 pounds per square inch.
Poisson's ratio of concrete, 0.20.
Poisson's ratio of foundation rock, 0.08.
Coefficient of thermal expansion of concrete, 0.0000056 per degree Fahrenheit.
Temperatures used in analysis are differences between average arch temperatures at time of joint closure and minimum operating temperatures. Operating temperatures assumed to vary linearly from upstream to downstream faces.
Unit weight of concrete, 150 pounds per cubic foot.
Construction grouting program assumed as follows:

1. Concrete placed to elevation 3300. Reservoir water surface elevation 3150.
2. Dam grouted to elevation 3300 when temperature of concrete in dam varies from 36°F. at elevation 3060 to 42°F. at elevation 3300. Concrete placed to elevation 3480. Reservoir water surface remains at elevation 3150.
3. Dam grouted to elevation 3480 when temperatures of concrete in dam are at 45°F. between elevations 3300 and 3360 and at 50°F. between elevations 3360 and 3480. Concrete is placed to elevation 3715. Water in the reservoir raised from elevation 3150 to elevation 3420.
4. Dam is grouted to elevation 3660 when concrete in dam is cooled to 50°F. between elevations 3480 and 3600, and to 55°F between elevations 3600 and 3660. Reservoir water surface raised to elevation 3700. The effects of earthquake, earthfill and tailwater are included.

NOTES

E= Stress at extrados of arch.
I= Stress at intrados of arch.
S_A=Maximum arch shear stress. (+) Indicates downstream shear.
□ Crown of arch.
○ Intermediate arch points.
All stresses are in pounds per square inch. (+) Compression; (-) Tension.
All arches analyzed as unsymmetrical with unsymmetrical loading.
Arch stresses are acting in horizontal directions parallel to the edges of the arches.

60 0 60 120
SCALE OF FEET

STUDY A-22d (MODIFIED GROUTING PROGRAM)

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MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZ.-UTAH

GLEN CANYON DAM
COMPLETE TRIAL LOAD ANALYSIS
ARCH STRESSES

DRAWN H.B.-P.L. SUBMITTED
TRACED R.D.B. RECOMMENDED
CHECKED C.W.J. APPROVED
DENVER, COLO., JULY 12, 1963 | 557-DC-161

Figure 47.—Dam study A-22d (final), complete trial-load analysis—Arch stresses.

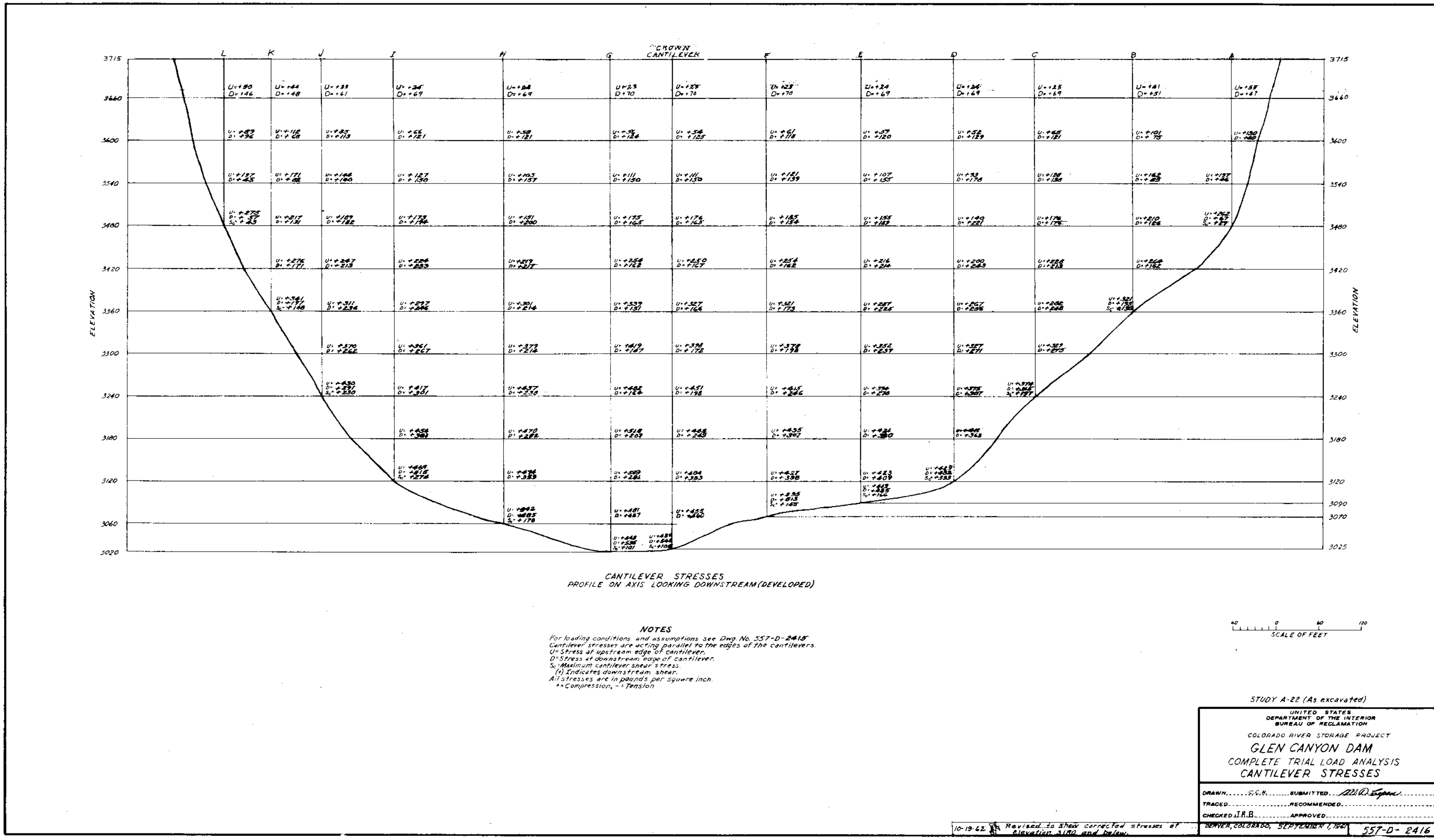


Figure 48.—Dam study A-22a (final), complete trial-load analysis—Cantilever stresses.

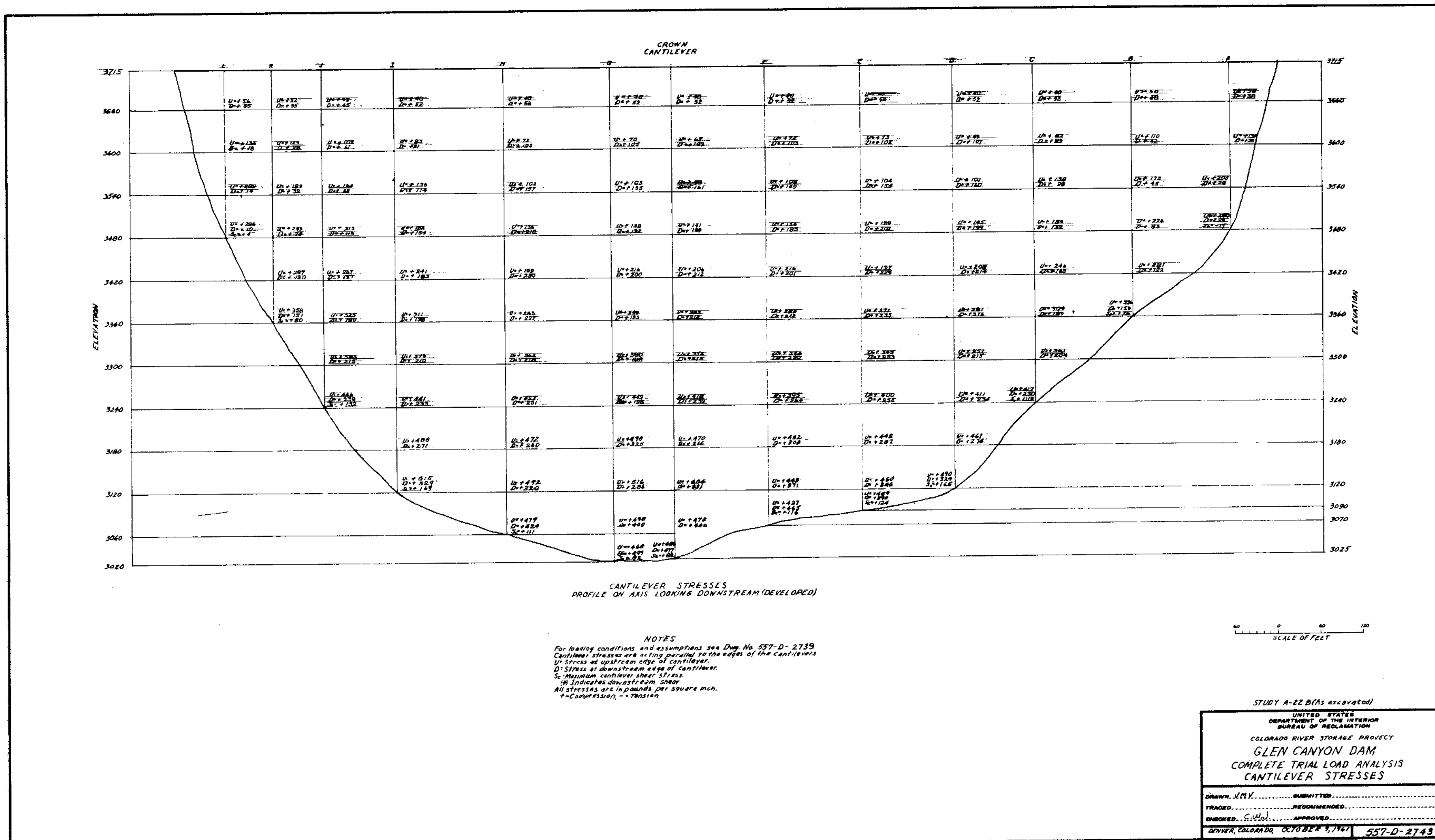


Figure 49.—Dam study A-22b (final), complete trial-load analysis—Cantilever stresses.

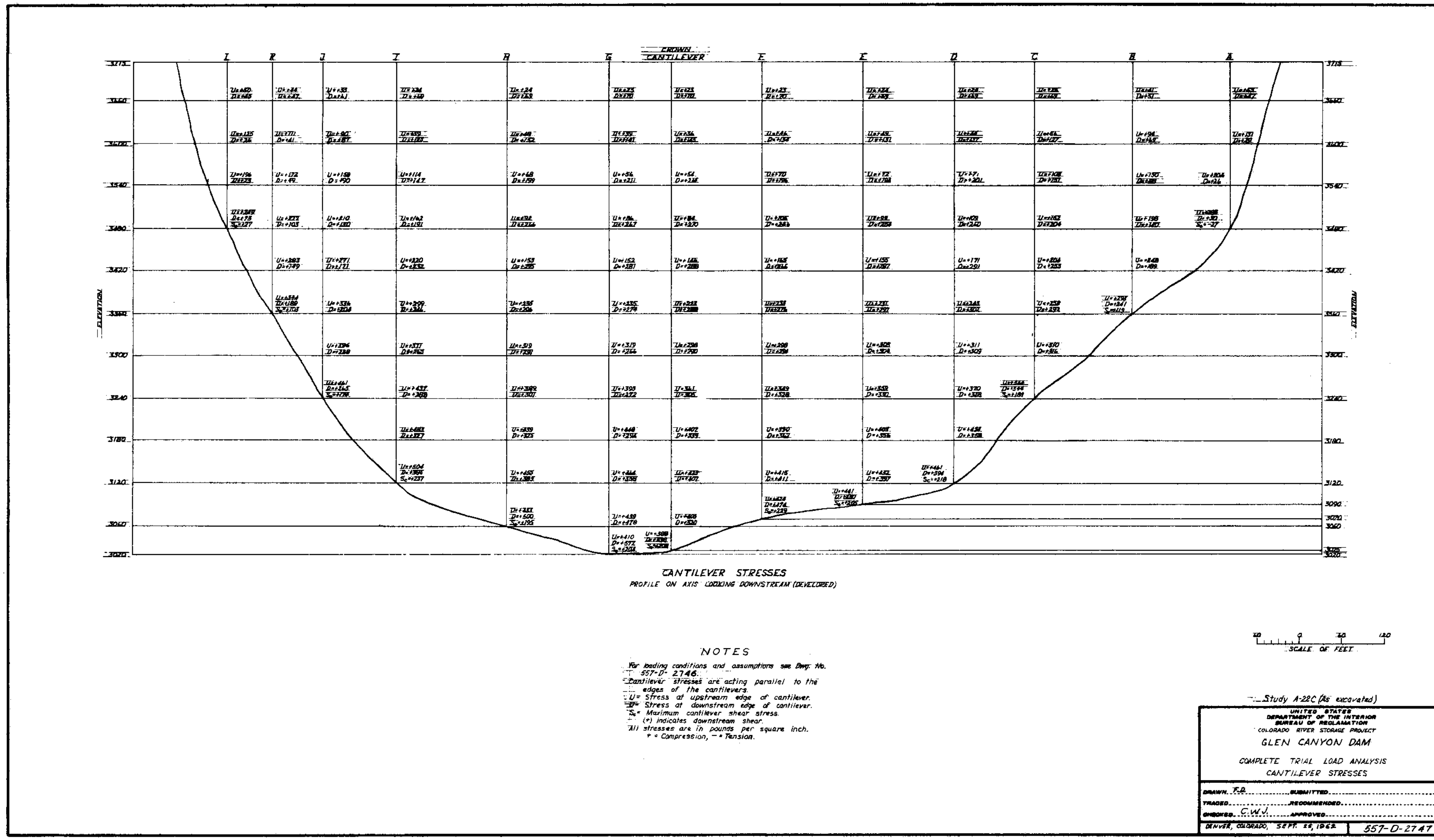
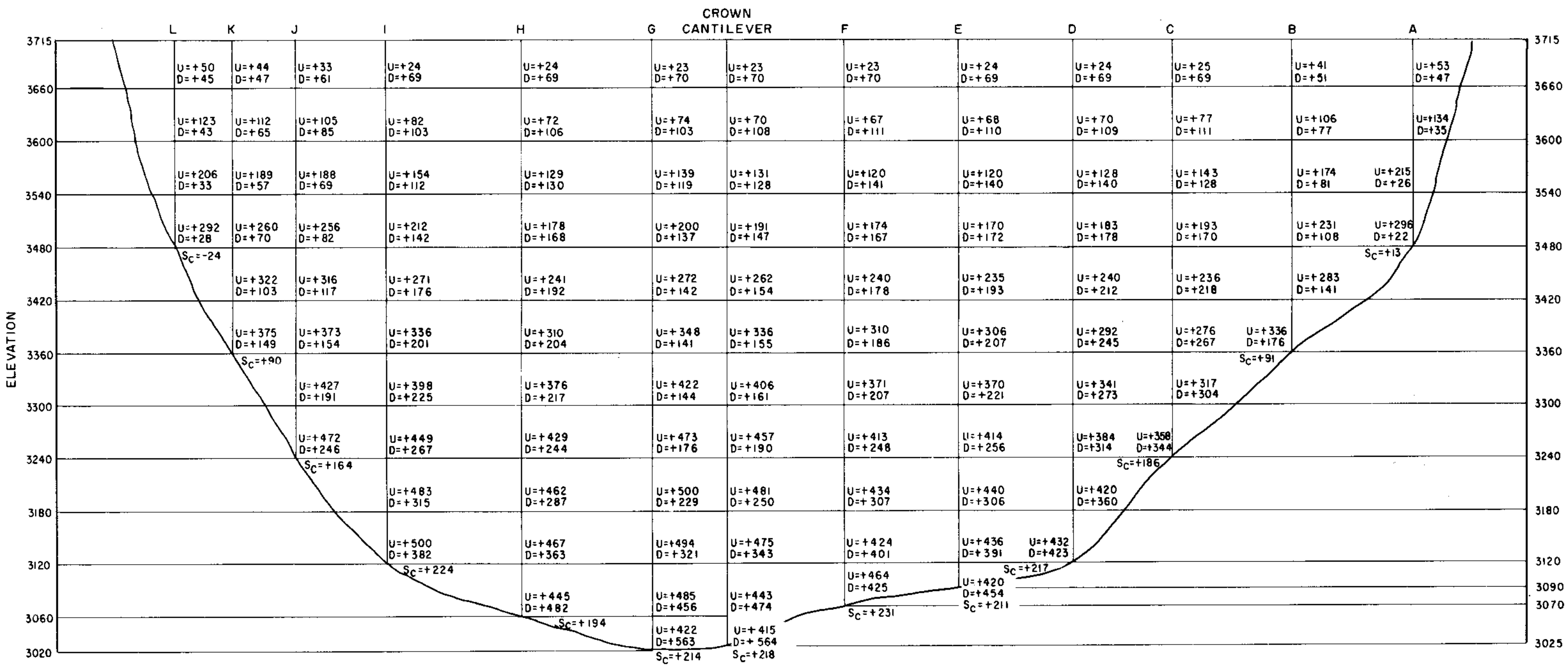


Figure 50.—Dam study A-22c (as excavated), complete trial-load analysis—Cantilever stresses.



CANTILEVER STRESSES

PROFILE ON AXIS LOOKING DOWNSTREAM (DEVELOPED)

NOTES

For constants, assumptions and loading conditions see Drawing 557-DC-161.
Cantilever stresses are acting parallel to the edges of the cantilevers.
U = Stress at upstream edge of cantilever.
D = Stress at downstream edge of cantilever.
S_c = Maximum cantilever shear stress.
(+) Indicates downstream shear.
All stresses are in pounds per square inch.
(+) Compression; (-) Tension

60 0 60 120
SCALE OF FEET

STUDY A-22d (MODIFIED GROUTING PROGRAM)

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MIDDLE RIVER DIV-GLEN CANYON UNIT-ARIZ-UTAH

GLEN CANYON DAM

COMPLETE TRIAL LOAD ANALYSIS
CANTILEVER STRESSES

DRAWN H.B.-P.L. SUBMITTED
TRACED R.D.B. RECOMMENDED
CHECKED C.W.J. APPROVED

DENVER, COLO., JULY 12, 1963 | 557-DC-162

Figure 51.—Dam study A-22d (final), complete trial-load analysis—Cantilever stresses.

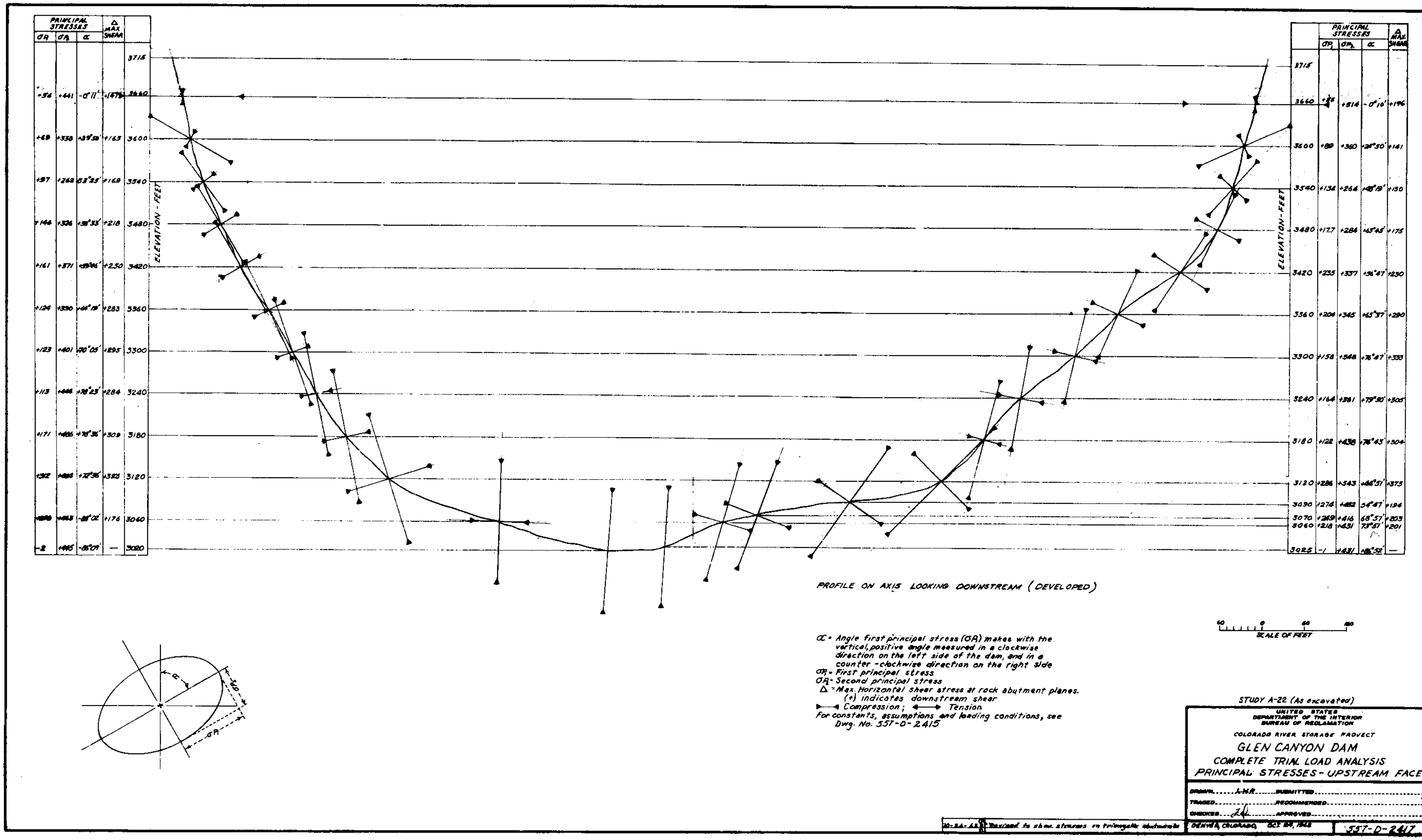
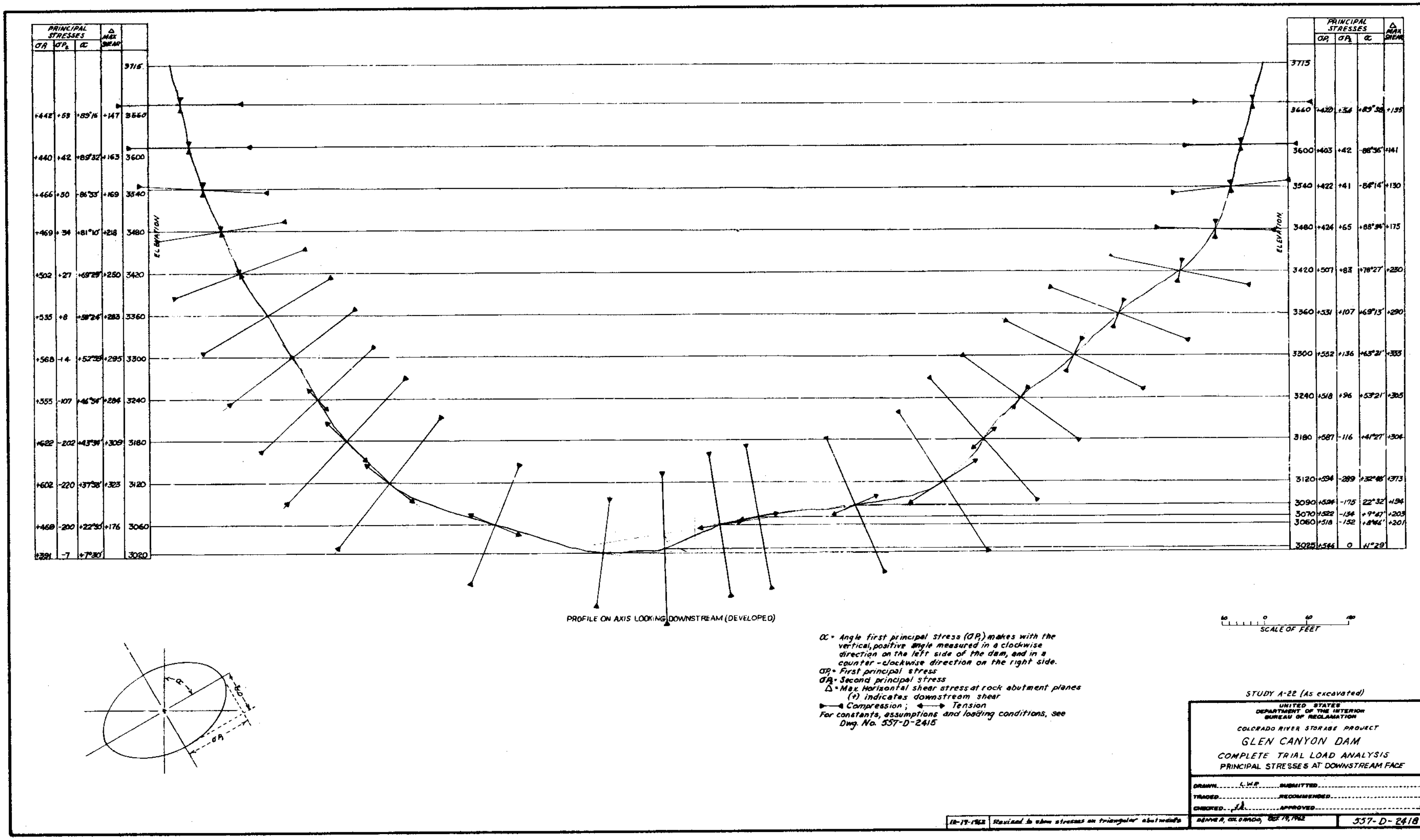


Figure 52.—Dam study A-22a (final), complete trial-load analysis—Principal stresses at upstream face.



Principal stresses σP1	Principal stresses σP2	α	Δ Max shear	Elevation
				3716
+442	+53	+89°16'	+147	3660
+440	+42	+89°32'	+163	3600
+466	+50	-86°33'	+169	3540
+469	+34	+81°10'	+218	3480
+502	+27	+69°29'	+250	3420
+535	+8	+58°24'	+283	3360
+568	-14	+52°39'	+295	3300
+555	-107	+46°54'	+284	3240
+622	-202	+43°34'	+309	3180
+602	-220	+37°38'	+323	3120
+468	-200	+22°30'	+176	3060
+391	-7	+7°30'		3020

Elevation	Principal stresses σP1	Principal stresses σP2	α	Δ Max shear
3715				
3660	+420	+54	+89°38'	+135
3600	+403	+42	-88°36'	+141
3540	+422	+41	-84°14'	+130
3480	+424	+65	+88°34'	+175
3420	+507	+83	+78°27'	+230
3360	+531	+107	+69°13'	+290
3300	+552	+136	+63°21'	+333
3240	+518	+96	+53°21'	+305
3180	+587	-116	+41°27'	+304
3120	+594	-289	+32°48'	+373
3090	+524	-175	22°32'	+194
3070	+522	-134	+9°47'	+203
3060	+518	-152	+8°46'	+201
3025	+546	0	11°29'	

Figure 53.—Dam study A-22a (final), complete trial-load analysis—Principal stresses at downstream face.

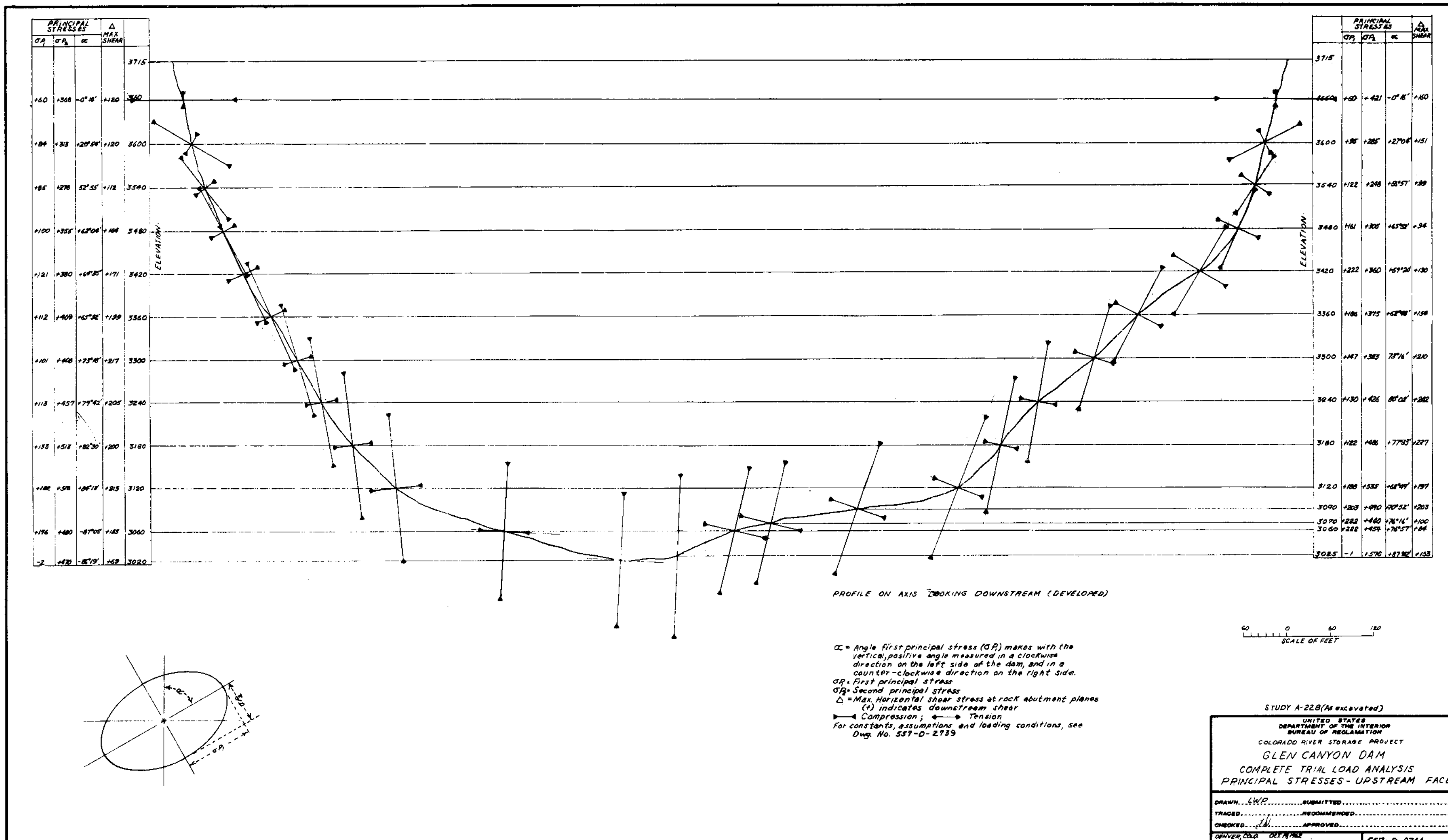


Figure 54.—Dam study A-22b (final), complete trial-load analysis—Principal stresses at upstream face.

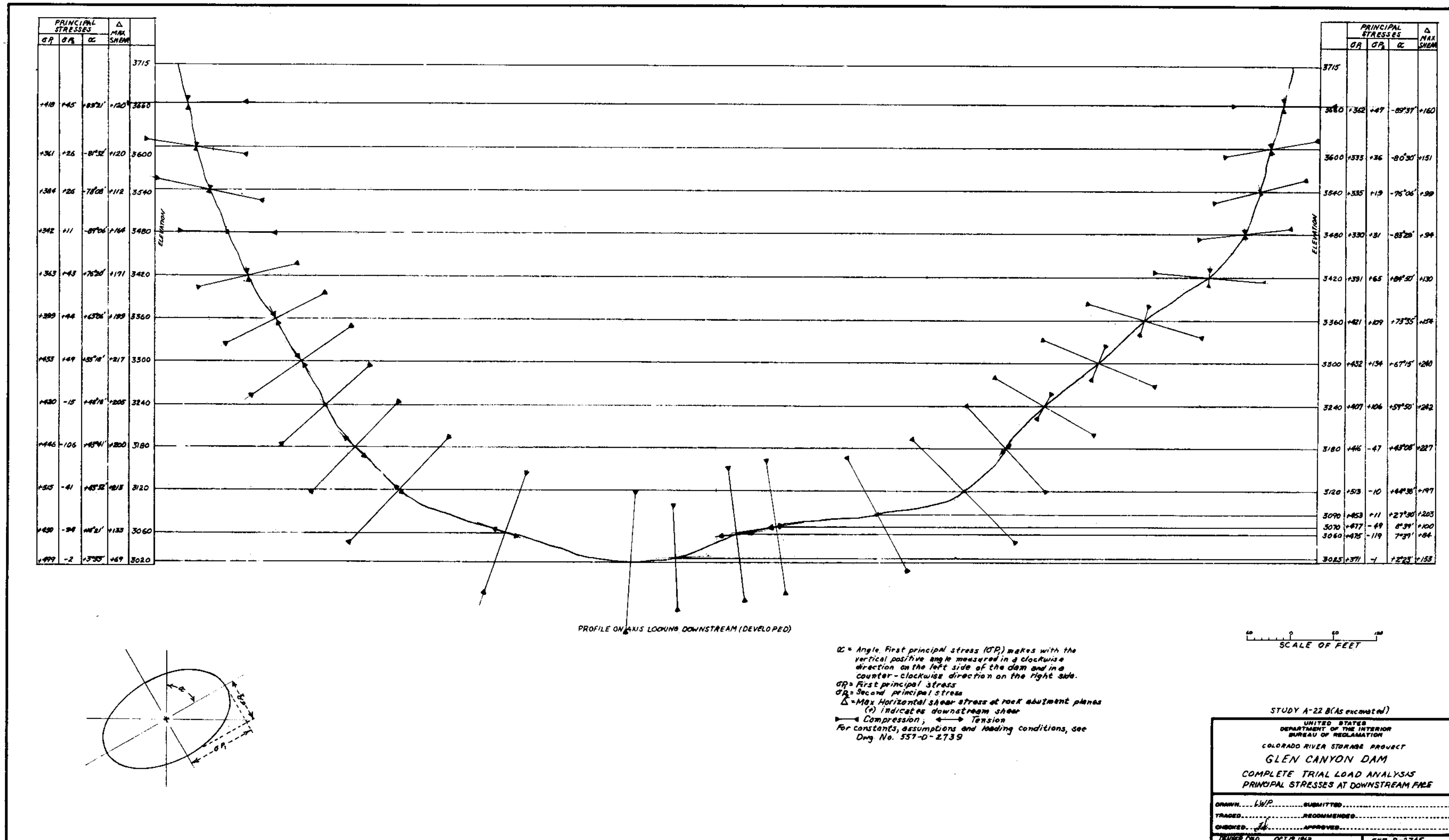


Figure 55.—Dam study A-22b (final), complete trial-load analysis—Principal stresses at downstream face.

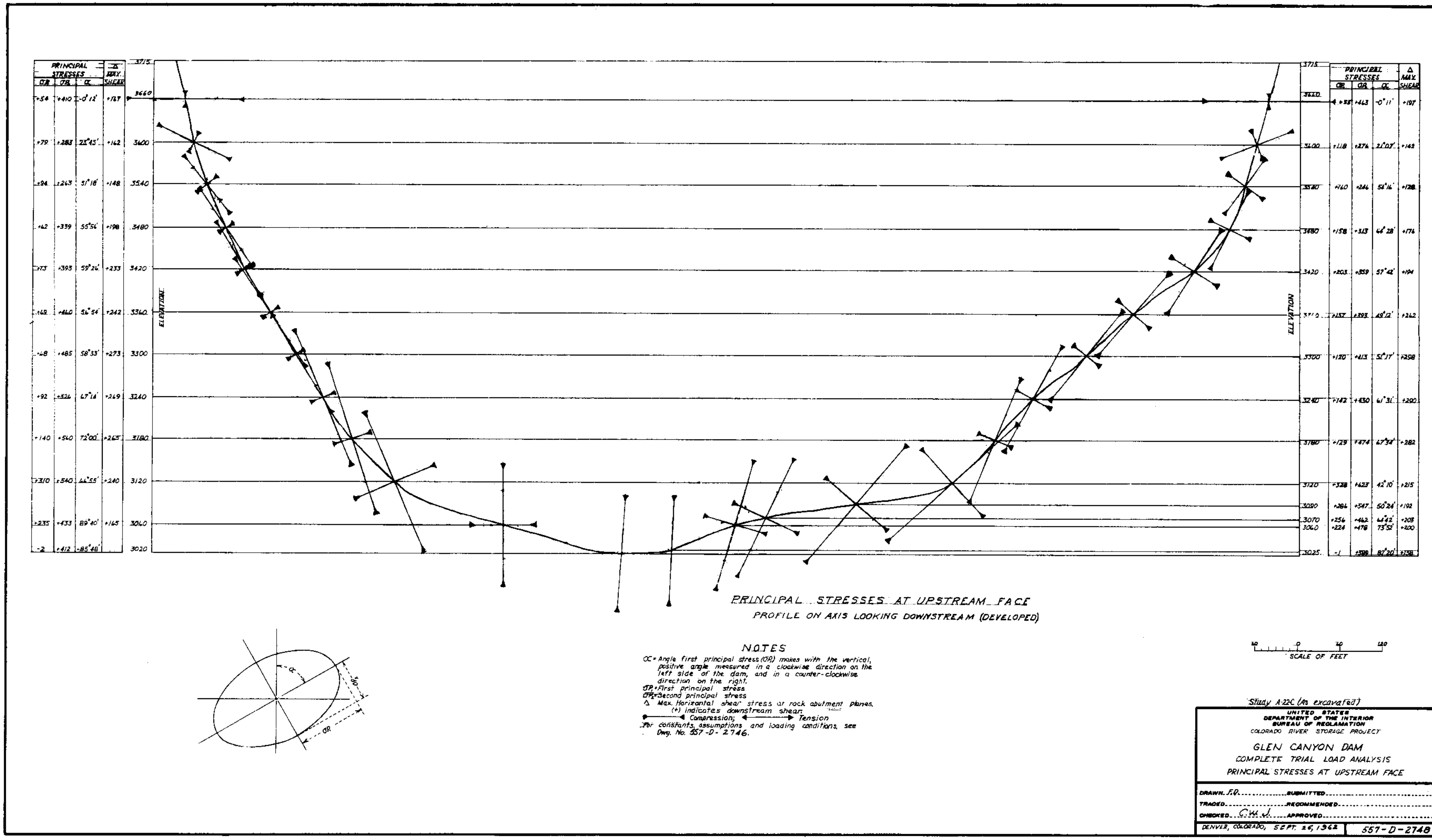


Figure 56.—Dam study A-22c (final), complete trial-load analysis—Principal stresses at upstream face.

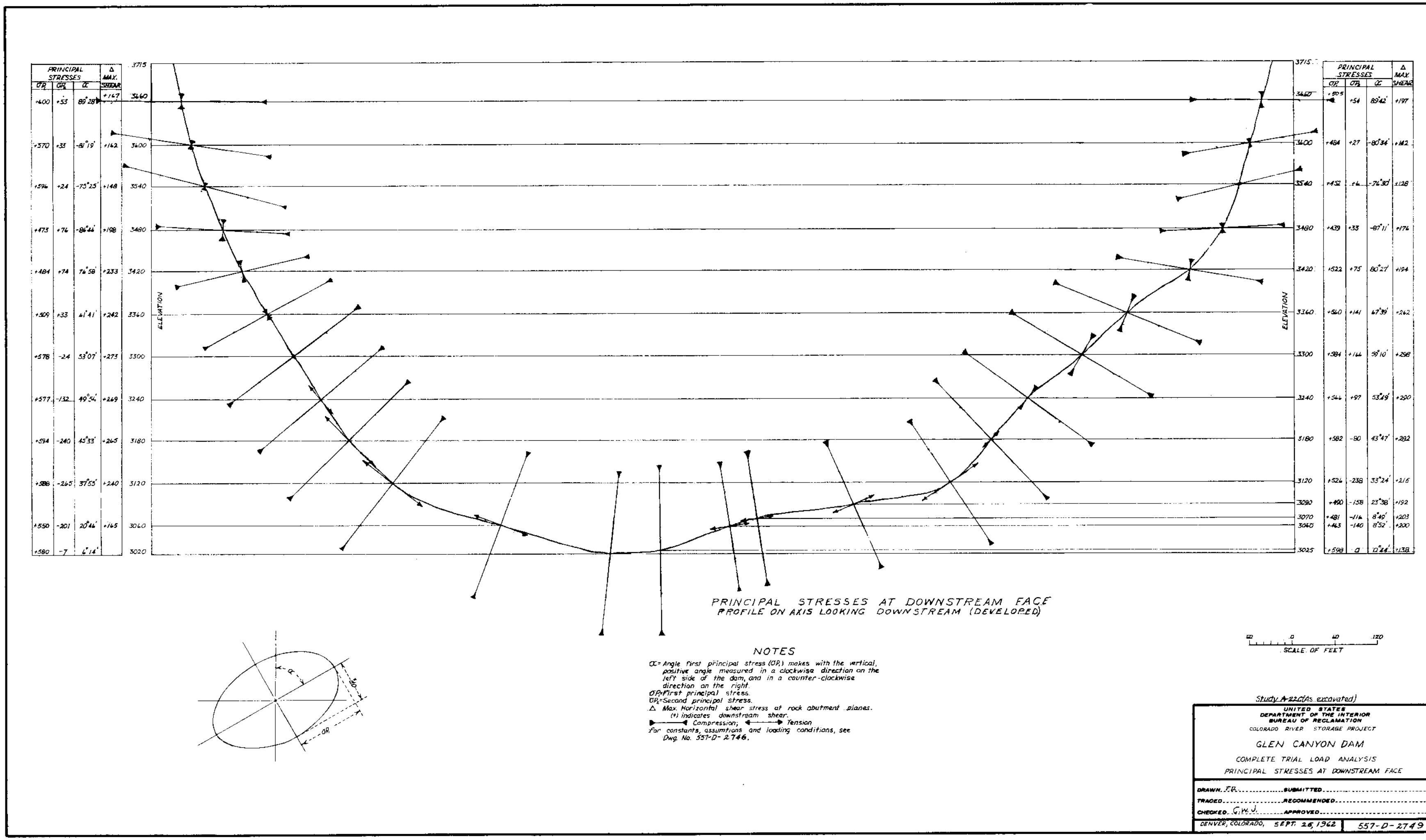
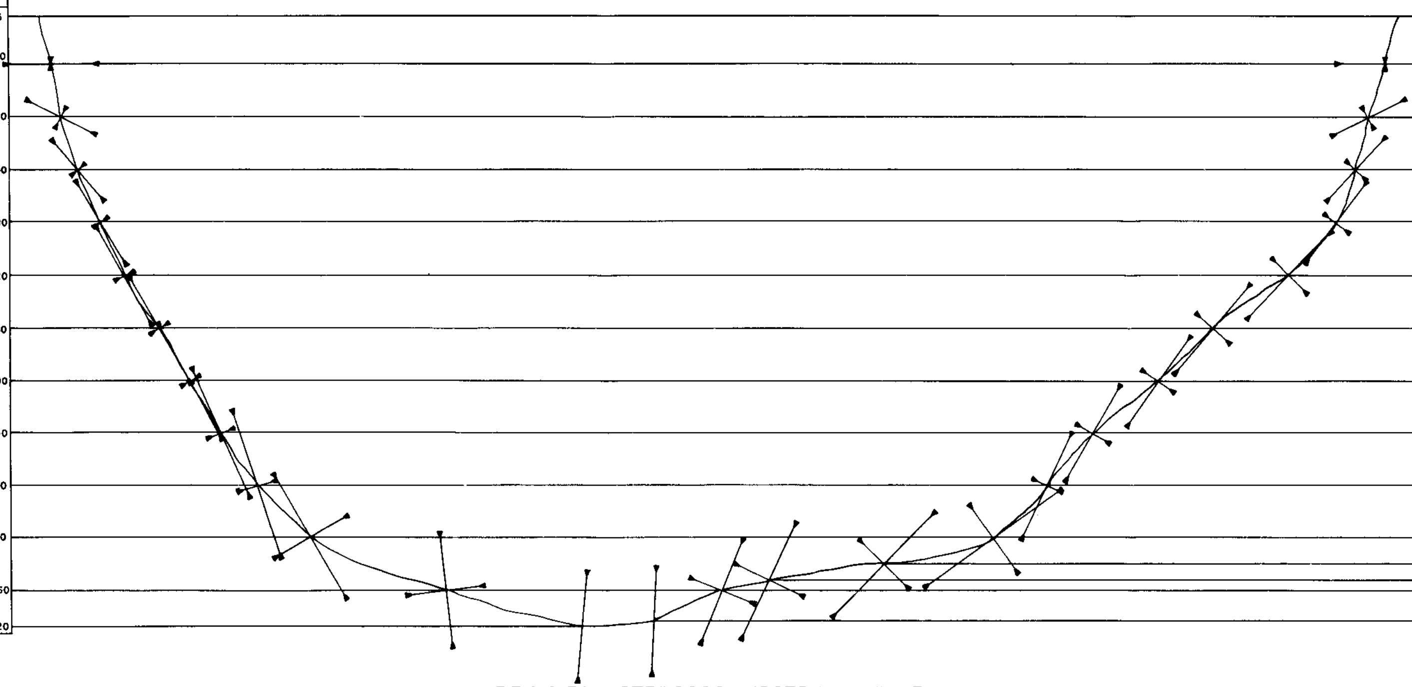


Figure 57.—Dam study A-22c (final), complete trial-load analysis—Principal stresses at downstream face.

Left abutment

PRINCIPAL STRESSES σ_{P_1}	PRINCIPAL STRESSES σ_{P_2}	PRINCIPAL STRESSES α	Δ MAX SHEAR	ELEV.
				3715
+54	+359	-0°48'	+185	3660
+90	+295	+26°37'	+180	3600
+69	+317	+49°11'	+174	3540
+70	+377	+58°53'	+191	3480
+73	+436	+66°55'	+204	3420
+82	+477	+59°30'	+239	3360
+85	+514	+61°34'	+250	3300
+112	+547	+65°34'	+248	3240
+155	+673	+71°51'	+244	3180
+330	+557	+59°52'	+225	3120
+302	+447	+83°07'	+186	3060
−3	+425	+95°04'		3020

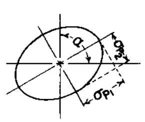


Right abutment

ELEV.	PRINCIPAL STRESSES σ_{P_1}	PRINCIPAL STRESSES σ_{P_2}	PRINCIPAL STRESSES α	Δ MAX SHEAR
3715				
3660	+55	+381	-0°50'	+209
3600	+99	+316	+25°58'	+163
3540	+108	+337	+47°00'	+145
3480	+123	+397	+53°13'	+166
3420	+193	+473	+45°21'	+195
3360	+174	+454	+49°34'	+236
3300	+154	+427	+54°51'	+250
3240	+151	+426	+60°08'	+282
3180	+129	+454	+64°42'	+287
3120	+324	+645	+35°26'	+218
3090	+273	+563	+45°24'	
3070	+298	+501	+64°38'	
3060	+272	+436	+67°44'	+226
3025	−1	+416	+87°38'	

PRINCIPAL STRESSES - UPSTREAM FACE

PROFILE ON AXIS LOOKING DOWNSTREAM (DEVELOPED)



NOTES

α = Angle first principal stress (σ_{P_1}) makes with the vertical, positive angle measured in a clockwise direction on the left side of the dam, and in a counterclockwise direction on the right side of the dam.

σ_{P_1} = First principal stress.

σ_{P_2} = Second principal stress.

Δ = Maximum horizontal shear stress at rock abutment planes. (+) Indicates downstream shear.

►—◄ Compression ; ◄—► Tension.

For constants, assumptions and loading conditions see Drawing 557-DC-161.

60 0 60 120
SCALE OF FEET

STUDY A-22d (MODIFIED GROUTING PROGRAM)

ALWAYS THINK SAFETY

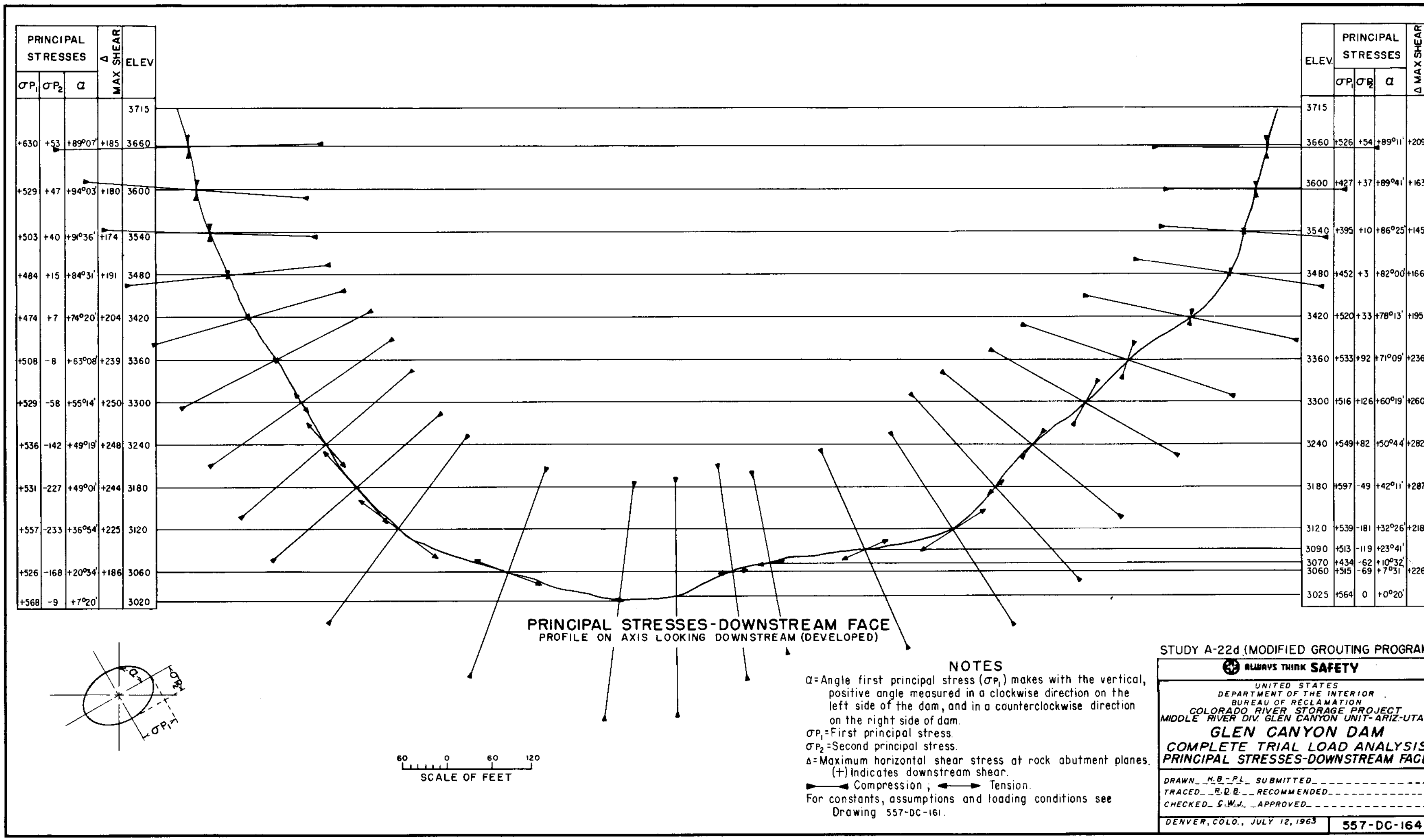
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZ.-UTAH

GLEN CANYON DAM

COMPLETE TRIAL LOAD ANALYSIS
PRINCIPAL STRESSES-UPSTREAM FACE

DRAWN H.B.-P.L. SUBMITTED
TRACED C.D.P. RECOMMENDED
CHECKED C.W.J. APPROVED
DENVER, COLO., JULY 12, 1963 — 557-DC-163

Figure 58.—Dam study A-22d (final), complete trial-load analysis—Principal stresses at upstream face.



Principal stresses σP1	Principal stresses σP2	Principal stresses α	Δ Max shear	Elev
				3715
+630	+53	+89°07'	+185	3660
+529	+47	+94°03'	+180	3600
+503	+40	+91°36'	+174	3540
+484	+15	+84°31'	+191	3480
+474	+7	+74°20'	+204	3420
+508	-8	+63°08'	+239	3360
+529	-58	+55°14'	+250	3300
+536	-142	+49°19'	+248	3240
+531	-227	+49°01'	+244	3180
+557	-233	+36°54'	+225	3120
+526	-168	+20°34'	+186	3060
+568	-9	+7°20'		3020

Elev.	Principal stresses σP1	Principal stresses σP2	Principal stresses α	Δ Max shear
3715				
3660	+526	+54	+89°11'	+209
3600	+427	+37	+89°41'	+163
3540	+395	+10	+86°25'	+145
3480	+452	+3	+82°00'	+166
3420	+520	+33	+78°13'	+195
3360	+533	+92	+71°09'	+236
3300	+516	+126	+60°19'	+260
3240	+549	+82	+50°44'	+282
3180	+597	-49	+42°11'	+287
3120	+539	-181	+32°26'	+218
3090	+513	-119	+23°41'	
3070	+434	-62	+10°32'	
3060	+515	-69	+7°31'	+226
3025	+564	0	+0°20'	

Figure 59.—Dam study A-22d (final), complete trial-load analysis—Principal stresses at downstream face.

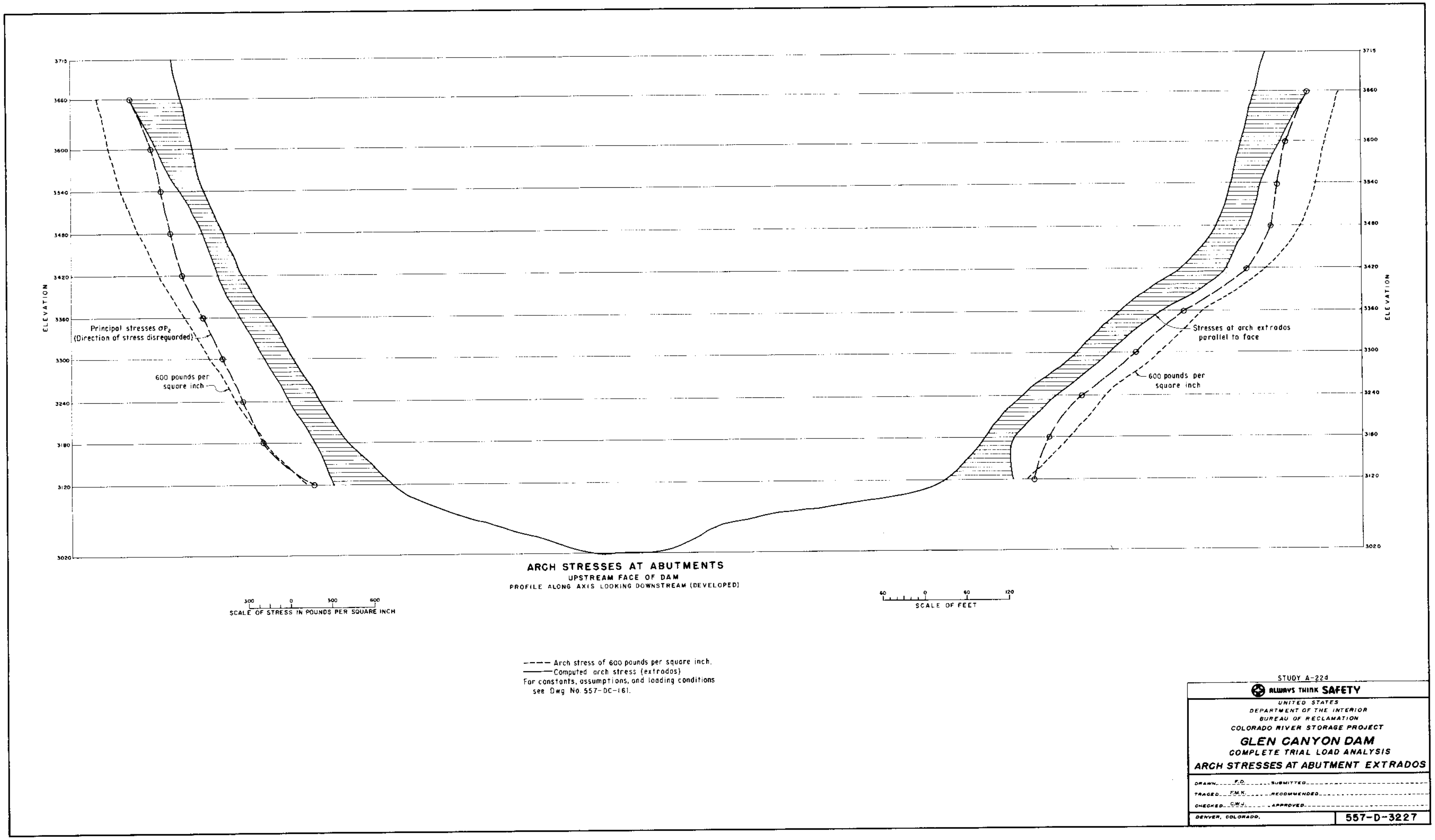


Figure 60.—Dam study A-22d (final), complete trial-load analysis—Arch stresses at abutment extrados.

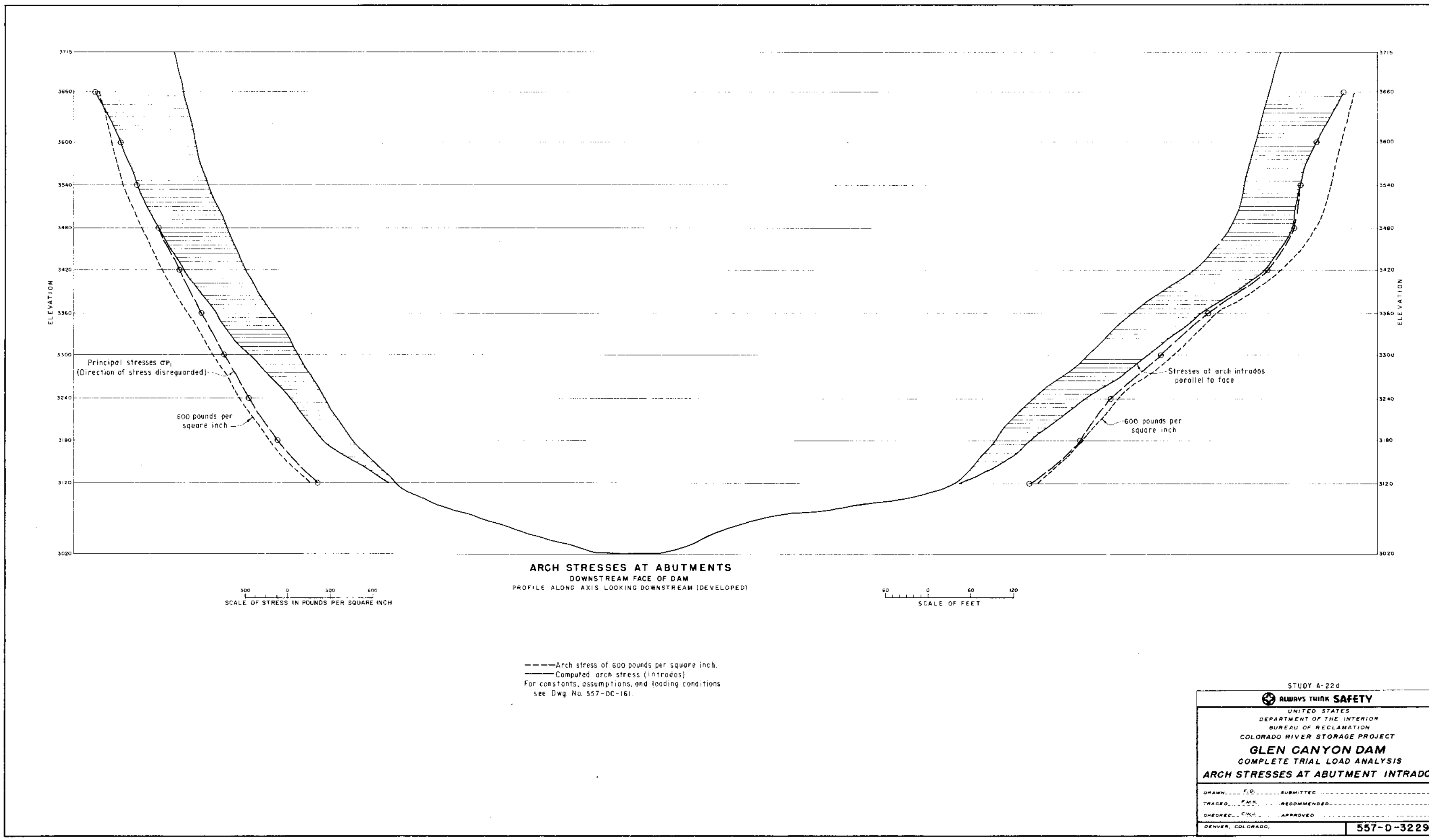


Figure 61.—Dam study A-22d (final), complete trial-load analysis—Arch stresses at abutment intrados.

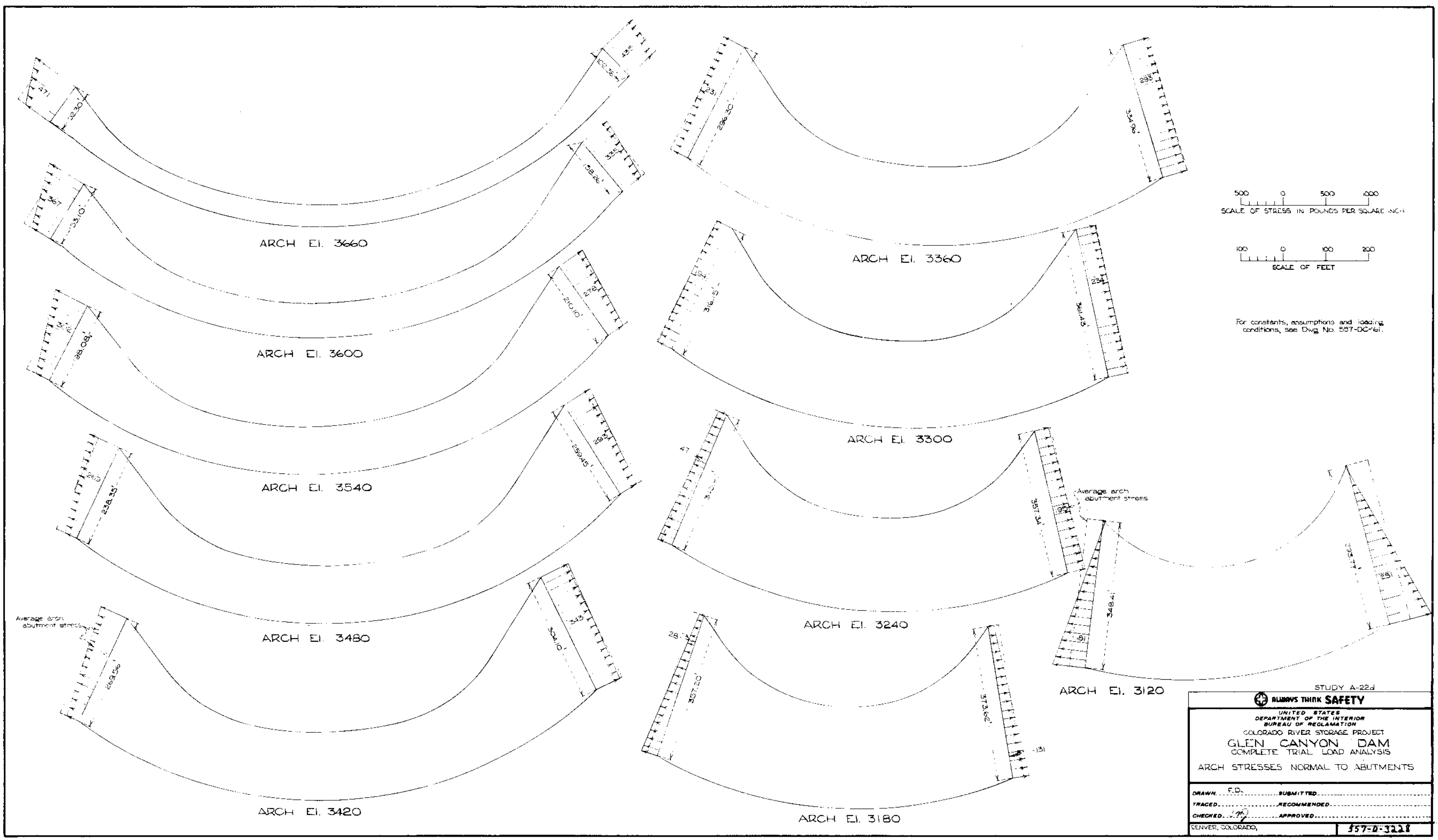


Figure 62.—Dam study A-22d (final), complete trial-load analysis—Arch stresses normal to abutments.

Precooling to obtain a maximum 50° F. placing temperature was required because of the size of blocks to be placed in the dam and because of the relatively high placing temperatures which would normally occur at the site without such measures. Blocks as large as 70 by 190 feet were to be in contact with the foundation. Larger blocks than this, up to 191.28 feet on one side and 210.88 feet on the other side, occurred at the termination of several longitudinal joints in the dam but were located where the foundation restraint was materially reduced. Large blocks such as these would crack severely unless the temperature drop from the maximum concrete temperature to the grouting temperature was controlled to a maximum of about 35° F. This would be possible only if the maximum placing temperature was held to 50° F. or less. Without this restriction, placing temperatures could be as high as 80° F. in the summer with a resulting temperature drop of about 70° F.

Postcooling was performed by circulating cold water through embedded cooling coils placed on the top of each 7-1/2-foot construction lift in the dam. Artificial cooling of the concrete was primarily required because Glen Canyon Dam was designed as an arch dam with both the transverse and longitudinal contraction joints grouted ahead of the rising reservoir. Only by postcooling could the concrete be cooled and grouted in the relatively short construction period. The cooling systems further controlled the temperature rise so that the peak temperatures obtained would be appreciably lower than those which would occur without pipe cooling. Details of the concrete cooling systems are shown on figure 63. Temperature studies using the adiabatic temperature rise obtained from laboratory studies[3] indicated that the temperature rise could be limited to 25° F. for blocks adjacent to rock. This meant that the maximum temperature drop would be limited to 35° F. for those areas where restraint was high.

The specifications requirement for the operation of the cooling systems provided for a 12-day initial cooling period for each 7-1/2-foot placement lift to be followed by final cooling of each 60-foot grouting lift to its final temperature. The final cooling period was estimated to take about 52 days. Refrigerated water was required for all final cooling because river water temperatures were not sufficiently low to accomplish the desired cooling. Temperatures at the time of grouting were originally to be 40° F. in the lower part of the dam, ranging from 40° F. at elevation 3450 to 50° F. at the top of the dam. Later analyses, however, showed that higher closure temperatures could be permitted without significantly changing the stresses in the dam. Final cooling was then directed to be 40° F. up to elevation 3300, 45° F. between elevations 3300 and 3360, 50° F. between elevations 3360 and 3600, and 55° F. above elevation 3600.

Longitudinal joints were placed in all blocks except the end blocks of the dam, and were terminated before they reached the downstream face of the dam. So that these joints would not continue to the face, the specifications required that all concrete in the upstream and downstream blocks separated by such longitudinal joints be cooled to 50° F. prior to placing concrete above the termination of the joint. Reinforcement was also placed over the top of the joints to minimize any cracking tendency to the face.

The specifications provided for artificial cooling of backfill and tunnel plug concrete in the two diversion tunnels and in the mass concrete beneath the machine shop. In the diversion tunnels, such cooling was necessary so that periphery grouting of the backfill and plug concrete could be accomplished as soon as possible after placement of the concrete. Cooling of the mass concrete beneath the machine shop was deemed necessary because of the dimensions of the mass and the required location of control joints. The 60° F. temperature in both places was the estimated final stable-state temperature of the adjacent rock.

32. CONTRACTION JOINTS. Glen Canyon Dam was constructed in blocks which were separated from other blocks by transverse and longitudinal contraction joints. The purpose of the block construction was to confine the volumetric shrinkage cracks to predetermined planes throughout the mass concrete, which cracks could be grouted to form a monolithic structure after full volumetric shrinkage was obtained.

The spacing of the transverse contraction joints was governed primarily by the location of penstock and river outlet pipes through the dam. This resulted in 40- and 70-foot spacings of the transverse joints in the central portion of the dam. Sixty-foot spacings were selected for the joints near the abutments to obtain block proportions which would facilitate concrete placement. One longitudinal joint was placed in each block except the end blocks. These longitudinal joints

[3]Concrete Laboratory Report No. C-526A, "Preliminary Laboratory Concrete Mix Investigations," Glen Canyon Dam, Glen Canyon Unit—Middle River Division—Colorado River Storage Project, July 28, 1958. (Unpublished.)

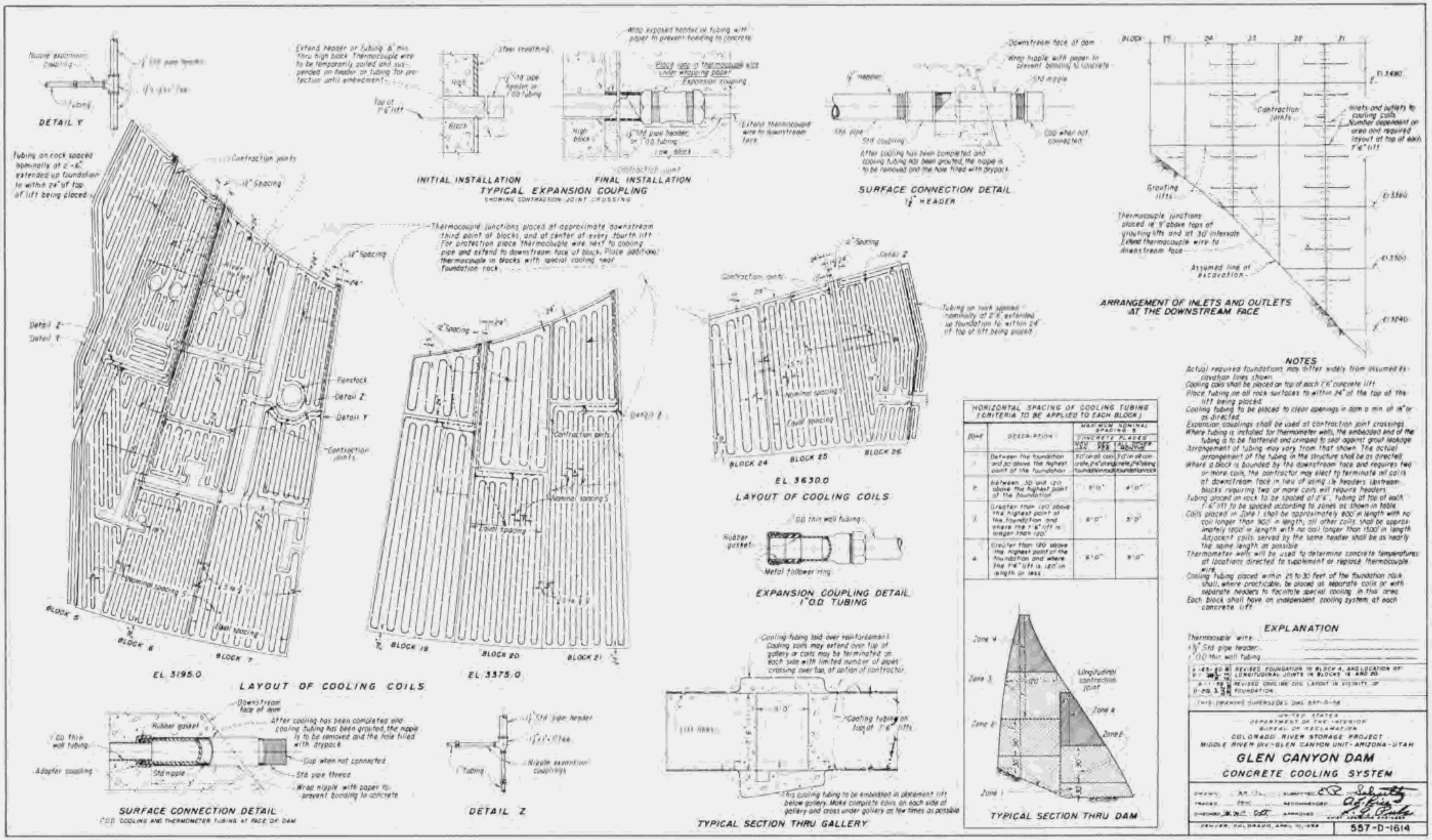


Figure 63.—Concrete cooling system in dam.

were deemed necessary because of the thickness of the dam, this thickness being 295 feet at the base at the reference plane. The longitudinal joints were staggered across adjacent rows so as not to form a continuous joint throughout the length of the dam. In the central portion of the dam, longitudinal joints were located 75 feet and 106.5 feet downstream of the axis of the dam, resulting in maximum 130- and 161.5-foot lengths of upstream blocks, since the axis of the dam was 55 feet downstream of the vertical part of the upstream face. Near the abutments, the upstream blocks were made 151 and 179.5 feet long in order to better balance the length of the upstream and downstream blocks. The layout of the joints is shown on figure 64.

The transverse contraction joints were keyed vertically to resist horizontal shear, and the longitudinal contraction joints were keyed horizontally to resist vertical shears. Grouting systems with grout outlets on the joints were embedded in the concrete to grout the joints after all excess heat had been removed from the concrete. Figures 65 and 66 show some details of these grouting systems.

Tests of the foundation rock at the damsite indicated that the sandstone in the abutments would undergo deformation which was not entirely elastic. In order to compress the sandstone and obtain elastic properties in the rock, provisions were made to apply water at a pressure of about 200 pounds per square inch in the end contraction joints to obtain the expected inelastic "set." Double metal seals were installed completely around each grouting lift in the end joints to permit the application of water under this higher than normal pressure. This pressure would be held for only a short time, and then would be released and the joints drained of water. Grouting of the joints would then be accomplished through the Bureau's normal grouting system with normal grouting pressures. Owing to the inelastic behavior of the abutment rock, subsequent filling and lowering of the reservoir could conceivably result in opening up of the end contraction joint at each end of the dam. Since the standard grouting systems are filled with grout, they are not available for any further grouting operations. Systems of reinjectable grout valves were therefore installed in these end joints in addition to the normal systems. The Capitaine reinjectable valve from France was selected as being the best available valve at the time for this purpose. For details of the reinjectable grouting system see figure 67.

33. GALLERIES, ADITS, AND CHAMBERS. (a) *General.*—The galleries and adits serve as access to the interior of the dam for inspection of the dam behavior and to service the dam after construction. The isometric drawings, figures 68 and 69, show the location of all galleries and adits in the dam. All the galleries in the dam are 5 by 7.5 feet except the grouting adits which are 6 by 7.5 feet, the adits to the powerplant which are 7 feet 8 inches by 8 feet 7 inches, and electrical service adits which are 4 feet by 7 feet 6 inches. The vertical stairwells are 6 feet in diameter. All longitudinal galleries were laid out on chords between contraction joints. Inclined galleries with a slope of 7-1/2 to 10 and vertical stairwells are provided with metal stairs. Vertical stairwells were limited to about 45 feet where possible, with frequent landings provided on the spiral stairs. Gutters, 12 inches wide and of varying depth, are located in all galleries and adits where the possibility of drainage is expected. Ventilation of the gallery system is accomplished by means of fans located in various areas of the dam. All galleries are located a minimum distance from the upstream face of the dam equal to 10 percent of the hydraulic head at the gallery floor.

(b) *Foundation Gallery.*—The centerline of the foundation gallery is located 14.5 feet downstream of the axis of the dam. It approximates the profile of the foundation as closely as possible. A 5-foot minimum clearance between the gallery and the excavated surface was established. The primary function of the gallery was to provide an area from which the main grout curtain ("A" holes) figure 14, were drilled and grouted and from which the upstream drainage curtain holes were drilled. This gallery also provides access to the foundation tunnels and to the plumbline well reading stations.

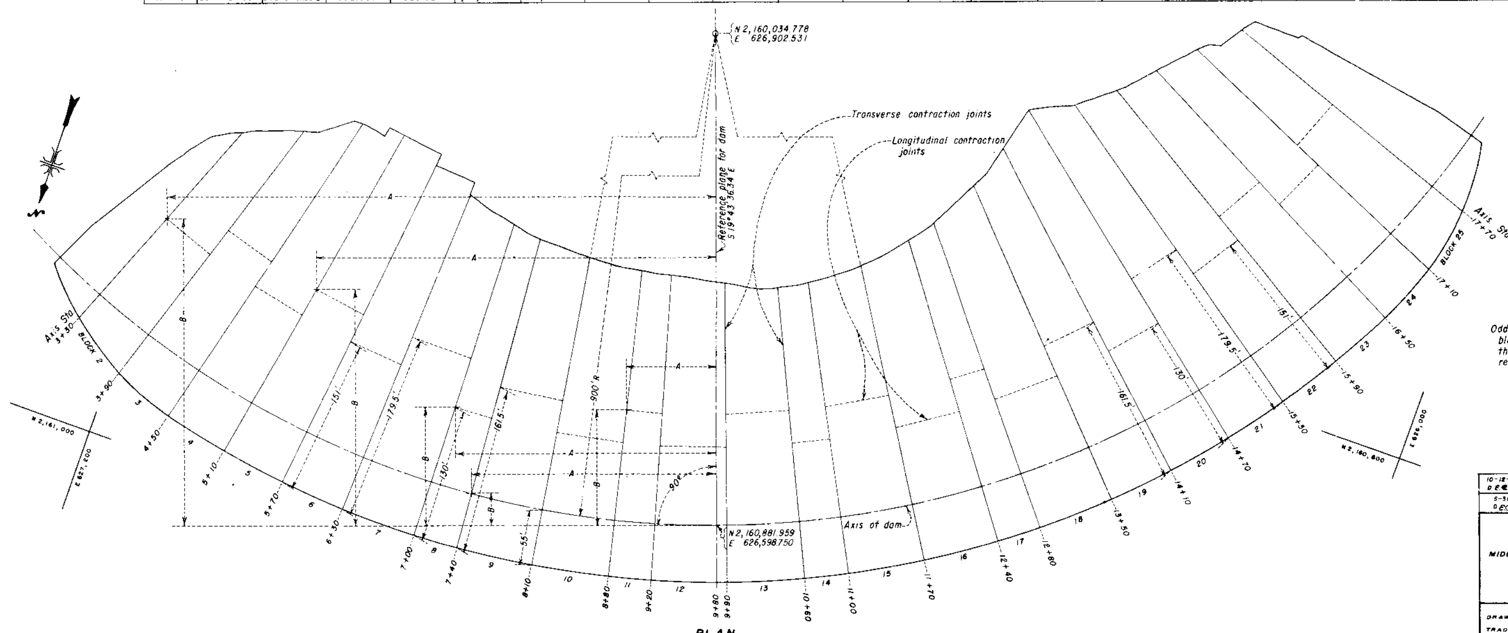
(c) *Drainage Gallery.*—The centerline of the drainage gallery is located 147.5 feet downstream of the axis of the dam. It also approximates the profile of the abutments and foundation as closely as possible. The 5-foot minimum clearance between the gallery and the excavated surface also was used. The gallery originally terminated at elevation 3187.50 but was extended to elevation 3427.50 on each side to connect the grouting adits for ease in movement of materials and equipment for grouting the abutments. The gallery collects drainage water and by means of gutters and piping carries the water to the sump in the foundation gallery.

(d) *Pump Chamber and Pump Chamber Gallery.*—The pump chamber gallery is located at elevation 3187.50, 24.5 feet downstream of the axis of the dam. It connects the powerplant with the adits in blocks 7 and 18. It provides access to the sump pump

AXIS STATION	DEFLECTION OF TRANS. JOINTS FROM REFERENCE PLANE	BEARING TRANS. JOINTS	INTERSECTION—TRANS. JOINTS AND AXIS OF DAM			
			A	B	COORDINATES N	COORDINATES E
3+30	41°22'49.03"	S21°39'12.69"W	594.948	224.695	2,160,871.267	627,234.625
3+90	37°33'38.04"	17°50'01.70"	548.640	186.562	2,160,891.532	627,178.162
4+50	33°44'27.05"	14°00'50.71"	499.894	151.597	2,160,907.990	627,120.476
5+10	29°55'16.07"	10°11'39.73"	448.927	119.958	2,160,920.569	627,061.820
5+70	26°06'05.08"	6°22'28.74"	395.965	91.785	2,160,929.213	627,002.457
6+30	22°16'54.09"	2°33'17.75"	341.244	67.202	2,160,933.883	626,942.651
7+00	17°49'31.27"	S1°54'05.06"E	275.505	43.205	2,160,934.282	626,872.669
7+40	15°16'43.95"	4°26'52.39"	237.166	31.811	2,160,932.067	626,832.734
8+10	10°49'21.13"	8°54'15.21"	168.991	16.008	2,160,923.931	626,763.226
8+80	6°21'58.31"	13°21'38.03"	99.794	5.550	2,160,910.419	626,694.561
9+20	3°49'10.99"	15°54'25.35"	59.956	1.999	2,160,900.314	626,655.861
9+90	0°38'11.83"	20°21'48.17"	10.000	0.056	2,160,878.532	626,583.356
10+60	5°05'34.65"	24°49'10.99"	79.895	3.553	2,160,851.647	626,524.743
11+00	7°38'21.97"	27°21'58.31"	119.645	7.988	2,160,834.056	626,488.823
11+70	12°05'44.79"	31°49'21.13"	188.592	19.981	2,160,799.494	626,427.970
12+40	16°33'07.61"	36°16'43.95"	256.399	37.295	2,160,760.309	626,369.987
12+80	19°05'54.94"	38°49'31.27"	294.475	49.539	2,160,735.932	626,338.277
13+50	23°33'17.75"	43°16'54.09"	359.665	74.990	2,160,689.970	626,285.504
14+10	27°22'28.74"	47°06'05.08"	413.826	100.783	2,160,647.410	626,243.227
14+70	31°11'39.73"	50°55'16.07"	466.149	130.126	2,160,602.128	626,203.880
15+30	35°00'50.71"	54°44'27.05"	516.400	162.890	2,160,554.326	626,167.637
15+90	38°50'01.70"	58°33'38.04"	564.357	198.929	2,160,504.215	626,134.658
16+50	42°39'12.69"	62°22'49.03"	609.807	238.082	2,160,452.018	626,105.091
17+10	46°28'23.68"	66°12'00.01"	652.548	280.176	2,160,397.968	626,079.067
17+70	50°17'34.66"	S70°01'11.00"E	692.389	325.024	2,160,342.305	626,056.702

AXIS STATION	INTERSECTION—130' LONG. JOINTS AND TRANS. JOINTS				INTERSECTION—161.5' LONG. JOINTS AND TRANS. JOINTS			
	A	B	COORDINATES N	COORDINATES E	A	B	COORDINATES N	COORDINATES E
3+30								
3+90								
4+50								
5+10								
5+70								
6+30								
7+00	252.546	114.605	2,160,859.323	626,875.158				
7+40	217.402	104.160	2,160,857.293	626,838.551	209.101	134.547	2,160,825.888	626,840.993
8+10	154.908	89.674	2,160,849.835	626,774.835	148.994	120.614	2,160,816.715	626,779.711
8+80	91.478	80.087	2,160,837.449	626,711.892	87.985	111.393	2,160,806.802	626,719.171
9+20	54.959	76.833	2,160,828.186	626,676.417	52.861	108.263	2,160,797.893	626,685.051
9+90	9.166	75.051	2,160,808.219	626,615.453	8.816	106.549	2,160,778.687	626,626.415
10+60	73.237	78.257	2,160,783.575	626,556.225	70.440	109.633	2,160,754.984	626,569.448
11+00	109.674	82.322	2,160,767.449	626,523.299	105.487	113.543	2,160,739.474	626,537.778
11+70	172.876	93.316	2,160,735.768	626,467.517	166.275	124.117	2,160,709.003	626,484.127
12+40	235.032	109.187	2,160,699.848	626,414.365	226.058	139.382	2,160,674.455	626,433.005
12+80	269.936	120.411	2,160,677.503	626,385.299	259.629	150.177	2,160,652.962	626,405.047
13+50	329.693	143.741	2,160,635.371	626,336.923	317.105	172.617	2,160,612.439	626,358.519
14+10	379.341	167.384	2,160,596.357	626,298.169	364.857	195.357	2,160,574.915	626,321.245
14+70	427.303	194.282	2,160,554.849	626,262.101				
15+30								
15+90								
16+50								
17+10								
17+70								

AXIS STATION	INTERSECTION—151' LONG. JOINTS AND TRANS. JOINTS				INTERSECTION—179.5' LONG. JOINTS AND TRANS. JOINTS			
	A	B	COORDINATES N	COORDINATES E	A	B	COORDINATES N	COORDINATES E
3+30	531.487	296.728	2,160,782.042	627,199.202				
3+90	490.118	262.662	2,160,800.144	627,148.762	472.745	285.254	2,160,773.014	627,140.033
4+50	446.572	231.427	2,160,814.848	627,097.228	430.742	255.127	2,160,787.196	627,090.327
5+10	401.041	203.163	2,160,826.085	627,044.830	386.825	227.864	2,160,798.035	627,039.785
5+70	353.729	177.995	2,160,833.807	626,991.799	341.190	203.588	2,160,805.483	626,988.634
6+30	304.845	156.034	2,160,837.978	626,938.371	294.039	182.406	2,160,809.507	626,937.101
7+00					237.393	161.729	2,160,809.851	626,876.800
7+40								
8+10								
8+80								
9+20								
9+90								
10+60								
11+00								
11+70								
12+40								
12+80								
13+50								
14+10								
14+70					401.665	236.626	2,160,523.645	626,300.527
15+30	461.317	241.515	2,160,498.907	626,246.025	444.965	264.857	2,160,482.455	626,269.297
15+90	504.159	273.710	2,160,454.142	626,216.565	486.288	295.910	2,160,439.276	626,240.881
16+50	544.761	308.687	2,160,407.513	626,190.152	525.450	329.647	2,160,394.300	626,215.404
17+10	582.942	346.291	2,160,359.228	626,166.904	562.278	365.918	2,160,347.727	626,192.980
17+70					596.603	404.562	2,160,299.763	626,173.708



TOP OF LONGITUDINAL JOINTS			
BLOCK No.	ELEV.	BLOCK No.	ELEV.
2	3570	14	.480
3	3510	15	3375
4	3525	16	3480
5	3435	17	3375
6	3465	18	3480
7	3375	19	3420
8	3480	20	3510
9	3375	21	3450
10	3480	22	3510
11	3375	23	3510
12	3480	24	3570
13	3375	25	3540

NOTE

Odd numbered blocks shall be the low blocks. In each block, below the termination of the longitudinal joint, the portion downstream of the joint shall be high in relation to the upstream portion.

10-12-59 D.E.C.	REVISED NOTE AND DOWNSTREAM TOE, RELOCATED LONG. JOINTS IN BLOCKS 19 AND 20, AND CHANGED TOP OF JOINT IN BLOCK 22
5-30-58 D.E.C.	REVISED IN ACCORDANCE WITH DWG. 557-D-435 (REV. 6-30-58) AND CHANGED TOP OF JOINT IN BLOCKS 2 AND 25

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT- ARIZONA-UTAH

GLEN CANYON DAM

CONTRACTION JOINT LAYOUT

DRAWN R.S.P. SUBMITTED [signature]
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DENVER, COLORADO, JUNE 18, 1957 — 557-D-433

Figure 64.—Contraction joint layout in dam.

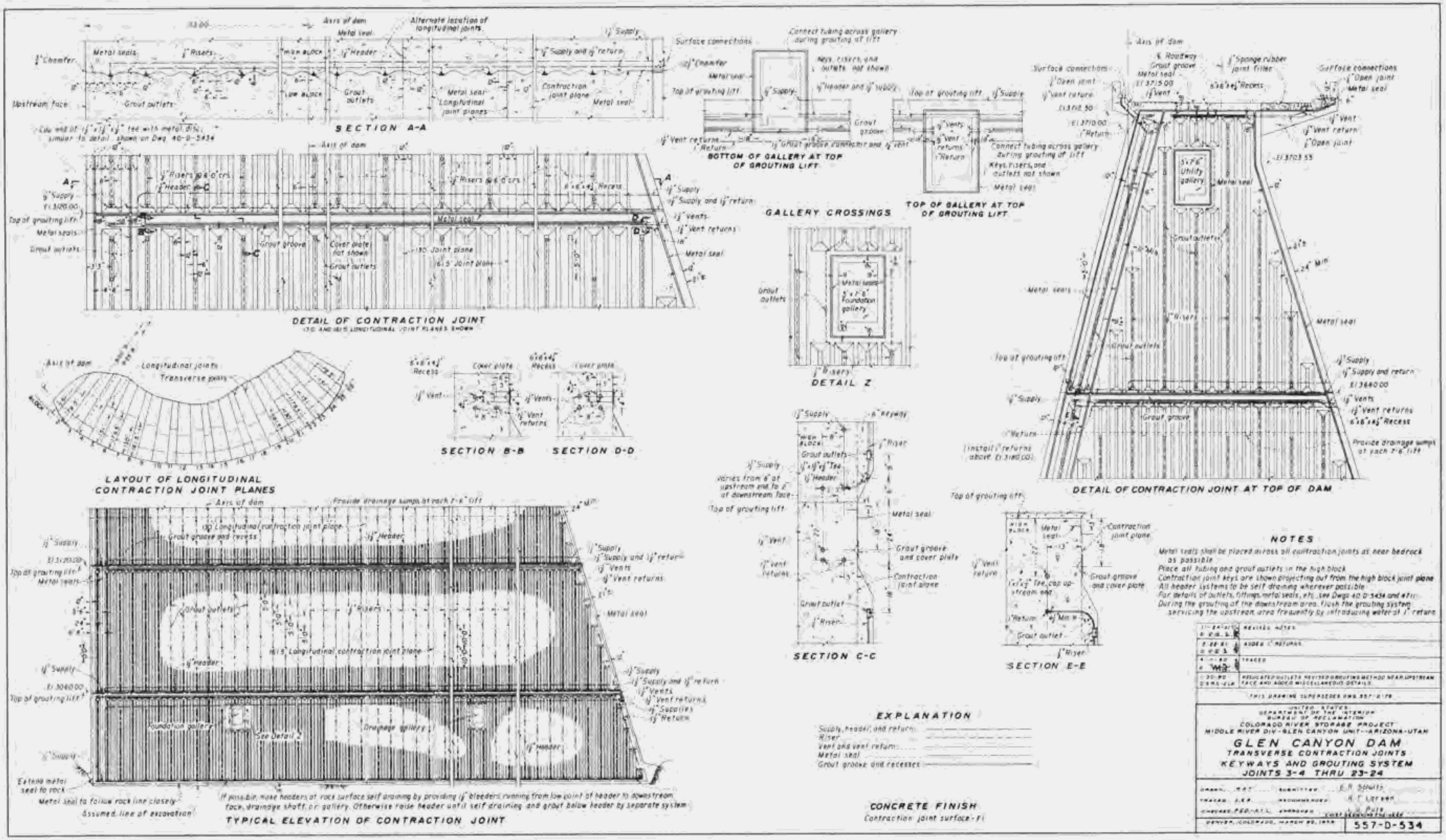

Figure 65.—Dam transverse contraction joints—Keyways and grouting system at joints 3-4 through 23-24.

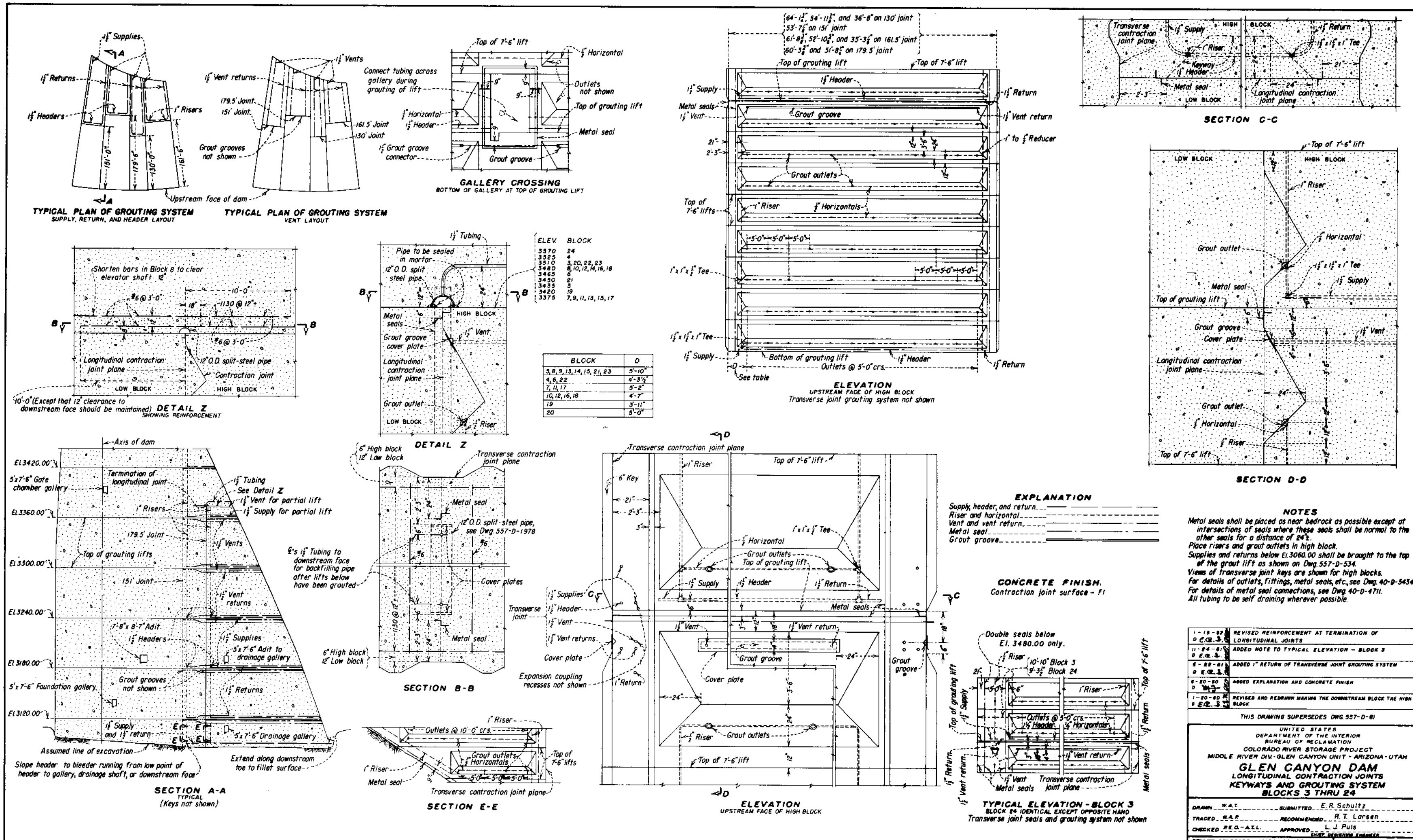

Figure 66.—Dam longitudinal contraction joints—Keyways and grouting system at blocks 3 through 24.

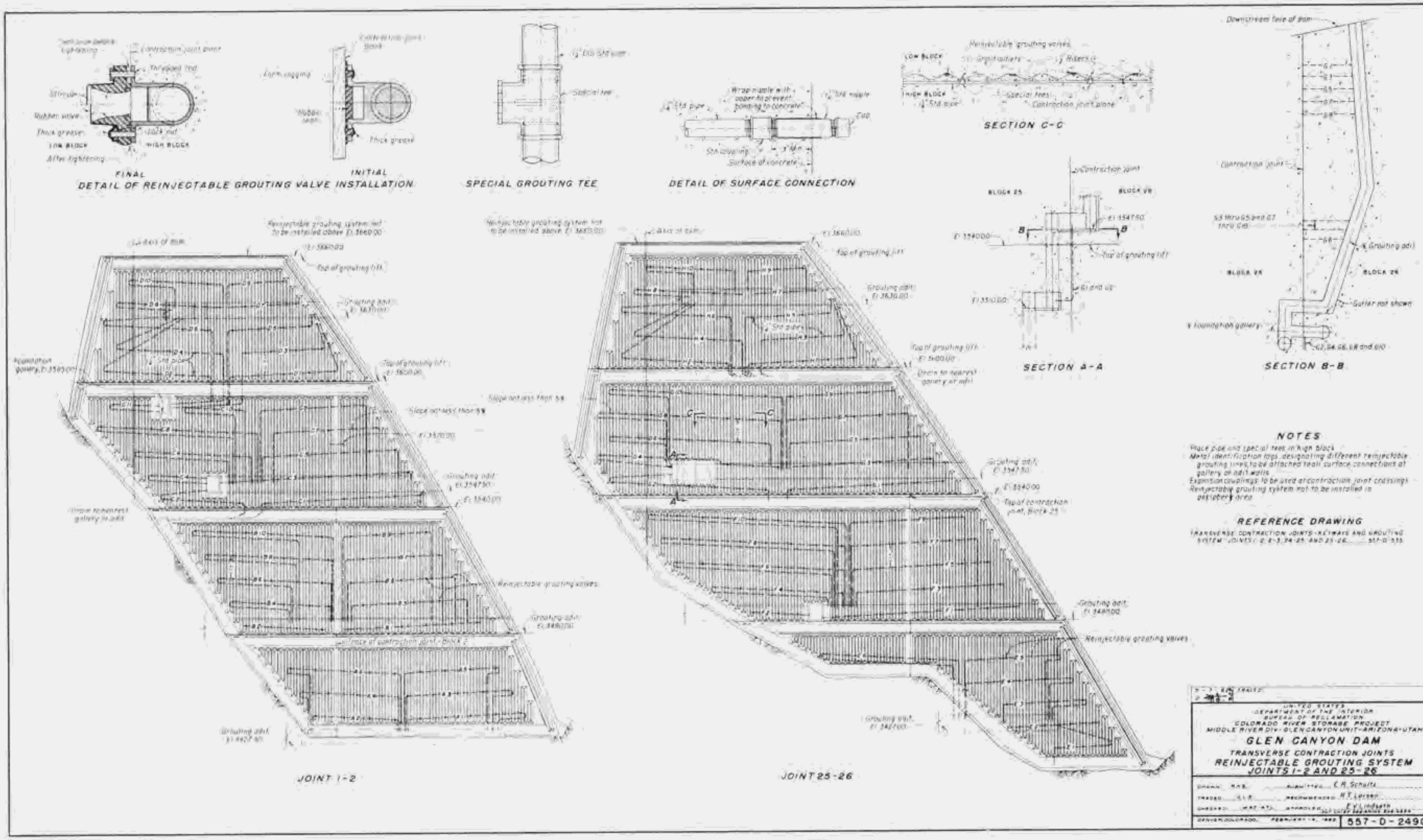

Figure 67.—Dam transverse contraction joints—Reinjectable grouting system of joints 1-2 and 25-26.

and equipment in the pump chamber and to three plumbline well reading stations.

(e) *Gate Chamber and Gate Chamber Gallery.*—The gate chamber gallery is located in the left abutment at elevation 3390.00, 2 feet downstream of the axis of the dam. It connects the elevator lobby in block 8 with the foundation gallery in block 3. It provides access to the 96-inch ring-follower gates, controls, and equipment in the gate chamber and to the upper end of the outlet pipes for inspection and/or repairs. It also provides access to a fan chamber and two plumbline well reading stations.

(f) *Filling Line Chambers and Filling Line Gallery.*—The filling line gallery is located at elevation 3480.00 and is 20 feet upstream of the axis of the dam. Adits connect this gallery with the elevator lobbies and with both end foundation galleries. The filling line gallery provides access to the penstock filling line chambers and piping and to the upper end of the penstocks for inspection or repairs. The gallery also provides access to plumbline reading stations and fan chambers.

(g) *Utility Gallery.*—The utility gallery is at elevation 3697.50 and is 12 feet downstream of the dam axis, extending for the entire length of the dam. Access to the utility gallery is provided from the roadway on top of the dam through 3-foot 6-inch-diameter shafts located in blocks 1 and 26. These shafts extend to the adits to the downstream face which are connected to the utility gallery. Access is also provided by adits from the elevator towers. This gallery contains water supply service for the dam and powerplant and sewage pipes from the visitors' facilities.

The suspension chambers for the plumbline wells are located in this gallery. The gate service shaft also is accessible from the gallery. This 5- by 11-foot vertical shaft just downstream from the gate chamber in block 6 provides a means for moving gate parts from the dam to the roadway level.

(h) *Grouting Adits.*—The grouting adits were added to the original system of galleries and adits, except for the adit at 3480 which was enlarged, to accommodate the foundation grouting program. The adits are 6 feet by 7 feet 6 inches with a 12-inch-wide gutter on the abutment side. These adits were interconnected with the foundation and drainage gallery where possible to provide easier access for grouting. The vertical spacing of the adits was about 60 feet. Some of the adits were extended to the downstream face of the dam. This was done to provide access to the adits from outside the dam for ease in setting up grouting equipment and supplies. A minimum clearance of 5 feet was provided between the adit and contraction joints and about 30 feet between the adit and the excavated surface. As a safety feature, all adits that exit on the downstream dam face have doors at their upstream entrance.

(i) *Powerplant Adits.*—The two powerplant adits are located at elevation 3187.50. They provide access to the powerplant from the two elevators in the dam. The adits are 6 feet 9-1/2 inches by 8 feet 3 inches. A pipe chase is located under the floor of the adits to contain water and sanitary piping to the powerplant. Since visitors are conducted to the powerplant through these adits, the adits have received special treatment. The walls are of ceramic tile, a false ceiling has been installed, and the floors are covered with terrazzo.

(j) *Electrical Service Adits.*—The electrical service adits are located in blocks 6 and 19. They extend between the downstream face at elevation 3187.50 and elevation 3202.50 at the transformer area. These 4- by 7.5-foot adits carry the electrical conduits to the transformer chamber.

(k) *Miscellaneous Adits and Chambers.*—Adits from the elevator shafts to plumbline well reading stations are located at elevations 3585 and 3285 in block 8 and at elevations 3585, 3480, and 3322.50 in block 17.

Access to the water tank chamber is available from the elevator shaft in block 8 at elevation 3345.

34. REINFORCEMENT OF OPENINGS IN DAM. (a) *General.*—All openings in the dam are reinforced for the calculated stresses in the dam in the area in which they are located. The reinforcement required was based on the tension stresses at openings as determined by photoelastic studies of openings in an infinite plate[4] [5]. Special studies were made for the larger openings and for openings located near contraction joints where grout pressure in a joint

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[4] Interoffice Bureau of Reclamation Memorandum of January 16, 1950—"Stress Distribution on Radial Lines Around a Circular Hole in an Infinite Plate." (Unpublished.)

[5] Interoffice Bureau of Reclamation Memorandum of January 15, 1959—"A Photoelastic Stress Analysis of Rectangular Openings in an Infinite Plate." Revised February 14, 1962. (Unpublished.)

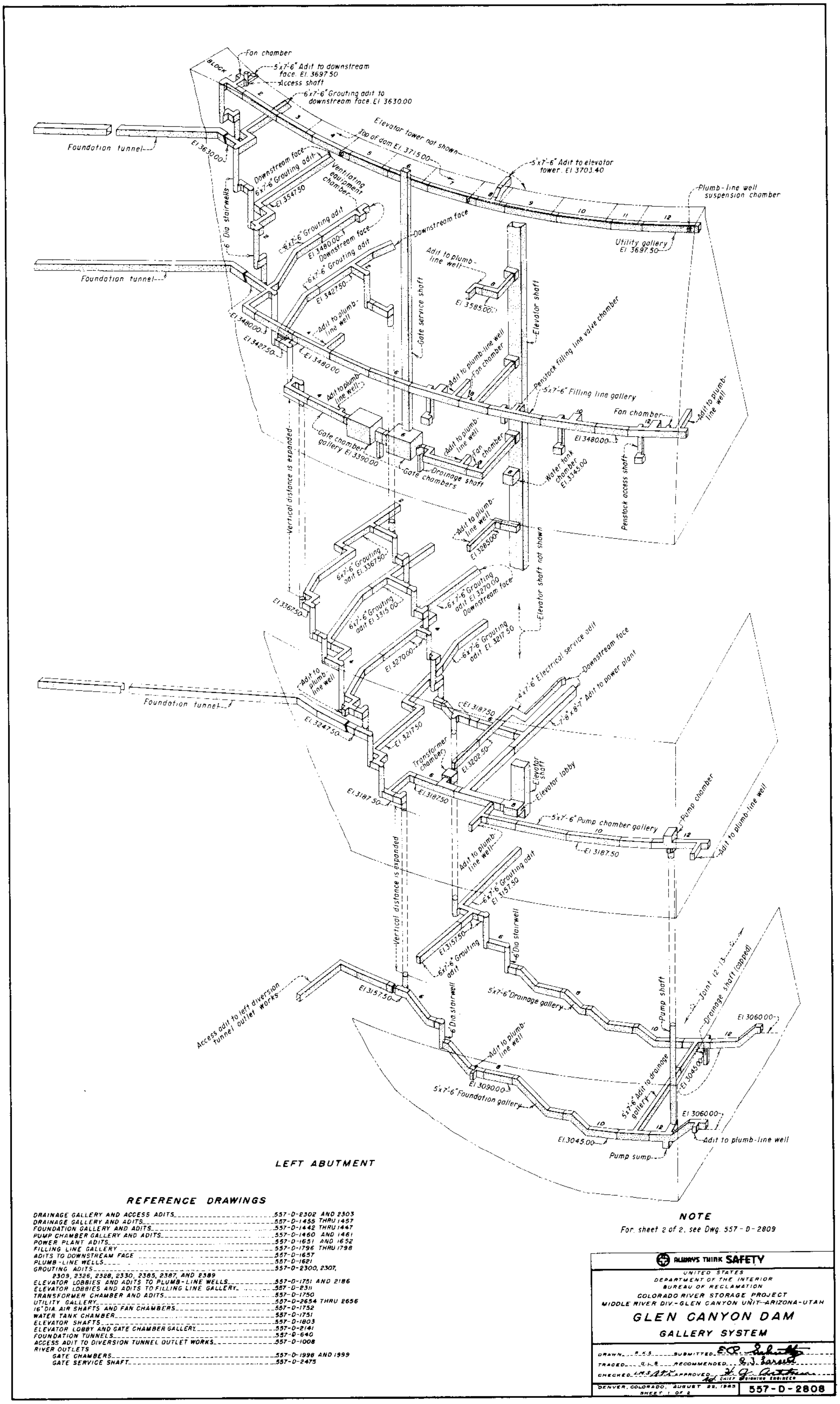


Figure 68.—Dam gallery system—Left abutment.

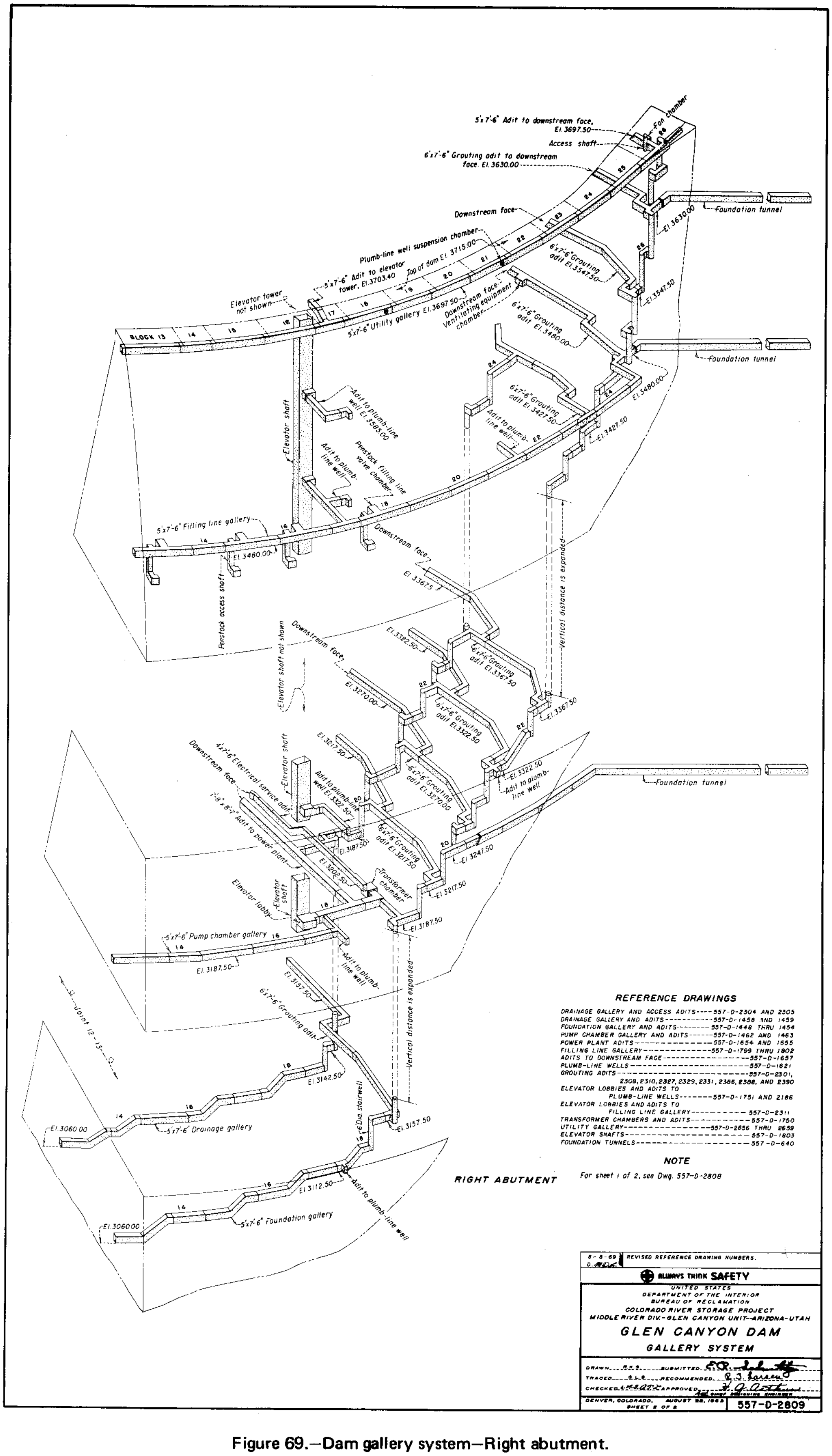


Figure 69.—Dam gallery system—Right abutment.

would affect the opening. All openings in the dam over 24 inches in diameter were reinforced.

(b) *Loading Conditions.*—Openings were reinforced for the stresses produced by the following load conditions:

(1) Dead load of the completed dam with reservoir water surface at elevation 3240.

(2) Arch and cantilever loads of the completed dam with water surface elevation 3700 and including earthquake loading.

(3) *Grout pressures in contraction joints adjacent to openings.*—Grout pressure in the joints of 50 pounds per square inch at top of grout lifts and increasing by 0.75 pound per square inch for every foot below the top of the lift.

(4) *Temperature effects at utility gallery.*—To prevent cracking due to the temperature differential between the roadway and faces of the dam and the utility gallery, the utility gallery reinforcement was increased to 1-inch-diameter bars at 6 inches top and at 12 inches sides and bottom, with 3/4-inch-diameter bars at 12 inches longitudinally.

When one load reduced the tensile stress produced by another load, 75 percent of the computed load reducing the tensile stress was used to reduce the tensile stress.

Allowable stress in the reinforcement was 25,000 pounds per square inch except for reinforcement required for grouting loads. Since this load occurred only during grouting of the contraction joints, the stress in the reinforcement was allowed to increase to 30,000 pounds per square inch. Reinforcement bars were lapped a minimum of 24 bar diameters; however, in the areas where the allowable stress was increased to 30,000 pounds per square inch the maximum lap was increased to 30 bar diameters.

35. ROADWAY AND PARAPETS. (a) *General.*—A roadway was provided on top of the dam at elevation 3715 for servicing the dam and appurtenant features and for access between the spillways. Details of the roadway and parapets are shown on figure 70.

The required roadway width was 35 feet to accommodate the 165-ton gantry crane. Since the top width of the structural dam was only 25 feet, a downstream cantilever of 14 feet 6 inches was required to provide the additional roadway. An upstream cantilever was also required at the left abutment to provide an area for the crane to pick up loads from trucks. A sidewalk and parapet were provided at each side of the roadway. The sidewalks were raised 9 inches above the roadway to provide a curb. The upstream sidewalk is 18 inches wide and the downstream sidewalk is 3 feet wide. The parapets are 18 inches thick and the top is at elevation 3719, 3 feet 3 inches above the sidewalks. To prevent the cantilevers from acting as a stiff longitudinal arch at the top of the dam, 1/2-inch open joints were formed in the cantilevers, sidewalks, and parapets at approximately 20-foot centers.

The roadway is lighted by lighting units embedded in the parapets. Lighting units are spaced at about 30-foot centers, alternating in the upstream and downstream parapets. Power receptacles are provided in the downstream parapet for the gantry crane.

Drainage of the roadway is accomplished by sloping the roadway to the center where the runoff is collected and carried to the reservoir by 8-inch-diameter cast iron soil pipe.

(b) *Structural Design.*—The design was based on concrete having a compressive strength of 3,000 pounds per square inch at 28 days. The allowable working stresses are shown on figure 71. Design loads are as follows:

(1) *Cantilevers.*—The cantilevers were designed for the dead load of the cantilever plus the following live load conditions:

Gantry crane loaded and trailer hauling unit unloaded;
Gantry crane unloaded and trailer hauling unit loaded; and
300 pounds per square foot live load.

The load from the gantry crane wheels was 51,250 pounds loaded and 48,125 pounds unloaded, with wheels spaced at 15-1/2-inch centers. The axle loads from the 35-ton trailer hauling unit were 25,800 pounds loaded and 4,800 pounds unloaded with the axles at 4-foot centers.

(2) *Roadway.*—Because of the temperature differential between the top of the dam and the utility gallery at elevation 3697.50, two layers of 1-3/8-inch-diameter bars at 12-inch centers across the roadway tied by 1-inch-diameter bars at 12-inch centers were placed in the top of the dam to control temperature cracking.

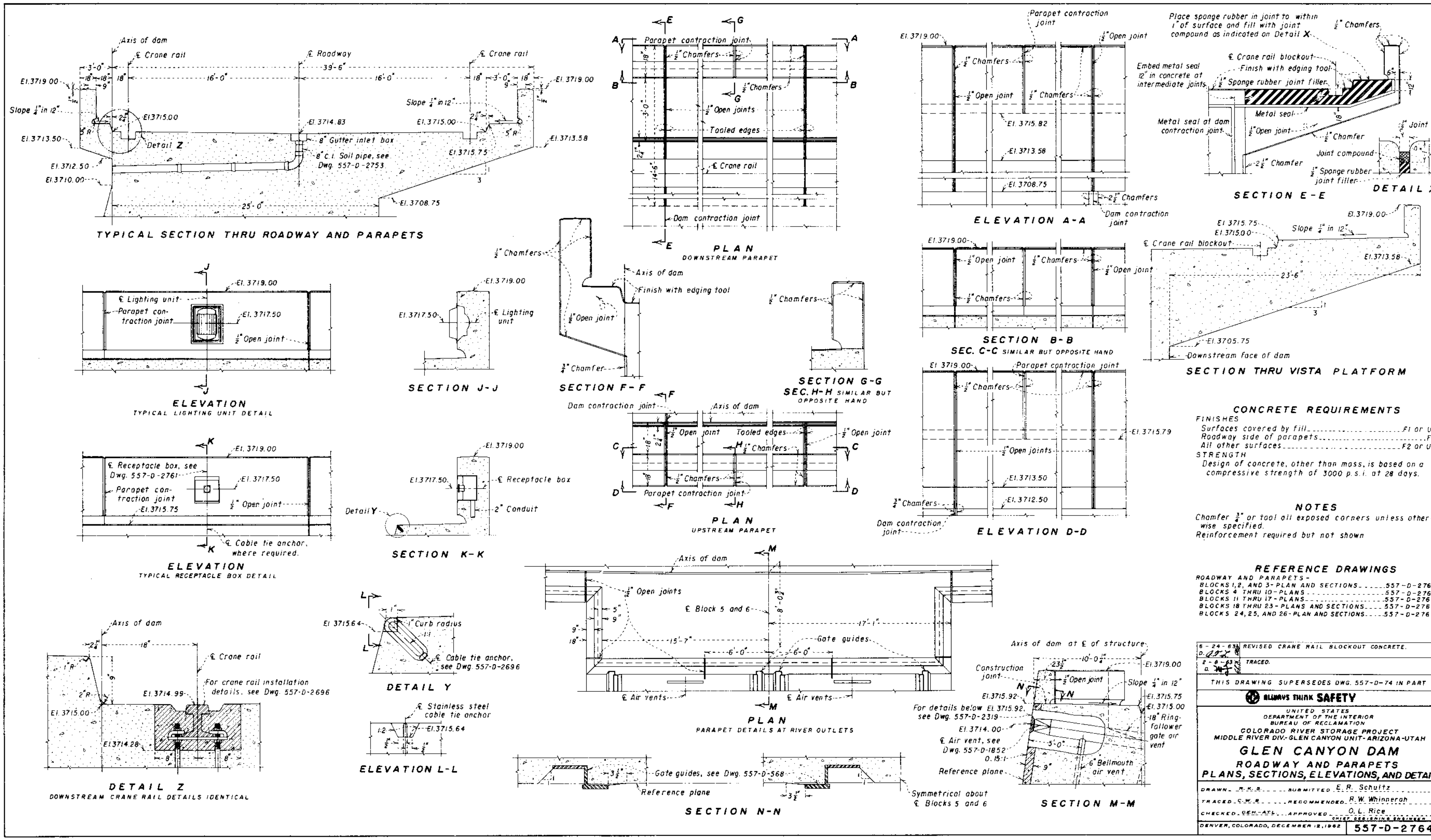


Figure 70.—Dam roadway and parapets—Plans, sections, elevations, and details.

WORKING STRESSES FOR CONCRETE

		2,000	2,500	3,000	3,750
STRENGTH CLASSIFICATION, LB. PER SQ. INCH AT 28 DAYS	f'_c	2,000	2,500	3,000	3,750
FLEXURE: f_c, f_t, lb. per sq. inch					
Extreme fiber stress in compression	$0.45 f'_c = f_c$	900	1,125	1,350	1,688
Extreme fiber stress in tension (for plain concrete footings only)	$0.03 f'_c = f_t$	60	75	90	113
SHEAR: V_c, lb. per sq. inch					
Beams with no web reinforcement	$0.03 f'_c$ (max. 90) $= V_c$	60	75	90	90
Beams with properly designed web reinforcement (when V_c is in excess of $0.06 f'_c$ web reinforcement should provide for total shear)	$0.12 f'_c$ (max. 360) $= V_c$	240	300	360	360
Footings	$0.03 f'_c$ (max. 75) $= V_c$	60	75	75	75
‡ **BOND:** u, lb. per sq. inch of surface area of bar					
※ Top bars	$0.07 f'_c$ (max. 245) $= u$	140	175	210	245
In two-way footings (except top bars)	$0.08 f'_c$ (max. 280) $= u$	160	200	240	280
All others	$0.10 f'_c$ (max. 350) $= u$	200	250	300	350
BEARING; f_c, lb. per sq. inch					
Full area loaded	$0.25 f'_c = f_c$	500	625	750	938
Load on partial area, maximum	$0.375 f'_c = f_c$	750	938	1,125	1,405

※ Top bars are horizontal bars so placed that more than 12 inches of concrete is cast in the member below the bar. In case of uncertainties regarding classification of horizontal bars, use bond stress for top bars.

‡ Deformations for high-bond bars shall conform to the requirements of A.S.T.M. Designation A. 305-(latest edition)

CONSTANTS					
	n	15	12	10	8
	p	0.0091	0.0113	0.0136	0.0170
	j	0.87	0.87	0.87	0.87
	k	0.40	0.40	0.40	0.40
	R	157	196	235	294

WORKING STRESSES FOR REINFORCEMENT

REINFORCEMENT		
Tension in flexural members with or without axial loads. Intermediate and hard-grade steel	f_s	20,000
Tension in web reinforcement Intermediate grade steel	f_s	16,000
Compression in column verticals and flexural members (Intermediate grade steel)	f_s	16,000
Compression in column verticals (hard grade steel)	f_s	20,000

NOTE

For conditions not listed, use the report of the Joint Committee, for plain and reinforced concrete, published in A.S.C.E. Proceedings, June 1940, and if the report does not cover the conditions, use "Standards of Design for Concrete" No. 3YB by U.S. Navy Department Bureau of Yards and Docks. Values of bond and maximum values of shear from A.C.I. Bldg. Code 318-56.

ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

WORKING STRESSES
FOR CONCRETE AND REINFORCEMENT
20,000 PSI HIGH-BOND BARS

DRAWN H.H.B. SUBMITTED
TRACED V.M.M. RECOMMENDED
CHECKED APPROVED Grant Bloodgood
ASSOC. CHIEF ENGINEER

DENVER, COLORADO, AUG. 15, 1956 | 103-D-563

Figure 71.—Working stresses for concrete and reinforcement—20,000 p.s.i. high-bond bars.

(3) *Sidewalks and parapets.*—To control cracking, the sidewalks were reinforced with 5/8-inch-diameter bars at 9-inch centers each way and the parapets were reinforced with 5/8-inch-diameter bars at 9-inch centers vertically and 5/8-inch-diameter bars at 6-inch centers horizontally.

36. ELEVATOR SHAFTS AND TOWERS. (a) *General.*—The elevator shafts and towers are located in blocks 8 and 17. The towers are 23 by 35 feet and are shown on figures 72 and 73. The shaft openings are 9 feet 6 inches by 14 feet 8 inches and are shown on figure 74. The elevators are used for both freight and passenger service and provide access to the numerous galleries. The visitors are transported from the top of the dam to the level of the powerplant by means of the elevators. There are six floors in the tower. These are:

Floor	Elevation
Elevator machinery	3739.65
Men's restrooms	3728.40
Floatwell room (block 8 only)	3728.40
Main lobby	3715.90
Women's restrooms	3703.40
Ventilating equipment	3692.15
Electrical equipment	3680.90

The access to elevator shaft escape ladder system is located at elevation 3680.90.

Vista platforms and canopies are located adjacent to the elevator towers. These provide vista points overlooking the powerplant and the canyon below the dam (see fig. 75).

(b) *Tower Design.*—The design of the towers was based on a compressive strength in the concrete of 3,000 pounds per square inch at 28 days. The allowable working stresses were as shown on figure 71.

(1) *Floors and roof.*—In addition to dead load, the floors were designed for the following live loads:

Floors elevation 3739.65 and 3715.90—400 pounds per square foot or a 5,500-pound motor generator over a 4- by 3-foot area.
All other floors and stairways—150 pounds per square foot.
Roof—40 pounds per square foot.

(2) *Walls.*—The exterior walls and walls of the elevator shaft were designed for the following loads:

Loads imposed by the floor slabs and beams.
Earthquake forces of 0.1g horizontal and 0.05g vertical.
Windload of 20 pounds per square foot (exterior walls only).

Interior walls other than those forming the shaft were separated from the floors above by 1/2-inch joint filler so that they cannot carry loads from the floor above.

(c) *Vista Platform and Canopy Design.*—Since the platform is an extension of the roadway cantilever, it was designed for the same loads as the roadway cantilevers in addition to the loads imposed by the canopy. The canopy was designed for dead load, a live load of 40 pounds per square foot, and a windload of 20 pounds per square foot over the projected area.

(d) *Shafts.*—The elevator shafts in mass concrete were reinforced using the same criteria as for openings in the dam (sec. 34).

(e) *Floatwell.*—The top of the floatwell is at elevation 3731.40. Reservoir level recording instruments were installed in the floatwell room. The floatwell was also reinforced using the criteria for openings in dam.

37. STRUCTURE DRAINAGE. In addition to the drainage curtains drilled in the foundation of the dam (sec. 14), 5-inch-diameter vertical drain holes on 10-foot centers near the upstream face of the dam have been formed in the mass concrete of the dam to collect any possible seepage through the dam and reduce possible uplift at construction lifts. These drains empty into the gutters of the penstock filling line gallery and the foundation gallery (see fig. 76).

The sump in the foundation gallery (block 12) collects all drainage water below elevation 3187.50 from the foundation and drainage galleries. The water is pumped to the 30-inch pipe in the pump chamber (see figs. 77 and 78). This water and the drainage water above elevation 3187.50 drains by gravity to the catch basin at the downstream face of the dam as shown on figure 79.

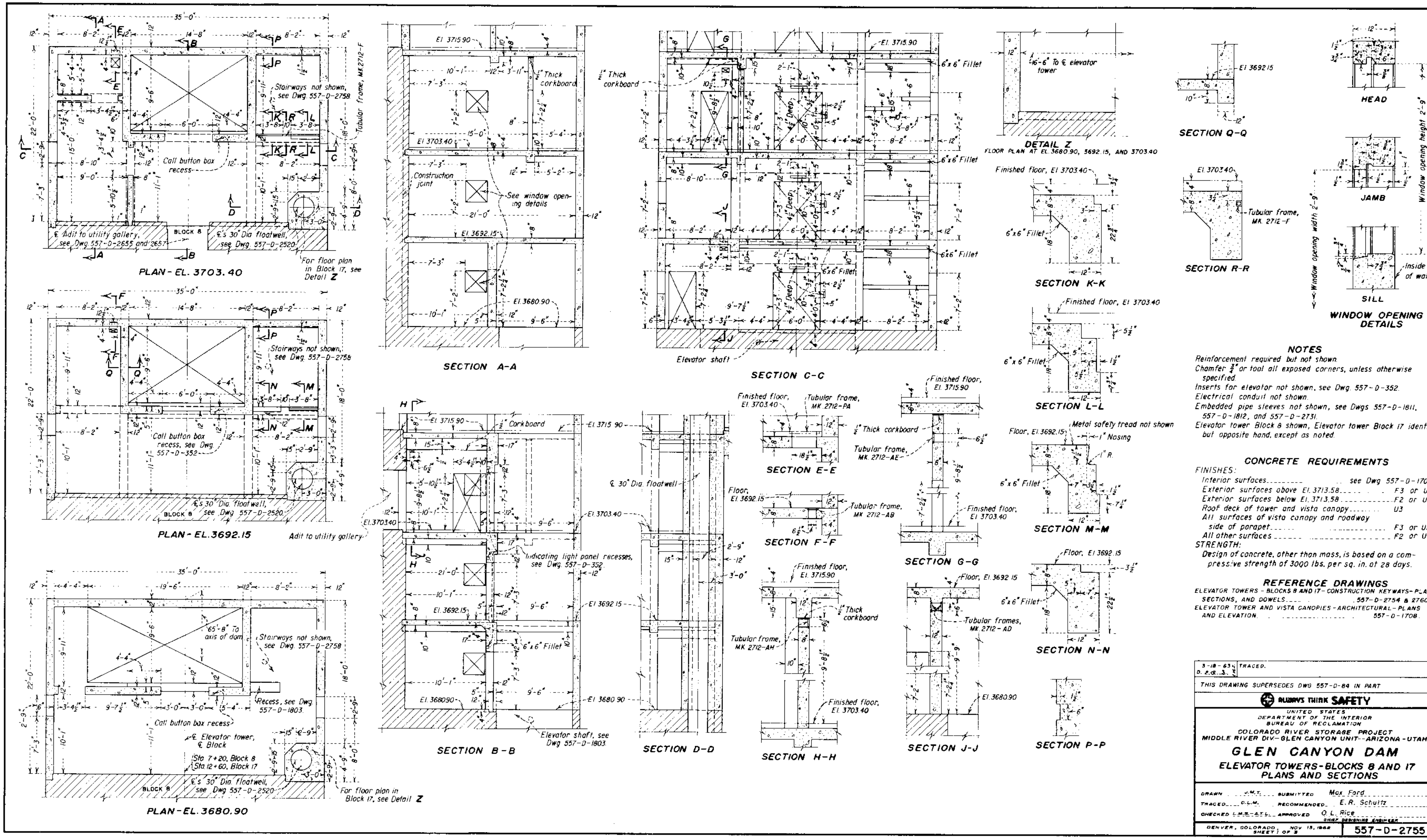


Figure 72.—Dam elevator towers, blocks 8 and 17—Plans and sections. (Sheet 1 of 3.)

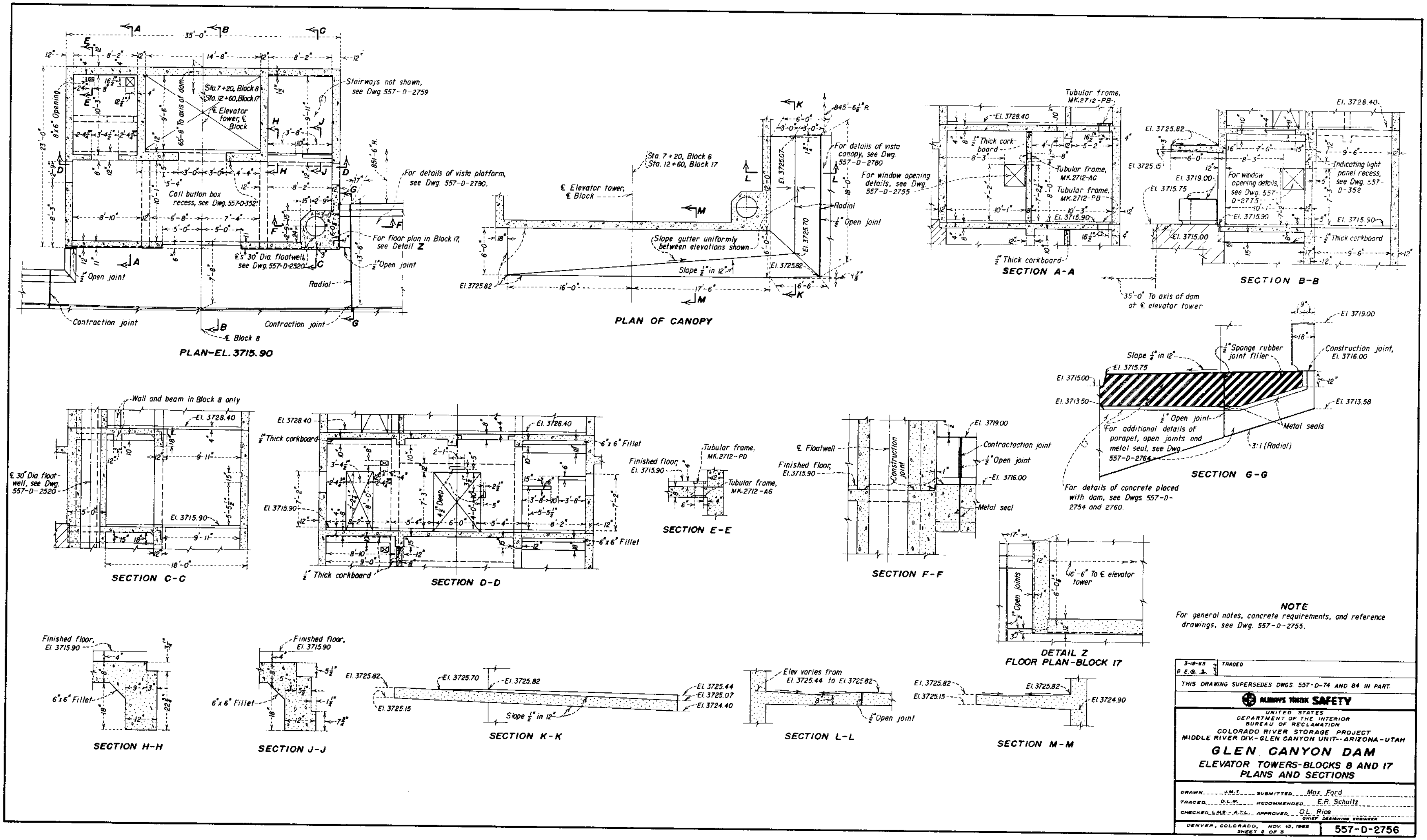


Figure 72.—Dam elevator towers, blocks 8 and 17—Plans and sections. (Sheet 2 of 3.)

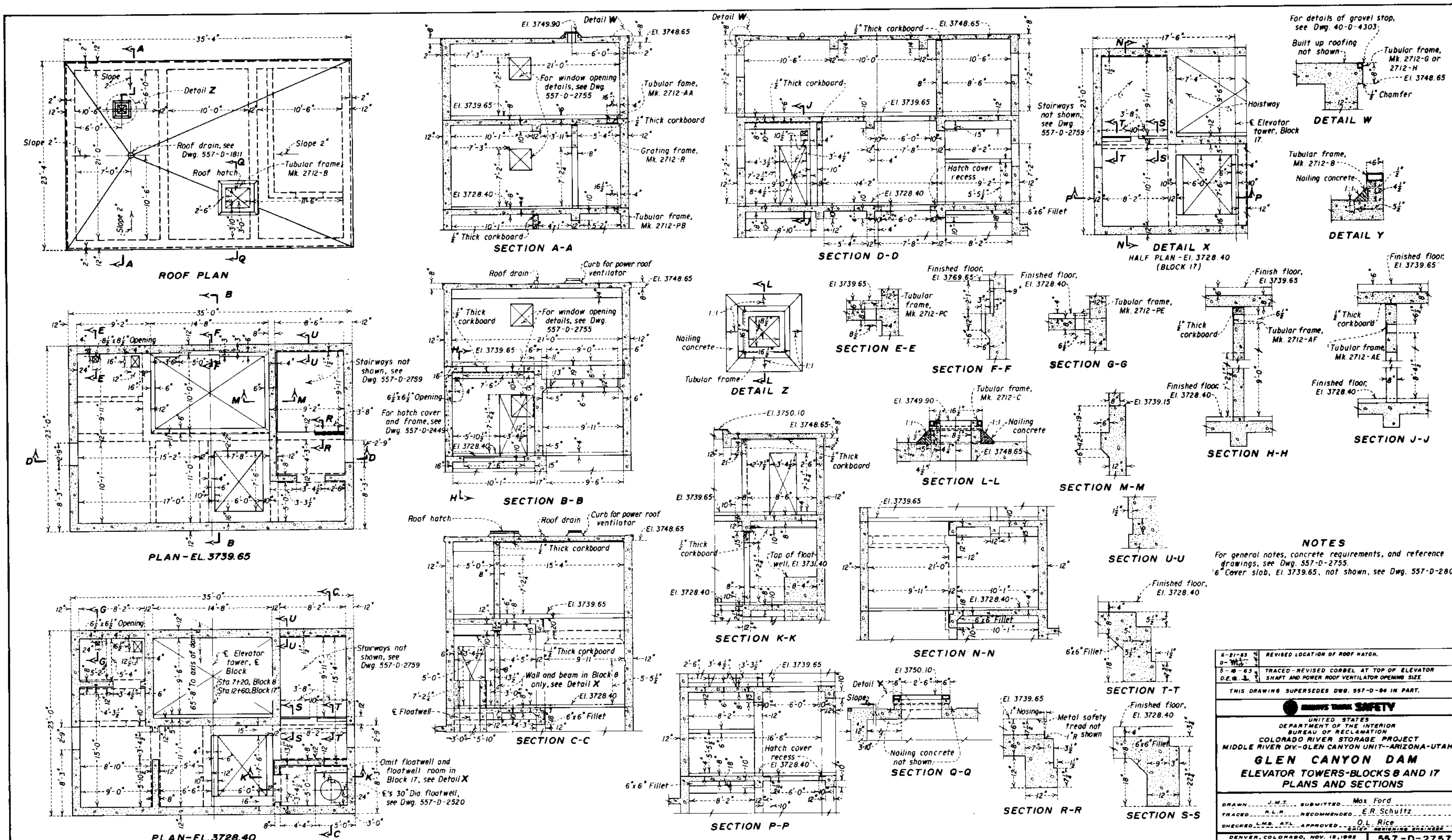


Figure 72.—Dam elevator towers, blocks 8 and 17—Plans and sections. (Sheet 3 of 3.)

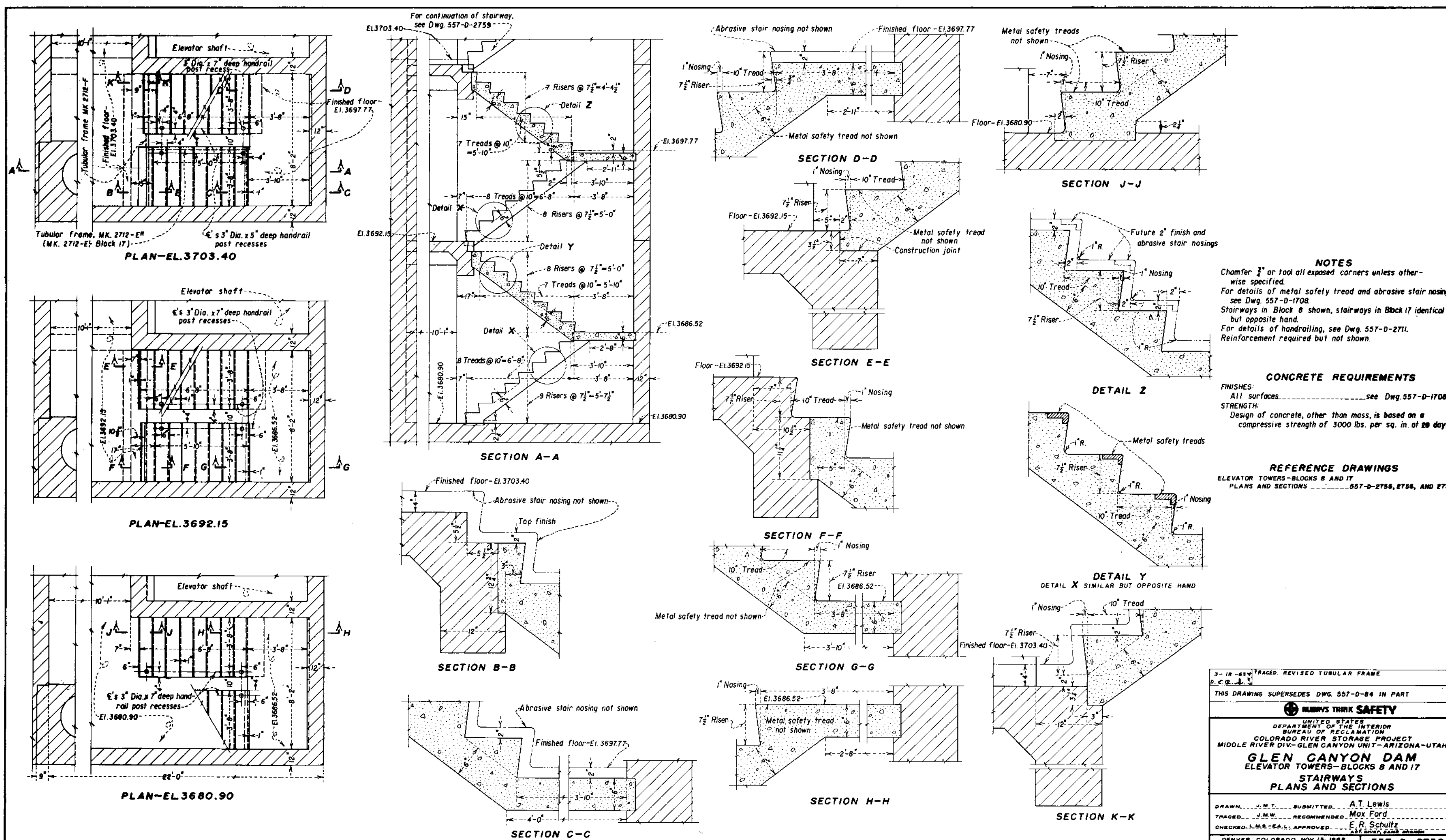


Figure 73.—Dam elevator towers, blocks 8 and 17 stairways—Plans and sections. (Sheet 1 of 2.)

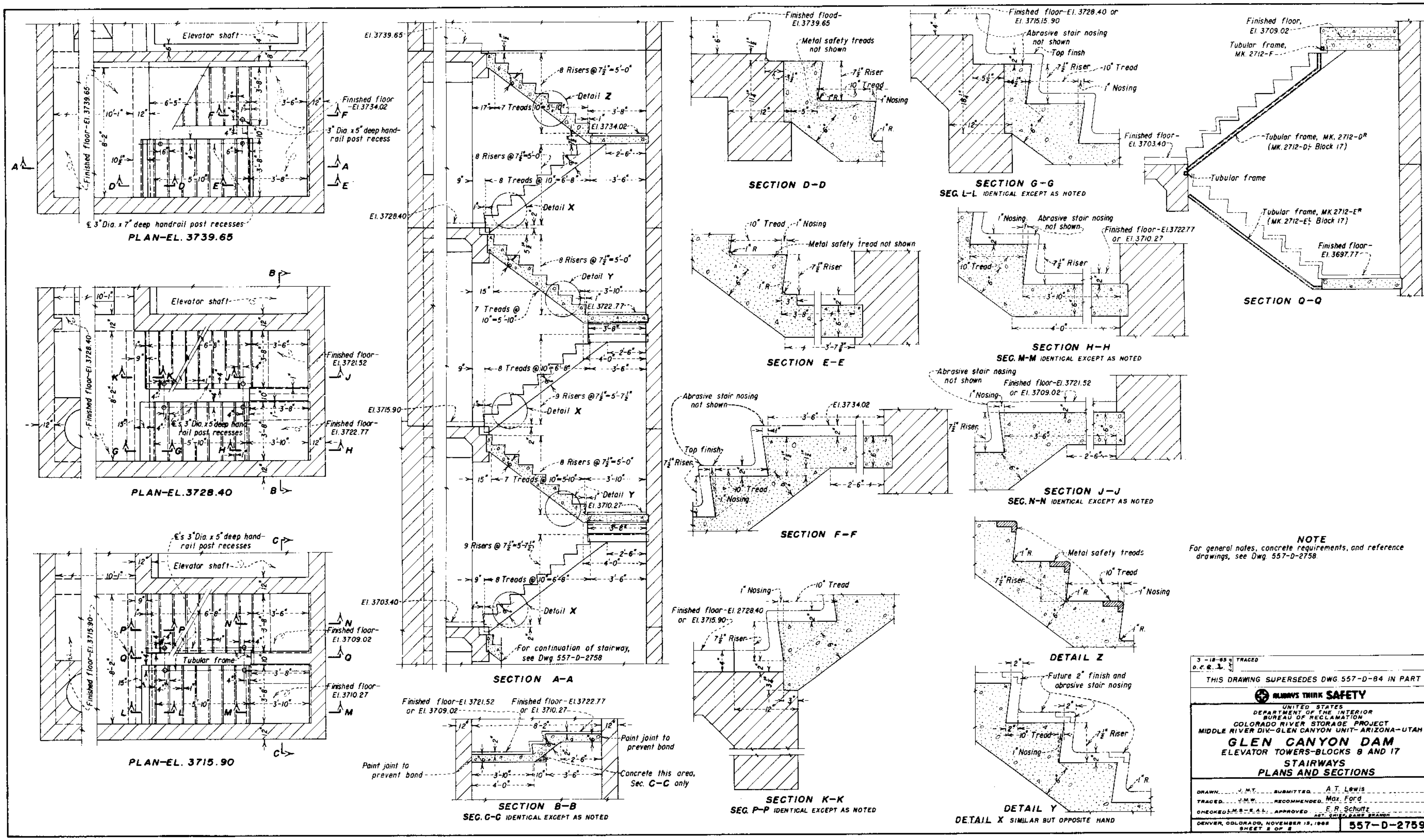


Figure 73.—Dam elevator towers, blocks 8 and 17 stairways—Plans and sections. (Sheet 2 of 2.)

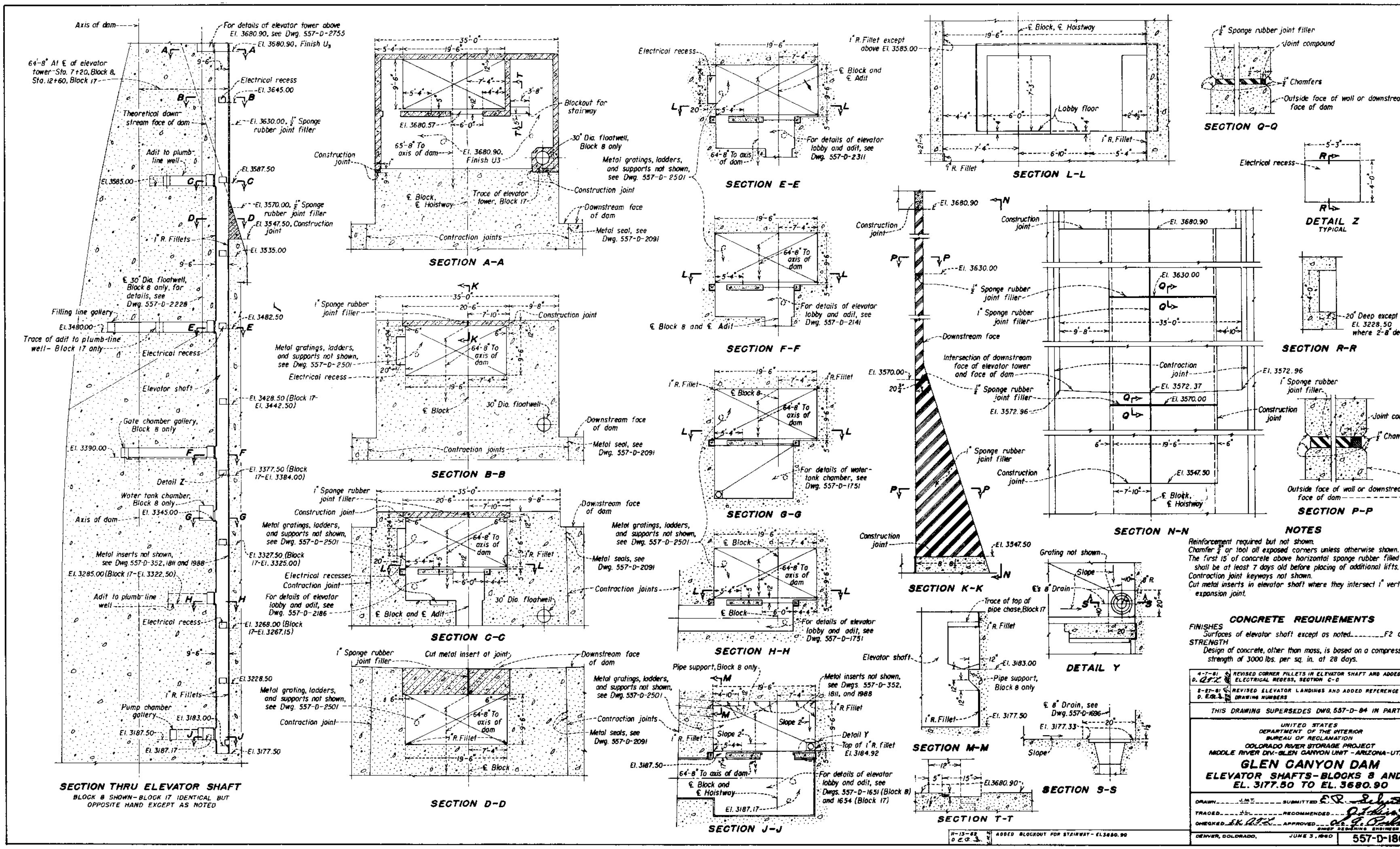


Figure 74.—Dam elevator shafts, blocks 8 and 17—Elevation 3177.50 to elevation 3680.90.

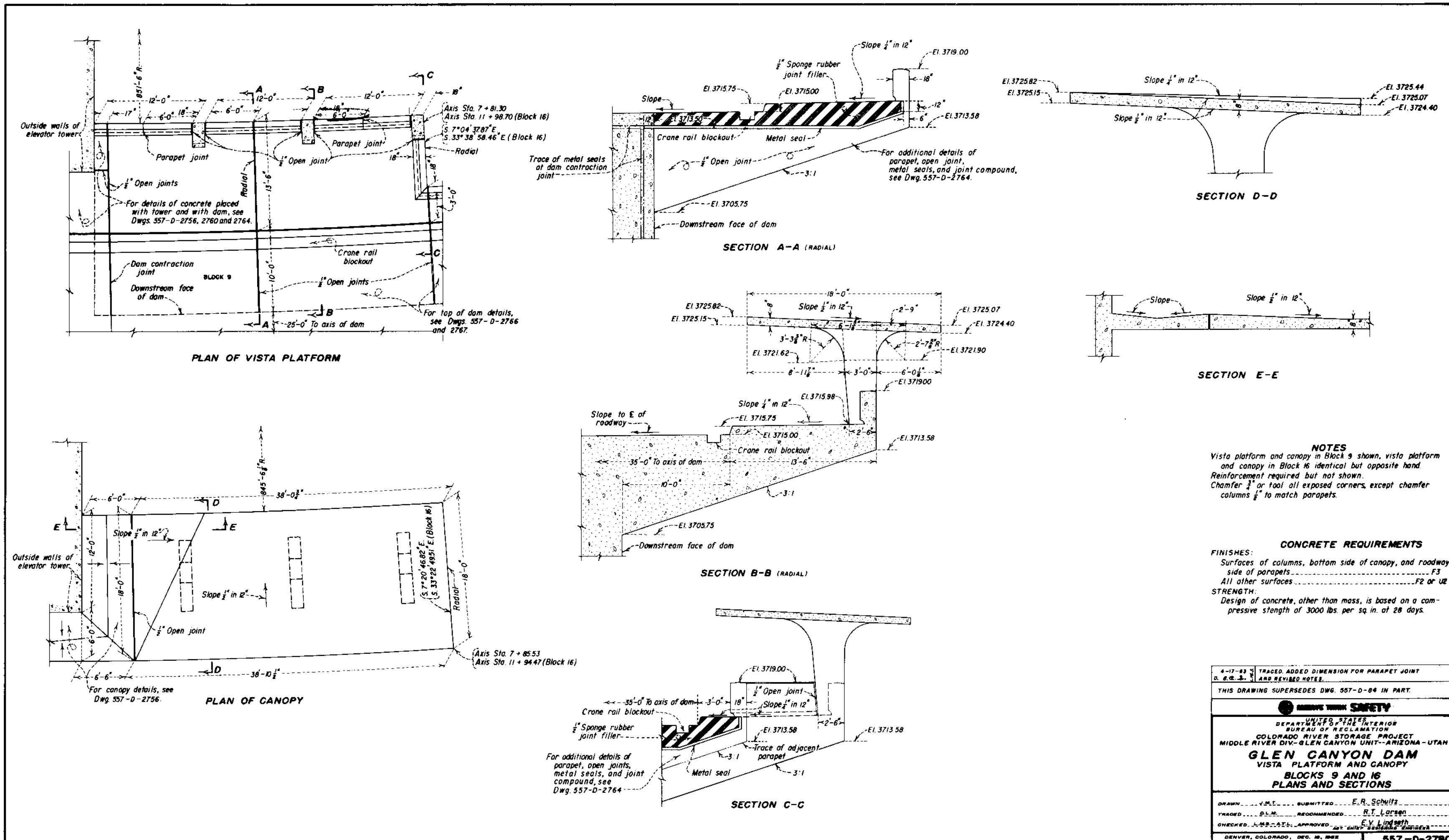


Figure 75.—Dam vista platform and canopy, blocks 9 and 16—Plans and sections.

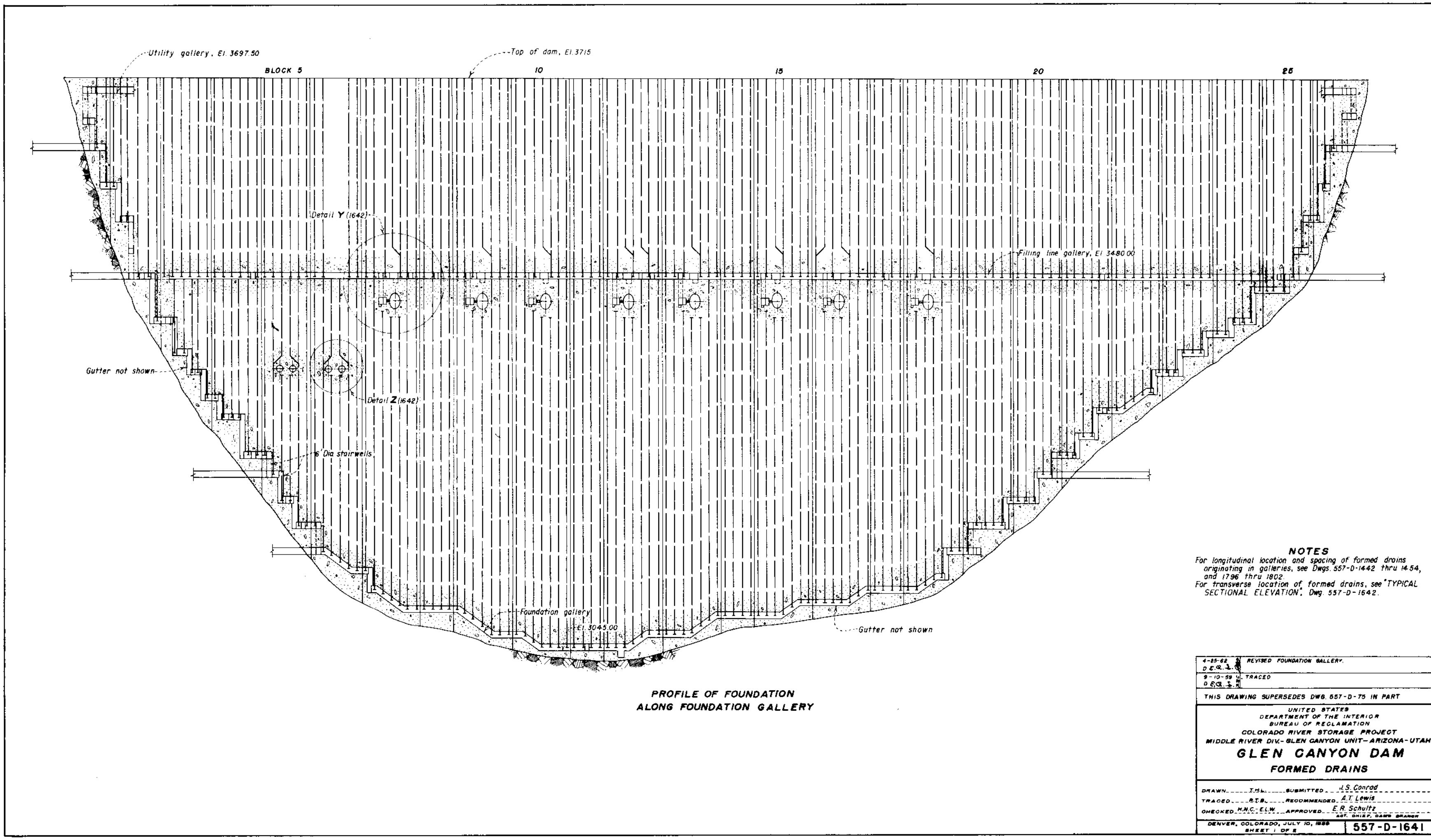


Figure 76.—Formed drains in dam. (Sheet 1 of 2.)

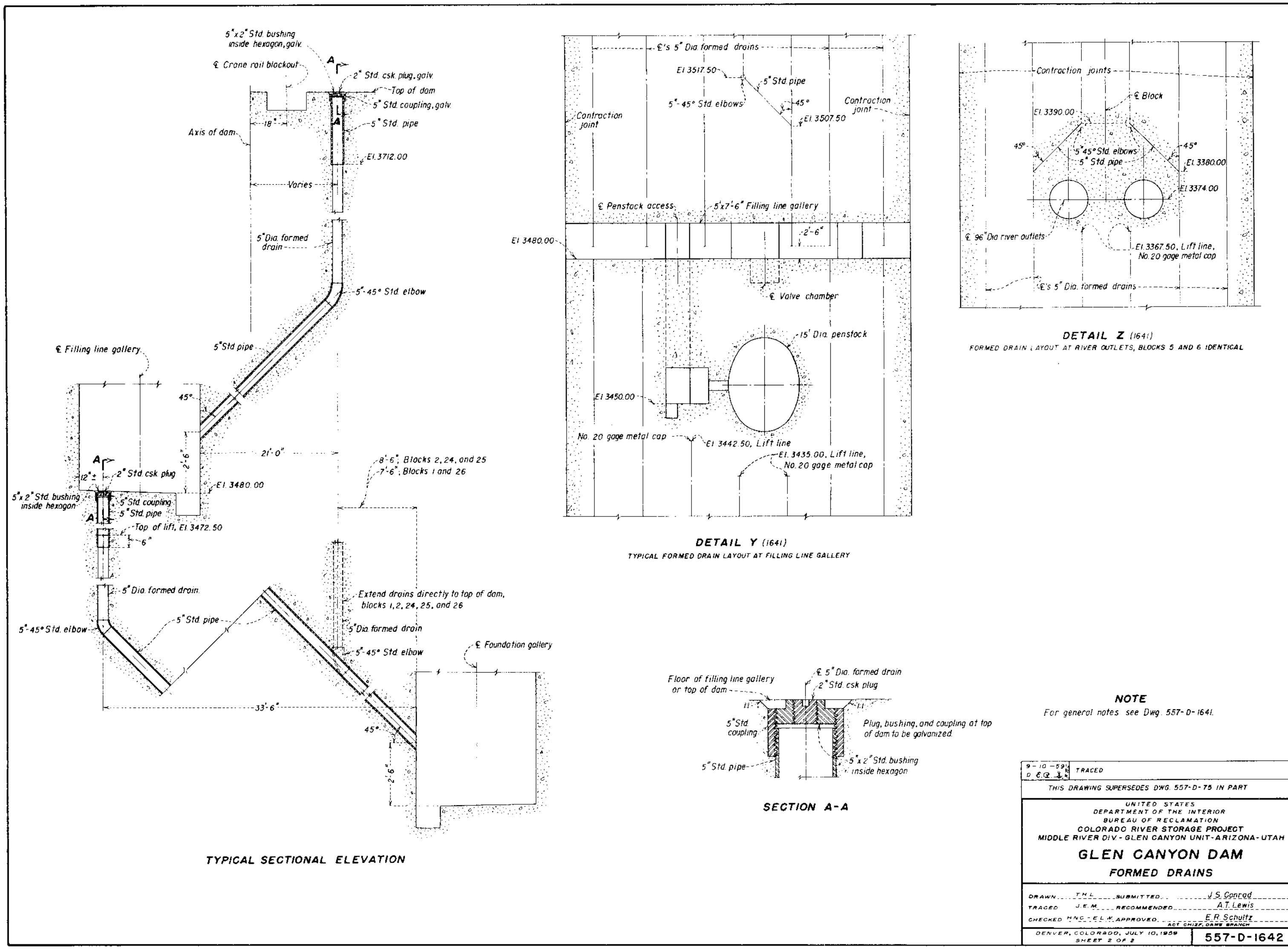


Figure 76.—Formed drains in dam. (Sheet 2 of 2.)

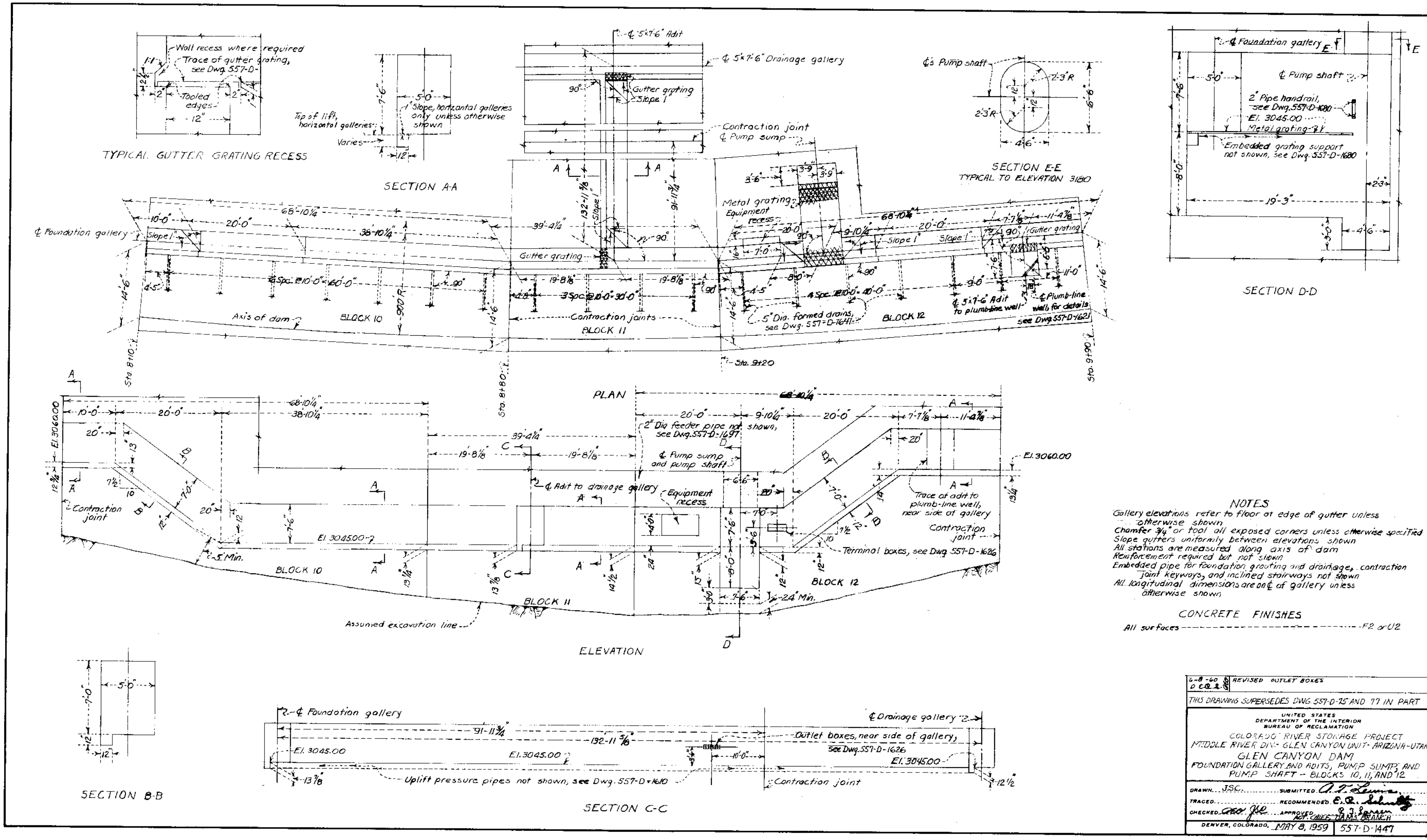


Figure 77.—Dam foundation gallery and adits, pump sump, and pump shaft—Blocks 10, 11, and 12.

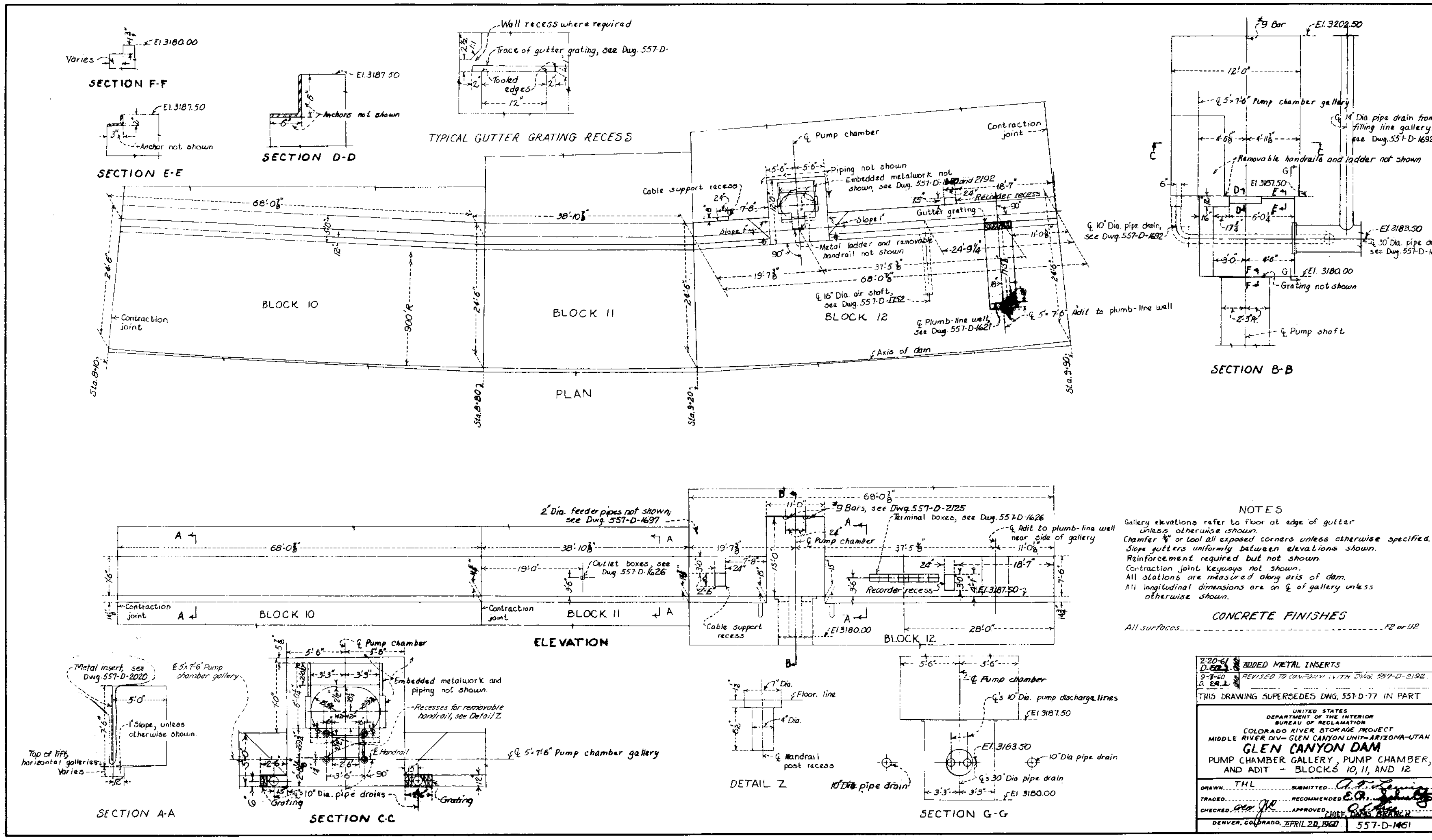


Figure 78.—Dam pump chamber gallery, pump chamber, and adit—Blocks 10, 11, and 12.

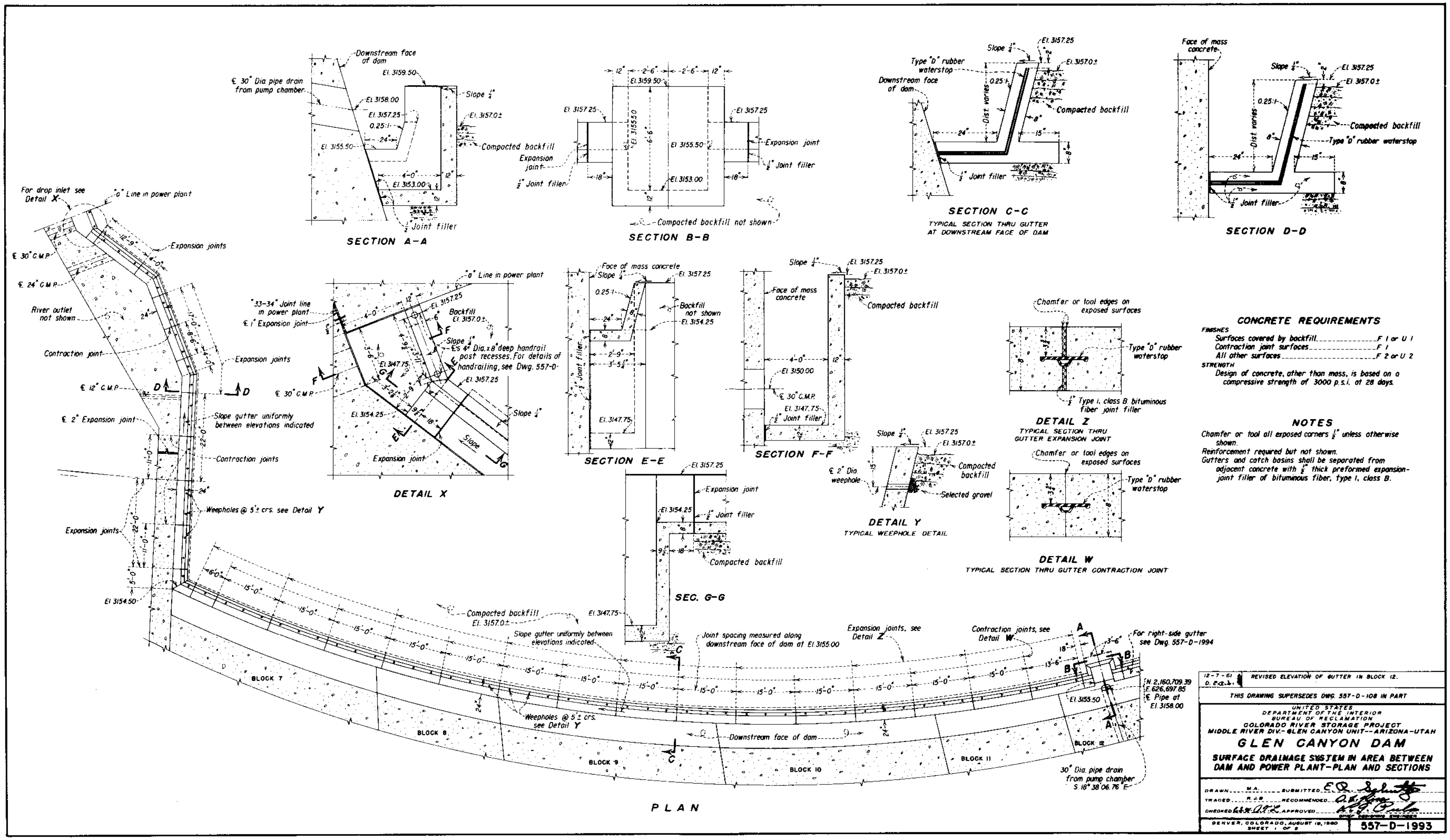


Figure 79.—Surface drainage system in area between dam and powerplant—Plan and sections. (Sheet 1 of 2.)

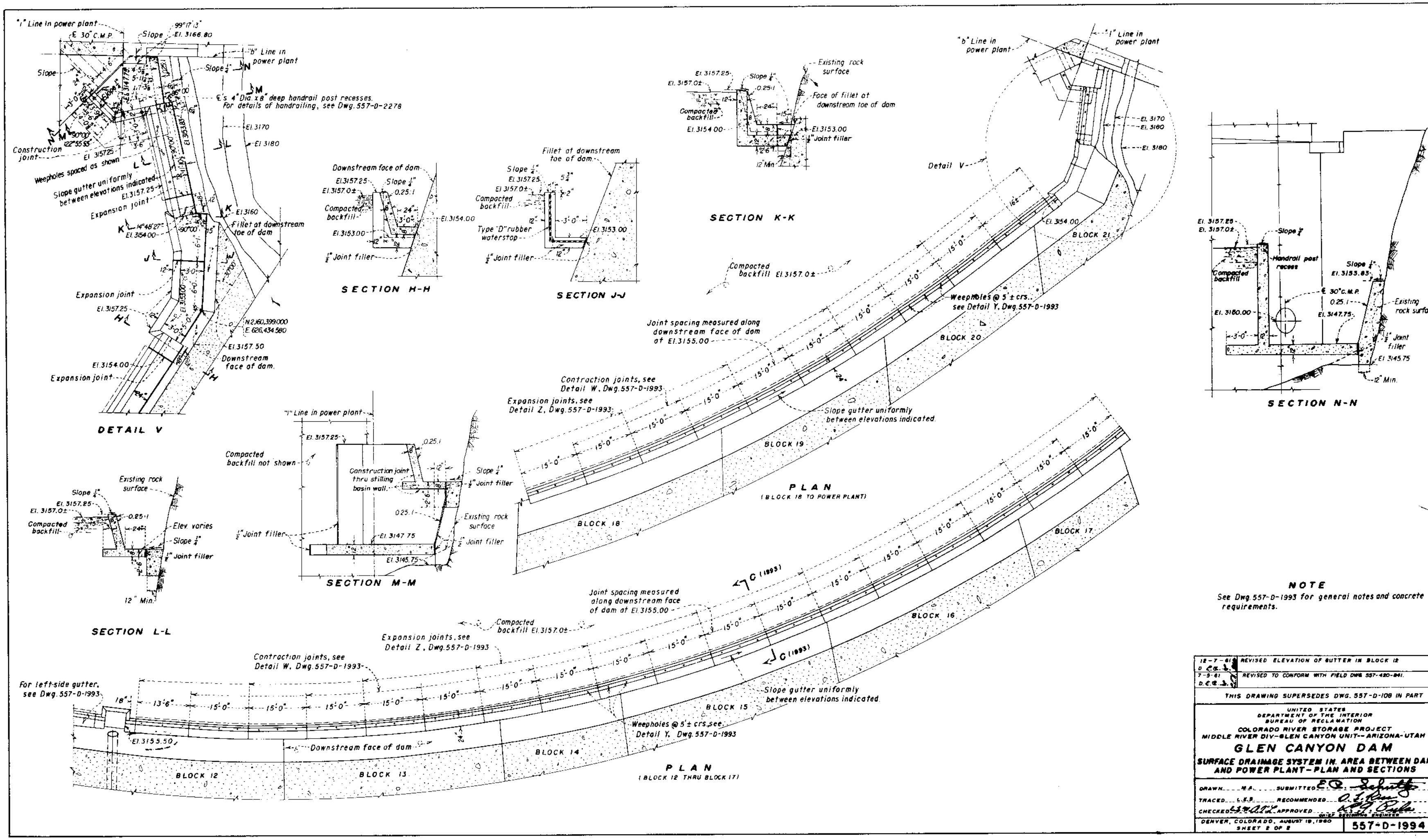


Figure 79.–Surface drainage system in area between dam and powerplant–Plan and sections. (Sheet 2 of 2.)

The surface drainage system in the backfill area between the dam and powerplant was designed for the following conditions:

(1) Drainage from the 30-inch pipe in the dam.

(2) Seepage through the foundation of the dam above elevation 3157.0.

(3) A 2-inch-per-hour rainfall at the dam area.

The water flows by gravity from the catch basin in block 12 to catch basins at the left and right abutments. The water flows from the catch basins to the tailrace area through 30-inch pipes (figs. 80, 81, and 82). The 30-inch diameter was specified to facilitate any required future cleaning.

The subdrainage system for the backfill between the dam and the powerplant is shown on figure 83. The purpose of the subdrainage system was to prevent the seepage past the dam from saturating the backfill up to the level of the penstocks. Also, the drainage system will provide subsurface moisture control for surface sprinkling required for any future landscaping of the area.

3. Structural Behavior Testing Apparatus

38. GENERAL. The designs of arch dams are made in accordance with accepted analytical methods using loads that will be applied to the structure and properties of the concrete from which the structure is built. In the design of any structure, certain assumptions are required to fulfill conditions of the analysis and certain factors are used that have been gained by experience from similar structures constructed in the past.

In order to determine the manner in which a dam behaves during the periods of reservoir filling and service operation, measurements are made on the structure to determine actual values of behavior criteria in terms of the strain, temperature, stress, deflection, and deformation of the foundation. Properties of the concrete from which the dam is constructed, such as temperature coefficient, modulus of elasticity, Poisson's ratio, and creep, are determined in the laboratory.

Knowledge of the behavior of Glen Canyon Dam may be gained by studying the service action of the dam using measurements of an external and an internal nature. Of primary importance is the information by which a continuing assurance of the structural safety of the dam can be gaged. Of secondary importance is information on structural behavior and the properties of concrete that may be used to give added criteria for use in the design of future concrete arch dams.

At Glen Canyon Dam, three general methods of measurement are used to gain this essential information; each method having a separate function in the overall scheme. One method of measurement involves four types of instruments that are embedded in the mass concrete of the structure. The second method involves two types of precise surveying measurements of an external nature. The other method involves two types of measurements of deformation of the rock of the foundation and abutments.

Data obtained from all three methods of measurements are correlated to determine the behavior of the structure.

The following subsections describe the layouts of the measurement systems and the locations and use of the various devices. Section 176 describes their installation and operation.

(a) *Embedded Instruments.*—The installation of embedded instruments consists of 1,142 strain meters, 60 stress meters, 264 jointmeters, and 74 resistance thermometers placed in the mass concrete of the dam and terminated by means of electrical cable connecting the instruments to 74 terminal boards and 116 outlet boxes located at some 60 appropriate reading stations in the system of galleries throughout the dam. The location of the instruments and details of their installation are shown on figures 84 through 91. Readings from the instruments are made periodically by means of special portable-type wheatstone bridge test sets. All data are recorded on a series of 18 appropriate data sheets.

Data supplied by the strain meters, stress meters, and jointmeters are in terms of total resistance of the meter and in terms of resistance ratio of the two coils contained in the meter. Data supplied by the resistance thermometers are in terms of resistance of the coil of the thermometer.

The strain meters, stress meters, and resistance thermometers embedded in the mass concrete of Glen Canyon Dam will furnish data over a long period of time for determining the stress behavior of the structure.

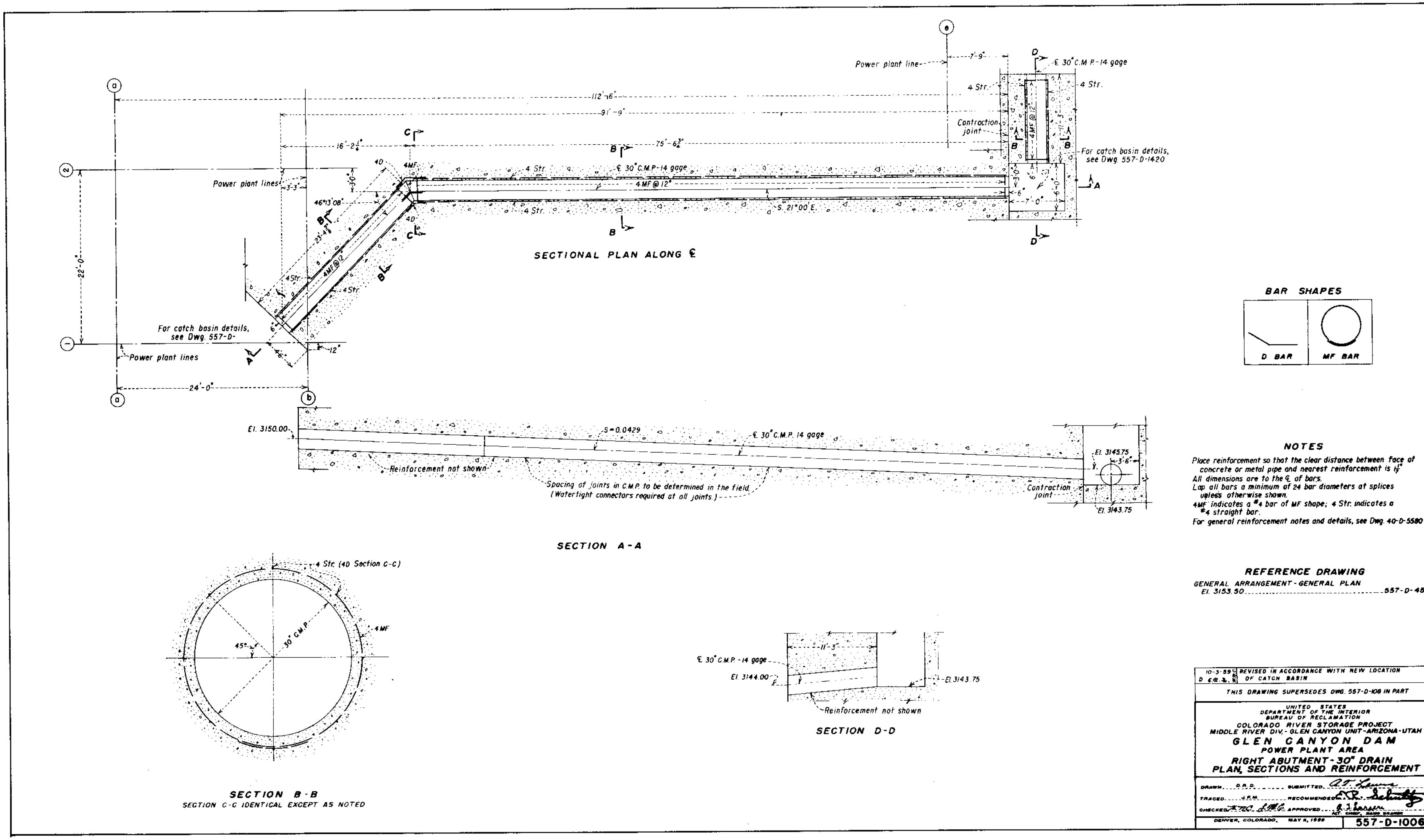


Figure 80.—Right abutment 30-inch drain in dam powerplant area—Plan, sections, and reinforcement.

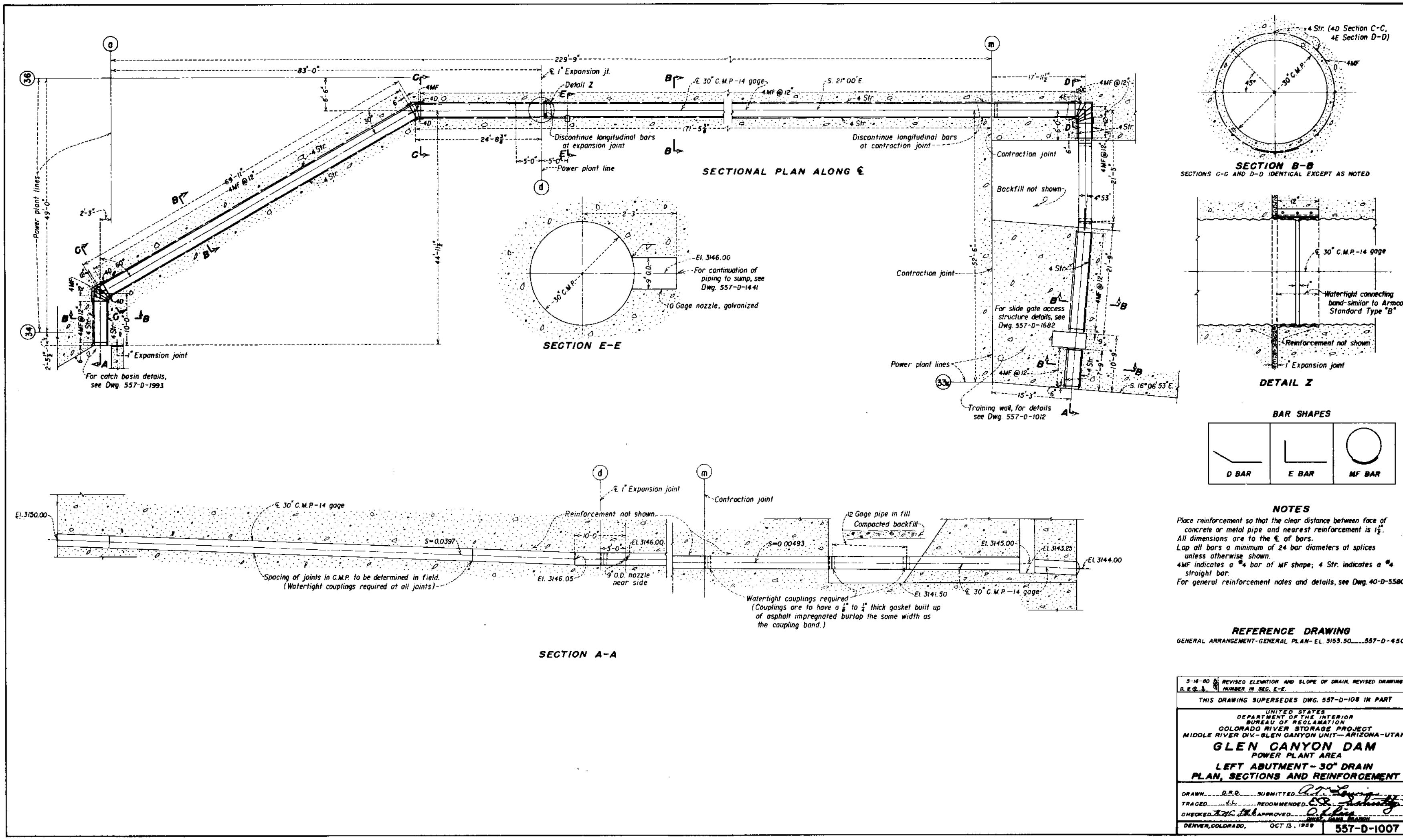


Figure 81.—Left abutment 30-inch drain in dam powerplant area—Plan, sections, and reinforcement.

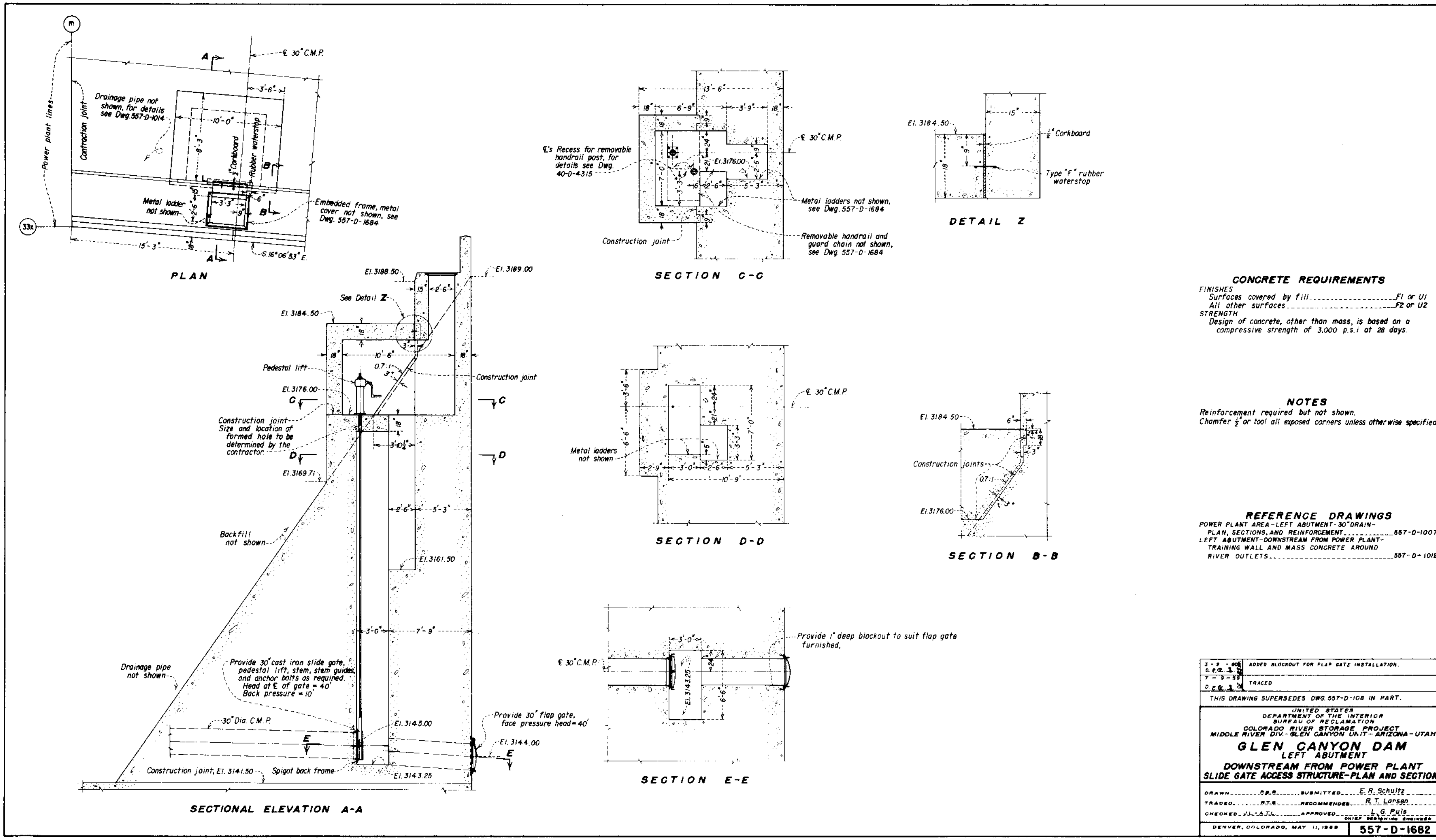


Figure 82.—Left abutment of dam downstream from powerplant—Plan and sections of slide gate access structure.

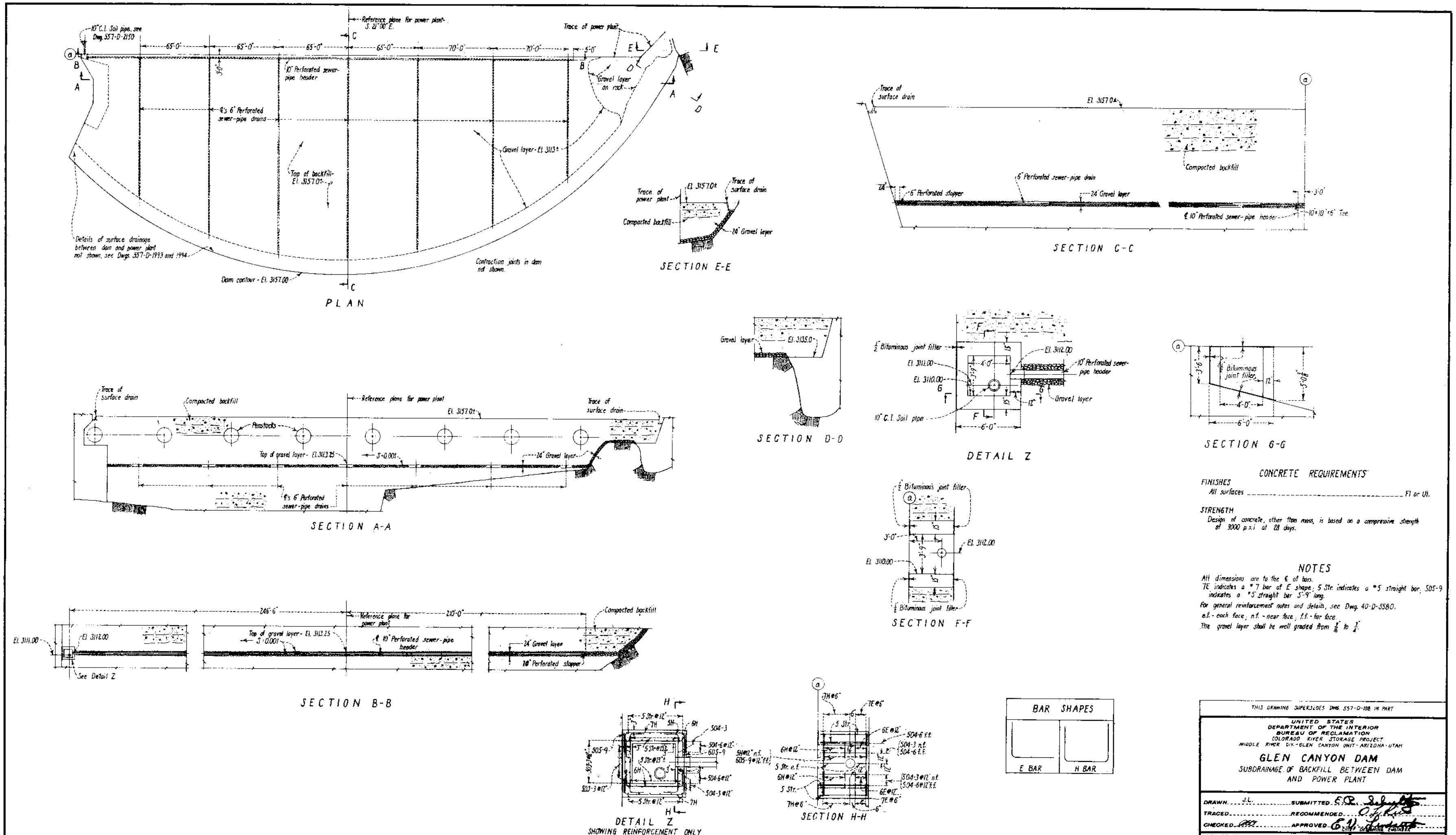


Figure 83.—Subdrainage of backfill between dam and powerplant.

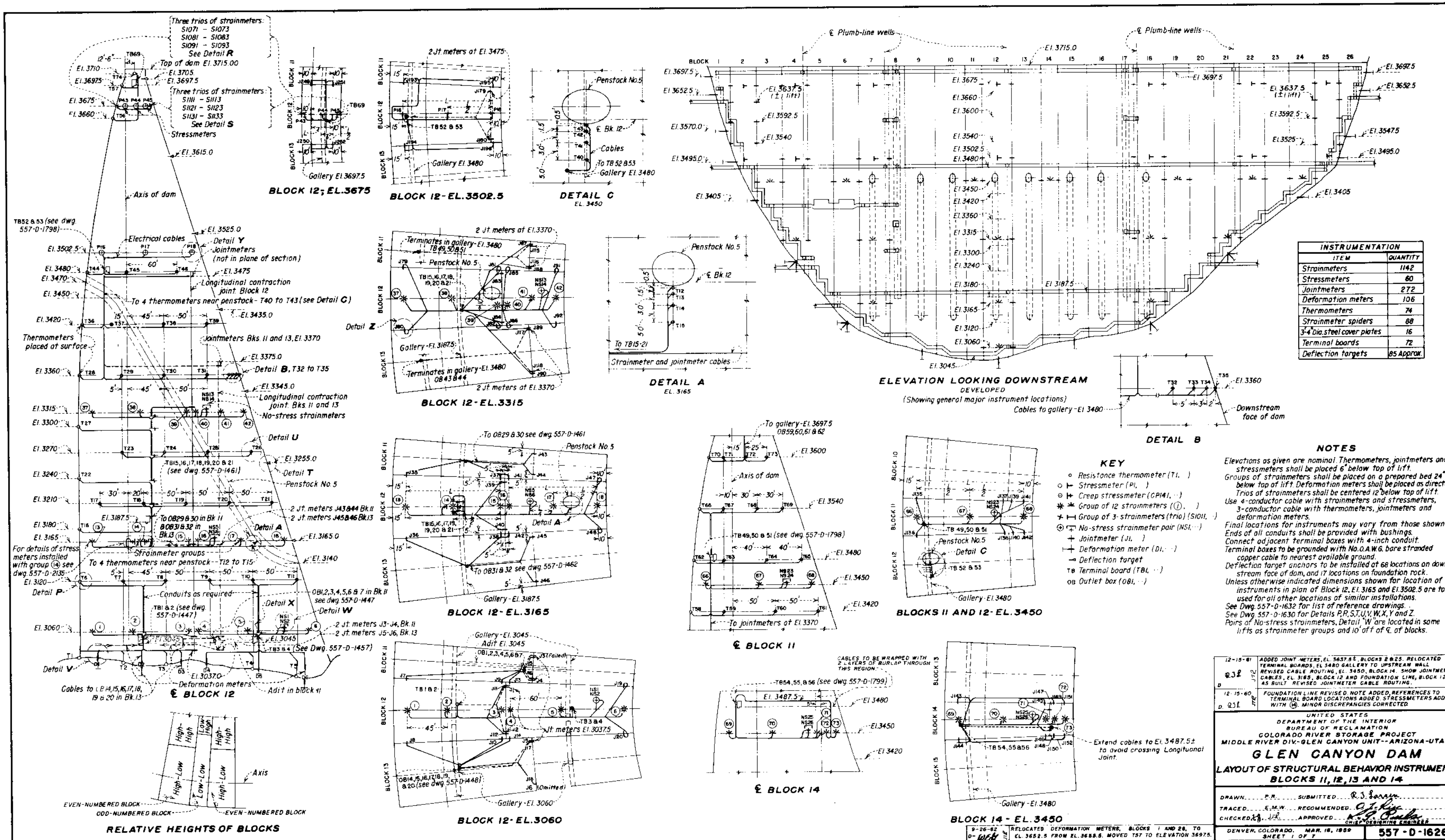


Figure 84.—Layout of structural behavior instruments in dam—Blocks 11, 12, 13, and 14.

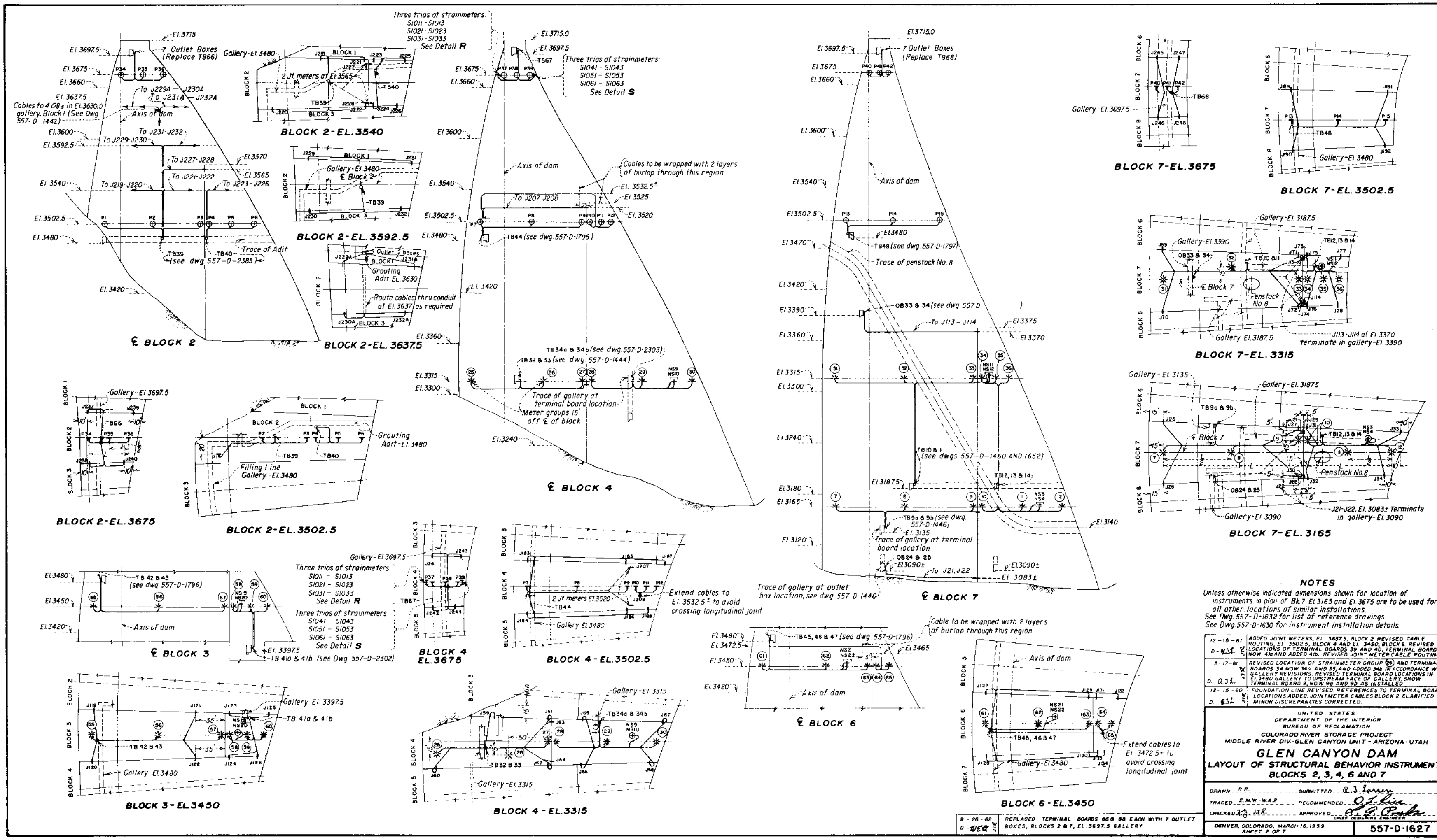


Figure 85.—Layout of structural behavior instruments in dam—Blocks 2, 3, 4, 6, and 7.

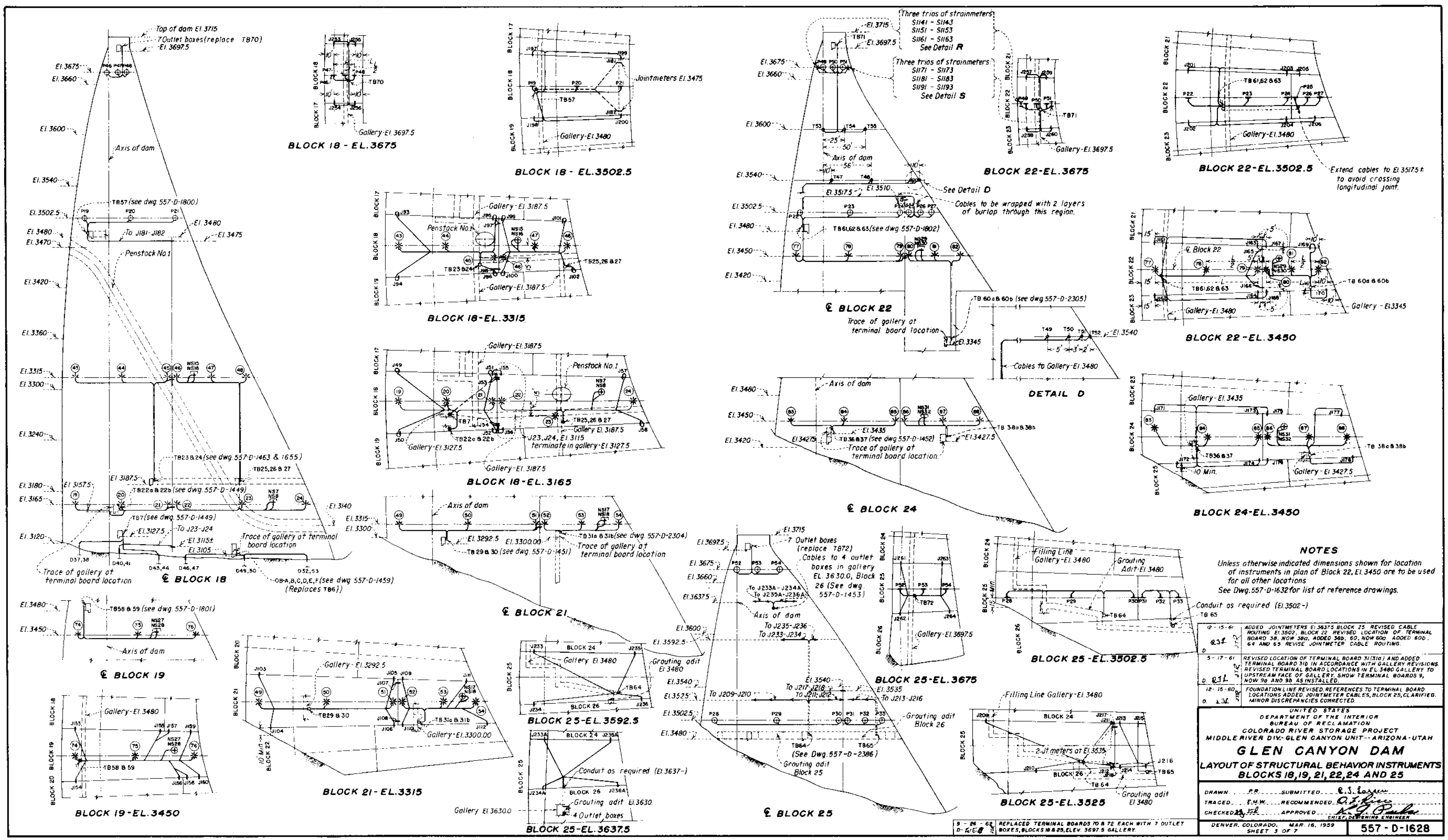


Figure 86.—Layout of structural behavior instruments in dam—Blocks 18, 19, 21, 22, 24, and 25.

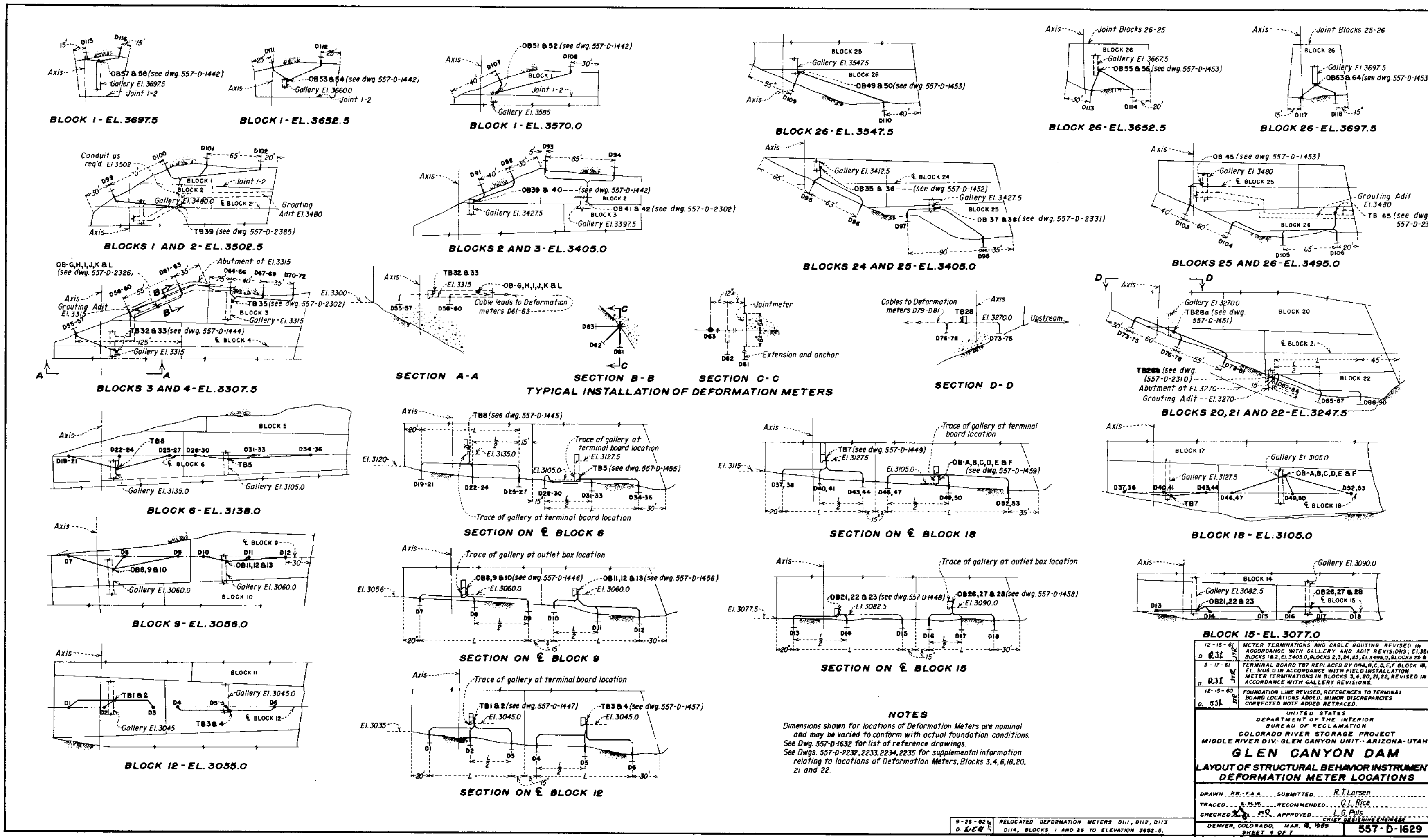


Figure 87.—Layout of structural behavior instruments in dam—Deformation meter locations.

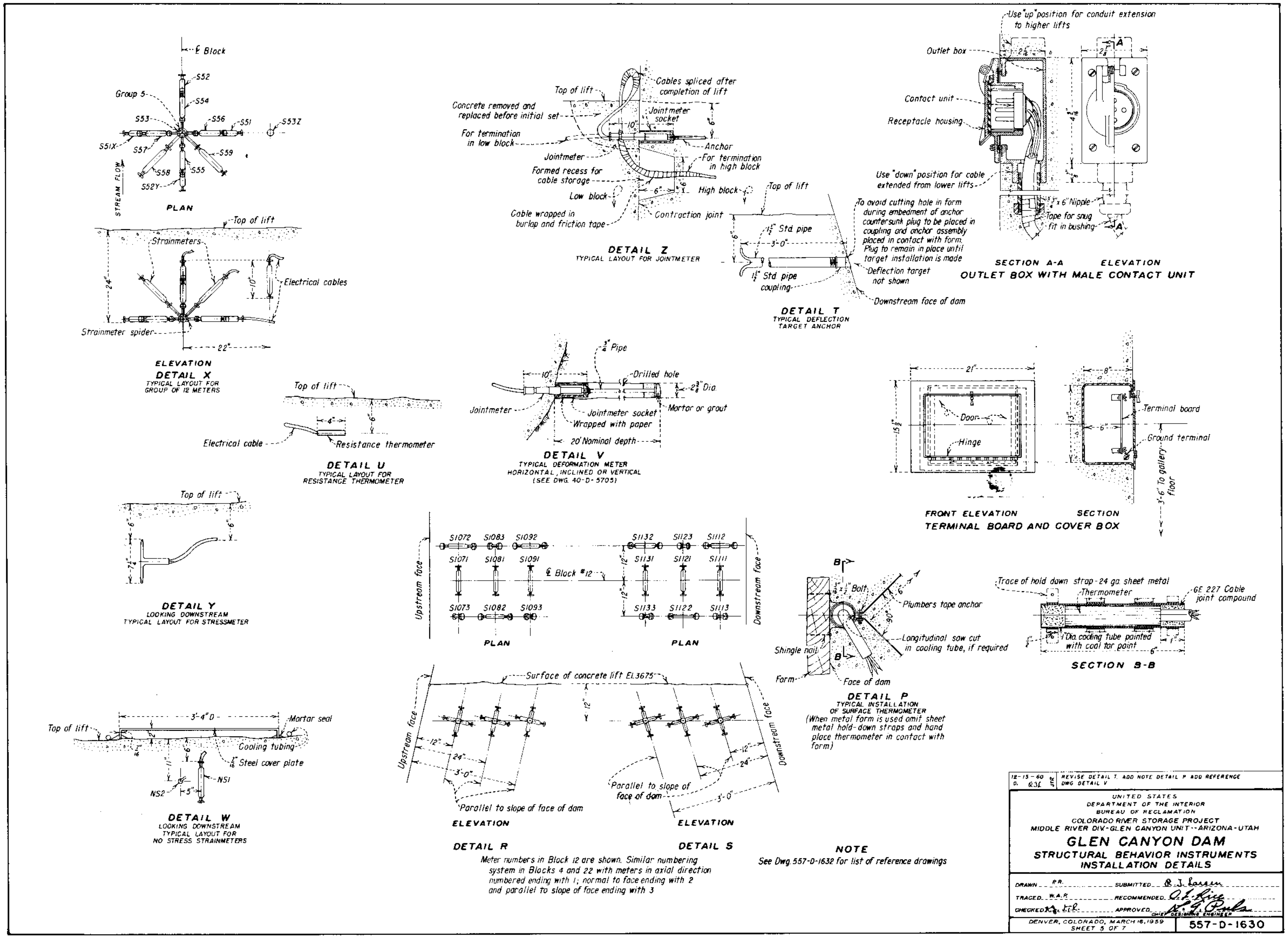


Figure 88.—Dam structural behavior instruments—Installation details.

Reading Station	Terminal Board Nos.	Outlet Box Nos.	Location Block - Elev.	No & Size Conduits	No & Size Nipples	Strainmeters — Group and No-Stress	Stress Meters	Deformation Meters	Jointmeters	Thermometers	Board Term.	Total Term.
57		63,64	26-3697.6		2-3/4"			D117,D118				2
56		97-103	25-3697.5		7-3/4"		P52-P54		J261-J264		7	7
55	71		22-3697.5		2-3½"	S1141-S1143, S1151-S1153, S1161-S1163, S1171-S1173, S1181-S1183, S1191-S1193	P49-P51		J257-J260	T53-T55	28	28
54		87-93	18-3697.5		7-3/4"		P46-P48		J253-J256		7	7
53	69		12-3697.5	1-3/4"	2-3½"	S1071-S1073, S1081-S1083, S1091-S1093, S1111-S1113, S1121-S1123, S1131-S1133	P43-P45		J249-J252	T56,T57,T74	28	28
52		59,60,61,62	11-3697.5		4-3/4"					T70-T73		4
51		80-86	7-3697.5		7-3/4"		P40-P42		J245-J248			7
50	67		4-3697.5		1-4"	S1011-S1013, S1021-S1023, S1031-S1033, S1041-S1043, S1051-S1053, S1061-S1063	P37-P39		J241-J244		25	25
49		73-79	2-3697.5		7-3/4"		P34-P36		J237-J240			7
48		57,58	1-3697.5		2-3/4"			D115,D116				2
47		55,56	26-3667.5		2-3/4"			D113,D114				2
46		53,54	1-3660	2-3/4"				D111,D112				2
45		51,52	1-3585		2-3/4"			D107,D108				2
44		49,50	26-3547.5		2-3/4"			D109,D110				2
43	65		26-3480	2-2", 1-1½"			P31-P33	D104-D106	J213-J216		10	10
42	64		25-3480 US	3-2", 1-1½"			P28-P30	D103	J209-J212, J217, J218, J233-236		14	14
41												
40	63		22-3480 US	1-3½"	1-4"				J201-J206	T47-T52	12	99
	62		22-3480 US	1-2½"	1-4"	S791X-S799	P22-P24		J161-J168		22	
	61		22-3480 US		1-4"	(77),(78), S791					25	
	60A,(60B)	[44A]	22-3345 DS		2-4", 1-3/4"	(80),(81), S821-S823Z, (S824-S829, NS29, NS30)			(J169),(J170)		30,(10)	
39	59		19-3480		1-4"	S761X-S769, NS27, NS28			J153-J159		20	45
	58		19-3480		1-4"	(74),(75), S761					25	
38	57		18-3480	1-3"	1-1½"		P19-P21		J181, J182, J197-J200		9	9
37	56		14-3480		1-4"	S734-S739, NS25, NS26			J143-J152		18	72
	55		14-3480		1-4"	(71),(72), S731-S733Z					30	
	54		14-3480		1-4"	(69),(70)					24	
36		43,44	13-3480 US		2-3/4"				J117, J118		2	2
35												
	52		12-3480		1-3½"		P16-P18		J179, J180, J193-J196	T28-T46	28	28
34												
	50		11-3480		2-3½"	S684-S689, NS23, NS24			J115, J116, J135-J142	T58-T69	30	60
	49		11-3480		1-3½"	(66),(67), S681-S683Z					30	
33	48		7-3480	1-3"			P13-P15		J189-J192		7	7
32	47		6-3480		1-4"	(65)			J127-J134		20	70
	46		6-3480		1-4"	(63),(64), S629					25	
	45		6-3480		1-4"	(61), S621-S628, NS21, NS22					25	
31	44		4-3480 US	1-3½", 1-1½"			P7-P12		J183-J188, J207, J208		14	14
30	43		3-3427.5 US		1-4"	(55),(56), S578, S579			J119-J122		30	82
	42		3-3427.5 US		2-4"	S571-S577					10	
	41A,(41B)		3-3397.5 DS		2-4"	(59),(58), ((60) NS19, NS20)			(J123, J126)		24,(18)	
29	40		2-3480 DS	2-2"			P4-P6		J223-J226		7	7
28	39		2-3480 US	2-3", 1-2"			P1-P3	D99-D102	J219-J222, J227-J232		17	17
27	38A,(38B)		24-3427.5 DS	2-4"		(84), S871-S875, ((85) S876-S879, NS31, NS32)			(J177, J178)		20,(20)	92
	37		24-3435 US	2-4"		(86), S846-S849			J171-J176		22	
	36		24-3435 US	1-4"		(83), S841-S845					20	
26		39,40,(41,42)	2-3427.5 US (2-3439.5)		4-3/4"			D91, D92, (D93, D94)				2,(2)
25		35,36,(37,38)	24-3412.5 US, (25-3427.5)		4-3/4"			D95, D96, (D97, D98)				2,(2)
24		33,34	7-3390 US		2-3/4"				J113, J114			2
23 c	35	G-L	3-3315 (US) & DS		1-4", 6-3/4"			(D58-D63) D64-D72			(6), 9	102
b	34A,(34B)		4-3315 DS	2-4"		(28), S291-S295, ((29) S296-S299, NS9, NS10)			(J65-J68)		20,(22)	
a	33		4-3315 US	2-4"		S271X-S279		D55-D57	J59-J64		20	
	32		4-3315 US	1-4"		(25),(26), S271					25	
22	31A,(31B)		21-3300 DS	2-4"		(52),(53), ((54) NS17, NS18)			(J109-J112)		24,(18)	84
	30		21-3292.5 US	2-4"		(51)			J103-J108		18	
	29		21-3292.5 US	1-4"		(49),(50) †					24	
21	28A,(28B)		21-3247.5 US, (3270 DS)		2-4"			D73-D81 (D82-D90)			9,(9)	18
20	27		18-3187.5 DS	2-4"		(24), NS7, NS8, NS15, NS16			J57, J58, J99-J102		22	82
	26		18-3187.5 DS	1-4"	1-4"	(22),(23), S484-S489					30	
	25		18-3187.5 DS		1-4"	(46),(47), S481-S483Z					30	
19	24		18-3187.5 US	2-4"		S454-S459			J93-J98		12	86
	23		18-3187.5 US	1-4"	1-4"	(43),(44), S451-S453Z					30	
	22A,(22B)		18-3187.5 US		2-4"	(19),(20), S211, (S211X-S219)			(J49-J56)		25,(19)	
18		31,32, 32A THRU 32C	13-3187.5 US		5-3/4"				J36, J38, J42, J45, J46			5
17	21		12-3187.5 US	2-4"	1-2½"					T19-T27	9	188
	20		12-3187.5 US	1-4", 2-1"		(42), NS13, NS14			J79-J92	T17, T18	30	
	19		12-3187.5 US	2-4", 1-2"		(40),(41), S394-S399					30	
	18		12-3187.5 US	2-4", 1-2"		(37),(38), S391-S393Z	CP141, CP141X				30	
	17		12-3187.5 US		2-3½"	(18), NS5, NS6	CP142, CP142Y		J39, J40, J47, J48	T12-T16	29	
	16		12-3187.5 US		2-3½"	(16),(17), S154-S159	CP143, CP143Z				30	
	15		12-3187.5 US		2-3½"	(13),(14), S151-S153Z					30	
16		29,30, 30A THRU 30C	11-3187.5 US		5-3/4"				J35, J37, J41, J43, J44			5
15	14		7-3187.5 DS	2-4"		(36), NS3, NS4, NS11, NS12			J31-J34, J75-J78		24	84
	13		7-3187.5 DS	1-4"	1-4"	(34),(35), S124-S129					30	
	12		7-3187.5 DS		1-4"	(10),(11), S121-S123Z					30	
14	11		7-3187.5 US	2-4"		S334-S339			J69-J74		12	84
	10		7-3187.5 US	1-4"	1-4"	(31),(32), S331-S333Z					30	
	9A,(9B)		7-3135.0 US		2-4"	(7),(8), S91-S93Z, (S94-S99)			(J25-J30)		30,(12)	
13	8		6-3135 US		1-2½"			D19-D27			9	9
12	7		18-3112.5 US		1-3"			D37-D45*	J23, J24		8	8
11		A-F	18-3105 DS		6-3/4"			D46-D54*				6
10	5		6-3105 DS		1-2½"			D28-D36			9	9
9		26,27,28	15-3090 DS		3-3/4"			D16, D17, D18				3
8		24,25	7-3090 US		2-3/4"				J21-J22			2
7		21,22,23	15-3082.5 US		3-3/4"			D13, D14, D15				3
6		14,15,16,17,18,19,20*	13-3060 US	3-3/4"	3-3/4"				J5, J6*, J8, J10, J14, J17, J18			6
5		11,12,13	9-3060 DS		3-3/4"			D10, D11, D12				3
4		8,9,10	9-3060 US		3-3/4"			D7, D8, D9				3
3	4		12-3045 DS	1-4", 1-2"	1-2"	(8), NS1, NS2		D4, D5, D6	J19, J20	T4, T5, T9-T11	24	48
	3		12-3045 DS	1-4"		(4),(5)					24	
2	2		12-3045 US	1-4", 1-2"	2-2"	(3)		D1, D2, D3	J1, J2, J11, J12	T1-T3, T6-T8	25	49
	1		12-3045 US	1-4"		(1),(2) †					24	
1		1,2,3,4,5,6,7	11-3045 US	5-3/4"	2-3/4"				J3†, J4, J7, J9, J13, J15, J16			7

* OB20 Not installed DS Indicates downstream block
US Indicates upstream block

* D39, D42, D45, D48, D51, D54 Not installed †D3 Failed *J6 Not installed

NOTES

Conduits enter top and nipples enter bottom of cover boxes.
All cover boxes interconnected by 4" conduit.
† Ringed numbers indicate groups of 12 strainmeters as shown in Detail "X" Dwg. 557-D-1630.
See Dwg. 557-D-1632 for list of reference drawings.

1-30-67 AS BUILT
12-15-60 ADD STRESS METERS CP141--CP143Z DEFORMATION METERS D39, D42, D45, D48, D51, D54 NOT INSTALLED

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT--ARIZONA-UTAH
GLEN CANYON DAM
INSTRUMENT TERMINATIONS

DRAWN ... SUBMITTED ...
TRACED ... RECOMMENDED ...
CHECKED ... APPROVED ... CHIEF DESIGNING ENGINEER

DENVER, COLORADO, MARCH 16, 1960 SHEET 6 OF 7 — 557-D-1631

Figure 89.—Dam instrument terminations.

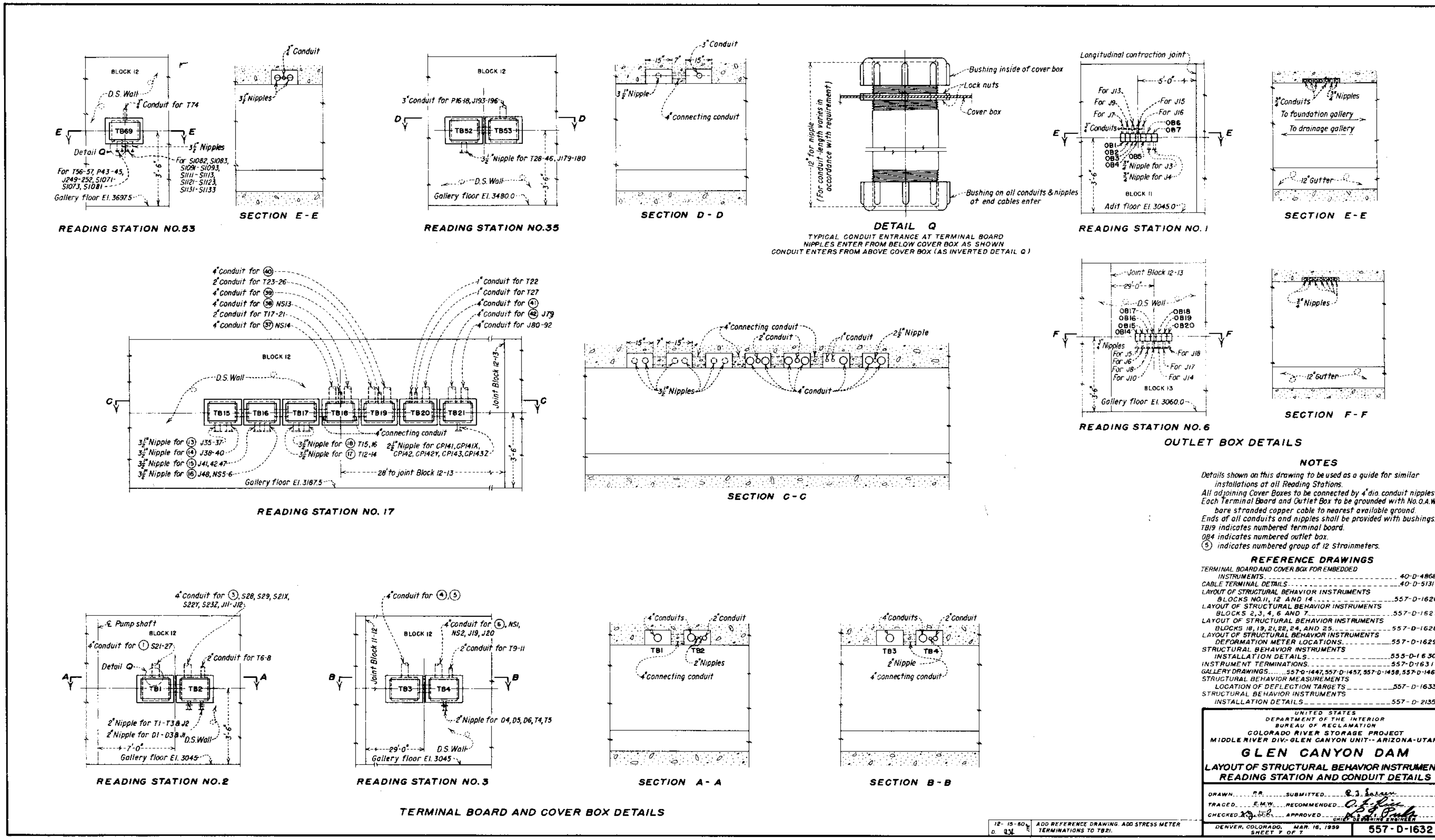


Figure 90.—Layout of structural behavior instruments in dam—Reading station and conduit details.

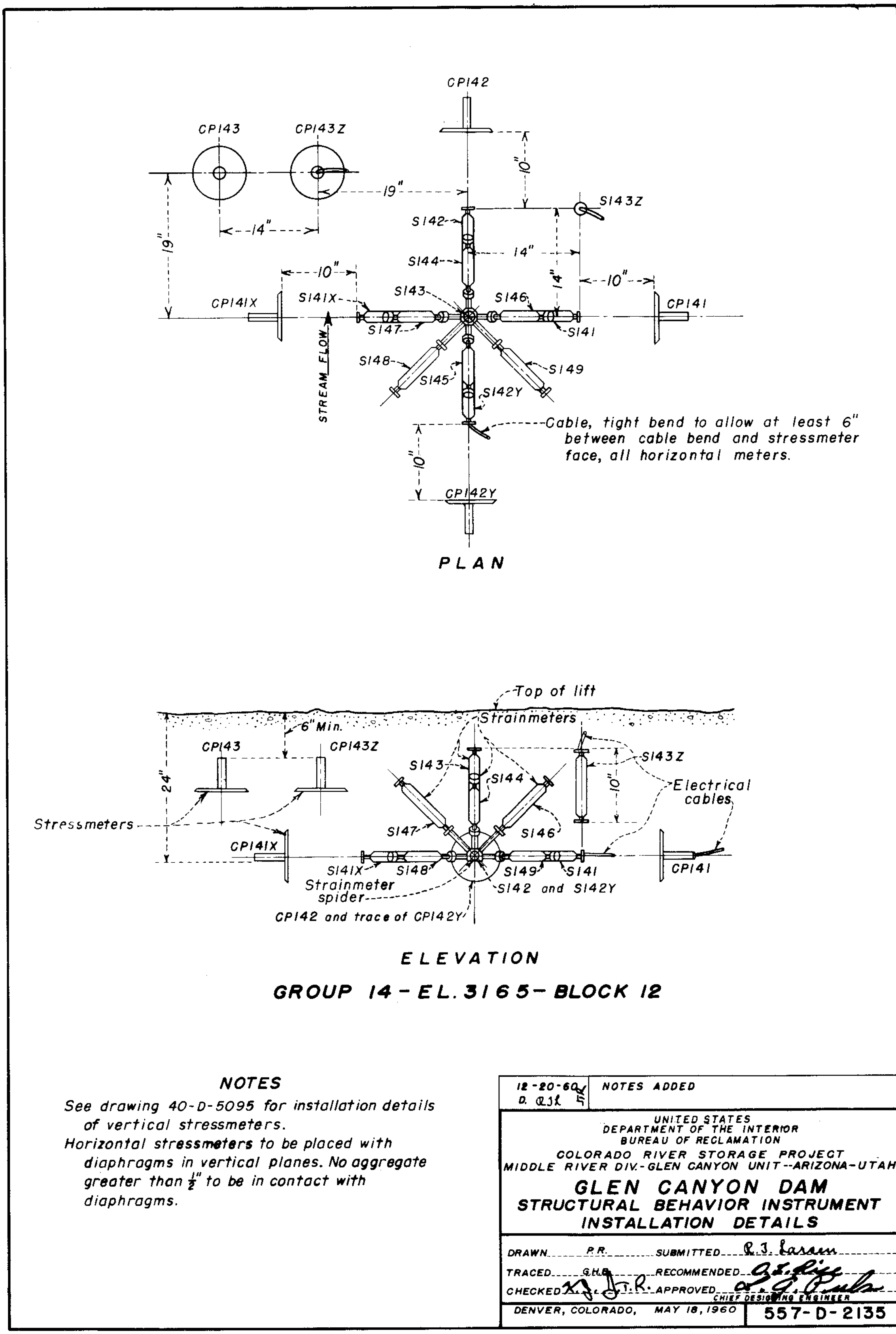


Figure 91.—Dam structural behavior instrument installation details.

The jointmeters detect the condition of joint opening until the joints are grouted, and after that time serve as the means of determining the effectiveness of the grouted joint.

The strain meter installation ranks first in numbers and importance. On 16 radial lines through the dam, groups of strain meters in three-dimensional configuration are embedded in the mass concrete to measure volume changes from which the stresses can be computed. The strain meters also measure concrete temperature at their locations. The groups of strain meters on the radial lines are installed near the base of the maximum section of the dam and in arch elements at three elevations in the dam. Each group of strain meters comprises 12 instruments installed as a cluster. Each radial line of strain meter groups contains one pair of "no-stress" strain meters. The patterns of radial lines are generally on the centerlines of blocks and were selected to define a system of horizontal and vertical planes that represent approximate arch and cantilever elements on which stress information is obtained by the trial-load design analysis of the dam. Stress information at a number of arch-cantilever intersections common to both methods of investigation is thus available.

Cantilever and arch stresses are determined at the base of the maximum section where maximum cantilever stresses are to be expected. Similarly the stresses are determined at sections in three arches which are about uniformly spaced through approximately the lower two-thirds of the dam and at elevations where the distributions of arch and cantilever stresses are desired. To obtain stress distribution in each of the sections, the groups of strain meters are placed at six locations between the upstream and downstream faces of the dam at the base of the maximum section in block 12 at elevation 3060; in blocks 7, 12, and 18 at elevation 3165; and in blocks 4, 7, 12, 18, and 21 at elevation 3315. At elevation 3450 six groups are installed, similarly, in each of blocks 3, 22, and 24; five groups in each of blocks 6 and 14; and three groups in each of blocks 11 and 19. Where six groups are installed, three are upstream and three downstream from the longitudinal joint. Where five groups are installed, three are upstream and two downstream from the longitudinal joint. Where three groups are installed, there is no longitudinal joint. The strain meter groups record length changes which are used in the computation of true structural stresses.

In the stress analysis, data are required regarding the volume changes in the concrete that take place in the absence of stress. "No-stress" strain meters are installed to supply this information. These strain meters are installed under a free surface in the interior of the dam so that the instruments are not affected by vertical loads.

Since temperature change is one of the most important contributory factors to the internal stresses in an arch dam, an array of resistance thermometers is installed in a grid pattern near the maximum section, block 12 below elevation 3480, to record this factor. A similar grid pattern of resistance thermometers is installed in block 11 between elevations 3420 and 3600, and in block 22 at elevations 3540 and 3600. In addition, resistance thermometers spaced at equal intervals from the base to the top of the dam at the upstream face of the dam in blocks 11 and 12 record lake temperatures at various depths.

Provision for measurement of arch stresses in two arch elements at elevation 3502.5 and elevation 3675 was made by installing stress meters at those elevations. Six stress meters were installed on the centerlines of each of blocks 2, 4, 22, and 25, and three on the centerlines of each of blocks 7, 12, and 18, all at elevation 3502.5. Three stress meters were installed on the centerlines of each of blocks 2, 4, 7, 12, 18, 22, and 25 at elevation 3675.

An array of six stress meters is installed in conjunction with a strain meter group at an interior location in the mass concrete of block 12, elevation 3165. Pairs of the stress meters are placed on the same three major orthogonal axes as strain meters to form a three-dimensional configuration for investigating the strain-stress relation at a location in mass concrete.

At elevations 3165 and 3450, block 12, four thermometers are placed at varying distances from penstock 5 to determine the temperature gradient in the concrete due to the temperature of water flowing through the penstock. Similar installations of four thermometers each at elevation 3360, block 12, and elevation 3540, block 22, near the downstream face of the dam, are to determine the temperature gradients at varying distances beneath the concrete surface due to air temperature and solar radiated heat.

Trios of mutually perpendicular strain meters are installed near the upstream and downstream faces of the dam at elevation 3675 in blocks 4, 12, and 22, to determine strain gradients near the surfaces.

In conjunction with the installations of strain meter groups and stress meter arrays at the various locations throughout the dam, jointmeters are placed on the radial contraction joints, and on the longitudinal contraction joints where they exist, at the same elevations as the strain meters and the stress meters. Two jointmeters were installed across and near the upper ends of the longitudinal contraction joints in each of blocks 2, 4, 12, 18, and 25.

Three mutually perpendicular strain meters were mounted on the rock of the top and downstream wall of the control cable tunnel in the west canyon wall downstream from the right abutment of the dam at approximately elevation 3325. Those strain meters are parallel to and across the "A" joint in the rock formation. The strain meters are for the purpose of measuring relative movements of the rock on opposite sides of the "A" joint.

Strain meters, stress meters, jointmeters, and deformation meters are all of the unbonded resistance wire type and are all modifications of the basic Carlson strain meter. Those meters are read electrically, using special wheatstone bridge test sets, from reading stations, most of which are in the galleries of the dam. Embedded insulated electrical cables extend from the meters to terminations at the reading stations. Embedded resistance thermometers are likewise connected to similar terminations and are read electrically from the same reading stations.

(b) *Observation of Dam Movement by Surveying Methods.*—The two types of precise surveying measurements used for measuring the structural deformation of the dam comprise: (1) a system for horizontal angular measurements made by a first order theodolite from six piers located downstream from the dam and from two auxiliary piers on the dam to targets on the dam and on the abutments, and to three deformation points in wells near the toe of the dam; and (2) measurements of deflection from five plumblines installed in formed wells that extend from near the foundation to the top of the dam. Measurements from the targets and measurements from the plumblines are correlated. A periodic check is made over the deformation points and the system of targets which are installed in a grid pattern at 68 points on the downstream face of the dam and at 17 points on the abutments. The grid pattern of targets defines arch and cantilever elements of the dam.

Periodic observations of the targets and deformation points are made from the three pairs of piers which are located at three elevations on the abutments and downstream from the dam. An additional pair of theodolite piers located farther downstream complete quadrilaterals with the six piers from which measurements to the targets are made. The layout of the system of targets and deformation points, and the layout of the system of piers, are shown on figures 92 and 93, respectively.

Using each pair of theodolite piers, accurate angular measurements are made between each target on the downstream face of the dam and the opposite theodolite pier. The auxiliary theodolite piers on the toe of the dam are used to make measurements on the targets on the abutments. All of these measurements are related by triangulation to measured baselines located on the abutments downstream of the dam and to computed baselines between pairs of piers. Differences between the successively measured locations of each target furnish the path of the target as it moves due to the deformation of the dam. When the paths of the targets are projected to the coordinate system for the dam, they may be resolved into components of deflection that are radial and tangential to the axis of the dam. Measured angular and linear data are recorded on a series of appropriate data sheets.

(c) *Plumblines.*—Measurements of a nature similar to those obtained from the surveying measurements method are the measurements of deflection that are obtained from the five plumblines installed in the dam. The plumblines are installed in 12-inch-diameter formed vertical wells in blocks 4, 7, 12, 18, and 21, as shown on figure 84. Reading stations in the galleries are provided on the lower end of each plumbline near the foundation, and at intermediate elevations, as shown on figure 94.

Each plumbline consists of a single strand of 0.030-inch-diameter, stainless steel wire suspended from an aluminum spider and stainless steel holding chuck in an airtight enclosure below the roadway and at the upper end of the line. The airtight enclosures are accessible through watertight manhole covers. Each plumbline supports a 26-pound cylindrical weight at the bottom. At the lowest reading station on each plumbline, a container of oil is provided for damping vibrations of the suspended mass. At the reading stations, doorframes are set in the concrete of the gallery wall and doors seating against sponge rubber seals are provided as closers. At each reading station on each plumbline, an anchor-plate assembly is installed for supporting a portable measuring apparatus. The anchor-plate assembly is positioned so that the upstream and downstream movement, as well as the cross stream movement, of the plumbline is measured from two positions, one position normal to and the other parallel to the axis of the dam. Measurements are made using a measuring apparatus which consists of a

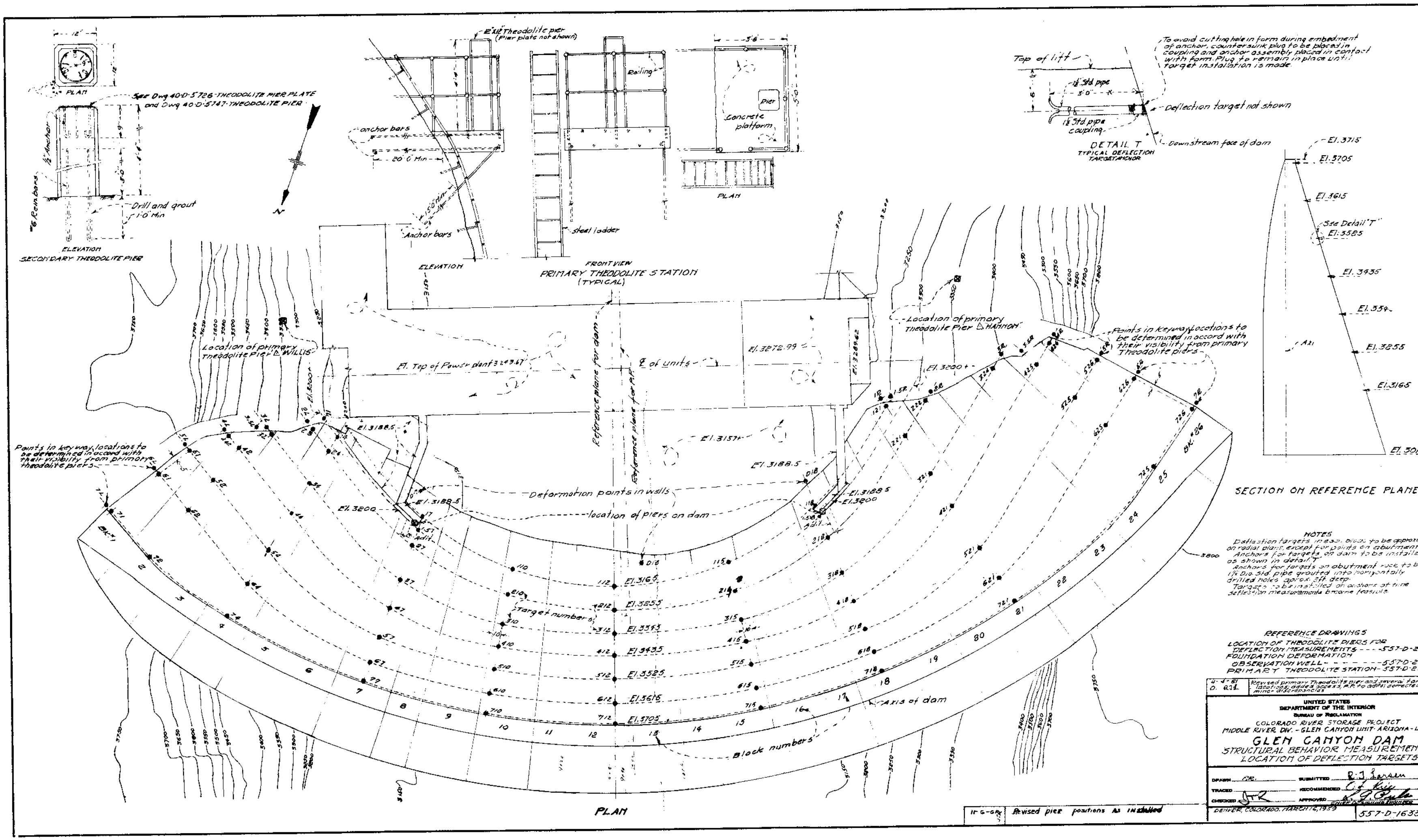


Figure 92.—Dam structural behavior measurements—Location of deflection targets.

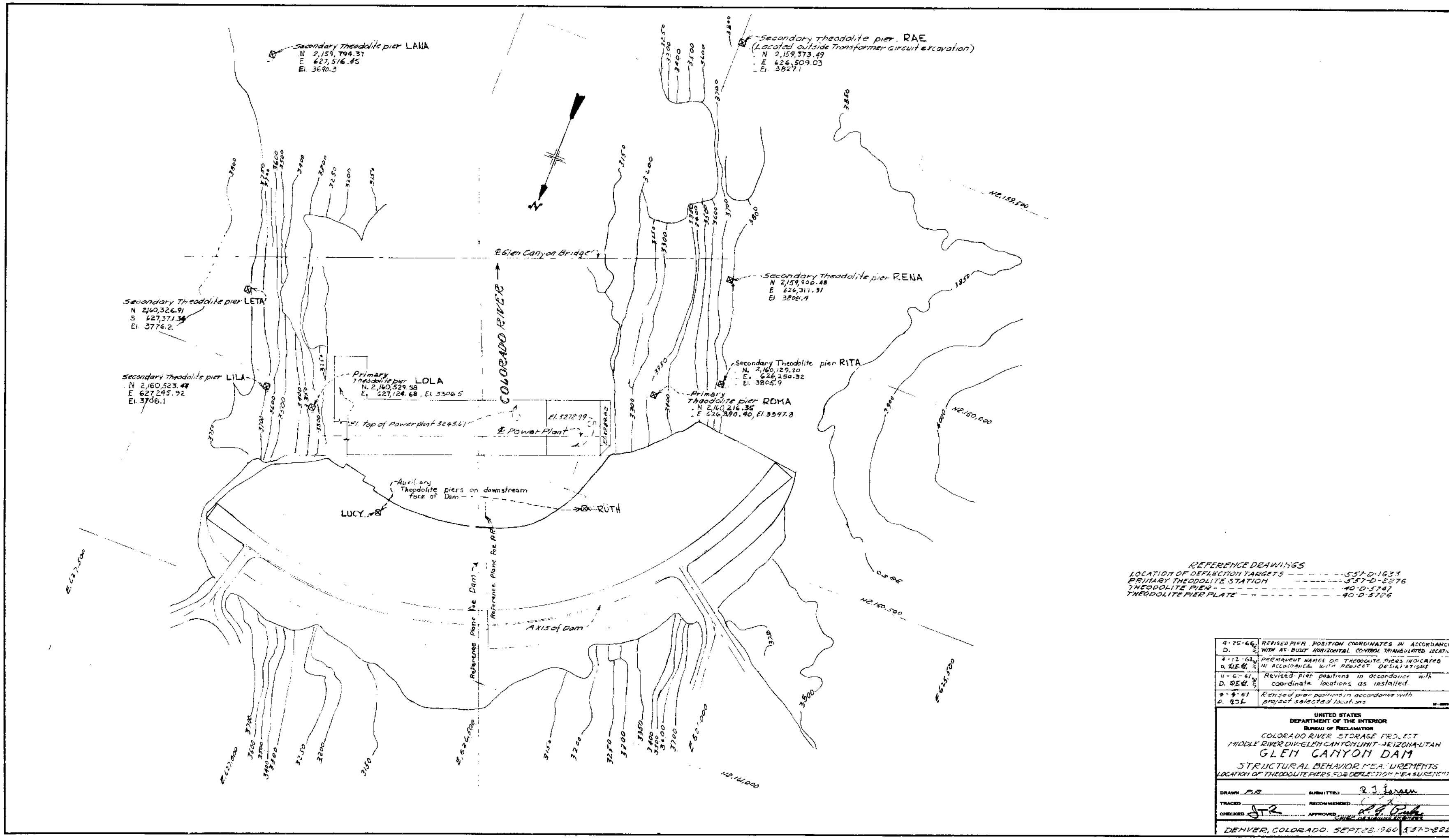


Figure 93.—Dam structural behavior measurements—Location of piers for deflection measurements.

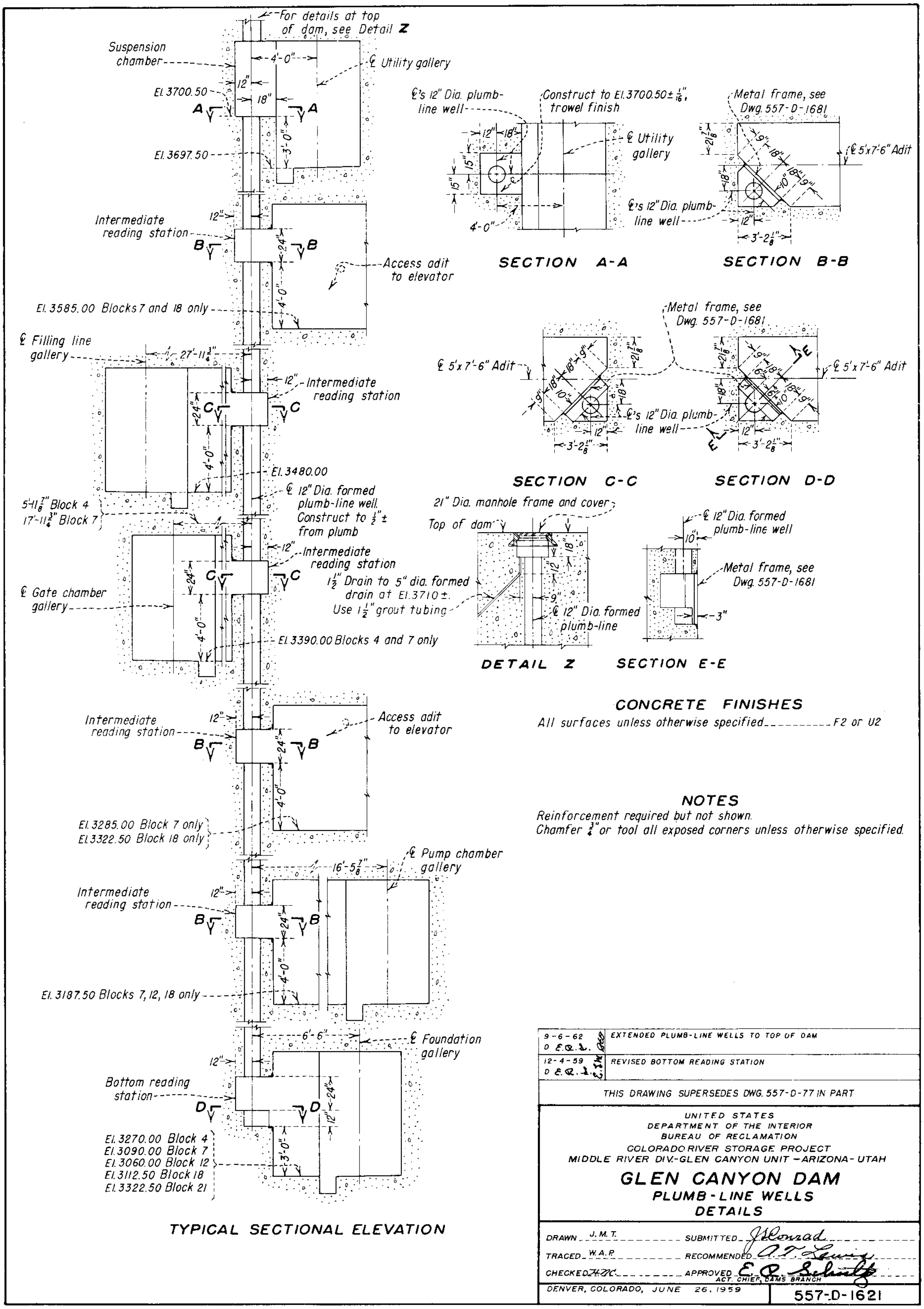


Figure 94.—Plumbline wells for deflection measurements in dam.

microscope mounted on a micrometer slide carriage. The apparatus is portable and is attached to upright supports of the anchor-plate assembly by knurled headed hold-down screws. Movement of the plumbline is determined in terms of the differences between successive readings made on indexes on the support assembly and readings on the position of the plumbline.

The doors of the reading station are kept locked except when readings are being made to prevent unauthorized personnel from disturbing the plumbline. Observed data are recorded on an appropriate data sheet.

(d) *Uplift Pressure Pipes.*—Hydrostatic uplift at the base of the dam is measured by 41 pipes in 7 lines of from 5 to 7 pipes each. The lines of pipes are located in blocks 2, 4, 5, 7, 11, 16, 19, and 25. The layout of the system of pipes is shown on figure 95.

When a pipe is under pressure, the pressure is measured by means of a Bourdon-tube pressure gage calibrated in feet of water, attached through a gage cock to the uplift pipe. When zero pressure is indicated in a pipe, the water level is determined by sounding. Continued zero pressure with water standing at the level of the top of the pipe is investigated further by adding a transparent standpipe section to the pipe and observing the level to which the water rises. Uplift pressure readings are recorded on appropriate data sheets.

(e) *Drain Flow Measurements.*—Drain flow measuring weirs are installed, as the need arises, in the drainage gutters of the galleries and adits of the dam. The locations of those weirs are selected so that measurements of the flows of drainage water can be localized to specific zones in the dam. Locations of the weirs are shown, and added to as required, on drawings No. 557-420-1353 and 557-420-1354.*

Measurements of the flows of drainage water over the weirs are made on a monthly schedule and recorded on appropriate data sheets.

(f) *Rock Deformation Measurements.*—Two systems were installed for measurement of deformation of the rock of the abutments and foundation of the dam. One of those systems consists of 112 deformation meters which utilize Carlson-type meters as measuring elements. These meters are in 17 general locations at the surface of contact between the concrete of the dam and the rock of the foundation and abutments. Anchors extend 20 feet into the rock from the contact surface and the deformation meters measure the rock deformation which occurs in that distance. Data from these meters are recorded on the data sheets with the embedded instrument data.

The other system consists of Invar tapes in four tunnels extending into the rock of the dam abutments. Two of the tunnels are in each abutment at elevations 3480 and 3630. These tunnels which are located about 85 and 235 feet, respectively below the crest of the dam, extend into the rock mass from the keyways. The tunnels are excavated normal to the centerline of the canyon and have a slope equal to 0.01.

The principal components of each gage (fig. 96) are a surveyor's tape (Invar) having a temperature coefficient of expansion equal to 0.22×10^{-6} per degree F., and a measuring head (fig. 97). Three gages are installed in series in each tunnel. These gages measure rock movements over a total length of 225 feet with the first reference point about 10 feet from the face of the keyway. Within this 225-foot distance, rock movements are found for tape spans equal to 50, 75, and 100 feet, respectively.

All components of a tape gage are installed on a tunnel wall about 5 feet above the floor. Each of the three tapes is supported at one end by a spring-loaded yoke, and at the other end by a fixture which is attached solidly to the rock. The tapes are supported at about 20-foot intervals by hangers.

When making a measurement with this device, two operations are necessary. First, the tape is pretensioned to 30 pounds by adjusting the lengths of the yoke springs (surveyor's tension handles). Second, the micrometer is adjusted until the spindle is within about 0.001 inch from the steel ball attached to the yoke. This last step is accomplished by connecting a 1.5-volt flashlight battery, a 3,000-ohm resistor, and a milliammeter in series with the yoke and the micrometer. The micrometer is insulated from the yoke. A slight movement of the milliammeter indicates that the micrometer has been positioned properly for making a reading. Each measuring head is protected from dripping water and rock spalls by a metal deflector. Components of the tape gages are fabricated from cadmium-plated steel, stainless steel, brass, or nylon.

*Not included.

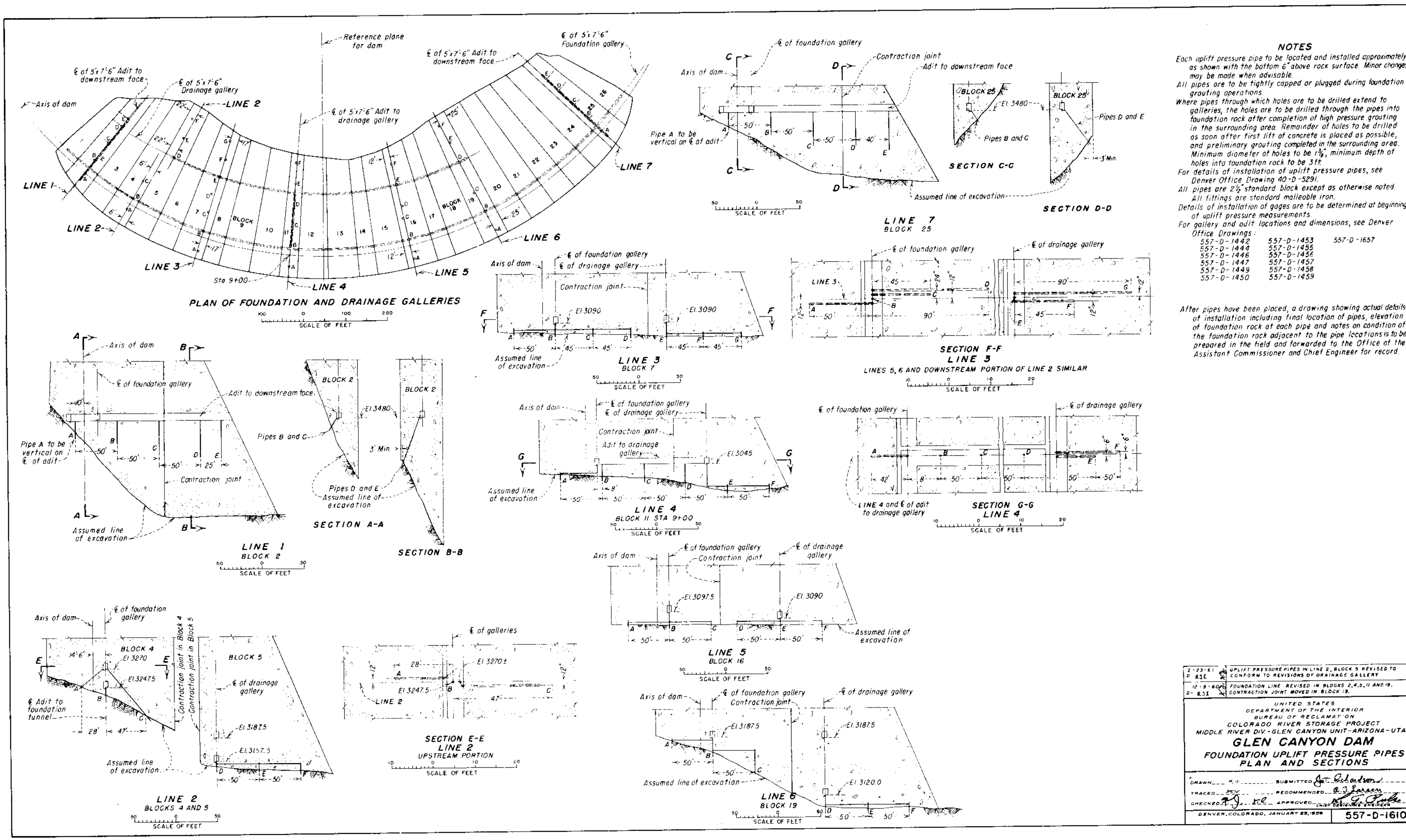
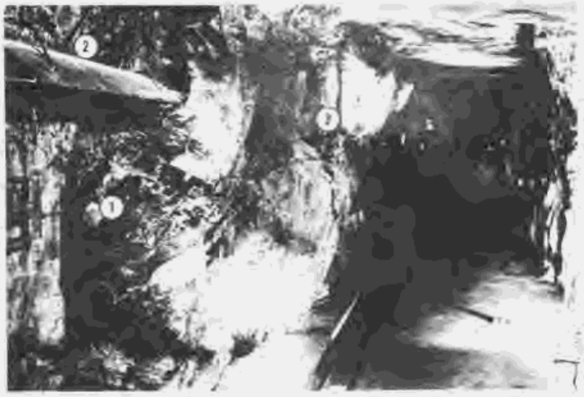
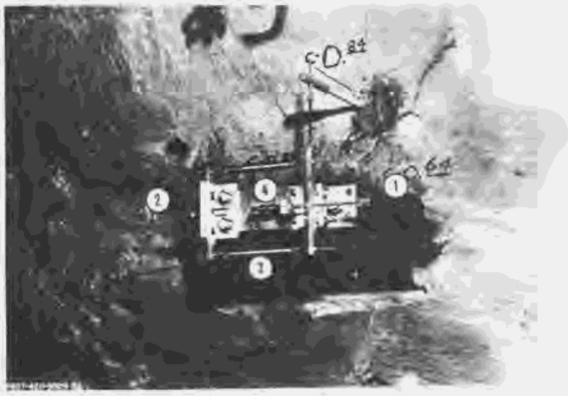


Figure 95.—Dam foundation uplift pressure pipes—Plan and sections.



(1) Surveyor's tape.
(2) Deflector.
(3) Tape hanger.

Figure 96.—Tape gage on tunnel wall, 50-foot tape span, Glen Canyon Dam. P557-420-10481NA.



(1) Tape (floating end).
(2) Tape (fixed end).
(3) Surveyor's tension handle.
(4) Micrometer.

Figure 97.—Measuring head for tape gage, Glen Canyon Dam. P557-420-9329NA.

It is anticipated from this latter installation that additional important information will be learned on: (1) the effect of long-term loading on foundation deformations, (2) deformation moduli of a rock mass, and (3) abutment-rock deformations.

Suitable conclusions on the behavior of loaded rock masses in general must be deferred until foundation-deformation data are available for many types of formations.

(g) *Seismograph Station.*—A seismograph station is operated by the Bureau in cooperation with the Coast and Geodetic Survey. It is located about 11 miles northwest of the Government townsite. A concrete vault in a hillside contains three seismometers, a recorder, time apparatus, radio equipment, and an accelerograph. The station records distant earthquake shocks as well as shocks in the general vicinity of the project. Records from the station, in addition to indicating the magnitude of shocks in the vicinity of the project, will serve to determine possible change in local seismicity that may occur in the area due to the weight of the reservoir on the earth's surface. Operation of the station is in accordance with instructions furnished by the Coast and Geodetic Survey. Records from the station are sent to the Washington, D. C. office of the Survey where they are processed. Copies of the resulting interpretation are furnished the project, the regional office in Salt Lake City, and the Office of Chief Engineer. Periodic inspections of the station instruments are performed by field personnel of the Coast and Geodetic Survey.

(h) *Computation and Plotting.*—Results for all systems of measurements except uplift pressures and drain flows are computed in Denver using the recorded, measured data which are transferred from the data sheets to punch cards and processed by use of an electronic computer. Plotting of the computed results is accomplished by use of an electronic plotter.

The plotted results from the data obtained from the stress meters and from the strain meter groups show variations of stress with time. Plots derived from data obtained from "no-stress" strain meter pairs, strain meter trios, jointmeters and deformation meters show length changes with respect to time.

Plots derived from the data in the form of deflections for the various arches and various cantilever sections of the dam furnish a record of the movement of the dam.

Plots, with respect to time, of the plumbline data are made in the form of radial and tangential deflections of the top of the dam, and of each intermediate elevation and furnish a time record of the movement of the four arch and three cantilever elements of the dam in which the plumblines are installed. These data are correlated with data from the triangulation measurements for information to detect possible deformation of the foundation of the dam.

(i) *Records and Reports.*—Readings of the instruments embedded in the dam, deformation meters, Invar tapes, uplift pressure pipes, the drain flows, and measurements on the plumblines are made in accordance with the schedule shown by the designers' operating criteria.[6] These data are recorded on appropriate data sheets and transmitted as a monthly report to the Engineering and Research Center, Denver.

Daily records of air and water temperature, reservoir and tailwater elevations, and any other data that may have an effect on the structural action of the dam, along with comments concerning the operation of the apparatus or the measurements are included with the report.

Measurements of the movement of the deflection targets on the face of the dam and measurements over the system of theodolite piers are made at a less frequent interval than the other measurements. The schedule for these triangulation measurements is also indicated by the designers' operating criteria. These data are transmitted to Denver as they become available.

Check measurements of embedded instrument cable resistances are made semiannually. As these data become available, they are transmitted to Denver with the monthly reports of instrument readings on appropriate data sheets. The Bourdon-tube pressure gages used for uplift pressure measurements are cleaned and recalibrated annually.

C. DAM ELECTRICAL SYSTEM

39. ELECTRICAL SYSTEM FOR DAM AND RELATED STRUCTURES. The extent of the electrical system serving Glen Canyon Dam and related structures generally appears on figures 98 and 99. The major portion of the system was installed under specifications No. DC-5750, and the following materials were installed under specifications No. DC-4825: Embedded electrical materials including portions of electrical conduit and grounding systems; embedded parts of lighting fixtures along the crest roadway on the dam, on the right abutment service bridge, and at the river outlet valve structure; and the gantry crane power outlet receptacles along the crest roadway. The system serves electrically operated equipment and lighting systems located in and on the dam proper, the elevator towers, the spillway gate structures, and the river outlet valve structure. As shown on figures 98 and 99, the electrical system employed for the dam, elevator towers, and spillway gate structures is served by two transformer banks located in the dam. Each transformer bank is served by a primary service circuit originating at switchgear in Glen Canyon Powerplant. These primary service circuits are energized nominally at 4,160 volts, 3-phase, 60 cycles, and each circuit is connected to and served by a circuit breaker in the powerplant 4,160-volt switchgear. The transformer banks provide service at nominally 480 volts, 3-phase, 60 cycles to two main power distribution panelboards located in the dam. From these two main power distribution panelboards, power distribution circuits energized nominally at 480 volts, 3-phase, 60 cycles are extended to distribution panelboards in the dam and elevator towers. The two spillway gate structures are each served by a power circuit energized nominally at 480 volts, 3-phase, 60 cycles. The power circuits to the spillway structures are connected to and served by circuit breakers in power panelboards located in the dam. Electrical service to the river outlet valve structure is provided by a power service circuit energized nominally at 480 volts, 3-phase, 60 cycles and connected to and served by a circuit breaker in one of the powerplant's unit-sub switchgear. This power service circuit extends to and serves a distribution board at the river outlet valve structure.

The power and lighting circuits are generally contained within a rigid steel conduit system and within equipment enclosures. Conduit systems employed in the gallery system of the dam and areas not normally available to public view are for the most part exposed conduit systems; otherwise, the conduit systems are generally embedded in the concrete of structures or are otherwise concealed from view. Also, electrical distribution and control equipment employed in the electrical system is contained within exposed surface-mounted type metal enclosures, and the enclosures are generally located in galleries, rooms, and areas not normally accessible to the public.

[6] "Designers' Operating Criteria—Glen Canyon Dam, Powerplant, and Switchyard—Glen Canyon Unit, Middle River Division, Colorado River Storage Project," Bureau of Reclamation, June 1965. (Unpublished.)

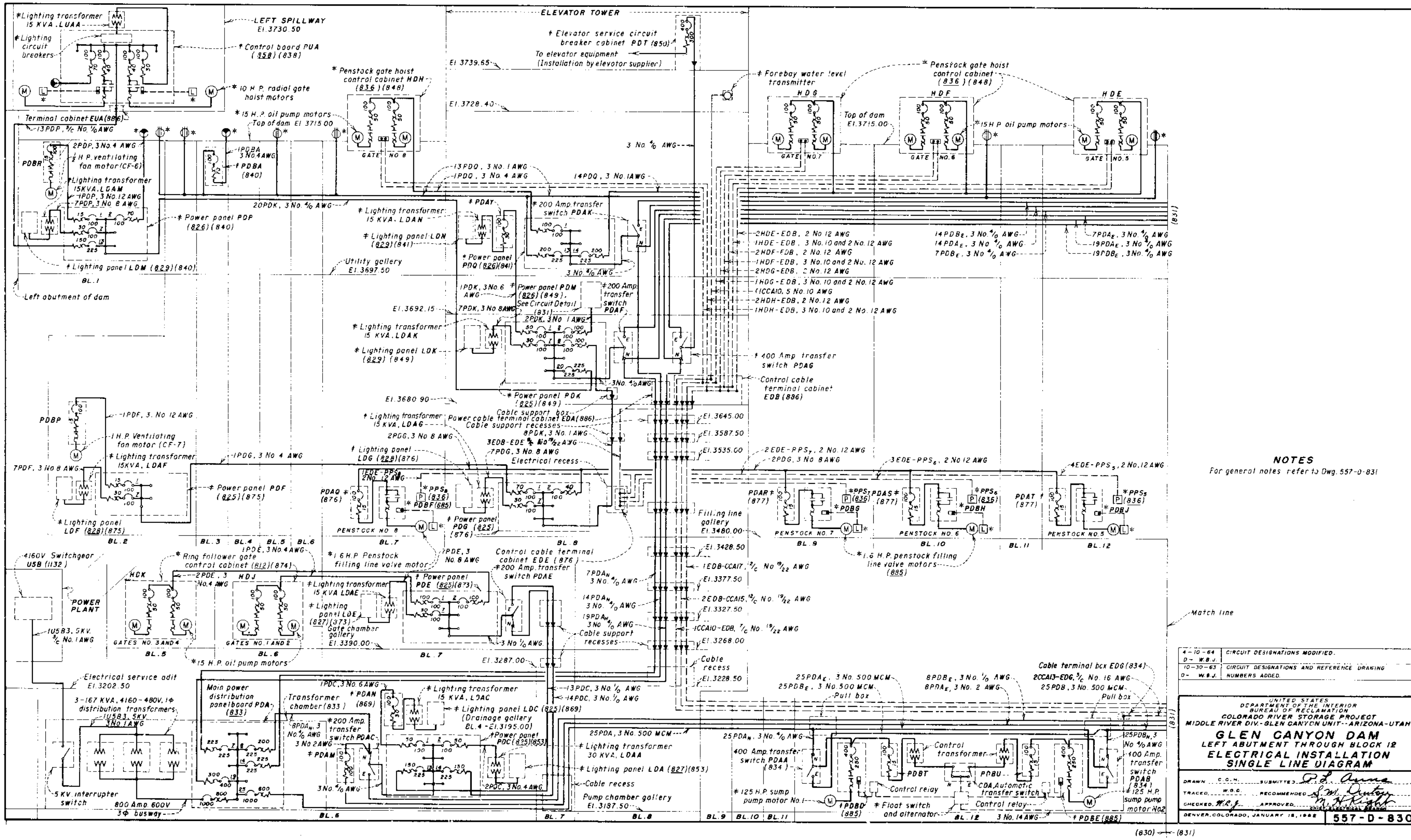


Figure 98.—Single-line diagram of electrical installation in dam—Left abutment through block 12.

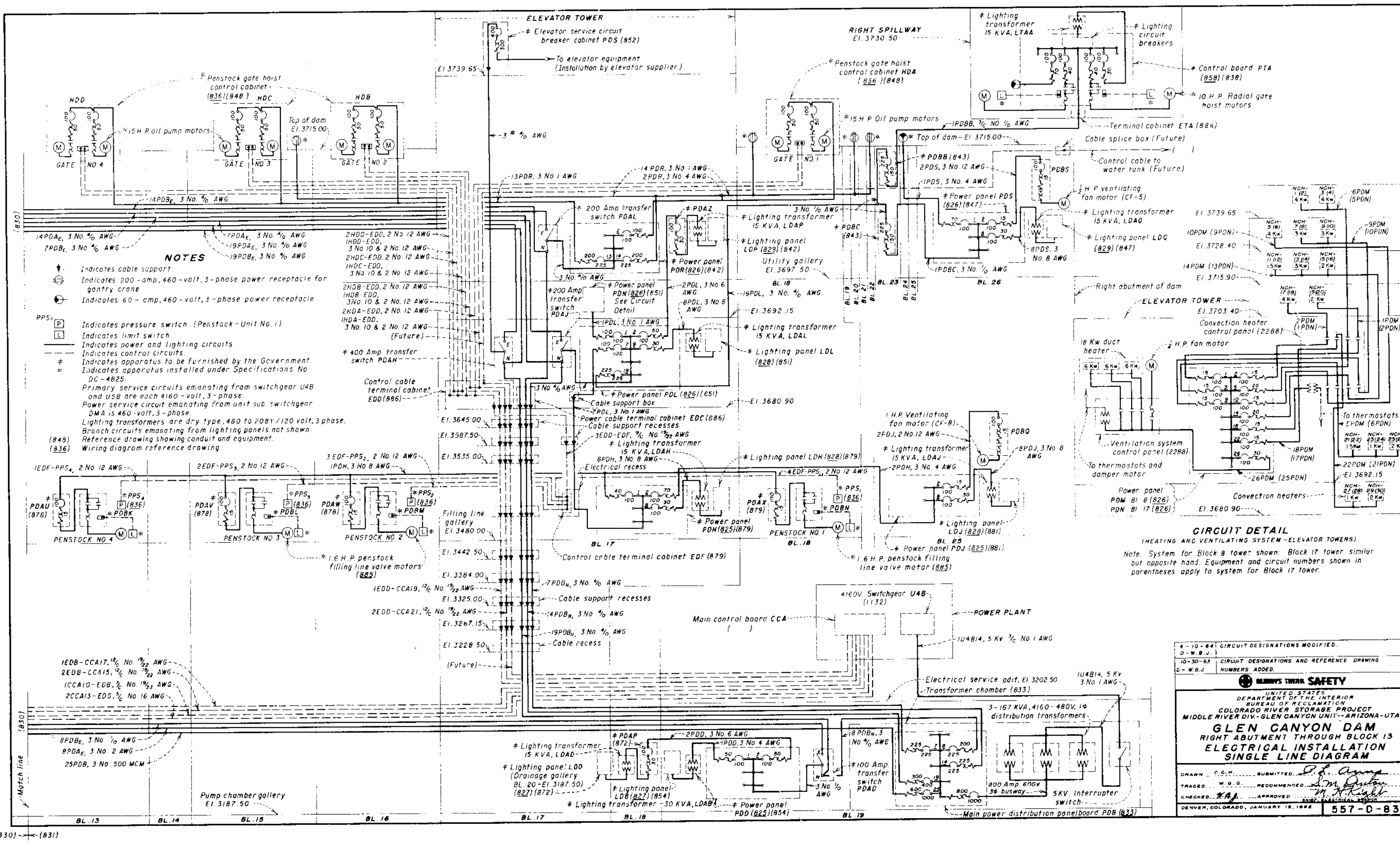


Figure 99.—Single-line diagram of electrical installation in dam—Right abutment through block 13.

The power distribution circuits and branch power circuits operating nominally at 480 volts and which originate and emanate from the main power distribution panelboards and branch power panelboards in the dam and related structures are afforded overcurrent protection by automatic-trip, molded-case-type circuit breakers contained in the panelboards. The breakers are also manually operable and provide a means of disconnecting the power distribution circuits and branch power circuits from their source of supply. At locations where tap circuits are connected to the branch power circuits, and also at locations where loads and equipment served by the branch circuits are not immediately adjacent to the branch power panelboards, individual circuit breakers are provided to afford overcurrent protection and to provide a means of locally disconnecting the tap circuits and equipment from the branch power circuits. The individual breakers are also automatic-trip, molded-case type.

Electrically operated equipment and loads served by the electrical power distribution system in the dam proper and the elevator towers include the gallery drainage system sump pump motors, ventilating and heating system equipment and components, penstock filling-line valve motors, penstock gate hoist oil pump motors, river outlet ring-follower gate oil pump motors, gantry crane power outlet receptacles, elevator machinery motors, and lighting system transformers. Equipment served at the spillway gate structures includes radial gate hoist motors and lighting system transformers. Equipment served at the river outlet valve structure includes hollow-jet valve oil pump motors and a lighting system transformer.

All motors of the various items of motor-operated equipment employed in the dam and related structures are afforded overload protection by thermal overload protective devices incorporated in the motor starting equipment serving the motors. The overload protective devices are a manual reset type. Motor starters are installed either adjacent to or in the control cabinets containing the motors which the starters serve and operate. Most of the motor starters are normally controlled by pushbutton stations located at or adjacent to motor starter locations. The gallery drainage system sump pump motor starters are automatically controlled by float switch equipment located in the sump pump chamber, but can also be controlled manually by selector switch units also located in the pump chamber. A selector switch is provided for each pump unit so that a pump can be placed on automatic operation or started and stopped manually by means of the switch as desired. Pump motors in penstock gate hoist control cabinets are provided with an automatic starting system controlled through pressure switches which becomes effective after a penstock gate is raised. This automatic starting system functions in conjunction with the gate position restoring system. Should a gate partially descend after being raised, the pump motor automatically starts and provides hydraulic system oil pressure necessary to automatically return or restore the gate to its fully raised position. Upon restoration of the gate to the proper raised position, the stopping of the motor is effected through action of pressure switches incorporated in the hydraulic system.

As indicated on figures 98 and 99, the gallery drainage system sump pump motors, the elevator machinery motors, and branch power panelboards PDC, PDD, PDE, PDK, PDL, PDQ, and PDR in the dam and elevator towers are each served through transfer switches. The transfer switches are manually operable and provide a means of connecting the indicated motors and panelboards to one of two power distribution circuits—a normal circuit and an emergency circuit. The apparent important or critical loads include the sump pumps, elevator machinery penstock gate hoists, river outlet ring-follower gates, and elevator towers heating, ventilating, and lighting systems.

The operation and control of the major portion of electrically operated equipment in the dam and related structures is normally accomplished by operation of controls locally provided at the location or site of equipment being operated. In a few instances, however, control circuits extend between equipment in the dam to control boards in the powerplant. The forebay (reservoir) water surface elevation is made available at a receiving instrument in the powerplant through a control circuit extending from a float-actuated transmitting instrument located in the elevator tower in block 8 at the top of the dam.

From each of the eight penstock gate hoist control cabinets located in the gate hoist structures of the dam, control circuits partially controlling penstock gate operation extend to main control board CCA in the powerplant. For each penstock gate, gate position indicating lights, a manually operable emergency complete unit shutdown switch to effect emergency closure of the gate, and a penstock gate test switch to test emergency closure of the gate are provided at the control board. Also, at the control board, the control circuits are so connected that automatic closure of a penstock gate will be effected through operation of generator main overspeed switch, governor oil level switch, or governor oil pressure switch. These gate control circuits operate at 125 volts direct current, and the direct current is supplied by the powerplant station battery. Additional control circuits operating at 110 volts, 60 cycles, also extend from the penstock gate hoist control cabinets to the penstock filling line valve locations in the filling line gallery of the dam.

Pressure switches connected to the penstock filling line piping are so connected in the penstock gate control circuits that a closed gate cannot be raised (opened) until the penstock associated with the gate has first been filled with water by means of the filling line valve. An alarm circuit is provided between the sump pump chamber in the dam and main control board CCA in the powerplant. Abnormally high water in the drainage sump in the dam will cause a contact of the sump pump control float switch to close and thereby initiate an alarm signal at the control board. Further, should a thermal overload device in one of the sump pump motor starters operate or the control circuit operating the pumps be transferred from the normal to the emergency source of supply, contacts on the thermal overload device or on the automatic transfer switch for the control circuit will close to initiate an alarm at the control board. The alarm control circuit operates at 125 volts direct current with the direct current being supplied by the powerplant station battery.

Lighting systems employed in the dam and related structures provide general utilitarian illumination in galleries, adits, and machinery spaces in the dam; in rooms of the elevator towers; on the crest roadway of the dam and service bridge; on the spillway gate structures; at the river outlet valve structure; for the left and right abutment covered walkways between the powerplant and dam; in the pipe chase and vaults for the outlet pipe expansion joints located in the left abutment mass concrete between the powerplant and dam; and for the slide gate access shaft and parking area at the left abutment downstream from the powerplant. The lighting systems for the covered walkways between the powerplant and dam, in the pipe chase and vaults for the outlet pipe expansion joints, and for the slide gate access shaft and parking area at the left abutment downstream from the powerplant are served from lighting distribution panelboards in the powerplant. Otherwise, the lighting systems for the dam and related structures are served from lighting panelboards located in the gallery system of the dam, in the elevator towers, and on the spillway gate and river outlet valve structures. The lighting panelboards are supplied through dry-type, air-cooled distribution transformers energized from the power distribution system in the dam and related structures. The transformers are generally situated adjacent to the respective lighting panelboards served by the transformers. The lighting system service voltages are nominally 208Y/120 volts, 3-phase, 4-wire (grounded neutral), 60 cycles, and all lamps energized from lighting system circuits are rated at 115 volts.

Lighting outlets within the gallery system of the dam consist generally of lampholder devices employing bare lamps. Lighting fixtures employed elsewhere are industrial and commercial types. Convenience outlets are provided in the same general areas and locations as are lighting outlets and fixtures. The convenience outlets are 2-wire, 3-pole, (grounded pole) types providing energy at nominally 120 volts, single-phase, 60 cycles at the outlet receptacles.

All lighting system panelboards contain automatic-trip, molded-case-type circuit breakers which afford short circuit and overload protection to the branch circuits originating at and emanating from the panelboards. Most of the branch circuits serving lighting outlets and fixtures are controlled by conventional lighting control switches. Switches are located at gallery entrances and intersections, and near or adjacent to access doors of rooms and compartments. For some of the longer circuits employed in the gallery lighting system, mechanically held magnetic contactors (remote control switches) are utilized to energize the circuits. The contactors are generally located in the lighting panelboards or mounted near the entrances to the grouting adits and are controlled by conventional momentary-contact-type lighting circuit switches.

The operation of crest roadway lighting units on the dam and on the service bridge is controlled by a time switch located in the utility gallery of the dam. The time switch, through control relays, controls the operation of mechanically held magnetic contactors (remote control switches) which energize the circuits serving the crest roadway lighting units. The contactors are located in the utility gallery lighting panelboards and the control relays are located adjacent to the panelboards. The roadway lighting units are mounted in the roadway parapets of the dam and service bridge. The canopy lighting units and the floodlights on the elevator towers, and the lighting units on the bridges and abutments of the spillway structures are controlled directly through individual time switches located adjacent to the elevator tower lighting panelboards or contained in the spillway structure control boards. The time switches are all a synchronous-motor-driven type and have an astronomic-type dial. The on and off periods of the switches can be manually adjusted or set. The operation of lighting units and lighting outlets in public areas of the elevator towers and at elevator landings in the elevator towers and dam, and the floodlighting units on the spillway structures is controlled directly by the circuit breakers serving the circuits to these lighting units and outlets.

The dam and related structures are provided with a grounding system which generally consists of main runs of bare copper cable extending exposed along galleries and otherwise embedded in concrete of structures. The main runs of ground cable extend from and are connected to the powerplant ground mat. Electrical equipment enclosures and cabinets, metal conduits, structure handrailings and metalwork, machinery bases, and crane rails are connected to main ground cable runs with branch ground cable taps and extensions to provide a basically common and interconnected grounding system.

40. GENERAL. One spillway is provided on each abutment. Each spillway consists of an approach channel, intake structure, spillway tunnel, and deflector bucket. Spillway discharges are controlled by two 40- by 52.5-foot radial gates in each intake structure. General plans and profiles of the spillways are shown on figures 100 and 101.

41. HYDRAULICS. To determine required spillway capacity, two inflow floods were considered:

(1) A maximum probable snowflood which had a peak inflow of 380,000 cubic feet per second and a 122-day volume of 29,060,000 acre-feet.

(2) A maximum probable rainflood which had a peak inflow of 417,000 cubic feet per second and a 6-day volume of 2,063,000 acre-feet.

Studies indicated the snowflood to be the critical flood and it was therefore adopted as the inflow design flood.

The design flood was routed through the reservoir, assuming that the reservoir was at elevation 3700, top of conservation storage, at the beginning of the flood. The maximum reservoir water surface obtained from the routing was elevation 3711 and the maximum outflow was 300,000 cubic feet per second. The spillways were sized to have a combined capacity of 276,000 cubic feet per second with the reservoir at elevation 3711. The remaining 24,000 cubic feet per second would be released through the river outlets and turbines.

The locations of the spillways were set to provide a satisfactory alinement with the river. This resulted in the intake structures being located well back of the canyon rim. Intake channels were provided for flow from the reservoir at the canyon rim to the intake structures.

The intake structures were designed to control releases and provide an entrance to the tunnels. The concrete crest is at elevation 3648.00. Two 40- by 52.5-foot radial gates are installed at the crest of each intake structure to control releases. Discharge curves for the radial gates are shown on figure 102. The crest shape is designed to follow the under nappe for a gate opening of 10 feet. The coefficient of discharge for the uncontrolled crest with maximum reservoir water surface is 3.46. The crest and gates are set at a converging angle of 84° from the centerline of the tunnel to start the transition and to decrease the tunnel portal width.

The spillway tunnels for the greater part of their length are 41 feet in diameter. The transition section downstream from the intake structure changes from a flat-arch-roof section 89 feet wide by 52 feet high to a circular section 48 feet 3 inches in diameter. From this point there is a further transition of the circular section to the 41-foot-diameter tunnel. The tunnels were designed to flow partially full, and at all sections the depth of the water will be 0.7 times the height of tunnel, or less.

At the downstream portals a concrete deflector bucket was designed to lift the jet of water a safe distance into the center of the river channel and also to deflect the jet away from the canyon wall.

42. MODEL STUDIES. The original layout for the spillway tunnels was based on data obtained from other tunnel spillways built by the Bureau, routing of the flood through the tunnels, and adapting the lower ends of the tunnels to the diversion scheme.

Extensive hydraulic model studies of both spillway tunnels were made on a 1:63.48 scale model.[1] These studies aided in determining the final dimensions of the approach channel, the transition section of the tunnel, and the location and final shape of the deflector buckets. Model studies indicated that the maximum velocity in the spillway tunnel will be 162 feet per second. Discharge curves shown on figure 102 were determined by the model studies.

43. CONCLUSIONS FROM MODEL STUDIES. In addition to the maximum spillway tunnel velocity stated above, other conclusions from the model studies are:

(1) The alinement of the tunnels was satisfactory for diversion flows and spillway flows.

(2) Preliminary tests on a 1 to 88 scale model indicated that the most satisfactory invert angle for the flip buckets was 35°. Subsequent tests on the 1 to 63.48 spillway model confirmed this.

(3) A low curved concrete wall placed adjacent to each canyon wall protected the canyon walls from further undermining and erosion damage by diversion flows.

[1] "Hydraulic Model Studies of the Spillways and Outlet Works—Glen Canyon Dam," Hydraulic Laboratory Report No. Hyd-469, Bureau of Reclamation, February 18, 1964 (unpublished).

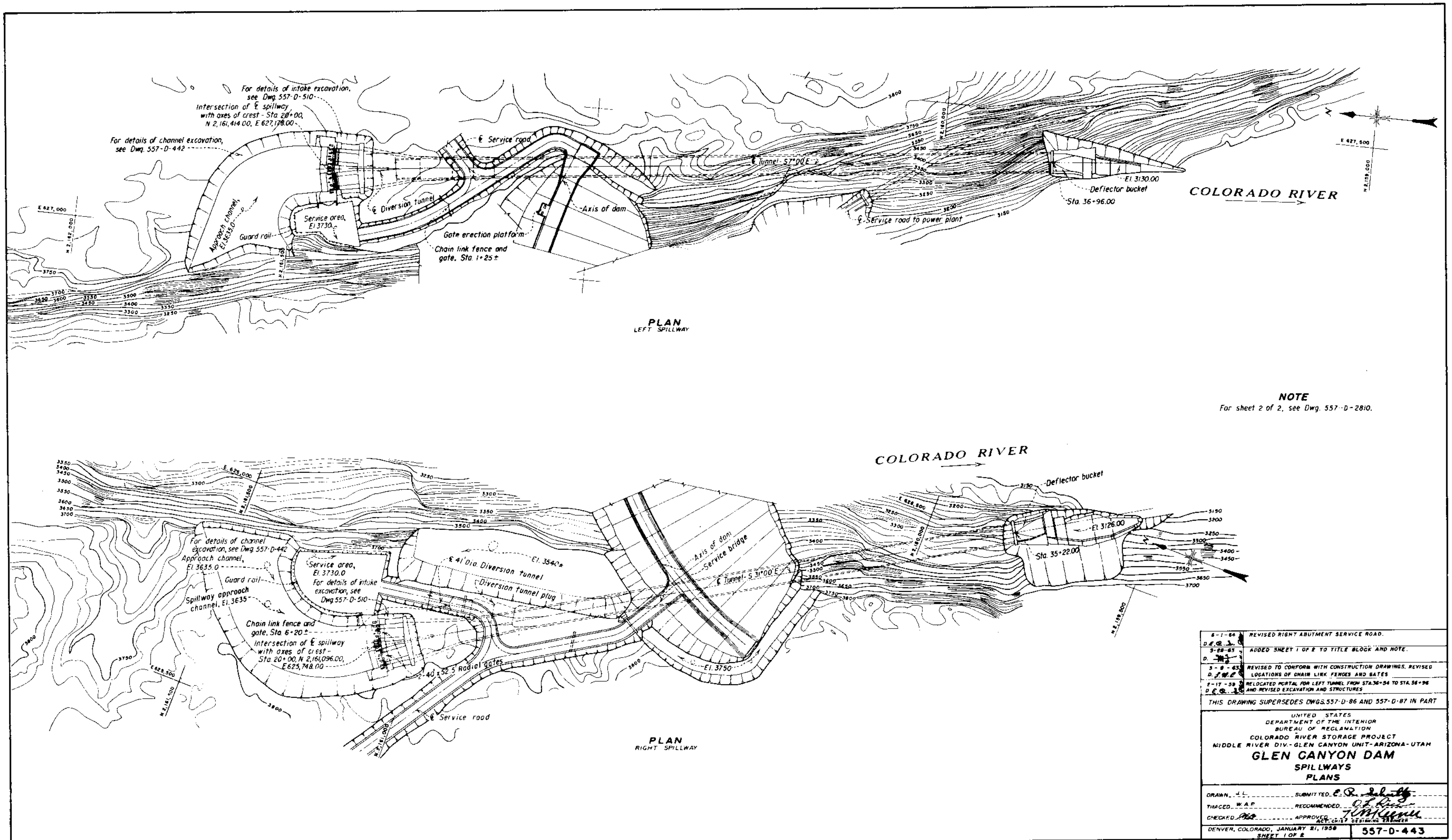


Figure 100.—Dam spillways—Plans.

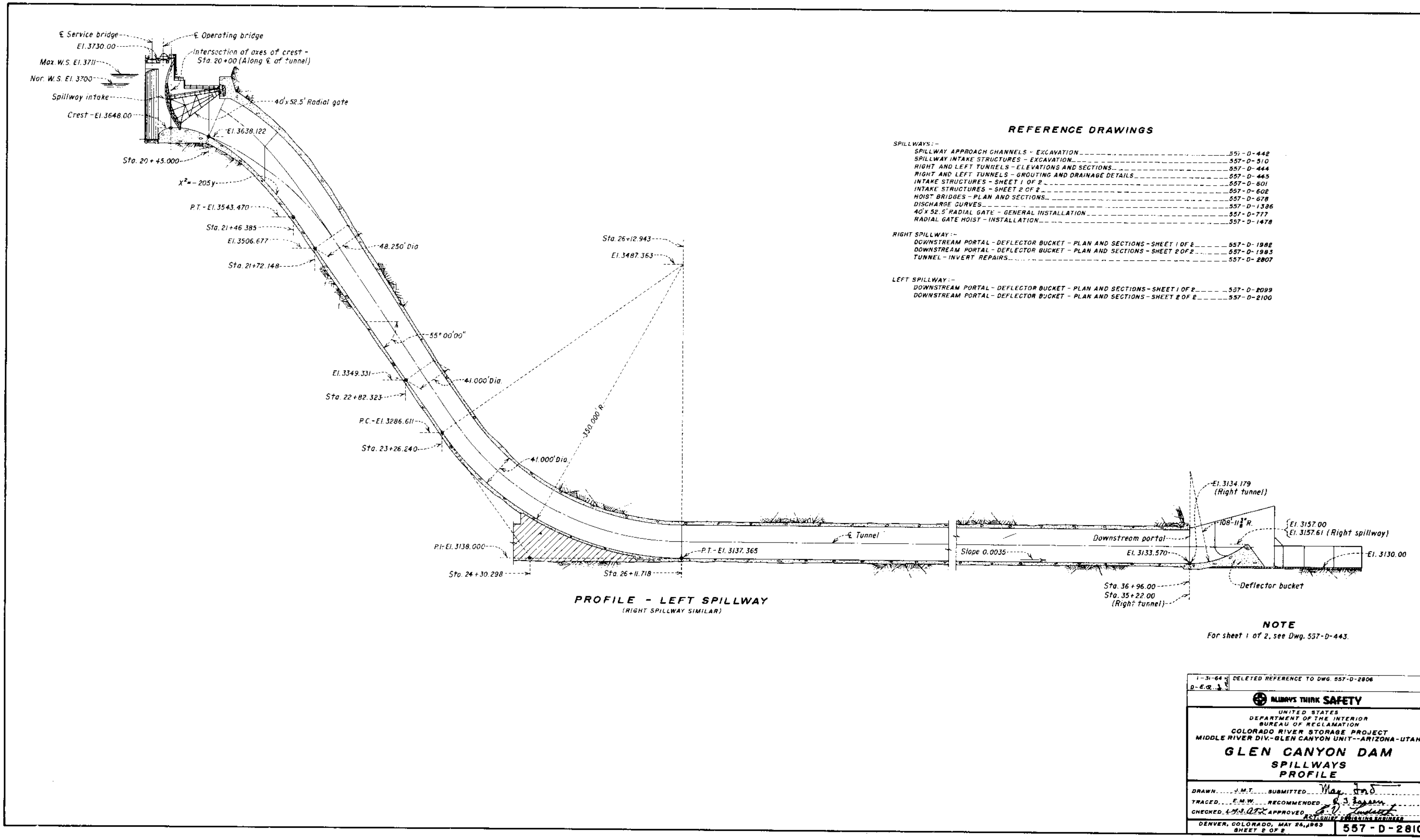


Figure 101.—Dam spillways—Profile.

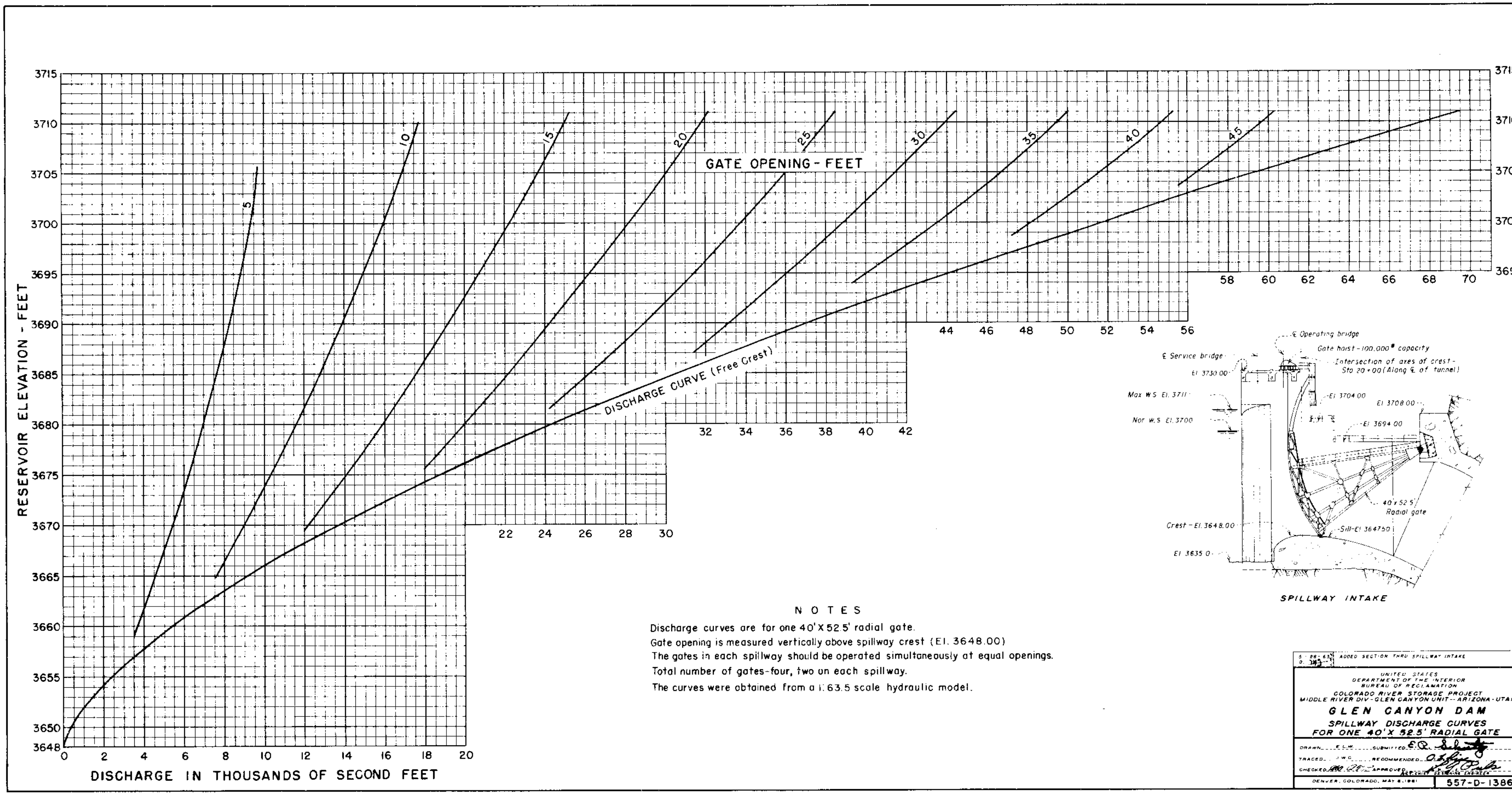


Figure 102.—Spillway discharge curves for one 40- by 52.5-foot radial gate.

(4) The spillway approach channels were greatly reduced from their original size and still provided extremely smooth flow conditions.

(5) Flow through the crest sections was excellent and no adverse pressure conditions were noticed. The maximum discharge of 138,000 cubic feet per second per tunnel was obtained at reservoir elevation 3711, the value used for design purposes.

(6) The preliminary tunnel transition was too abrupt. A surface fin formed in the center of the tunnel and pressures on the sidewalls were in the cavitation range.

(7) The longer recommended transition (fig. 104) was adequately streamlined and provided smooth flow conditions with no adverse pressures on the sidewalls.

(8) Flow in the 41-foot-diameter tunnels was excellent at all discharges.

(9) The preliminary downstream circular-to-rectangular tunnel transition was too short, as indicated by severely subatmospheric pressures in the lower corners. Increasing the transition length from 70 to 100 feet increased the pressures to a satisfactory value. This transition was eliminated in the recommended design.

(10) The preliminary deflector buckets, which were rectangular in cross section, were replaced by a bucket in which the circular invert of the tunnel intersected the vertical curve of the bucket. This type of bucket also eliminated the need for the circular-to-rectangular transition.

(11) The deflector buckets were moved upstream to the tunnel portals, eliminating about 200 feet of open channel.

(12) The outside walls of the buckets were turned inward 7 feet to direct the flow in a more favorable pattern at their impact points.

(13) The outside wall of the left bucket was extended 32.5 feet downstream from the lip to deflect the flow from the canyon wall.

(14) Pressure measurements on the wall and invert of the left bucket showed that pressures equivalent to 211 feet of water would be considered in the structural design of the bucket.

44. STRUCTURAL DESIGN. (a) *Intake Structure.*—The layout of the intake structure is shown on figure 103. The design was based on concrete having a compressive strength of 3,000 pounds per square inch at 28 days. The allowable working stresses, except as noted below, are shown on figure 71.

The crest and piers were designed for earthquake, hydrostatic pressure, uplift, and service bridge loadings. Earthquake loadings of 0.1 g horizontally and 0.05 g vertically were used. The horizontal component of thrust from the radial gates is carried directly to the rock through a massive beam above the portals of the tunnels. This beam was designed to support the vertical component of the thrust as well as the reaction of the weight of the gate and trunnion assemblies. The bridge was designed for the load imposed by the gate hoists and for a 20-ton truck-mounted crane.

The period of vibration of the center pier was found to be in the earthquake resonance zone and, therefore, this pier was designed for 0.7 g earthquake; however, the allowable stresses in the reinforcement were increased to 40,000 pounds per square inch for this condition. A split sewer-pipe drainage system was placed in the walls and on the foundation of the crest structure for uplift pressure relief.

(b) *Tunnel.*—The arrangement of each tunnel and transition is shown on figure 104.

The thickness of the tunnel lining for the circular sections was made 0.8 inch per foot of the tunnel diameter. The lining was reinforced where required for tension caused by the dam and reservoir waterloads. The stresses in the vicinity of the tunnels were determined by Newmark's method of computation of stresses in elastic foundations. Stresses in the lining caused by the stresses in the rock were determined by both analytical and model studies.[2]

Moments, thrusts, and shears in the noncircular transition section of the tunnels were determined by photoelastic model studies[3] of various sections for hydrostatic load, dead load, grout load, and rock load based on the assumption that the height of the rock acting on the structures was equal to 0.35 of the excavated width. A grout load of 3,600 pounds per square foot over a 90° sector of the arched roof, or between spring lines, whichever was least, were used for design. The results of these studies were used to determine final concrete thickness and reinforcement.

[2] Interoffice Memorandum to Chief, General Engineering Branch, from H. Boyd Phillips and Ira E. Allen, "Electrical Analogy Tray Study of Pressures on Spillway Tunnel Lining, Glen Canyon Dam," October 29, 1956 (unpublished).

[3] Interoffice Memorandum to Chief, Dams Branch, from H. Boyd Phillips and Ira E. Allen, "Stress Analysis of Transition Section, Spillway Tunnel—Glen Canyon Dam," November 19, 1957 (unpublished).

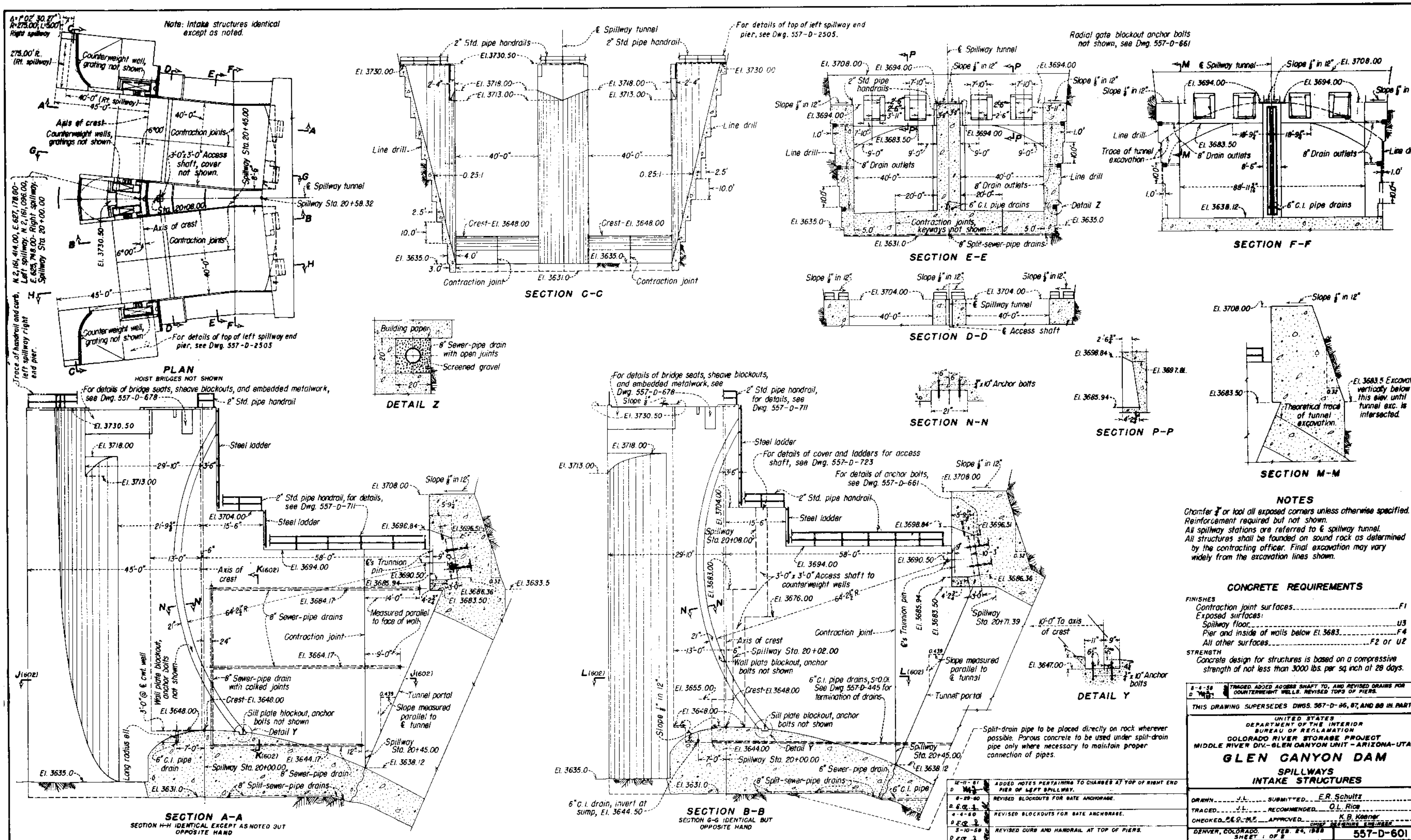
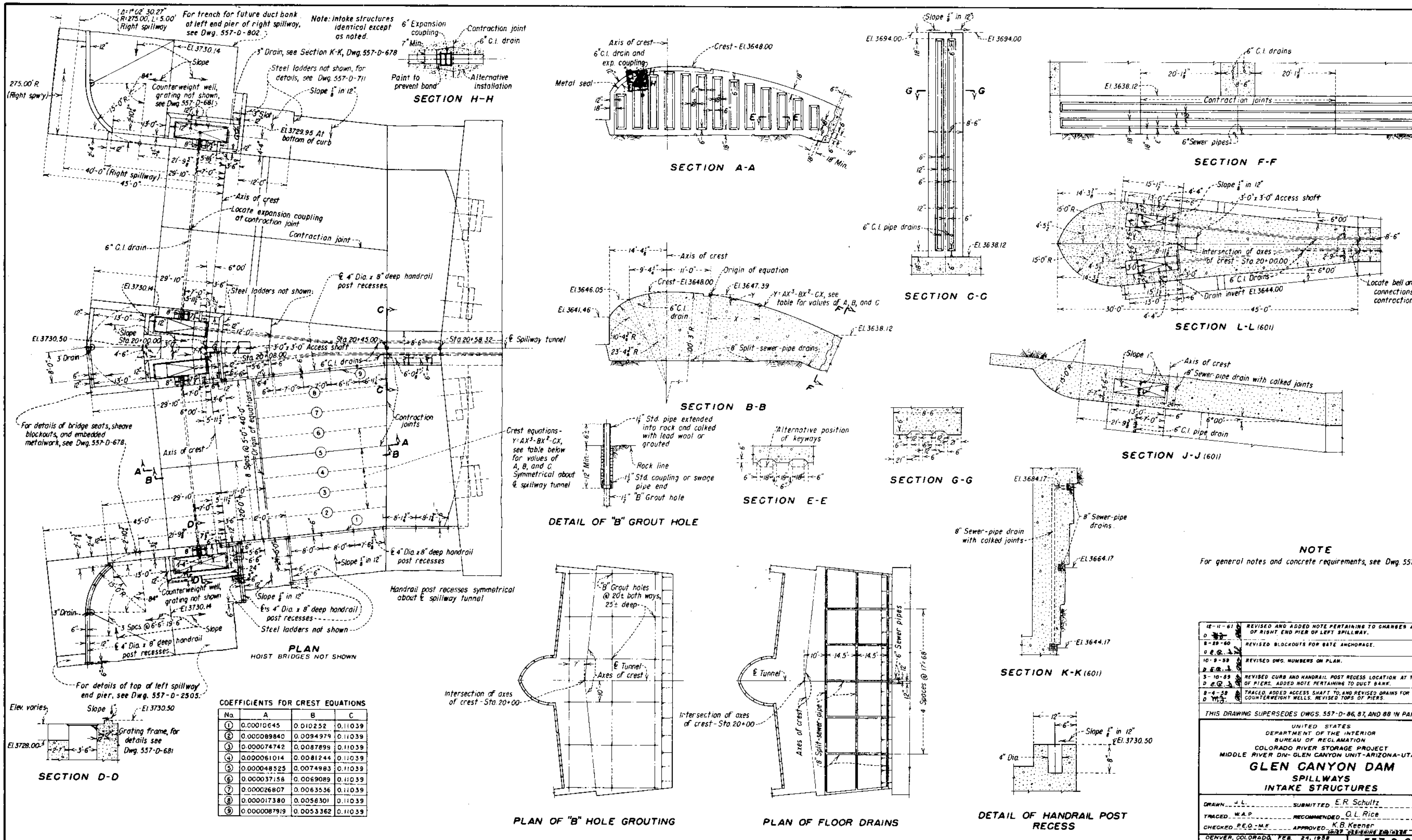


Figure 103.—Spillways intake structures. (Sheet 1 of 2.)

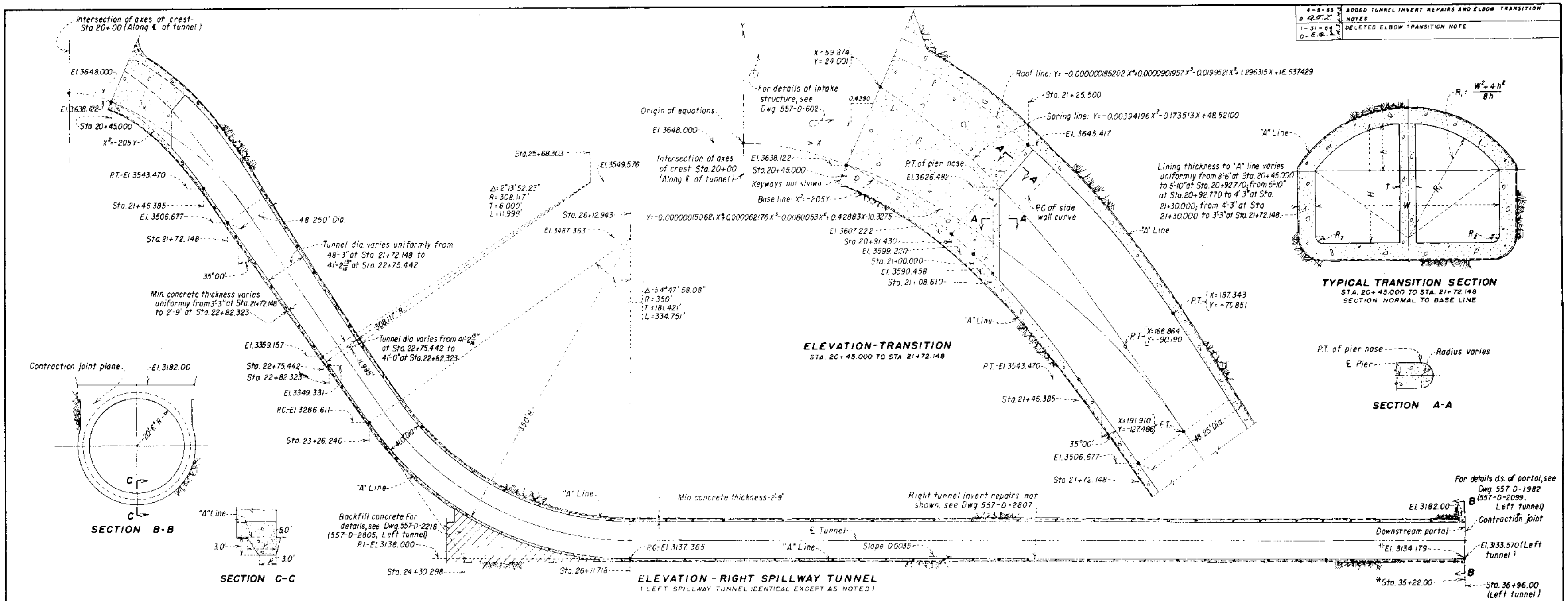


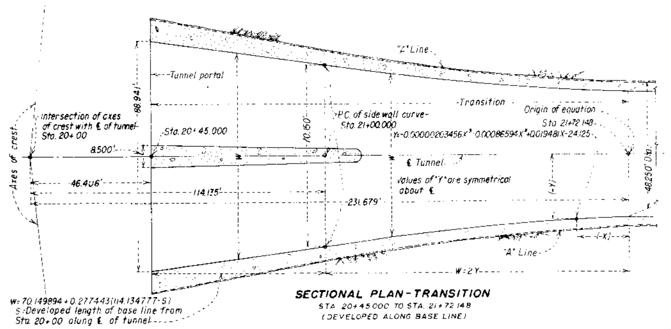
COEFFICIENTS FOR CREST EQUATIONS

No.	A	B	C
1	0.00010645	0.010252	0.11039
2	0.000089840	0.0094979	0.11039
3	0.000074742	0.0087899	0.11039
4	0.000061014	0.0081244	0.11039
5	0.000048525	0.0074983	0.11039
6	0.000037158	0.0069089	0.11039
7	0.000026807	0.0063536	0.11039
8	0.000017380	0.0058301	0.11039
9	0.0000087919	0.0053362	0.11039

Figure 103.—Spillways intake structures. (Sheet 2 of 2.)

4-3-63	ADDED TUNNEL INVERT REPAIRS AND ELBOW TRANSITION NOTES
1-31-61	DELETED ELBOW TRANSITION NOTE





BASE LINE		ROOF LINE		ELEMENTS OF TRANSITION SECTIONS					
Station	Elevation	Station	Elevation	H	h	W	R_1	R_2	T
20+45.000	3638.122	20+65.903	3685.735	52'-0"	15'-0"	88'-11 5/16"	73'-5 1/16"	0	8'-6"
20+48.650	3636.455	20+71.051	3683.651	52'-2 3/8"	16'-0 5/16"	87'-9 15/16"	68'-1 5/16"	5 3/16"	8'-3 7/8"
20+52.230	3634.693	20+75.995	3681.331	52'-4 1/8"	16'-10 7/8"	86'-8 5/8"	64'-0 11/16"	10 7/16"	8'-1 11/16"
20+55.760	3632.839	20+80.763	3678.826	52'-4 3/16"	17'-7 3/4"	85'-7 3/4"	60'-9 1/2"	15 7/16"	7'-11 9/16"
20+59.250	3630.875	20+85.418	3676.144	52'-3 7/16"	18'-3"	84'-6 1/16"	58'-0 3/16"	20 1/2"	7'-9 3/16"
20+62.680	3628.835	20+89.905	3673.356	52'-2 1/4"	18'-9 7/16"	83'-4 3/4"	55'-8"	2'-1 9/16"	7'-7 5/16"
20+66.080	3626.700	20+94.288	3670.454	52'-0 11/16"	19'-3 1/4"	82'-3 1/8"	53'-6 3/8"	2'-6 7/8"	7'-5 1/8"
20+69.440	3624.478	20+98.561	3667.464	51'-11 1/16"	19'-8 7/16"	81'-2"	51'-7 11/16"	3'-0 5/8"	7'-3"
20+72.710	3622.211	21+02.671	3664.448	51'-9 7/16"	20'-1 1/4"	80'-0 3/4"	49'-10 7/8"	3'-6"	7'-0 7/8"
20+75.940	3619.869	21+06.689	3661.372	51'-7 5/16"	20'-5 11/16"	78'-11 7/16"	48'-3 3/16"	3'-11 13/16"	6'-10 11/16"
20+79.140	3617.448	21+10.632	3658.236	51'-6 3/8"	20'-9 15/16"	77'-10 11/16"	46'-9 3/16"	4'-6 1/16"	6'-8 9/16"
20+82.300	3614.960	21+14.494	3655.056	51'-5 1/2"	21'-2"	76'-8 11/16"	45'-4 7/8"	5'-0 11/16"	6'-6 7/16"
20+85.370	3612.449	21+18.219	3651.889	51'-3 15/16"	21'-5 7/8"	75'-7 1/8"	44'-0 1/8"	5'-7 7/16"	6'-4 3/16"
20+88.430	3609.854	21+21.906	3648.657	51'-3"	21'-9 11/16"	74'-6 1/8"	42'-8 3/4"	6'-2 9/16"	6'-2 1/8"
20+91.430	3607.222	21+25.500	3645.417	51'-2 3/16"	22'-1 5/16"	73'-4 13/16"	41'-6 1/4"	6'-9 13/16"	6'-0"
20+94.400	3604.530	21+29.037	3642.139	51'-1 9/16"	22'-4 13/16"	72'-3 1/2"	40'-4 5/16"	7'-5 3/8"	5'-9 7/8"
20+97.310	3601.809	21+32.485	3638.860	51'-1 1/16"	22'-8 1/4"	71'-2 1/4"	39'-3 3/16"	8'-1 5/16"	5'-7 11/16"
21+00.000	3599.220	21+35.656	3635.767	51'-0 11/16"	22'-11 5/16"	70'-1 13/16"	38'-3 3/8"	8'-9 1/8"	5'-5 11/16"
21+00.200	3599.024	21+35.851	3635.535	51'-0 11/16"	22'-11 9/16"	70'-0 7/8"	38'-2 1/2"	8'-9 3/8"	5'-5 9/16"
21+03.040	3596.209	21+39.222	3632.201	51'-0 7/16"	23'-2 3/4"	68'-11 5/8"	37'-2 5/16"	9'-6 1/8"	5'-3 7/16"
21+05.860	3593.335	21+42.512	3628.824	51'-0 1/4"	23'-5 3/4"	67'-10 7/16"	36'-3 1/8"	10'-2 3/4"	5'-1 1/4"
21+08.610	3590.458	21+45.705	3625.466	51'-0 1/16"	23'-8 5/8"	66'-9 1/2"	35'-4 7/16"	10'-11 1/2"	4'-11 1/8"
21+11.310	3587.561	21+48.823	3622.105	50'-11 15/16"	23'-11 1/2"	65'-8 7/8"	34'-6 7/16"	11'-8 3/16"	
21+14.000	3584.605	21+51.912	3618.692	50'-11 13/16"	24'-1 15/16"	64'-8 1/4"	33'-8 13/16"	12'-5 1/4"	
21+16.670	3581.601	21+54.961	3615.241	50'-11 3/8"	24'-4 1/8"	63'-7 13/16"	32'-11 11/16"	13'-2 3/8"	

BASE LINE		ROOF LINE		ELEMENTS OF TRANSITION SECTIONS				
Station	Elevation	Station	Elevation	H	h	W	R_1	R_2
21+19.300	3578.573	21+57.946	3611.777	50'-11 3/8"	24'-6 1/4"	62'-7 3/8"	32'-3 1/8"	13'-11 7/16"
21+21.900	3575.514	21+60.878	3608.288	50'-11 1/8"	24'-8 7/8"	61'-7 11/16"	31'-7"	14'-8 3/16"
21+24.470	3572.425	21+63.756	3604.777	50'-10 11/16"	24'-9 3/8"	60'-7 9/16"	30'-11 3/8"	15'-5 3/8"
21+26.970	3569.359	21+66.536	3601.300	50'-10 3/16"	24'-11 1/8"	59'-8 3/4"	30'-4 1/4"	16'-2 7/16"
21+29.480	3566.219	21+69.307	3597.747	50'-9 9/16"	25'-0 1/4"	58'-9 5/8"	29'-9 3/8"	16'-11 1/4"
21+31.960	3563.056	21+72.023	3594.175	50'-8 7/8"	25'-1 1/16"	57'-10 3/8"	29'-3"	17'-7 13/16"
21+34.390	3559.912	21+74.651	3590.630	50'-7 13/16"	25'-1 3/8"	57'-0 1/2"	28'-9"	18'-4 1/4"
21+36.800	3556.711	21+77.258	3587.025	50'-6 5/8"	25'-1 5/8"	56'-2 1/8"	28'-3 1/4"	19'-0 7/8"
21+39.180	3553.507	21+79.798	3583.420	50'-5 5/16"	25'-1 7/8"	55'-4 1/4"	27'-9 13/16"	19'-8 3/8"
21+41.530	3550.289	21+82.283	3579.804	50'-3 15/16"	25'-1 9/16"	54'-7 9/16"	27'-4 7/8"	20'-4"
21+43.880	3547.017	21+84.745	3576.130	50'-2 1/8"	25'-1"	53'-10 5/8"	27'-0 1/8"	20'-11 3/8"
21+46.180	3543.763	21+87.131	3572.477	50'-0 3/16"	25'-0 1/16"	53'-2 1/2"	26'-7 3/4"	21'-6 3/16"
21+46.385	3543.470	21+87.343	3572.149	50'-0"	25'-0"	53'-1 1/2"	26'-7 5/16"	21'-6 13/16"
21+48.470	3540.493	21+89.312	3569.090	49'-10 3/16"	24'-11 1/8"	52'-6 5/16"	26'-3 9/16"	22'-0 9/16"
21+50.770	3537.208	21+91.484	3565.716	49'-8 1/16"	24'-10 3/16"	51'-10 3/4"	25'-11 11/16"	22'-6 1/8"
21+53.060	3533.938	21+93.646	3562.356	49'-6 5/16"	24'-9 1/4"	51'-3 3/4"	25'-8 1/16"	22'-10 13/16"
21+55.360	3530.653	21+95.818	3558.982	49'-4 11/16"	24'-8 5/16"	50'-9 1/4"	25'-4 3/4"	23'-3 1/8"
21+57.650	3527.382	21+97.981	3555.622	49'-2 13/16"	24'-7 5/16"	50'-3 5/16"	25'-1 3/4"	23'-6 3/8"
21+59.950	3524.098	22+00.153	3552.248	49'-0 13/16"	24'-6 1/2"	49'-9 7/8"	24'-11"	23'-9 1/2"
21+62.240	3520.827	22+02.315	3549.888	48'-11 1/16"	24'-5 5/16"	49'-5"	24'-8 1/2"	23'-11 3/8"
21+64.530	3517.557	22+04.478	3545.529	48'-9 5/16"	24'-4 3/8"	49'-0 1/4"	24'-6 3/8"	24'-1 1/8"
21+66.830	3514.272	22+06.650	3542.154	48'-7 3/16"	24'-3 1/16"	48'-9 11/16"	24'-4 5/16"	24'-2"
21+69.120	3511.001	22+08.813	3538.795	48'-5 7/16"	24'-2 3/4"	48'-6 1/4"	24'-3"	24'-2 3/16"
21+71.420	3507.717	22+10.985	3535.420	48'-3 3/16"	24'-1 13/16"	48'-3 3/8"	24'-1 13/16"	24'-1 5/8"
21+72.148	3506.677	22+11.672	3534.352	48'-3"	24'-1 1/2"	48'-3"	24'-1 1/2"	24'-1 1/2"

TRANSITION DATA
STA. 20+45.000 TO STA. 21+72.148
SECTIONS NORMAL TO BASE LINE

2-17-59	RELOCATED LEFT TUNNEL PORTAL FROM STA. 36+94 TO STA. 36+96 AND REVISED INVERT ELEVATION
12-29-58	RELOCATED LEFT TUNNEL PORTAL FROM STA 36+54 TO STA 36+94 AND REVISED INVERT ELEVATION.

CONCRETE REQUIREMENTS

FINISHES
Tunnel below ℄ or spring line .... F4 or U3
Tunnel below ℄ from Sta. 23+20 to Sta 26+15 .... F4 with special finish or U3
All other surfaces .... F2 or U2

STRENGTH
Concrete design is based on a compressive strength of not less than 3000 lbs. per sq. inch at 28 days.

NOTES

Reinforcement required but not shown
Chamfer all exposed corners 3/4" unless otherwise specified.
"A" Line is line of minimum concrete thickness.

THIS DRAWING SUPERSEDES DWGS. 557-D-86 AND 87 IN PART

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZONA-UTAH

GLEN CANYON DAM
SPILLWAYS
RIGHT AND LEFT TUNNELS
ELEVATIONS AND SECTIONS

DRAWN ... SUBMITTED E.R. Schultz
TRACED ... RECOMMENDED O.L. Rice
CHECKED ... APPROVED K.B. Keener
CHIEF DESIGNING ENGINEER
DENVER, COLORADO, DEC. 6, 1957
557-D-444

Figure 104.–Spillways, right and left tunnels–Elevations and sections.

The rock surrounding the tunnels upstream from the downstream end of the elbow and at the downstream portal was grouted from radial holes ranging from 20 to 40 feet in depth, spaced at up to 20-foot centers measured along the centerline of the tunnels. Drainage holes 3 inches in diameter, drilled radially 25 feet deep at 20-foot centers, were also provided in the tunnels from the entrances to a point 400 feet downstream from the elbows. Grouting and drainage details are shown on figure 105.

The inverts of the horizontal portions of both tunnels were eroded during diversion. Repairs were made with epoxy-bonded concrete and epoxy-bonded epoxy mortar to insure the serviceability of the tunnels.

(c) *Deflector Bucket.*—The layout of the left spillway deflector bucket is shown on figure 106. The layout for the right deflector bucket is similar but opposite hand.

The reinforced concrete deflector buckets were designed for dead weight, static and dynamic forces from water at maximum discharge, and uplift pressure. In general, the bucket was designed as a monolithic structure resting on an elastic foundation. The dynamic forces from the deflecting jet were determined from the hydraulic models.[1]

Seams and joints were observed in the rock at the outlet portal of the left tunnel. To prevent the rock from slabbing off, 1-1/2-inch-diameter post-tensioned anchors 20 feet long were installed as shown on figure 22. These were tensioned by applying a torque of between 450 and 550 foot-pounds.

45. 40- BY 52.5-FOOT RADIAL GATES. (a) *Description.*—Four radial gates are installed in the spillway intake structures and are used to regulate discharges into the spillway tunnels. The gates were manufactured by Vereinigte Osterreichische Eisen und Stahlwerke (VOEST), Linz-Donau, Austria under invitation No. DS-5192. The embedded metalwork for the gates was manufactured by Dixie Steel and Supply Company, Inc., Tuscaloosa, Ala., under invitation No. DS-5077.

The radial gates are located near the crest of the spillway intake structures, upstream from the right and left abutments of the dam. The general installation is shown on figure 107. Each gate is operated by a wire-rope drum hoist on the operating bridge with movable counterweights in the pier and end wall of the spillway.

[1]Op. cit. p. 145.

The estimated weight of the movable parts of each gate is 326,000 pounds, and the estimated weight of the nonmovable parts is 64,000 pounds. Figure 108 shows the shop assembly of one gate.

(b) *Design.*—The gate was designed for a head of 52.5 feet for two conditions; namely, (1) water surface at top of gate, elevation 3700.0 (gate closed), and (2) water surface at top of gate, elevation 3715.0 (gate partially raised). The hydraulic force on the skinplate and vertical stiffeners is transmitted to the four horizontal girders. The load from the girders is carried to the arms and trunnion pins which are located at the center of curvature of the skinplate. The load from the trunnion pins is carried to the pedestals which are attached to base castings embedded in the concrete headwall of the spillway tunnel. The pedestals were alined and held in place by bolts and jacks before making the final placement of concrete in the headwall blockouts. The trunnion pin bushing is provided with seals to prevent entry of foreign matter.

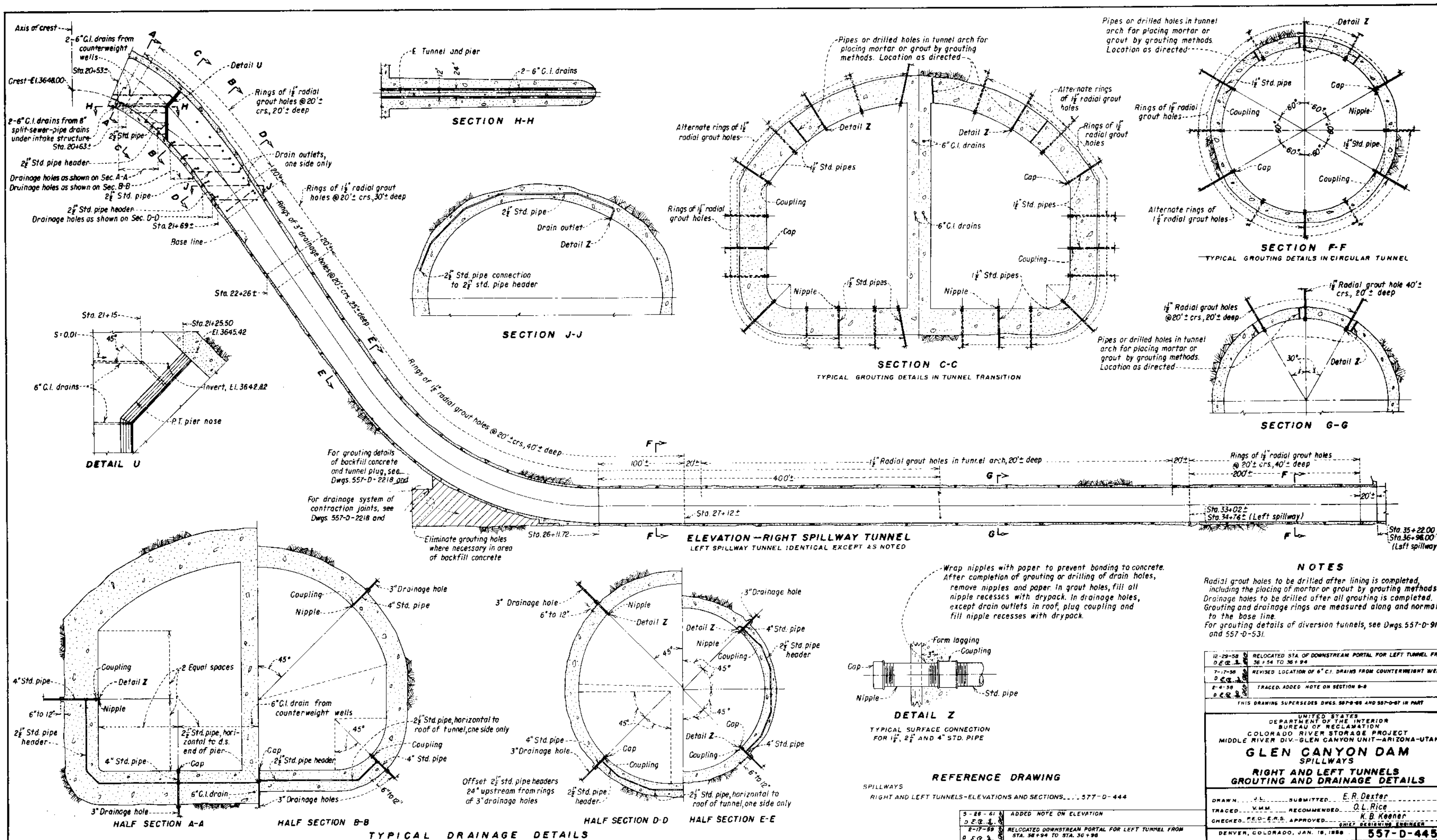
Grease fittings are provided for lubrication of the bushing. Lateral movement of the gate is controlled by four guide shoes on each side.

The rubber side seals are provided with brass bars to reduce the sliding friction. The flat bar rubber bottom seal extends below the lower edge of the skinplate and is compressed to prevent leakage when the gate is seated.

Two sets of hoist wire ropes and two counterweight ropes are connected to the lifting brackets on the skinplate. Part of the lifted load is carried to the hoist and the remainder to the counterweights in the pier and end wall. The maximum load on the ropes is 262,000 pounds.

The design was based on the following unit stresses expressed in pounds per square inch:

Combined longitudinal and transverse stresses in skinplate	20,000
Tension	
Net sectional of rolled members	18,000
Upset anchor bolts	15,000
Bending	
Compression in girders and rolled members	18,000
Trunnion pins	16,000
Shear	
Webs of girders and rolled members	12,000
Rivets	13,500
Ribbed bolts	15,000



Figures 105.—Spillways, right and left tunnels—Grouting and drainage details.

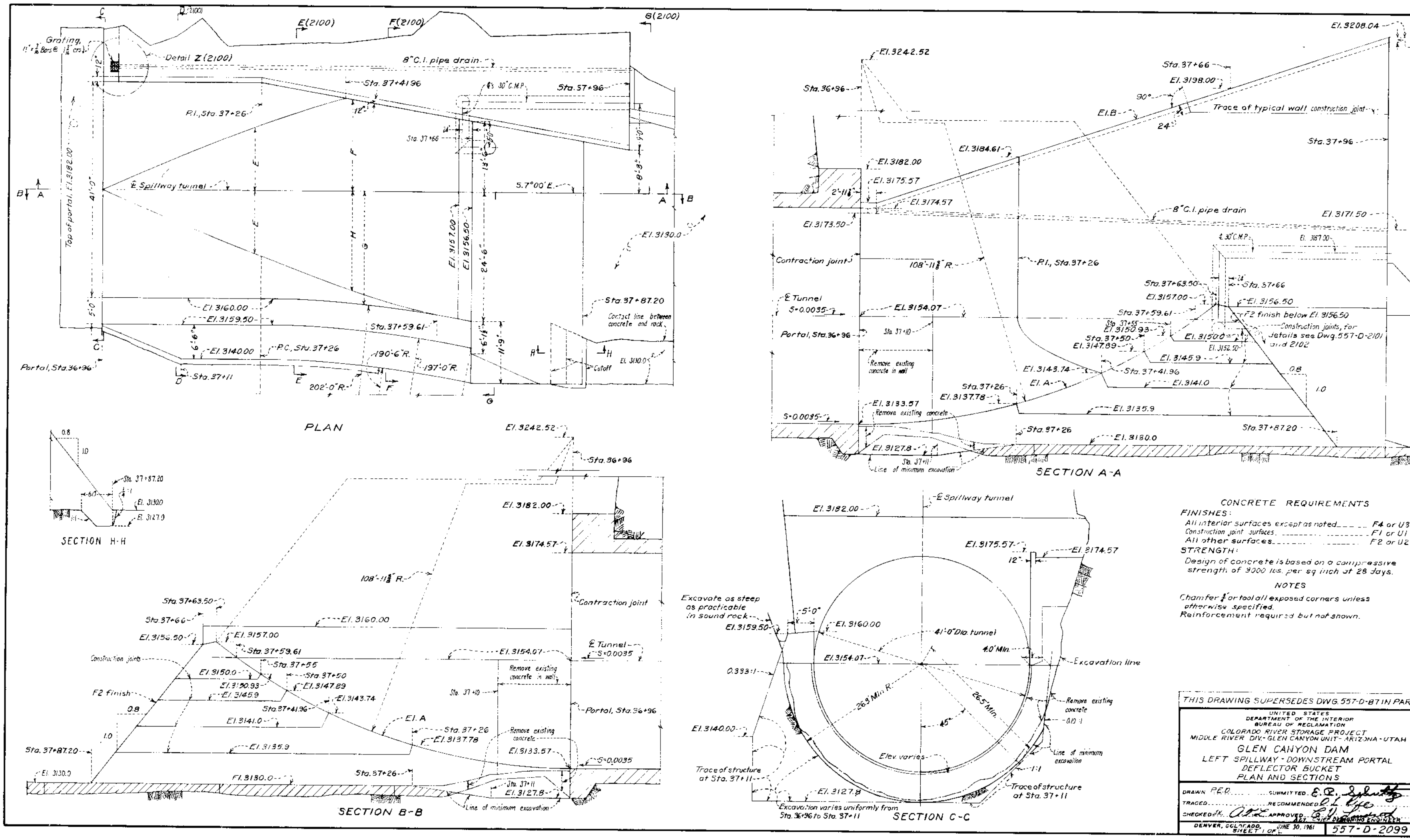
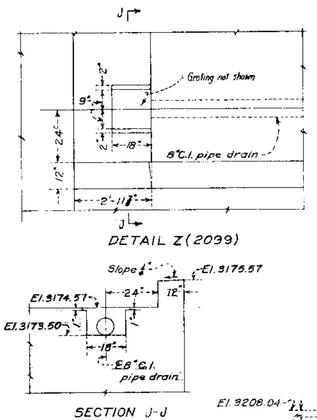
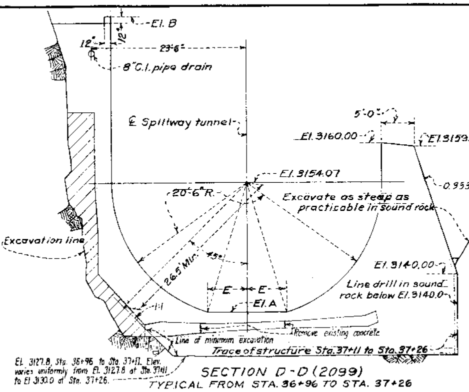
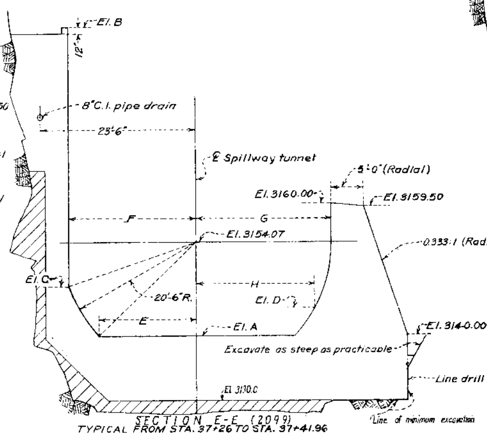
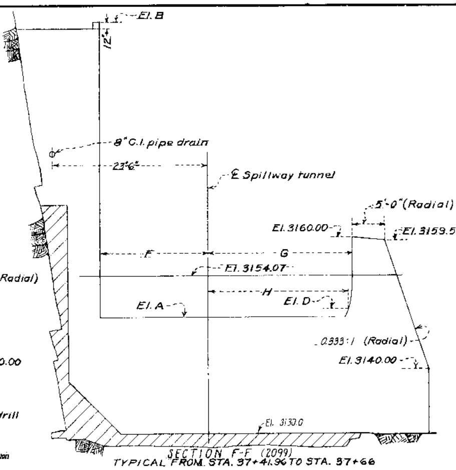


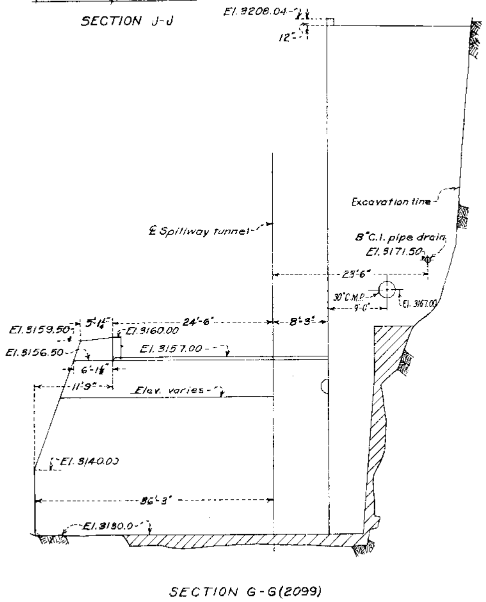
Figure 106.–Left spillway downstream portal deflector bucket–Plan and sections. (Sheet 1 of 2.)











STATION	El. A	El. B	E
36+96	3133.57	3175.57	0
36+98	3133.59	3175.57	10⅝"
37+00	3133.64	3175.91	20¾"
37+02.50	3133.76	3176.75	2'-9¾"
37+05	3133.94	3177.58	3'-10⅝"
37+07.50	3134.18	3178.42	4'-11½"
37+10	3134.47	3179.26	6'-0¼"
37+12.50	3134.83	3180.09	7'-0¾"
37+15	3135.24	3180.93	8'-1¼"
37+17.50	3135.71	3181.77	9'-1⅛"
37+20	3136.25	3182.60	10'-1⅛"
37+23	3136.97	3183.61	11'-3⅜"
37+26	3137.78	3184.61	12'-5⅜"

TRANSITION DATA

STATION	El. A	El. B	El. C	E	F	G	El. D
37+28	3138.38	3185.28	3150.30	13'-2¼"	20'-1⅜"	20'-6⅛"	3138.38
37+30	3139.01	3185.95	3148.76	13'-10⅞"	19'-9⅜"	20'-6½"	3139.01
37+32.50	3139.87	3186.79	3147.34	14'-9⅜"	19'-4⅜"	20'-7¼"	3139.87
37+35	3140.79	3187.62	3146.19	15'-7⅜"	18'-11⅛"	20'-8⅝"	3140.79
37+37.50	3141.78	3188.46	3145.21	16'-4⅞"	18'-5⅞"	20'-9⅜"	3141.78
37+40	3142.85	3189.30	3144.35	17'-1⅞"	18'-0⅝"	20'-11⅞"	3142.85
37+41.96	3143.74	3189.95	3143.74	17'-8½"	17'-8½"	21'-1⅝"	3143.74
37+42.50	3143.99	3190.15			17'-7⅜"	21'-2⅜"	3143.99
37+45	3145.21	3190.97			17'-2⅜"	21'-4¾"	3145.21
37+47.50	3146.51	3191.81			16'-8⅞"	21'-7⅜"	3146.51
37+50	3147.89	3192.64			16'-3⅝"	21'-11⅛"	3147.89
37+52.50	3149.37	3193.48			15'-10⅝"	22'-3"	3149.37
37+55	3150.93	3194.32			15'-5¼"	22'-7⅛"	3150.93
37+57.50	3152.59	3195.15			14'-11⅞"	22'-11⅝"	3152.59
37+59.61	3154.07	3195.86			14'-7⅜"	23'-3¾"	3154.07
37+60	3154.35	3195.99			14'-6⅝"	23'-4⅜"	
37+63.50	3157.00	3197.16			13'-11¼"	24'-0⅛"	
37+66	3156.50	3198.00			13'-6"	24'-6"	
37+96		3208.04			8'-3"	---	

VALUES OF OFFSET H

El. D \ STA.	37+28	37+30	37+32.50	37+35	37+37.50	37+40	37+41.96	37+42.50	37+45	37+47.50	37+50	37+52.50	37+55	37+57.50	37+59.61
3138.38	13'-2¼"														
3139.01	13'-11"	13'-11⅜"													
3139.87	14'-9½"	14'-9⅞"	14'-10⅝"												
3140.79	15'-7½"	15'-7⅞"	15'-8⅝"	15'-9¾"											
3141.78	16'-5"	16'-5⅜"	16'-6¼"	16'-7¼"	16'-8¾"										
3142.85	17'-2"	17'-2⅜"	17'-3⅛"	17'-4¼"	17'-5½"	17'-7⅝"									
3143.74	17'-8⅜"	17'-9"	17'-9¾"	17'-10⅞"	18'-0⅛"	18'-2¼"	18'-4"								
3143.99	17'-10⅜"	17'-10¾"	17'-11⅛"	18'-0¾"	18'-2⅛"	18'-4"	18'-5¾"	18'-6¼"							
3145.21	18'-6"	18'-6⅜"	18'-7⅛"	18'-8¼"	18'-9¾"	18'-11⅜"	19'-1⅜"	19'-1⅜"	19'-4½"						
3146.51	19'-0¾"	19'-1⅛"	19'-1⅞"	19'-3"	19'-4⅜"	19'-6½"	19'-8¼"	19'-8¾"	19'-11⅜"	20'-2⅜"					
3147.89	19'-6½"	19'-7"	19'-7⅜"	19'-9"	19'-10⅛"	20'-0⅜"	20'-2⅛"	20'-2⅞"	20'-5¼"	20'-8¼"	20'-11⅝"				
3149.37	19'-11½"	19'-11⅞"	20'-0⅜"	20'-1⅛"	20'-3⅞"	20'-5¼"	20'-7"	20'-7½"	20'-10⅛"	21'-1⅛"	21'-4½"	21'-8⅜"			
3150.93	20'-3¼"	20'-3⅝"	20'-4⅜"	20'-5½"	20'-7"	20'-8⅞"	20'-10⅜"	20'-11⅛"	21'-1⅞"	21'-4⅜"	21'-8¼"	22'-0"	22'-4⅜"		
3152.59	20'-5⅛"	20'-5⅞"	20'-6⅝"	20'-7⅞"	20'-9¼"	20'-11⅛"	21'-0⅞"	21'-1½"	21'-4⅛"	21'-7¼"	21'-10½"	22'-2¼"	22'-6½"	22'-11"	
3154.07	20'-6⅛"	20'-6½"	20'-7¼"	20'-8⅝"	20'-9⅝"	20'-11⅞"	21'-1⅜"	21'-2⅛"	21'-4⅞"	21'-7¾"	21'-11⅛"	22'-3"	22'-7⅛"	22'-11⅝"	23'-3⅜"

NOTE
For general notes and concrete requirements, see Dwg. 557-D-2099.

THIS DRAWING SUPERSEDES DWG. 557-D-87 IN PART

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZONA-UTAH
GLEN CANYON DAM
LEFT SPILLWAY-DOWNSTREAM PORTAL
DEFLECTOR BUCKET
PLAN AND SECTIONS

DRAWN: P.E.Q. SUBMITTED: E. R. Schultz
TRACED: RECOMMENDED: O. L. Rice
CHECKED: A.T.L. APPROVED: C. G. Lindstrom, CHIEF DESIGNING ENGINEER
DENVER, COLORADO, JUNE 30, 1961 SHEET 2 OF 2 557-D-2100

Figure 106.—Left spillway downstream portal deflector bucket—Plan and sections. (Sheet 2 of 2.)

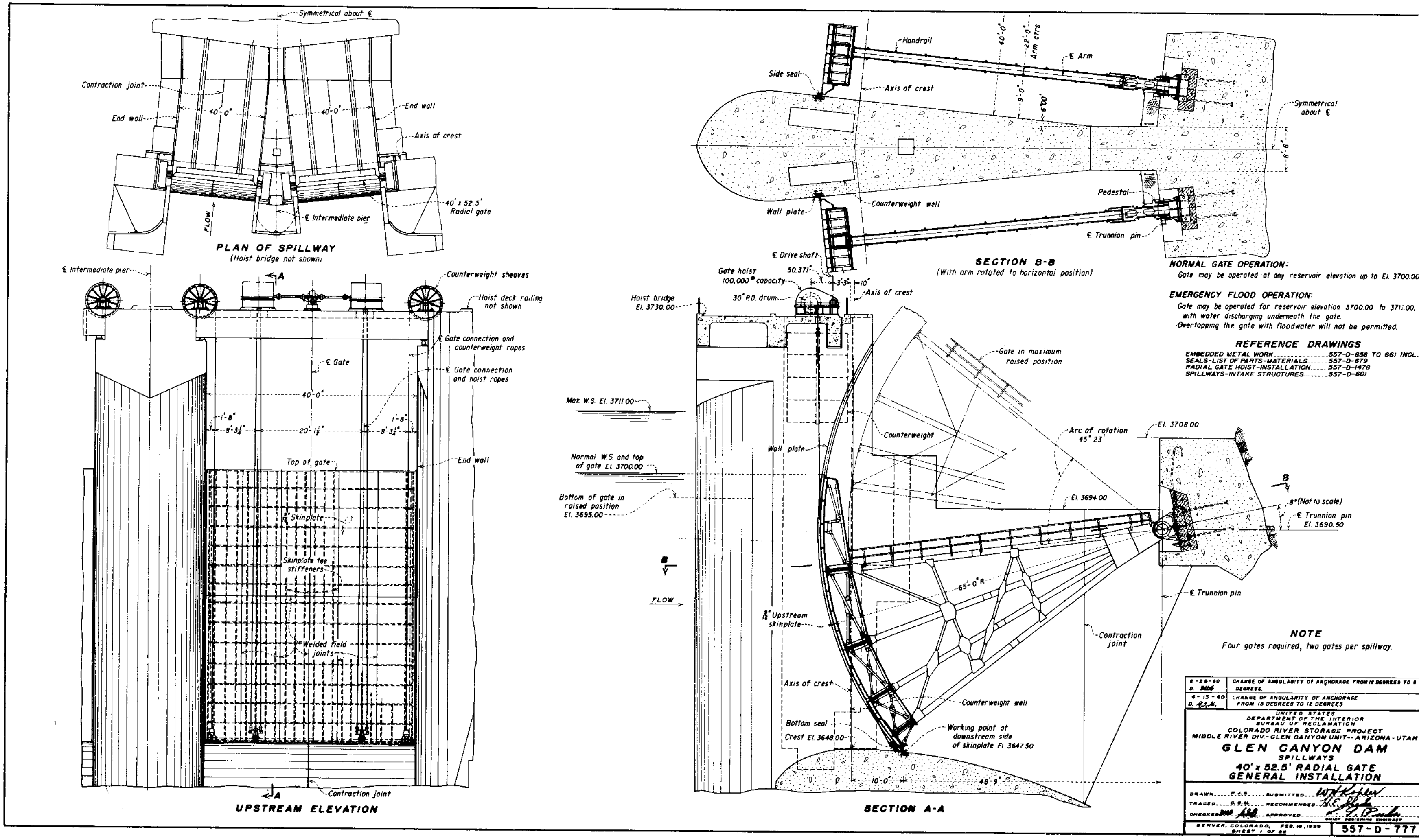


Figure 107.—Dam spillways, 40- by 52.5-foot radial gate—General installation.

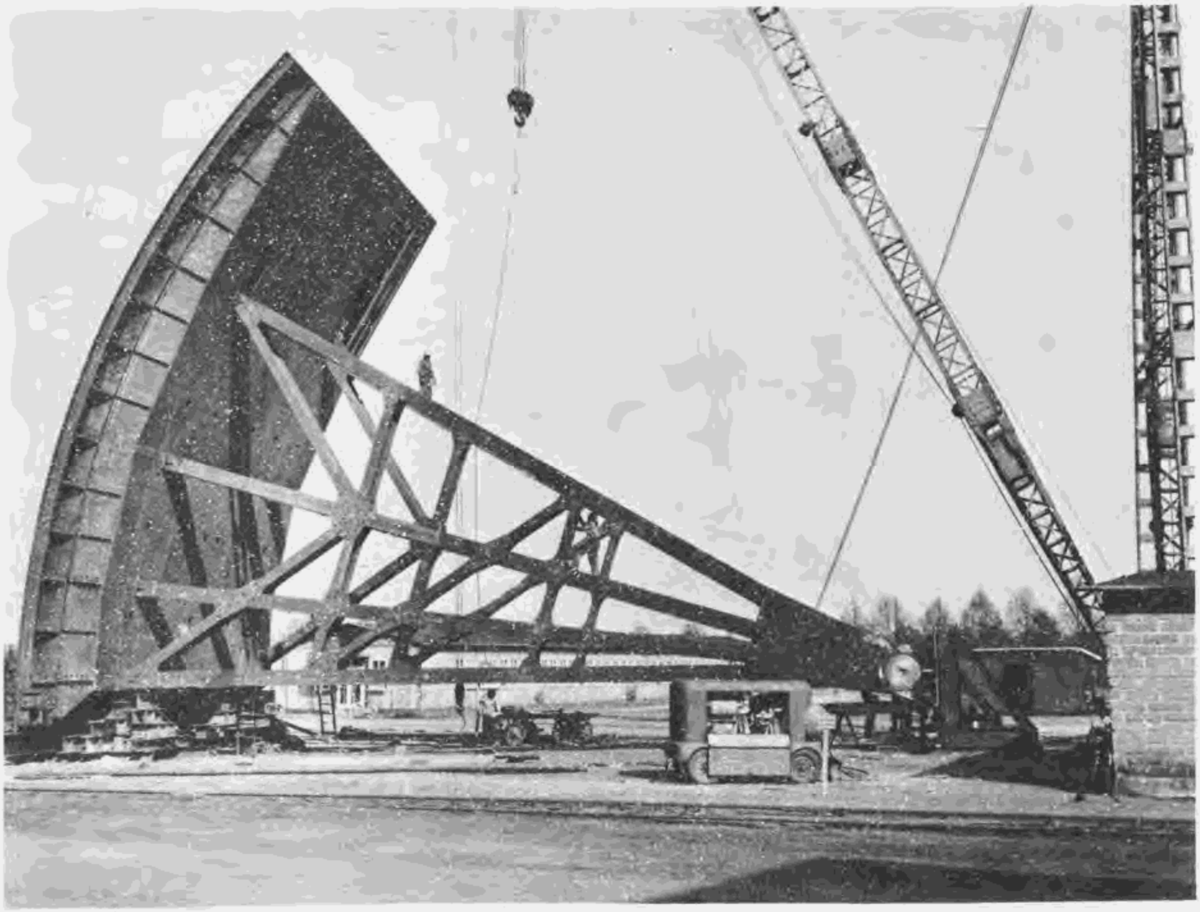


Figure 108.—40- by 52.5-foot radial gate for spillway—Side elevation of shop assembly. TPX-D-21976.

Bearing	
Rivets and ribbed bolts, single shear	24,000
Rivets and ribbed bolts, double shear	30,000
Trunnion pins	20,000
Bronze bushings, projected area	3,500
Compression	
Arm columns, maximum $\frac{l}{r} = 61$	14,910
Assumed coefficient of side seal friction, brass on stainless steel	F = 0.25
Assumed coefficient of trunnion pin friction, bronze or steel	F = 0.10

46. SPILLWAY RADIAL GATE HOISTS (a) *Description.*—Two 100,000-pound-capacity radial gate hoists are installed on the operating deck of each spillway for raising and lowering the radial gates. The gates were furnished under invitation No. DS-5213.

Each hoist consists of a center-drive unit and two drum units. The drum units are mounted on the hoist deck directly above the gate rope connections. The center-drive unit is mounted on the hoist deck on the centerline of the gate. There are two concrete counterweights, each weighing approximately 80,000 pounds, connected to each gate to reduce the lifting effort required of the hoist. A single 2-1/4-inch-diameter 6 by 37 stainless steel wire rope with independant wire-rope core is reeved over a 6-foot-diameter sheave connecting each counterweight to the gate. The counterweights are located in wells in the piers adjacent to each gate.

The center-drive unit consists of a 10-horsepower, 350-r.p.m., gear motor with a motor-mounted, electrical-release, disk-type brake and a commercial 50 to 1 worm gear reducer with a double extended output shaft. A floating shaft and two flexible couplings connect the center unit to each drum unit.

Each drum unit consists of two stages of spur reduction gearing and a 30-inch pitch-diameter drum. The drum is grooved right and left hand for 2 parts of 1-1/2-inch-diameter 6 by 37 stainless steel wire rope with independant wire-rope core. The ropes extend from the drum and are attached to an equalizer bar located on the upstream side of the gate.

The drum unit base is of welded steel construction. Antifriction roller bearing pillow blocks support the drum and spur gear shafts. A double-acting, screw-type, snap action limit switch is installed on one drum unit of each hoist to limit the travel in both the hoisting and lower directions.

Each counterweight consists of four reinforced concrete blocks suspended on a steel hanger. The bottom block of each counterweight is provided with a steel lifting bar at each end to permit assembly of all four blocks onto the hanger. Guide wheels are mounted on top of each counterweight to prevent rotation and binding in the wells due to the twist of the wire rope. The counterweight sheaves are bronze bushed and are designed to rotate on the sheave shaft. Each sheave and shaft is supported by two welded steel pedestals mounted on the piers.

(b) *Design.*—The hoists were designed with a safety factor of 5 based on the ultimate strength of the material and assuming 60 percent of the total rated torque applied on one end unit. Also, each part was designed so that the stress will not exceed 80 percent of the yield point of the material, based on the breakdown torque of the motor and 50 percent of the torque applied at each end unit. The hoist ropes were designed with a factor of safety of 5.5 and the counterweight ropes with a factor of safety of 4.4 on the breaking strength of the ropes.

Electric motor control equipment is discussed in section 39.

CHAPTER VI. *Design*—OUTLET WORKS

47. GENERAL. River outlets having a capacity of 15,000 cubic feet per second with the reservoir at elevation 3490, which is the minimum water surface for power operation, were installed in the dam near the left abutment. The river outlets provide for releases for downstream commitments when the powerplant is not in operation and during the period of final closure of the diversion tunnels. The outlets will also be used to maximum capacity during maximum flood releases. The centerline of the intake is at elevation 3374 which is about 30 feet above the estimated 100-year silt level in the reservoir.

The outlet works consists of four 96-inch-diameter steel pipes with cast iron bellmouth intakes, hollow-jet valves for regulation, and ring-follower gates for emergency closure. A bulkhead gate, which operates under balanced head, is provided at the upstream face of the dam to provide access for servicing the ring-follower gates. A reinforced concrete trashrack structure with structural steel bars protects the entrance. The location of the outlet works is shown on figures 109 and 110.

To avoid excessive velocities in the outlet pipes, a criterion was established limiting the maximum discharge of each outlet pipe to 3,750 cubic feet per second except in cases of emergency. Discharge curves are shown on figure 111.

48. SELECTION OF LOCATION. The best arrangement for the outlets in the dam was found to be two parallel outlets in each of two 60-foot-wide blocks. The centerline distance between the outlets in the dam, 15 feet 7 inches, was dictated by the required clearance for the bulkhead gate frame metalwork around the bellmouth intake. The minimum distance between the centerlines of the outlets and the radial contraction joints in the dam was set at 1-1/2 outlet diameters. Radii of bends are 4 diameters except where lack of space required using a bend of 3 diameters.

The ring-follower gates are located in a chamber (fig. 112) in the dam 60 feet downstream from the face of the dam. To facilitate installation of the gates, blockouts were provided in the gate chamber floor. A vertical shaft from the chamber to the roadway at the top of the dam was provided for removing gate parts for servicing. Special expansion joints in vaults, to allow for movement in three directions, were designed for the outlet pipes where they leave the dam. These joints accommodate the movements of the dam when the dam is fully loaded (fig. 113).

Since the powerplant structure occupies the entire bottom of the canyon from abutment to abutment, it was necessary to locate the outlets in the mass concrete beneath the service bay and machine shop as they leave the dam. In this area, they were set two above each other at minimum spacing. Beyond the machine shop the outlets were encased in concrete and located below the powerplant parking area. The hollow-jet regulating valves are located about 700 feet downstream from the axis of the dam. In this area, the outlet pipes are spread apart and are all brought to elevation 3175. The layout of the downstream end of the river outlets is shown on figure 114. The location of the valves and their operation were studied in a hydraulic model.[1]

49. STRUCTURAL DESIGN. (a) *General.*—The design of the river outlets was based on concrete having a compressive strength of 3,000 pounds per square inch at 28 days for structural concrete and 2,500 pounds per square inch at 28 days for mass concrete.

The allowable working stresses are shown on figure 71, except that the allowable stress in the reinforcement around the pipes in the dam was increased to 25,000 pounds per square inch.

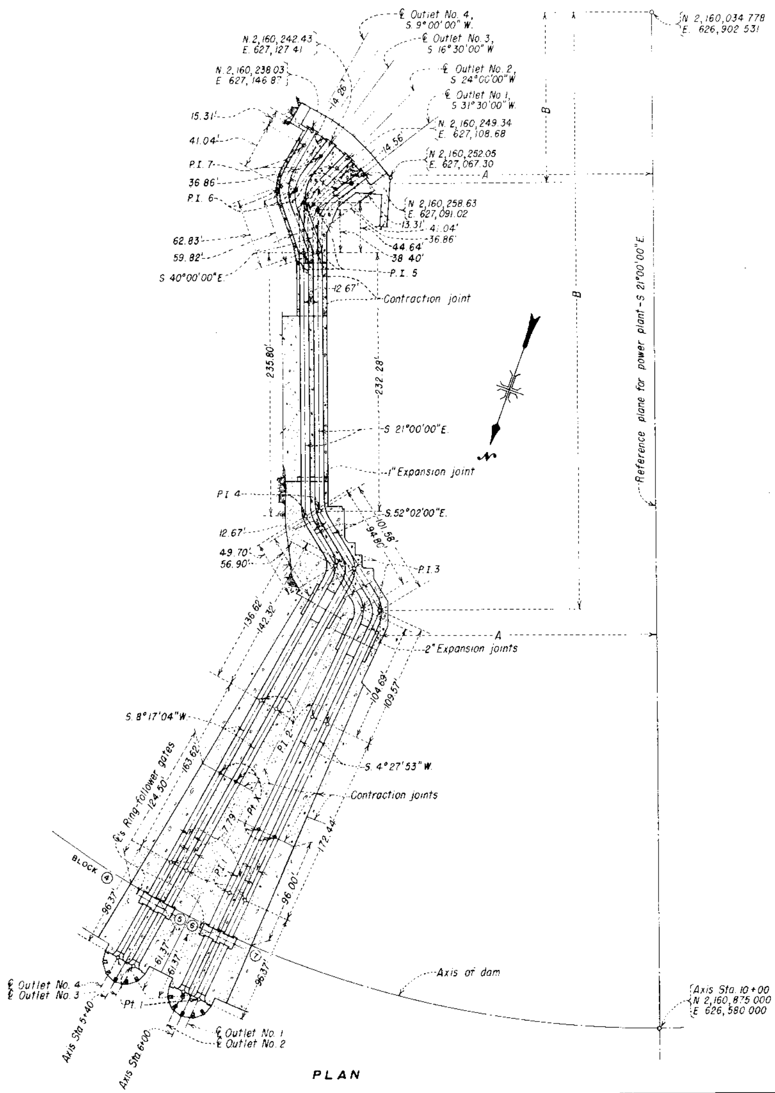
(b) *Trashrack Structures.*—The concrete trashrack structures (fig. 115) were designed for a differential waterload of 20 feet, temperature effects, and dead load.

(c) *Outlet Pipes.*—The reinforcement requirements around the outlets and in the surrounding mass concrete were as follows (fig. 110):

(1) Bellmouth casting at intake. It was assumed that the casting could take no tensile stress; therefore the opening was reinforced for the total tensile forces due to dam stresses, internal bursting pressure, and temperature effects.

(2) From the downstream end of the bellmouth intake to the upstream edge of the ring-follower gate blockout and from the

[1] "Hydraulic Model Studies of the Spillways and Outlet Works—Glen Canyon Dam." Hydraulic Laboratory Report No. 469, Bureau of Reclamation, February 18, 1964 (unpublished).



	A	B	COORDINATES		BEND DATA			TYPE OF BEND
			N	E	Δ	R	T	
OUTLET No. 1								
Pt. 1			2,160,993.83	626,969.60				
P.I. 1			2,160,897.76	626,962.10	50°00'00"	32.00'	14.92'	Vertical
P.I. 2			2,160,725.85	626,948.68	50°00'00"	32.00'	14.92'	Vertical
P.I. 3	243.63'	529.71'	2,160,616.62	626,940.15	56°29'53"	24.00'	12.90'	Horizontal
P.I. 4	296.00'	442.67'	2,160,554.12	627,020.23	31°02'00"	32.00'	8.88'	Horizontal
P.I. 5	296.00'	171.98'	2,160,301.41	627,117.24	53°09'54"	32.00'	16.01'	Combined
P.I. 6	266.75'	149.54'	2,160,269.98	627,097.98	10°00'00"	32.00'	2.80'	Vertical
Pt. X			2,160,836.50	626,957.32				
OUTLET No. 2								
Pt. 1			2,160,992.62	626,985.14				
P.I. 1			2,160,896.55	626,977.64	50°00'00"	32.00'	14.92'	Vertical
P.I. 2			2,160,724.63	626,964.21	50°00'00"	32.00'	14.92'	Vertical
P.I. 3	259.80'	527.41'	2,160,620.26	626,956.06	56°29'53"	24.00'	12.90'	Horizontal
P.I. 4	308.67'	446.18'	2,160,561.94	627,030.80	31°02'00"	32.00'	8.88'	Horizontal
P.I. 5	308.67'	165.75'	2,160,300.13	627,131.30	45°42'04"	32.00'	13.48'	Combined
P.I. 6	279.65'	136.73'	2,160,262.64	627,114.61	9°00'00"	32.00'	2.52'	Vertical
Pt. X			2,160,835.29	626,972.85				
OUTLET No. 3								
Pt. 1			2,160,987.24	627,033.34				
P.I. 1			2,160,891.87	627,019.46	50°00'00"	32.00'	14.92'	Vertical
P.I. 2			2,160,729.96	626,995.88	50°00'00"	32.00'	14.92'	Vertical
P.I. 3	266.67'	491.42'	2,160,589.13	626,975.37	60°19'04"	24.00'	13.95'	Horizontal
P.I. 4	296.00'	442.67'	2,160,554.12	627,020.23	31°02'00"	32.00'	8.88'	Horizontal
P.I. 5	296.00'	210.39'	2,160,337.27	627,103.48	19°00'00"	32.00'	5.35'	Horizontal
P.I. 6	315.47'	153.83'	2,160,291.45	627,141.92	56°43'16"	32.00'	17.27'	Combined
P.I. 7	293.03'	124.58'	2,160,256.10	627,131.45	6°11'34"	32.00'	1.73'	Vertical
Pt. X			2,160,802.88	627,006.50				
OUTLET No. 4								
Pt. 1			2,160,984.99	627,048.76				
P.I. 1			2,160,889.63	627,034.88	50°00'00"	32.00'	14.92'	Vertical
P.I. 2			2,160,727.71	627,011.30	50°00'00"	32.00'	14.92'	Vertical
P.I. 3	283.04'	488.77'	2,160,592.52	626,991.62	60°19'04"	24.00'	13.95'	Horizontal
P.I. 4	308.67'	440.18'	2,160,561.94	627,030.80	31°02'00"	32.00'	8.88'	Horizontal
P.I. 5	308.67'	210.39'	2,160,341.81	627,115.30	19°00'00"	32.00'	5.35'	Horizontal
P.I. 6	329.12'	150.98'	2,160,293.68	627,155.69	49°14'04"	32.00'	14.66'	Combined
P.I. 7	308.60'	115.44'	2,160,253.14	627,149.27	5°34'01"	32.00'	1.56'	Vertical
Pt. X			2,160,800.64	627,021.92				

NOTE

For list of reference drawings, see Dwg. 557-D-566

6-1-64 D E.Q.S.	MINOR REVISIONS
2-16-59 D E.Q.S.	REVISED HOLLOW-JET VALVE AREA
7-14-58 D E.Q.S.	REVISED DOWNSTREAM END OF RIVER OUTLETS
2-18-58 D E.Q.S.	ADDED CONTRACTION JOINT AND REVISED EXPANSION JOINT.

THIS DRAWING SUPERSEDES DWG. 557-D-96 IN PART

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZONA-UTAH

GLEN CANYON DAM
RIVER OUTLETS
PLAN

DRAWN E.A.L. SUBMITTED E.R. Schultz
TRACED R.T.S. RECOMMENDED O.L. Rice
CHECKED Ho APPROVED [signature] CHIEF DESIGNING ENGINEER

DENVER, COLORADO, DECEMBER 26, 1957 SHEET 1 OF 2 — 557-D-565

Figure 109.—Plan of river outlets.

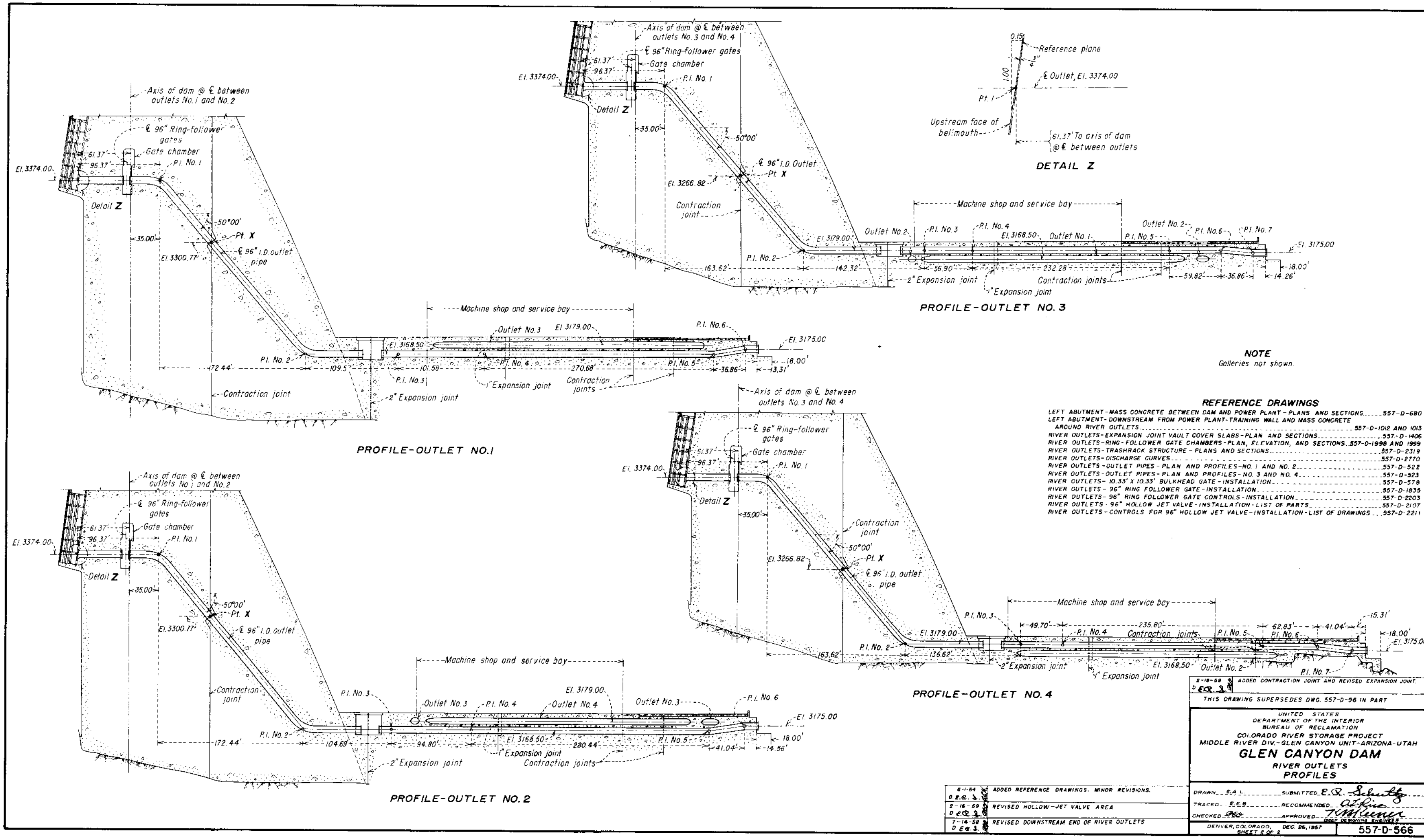


Figure 110.—Profile of river outlets.

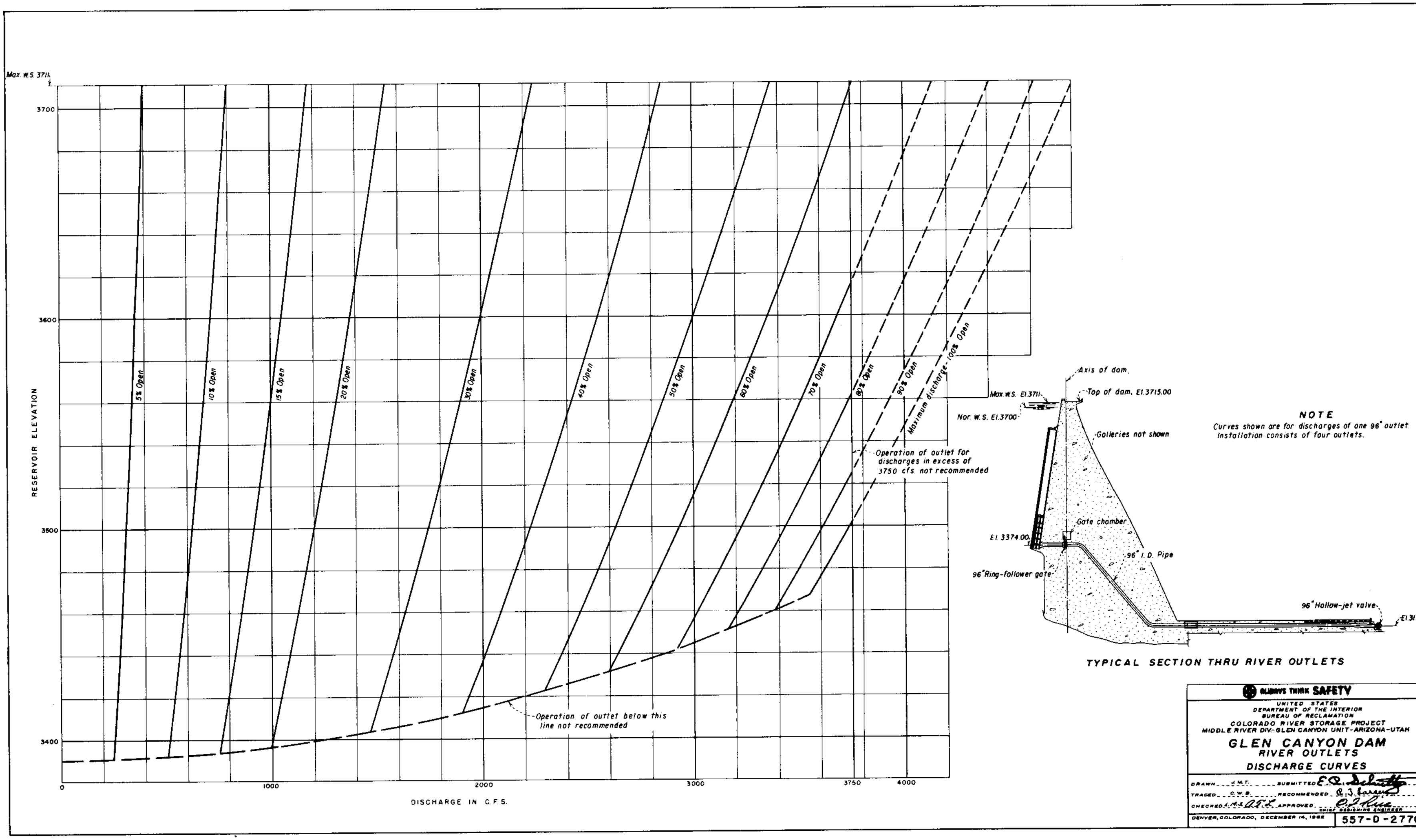


Figure 111.—Discharge curves of river outlets.

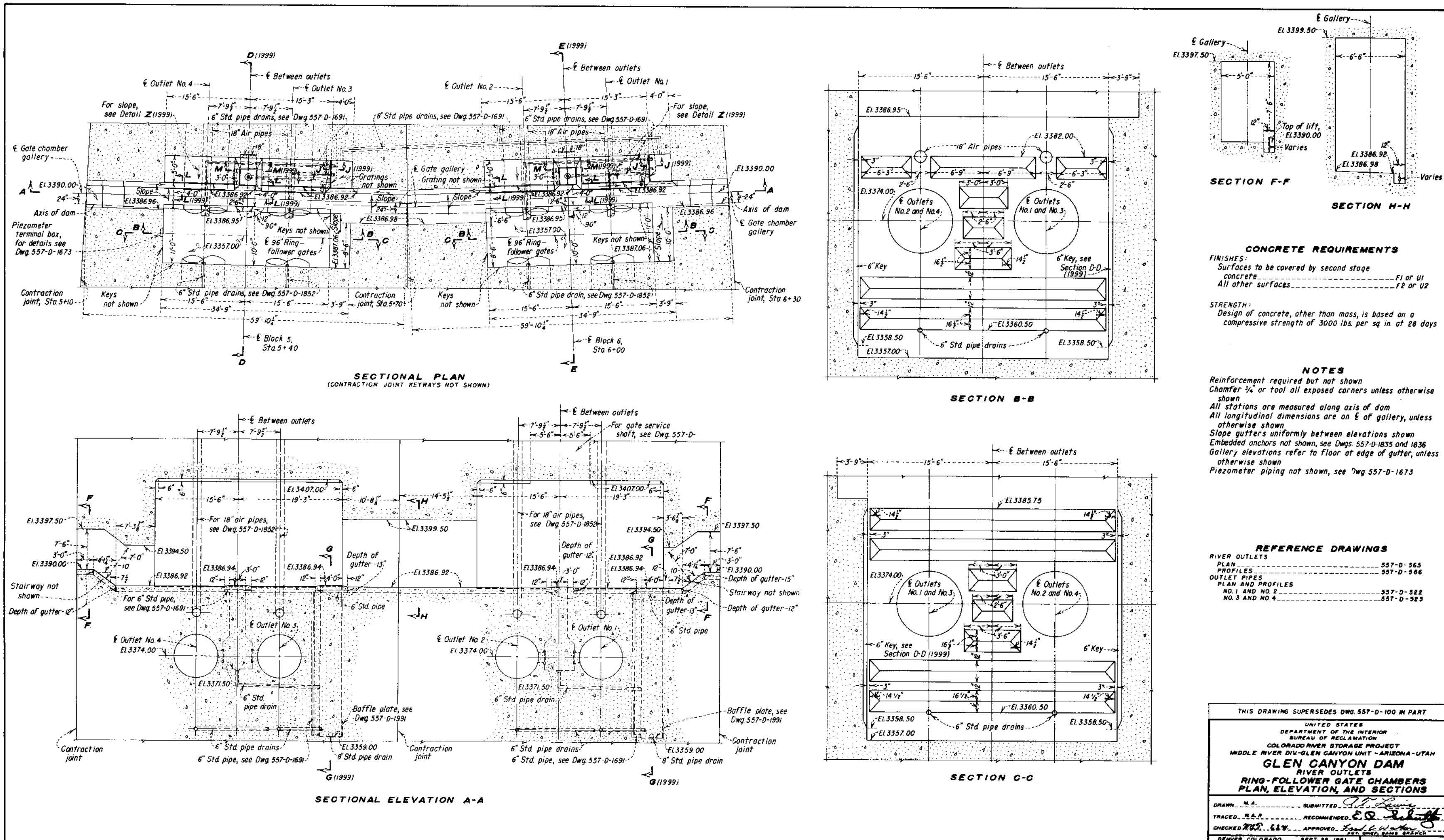


Figure 112.—River outlets, ring-follower gate chambers—Plan, elevation, and sections. (Sheet 1 of 2.)

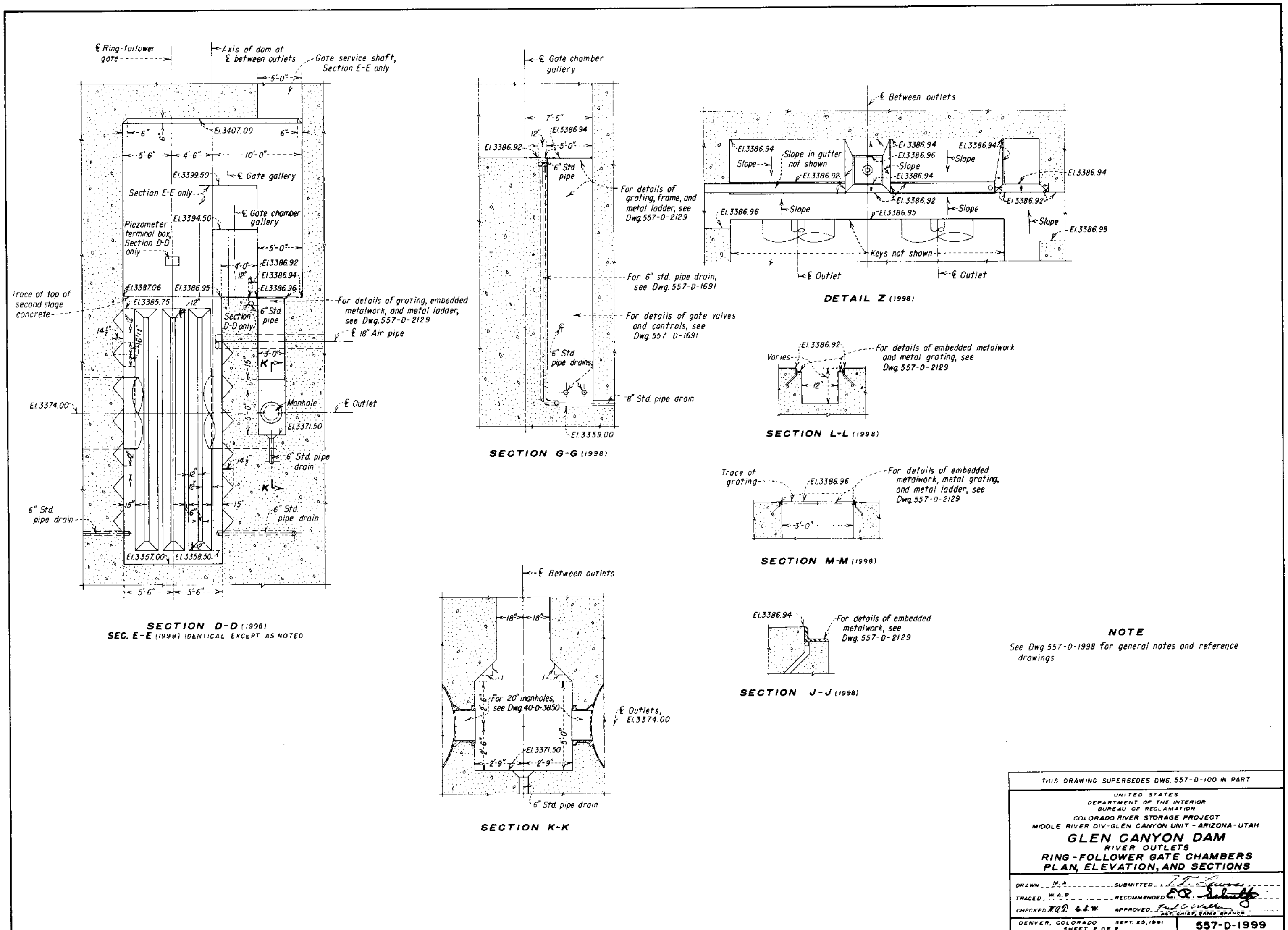


Figure 112.—River outlets, ring-follower gate chambers—Plan, elevation, and sections. (Sheet 2 of 2.)

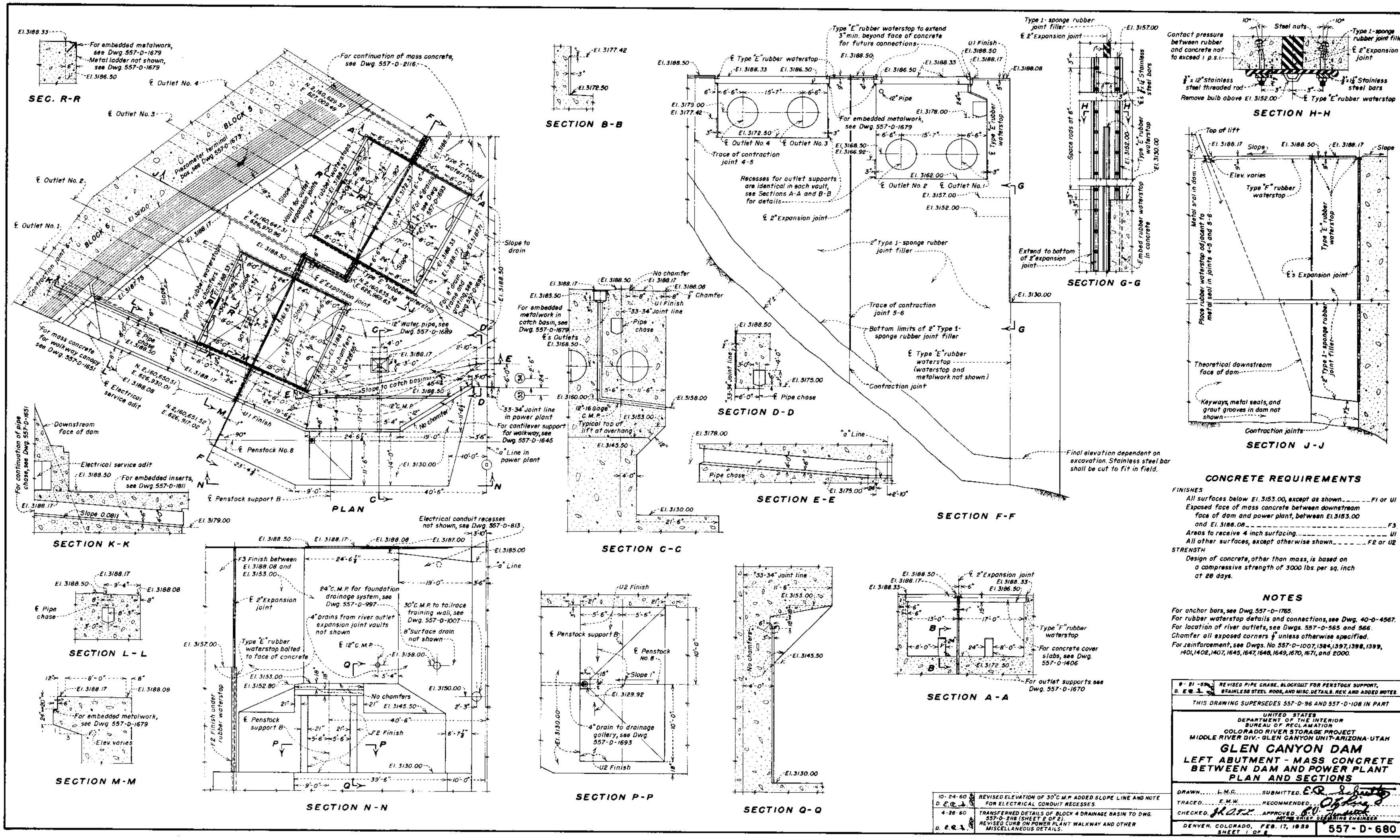


Figure 113.—Left abutment, mass concrete between dam and powerplant—Plan and sections.

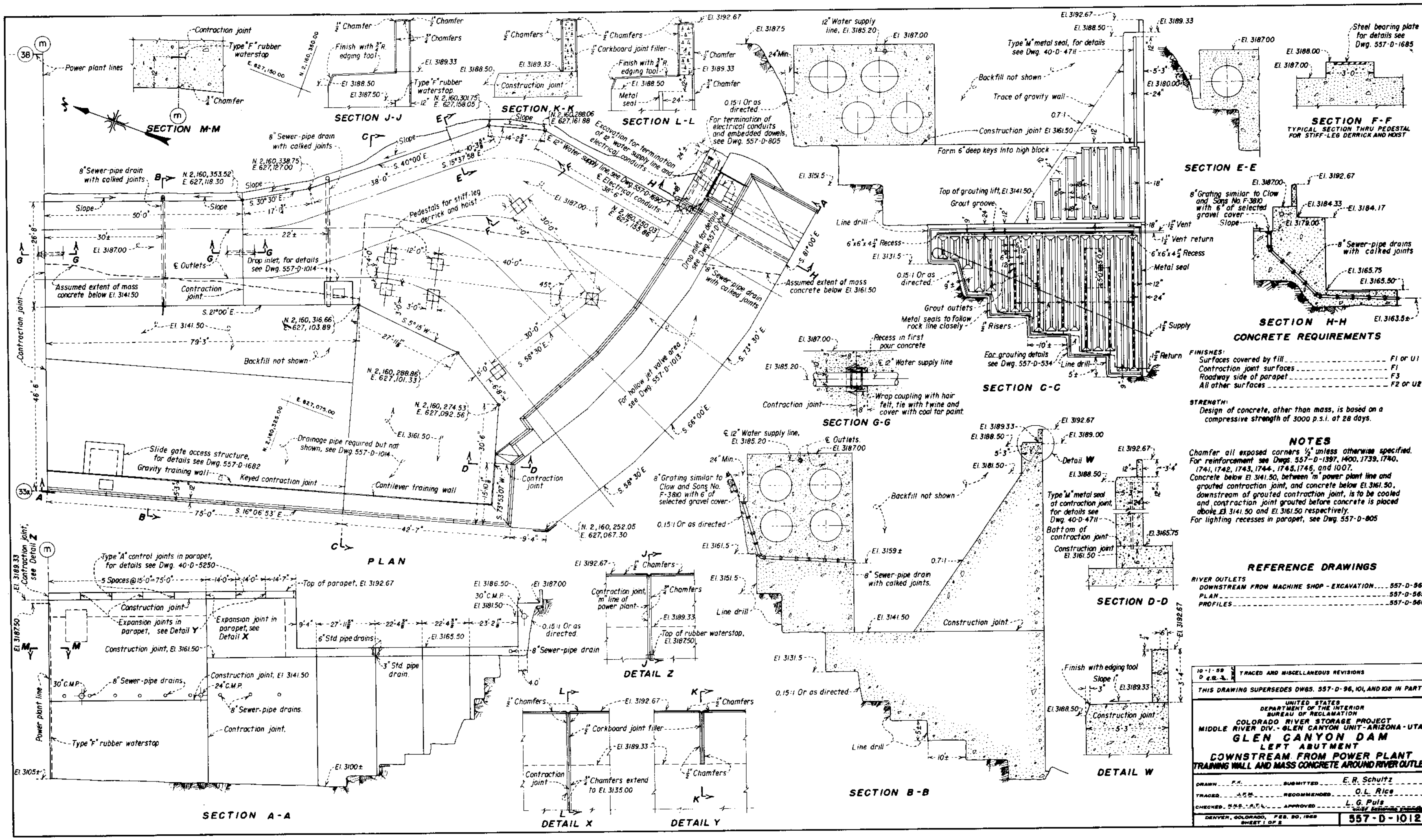


Figure 114.—Left abutment downstream from powerplant—Training wall and mass concrete around river outlets. (Sheet 1 of 2.)

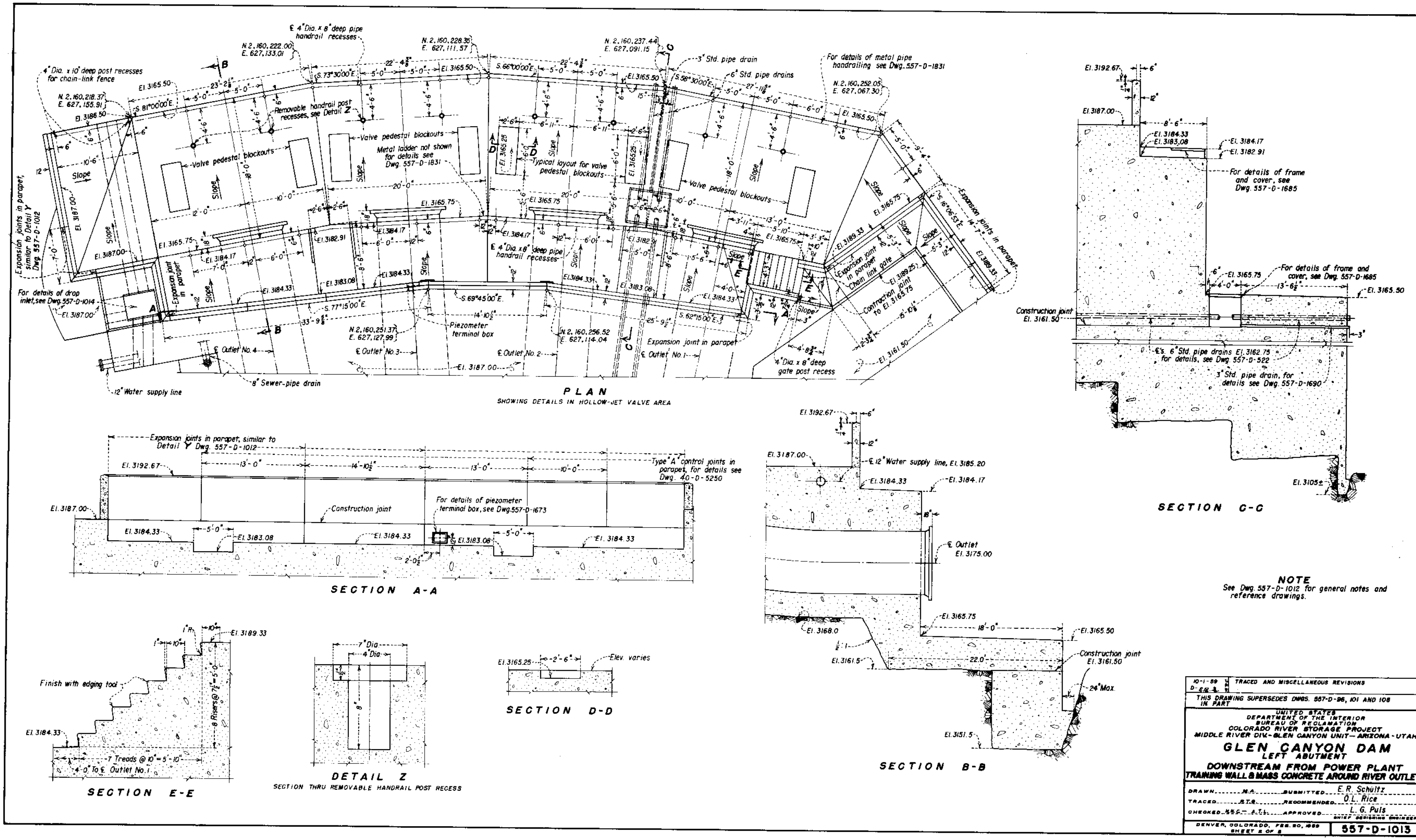


Figure 114.—Left abutment downstream from powerplant—Training wall and mass concrete around river outlets. (Sheet 2 of 2.)

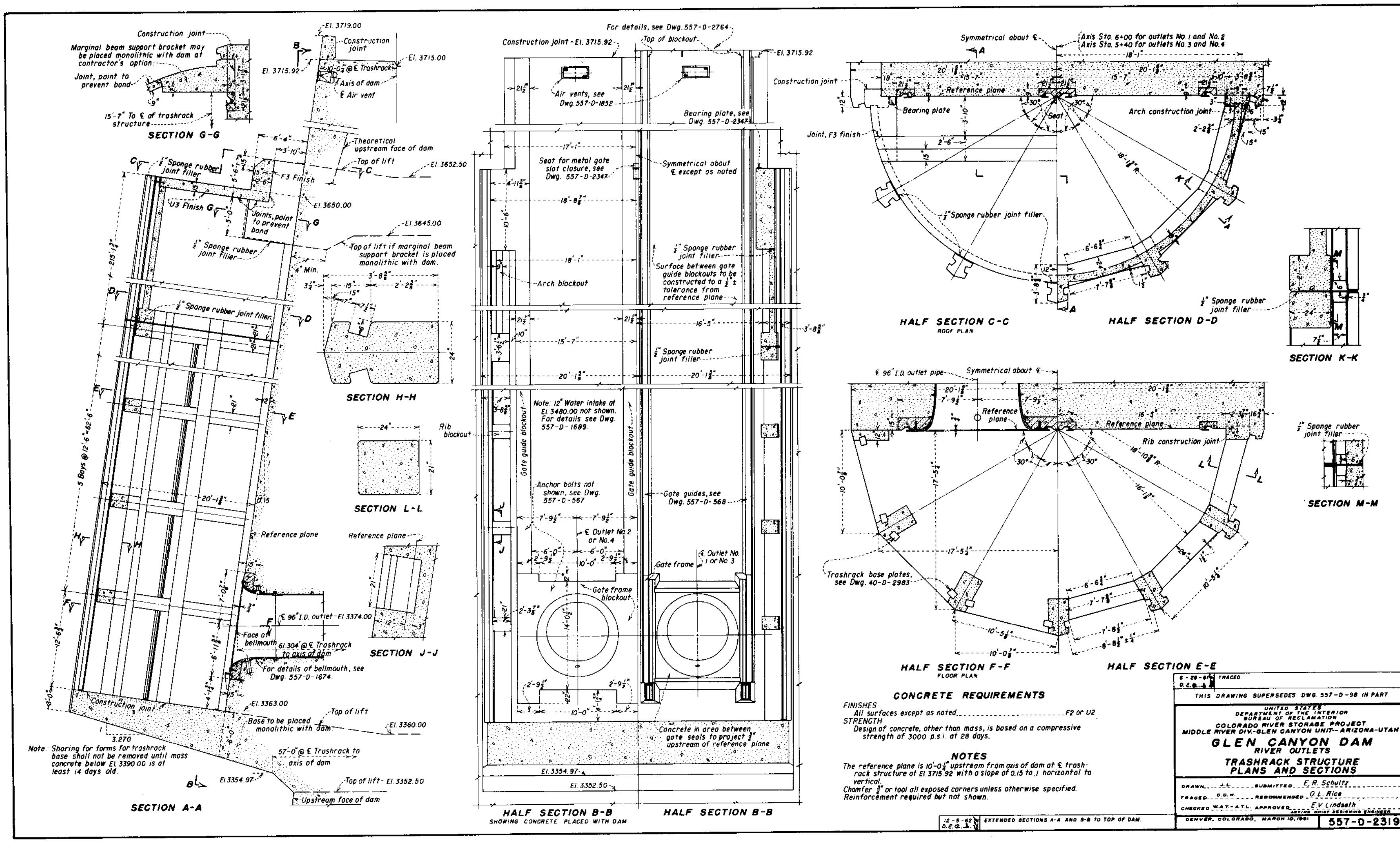


Figure 115.—River outlets trashrack structure—Plans and sections.

downstream edge of the ring-follower gate blockout to a point 40 feet upstream from the upstream edge of the expansion joint vault, the outlets were reinforced for tensile forces due to dam stresses and temperature effects.

(3) From a point 40 feet upstream from the upstream face of the expansion joint vault to the upstream face of the expansion joint vault, the outlets were reinforced for tensile stresses due to dam stresses, internal bursting pressures, and temperature effects. The outer face of the surrounding mass downstream from the downstream face of the dam was also reinforced to further control surface cracking.

(4) From the downstream face of the expansion joint vaults to the downstream end of the machine shop (m-line), the outlets were reinforced for tensile forces due to internal bursting pressure and for unbalanced forces in the pipe bends.

The outer face of the surrounding mass concrete was also reinforced in order to control surface cracking.

(5) From the downstream end of the machine shop to the hollow-jet valves, the outlets were reinforced for tensile forces due to internal bursting, truck and trailer loads, and unbalanced forces in the pipe bends. The outer face of the surrounding mass concrete was also reinforced in order to control surface cracking.

The second-stage concrete around the ring-follower gates was reinforced for tensile forces due to internal bursting pressure within the gate frames and bonnets.

50. 10.33- BY 10.33-FOOT BULKHEAD GATE. (a) *Description.*--The bulkhead gate is used for emergency closure of the four river outlets as shown on figure 116. When not in use, the gate is stored on the erection and storage platform at the top of the dam. The gate was manufactured by Johnson Machine Works, Chariton, Iowa, under invitation No. DS-5493. The frames were manufactured by Steward Machine Co., Birmingham, Ala., and the anchor bolts by Fulton Shipyard, Antioch, Calif., under invitation No. DS-5370.

(b) *Design.*--The design head is 343 feet with maximum water surface at elevation 3711.00. A lifting frame is connected to the gate for the raising and lowering operations. The gate and lifting frame slide in the guides embedded in the upstream face of the dam. The lifting frame is attached to the gantry crane on the top of the dam. The maximum travel of the gate in the guides is approximately 355 feet.

The gate consists of a skinplate supported on horizontal wide-flange beams which are connected to vertical end beams along the sides of the gate. The hydraulic force on the skinplate is transmitted to the horizontal beams and the reactions are carried through the vertical end beams to the seats embedded in the upstream face of the dam. The gate is operated only under balanced pressure.

Double-stem rubber seals are mounted on the downstream side of the skinplate and contact the embedded seal seats when the gate is in the lowered position. The water pressure on the upstream side of the seals provides effective contact with the seats preventing leakage into the river outlet conduit. Lateral movement of the gate is controlled by means of guide shoes mounted at the four corners of the gate. The shoes engage the guides along the upstream face of the dam.

The estimated movable weight of the gate is 38,000 pounds. The estimated weight of the gate frames and anchorage for the four river outlets is 153,500 pounds.

The gate was designed for a head of 343 feet. The design was based on the following allowable unit stresses in pounds per square inch:

Combined longitudinal and transverse stresses in skinplate	20,000
Tension in extreme fibers of members subjected to bending	15,000
Compression in extreme fibers of members subjected to bending	14,000
Shear on gross area webs	9,500
Shear on rivets	11,500
Shear on ribbed bolts	15,000
Bearing on ribbed bolts and rivets	23,000

51. 96-INCH RING-FOLLOWER GATES AND CONTROLS. (a) *Description.*—Four ring-follower gates and two sets of controls are provided for normal closure of the river outlets when the hollow-jet valves are not in use and for emergency closure under full flow if the hollow-jet valves should become inoperable. Ring-follower gates were selected because of the absence of any obstruction in the fluidway of the high-velocity flow. The ring-follower gates (fig. 117) were manufactured by Goslin-Birmingham Manufacturing Co., Inc., under invitation No. DS-5269.

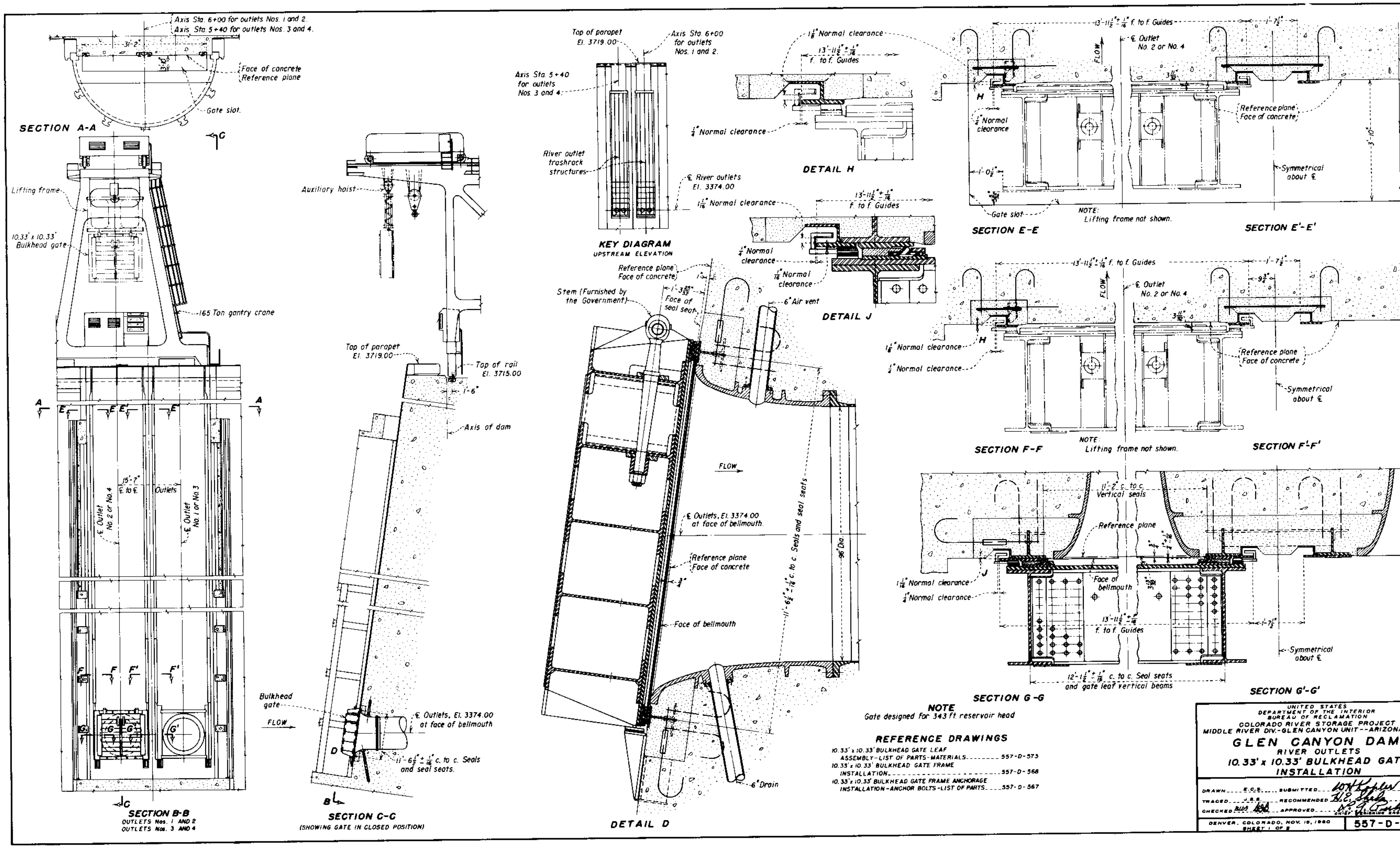


Figure 116.—River outlets—10.33- by 10.33-foot bulkhead gate installation.

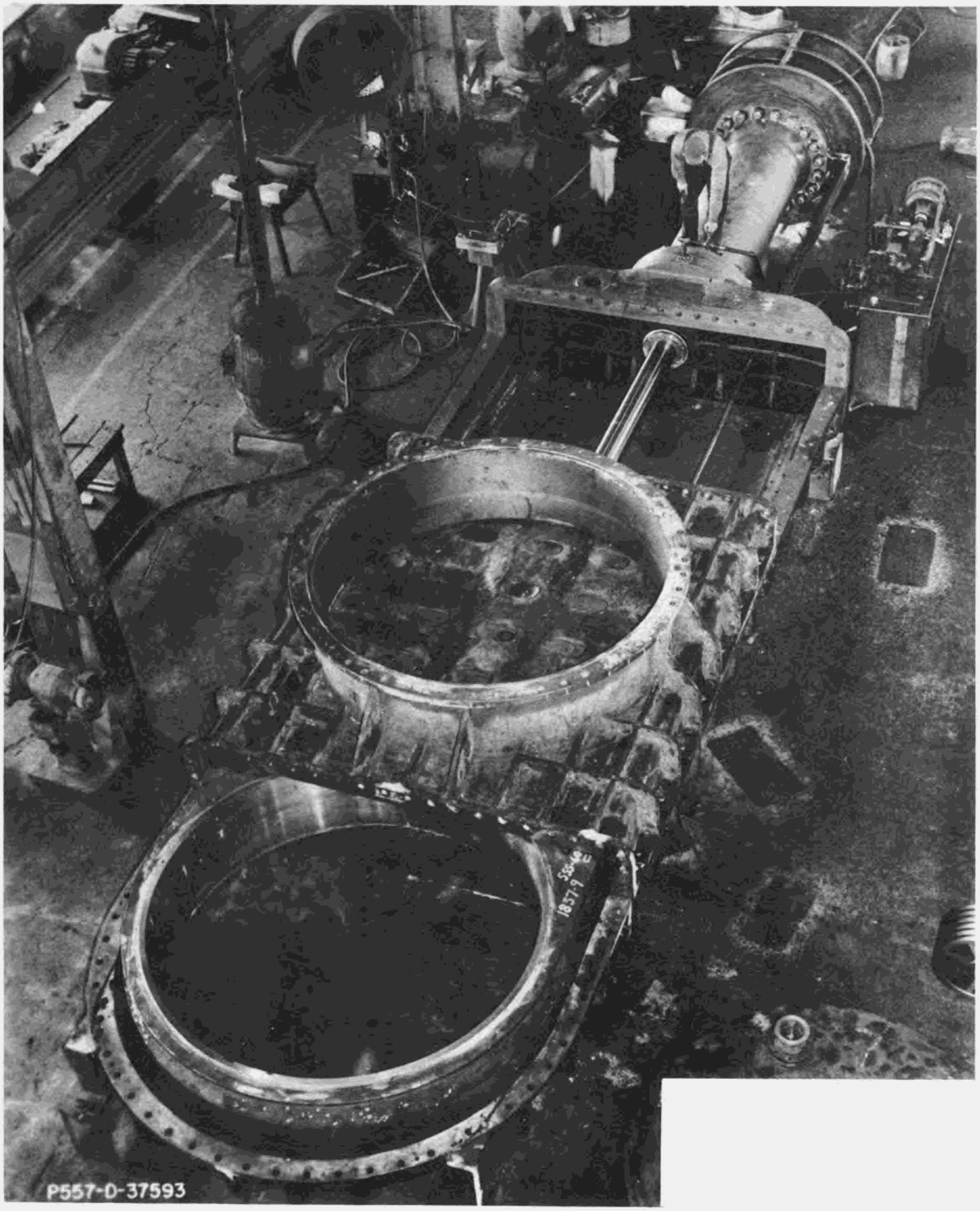


Figure 117.—96-inch ring-follower gate. The gate is shown in a horizontal position in the shop with the upper and lower upstream bonnets removed and is in the process of being tested.

The controls were manufactured by Kendo, Inc., under invitation No. DS-5498.

The ring-follower gates and controls are located upstream from the hollow-jet valves, near the axis of the dam, in the ring-follower gate chambers as shown on figure 118. A set of controls is located adjacent to each pair of ring-follower gates as shown on figure 119.

(1) *Ring-follower gate.*—The ring-follower gate is of cast and welded steel construction and consists basically of a body, a leaf, and a hydraulic hoist which is an integral part of the gate. The gate is of the slide type with a leaf made to include a follower ring having a circular opening equal to the diameter of the pipe to provide an unobstructed water passage when the leaf is in the open position as shown on figure 120. The hoist has a 27-inch-diameter cylinder and a travel of 8 feet 8 inches. A direct-reading position indicator is located adjacent to the hoist cylinder. The estimated weight of each ring-follower gate is 140,000 pounds.

(2) *Controls.*—Each control cabinet is of steel construction and encloses the control system for two ring-follower gates as shown on figure 121. A 110-gallon oil supply tank is located on top of each control cabinet to keep the minimum oil level above the gate cylinders. An oil-level gage is located on each tank. The estimated weight of one set of controls is 10,000 pounds.

(b) *Design.*—The ring-follower gate was designed to operate under a maximum reservoir head of 337 feet and a hoist oil pressure of 2,000 pounds per square inch. The hoist capacity was based on the weight of the leaf plus the sliding friction of the gate seals, using a friction coefficient of 0.6. Nickel-copper alloy seats were used on the gate body and bronze seats on the leaf to provide a low-friction and rust-resisting sliding surface. The maximum waterload on the gate leaf produces a bearing pressure between the sliding surfaces of the leaf and body seats of 715 pounds per square inch. The piston stem was made of nickel-copper alloy to prevent rusting. A hand-operated mechanical latching device was provided on the upper cylinder to hold the gate in the open position.

The equipment in each control cabinet consists of two oil pumps, each having a capacity of approximately 10-1/2 gallons per minute when pumping oil at 2,000 pounds per square inch; two 15-horsepower, 440-volt, 3-phase, 60-cycle electric motors; connecting piping, and hydraulic and electric controls. Remote controls were not provided.

(c) *Design Stresses.*—

(1) *Tension.*—The allowable design stresses in tension for the following materials were based on the yield point or the ultimate strength of the material. The smaller of the tabulated values was used in each instance.

Material	Type	Percent of yield point	Percent of ultimate tensile strength
Steel	Rolled or forged	40	25
Bolt steel	Rolled or forged	25	16.5
Cast steel	Castings	33	20
Brass or bronze	Rolled or cast	33	16.5

(2) *Compression.*—The allowable design stresses in compression used for the materials listed above were the same as for tension.

(3) *Shear.*—Allowable design stresses in shear were not more than 0.6 the allowable design stresses in tension.

(4) *Structural steel.*—Allowable design stresses for structural steel in tension or compression was not more than 20,000 pounds per square inch. In general, structural steel stresses were based on the American Institute of Steel Construction "Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings," 1961.

(5) *Hoist cylinders.*—Allowable design stresses for hoist cylinders were based on the recommendations of the ASME Boiler and Pressure Vessel Code—Unfired Pressure Vessels—Section VIII.

52. STEEL OUTLET PIPES. (a) *Description.*—Four outlet pipes (figs. 122 and 123) are provided to pass water through the dam when the reservoir water surface is below penstock intakes, and to supplement turbine and/or spillway discharge when required for flood control. The outlet pipes were installed under specifications No. DC-4825. Fabrication was performed under invitation No. DS-5052.

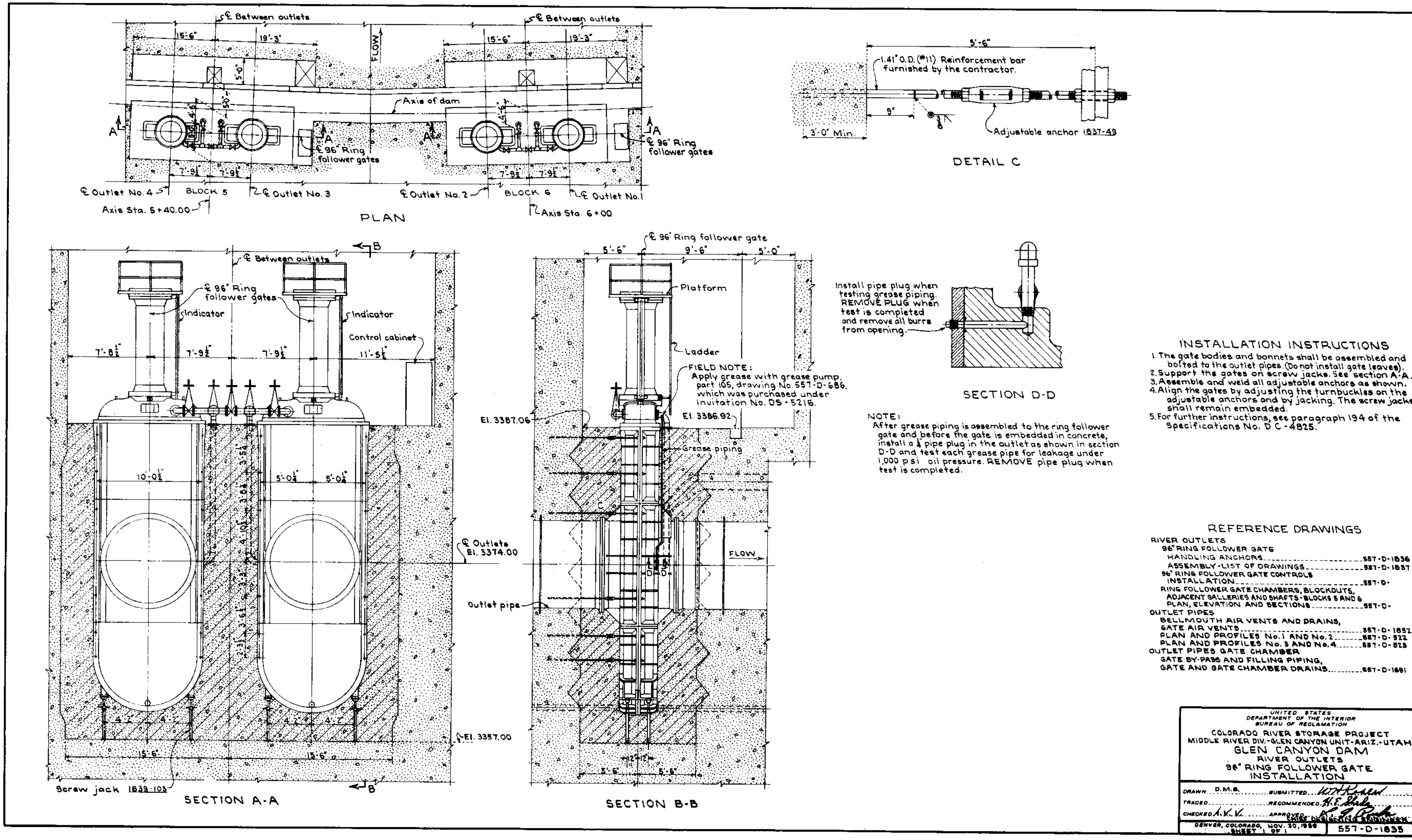


Figure 118.—River outlets, 96-inch ring-follower gate installation.

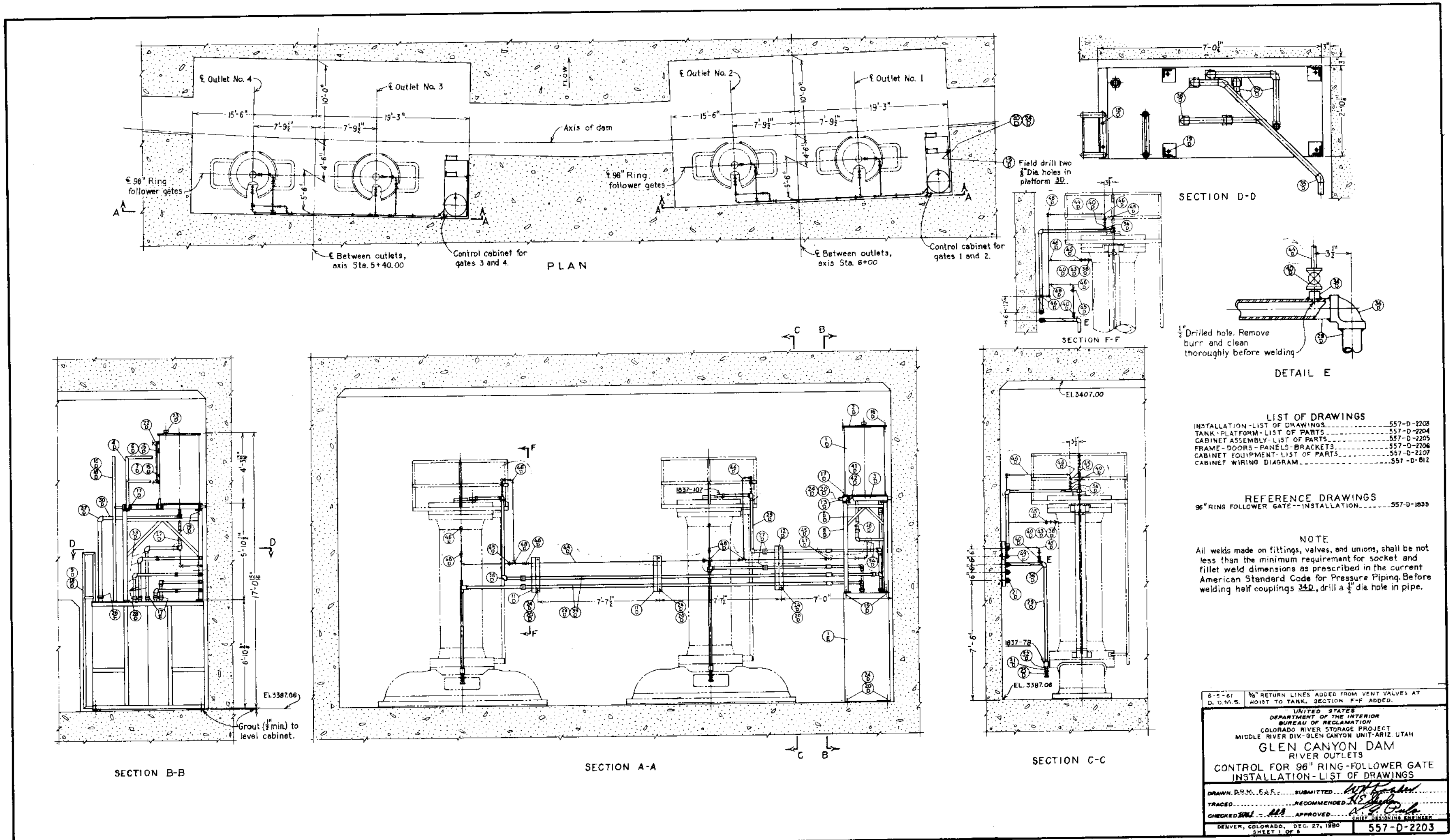


Figure 119.—River outlets, 96-inch ring-follower gate control—Installation.

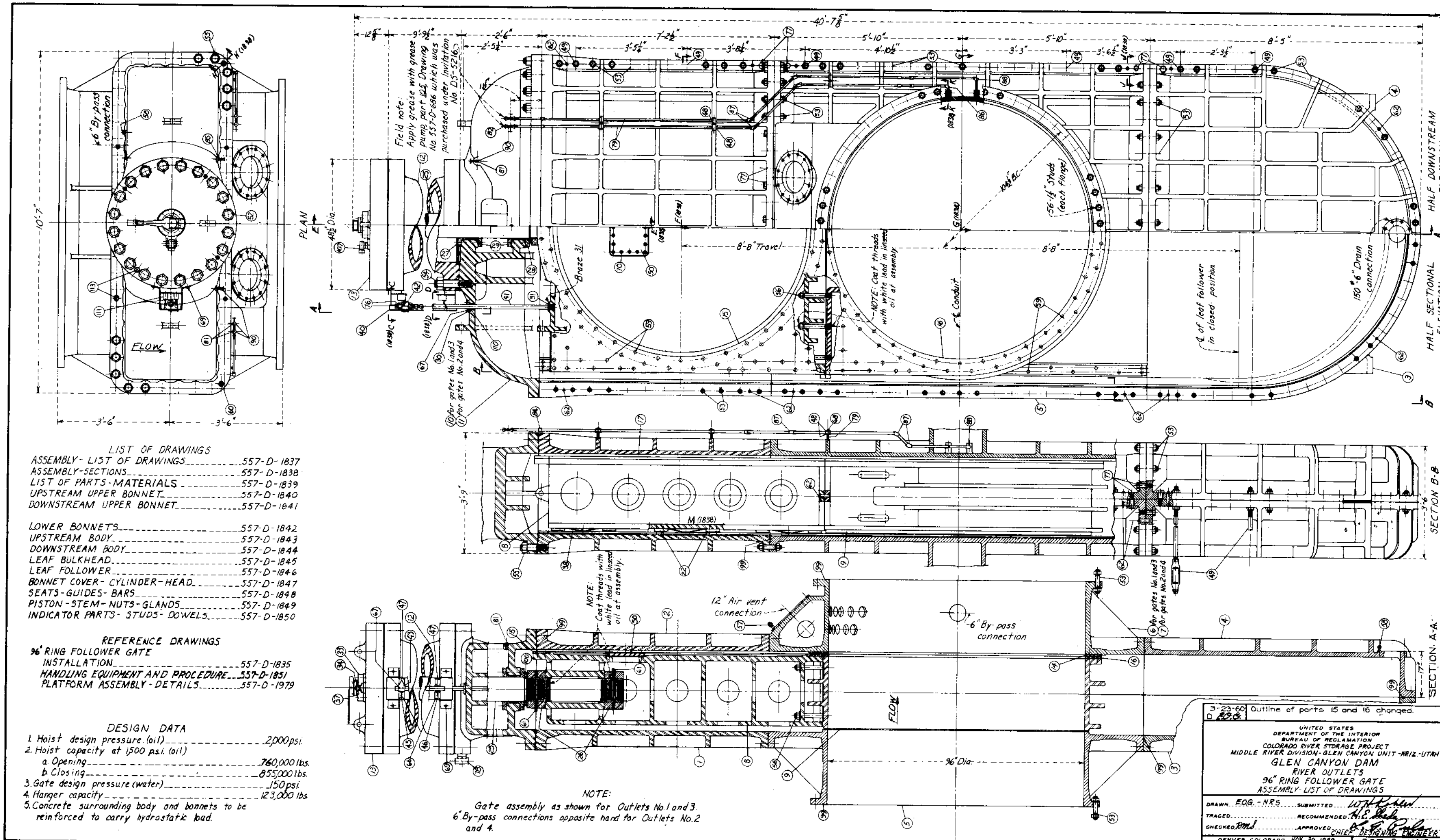
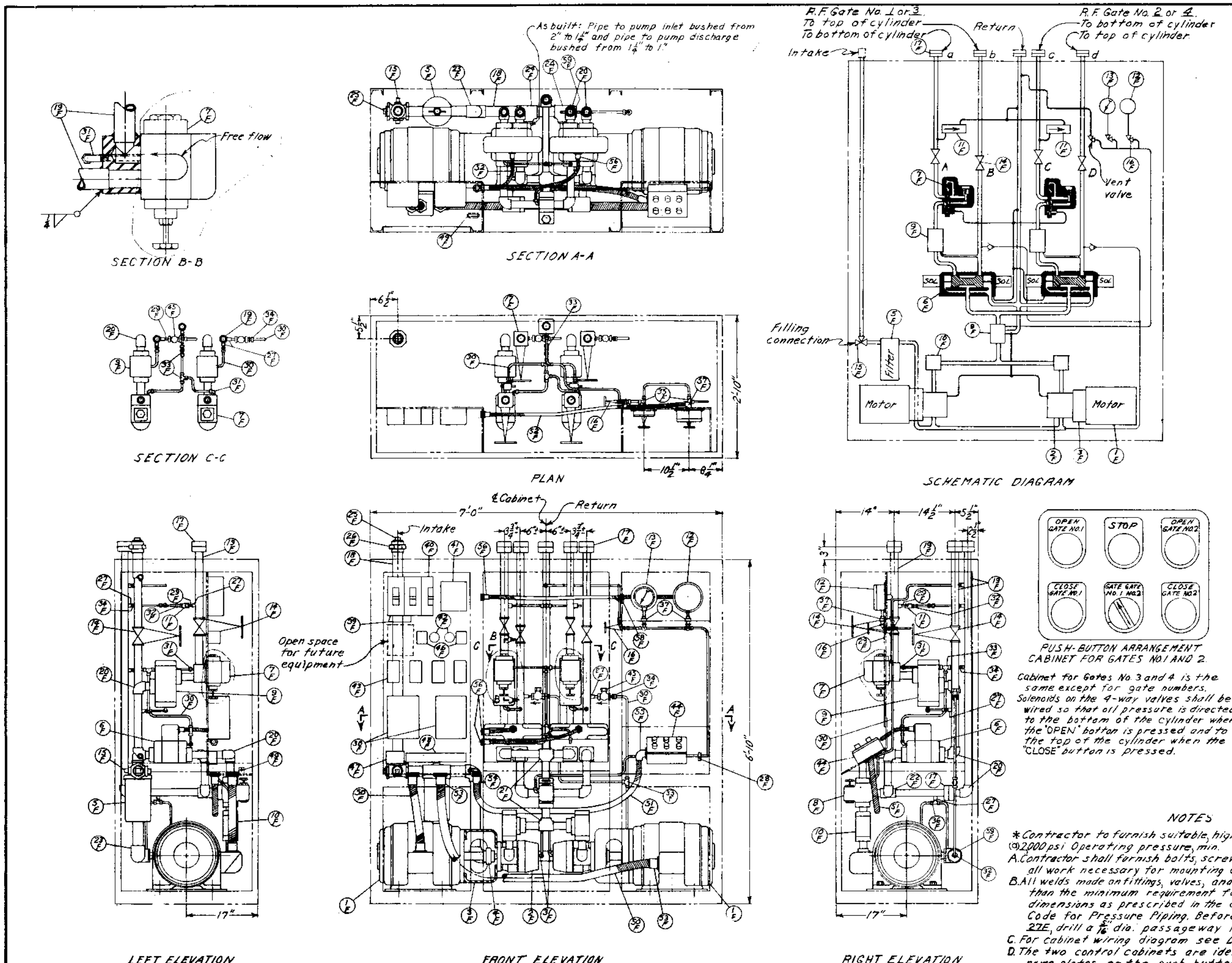


Figure 120.—River outlets, 96-inch ring-follower gate assembly.



LIST OF PARTS- EQUIPMENT FOR TWO CABINETS

PART NO.	NO. REQ'D.	DESCRIPTION	MATERIAL *
1F	4	Electric motor, 15 H.P., 1200 R.P.M.	See specifications
2F	4	Oil pump, $10\frac{1}{2}$ G.P.M.	
3F	4	Motor-pump adapter	
4F	4	Flexible coupling	
5F	2	2"-Oil filter	
6F	4	$1\frac{1}{4}$"-Four-way valve	
7F	4	$1\frac{1}{4}$" By-pass valve	
8F	2	$1\frac{1}{4}$" Relief valve	
9F	4	$1\frac{1}{4}$" Pilot operated check valve	
10F	4	$1\frac{1}{4}$" Angle check valve	
11F	4	Thermal relief valve	
12F	2	Pressure switch	
13F	2	Pressure gage	
14F	8	$1\frac{1}{4}$"-600# Wedge gate valve, socket welding (a)	Cast steel, SEE SPECS.
15F	2	2"-Three-way cock valve, screwed	Brass
16F	6	$\frac{1}{4}$"-3000# Bar stock angle valve, female (a)	Steel
17F	12	$1\frac{1}{4}$"-O Ring flange union, COMPLETE WITH O RING AND CAP SCREWS, SOCK. WELDING (a)	Forged steel, SEE SPECS.
18F	21 Ft	2"-Black pipe-schedule 40	Steel
19F	70 Ft	$1\frac{1}{4}$" Black pipe-schedule 80	Steel
20F	30	$1\frac{1}{4}$"-3000#-90° Elbow, socket welding	Forged steel
21F	4	$1\frac{1}{4}$"-3000# Tee, socket welding	Forged steel
22F	2	$1\frac{1}{4}$"-3,000# Cross, socket welding	Forged steel
23F	4	2"-150#-90° Elbow, screwed	Malleable iron
24F	4	2"-150# Tee, screwed	Malleable iron
25F	4	2" Solid square head pipe plug	Malleable iron
26F	2	2"-150# Union, female, screwed, GROUND JOINT	Malleable iron
27F	20	$\frac{1}{4}$"-3000# Half coupling, screwed	Steel
28F	2	Tube clip for $\frac{3}{8}$" hydraulic tubing	Brass
29F	12	$\frac{1}{4}$"x$1\frac{1}{2}$" Long-black pipe nipple, SCHEDULE 80, THREADED.	Steel
30F	80 Ft	$\frac{3}{8}$" O.D. x .049" Wall hydraulic tubing	Steel J.I.C Standard
31F	18	Male elbow, 90°, $\frac{1}{4}$" MPT to $\frac{3}{8}$" tube (a)	
32F	6	Male connector, $\frac{3}{8}$" MPT to $\frac{3}{8}$" tube (a)	
33F	12	$\frac{3}{8}$" Union tee, $\frac{3}{8}$" tube (a)	
34F	20	Male connector, $\frac{1}{4}$" MPT to $\frac{3}{8}$" tube (a)	
35F	4	$\frac{3}{8}$" Tube tee with $\frac{1}{4}$" M.P.T. on run (a)	
36F	2	$\frac{3}{8}$" Tube tee with $\frac{1}{4}$" M.P.T. outlet (a)	
37F	4	$\frac{1}{4}$"-2000# Male and female union, SCREWED, GROUND JOINT	Forged steel
38F	4	Magnetic motor starter	See specifications
39F	4	Circuit breaker, 50 amp, 3 pole	
40F	2	Circuit breaker, 15 amp, 2 pole	
41F	2	Control transformer, 1000 VA	
42F	4	Surface rotary switch, 1 pole	
43F	8	Control relay	
44F	2	Push-button station	
45F	4	$\frac{1}{4}$"-600# Lift check valve, screwed, no spring (a)	Steel
46F	4	Fuse cutout base, 2 pole	See specifications
47F	2	Terminal block, 3 circuit, 100 amp.	
48F	2	Terminal block, 24 circuit, 25 amp.	
49F	2	Cartridge heater	
50F	17 Ft	$1\frac{1}{2}$" Flexible metal conduit	Steel
51F	12 Ft	1" Flexible metal conduit	Steel
52F	19 Ft	$\frac{1}{2}$" Flexible metal conduit	Steel
53F	8	$1\frac{1}{2}$" Flexible metal conduit connector (straight)	Steel
54F	2	1" Flexible metal conduit connector (straight)	Steel
55F	2	1" Flexible metal conduit connector (90°)	Steel
56F	10	$\frac{1}{2}$" Flexible metal conduit connector (straight)	Steel
57F	2	$\frac{1}{2}$" Flexible metal conduit connector (90°)	Steel
58F	2	$\frac{1}{2}$" Conduit clamp	Steel
59F	2	2"x$\frac{3}{8}$" Hex. bushing, screwed	Steel
60F			

PUSH-BUTTON ARRANGEMENT CABINET FOR GATES NO 1 AND 2.

Cabinet for Gates No. 3 and 4 is the same except for gate numbers. Solenoids on the 4-way valves shall be wired so that oil pressure is directed to the bottom of the cylinder when the "OPEN" button is pressed and to the top of the cylinder when the "CLOSE" button is pressed.

NOTES

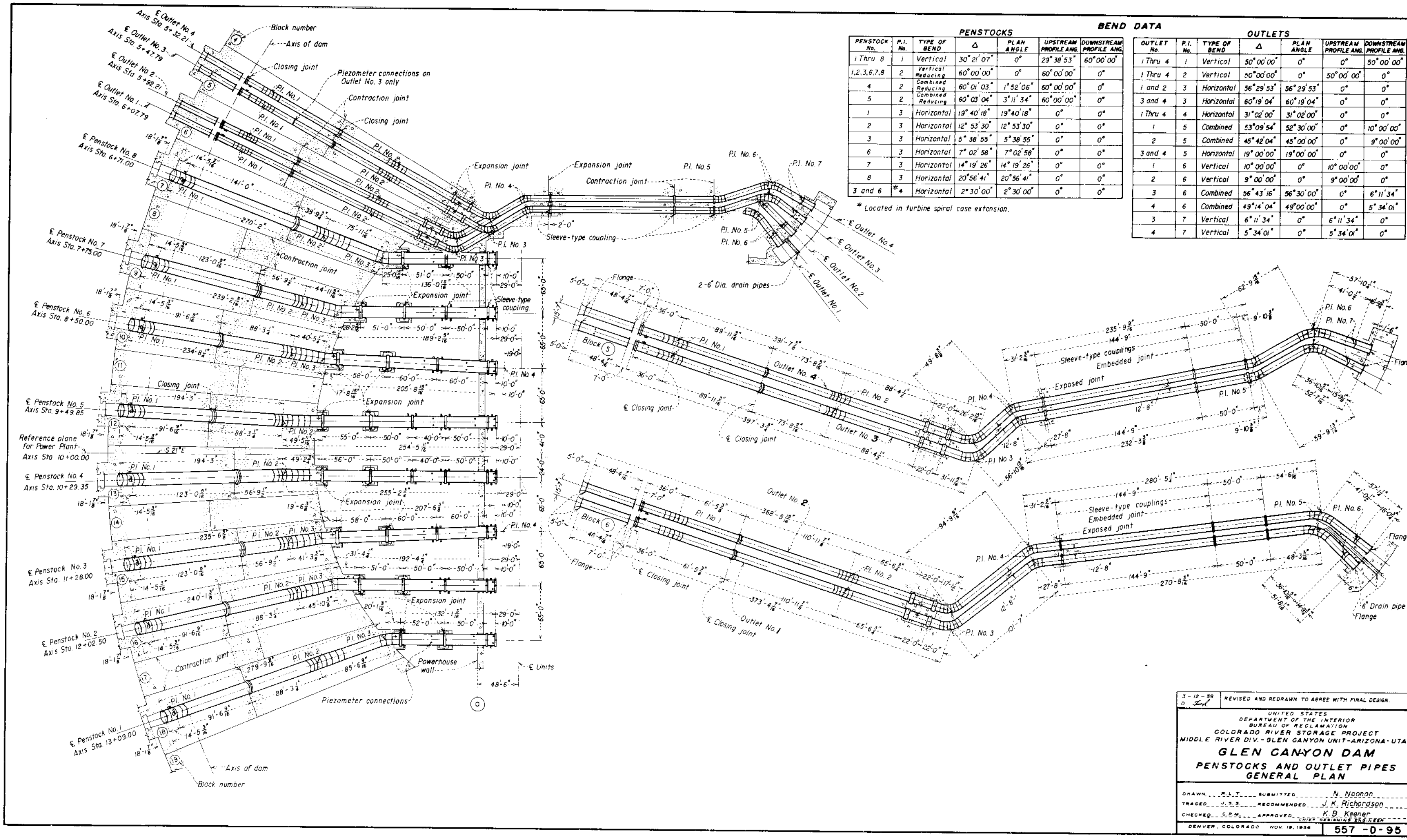
*Contractor to furnish suitable, high-class, commercial materials.
(a) 2000 psi Operating pressure, min.
A. Contractor shall furnish bolts, screws, etc. required and perform all work necessary for mounting equipment in cabinet.
B. All welds made on fittings, valves, and unions, shall be not less than the minimum requirement for socket and fillet weld dimensions as prescribed in the current American Standard Code for Pressure Piping. Before welding half couplings, 27F, drill a $\frac{1}{16}$" dia. passageway in pipe.
C. For cabinet wiring diagram see Drawing 557-D-812
D. The two control cabinets are identical except for the name plates on the push-button station.

10-9-61 A: AS BUILT: BY-PASS VALVE 7F MOUNTED WITH ADJUSTMENT HANDLE DOWNWARD. PUMP INLET AND OUTLET BUSHED AS NOTED.
6-5-61 B: TUBING LINES, WITH $\frac{1}{4}$" CHECK VALVE 45F IN EACH LINE, ADDED BETWEEN INTAKE LINE AND EACH TOP OF CYLINDER LINE.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIVISION-GLEN CANYON UNIT-ARIZ-UTAH
GLEN CANYON DAM
RIVER OUTLETS
CONTROL FOR 96" RING-FOLLOWER GATE
CABINET EQUIPMENT-LIST OF PARTS

DRAWN R.S.O. SUBMITTED [signature]
TRACED RECOMMENDED [signature]
CHECKED [initials] APPROVED [signature] CHIEF MECHANICAL BRANCH
DENVER, COLORADO, DEC. 23, 1960 SHEET 5 OF 6
557-D-2207

Figure 121.—River outlets, 96-inch ring-follower gate—Cabinet equipment.



BEND DATA

PENSTOCKS

PENSTOCK No.	P.I. No.	TYPE OF BEND	Δ	PLAN ANGLE	UPSTREAM PROFILE ANG.	DOWNSTREAM PROFILE ANG.
1 Thru 8	1	Vertical	30°21'07"	0°	29°38'53"	60°00'00"
1,2,3,6,7,8	2	Vertical Reducing	60°00'00"	0°	60°00'00"	0°
4	2	Combined Reducing	60°01'03"	1°52'06"	60°00'00"	0°
5	2	Combined Reducing	60°03'04"	3°11'34"	60°00'00"	0°
1	3	Horizontal	19°40'18"	19°40'18"	0°	0°
2	3	Horizontal	12°53'30"	12°53'30"	0°	0°
3	3	Horizontal	5°38'55"	5°38'55"	0°	0°
6	3	Horizontal	7°02'58"	7°02'58"	0°	0°
7	3	Horizontal	14°19'26"	14°19'26"	0°	0°
8	3	Horizontal	20°56'41"	20°56'41"	0°	0°
3 and 6	*4	Horizontal	2°30'00"	2°30'00"	0°	0°

* Located in turbine spiral case extension.

OUTLETS

OUTLET No.	P.I. No.	TYPE OF BEND	Δ	PLAN ANGLE	UPSTREAM PROFILE ANG.	DOWNSTREAM PROFILE ANG.
1 Thru 4	1	Vertical	50°00'00"	0°	0°	50°00'00"
1 Thru 4	2	Vertical	50°00'00"	0°	50°00'00"	0°
1 and 2	3	Horizontal	56°29'53"	56°29'53"	0°	0°
3 and 4	3	Horizontal	60°19'04"	60°19'04"	0°	0°
1 Thru 4	4	Horizontal	31°02'00"	31°02'00"	0°	0°
1	5	Combined	53°09'54"	52°30'00"	0°	10°00'00"
2	5	Combined	45°42'04"	45°00'00"	0°	9°00'00"
3 and 4	5	Horizontal	19°00'00"	19°00'00"	0°	0°
1	6	Vertical	10°00'00"	0°	10°00'00"	0°
2	6	Vertical	9°00'00"	0°	9°00'00"	0°
3	6	Combined	56°43'16"	56°30'00"	0°	6°11'34"
4	6	Combined	49°14'04"	49°00'00"	0°	5°34'01"
3	7	Vertical	6°11'34"	0°	6°11'34"	0°
4	7	Vertical	5°34'01"	0°	5°34'01"	0°

3-12-59 D — REVISED AND REDRAWN TO AGREE WITH FINAL DESIGN.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZONA-UTAH

GLEN CANYON DAM
PENSTOCKS AND OUTLET PIPES
GENERAL PLAN

DRAWN R.L.T. SUBMITTED N. Noonan
TRACED J.S.S. RECOMMENDED J. K. Richardson
CHECKED C.P.M. APPROVED K. B. Keener, CHIEF DESIGNING ENGINEER
DENVER, COLORADO NOV. 19, 1956 — 557-D-95

Figure 122.—Dam penstocks and outlet pipes—General plan.

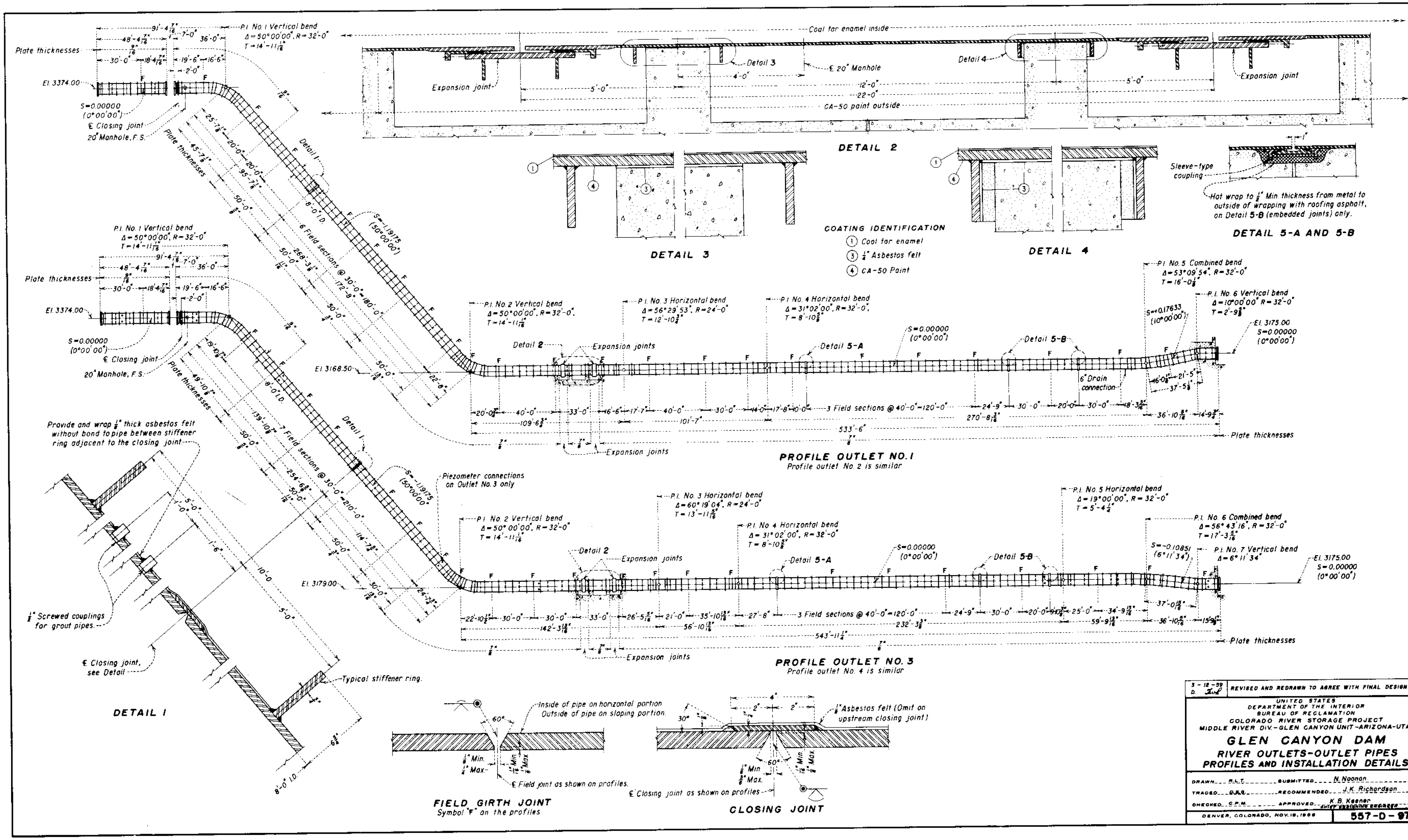


Figure 123.—Dam river outlet pipes—Profiles and installation details.

The outlet pipes have an inside diameter of 8 feet. They begin at the downstream ends of cast iron bellmouths in the upstream face of the dam at centerline elevation 3374.0 near the left abutment. Each pipe runs level for about 90 feet, then slopes downward and emerges from the dam and passes around the powerplant to the outlet structure. Outlet pipes No. 1 and 2 emerge from the dam at centerline elevation 3168.5, run level to the outlet structure and there lift to centerline elevation 3175.0. Pipes No. 3 and 4 emerge from the dam at centerline elevation 3179.0, run level to the outlet structure and there drop to centerline elevation 3175.0, in line with the other outlet pipes. A ring-follower gate is installed in the first level run of each pipe. The outlet pipes terminate at hollow-jet type discharge regulating valves. Each outlet pipe is approximately 930 feet long.

Minimum and maximum outlet pipe plate thicknesses are 9/16 and 7/8 inches. The outlet pipes are totally embedded in dam and encasement concrete except where they are exposed at expansion joints and at hollow-jet valves.

The outlet pipes have 6-inch gate bypass and filling piping systems at the ring-follower gates. These systems provide complete flexibility of outlet pipe filling and ring-follower gate bypass for each pair of outlet pipes. That is, pipes No. 1 and 2 may be filled from each other, and pipes No. 3 and 4 may be filled from each other; but there is no connection between pipes No. 1 and 2 and pipes No. 3 and 4.

A teetering section with double-end expansion joints was installed in each outlet pipe across the joint between the dam and the adjacent mass concrete. Sleeve-type coupling joints were located at each contraction joint in mass concrete encasements and discharge valve structure.

(b) *Design.*—The outlet pipes were designed for static head when the reservoir water surface is at maximum elevation 3711.0. For design considerations no water-hammer head was superimposed on static head because the outlet pipe gate and valve controls do not operate rapidly enough to cause significant water hammer.

The maximum designed discharge through each outlet pipe is 3,750 cubic feet per second. The corresponding average velocity in the 8-foot-diameter pipes is 74.60 feet per second. The pipe diameter was chosen for best balance between factors representing desired discharge, energy dissipation, and maximum allowable velocity short of destructive cavitation and vibration. The maximum designed velocity of 74.60 feet per second is about 10 feet per second faster than that used in previous outlet pipe designs.

The special double-end expansion joints previously mentioned were designed to accommodate calculated deflections of the dam amounting to 1-1/4 inches downstream, 1 inch vertically downward, and 1 inch laterally at the point where the pipes emerge from the dam. Sleeve-type couplings were designed by the subcontractor, R. H. Baker and Co.

Outlet pipe sections, including expansion joints and stiffener rings, were fabricated of steel plates conforming to ASTM Designation A 201, grade B, firebox quality. Middle rings and sealing glands of sleeve-type couplings were fabricated of steels conforming to ASTM Designations A 212, grade B, and A 7, respectively.

All permanent joints were welded, excepting flanged connections to ring-follower gates which were embedded in concrete. All girth and longitudinal welds in outlet pipe shells, expansion joint inner and outer sleeves, and sleeve-type coupling middle rings were fully radiographed in accordance with section VIII of the ASME Boiler and Pressure Vessel Code at that time current. Code basic design working unit stresses ASTM A 201 and 212 steels used were 15,000 and 17,500 pounds per square inch, respectively. Joint efficiency was 90 percent for both steels.

Completed sections of outlet pipes, including expansion joints and sleeve-type couplings, were hydrostatically tested at pressures computed from the formula:

$$P = \frac{43{,}000T}{D}$$

where:

P = test pressure in pounds per square inch,
T = minimum thickness, in inches, of plate course in section tested, and
D = inside diameter of pipe in inches.

A 20-inch inside-diameter manhole is located immediately downstream from the ring-follower gate of each outlet pipe. Another similar manhole is located in the teetering section at the joint between the dam and adjacent mass concrete.

Outlet pipe No. 3 only has seven piezometer orifice stations distributed along its entire length. Piezometer orifices at each station are manifolded, and single 3/4-inch pipes lead from each manifold to terminal boxes embedded in concrete faces in the ring-follower gate chamber, the downstream face of the dam, or the discharge valve structure. The piezometer installation is shown on figure 124.

(c) *Installation and Coating.*—The outlet pipe sections were installed under the prime contract. Interior surfaces were coated with coal-tar primer and coal-tar enamel. Exterior embedded surfaces were not coated. Exterior surfaces exposed in the teetering section vaults were coated with phenolic-resin aluminum paint.

53. 96-INCH HOLLOW-JET VALVES AND CONTROLS. (a) *Description.*—Four 96-inch hollow-jet valves and two sets of controls are provided at the discharge end of the river outlets to regulate the flow of water up to a maximum reservoir head of 536 feet. The hollow-jet valves were manufactured by Goslin-Birmingham Manufacturing Co., under invitation No. DS-5363. The controls were manufactured by the Rucker Co., under invitation No. DS-5503.

The hollow-jet valves and controls are located at the discharge end of the river outlets as shown on figures 125 and 126. A set of controls is located on a concrete platform above and between each pair of hollow-jet valves.

(1) *Hollow-jet valve.*—The hollow-jet valve is of cast and welded steel construction and consists basically of a circular body and movable concentric needle which forms an annular passage and seals in the entrance throat. The hollow-jet valve is hydraulically operated by a cylinder within the annular passage which is concentrically positioned by six radial splitters as shown on figure 126. The cylinder is 58 inches in diameter and the needle has a travel of 33-1/2 inches from fully open to fully closed position. As the valve is opened, water flows past the periphery of the needle in the shape of a cylindrical ring along the inside of the valve body. The splitters cut the ring into sectors and the water discharges from the valve in six separate jets. The estimated weight of each hollow-jet valve is 135,000 pounds.

(2) *Controls.*—Each control cabinet is of steel construction and encloses the control system for two hollow-jet valves. A cable-driven dial-type position indicator, for each hollow-jet valve, is located on the cabinet control panel to show the percentage of valve opening. Two 850-gallon oil tanks are located adjacent to the control cabinets. An oil level gage is located on the side of each oil tank. The estimated weight of each control system is 7,300 pounds.

(b) *Design.*—

(1) *Hollow-jet valve.*—The hollow-jet valve was designed to regulate the discharge from the 96-inch outlet pipes under any head up to 536 feet. The plunger capacity was based on the sliding friction of the plunger and packings, and the hydrostatic force from full reservoir head acting on the upstream face of the needle. V-type packings are provided to minimize leakage past the plunger. Stainless steel seats are provided on both the upstream body and the needle. Stainless steel cladding was used on the exterior surfaces of the plunger to prevent rusting.

(2) *Controls.*—The hydraulic controls were designed to operate at 850 pounds per square inch for closing and 500 pounds per square inch for opening. The equipment in each cabinet consists of two oil pumps, each having a capacity of approximately 24 gallons per minute when pumping oil at 1,000 pounds per square inch; two 15-horsepower, 440-volt, 3-phase, 60-cycle electric motors; connecting piping; and hydraulic and electric controls. Remote controls were not provided.

(c) *Design Stresses.*—

(1) *Tensile.*—The allowable design stresses in tension for the following materials were based on the yield point or the ultimate tensile strength of the material. The smaller of the tabulated values was used in each instance:

Material	Type	Percent of yield point	Percent of ultimate tensile strength
Steel	Rolled or forged	40	25
Bolt steel	Rolled or forged	25	16.5
Cast steel	Castings	33	20
Brass or bronze	Rolled or cast	33	16.5

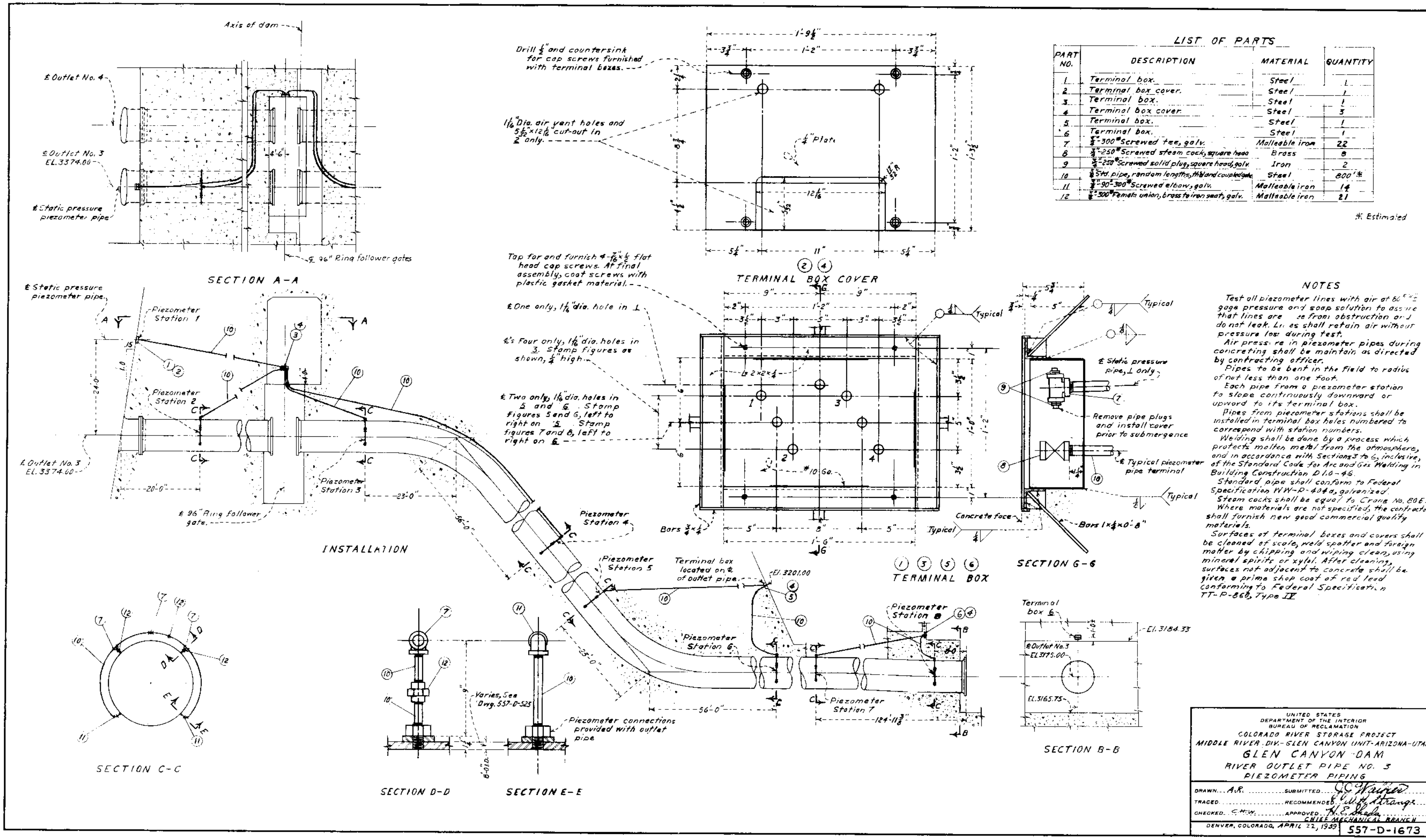


Figure 124.—River outlet pipe No. 3—Piezometer piping.

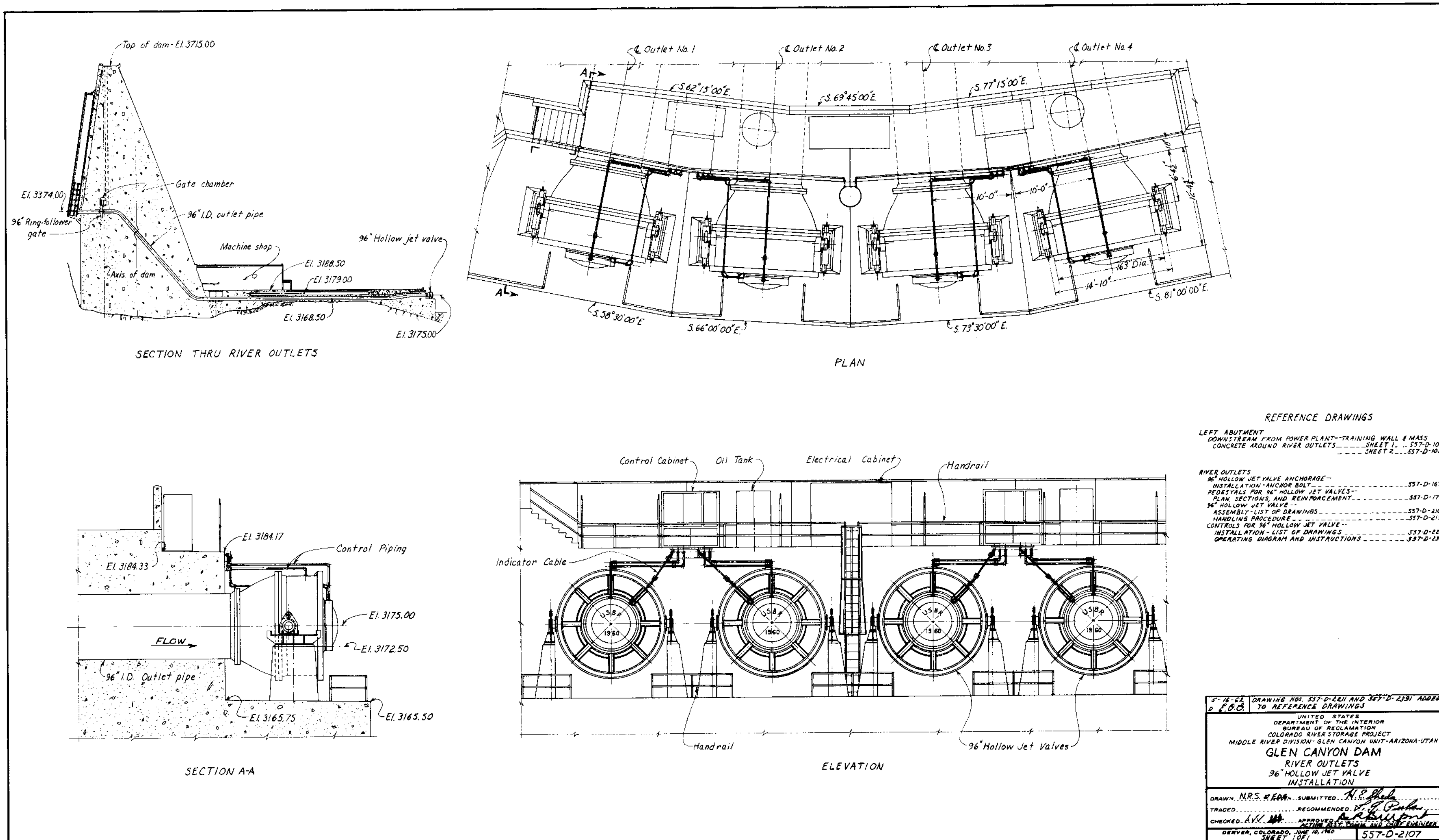


Figure 125.—River outlets 96-inch hollow-jet valve—Installation.

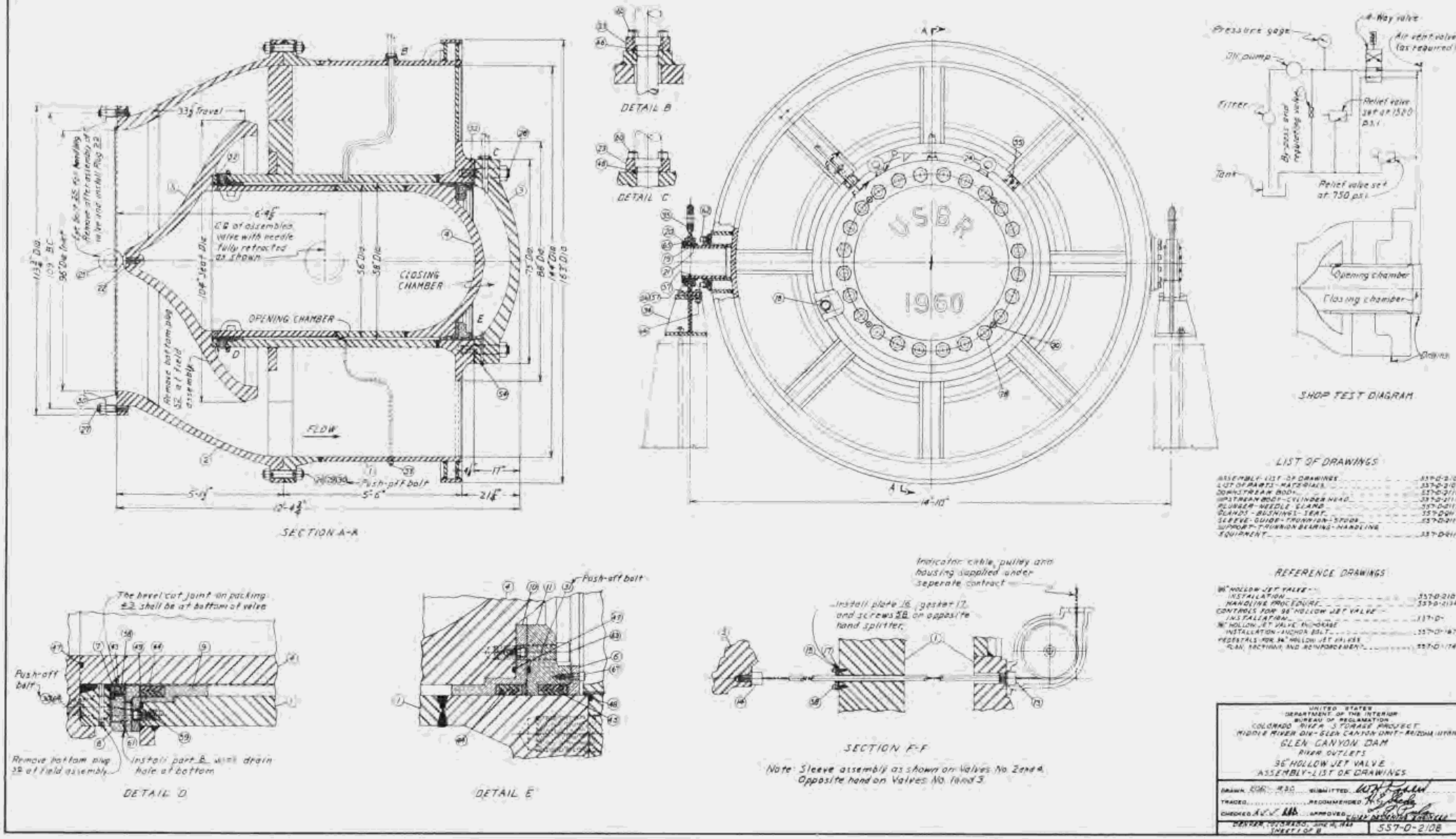


Figure 126.—River outlets 96-inch hollow-jet valve—Assembly.

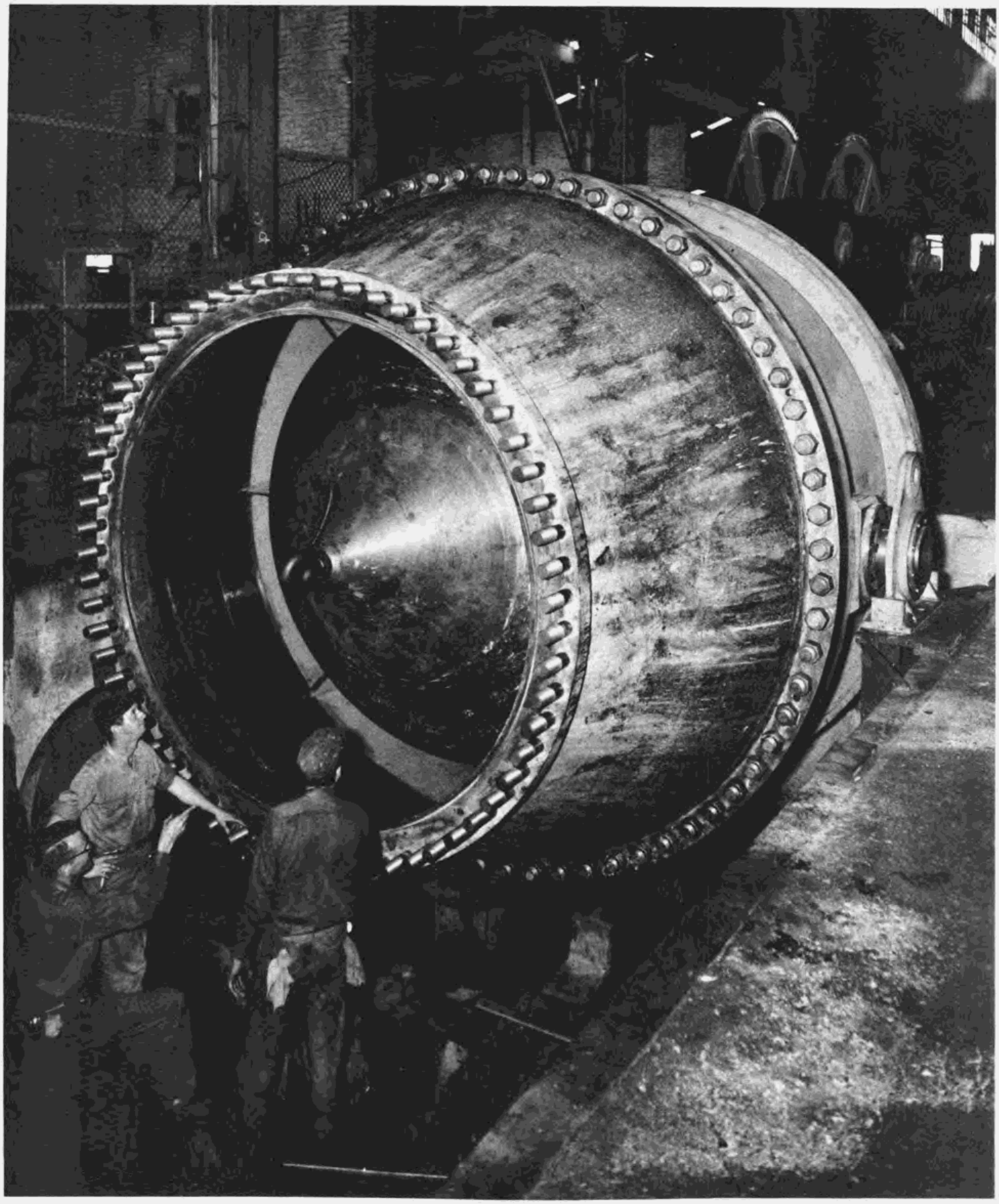


Figure 127.—Downstream view of 96-inch hollow-jet valve.

(2) *Compression.*—The allowable design stresses in compression used for the materials listed above were the same as for tension.

(3) *Shear.*—The allowable design stresses in shear were not more than 0.6 the allowable design stresses in tension.

(4) *Structural steel.*—Allowable design stresses for structural steel in tension or compression were not more than 20,000 pounds per square inch. In general, structural steel stresses were based on the American Institute of Steel Construction "Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings," 1961.

(5) *Hoist cylinder.*—The allowable design stresses for hoist cylinders were based on the recommendations of the ASME Boiler and Pressure Vessel Code—Unfired Pressure Vessels—Section VIII.

Figure 127 shows a downstream view of the 96-inch hollow-jet valve and figure 128 shows a front view of the 96-inch hollow-jet valve control cabinet.

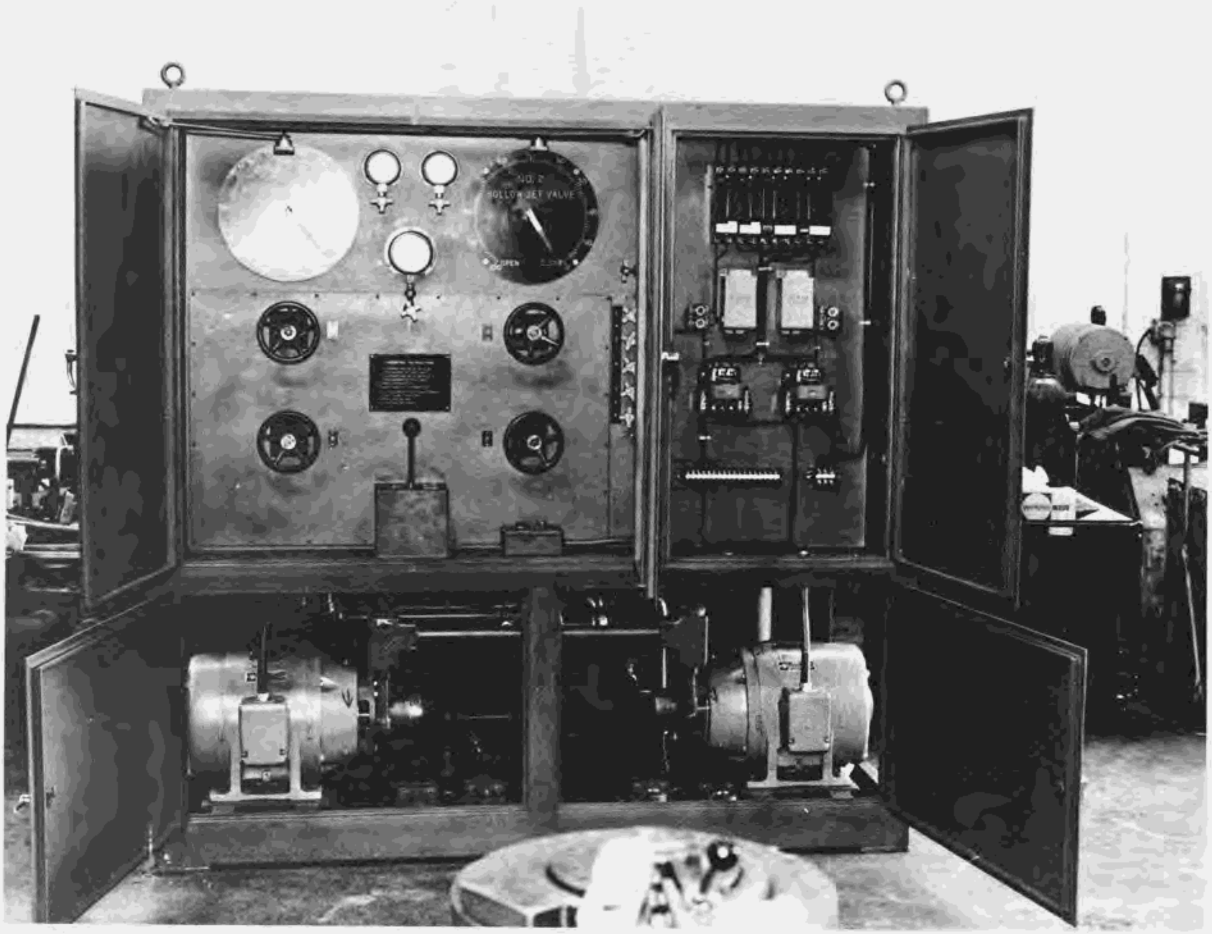


Figure 128.—Front view of 96-inch hollow-jet valve control cabinet. P557-D-31840.

CHAPTER VII. *Design*—POWER WATERWAYS

54. LOCATING PENSTOCKS. (a) *General.*—Eight 15-foot-diameter steel penstocks for delivering water to the turbines in the powerplant are embedded in the dam (figs. 129 and 130). The centerline of each penstock intake is at elevation 3470 which is about 45 feet above estimated silt elevation after 150 years of reservoir operation. The minimum water surface for power operation is elevation 3490. A 13.96- by 22.45-foot fixed-wheel hydraulically operated closure gate which can operate under unbalanced head was provided at the upstream face of the dam for each penstock. An unlined transiton section from rectangular to round was provided at the entrances of the penstocks for a smooth increase in flow velocity and to reduce the width required for each closure gate (fig. 131). A reinforced concrete trashrack structure (fig. 132), with structural steel trashracks, protects the entrances. Guides and seats for stoplogs were installed upstream from the closure gate to provide a means for inspecting the gate frames and guides if required. Special joints to take movement in three directions were designed for the penstocks, where they leave the dam, to provide for movements of the dam under variable load. These joints were set in special vaults at the toe of the dam (fig. 133). The penstocks are supported between the dam and the powerplant on reinforced concrete piers which, in turn, are carried to bedrock (figs. 134 and 135). The backfill in the area between the dam and powerplant was placed above the top of the penstocks, and a drainage system was provided as shown on figure 83.

(b) *Layout.*—The layout of the penstocks in the dam was dependent upon the following criteria:

(1) The penstocks had to be radial at the upstream face of the dam, and so located that the trashrack structures did not cross contraction joints.

(2) The minimum distance from the centerlines of the penstocks to the radial contraction joints in the dam had to be at least one and one-half pipe diameters.

(3) There were to be no bends in the penstocks between the dam and the powerplant.

(4) The minimum permissible bend radius was to be four pipe diameters.

(5) The spacing of the generators in the powerplant was to be 65 feet on centers.

In addition to the above criteria, there were limits to the permissible spacing of the contraction joints in the dam. To preclude difficulties in cooling of the blocks and grouting of the contraction joints, the minimum and maximum block widths, measured along the axis of the dam, were 40 and 70 feet, respectively. The layout criteria were satisfied by selecting a system of blocks using maximum and minimum permissible widths, bending two of the penstocks inside the powerplant structure, and anchoring horizontal bends to the mass concrete in the dam where the bends were near the downstream face of the dam.

55. STRUCTURAL DESIGN. (a) *General.*—The design was based on concrete having a compressive strength of 3,000 pounds per square inch at 28 days for structural concrete and 2,500 pounds per square inch at 28 days for mass concrete. The allowable working stresses are shown on figure 71 except that the allowable stress in the reinforcement around the pipe in the dam was increased to 25,000 pounds per square inch.

(b) *Trashrack Structures.*—The concrete trashrack structures were designed for a differential waterload of 20 feet, temperature effects, and dead load. The gate guide supports were designed for unwatering the gate guide area with normal water surface.

(c) *Gate Hoist Structure.*—The gate hoist structure was designed for the following conditions:

(1) A 25° F. temperature change.

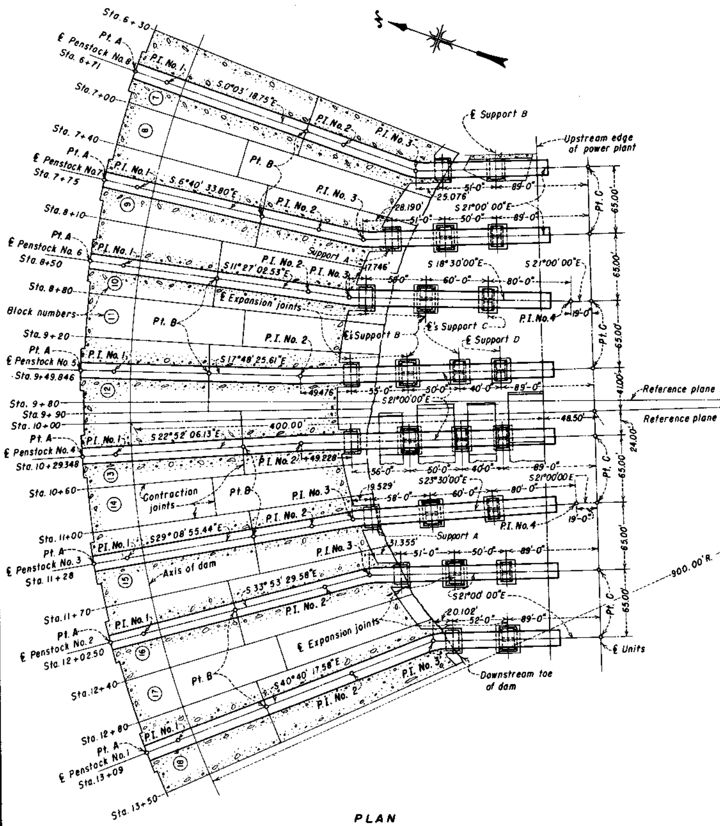
(2) A 20-foot differential waterhead around structure and 5° F. temperature change.

(3) Stoplogs in place—water on sides—thrust from stoplog seats.

(4) A pullout hoist load of 668,000 pounds which included the force to overcome any jamming of gate during lifting operation.

The gate hoist structure has a removable cover (see fig. 136). For details of the gate hoist structure see figure 137.

(d) *Gate Hoist and Stem Storage Platform.*—The gate hoist and stem storage structure is shown on figure 138. It was designed to store the gate hoist and the



PLAN

TABLE OF COORDINATES

	Pt. A	P.I. No. 1	Pt. B	P.I. No. 2	P.I. No. 3	P.I. No. 4	℄ Exp. Joint at Support A	℄ Support B	℄ Support C	℄ Support D	Pt. C
Penstock No. 1	N 2,160,754.194 E 626,284.357	N 2,160,729.476 E 626,305.597	N 2,160,660.046 E 626,365.255	N 2,160,593.093 E 626,422.786	N 2,160,528.208 E 626,478.540		N2,160,509.441 E 626,485.744	N2,160,460.895 E 626,504.379			N2,160,377.807 E 626,536.274
Penstock No. 2	N 2,160,822.143 E 626,373.612	N 2,160,795.090 E 626,391.785	N 2,160,719.090 E 626,442.839	N2,160,645.826 E 626,492.055	N 2,160,607.754 E 626,517.630		N2,160,578.481 E 626,528.867	N2,160,530.868 E 626,547.144	N2,160,484.189 E 626,565.062		N2,160,401.101 E 626,596.957
Penstock No. 3	N 2,160,863.180 E 626,440.525	N 2,160,834.717 E 626,456.399	N 2,160,727.284 E 626,516.315	N2,160,677.673 E 626,543.984	N 2,160,641.619 E 626,564.091	N2,160,442.133 E 626,650.830	N2,160,623.710 E 626,571.878	N2,160,570.521 E 626,595.006	N2,160,515.497 E 626,618.930		N2,160,424.394 E 626,657.639
Penstock No. 4	N 2,160,908.749 E 626,533.920	N 2,160,878.720 E 626,546.585	N2,160,765.364 E 626,594.395	N2,160,713.038 E 626,616.464			N2,160,667.080 E 626,634.106	N 2,160,614.799 E 626,654.174	N2,160,568.120 E 626,672.093	N 2,160,530.777 E 626,686.427	N2,160,447.688 E 626,718.322
Penstock No. 5	N 2,160,937.860 E 626,612.459	N 2,160,906.831 E 626,622.426	N2,160,819.672 E 626,650.422	N2,160,735.630 E 626,677.416			N2,160,689.440 E 626,695.147	N2,160,638.093 E 626,714.857	N 2,160,591.414 E 626,732.775	N2,160,554.071 E 626,747.110	N2,160,470.982 E 626,779.005
Penstock No. 6	N 2,160,964.423 E 626,714.225	N 2,160,932.481 E 626,720.696	N2,160,842.758 E 626,738.870	N2,160,756.245 E 626,756.393	N2,160,716.611 E 626,764.421	N2,160,512.014 E 626,832.879	N2,160,699.782 E 626,770.052	N2,160,644.780 E 626,788.456	N2,160,587.880 E 626,807.494		N2,160,494.276 E 626,839.688
Penstock No. 7	N 2,160,976.871 E 626,792.260	N 2,160,944.502 E 626,796.049	N2,160,822.301 E 626,810.352	N2,160,765.905 E 626,816.953	N2,160,721.268 E 626,822.178		N2,160,694.950 E 626,832.280	N2,160,647.338 E 626,850.557	N2,160,600.659 E 626,868.476		N2,160,517.570 E 626,900.370
Penstock No. 8	N 2,160,98[illegible].302 E 626,901.617	N 2,160,950.712 E 626,901.649	N 2,160,809.708 E 626,901.785	N 2,160,770.896 E 626,901.822	N2,160,694.976 E 626,901.895		N 2,160,671.565 E 626,910.882	N2,160,623.953 E 626,929.158			N2,160,540.864 E 626,961.053

NOTES

All stations shown are measured along axis of dam. For additional notes and reference drawings, see Dwg. 557-D-1004.

REFERENCE DRAWING

PENSTOCKS AND OUTLET PIPES-
GENERAL PLAN AND LIST OF DRAWINGS...557-D-513

TABLE OF HORIZONTAL LENGTHS (FT.)

	Pt. A to P.I. No. 1	P.I. No. 1 to Pt. B	Pt. B to P.I. No. 2	P.I. No. 2 to P.I. No. 3	P.I. No. 2 to Pt. C	P.I. No. 3 to P.I. No. 4	P.I. No. 3 to Pt. C
Penstock No. 1	32.591	91.540	88.276	85.549			161.102
Penstock No. 2	32.591	91.555	88.260	45.865			221.355
Penstock No. 3	32.591	123.011	56.805	41.282		217.529	
Penstock No. 4	32.591	123.025	56.790		284.228		
Penstock No. 5	32.591	91.545	88.271		283.476		
Penstock No. 6	32.591	91.546	88.270	40.439		215.746	
Penstock No. 7	32.591	123.035	56.781	44.942			218.190
Penstock No. 8	32.591	141.004	38.812	75.920			165.076

6-1-64 ADDED BUTTRESS WALLS.
D. E.Q. J.

THIS DRAWING SUPERSEDES DWG. 557-D-102 IN PART.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZONA-UTAH

GLEN CANYON DAM

PENSTOCKS
PLAN

DRAWN P.S.R. SUBMITTED [signature]
TRACED F.E.M. RECOMMENDED [signature]
CHECKED [signature] APPROVED [signature] CHIEF DESIGNING ENGINEER

DENVER, COLORADO, SEPT. 17, 1959 | 557-D-1003

Figure 129.—Plan of penstocks.

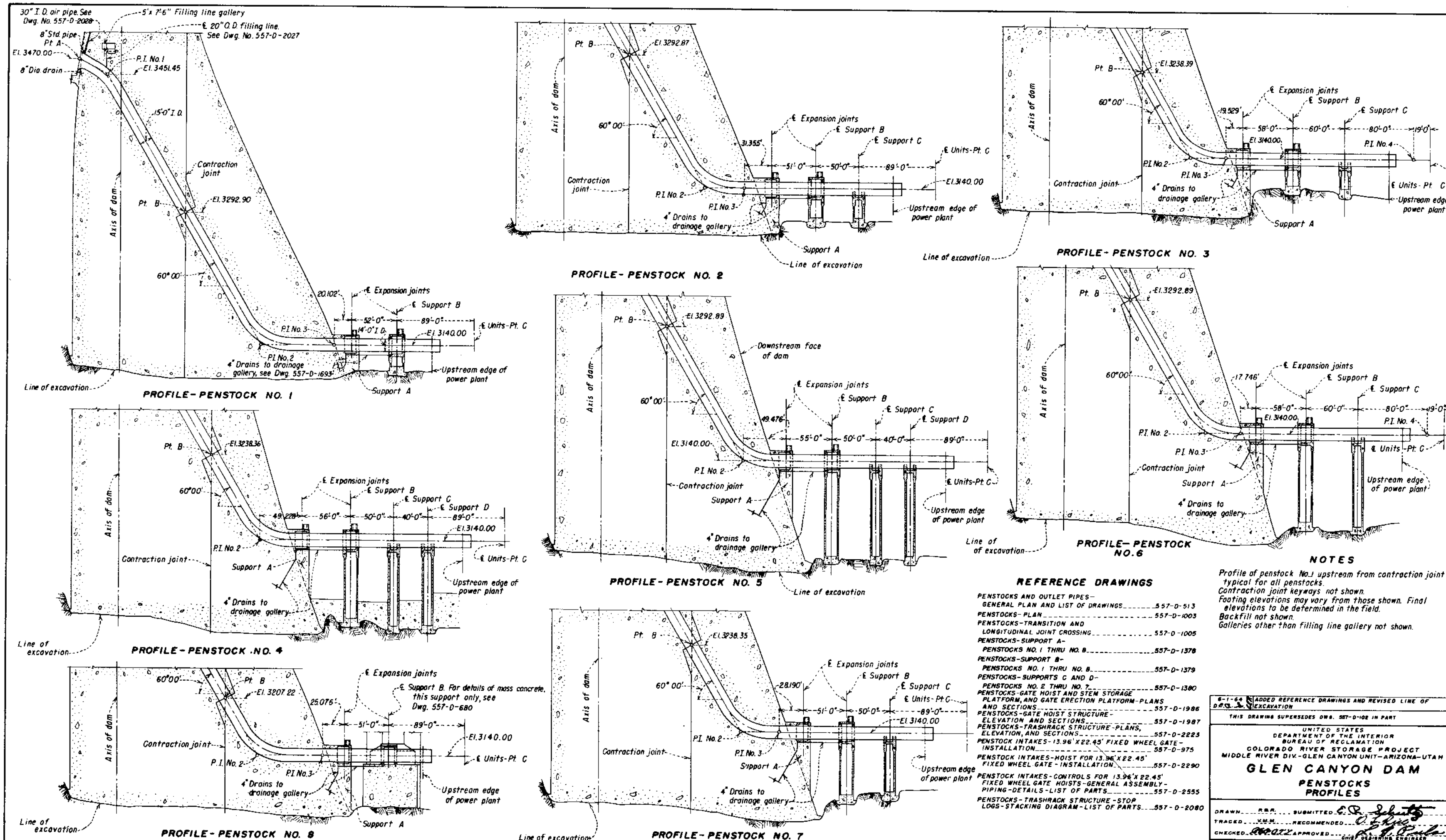
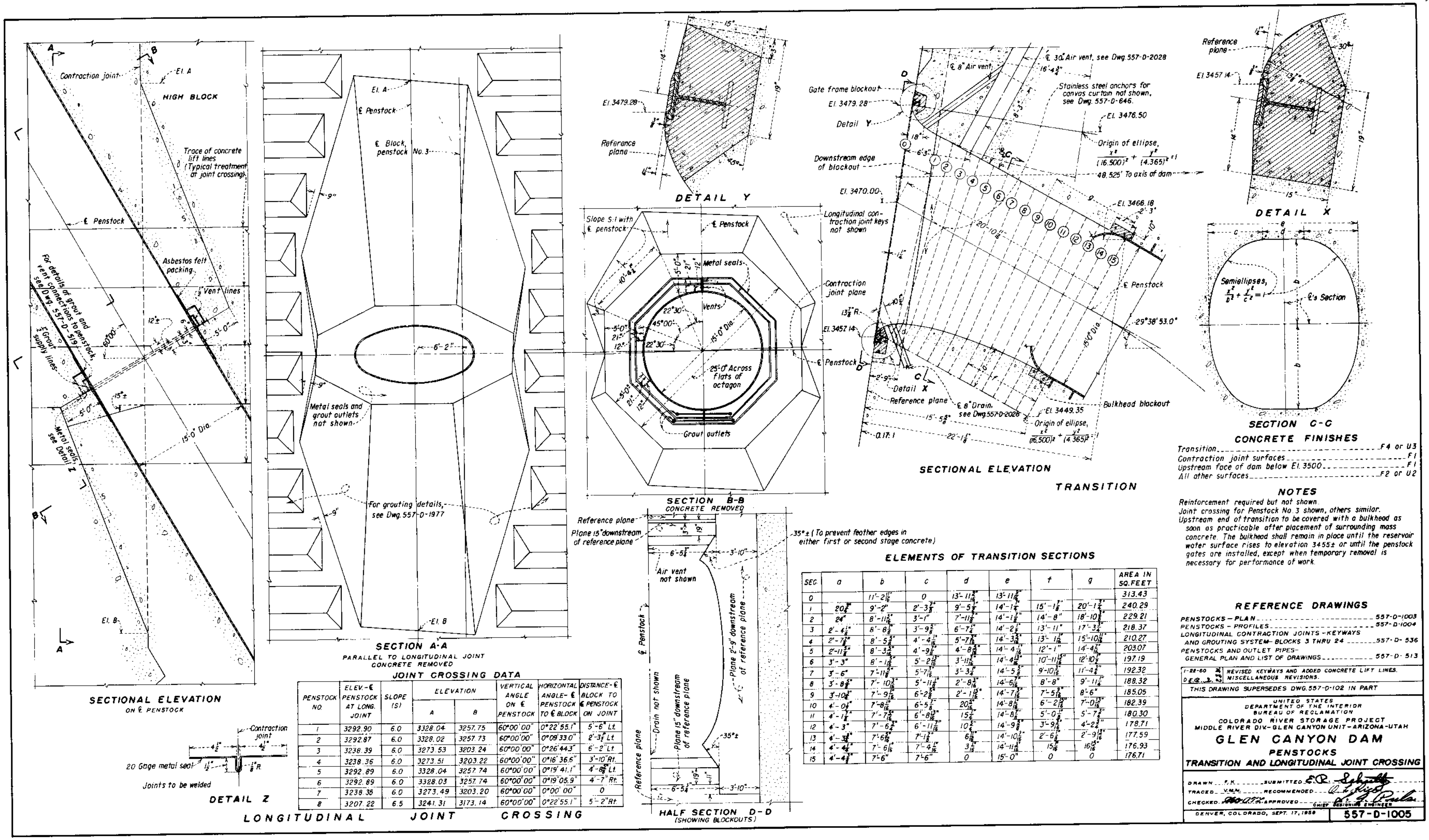


Figure 130.—Profile of penstocks.



JOINT CROSSING DATA

PENSTOCK NO.	ELEV.-℄ PENSTOCK AT LONG. JOINT	SLOPE (S)	ELEVATION A	ELEVATION B	VERTICAL ANGLE ON ℄ PENSTOCK	HORIZONTAL ANGLE-℄ PENSTOCK TO ℄ BLOCK	DISTANCE-℄ BLOCK TO ℄ PENSTOCK ON JOINT
1	3292.90	6.0	3328.04	3257.75	60°00'00"	0°22'55.1"	5'-6" Lt.
2	3292.87	6.0	3328.02	3257.73	60°00'00"	0°09'33.0"	2'-3½" Lt.
3	3238.39	6.0	3273.53	3203.24	60°00'00"	0°26'44.3"	6'-2" Lt.
4	3238.36	6.0	3273.51	3203.22	60°00'00"	0°16'36.6"	3'-10" Rt.
5	3292.89	6.0	3328.04	3257.74	60°00'00"	0°19'41.1"	4'-8⅝" Lt.
6	3292.89	6.0	3328.03	3257.74	60°00'00"	0°19'05.9"	4'-7" Rt.
7	3238.35	6.0	3273.49	3203.20	60°00'00"	0°00'00"	0
8	3207.22	6.5	3241.31	3173.14	60°00'00"	0°22'55.1"	5'-2" Rt.

LONGITUDINAL JOINT CROSSING

ELEMENTS OF TRANSITION SECTIONS

SEC	a	b	c	d	e	f	g	AREA IN SQ. FEET
0		11'-2 11/16"	0	13'-11 9/16"	13'-11 9/16"			313.43
1	20 3/4"	9'-2"	2'-3 7/8"	9'-5 1/2"	14'-1 1/4"	15'-1 7/8"	20'-1 3/4"	240.29
2	24"	8'-11 3/16"	3'-1"	7'-11 7/8"	14'-1 7/8"	14'-8"	18'-10 5/8"	229.21
3	2'-4 1/8"	8'-8 1/8"	3'-9 5/8"	6'-7 1/4"	14'-2 1/2"	13'-11"	17'-3 3/4"	218.37
4	2'-7 7/8"	8'-5 5/8"	4'-4 1/16"	5'-7 9/16"	14'-3 5/16"	13'-1 1/16"	15'-10 11/16"	210.27
5	2'-11 3/8"	8'-3 5/16"	4'-9 7/8"	4'-8 5/16"	14'-4 1/16"	12'-1"	14'-4 7/16"	203.07
6	3'-3"	8'-1 5/16"	5'-2 13/16"	3'-11 3/16"	14'-4 13/16"	10'-11 15/16"	12'-10 1/2"	197.19
7	3'-6"	7'-11 5/8"	5'-7 3/16"	3'-3 1/4"	14'-5 5/8"	9'-10 3/16"	11'-4 3/4"	192.32
8	3'-8 5/8"	7'-10 3/16"	5'-11 1/8"	2'-8 3/16"	14'-6 7/16"	8'-8"	9'-11 1/4"	188.32
9	3'-10 3/4"	7'-9 1/16"	6'-2 5/8"	2'-1 15/16"	14'-7 3/16"	7'-5 9/16"	8'-6"	185.05
10	4'-0 1/2"	7'-8 1/16"	6'-5 7/8"	20 5/16"	14'-8 1/16"	6'-2 15/16"	7'-0 13/16"	182.39
11	4'-1 7/8"	7'-7 5/16"	6'-8 13/16"	15 1/4"	14'-8 7/8"	5'-0 1/8"	5'-7 3/4"	180.30
12	4'-3"	7'-6 3/4"	6'-11 7/16"	10 3/4"	14'-9 5/8"	3'-9 1/16"	4'-2 3/4"	178.71
13	4'-3 3/4"	7'-6 5/16"	7'-1 7/8"	6 11/16"	14'-10 7/16"	2'-6 1/8"	2'-9 13/16"	177.59
14	4'-4 1/4"	7'-6 1/16"	7'-4 1/16"	3 7/16"	14'-11 3/16"	15 7/16"	16 15/16"	176.93
15	4'-4 3/8"	7'-6"	7'-6"	0	15'-0"	0	0	176.71

Figure 131.—Penstocks—Transition and longitudinal joint crossing.

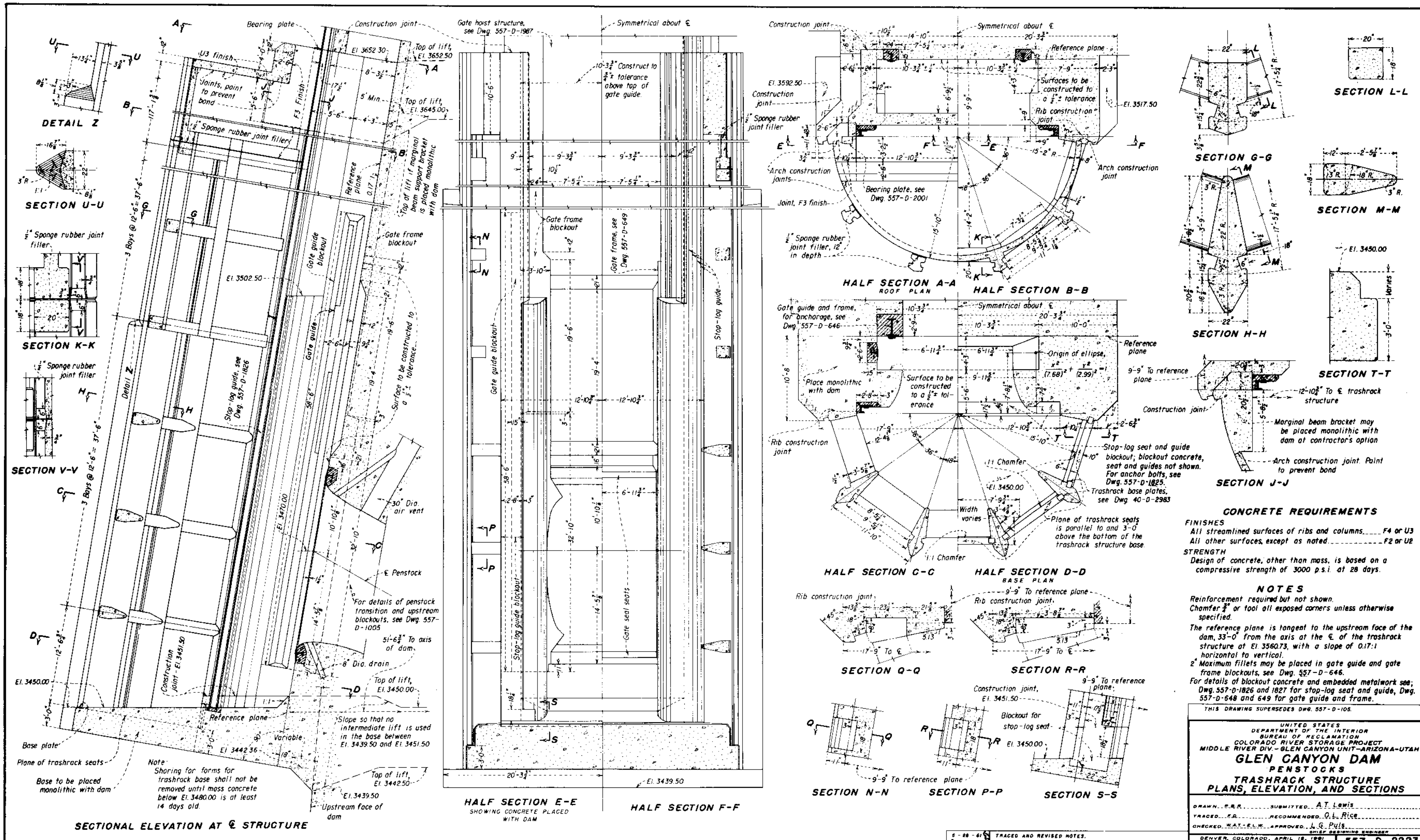
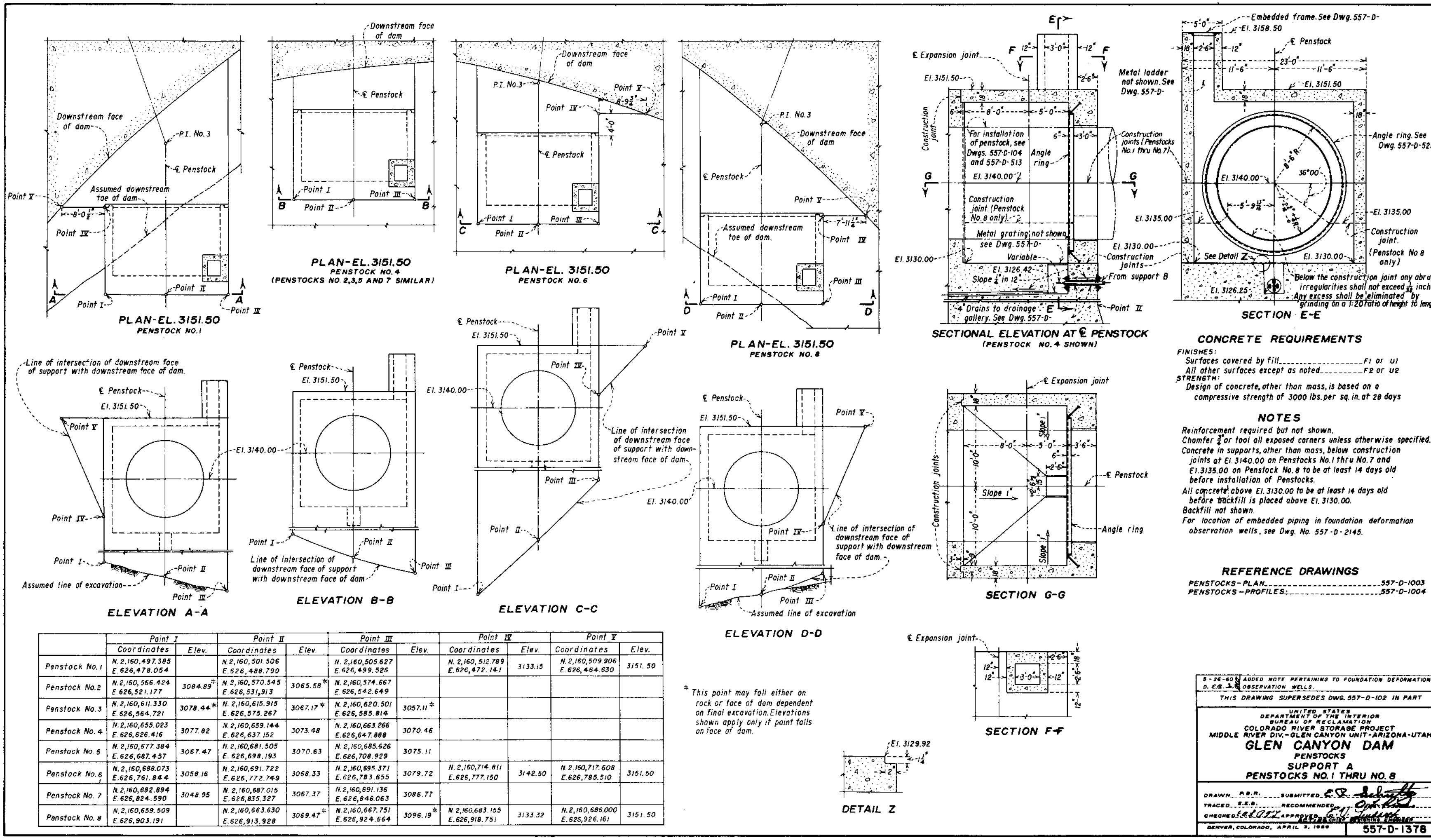


Figure 132.—Penstocks trashrack structure—Plans, elevation, and sections.



	Point I		Point II		Point III		Point IV		Point V	
	Coordinates	Elev.	Coordinates	Elev.	Coordinates	Elev.	Coordinates	Elev.	Coordinates	Elev.
Penstock No. 1	N. 2,160,497.385 E. 626,478.054		N. 2,160,501.506 E. 626,488.790		N. 2,160,505.627 E. 626,499.526		N. 2,160,512.789 E. 626,472.141	3133.15	N. 2,160,509.906 E. 626,464.630	3151.50
Penstock No. 2	N. 2,160,566.424 E. 626,521.177	3084.89*	N. 2,160,570.545 E. 626,531,913	3065.58*	N. 2,160,574.667 E. 626,542.649					
Penstock No. 3	N. 2,160,611.330 E. 626,564.721	3078.44*	N. 2,160,615.915 E. 626,575.267	3067.17*	N. 2,160,620.501 E. 626,585.814	3057.11*				
Penstock No. 4	N. 2,160,655.023 E. 626,626.416	3077.82	N. 2,160,659.144 E. 626,637.152	3073.48	N. 2,160,663.266 E. 626,647.888	3070.46				
Penstock No. 5	N. 2,160,677.384 E. 626,687.457	3067.47	N. 2,160,681.505 E. 626,698.193	3070.63	N. 2,160,685.626 E. 626,708.929	3075.11				
Penstock No. 6	N. 2,160,688.073 E. 626,761.844	3058.16	N. 2,160,691.722 E. 626,772.749	3068.33	N. 2,160,695.371 E. 626,783.655	3079.72	N. 2,160,714.811 E. 626,777.150	3142.50	N. 2,160,717.608 E. 626,785.510	3151.50
Penstock No. 7	N. 2,160,682.894 E. 626,824.590	3048.95	N. 2,160,687.015 E. 626,835.327	3067.37	N. 2,160,691.136 E. 626,846.063	3086.77				
Penstock No. 8	N. 2,160,659.509 E. 626,903.191		N. 2,160,663.630 E. 626,913.928	3069.47*	N. 2,160,667.751 E. 626,924.664	3096.19*	N. 2,160,683.155 E. 626,918.751	3133.32	N. 2,160,686.000 E. 626,926.161	3151.50

Figure 133.—Penstocks No. 1 through No. 8—Support A.

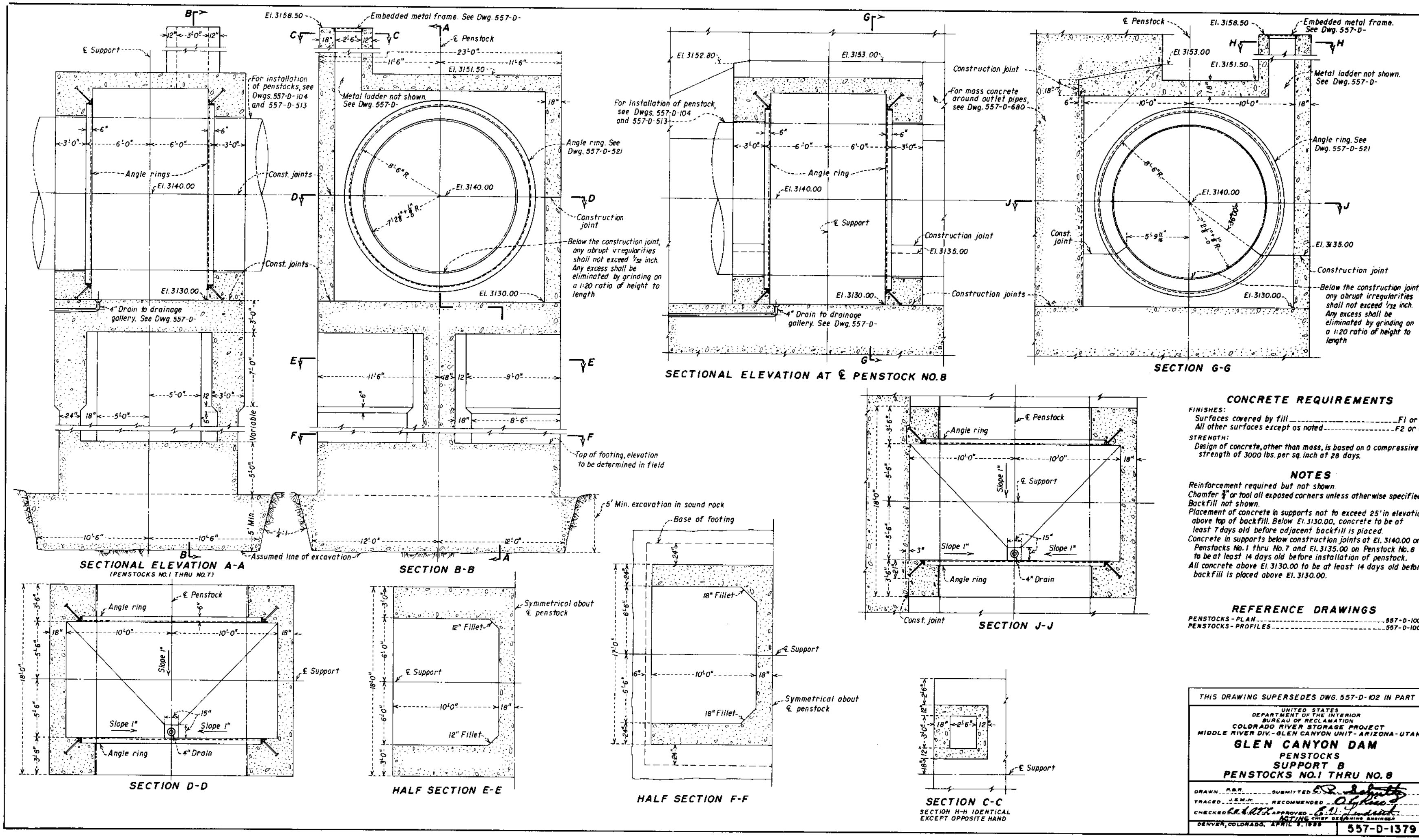


Figure 134.—Penstocks No. 1 through No. 8—Support B.

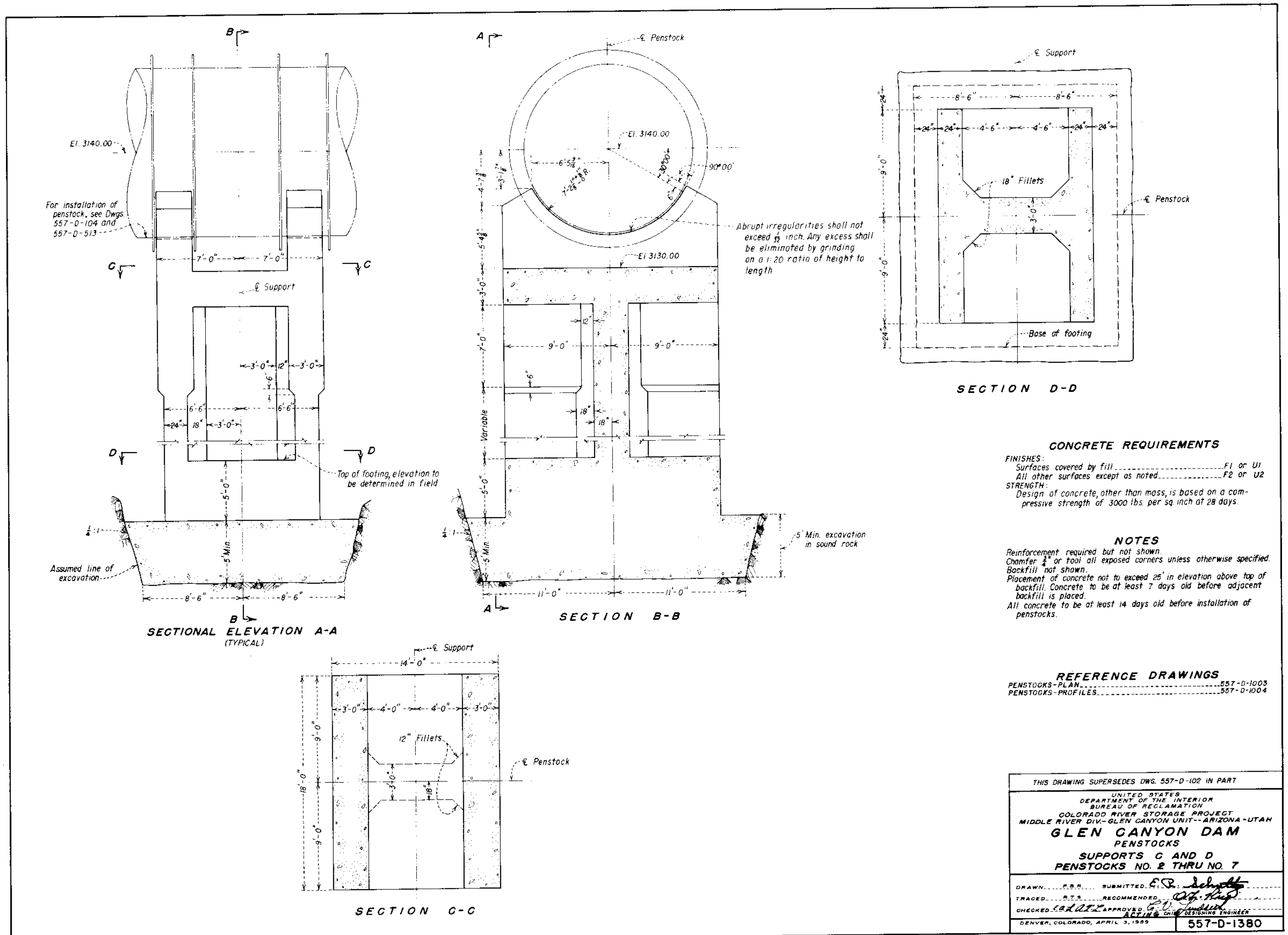


Figure 135.—Penstocks No. 2 through No. 7—Supports C and D.

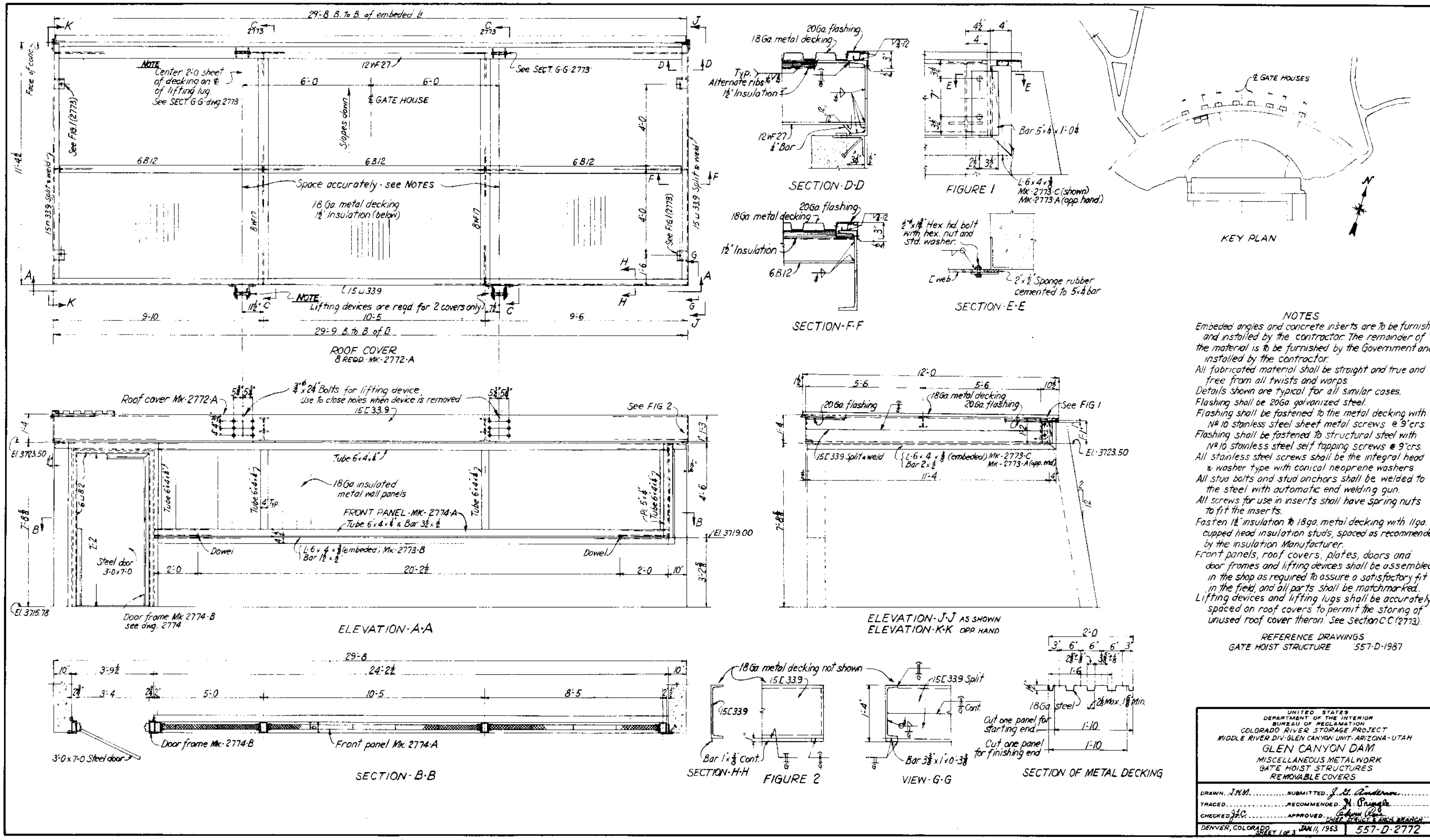


Figure 136.—Miscellaneous metalwork—Gate hoist structures removable covers.

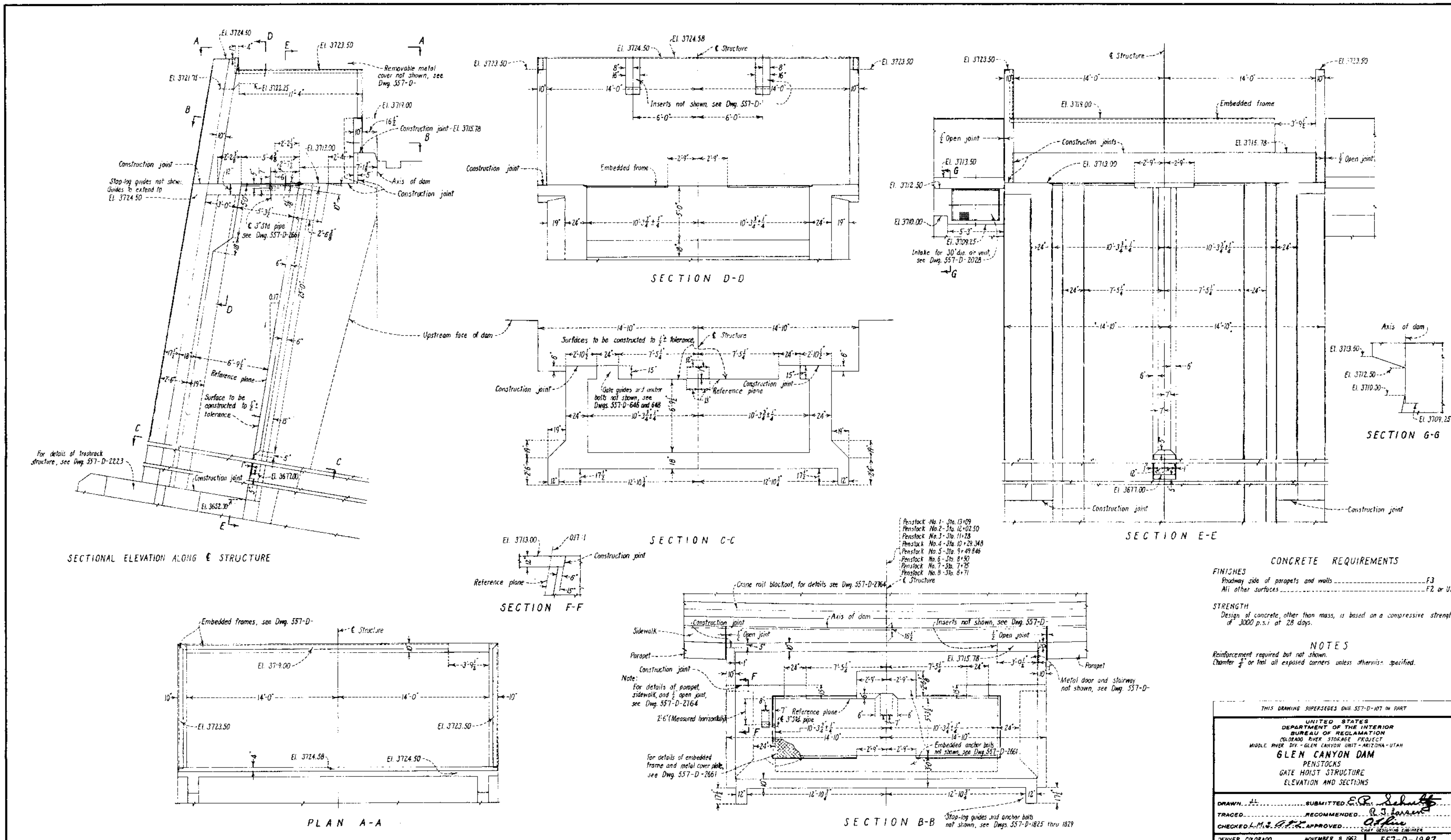


Figure 137.—Penstocks gate hoist structure—Elevation and sections.

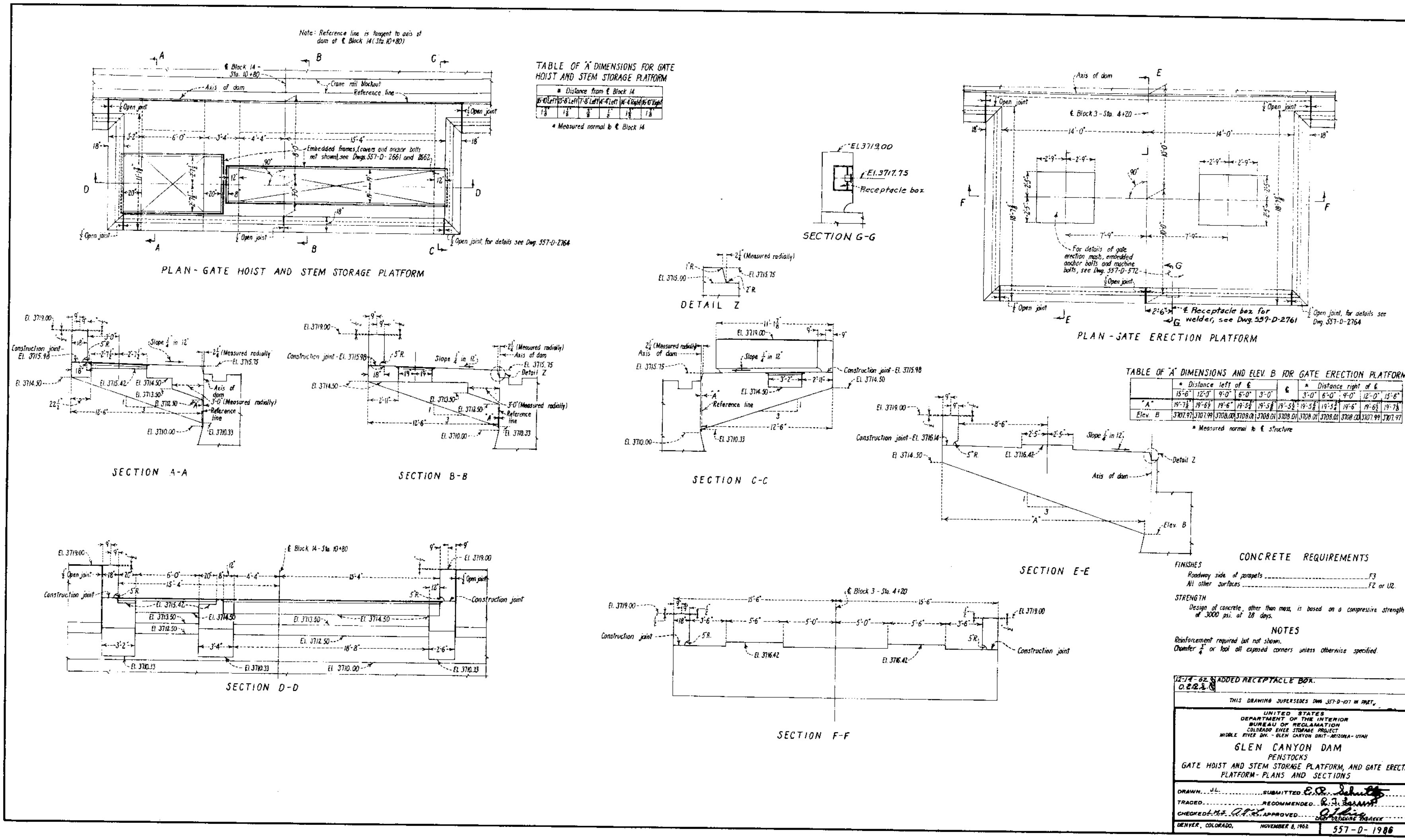


Figure 138.—Penstocks gate hoist and stem storage platform, and gate erector platform—Plans and sections.

hoist stems when a penstock fixed-wheel gate is being serviced. The structure was designed to support its own weight as well as the dead load weights of the gate hoist and stems.

(e) *Gate Erection Platform.*—The gate erection platform is also shown on figure 138. It was designed for the erection and servicing of the penstock fixed-wheel gates and the river outlet bulkhead gate. The river outlet bulkhead gate will be stored on the platform.

(f) *Penstock Concrete Reinforcement.*—The reinforcement requirements for the penstocks were broken into three sections, as follows:

(1) The approximately 21-foot-long upstream transition which has no steel liner.

(2) The circular section with steel liner extending from the downstream end of the transition to a point approximately 30 feet upstream from the downstream face of the dam.

(3) The circular section with steel liner from a point 30 feet upstream from the downstream face to the downstream face of the dam.

Since the transition section has no steel liner, cracks could open into adjacent galleries and openings which could result in excessive leakage. In order to control cracking, the transition section was reinforced for tensile forces caused by dam stresses, internal bursting pressures, and temperature effects.

The circular section with steel liner, except the approximately 30-foot-long downstream section, has enough mass concrete cover that tension cracks cannot propagate to the surface of the dam. There is no leakage problem since the penstock has a steel liner. Therefore this section of the penstock was reinforced only for tensile forces caused by dam stresses and temperature effects.

The downstream approximately 30-foot section presents no leakage problem since it has a steel liner; however, tension cracks could open to the downstream face of the dam in the area. In order to control cracking this section was reinforced for tensile forces due to dam stresses, temperature effects, and internal bursting pressure, including waterhammer effects. In addition, anchorage reinforcement was provided for the unbalanced forces in the horizontal bends in this area.

(g) *Supports.*—The supports shown on figures 129, 130, 133, 134, and 135 consist of the following component parts:

(1) Vaults (two upstream supports) or bearing walls

(2) H-column

(3) Footing

The vaults were designed for the following loads:

(1) Dead load of structure

(2) Backfill to elevation 3157

(3) Temperature effects

(4) Earthquake effects

(5) Penstock reactions

The bearing walls on the supports without vaults were designed for the following loads:

(1) Temperature effects

(2) Earthquake effects

(3) Penstock reactions

The overall dimensions of the H-columns were determined in part by the vaults or supporting walls above. The length to depth ratio was kept within the short column range. The individual legs of the H-column were given a small length-to-depth ratio to prevent buckling.

The H-column and its members individually were designed for the following loads:

(1) Dead weight from above

(2) Penstock reactions

(3) Earthquake effects

The footings were stepped with the top part having the same exterior dimensions as the H-column and the bottom part widened in both directions to increase stability and reduce foundation stresses.

The footings were designed for the following loads:

(1) Dead loads.

(2) Penstock reactions.

(3) Fill material (partly saturated).

(4) Earthquake effects.

A minimum of 1 percent of vertical reinforcement was placed in the H-columns.

56. TAILRACE TRAINING WALLS. Tailrace training walls are provided on each side of the tailrace downstream of the powerplant. On the left side of the upstream 75 feet of the wall is a gravity section and the rest of the wall is cantilevered off the mass concrete under the river outlets and hollow-jet valves. It is shown on figure 114. It also serves as a retaining wall to retain the fill for the powerplant parking area. The right training wall is of the line-drilled type and was doweled to the rock, figure 139.

The left training wall was designed for the following loads:

(1) Parking area fill material (saturated).

(2) 100-ton trailer unit including impact from bumping curb.

(3) Earthquake effect.

57. TAILRACE SLAB. To insure a minimum tailwater for the turbines, a tailrace slab (fig. 140) was constructed between the tailrace walls. This slab sloped up on a 6 to 1 slope from the draft tubes to form a weir at elevation 3132, 180 feet downstream of the powerplant. From the weir the slab sloped down until it was approximately 20 feet below riverbed. It was originally planned that the tailrace channel and weir would be constructed of riprap, but lack of suitable rock within economical haul distance dictated the change to a reinforced concrete slab. The riprap weir was tested in a hydraulic model.[1]

The maximum drawdown rate of the tailwater over the weir was established at 10 feet in 10 minutes. Studies showed that porous concrete drains at 10-foot centers each way provided adequate relief from uplift pressures so that, in general, an 8-inch slab would be stable. Near the left training wall, studies indicated that saturation of the fill and bedrock behind the wall caused higher uplift adjacent to the wall and the slab thickness was increased to 12 inches. At the weir, the slab was made 12 inches thick to provide additional stability for the weir.

During the April through July runoff period in 1965, discharges up to 50,000 cubic feet per second were released in order to obtain a rated head at Lake Mead and maintain Lake Powell at about elevation 3490. The discharge was obtained by using combinations of the discharges from the turbines, the diversion tunnel outlets, and the river outlets. The river outlets were used to their design capacity when the tunnel outlets were closed.

During the night of April 20, 1965, the slab was undermined and portions of it sank from sight. At the time of the failure the reservoir water surface was at about elevation 3490, four of the eight units in the powerplant were operating, the diversion tunnel outlets and outlet No. 1 were closed, outlet No. 2 was 25 percent open, outlets No. 3 and 4 were 90 percent open, and the tailwater was 4 to 6 feet lower than had been predicted and used in the model studies. Operating the river outlets at reservoir water surface elevation 3490, instead of at normal water surface elevation 3700 as used in the model, caused the jets to impinge closer to the weir.

Observation of flows in the tailrace area, after the failure, showed a reversal of flow due to a large eddy extending to the location of the weir. This reverse flow had an unexpected velocity estimated to be 15 feet per second. The operation of the outlets at the lower head, operation of only four powerplant units instead of eight, and the lower tailwater all intensified the eddy action over that predicted from the model. The scour from this reverse flow, caused by the large eddy, undermined the toe of the slab and rapidly removed large amounts of fine bedding material, causing collapse of the slab. The extensive use of the river outlets at near full capacity with the reservoir at about elevation 3490 and with only four of the powerplant units operating had not been anticipated and had not been checked in the model.

To date (June 1969) the floor slab has not been repaired or replaced, as observation has indicated no further significant erosion in the tailrace floor.

[1] "Hydraulic Model Studies of the Spillways and Outlet Works—Glen Canyon Dam—Colorado River Storage Project, Arizona," Hydraulic Laboratory Report No. Hyd-469, Bureau of Reclamation, February 18, 1964 (unpublished).

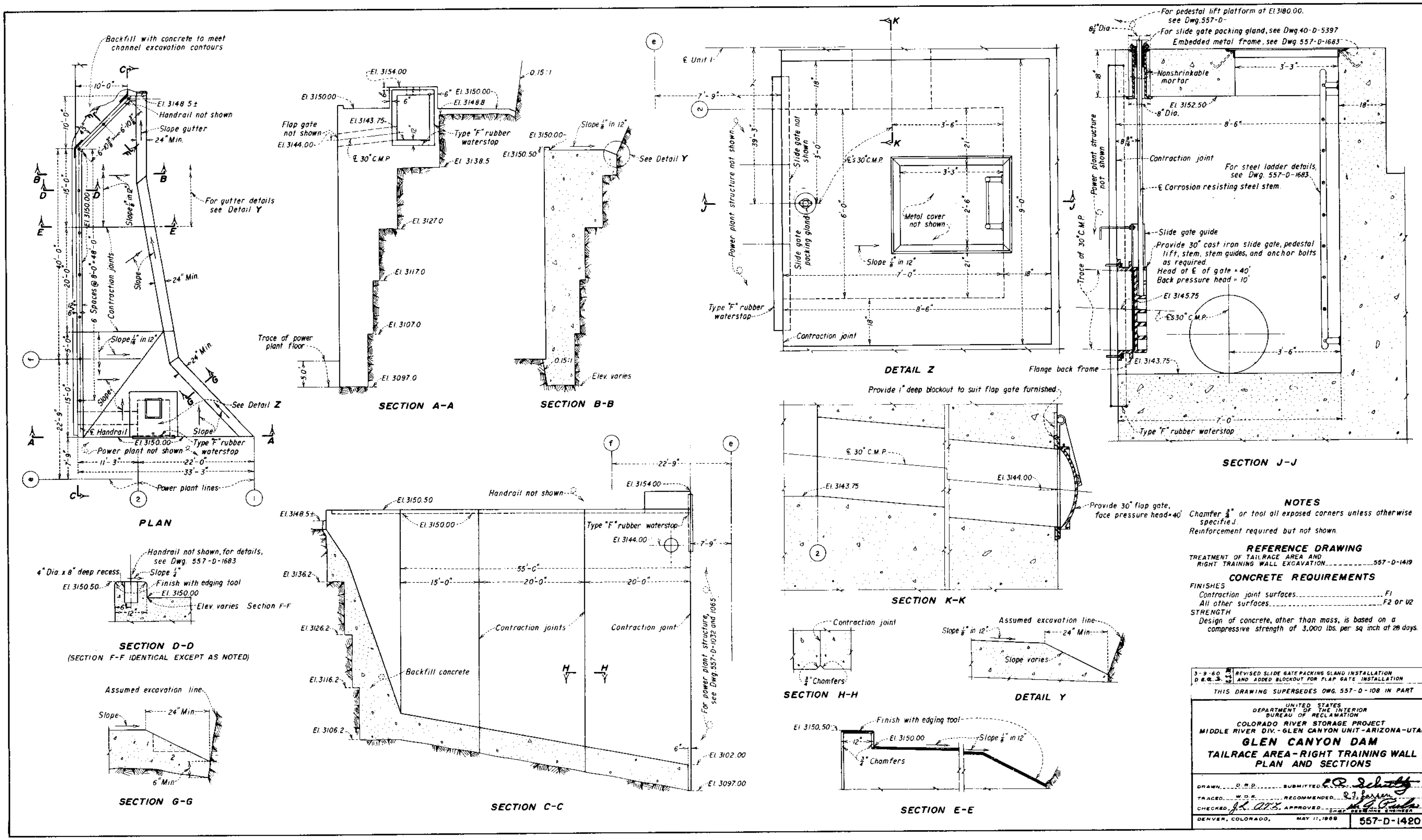


Figure 139.—Dam tailrace area—Right training wall—Plan and sections.

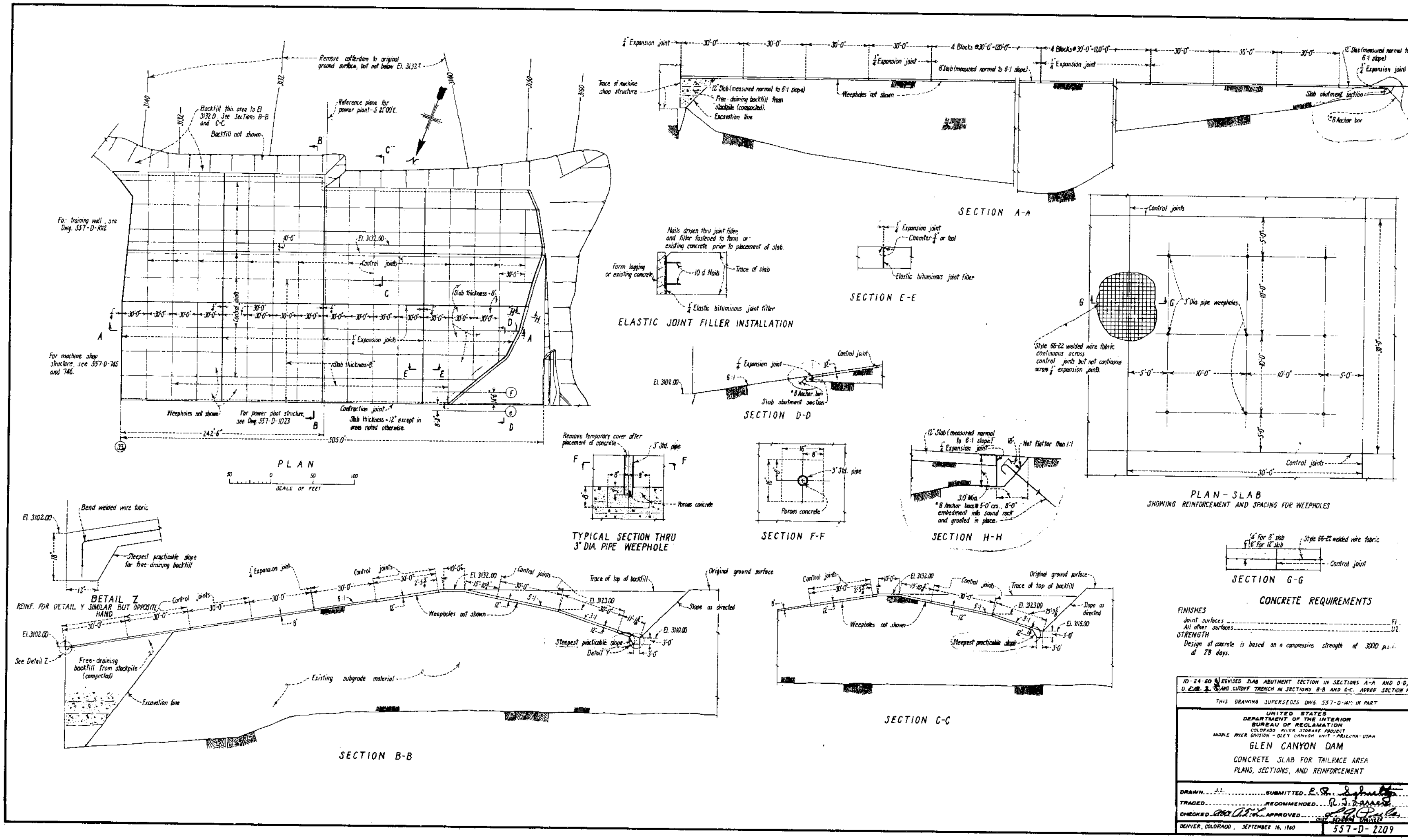


Figure 140.—Concrete slab for tailrace area—Plans, sections, and reinforcement.

58. PENSTOCK STEEL DESIGN. (a) *Description.*—Eight penstocks are provided to conduct water from the reservoir through the dam and to the eight turbines in the powerplant. The penstock alinements and profiles are shown in figures 122, 141, and 142. Fabrication was performed by a separate contract under invitation No. DS-5052.

The penstocks begin as 15-foot-inside-diameter pipes at the downstream ends of formed concrete rectangular-to-circular transitions in the upstream face of the dam at centerline elevation 3470.0. Extending downward toward the dam toe, they taper through reducing bends to an inside diameter of 14 feet, then level off at centerline elevation 3140.0 and emerge from the dam. Passing through fill for an average 150 feet, they then enter the powerplant. Sleeve-type coupled makeup sections 10 feet long connect to the turbine spiral cases. Makeup sections are designed and supported so that they may be disconnected and displaced laterally to permit access to penstocks for major inside inspection and maintenance. Penstocks in fill are sleeve-type coupled 50-foot sections, excepting those sections nearest the dam which are coupled with special double-end expansion joints. Penstocks in fill are supported by concrete piers which extend to bedrock. The average length of each penstock is about 610 feet. Minimum and maximum penstock plate thicknesses are 7/8 inch and 1-13/16 inches, respectively.

A 24-inch outside-diameter penstock filling header pipe extends across and above the penstocks near their upstream ends. Individual 20-inch outside-diameter valved riser pipes connect each penstock to the header.

(b) *Design.*—The penstocks were designed for a maximum total head of 695 feet including water hammer, measured at the centerlines of the generating units. The maximum designed static head is 560 feet at this point. Water hammer was calculated for the most severe combination of reservoir water surface elevation with normal discharge and a turbine wicket gate closure time of 5 seconds from full open position.

Each turbine discharge is 3,580 cubic feet per second at 450-foot rated head. The corresponding average velocities in each penstock are 20.26 and 23.26 feet per second for the 15- and 14-foot-diameter portions, respectively.

The special double-end expansion joints mentioned above in subsection (a) were designed to accommodate the displacement of the dam amounting to 2-1/4 inches downstream and 2-1/4 inches vertically downward at centerline elevation of the 14-foot-diameter pipes. Sleeve-type coupled field joints downstream from the expansion joints were also designed to allow small angular displacements in the alinements of the penstocks. The 14-foot-diameter penstock sections in earthfill were designed against flotation on the assumption of nonfluid fill material weighing not less than 40 pounds per cubic foot in water, or 102.4 pounds per cubic foot in air.

Penstock sections, including expansion joints and stiffener rings, were fabricated of steel plates conforming to ASTM Designation A 201, grade B. Middle rings and follower rings of sleeve-type couplings were fabricated of steels conforming to ASTM Designations A 212, grade B, and A 7, respectively.

All permanent joints were welded. All girth and longitudinal welds in penstock shells, expansion joint inner and outer sleeves, and sleeve-type coupling middle rings were fully radiographed in accordance with Section VIII of the ASME Boiler and Pressure Vessel Code. Code basic design working unit stresses for ASTM A 201 and A 212 steels used were 15,000 and 17,500 pounds per square inch, respectively. Joint efficiency was 95 percent for both steels. All field erection sections, including expansion joints and sleeve-type couplings were furnace stress relieved in accordance with the afore-mentioned code.

Completed sections of penstocks, including expansion joints and sleeve-type couplings were hydrostatically tested at pressures computed from the formula:

$$P = \frac{43{,}000\,T}{D}$$

where:

P = test pressure in pounds per square inch,

T = minimum thickness in inches, of plate course in section tested, and

D = inside diameter of pipe in inches.

A 24-inch-outside-diameter manhole is located in the first bend of each penstock. The formed inlet

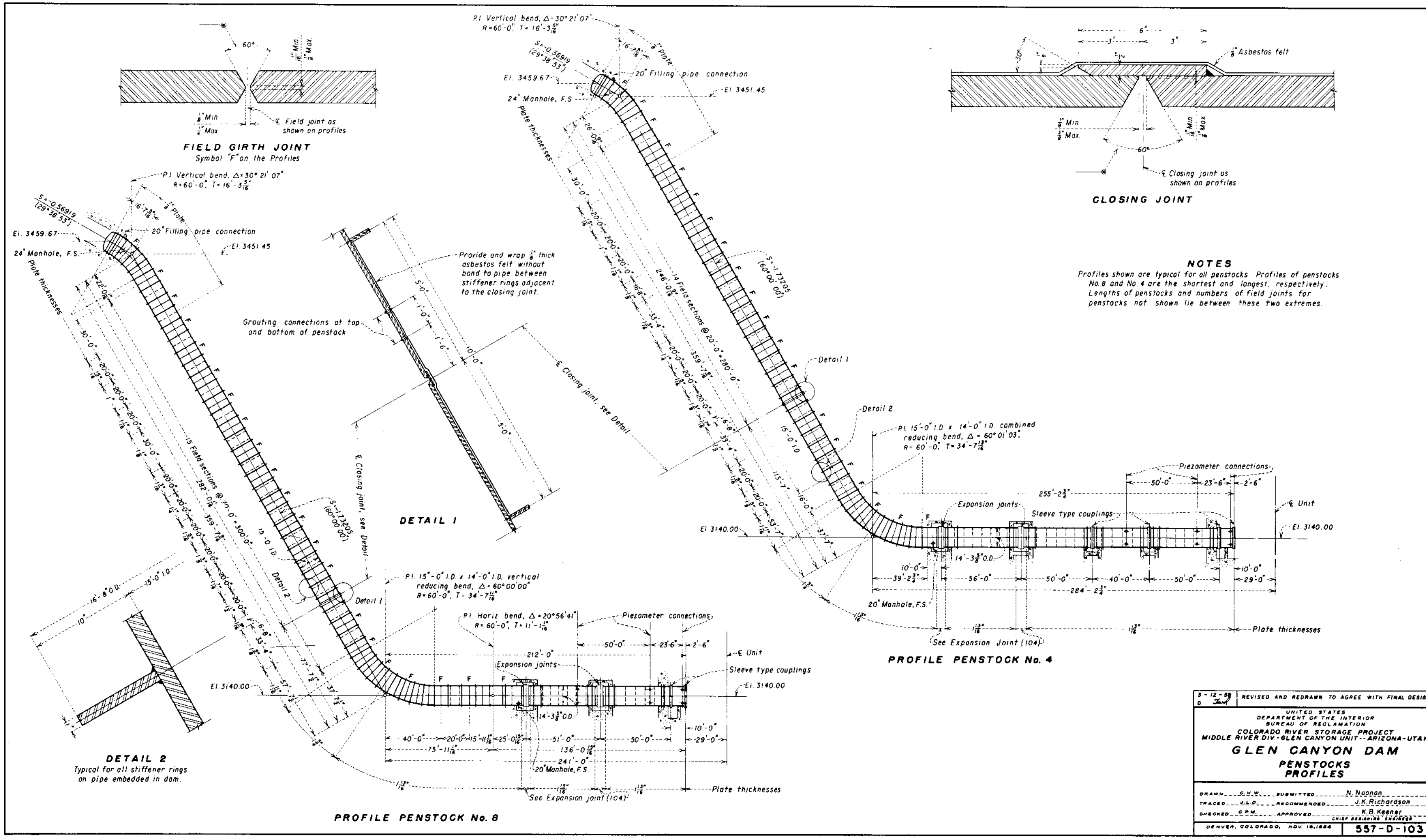


Figure 141.—Penstock profiles.

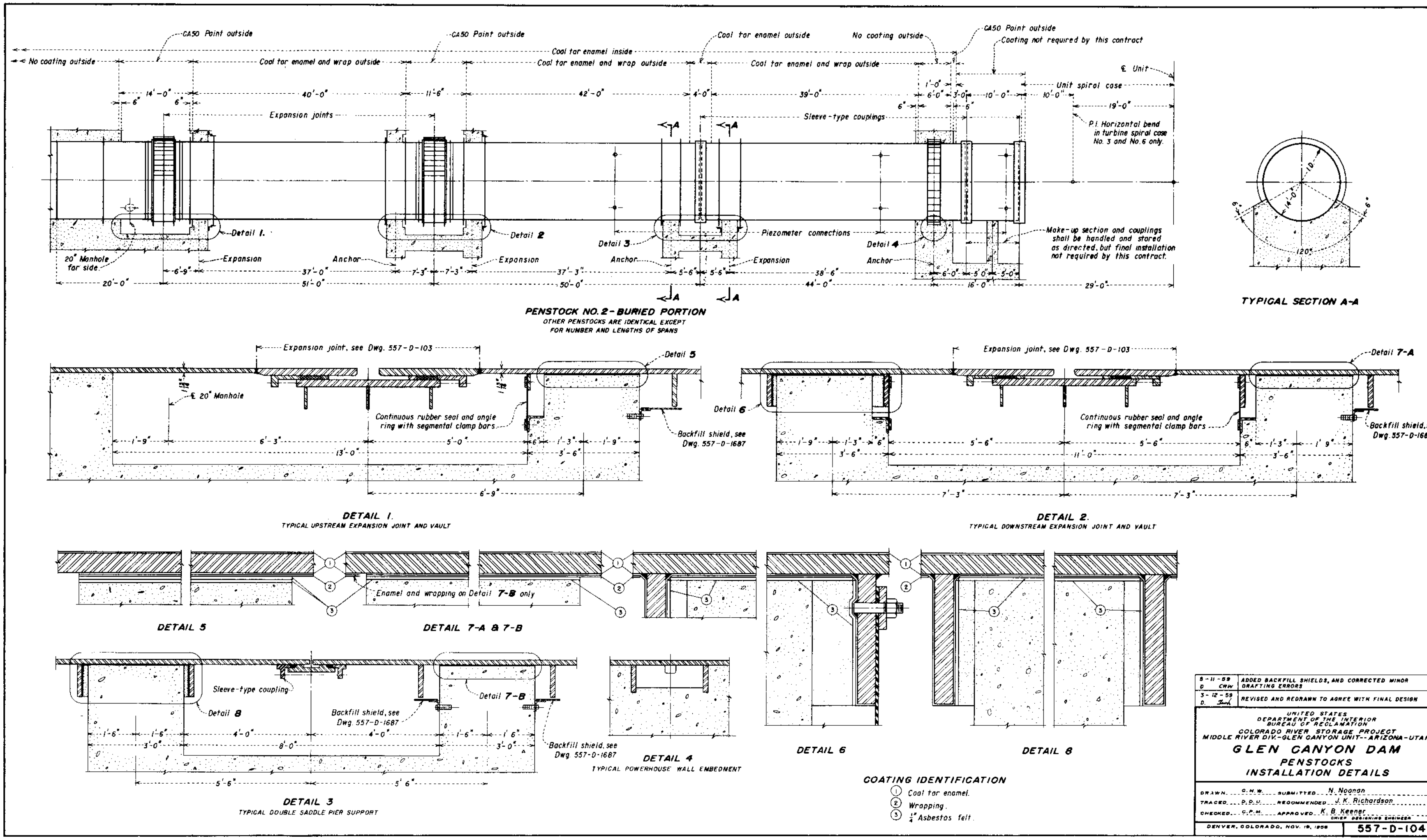


Figure 142.—Penstock installation details.

transitions may be reached by temporary ladders brought inside the penstocks through the manholes and laid on inverts. A 20-inch-inside-diameter manhole is installed in each penstock immediately upstream from the first expansion joint.

Two piezometer orifice stations 50 feet apart are located on each penstock in its level portion near the powerplant. Individual 3/4-inch pipes lead from each piezometer connection into the powerplant.

(c) *Installation and Coating.*—Penstock sections were installed under the prime contract, excepting makeup sections, which were installed under the completion contract, specifications No. DC-5750.

Interior surfaces of penstocks were coated with coal-tar primer and coal-tar enamel. Exterior surfaces in backfill, except the sleeve-type couplings, were coated with coal-tar primer, coal-tar enamel, bonded-asbestos felt, and kraft paper wrap. Exterior surfaces of sleeve-type couplings in backfill were coated with coal-tar primer and coal-tar enamel, and surrounded by sand shields. The penstock exterior embedded surfaces were not coated. Exterior surfaces of penstocks, makeup pieces, and sleeve-type couplings exposed to view in the powerplant were coated with phenolic-resin aluminum paint.

59. PENSTOCK TRASHRACKS. A total of 280 trashracks are provided for the penstock trashrack structures to protect the turbines from oversize trash. The 280 trashrack sections, approximately 9.33 feet wide by 12.5 feet high, as shown on figure 143 were furnished under invitation No. DS-5512.

The trashracks are installed in vertical slots in the penstock trashrack structures. The racks consist of 2- by 5/8-inch trash bars supported by 9- by 1-inch horizontal load carrying plates which transmit the load to the side members. The 4-3/8-inch clear opening between the trash bars conforms with the recommendations of the turbine manufacturer. The estimated weight of the 280 trashracks is 980,000 pounds.

The trashracks were designed for a differential head of 20 feet at a stress of 33,000 pounds per square inch.

60. PENSTOCK STOPLOGS. Twenty-three stoplog sections were required for the penstock intakes to provide a bulkhead for inspection and maintenance of the fixed-wheel gates and appurtenances. One set of stoplogs, sufficient for one unit, is provided for the penstock intakes.

The following log sections were furnished for stacking in accordance with drawing No. 557-D-2080*: 9 lower log sections, approximately 24.5 feet wide by 4.80 feet high, weighing 24,700 pounds each; 7 intermediate log sections, approximately 24.5 feet wide by 5.30 feet high, weighing 25,100 pounds each; and 7 top log sections, approximately 24.5 feet wide by 6.79 feet high, weighing 27,200 pounds each. A lifting frame was also furnished for grappling purposes. All material was furnished under invitation No. DS-5354.

The stoplogs are installed in vertical guides on the face of the dam and in the trashrack structures. The stoplogs are placed in the guides in one stack. Effective sealing is obtained by means of music-note-type rubber seals bolted to the log sections.

The stoplogs were designed for a maximum head of 250 feet at safe stress.

61. 165-TON GANTRY CRANE. (a) *Description.*—A 165-ton gantry crane (fig. 274) is installed on rails on top of the dam for the installation and maintenance of penstock gates and hoists and river outlet gates and stop logs. The crane was manufactured under invitation No. DS-5521 and is an outdoor, cab-operated, electric gantry crane with a trolley supporting a 165-ton main hoist and a 25-ton auxiliary hoist.

The main hoist is reeved 6 parts double with 1-1/2-inch-diameter, 6 by 37, improved plow steel, fiber-core wire rope. The block has a lifting eye in lieu of a hook. The hoist has a lift of 53 feet, is powered by a 40-horsepower, 1,200-r.p.m., direct-current motor, and operates at incremental speeds up to 3.7 feet per minute hoisting and 4.3 feet per minute lowering with a 165-ton load.

The auxiliary hoist is reeved 2 parts double with 1-inch-diameter, 6 by 37, improved plow steel, fiber-core wire rope. The block has a sister-type hook bored for a horizontal lifting pin. The hoist has a lift of 380 feet, is powered by a 40-horsepower, 1,200-r.p.m., direct-current motor and operates at incremental speeds up to 24 feet per minute hoisting and 28 feet per minute lowering with a 25-ton load.

*Not included.

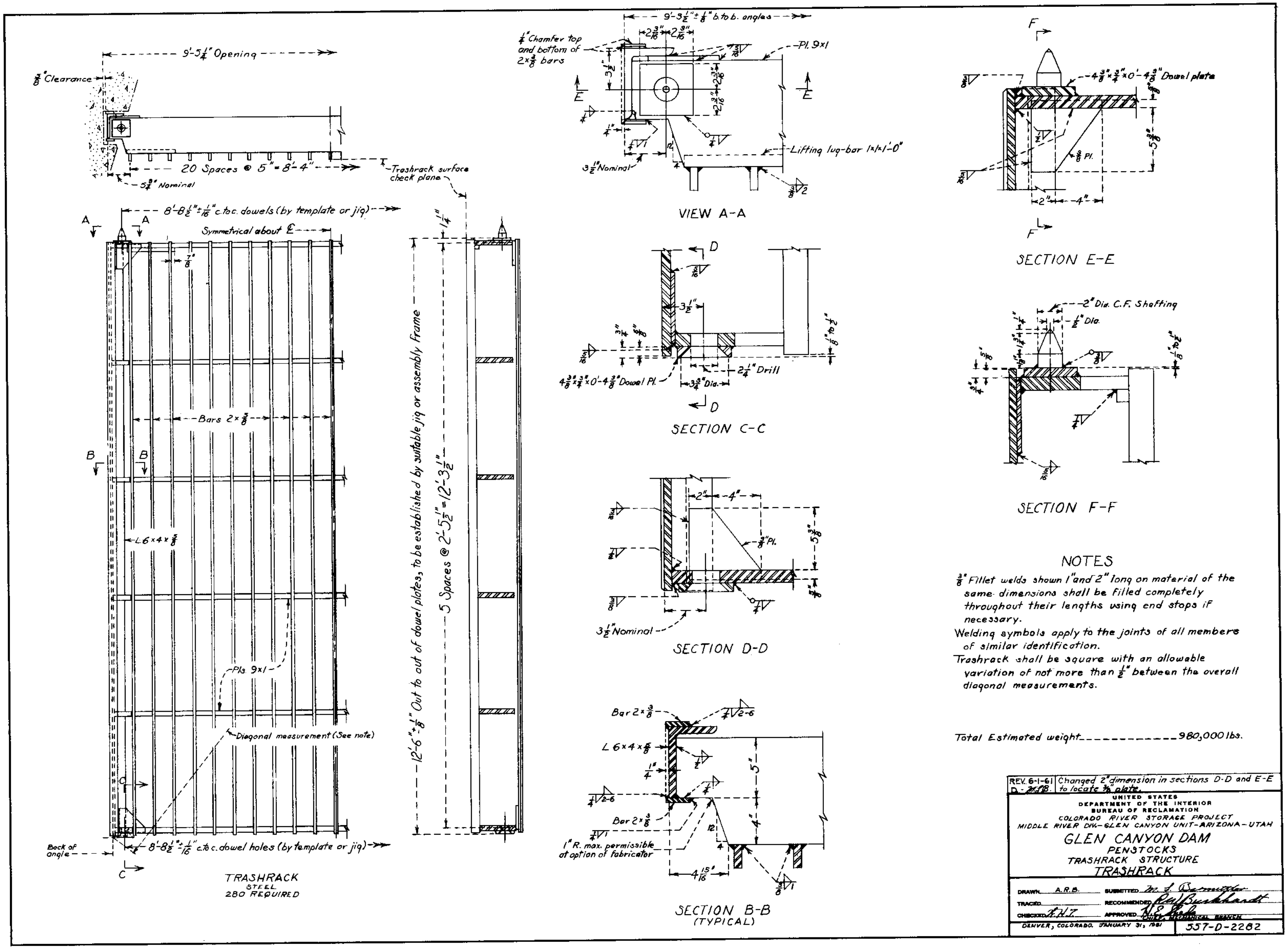


Figure 143.--Penstock trashracks.

The trolley is driven by a 15-horsepower, 1,200-r.p.m., alternating-current motor at 4.8 feet per minute in either direction, through a rack and pinion drive. The gantry is driven by four 5-horsepower, 1,150-r.p.m., alternating-current motors at incremental speeds up to 48 feet per minute in either direction.

A single motor-generator set installed in the control enclosure supplies direct-current power for both hoist motors, the hoist motor controls being electrically interlocked so that only one hoist may be energized at a given time. Full magnetic controllers for each motion of the crane are located in the operator's cab. Five-point-speed control is provided for each direction of the main hoist, auxiliary hoist, and gantry drive, and single-speed control for the trolley drive.

Each hoist motor is equipped with two alternating-current magnetic brakes, one on each side of the motor, which are released when the hoist motor circuit is energized. Each of these brakes is capable of overcoming the full-load torque of the motor with which it operates. The trolley drive is equipped with a similar brake which is capable of overcoming at least three-fourths of the full-load torque of the drive motor.

Each of the gantry drive motors is equipped with a hydraulic brake, all of which are actuated by a single foot pedal located in the operator's cab. Each brake is capable of overcoming the full-load torque of the motor with which it is associated, simultaneously with the other gantry drive brakes, with no more than 50 pounds of operating force on the foot pedal. Each brake is equipped with an automatic, spring-applied, parking feature.

Block-actuated limit switches on each hoist limit the upward travel of the hook. A two-contact-geared limit switch on the auxiliary hoist limits the lower travel of the hook and lights a red light in the operator's cab. Each contact is individually adjustable within the last 30 feet of travel.

A limit switch on the main power cable reel, adjustable within the last 10 feet of cable, stops the gantry before the cable is unwound lighting a green indicating light in the operator's cab and setting the parking brake. A momentary contact pushbutton station, located near the gantry travel master switch, permits the operator to move the gantry back into the limit switch zone.

The crane is equipped with a 120-volt permanent lighting system. Service outlets are located in the operator's cab, in the control room, and on the trolley access walkway. One 100-watt lamp is installed in the operator's cab and one in the control room, each with conveniently located wall switches. Two 100-watt lamps are located on the trolley access walkway. One 750-watt narrow-beam floodlight is mounted under the trolley access walkway. Two 500-watt medium-beam floodlights are mounted on the upstream side of the crane and two 500-watt wide-beam floodlights are mounted to illuminate the roadway under the crane. There is a 240-volt, 3-kilowatt heater installed in the operator's cab.

Spring bumpers are mounted on the gantry legs at one end of the crane only. Double-acting spring bumpers are provided on the trolley. Ladders and walkways provide access to the trolley, control room, and operator's cab.

(b) *Design.*—The crane was designed according to the Navy Department's standard of design for structural steel using a base stress of 16,000 pounds per square inch. Computations of stress took into account wind loads, all dead and live loads, and loads due to acceleration and deceleration. Allowable stresses due to combinations of these stresses were increased according to the Navy standard noted. A factor of safety of not less than five, based on the ultimate strength of the material and rated capacity of the crane, was used in the design of all mechanical parts.

62. 13.96- BY 22.45-FOOT FIXED-WHEEL GATES FOR PENSTOCK INTAKES. (a) *Description.*—Eight fixed-wheel gates are provided in the penstock intakes for emergency closure of the penstocks in case of damage to the penstocks or the powerplant equipment, and also to permit unwatering of the penstocks for inspection and maintenance. Normally, the gates will close only after flow through the penstocks has been stopped by closing the wicket gates. However, in an emergency the gate may be closed with water at full reservoir head flowing through the penstock. The gates were manufactured by Voest Co. of Linz/Donau, Austria, under invitation No. DS-5577. The frames were manufactured by Rockwell Engineering Co. of Blue Island, Ill., under invitation No. DS-5469. The general arrangement and some details of the 13.96- by 22.48-foot fixed-wheel gate installation are shown on figure 144.

Each gate is made in four units, fabricated from welded shapes and plate connected by rivets and rib bolts. Six wheels on each side of the gates carry the waterload to the tracks embedded in the downstream side of the gate slots. Rubber seals are mounted like a

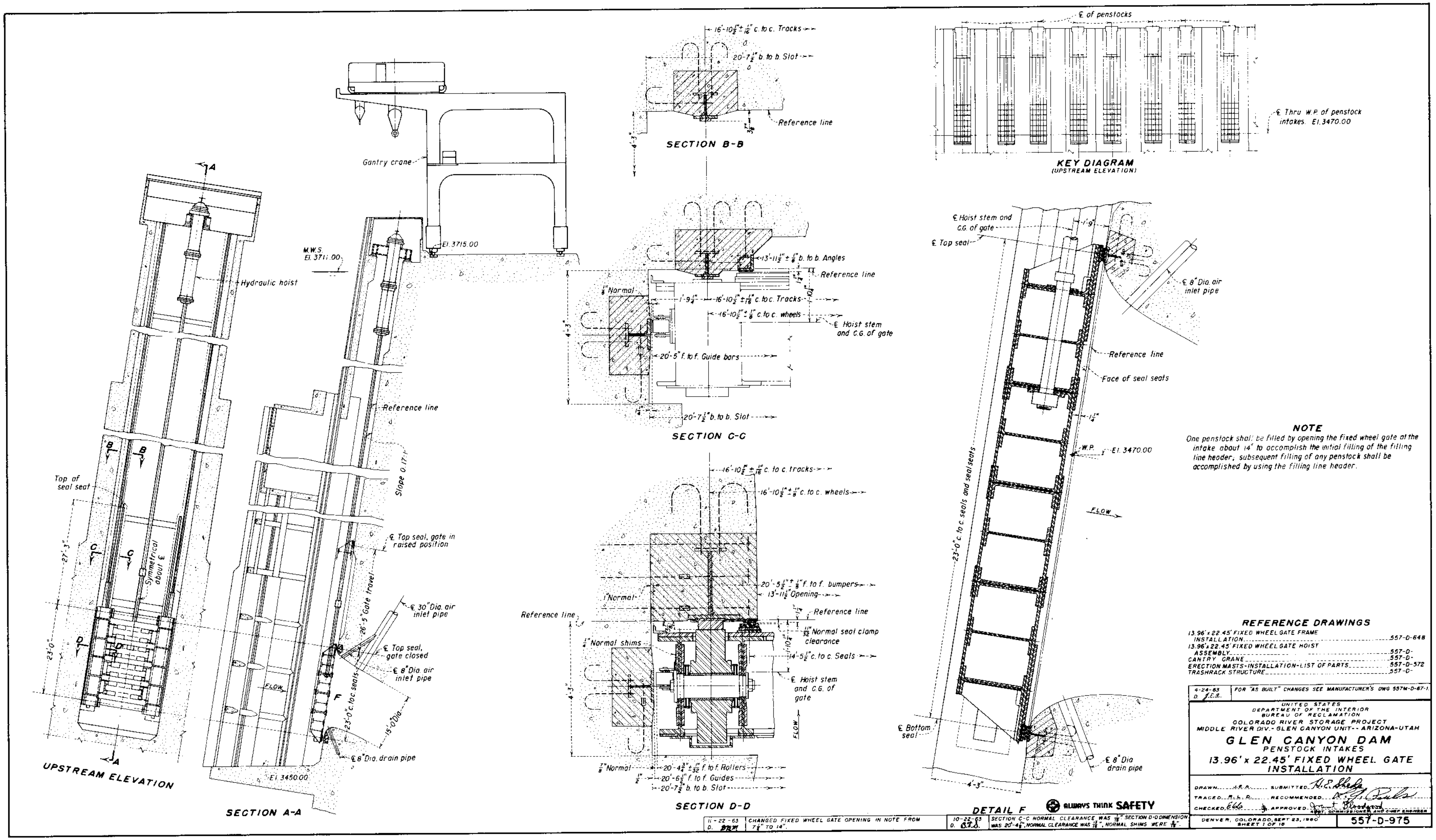


Figure 144.—13.96- by 22.45-foot fixed-wheel gate installation for penstock intakes.

picture frame on the downstream skinplate and bear against metal seats which surround the penstock intakes. Rollers at the sides of the gate provide lateral guidance for the gate and bear on embedded guides in the gate slot. A single hoist connection, located on the center of gravity, is provided. The gate travel is 26 feet 5 inches. The estimated weight of each gate is 231,000 pounds and of each frame is 70,000 pounds.

(b) *Design.*—Each gate was designed for a static head of 270 feet. The hoist connection at the gate was designed for the maximum hoist capacity.

Bending stresses in the horizontal and vertical beams were limited to 15,000 pounds per square inch tension and 14,000 pounds per square inch compression with the web shear limited to 9,500 pounds per square inch. Equivalent stress in the skinplate due to beam bending and skinplate bending was limited to 20,000 pounds per square inch. A portion of the skinplate was considered to be an integral part of the downstream beam flanges. The skinplate was also assumed to act as a haunched continuous beam with the supports on the centerline of the webs.

The stresses in the tracks and wheels were investigated by a method of calculating stresses due to the pressure of one elastic solid on another. The bushings in the wheels have self-lubricating inserts and may be greased when assembled. Friction forces of the loaded wheels, busings, seals, etc., were investigated to be certain that the gate would have sufficient weight to close without the aid of an externally applied force. Bearing pressure on the projected area of the bushing was limited to 4,000 pounds per square inch.

Figure 145 shows the downstream face of the gate as fabricated in the manufacturer's shop.

63. HOISTS AND CONTROLS FOR THE 13.96- BY 22.45-FOOT FIXED-WHEEL GATES. (a) *Description.*—A hydraulic hoist and control are provided to operate each of the eight 13.96- by 22.45-foot fixed-wheel gates. Normally, a hoist will be required to close the gate under balanced pressure to unwater the penstock for maintenance purposes, but may be required to close the gate under flow conditions in an emergency. Eight hydraulic hoists were furnished by the Pacific Coast Engineering Co., Alameda, Calif., under invitation No. DS-5601. The controls were manufactured by Auto-Control Laboratories, Inc., Los Angeles, Calif., under invitation No. (D) 90,638-A.

Figure 145.—Downstream elevation of shop assembly of 13.96- by 22.45-foot fixed-wheel penstock gate. P557-D-46362NA.

Some installation and assembly details are shown on figure 146. The hydraulic hoists are installed on the slope of the intake structure (0.170:1) at the upstream face of the dam and are supported by steel beams at elevation 3713.00. Each hoist is connected to the gate by a series of intermediate stems with an assembled length of 172 feet 9-3/4 inches. Each hoist piston has both packing and piston rings, is 30 inches in diameter, and has a stroke of 26 feet 5 inches. An individual control cabinet is located adjacent to each hoist in the control house, and contains all the hydraulic and electrical equipment for operating the hoist, and a gate position indicator.

The estimated weight of the eight complete hoists is 800,000 pounds and of the eight complete control systems is 42,000 pounds. Figures 147 and 148 show the shop assembly of the hoist and stems.

(b) *Design.*—The hoists were designed to open and close the gates normally with the water pressure balanced on both sides of the gates; to open any one of

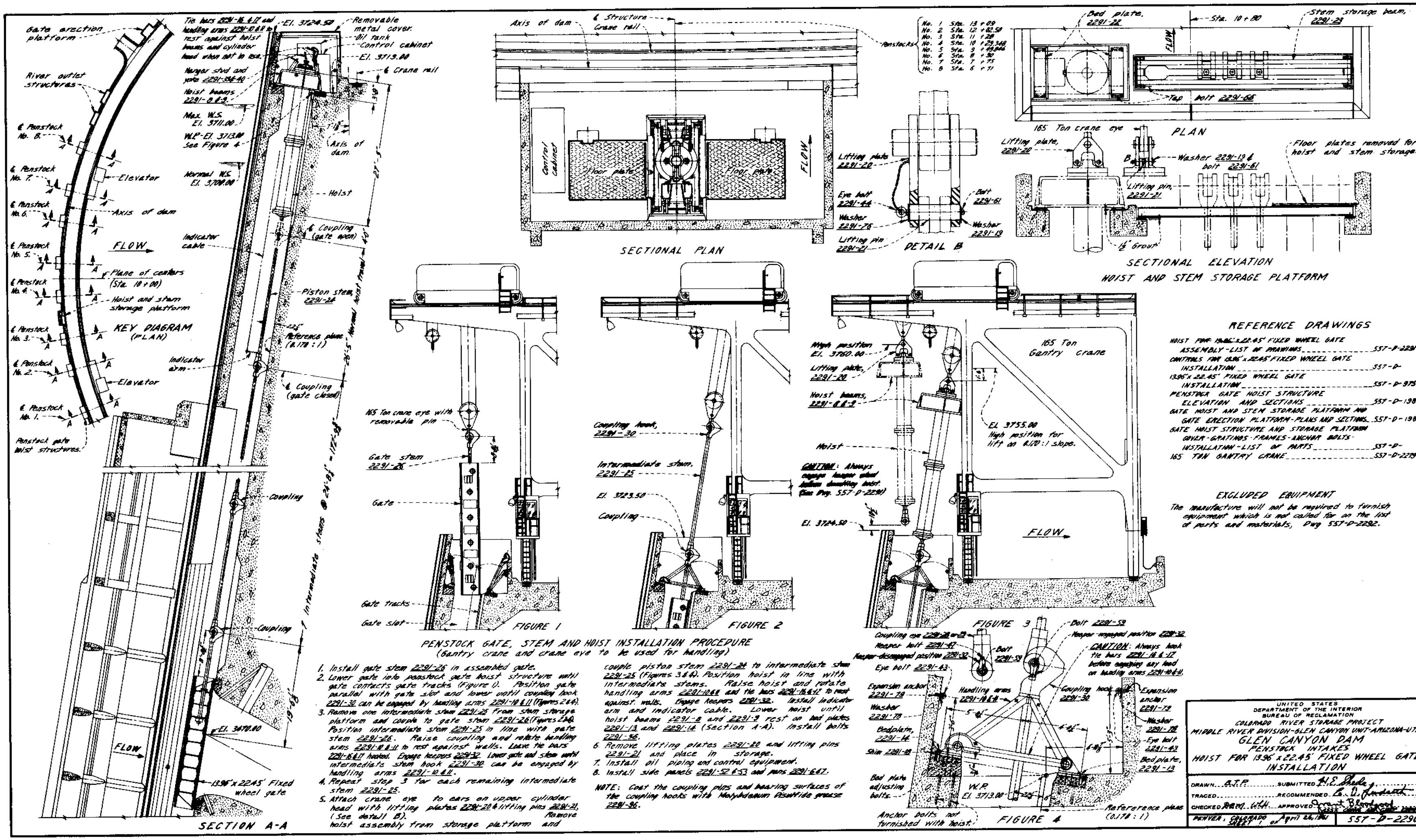


Figure 146.—Installation of hoist for 13.96- by 22.45-foot fixed-wheel gate—Penstock intakes.

Figure 147.—Intermediate stems and coupling hooks for the 13.96- by 22.45-foot fixed-wheel gate. P557-D-37604.

the gates 14 inches for conduit filling with maximum unbalanced pressures on the gate; and in an emergency, to close the gates with full flow to the turbines under a maximum head of 255 feet. The cylinder size for the hoists was determined by the sum of the weights of the gate and stems, plus maximum downpull, minus friction of the coupling hooks sliding on the concrete guide surface, and minus one-half the gate wheel and seal friction. This load would produce a cylinder pressure of approximately 1,200 pounds per square inch during emergency closure; however, normal balanced operations or conduit filling will require a maximum cylinder pressure of less than 600 pounds per square inch. The hoists and stems were designed for a relief valve setting of 1,000 pounds per square inch, using normal factors of safety. The pressure switch setting of 750 pounds per square inch provides a minimum lifting force at the hoists of 437,000 pounds which exceeds the 392,000-pound force required to open the gate for conduit filling. The hoists were designed to lower the gates, without operating the pumps, by bypassing oil from the underside of the piston directly to the top of the cylinder. The design time selected for opening the gates balanced was about 20 minutes, and for closing unbalanced less than 2 minutes, based on normal oil temperatures. The closing time will increase slightly as the oil temperature drops, and decrease slightly when the gate is subjected to downpull forces during emergency closure. A hanger stud is provided in the upper cylinder head of each hoist to engage the piston stem and support the weight of the stems and gate in the open position during gate installation or removal. During normal operation the hanger stud will not be engaged, as the piston was designed to support the gate and stems on oil confined under the piston.

Two constant-delivery pumps, having a combined delivery of 49 gallons per minute at a pressure of 1,000 pounds per square inch and a pump speed of 1,800

Figure 148.—Hydraulic hoist and intermediate stems for the 13.96- by 22.45-foot fixed-wheel gate. P557-D-37603.

revolutions per minute were selected for each cabinet to open the gate within the predetermined design time. Two 15-horsepower motors, using 440-volt, 3-phase, 60-cycle current were required to drive the pumps. The two independent pump units were used to provide protection against failure of a single pump unit. A 110-volt transformer was used to provide power for most of the control circuits and the cabinet heater; however, the control was designed to use a separate source of direct-current power from the powerplant to operate the emergency close circuit and the solenoid-operated, four-way valve which directs the flow of oil to open, close, or stop the gate.

The control was designed for gravity closure of the gate with the closing speed controlled by a throttle valve in the bypass piping from the bottom to the top of the cylinder. A restoring cycle was incorporated in the controls to maintain the gate automatically within close limits of the open position. A penstock pressure switch was included to prevent opening the gate until the penstock is full and the pressures on either side of the gate are essentially balanced. Gate position indication was provided in the cabinet using a vertical scale with a pointer driven by a stainless steel cable connected to the piston stem.

The control was designed for normal gate operation by pushbutton from the control cabinet and for emergency gate closure by selector switch or automatic protective devices from the powerplant. The emergency close circuit was designed to override all opposing signals initiated from the control cabinet.

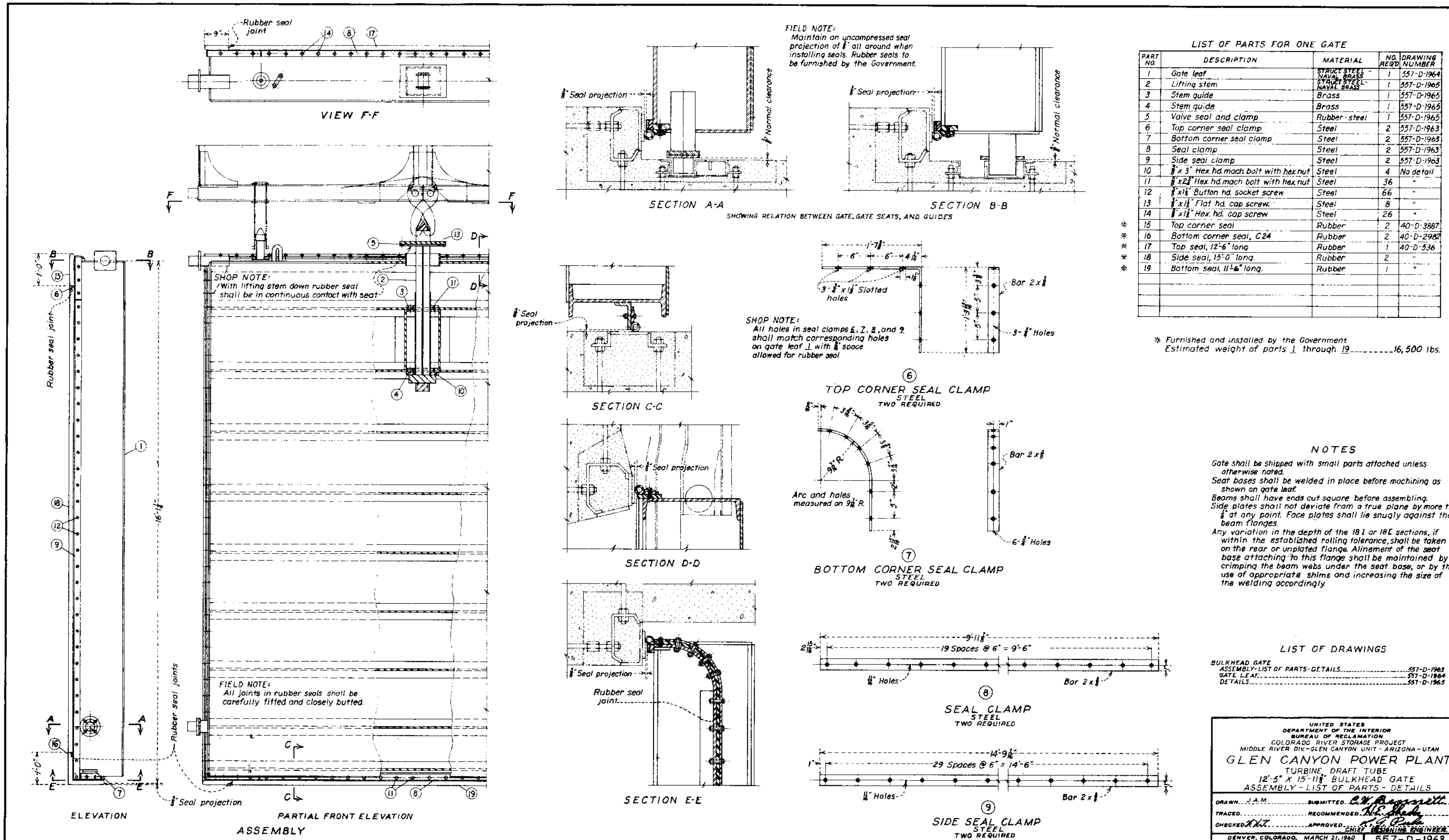


Figure 149.—Turbine draft tube 12.42- by 15.95-foot bulkhead gate assembly.

The 214-gallon oil tank was selected to provide storage and expansion capacity for the hydraulic oil used in the system. The control design was based on a nominal temperature of 35° F. and the use of a lightweight hydraulic oil with a viscosity of 153 Saybolt seconds universal at 100° F. and a viscosity index of 97.

The design stresses used were as follows:

(1) *Tension.*—The allowable design stresses in tension for the following materials were based on the yield point or the ultimate strength of the material. The smaller of the tabulated values was used in each instance.

Material	Type	Percent of yield point	Percent of ultimate tensile strength
Steel	Rolled or forged	40	25
Bolt steel	Rolled or forged	25	16.5
Cast steel	Castings	33	20
Brass or bronze	Rolled or cast	33	16.5

(2) *Compression.*—The allowable design stress in compression used for the materials listed above were the same as for tension.

(3) *Shear.*—The allowable design stresses in shear were not more than 0.6 of the allowable design stresses in tension.

(4) *Structural steel.*—The allowable design stresses for structural steel in tension or compression were not more than 20,000 pounds per square inch. In general, structural steel stresses were based on the American Institute of Steel Construction "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings," 1961.

(5) *Hoist cylinders.*—The allowable design stresses for the hoist cylinders were based on the recommendations of the ASME Boiler and Pressure Vessel Code—Unfired Pressure Vessels—Section VIII, 1962.

64. TURBINE DRAFT TUBE BULKHEAD GATES. (a) *Description.*—Twenty-four bulkhead are provided to seal the turbine draft tubes from the tailwater in the event that it is necessary to unwater the turbine units for inspection and maintenance of either equipment or structure. The 24 bulkhead gates and related equipment (fig. 149) were furnished under invitation No. DS-5341.

A gate having nominal dimensions of 12.42 feet wide by 15.95 feet high is available for each of the draft tube openings. The gates and lifting frame operate in steel guides embedded vertically in the concrete structure. The gates are raised and lowered by means of a 10-ton gantry crane. An automatically engaging lifting frame is used for raising and lowering the gates.

The gates are normally stored in the upper portion of the gate slots. The gates are supported by latches so located that the lifting lug on the gate is just below the deck slot covers and is therefore easily accessible from the deck. The latches consist of hinged bars which will automatically engage the gate guide lugs during the raising cycle, and may be laid back out of the path of the gate guide lugs to allow the gate to lower.

Inasmuch as the gates must be raised and lowered only under balanced head conditions, a filling valve is incorporated in the gate lifting stem of each gate for the purpose of creating the balanced head prior to raising the gates. In the event that high-pressure penstock water is unintentionally admitted to the turbine and draft tube while the gates are in place, a gate will blow off of its seat, shearing the bolts which fasten the lower guide lugs to each side of the gate. The top guide lugs are rigidly fixed to the gate, enabling the gate to swing out in the event that the lower guide lugs shear. The estimated weight of the 24 gates is 396,000 pounds.

(b) *Design.*—Maximum stresses are calculated on the basis of a dry draft tube and maximum tailwater surface, the maximum head being 78 feet. Under these conditions the combined stresses of the faceplate and beams are permitted to reach 24,000 pounds per square inch. Other stresses conform to the American Institute of Steel Construction.

CHAPTER VIII. *Design*—POWERPLANT

65. DESCRIPTION. Glen Canyon Powerplant, which extends from wall to wall of the canyon and is located 400 feet downstream from the axis of the dam, is of the indoor type with a structural steel superstructure enclosed with concrete curtain walls (figs. 4, 150, and 151). The intermediate structure and substructure are reinforced concrete. The powerplant consists of a machine shop bay, a service bay, a control area and eight generating unit bays. Each unit bay contains a 125,000-kv.-a., 90-percent power factor, 13,800-volt, 150-r.p.m., 60-cycle, vertical-shaft, hydraulic-turbine-driven, alternating-current generator. Each generator is driven by a 155,500-horsepower hydraulic turbine operating at 150 revolutions per minute under an effective head of 450 feet. There are nine 345-kilovolt, single-phase, 100,000-kv.a. transformers and three 230-kilovolt, single-phase, 100,000-kv.-a. transformers located on a transformer deck downstream of the generating units. Each pair of generators is electrically paralleled, at generator voltage, and connected to one bank of three single-phase transformers, by means of an isolated phase bus, through generator-voltage circuit breakers.

Two 300-ton overhead traveling cranes are required to handle the rotor and other heavy pieces of equipment in the generating units and service bay. A 75-ton crane is provided in the machine shop.

The powerplant was designed to be built in two stages; namely, the first stage or prime construction work and the second stage or completion work.

66. POWERPLANT ARRANGEMENT. Figures 150 through 161 show the final powerplant arrangement as constructed. The powerplant is laid out in an L shape, with the longitudinal centerline of the units perpendicular to the canyon walls. Unit bay 1 is located against the west or right canyon wall. The units are numbered 1 through 8 toward the east or left canyon wall. The service bay is adjacent to unit bay 8 and extends to the left canyon wall. The machine shop bay is also against the left canyon wall and is adjacent to and downstream of the service bay. A control area is situated on top of the superstructure of unit bays 1 and 2. Access to the control area is by means of a stair and two elevators in the cable and elevator tower located on the right side of unit 1. Downstream of the machine shop bay is a large parking area and river outlet valve structure. Vehicular access to the powerplant is provided by taking the 20-foot-wide service road and tunnel located in the left canyon wall. The tunnel terminates at the powerplant parking area at elevation 3188.5.

There are two covered walkways from the dam to the powerplant. The right walkway is a structural steel bridge extending from a balcony on unit 1 to block 18 of the dam. The left abutment walkway is placed on the outlet pipes anchor block at elevation 3188.50 and connects block 6 in the dam with the powerplant at unit 8.

(a) *Typical Generating Unit Bay.*—Units 3 through 8 are shown on figure 150, while units 1 and 2 are shown on figures 160 and 161, respectively. A typical unit bay has a superstructure 65 feet wide in the longitudinal direction and 81 feet deep in the transverse direction (upstream-downstream direction). The generator floor and electrical galleries are at elevation 3168.50. A visitors' walkway or balcony is provided at elevation 3188.50 inside the superstructure wall on the downstream side of the units. A transformer and transfer deck is located at the same elevation outside the superstructure on the downstream side of the plant. The turbine gallery, governor gallery, and service gallery are at elevation 3153.50. The pipe gallery is located downstream at elevation 3138.50. The penstock gallery, unwatering gallery, and inspection galleries are at elevation 3124.75. The centerline of the distributor is at elevation 3140.00 and the invert of the draft tube is at elevation 3102.00. Units 2 through 8 are set on mass concrete.

The configuration of unit 1 is quite different from a typical unit bay. A cable and elevator tower with a large air-conditioned lobby and a curved balcony for visitors is located on the right side of unit 1. Downstream of the tower, a sewage treatment structure is placed on top of the substructure concrete and on the right side of the transformer deck. The base of the entire unit 1 structure sets on rock instead of mass concrete.

Unit 8 also varies from the typical unit bay. The visitors' balcony at elevation 3188.50 goes around the unit on the service bay side and is supported by columns. A concrete stairway is located on the upstream wall of unit 8 to provide access from the covered walkway down to the landscaped area at elevation 3157.0.

(b) *Control Area.*—The control area (figs. 156, 158, and 161) is a two-floor structure designed to allow adequate space for offices, visitor facilities, fallout protection, and the necessary control equipment for the powerplant. The lowest floor is at elevation 3244.24 and provides space for the control

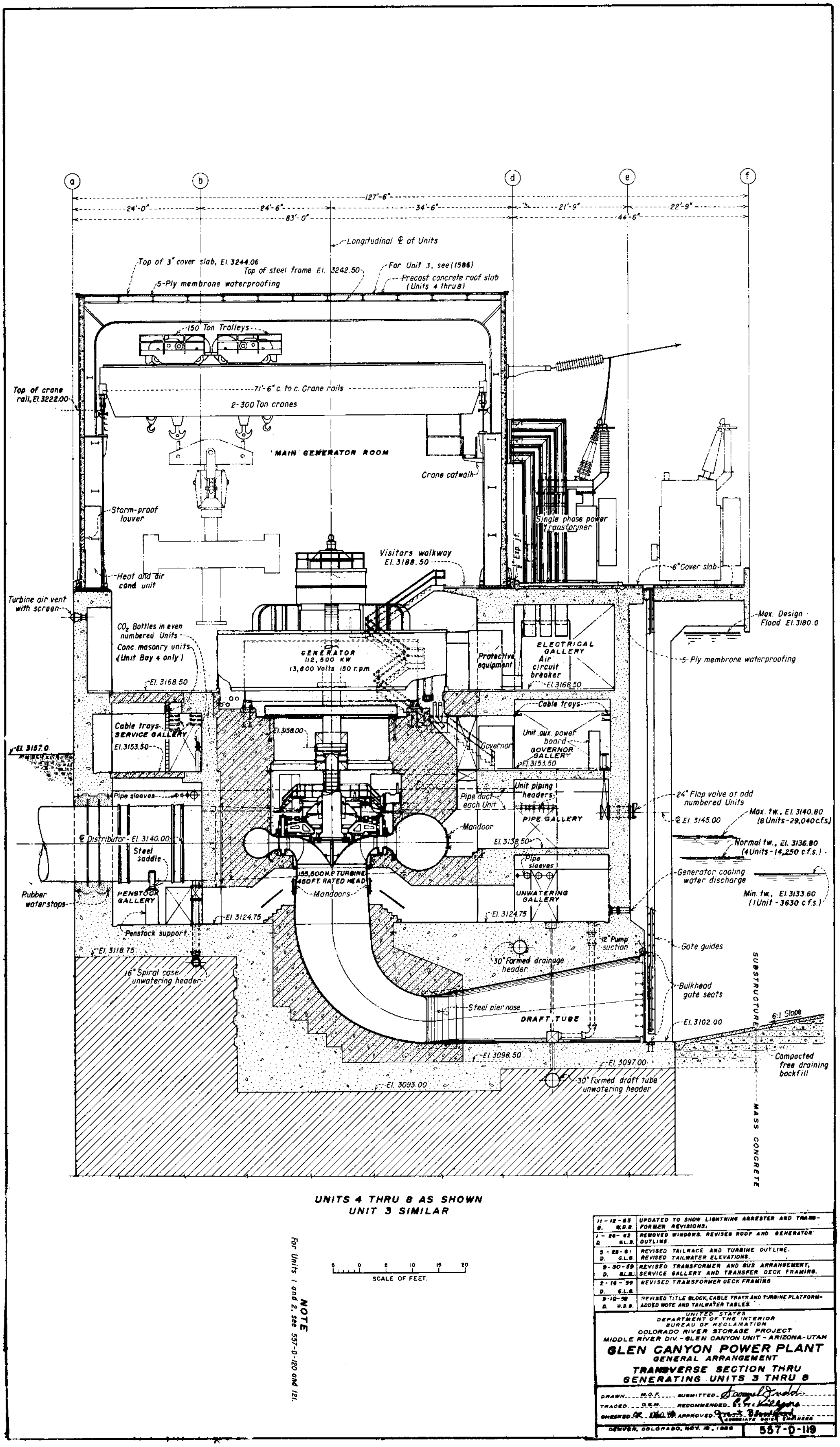


Figure 150.—Transverse section through generating units 3 through 8.

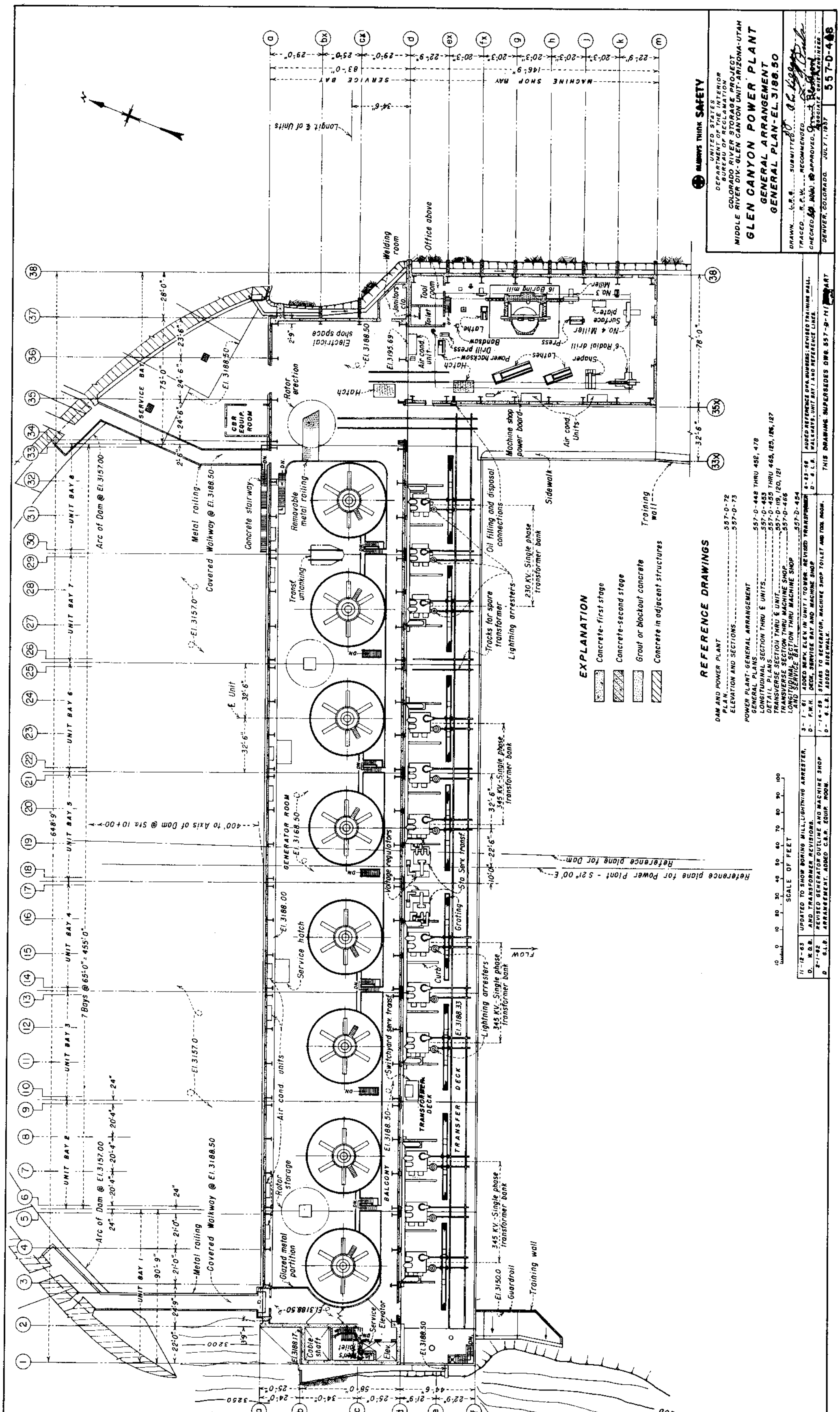


Figure 151.—Powerplant general arrangement—Plan, elevation 3188.50.

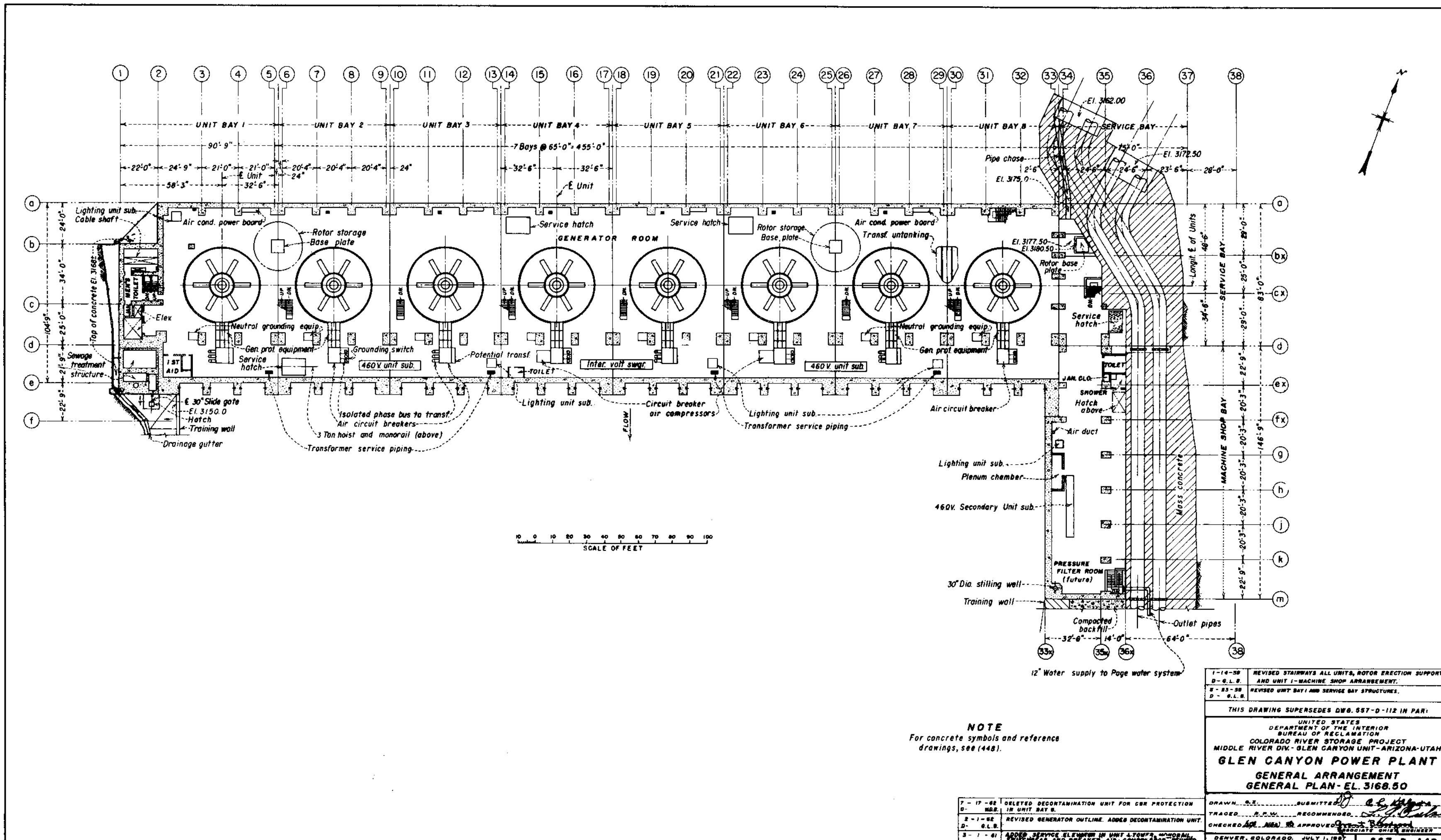


Figure 152.—Powerplant general arrangement—Plan, elevation 3168.50.

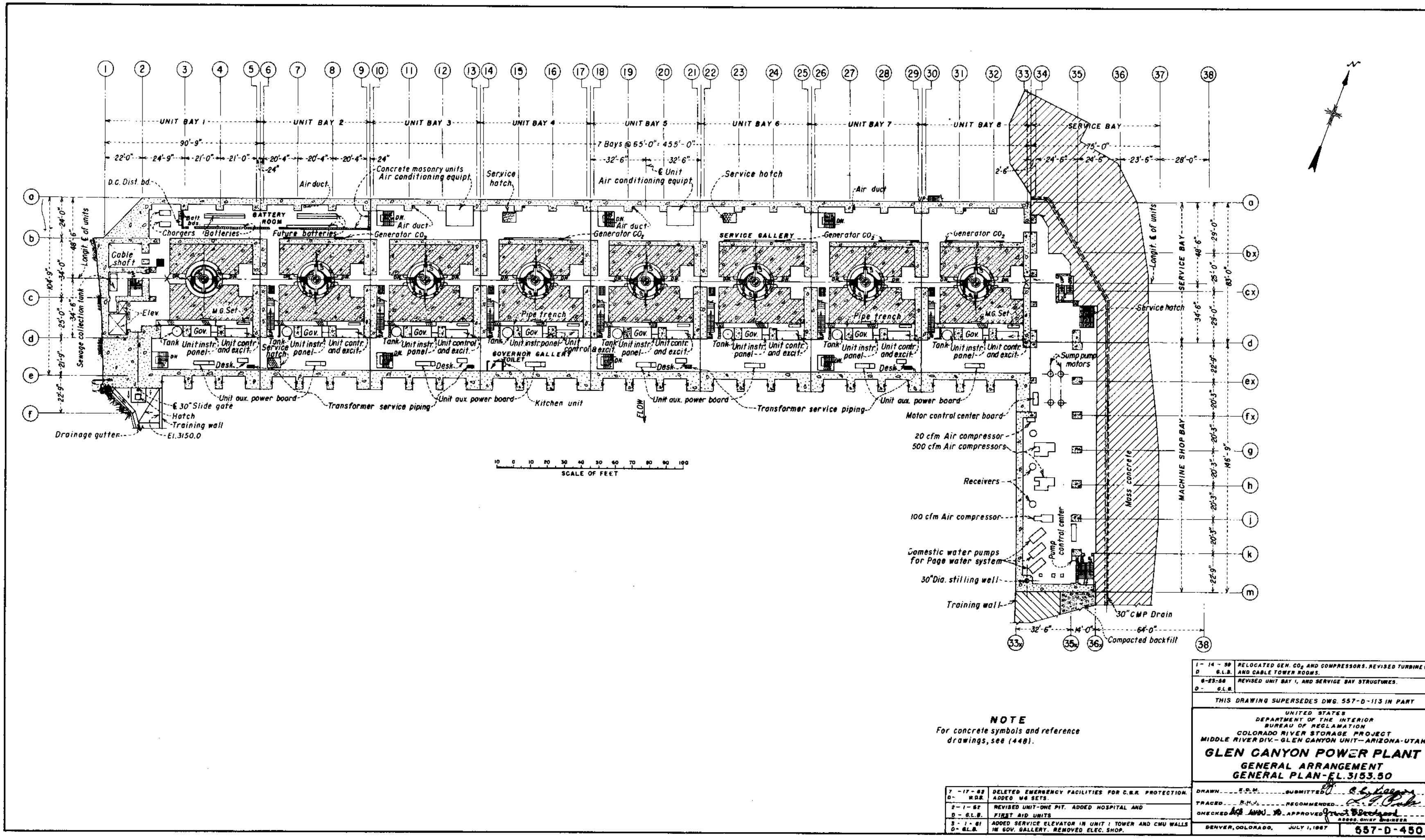


Figure 153.—Powerplant general arrangement—Plan, elevation 3153.50.

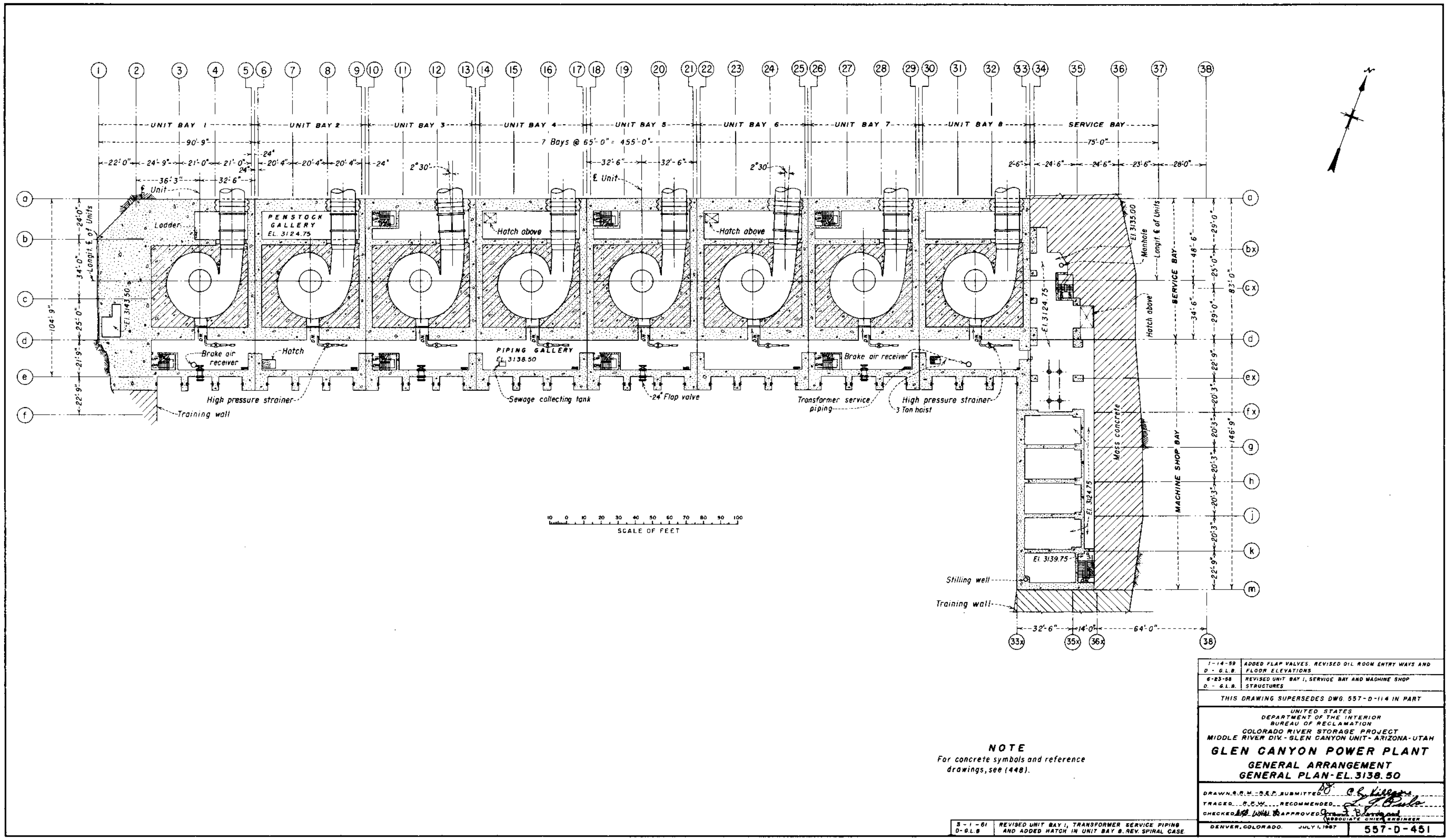


Figure 154.—Powerplant general arrangement—Plan, elevation 3138.50.

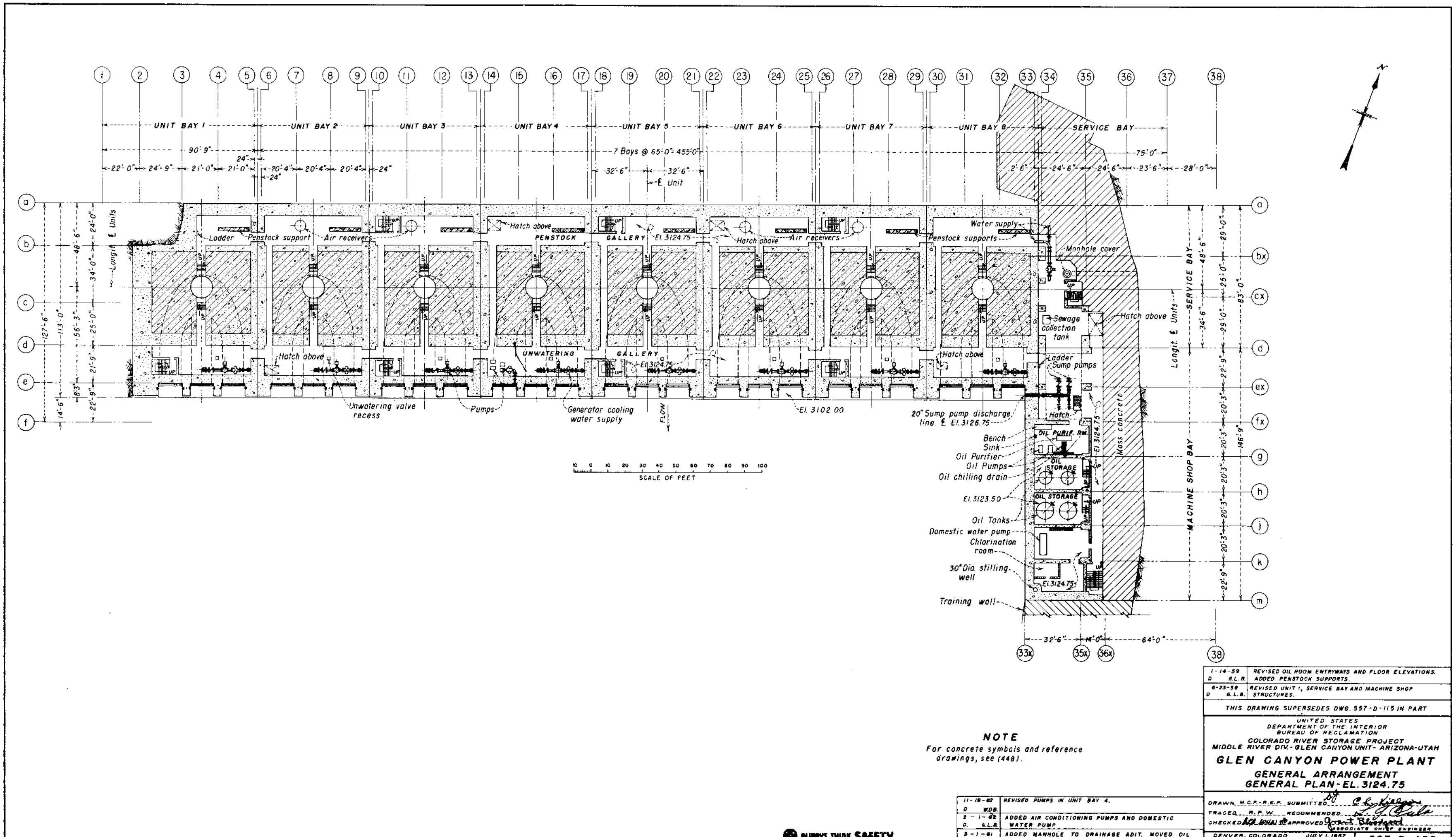


Figure 155.–Powerplant general arrangement–Plan, elevation 3124.75.

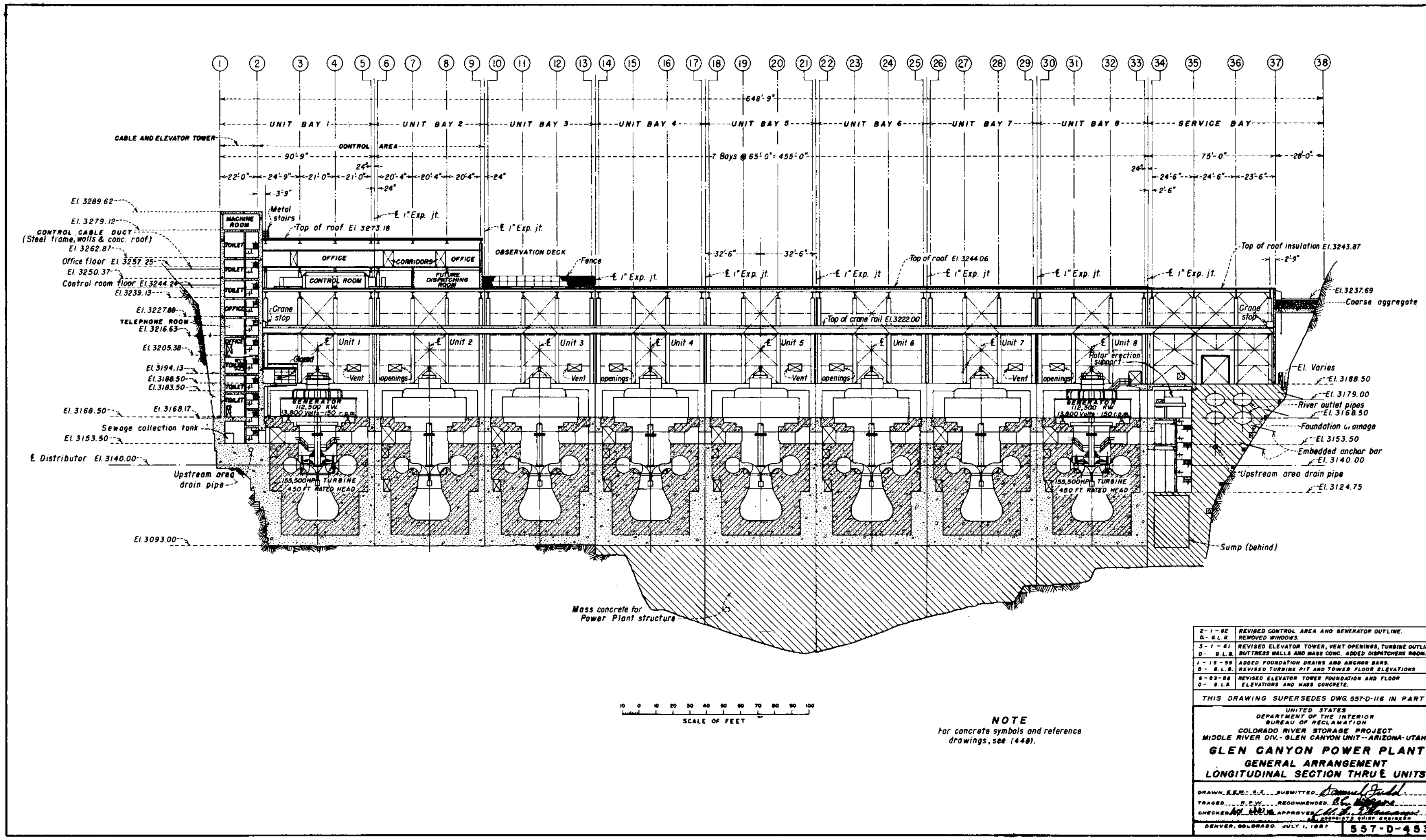


Figure 156.—Powerplant general arrangement—Longitudinal section through centerline of units.

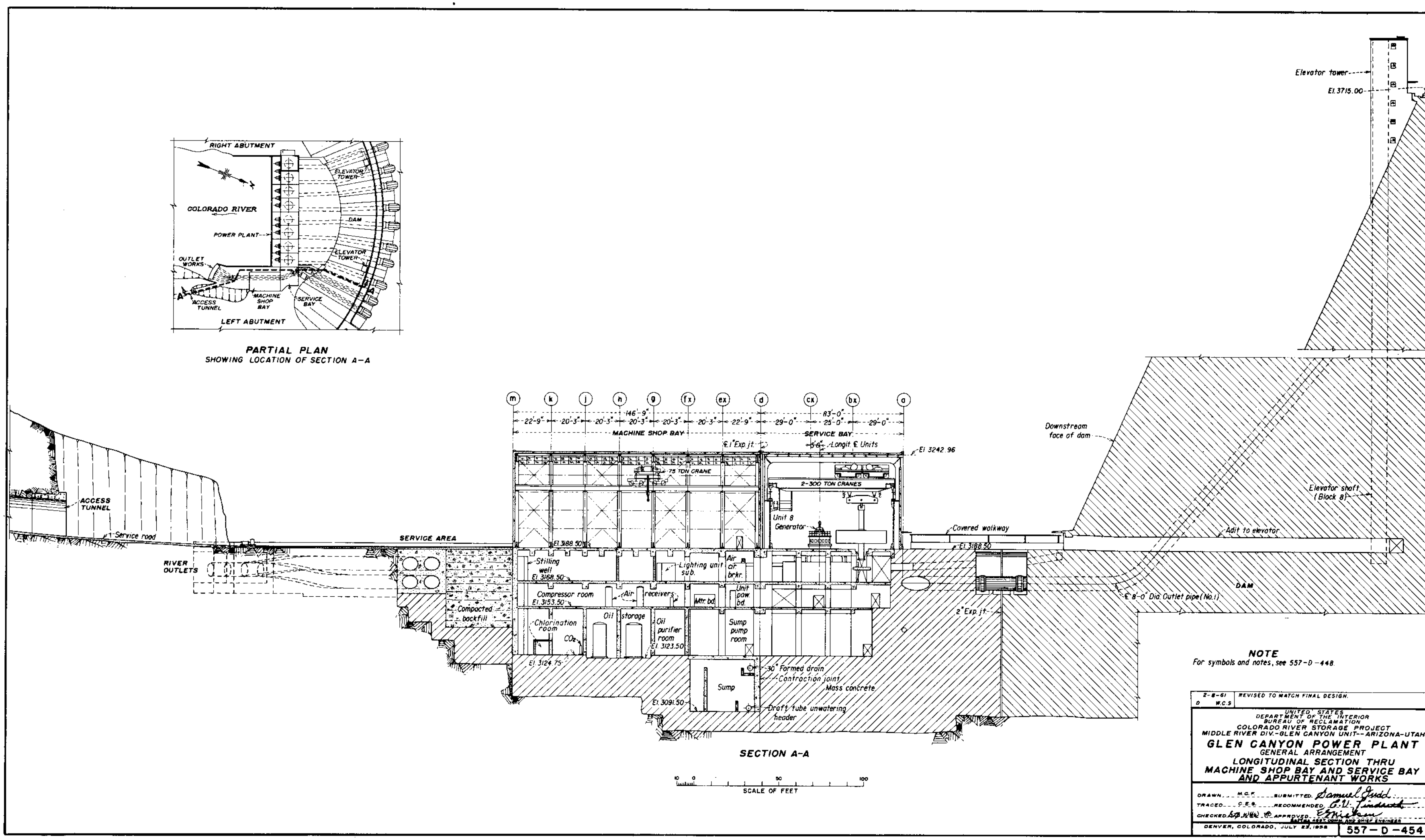


Figure 157.—Powerplant general arrangement—Longitudinal section through machine shop bay and service bay and appurtenant works.

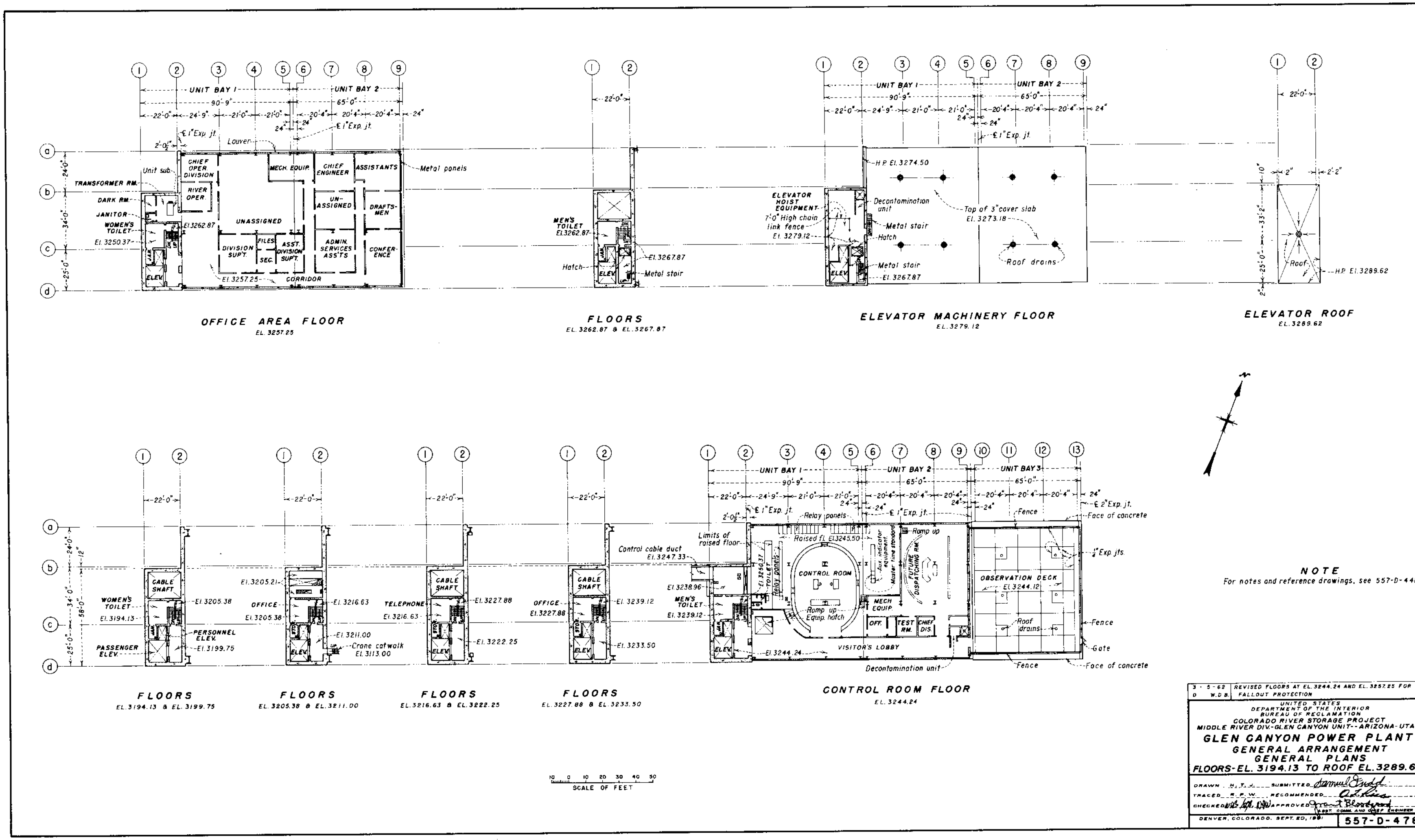


Figure 158.—Powerplant general arrangement—Plans, floor elevation 3194.13 to roof elevation 3289.62.

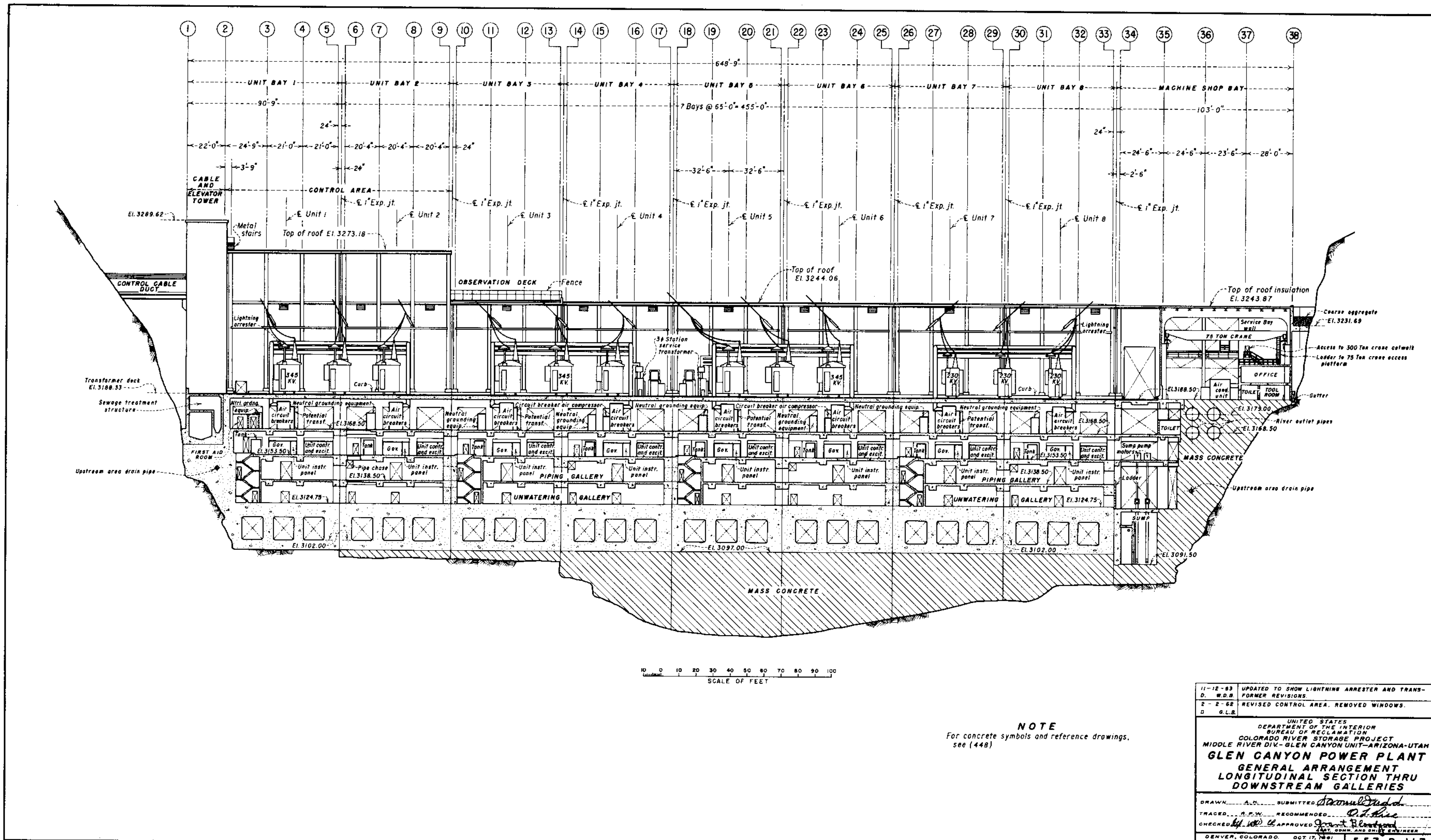


Figure 159.—Powerplant general arrangement—Longitudinal section through downstream galleries.

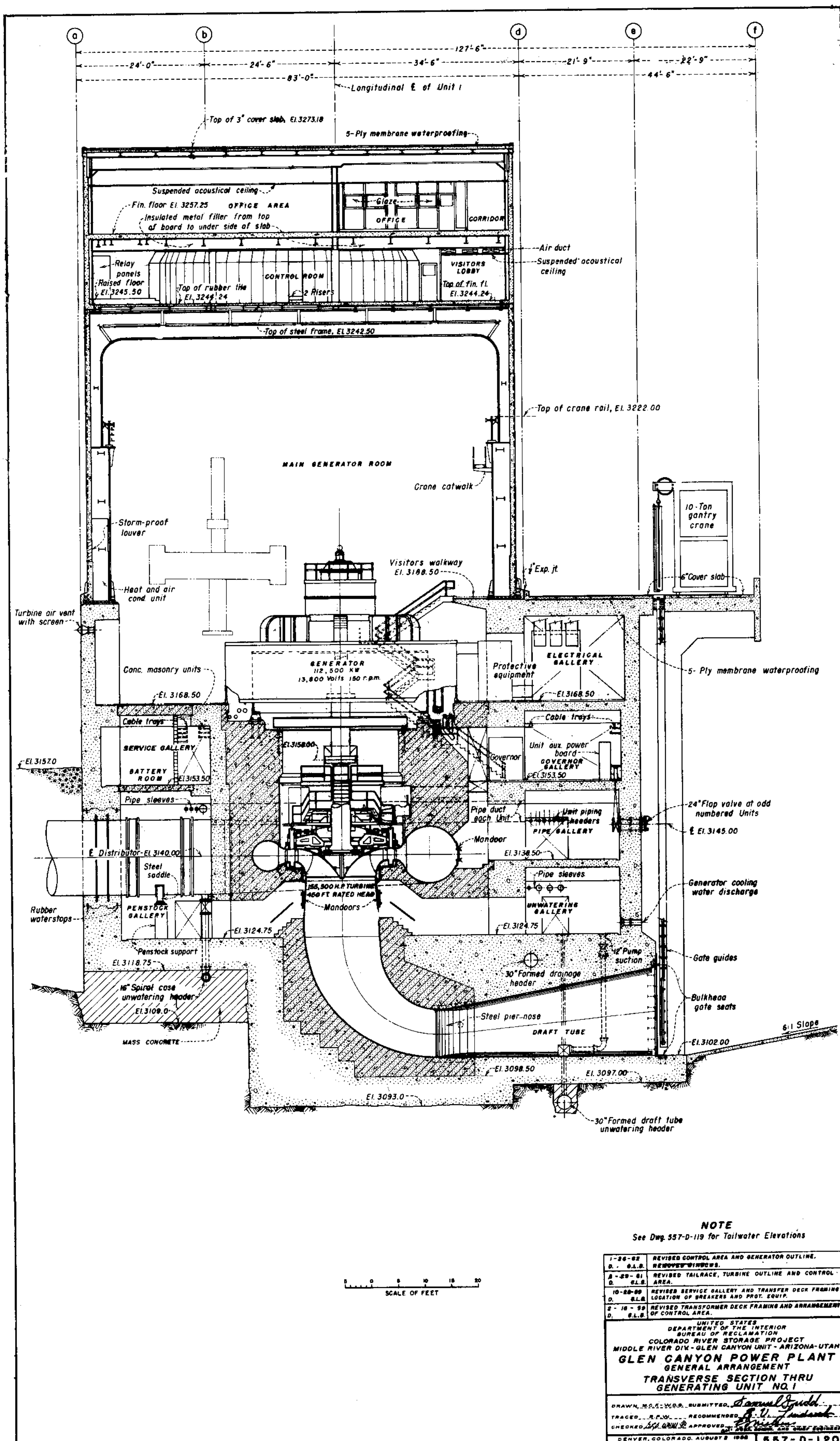


Figure 160.—Powerplant general arrangement—Transverse section through generating unit 1.

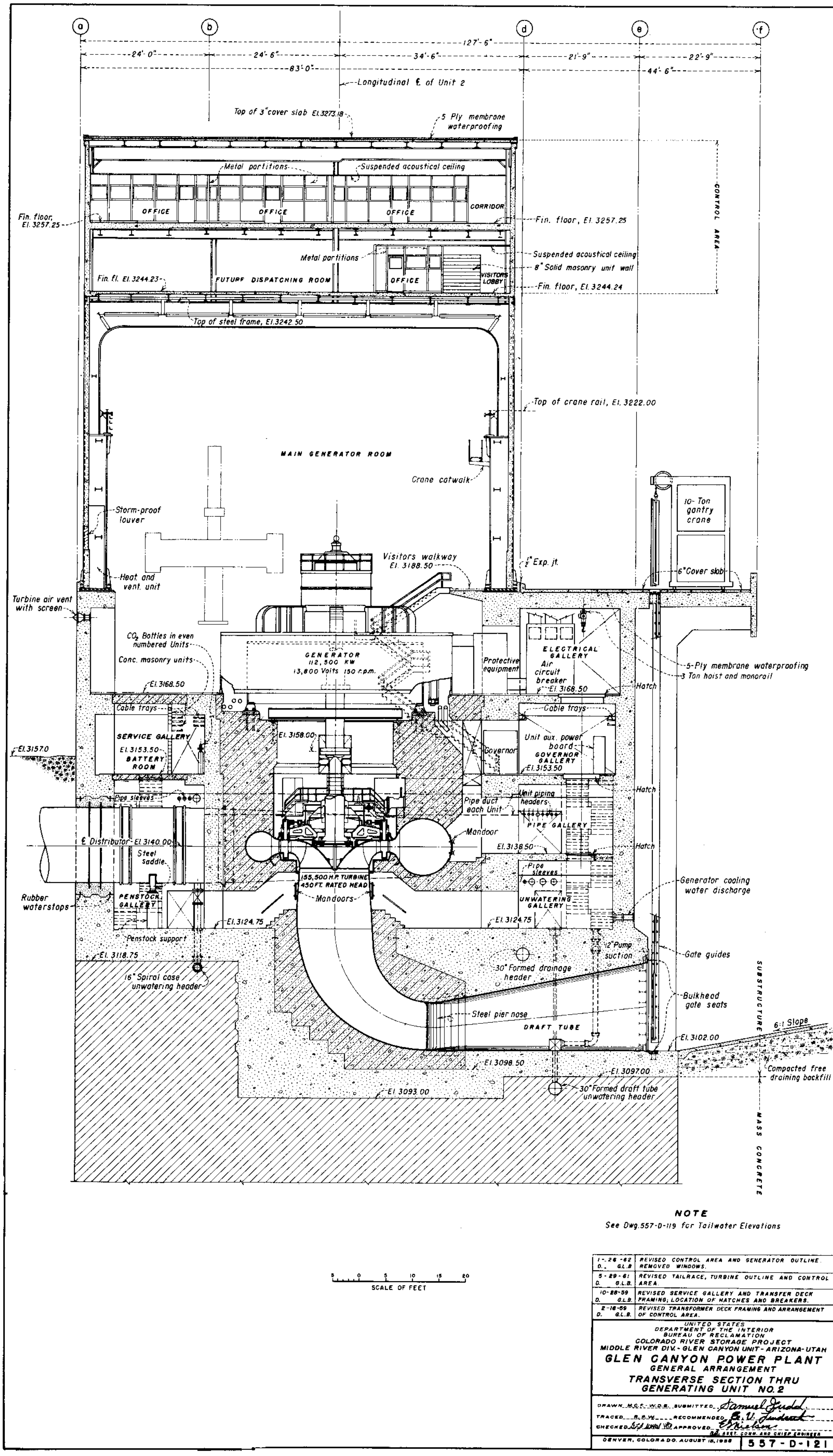


Figure 161.—Powerplant general arrangement—Transverse section through generating unit 2.

room, a future dispatching room, toilets, offices, mechanical equipment, and a decontamination area. The control boards are situated on top of a raised metal floor to provide space for cable spreading under the floor.

The second floor of the control area is at elevation 3257.25 and is arranged for office space as well as room for mechanical and electrical equipment.

The roof of unit bay 3 was designed to serve as an observation deck. This area was included as a part of the visitor facilities for the powerplant. Visitors may travel from the dam to the powerplant by a covered walkway. An air-conditioned lobby is provided at elevation 3188.50 so they may view the generator hall and also receive an orientation of the powerplant. The visitors can then go up the elevator to a lobby at elevation 3244.24. From there, they will be guided past the control room and out to the observation deck where they can view the overall project. Provisions were made to allow for future additions of pools, planters, fountains, and other visitor facilities on the observation deck.

The roof of the control area over unit bays 1 and 2 as well as the roof over unit bays 4 through 8 are constructed of precast concrete roof slabs, covered by a five-ply membrane waterproofing and protected by lightweight concrete cover slabs. The observation deck, which is the roof of unit bay 3, is constructed of a structural concrete floor slab, five-ply membrane waterproofing and a regular concrete cover slab. Tooled grooves and one-fourth-inch expansion joints are used in the cover slab to control cracking.

(c) *Service Bay.*—The main floor of the service bay is at elevation 3188.50 and is arranged to provide adequate areas for rotor erection, transformer handling, upper bearing bracket storage, and shop space. Floors are provided at elevations 3168.50, 3153.50, and at elevation 3124.75 for the service equipment, pumps, sewage collection tanks, and water supply equipment. The main access to the powerplant is through a metal rolling door, 20 feet wide and 28 feet 4 inches high, in the downstream wall of the service bay. When the transformers need maintenance, they can be brought into the powerplant through the main access door on transfer rails extending into the service bay area. A chemical, biological, and radiological (CBR) equipment room is located outside of the upstream wall at elevation 3188.50 on the outlet-pipes anchor block. Access to the CBR equipment room and the upstream area of the powerplant is through a metal rolling door, 16 feet wide and 16 feet high, on the upstream side of the service bay.

The roof of the service bay from the 34- to the 37-line is a cast-in-place concrete slab covered by four-ply membrane waterproofing on 1-inch insulation. The roof of the service bay from the 37-line to rock is a cast-in-place concrete slab covered by 5 feet of 1-1/2- to 3-inch coarse aggregate.

(d) *Machine Shop Bay.*—The machine shop bay superstructure is 146 feet 9 inches in length, 78 feet in width and 54 feet high. There is a 27-foot-wide roadway and a 4-foot 6-inch walkway on the tailrace side of the machine shop for access to the service bay and transformer deck. The machine shop houses a 16-foot boring mill and the necessary equipment for all operation and maintenance work. A 75-ton crane is provided to handle the equipment.

There are four floor levels in the machine shop bay to handle mechanical equipment, filters, plenum chambers, electrical equipment, air compressors, water supply pumps, oil storage tanks, and oil purifier equipment.

Access to the machine shop from the parking area is provided by a 20-foot-wide by 20-foot-high metal rolling door at the downstream end of the machine shop.

Transfer car rails extend from the service bay area into the machine shop area to facilitate moving heavy loads which cannot be reached by the cranes.

The roof of the machine shop bay from the 35-line to the 38-line is a cast-in-place concrete slab covered by a four-ply membrane waterproofing on 1-inch insulation. The roof from the 38-line to rock is a cast-in-place concrete slab covered by 5 feet of 1-1/2- to 3-inch coarse aggregate.

A. DEVELOPMENT OF THE GLEN CANYON POWERPLANT

67. GENERAL. A general discussion and review of some of the problems encountered in the development of the Glen Canyon Powerplant design and the determination of its final location is presented herein.

The first problem to solve was that of fitting a long multiunit powerhouse into a narrow canyon in conjunction with a concrete arch dam. To position the powerhouse parallel to the canyon walls would require the use of large tunnels in the canyon walls leading to smaller tunnels to feed water to each unit. This method of bringing water to each of the hydroelectric units had to be compared with the design using penstocks through the dam to a building placed crossways of the canyon (the design finally adopted). This latter is a careful balance of the shortest penstocks possible for economy with the design requirement that the excavation for the powerhouse end bays shall not remove rock which is needed for support of the arch dam at its abutments. These arch thrust lines angle into the canyon wall at different horizontal positions at different elevations and require progressively shorter lengths of the upstream powerhouse concrete wall at the lower floor elevations. Thus, the location of the powerhouse upstream wall in relationship to the axis of the arch dam becomes a function of the length of building, the depth of powerplant structure, and the dam design, particularly its radius of curvature and the thickness at its abutments.

In the case of Glen Canyon, the use of expensive intake towers in the reservoir and of water tunnels was discarded in favor of the use of penstocks through individual blocks in the dam with intakes on the face of the dam. The decision to place the powerhouse crossways of the canyon, however, caused many difficulties in building layout, influenced to a great extent the number of hydroelectric power units to use, and introduced the shape of the canyon floor in relationship to the powerhouse bottom concrete as an important factor in the design. Since the canyon cross section was U-shaped, it was desirable to locate the control bay and the service bays at the ends of the powerplant because their lowermost concrete was not as low as that of the units and therefore would not cut as deep vertically into the rock at the ends as would the generating units. A balance of costs for a minimum of excavation at the ends with a minimum of mass concrete fill under the center units was an objective in the layouts. However, the bottom elevations of the powerhouse were eventually established by the hydraulic turbine requirements of elevations for maximum power production, height of the centerline of the turbine distributor in relationship to varying tailwater elevations, and tailrace bottom in relationship to the draft tube floor.

68. DEVELOPMENT PHASES. The functional design, leading to the finished general arrangement of the powerplant, progressed through three distinct phases; namely reconnaissance, specifications, and post-specifications issuance or preconstruction. Comparative estimates were prepared, where necessary, to select the best one of several schemes in each phase.

(a) *Phase I—Reconnaissance.*—A favorable site for a major dam and powerplant had been selected by field investigations at mile 15 (or 15.3 miles upstream from Lee Ferry on the Colorado River).

In 1949 and 1950, most of the design data were far from being firm. The height of the dam had not been determined and consequently the reservoir storage capacity and the power head had to be assumed in order to start on layouts for rough feasibility and reconnaissance estimates. A preliminary draft of an interim report in 1949 had recommended a powerplant of 800,000-kilowatt capacity and operation of the Glen Canyon Reservoir with 26 million acre-feet maximum storage. From technical data, a reconnaissance estimate was made for six units producing 800,000 kilowatts of power with a 475-foot rated head. A tailwater at elevation 3140, assumed for the studies, was based upon the water surface of the proposed downstream Marble Canyon Reservoir. It was recognized at this time that a Glen Canyon Reservoir normal water surface above elevation 3725 for 30-million-acre-feet capacity would encroach on the Rainbow Bridge National Monument. A maximum capacity of 26 million acre-feet would lower the reservoir 25 feet to elevation 3700. (Later studies give 28 million acre-feet for this elevation.)

A request by the regional office for further studies to establish the number of units for an 800,000-kilowatt plant, led to a suggestion that an alternative plant rating of 970,200 kilowatts with a rated head of 534 feet be considered because it represented an optimum relation between turbine and generator capacities under the then present reservoir operation studies, which would produce the greatest annual generation at relatively low incremental cost. For this reason an additional unit was included in the five schemes that were studied. Three schemes were for surface powerplants having seven units, with a rating of 138,600 kilowatts for each unit and a total plant capacity of 970,200 kilowatts. One scheme was for a six-unit surface plant of 138,600 kilowatts each with an 831,600-kilowatt total capacity. The fifth scheme was a six-unit underground plant of the same capacity as the six-unit surface plant. All schemes had the centerline of the distributor at elevation 3140 except the six-unit surface plant which used elevation 3121.

One of the purposes of establishing at this time the number of units in the powerplant was their effect on the dam design. The number of penstocks affected the position of the intakes, the outlet pipes, and the jointing of the dam.

Because additional reservoir studies were needed, the regional office questioned the advisability of providing a plant of the higher plant capacity of 970,200 kilowatts and sought agreement on a plant of 800,000-kilowatt capacity.

(1) *Six-unit underground plant.*—The Denver office at this time (1950) was under administrative instructions to investigate the possibility of placing future powerplants underground at all sites having suitable rock. However, at most sites studied by the Bureau, the cost of an underground plant does not compare favorably with the cost of a surface plant. An underground plant requires much rock excavation, and the rock above the generator hall may require a massive reinforced concrete arch roof to support it; whereas a surface plant superstructure requires no excavation, the roof loads are relatively light, and the superstructure framing is much less expensive than the underground roof-supporting concrete. Therefore, an underground plant is usually at a disadvantage economically as compared to a surface plant.

A layout was made of an underground plant downstream from the left abutment of the dam. It consisted of an excavated hall containing six units, two station-service units, and a service bay and control bay at opposite ends of the hall. The transformers were in partitioned vaults along the generator floor. The elevation of the centerline of the distributor was approximately the same as in the seven-unit schemes at elevation 3140. An inclined shaft from the control bay to an aboveground switchyard was considered. An access tunnel above maximum tailwater was provided to the service bay from an unloading area under the overcanyon cableway (used in all the 1950 studies) which area was alongside a surface machine shop near the toe of the dam.

Several studies were made of the arrangement of the tailrace tunnels, the intake towers, and the intake water passageways. Suitable positioning of the intakes that would prevent excavation from interfering with the thrust lines of the arch dam and that would not require excessive forebay excavation in the canyon wall presented a problem.

Rough estimates of comparative cost of the six-unit surface and underground schemes were surprisingly close because the mass concrete requirements for the surface plant tended to balance the large amount of underground excavation that would have been necessary for an underground plant. However, because of the uncertainty about the cost per cubic yard for underground excavation, the underground scheme was not given further consideration in later studies.

In all six- and seven-unit schemes, an overhead cableway was provided above an unloading platform for equipment handling to save the expense of an access roadway from the canyon rim. Equipment was to be moved by transfer car on tracks from the unloading areas into the powerhouse as is done at the Bureau's Hoover Powerplant. Access for personnel was to be provided by elevators and galleries in the dam and by a walkway from the dam to the powerhouse.

(b) *Phase II—Specifications.*—Funds were not programed for design studies and estimates between 1950 and 1956. Comparative reconnaissance-type cost estimates were requested in March 1956 for four combinations of low and high dams and hydroelectric powerplants of about 800,000- and 1,000,000-kilowatt capacity to aid in economic power studies. Studies were made to determine the most suitable number of units. In these studies, the following conditions and requests were considered:

(1) Lower the centerline of the distributor to elevation 3121 to reduce the amount of mass concrete fill under the units. (This was later changed to elevation 3140.)

(2) Use the shortest penstock lengths possible without powerhouse excavation cutting into arch dam thrust lines.

(3) Use the shortest length of powerhouse to fit the canyon and avoid excessive excavation of canyon walls.

(4) Prepare estimates for eight-unit plants of 120,000 kilowatts each or 960,000 kilowatts total capacity for both 515- and 480-foot weighted average heads.

(5) Prepare estimates for seven-unit plants of 114,000 kilowatts each or 798,000 kilowatts total for both 515- and 480-foot weighted average heads.

(6) Lacking turbine data sheets, assume same data as used in the 1950 studies and use 65-foot-wide unit bays.

(7) Eliminate the two station-service units provided in the 1950 studies.

(8) Place an 80-foot-wide service bay at the end of the units and a control-machine shop area parallel to and joined to the upstream side of the central units.

(9) Provide an access road and turn-around area, instead of the overhead cableway.

Schemes were made for a seven-unit plant and for an eight-unit plant. At this time, some consideration was given to more than eight units at a smaller capacity per unit, namely 100,000 kilowatts each. Increasing the number to 10 units was a means of lowering the number of kilowatts per unit and reducing the speed of the machine. There was some concern about the high-strength steel needed on the generators which would operate at high peripheral speeds as used in the 1950 studies.

A 10-unit scheme was also prepared. Because of the length of the building, an excessive amount of canyon wall rock excavation would have resulted in this cross-canyon position, but to have turned it parallel to the canyon wall would have cost more for water tunnels than for the surface penstocks. This building with more units, although requiring slightly smaller bays, was longer than the seven- and eight-unit layouts and would have required much longer penstocks. The longer building would have forced the outlet pipes into tunnels along the canyon walls at increased cost over their surface location in the other schemes. Preliminary estimates were made of the penstocks only for both the high and low dams for each layout of 7, 8, and 10 units. The total cost of the penstocks for the 10-unit scheme was almost twice that for the 7-unit scheme for both high and low dams because of their greater length and number, although they would have been smaller in diameter.

Use of more than eight units appeared undesirable both from the economical and physical standpoints. Although it was decided not to firmly establish the number of units at this time, these studies did in fact crystalize the use of eight units for the powerplant.

The studies up to this time had been for both high and low dams with a 1,000-foot radius to the axis. A decision was reached to use the low dam and change the radius to 900 feet. The powerplant cross sections used for the 1950 studies were replaced with new ones based upon turbine data sheets for eight units of 112,500 kilowatts each for a total plant capacity of 900,000 kilowatts, and a design head of 510 feet. A study of the effect on the building size for machine speeds of 150 and 163.6 revolutions per minute showed a somewhat smaller size unit for the faster speed, but it was decided to use the slower speed to avoid any generator design difficulties. This speed of 150 revolutions per minute, a design head of 510 feet, and rated head of 450 feet, and eight units of 112,500-kilowatt capacity each are the statistics used throughout all later schemes and the specifications.

As a result of the studies so far prepared, decisions were made in May of 1956 to accept the conditions listed at the start of phase II (this subsection) together with the following modifications:

(1) Use of eight units at a rating of 112,500 kilowatts each totaling 900,000 kilowatts.

(2) Locate a machine shop in the powerhouse instead of alongside the warehouse in the town of Page, Ariz.

(3) Limit the speed of the units to 150 revolutions per minute, and when heads are higher than 450 feet the turbine wickets would be regulated to prevent overload of the generators.

(4) The centerline of the distributor was not to be lower than elevation 3140. This decision was made because of previous difficulties of leakage with higher draft tube gate pressures and probable rough turbine operation which have accompanied low distributor settings such as the elevation 3121 proposed for the six-unit surface plant.

(5) A firm elevation of 3183 for maximum tailwater surface was received, so the freeboard was based upon elevation 3188.50 instead of 3196 for the entrance door and approach road. This reduced the building height and lowered the building cost.

(6) A suggestion to provide warehouse space in the powerhouse was rejected as too expensive a provision. It was considered better to locate a permanent warehouse in the town of Page, Ariz., and provide only minor storage space in the powerhouse so as to keep the structure as small as possible.

(7) Because of the proximity of the powerhouse to the canyon wall, it was considered advisable to protect the roofs of the end bays and the service and control bays from small falling rocks

by use of a sand-cushion-type construction. Major slabs of rock were to be scaled during excavation.

Studies were made for locating the transformers on the upstream and downstream decks of the powerplant. After comparing the transformer locations, the scheme with the transformers on the upstream deck was abandoned, and the decision was made to locate the transformers on the downstream deck in all future schemes. It was also decided to use a structural steel superstructure with concrete curtain walls for the main units and the service bay. The control bay was to be all reinforced concrete.

Schemes for the location of the control bay along the upstream wall of units 1 and 2 and for its location along the upstream wall of units 4 and 5 were investigated. These schemes were discarded due to the disadvantage of this location for the electrical layout.

Another scheme was an attempt to combine the service bay and the control bay in the center with four units on each side. This scheme was also discarded because of the difficult access, the change of the penstock locations in the dam, crowding the outlet pipes farther into the canyon side walls and the intakes into the abutments, and putting the deep unit bays at the ends to increase excavation and shallow bays in the deepest part of the channel which added more mass concrete to the layout.

About this time, word was received that issuance of the prime construction contract was scheduled to be made earlier than previously announced, so the following major decisions were made to finalize the specifications layout for the powerplant:

(1) Locate the control bay at the right end of the powerplant.

(2) Locate the governor gallery on the downstream side of the powerplant.

(3) Locate the transformers on the downstream deck of the powerplant.

(4) Establish the centerline of the distributor at elevation 3140.

(5) Use expansion joints between each bay in the powerplant.

(6) Proceed with the general and structural arrangement drawings for the construction specifications.

As a result of the above decisions reached in the first part of June 1956, the best ideas of the previous schemes were incorporated into the specifications layout.

A check of the powerplant building for clearance with the thrust lines of a new dam design showed that the building could be moved 54 feet closer to the dam axis, or from 524 to 470 feet. Although this was not a final dam design, it was accepted for the purpose of meeting the announced construction specifications issuance date.

Design studies indicated that it was highly practicable to move the four outlet pipes to the left side of the dam and embed the bends in the mass concrete between the dam and the upstream side of the service bay, and then continue them along the outside service bay wall to an outlet valve house downstream of the service bay turning around area. By doing this it was possible to utilize the surface of the concrete embedding the outlet pipes as a floor and have a machine shop between the service bay and the dam.

Specifications No. DC-4825 for construction of Glen Canyon Dam and Powerplant were issued on January 11, 1957, which incorporated the above arrangement.

(c) *Phase III—Preconstruction.*—The Glen Canyon Powerplant, as shown on the construction specifications drawing "General Plan," was located 470 feet downstream from the dam axis. This location of the conventional "in-line" building (all bays alongside each other) and 665 feet long met the requirement of positioning all powerplant excavation outside the thrust lines in the abutments supporting the concrete arch dam design shown in the specifications.

Final reports of additional tests taken subsequent to the preparation and issuance of the specifications, required thickening the dam at the abutments to reduce the bearing pressures. This revised dam design changed the arch thrust lines in such a way as to require locating the powerplant downstream an additional 20 feet, or 490 feet from the dam axis.

Such a move was considered undesirable as it would have lengthened the eight penstocks and four outlet pipes, and would have increased the powerplant cost. It was therefore decided that studies should be made not only to save the cost of shifting the powerhouse 20 feet downstream but also to attempt to reduce the original 470 feet shown in the specifications and make further savings.

The specifications powerplant was drawn on a print of the new dam design, and its encroachment on the thrust lines studied. Specifications drawings of the control bay with vellum overlays were studied for room rearrangement. Only by eliminating rooms could the encroachment on the thrust lines be avoided. However, loss of such room area did not appear to be a desirable solution to the problem.

A repeat attempt to turn the machine shop from the specifications location between the dam and the service bay on the left side to a position alongside the upstream side of the service bay and unit 8, and shifting the powerhouse towards the left, did show a reduction of 25 feet or to a location 445 from the dam axis to the upstream wall of the units. Poor access and more expensive undersupport of the machine shop, as in previous similar schemes, led to the abandonment of this study.

Consideration was then given to changing the shape of the rectangular building as a means of solving the length distance problem. Layouts were made of a variety of building shapes, such as a curved powerhouse, V-, U-, and modified L-shapes.

Eight study schemes were drawn and cost estimates made. In the V-shaped schemes, the bends in the penstock and outlet pipes were too complicated or crossed the dam block joints. The curved plant and the U-shaped schemes created undesirable water passage conditions, equipment handling difficulties, or introduced undesirable structural problems.

For the above reasons, the curved plant, V-, and U-shaped schemes were abandoned. Although preliminary cost estimate showed the curved plant scheme to be as economical as any scheme, it was decided to return to the original rectangular-shaped building and modify the positions of the end bays, the control and service bays, and the machine shop, in relationship to the generating units. This decision was based upon the unsatisfactory results of the aforementioned schemes and upon a desire to retain the design as close as possible to the specifications design to avoid unnecessary costs.

The machine shop was moved from the upstream specifications location to downstream of the service bay. This reversal of the L-shape allowed the service bay and eight units to move closer to the dam and shortened the penstocks, but in so doing caused the control bay on the right end to encroach on the dam thrust lines.

Extensive studies were made to preserve the control bay as located in the specifications—at the end and alongside unit 1. Each floor area at the upstream end corner was reduced by rearranging rooms and the bay kept outside the thrust lines. This reduction in floor area was objectionable. The control bay was moved downstream until only a passageway connected it with unit 1, sufficient to provide for cable trays, pipe runs, and access between the structures. The shape was streamlined to avoid the thrust lines by placing the narrow elevator and stair shaft upstream of the control room. This scheme located the powerhouse 400 feet from the dam axis.

Further study showed that much of the floor space in the previous service bay would be lost to the mass concrete anchoring the bends in the outlet pipes. This lost room area was to be provided for in a new machine shop bay layout. The outlet pipes which had previously been outside the end of the powerhouse were now run under both the service bay and machine shop. To improve the design at the left end, it was decided to shift the building up to 10 feet to the right of the plane of centers. To avoid cutting the thrust lines, the width of the control bay was reduced from 65 to 50 feet. This reduction, however, provided insufficient room areas for a major powerplant control bay. Even if it had been possible to make the control bay narrower, the gain from a 15-foot shift of the powerhouse to the right would not have been sufficient to relieve the congestion in the service bay end caused by the outlet pipes.

Work was interrupted until receipt of the results of a meeting of the Consulting Board[1] to consider the rock in the dam abutments and review the dam design. A new dam design with thicker abutments was decided upon. This change required moving the powerhouse farther downstream. Since all schemes so far to reduce the length of powerhouse had various objections to them, it was decided that a drastic change was needed.

The control bay, as such, was eliminated except for a narrow tower located at the downstream corner of the end of unit 1. This tower contained a stairway, a

[1]The Consulting Board consisted of Julian Hinds, Chairman, John J. Hammond, Raymond E. Davis, Edward B. Burwell, Jr., and John W. Vanderwilt.

passenger elevator, and a cable shaft. A two-story structural steel structure called the control area was placed above the superstructure of units 1 and 2. This shortened the length of the upstream wall to 598 feet 9 inches anu permitted the building to be 400 feet from the dam axis. The units were offset 10 feet to the right of the plane of centers. The elevator tower added 24 feet and the widened machine shop added 25 feet for a total powerplant length of 655 feet, but these extensions were downstream of the a-line and outside of the thrust lines.

69. FINAL DESIGN. A review of the layouts, made in conjunction with previous dam designs, showed that with minor modifications the result of the previously detailed powerplant studies could be utilized for the final design of the powerplant structure in conjunction with the final dam design. New general arrangement drawings were prepared superseding all or in part the specifications drawings. The final design for construction shows:

(1) The upstream wall of the units and service bay is located 400 feet from the axis (70 feet less than the specifications design).

(2) The machine shop is located downstream instead of upstream of the service bay and has a passage on the tailrace side of the machine shop for access to the service bay.

(3) The control area is located above the typical superstructure of units 1 and 2, and the control bay at the end was eliminated.

(4) Access to the control area is by means of an elevator and stair tower, which provides for a cable shaft to the cable spreading and control area.

(5) The final upstream or a-line length is 598 feet 9 inches and the overall length from the elevator tower to the machine shop is 648 feet 9 inches. The elevator tower and machine shop are downstream 24 and 83 feet, respectively, from the a-line.

As a result of these studies in phase III (subsec. 68c), a new location utilizing shorter penstocks and other numerous layout improvements saved an estimated 1 million dollars over the specifications design when calculated by using the actual unit costs contained in the contractor's bid.

The revisions discussed above were incorporated in specifications No. DC-4825 by means of supplemental notices.

70. CHEMICAL, BIOLOGICAL, AND RADIOLOGICAL PROTECTION. Protection against radiological fallout at Glen Canyon Powerplant was provided. This was in accordance with Annexes 11, 24, and 33 of the National Plan for Civil Defense and Defense Mobilization which made the Bureau responsible for the continued operations of its power facilities during periods of national emergency.

In general, the criteria used in the design and arrangements follow the Corps of Engineers Manual EM 1110-2-5000 (1 Mar. 61).

In matters of design, the OCDM (now OCD) publication "Design and Review of Structures for Protection from Fallout Gamma Radiation," also known as the "Engineering Manual," dated July 1, 1961, was used.

71. POWERPLANT COMPLETION. The work for the completion of the powerplant consisted of placing second-stage concrete, blockout concrete, concrete floor surfacing and miscellaneous concrete; constructing partition walls and completing architectural finishes; installing air-conditioning, heating and ventilating systems; installing hydraulic turbines, penstock makeup pieces, governors, machine tools, piping and fire protection equipment; furnishing and installing sewage treating equipment, plumbing fixtures, and water supply piping and installing miscellaneous metalwork; installing power transformers, generators, generator voltage bus structures, switchgear, electric conduit, cables and electrical accessories.

B. POWERPLANT STRUCTURAL DESIGN

72. BASIC DATA. The following data and codes were used in the structural design of the Glen Canyon Powerplant:

(1) "Reinforced Concrete Design Data," Engineering Monograph No. 10, October 1952, U.S. Department of the Interior, Bureau of Reclamation.

(2) "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, June 1940, American Society of Civil Engineers.

(3) "Standards of Design for Concrete," U.S. Navy Department, Bureau of Yards and Docks, Publication No. 3Yb, November 15, 1929.

(4) "Building Code Requirements for Reinforced Concrete" (ACI 318-56), American Concrete Institute.

(5) "Steel Construction," Manual of the American Institute of Steel Construction (1956).

73. STABILITY ANALYSIS. A separate, thorough, and comprehensive stability analysis was made for unit bays 2 through 8, unit bay 1, the service bay, and the machine shop bay. Each of the above bays was separated by a 1-inch expansion joint and therefore was analyzed as a separate unit.

The bays were analyzed to determine the factors of safety for overturning, shear-friction, and flotation. Factors of safety were determined by the following formulas:

Flotation . . . . $\frac{\Sigma W}{\Sigma U}$

Overturning . . . . $\frac{\Sigma RM}{\Sigma OM}$

Shear-friction . . . . $\frac{(\Sigma W + U) f + cA}{\Sigma H}$

where:

A = area of base or horizontal section considered (in compression), square feet,

c = cohesion or unit shearing resistance, applied only to area in compression, kips per square foot,

f = coefficient of friction between concrete and foundation material or between concrete and concrete,

U = uplift, assumed acting over 100 percent of area,

ΣH = sum of horizontal forces,

ΣOM = overturning moment about toe,

ΣRM = righting moment about toe; and

ΣW = sum of vertical forces, except uplift; plus (+) is down.

See figure 162 for the general notes used for the stability analysis, the foundation properties and the allowable factors of safety for the conditions: (1) during construction and (2) with the structure completed and equipment operating.

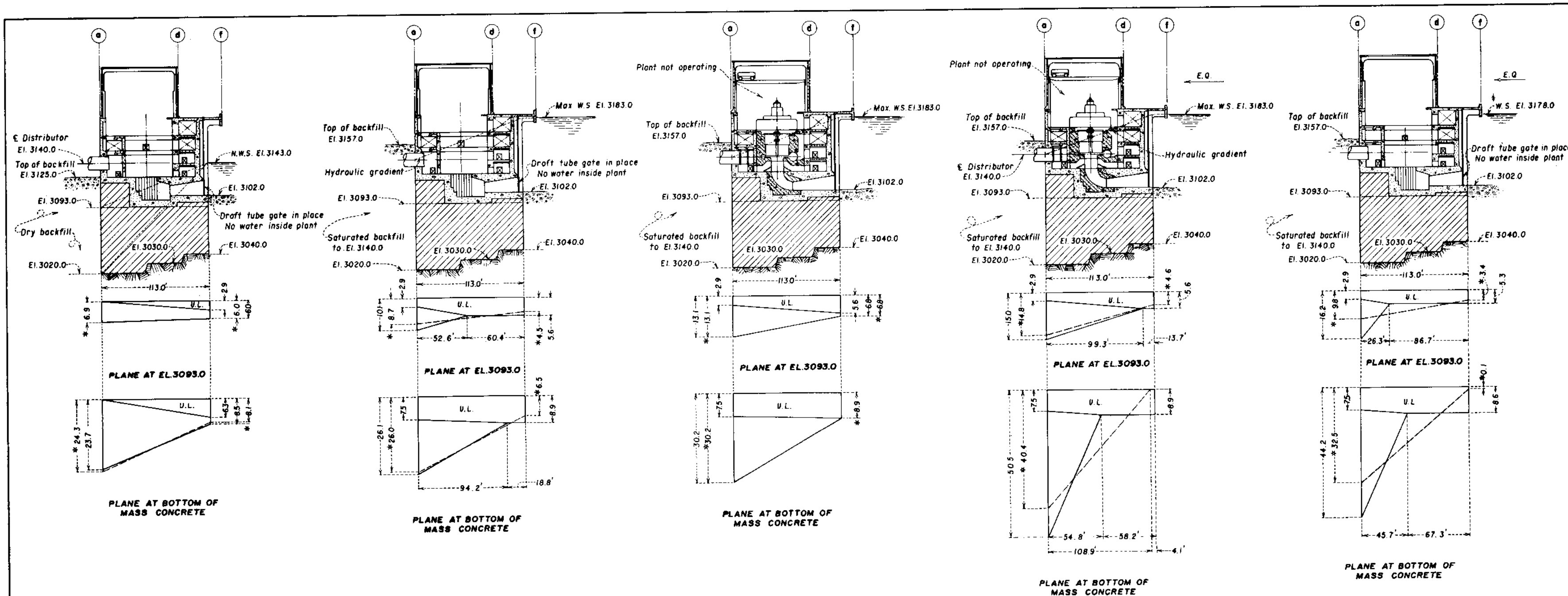
(a) *Units 2 Through 8.*—Units 2 through 8 were identical except for the excavation and mass concrete; therefore, the unit with the most severe condition was selected for the analysis. Stability was analyzed at two different planes, one at elevation 3093.00 and the other at elevation 3020.00. The plane at elevation 3093.00 was investigated since it was the lowest point of the first-stage concrete. The other plane at elevation 3020.00 was chosen since it was the lowest point of the excavation.

When the stability analysis showed a tension area in the base pressure diagram, the section was assumed to be cracked and full hydrostatic uplift was placed on the tension zone. The length of crack was determined by successive approximations until stability was achieved with a base pressure diagram having only compressive forces.

For the computed factors of safety for each stability case, see figures 162 and 163.

(b) *Unit Bay 1.*—Unit bay 1 was analyzed separately since it was quite different from units 2 through 8. The base of unit 1 was on rock instead of mass concrete; it had a tall narrow elevator tower on its right side and was placed against the right canyon wall. One hundred and fifty-four grouted 1-3/8-inch anchor bars as well as mass concrete were used below the upstream base slab to assure adequate stability of the unit. The effects of the tower and the water table in the canyon wall on the stability analysis can be seen on figure 164.

(c) *Service Bay.*—The stability analysis for the service bay consisted of three conditions; namely, (1) a construction condition, (2) the minimum operating condition, and (3) the maximum operating condition. The concrete anchor block for the outlet pipes upstream of the service bay was considered a part of the service bay for the stability analysis. Uplift pressures were considered under all conditions of loading. Full uplift was assumed on the foundation steps to the saturation line of the rock. Hydraulic forces from the outlet pipes were also considered in the analysis for the operating conditions. Grouted 1-3/8-inch anchor bars were used to improve the stability of the structure. For the computed factors of safety, see figure 165.



	FACTORS OF SAFETY		
	PLANE AT El. 3093.0		PLANE AT BOTTOM MASS CONC.
	UNIT 2	UNIT 3 THRU 8	
OVERTURNING	3.1	3.0	2.3
SHEAR-FRICTION	37.9	102.5	8.1
FLOTATION	4.5	4.4	5.0

CASE I

	FACTORS OF SAFETY		
	PLANE AT El. 3093.0		PLANE AT BOTTOM MASS CONC.
	UNIT 2	UNIT 3 THRU 8	
OVERTURNING	1.1	1.1	1.3
SHEAR-FRICTION	6.9	16.8	9.8
FLOTATION	1.3	1.3	1.9

CASE II

	FACTORS OF SAFETY		
	PLANE AT El. 3093.0		PLANE AT BOTTOM MASS CONC.
	UNIT 2	UNIT 3 THRU 8	
OVERTURNING	1.7	1.7	1.5
SHEAR-FRICTION	15.5	41.2	12.3
FLOTATION	2.4	2.3	2.4

CASE IIA

	FACTORS OF SAFETY		
	PLANE AT El. 3093.0		PLANE AT BOTTOM MASS CONC.
	UNIT 2	UNIT 3 THRU 8	
OVERTURNING	1.4	1.4	1.2
SHEAR-FRICTION	6.7	18.2	3.0
FLOTATION	2.3	2.2	2.2

CASE IIB

	FACTORS OF SAFETY		
	PLANE AT El. 3093.0		PLANE AT BOTTOM MASS CONC.
	UNIT 2	UNIT 3 THRU 8	
OVERTURNING	1.0	1.0	1.1
SHEAR-FRICTION	2.7	6.5	3.5
FLOTATION	1.3	1.3	1.8

CASE IIC

‡ Maximum tailwater elevation that structure safely resists under case II-C loading.

GENERAL NOTES

Factors of safety are defined by the following formulae:

Overturning... $\frac{\Sigma RM \text{ (righting mom.)}}{\Sigma OM \text{ (overturning mom.)}}$

Shear-friction = $\frac{(\Sigma W + U)f + cA}{\Sigma H}$; ΣW = summation of vertical forces; c = cohesion in kips /sq. ft.; A = area, in sq. ft. of base in compression.

Flotation..... $\frac{\Sigma W \text{ (vertical forces)}}{\Sigma U \text{ (uplift forces)}}$

Base pressures are in kips per sq. ft.

U.L. denotes uplift, plus (+) is down.

The following foundation properties were recorded in memo received from Concrete Laboratory Branch:

1. Coefficient of internal friction = 1.34. (values used in analyses: 0.8 concrete to concrete, and 0.6 concrete to rock.)
2. Shear strength (cohesion) of rock in powerplant region = 300 psi. (values used in analyses: concrete on rock = 100 psi., and concrete on concrete 300 psi.)
3. Bond strength in tension between concrete and rock = 70 psi. (value used in analyses = 0 psi.)
4. Established allowable bearing values concrete on rock.
   86 kips /sq. ft. without earthquake
   100 kips /sq. ft. with earthquake

NOTES

* Values denote base pressures without uplift on any part of the base.

Unit 2 factors of safety are recorded separately because the substructure is resting on a rock foundation at El. 3093.0

ALLOWABLE FACTORS OF SAFETY

	DURING CONSTRUCTION		STRUCTURE COMPLETED AND EQUIPMENT OPERATING	
	LOADING		LOADING	
	NORMAL	EXTREME	NORMAL	EXTREME
OVERTURN	1.1	1.1	1.2	1.1
SHEAR-FRICTION	2.5	1.1	3.5	2.0
FLOTATION	1.1	1.1	1.2	1.1

REFERENCE DRAWINGS

STRUCTURAL DESIGN DATA:
- UNIT BAYS 557-D-149
- SERVICE BAY 557-D-150
- MACHINE SHOP 557-D-151

STABILITY ANALYSES
- UNIT BAYS 2 THRU 8 .... 557-D-603,604
- UNIT BAY 1 .... 557-D-605,606
- SERVICE BAY 557-D-607
- MACHINE SHOP 557-D-608

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT--ARIZONA-UTAH

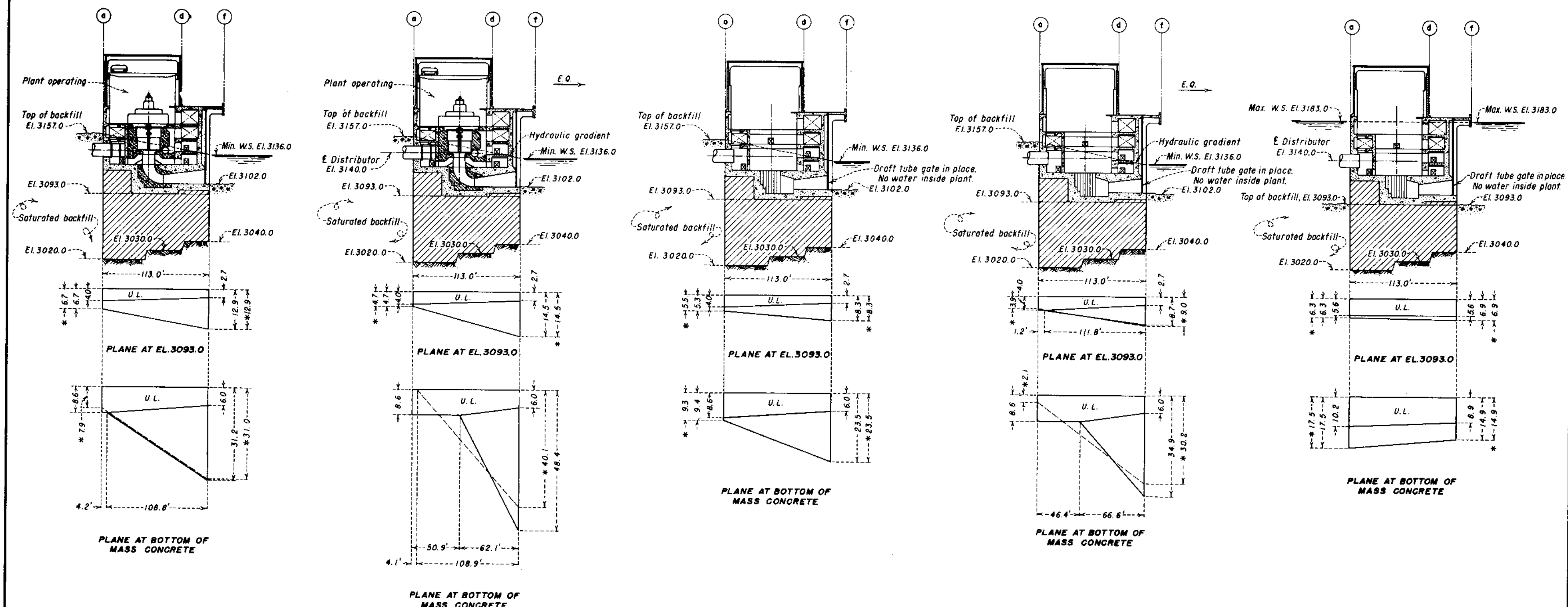
GLEN CANYON POWER PLANT

UNIT BAYS 2 THRU 8
STABILITY ANALYSES

DRAWN A.H. M.A. SUBMITTED H.H. Bennett
TRACED J.L.D. RECOMMENDED Samuel Judd
CHECKED [illegible] APPROVED E.V. [illegible]
ACT. CHIEF DESIGNING ENGINEER

DENVER, COLORADO, DEC. 4, 1967 SHEET 1 OF 2 | 557-D-603

Figure 162.—Powerplant stability analyses—Unit bays 2 through 8. (Sheet 1 of 2.)



	FACTORS OF SAFETY		
	PLANE AT El. 3093.0		PLANE AT BOTTOM MASS CONC.
	UNIT 2	UNIT 3 THRU 8	
OVERTURNING	2.2	2.1	1.5
SHEAR-FRICTION	10.0	26.3	5.4
FLOTATION	3.0	2.9	2.7

CASE III

	FACTORS OF SAFETY		
	PLANE AT El. 3093.0		PLANE AT BOTTOM MASS CONC.
	UNIT 2	UNIT 3 THRU 8	
OVERTURNING	1.9	1.8	1.2
SHEAR-FRICTION	6.6	17.5	2.3
FLOTATION	3.0	2.9	2.5

CASE III A

	FACTORS OF SAFETY		
	PLANE AT El. 3093.0		PLANE AT BOTTOM MASS CONC.
	UNIT 2	UNIT 3 THRU 8	
OVERTURNING	1.7	1.6	1.5
SHEAR-FRICTION	15.7	43.6	6.4
FLOTATION	2.1	2.0	2.2

CASE IV

	FACTORS OF SAFETY		
	PLANE AT El. 3093.0		PLANE AT BOTTOM MASS CONC.
	UNIT 2	UNIT 3 THRU 8	
OVERTURNING	1.5	1.4	1.2
SHEAR-FRICTION	9.7	27.3	2.5
FLOTATION	2.0	1.9	2.1

CASE IV A

	FACTORS OF SAFETY		
	PLANE AT El. 3093.0		PLANE AT BOTTOM MASS CONC.
	UNIT 2	UNIT 3 THRU 8	
OVERTURNING	1.2	1.2	1.7
SHEAR-FRICTION	∝	∝	70.8
FLOTATION	1.2	1.2	1.7

CASE V

NOTES

For general notes and reference drawings, see 557-D-603.

* Values denote base pressures without uplift on any part of the base.

Unit 2 factors of safety are recorded separately because the substructure is resting on a rock foundation at El. 3093.0.

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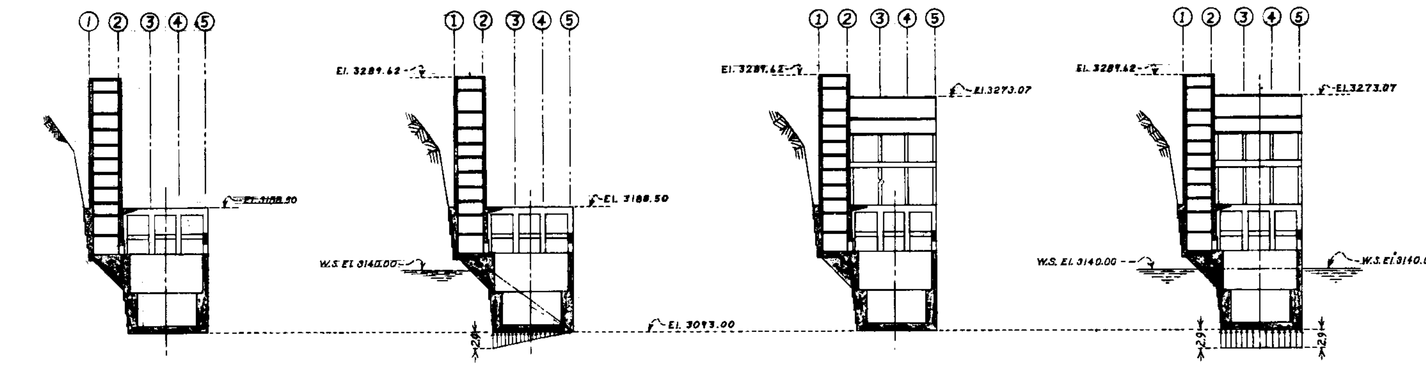
MIDDLE RIVER DIV.-GLEN CANYON UNIT--ARIZONA-UTAH

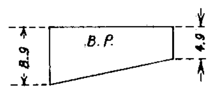
GLEN CANYON POWER PLANT

UNIT BAYS 2 THRU 8
STABILITY ANALYSES

DRAWN A.H.-M.A. SUBMITTED H.H. Bennett
TRACED A.P.W. RECOMMENDED Samuel Bedd
CHECKED APPROVED L.V. Lindeman
DENVER, COLORADO, DEC. 4, 1957
SHEET 2 OF 2
557-D-604

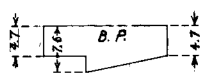
Figure 163.—Powerplant stability analyses—Unit bays 2 through 8. (Sheet 2 of 2.)





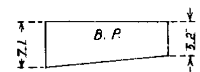
CASE I

Design: 1st Stage concrete to El. 3188.50, + tower complete. No tailwater and no backfill.



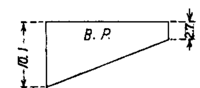
CASE II

Design: 1st Stage concrete to El. 3188.50, + tower complete. Backfill to El. 3125.0 @ U.S. side. Tailwater to El 3140.0 @ Abut. side.



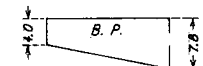
CASE III

Design: 1st Stage concrete complete. Steel superstructure complete. Backfill to El. 3157.0 @ U.S. side + no tailwater.



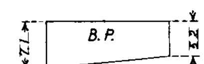
CASE III-A

Design: Case III + E.Q.(.1g) from Unit 2 side.



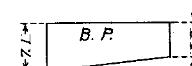
CASE III-B

Design: Case III + E.Q.(.1g) from Abut. side.



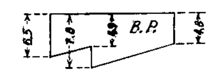
CASE III-C

Design: Case III + E.Q.(.1g) from U.S. side



CASE III-D

Design: Case III + E.Q.(.1g) from D.S. side.



CASE III-E

Design: 1st Stage concrete complete. Steel superstructure complete. Backfill to El. 3157.0 @ U.S. side. Tailwater to El. 3140.0 all around plant. Saturated backfill to El. 3140.0 @ U.S. side.

	Factors of Safety
Overturning	∞
Shear-friction	∞
Flotation	∞

	Factors of Safety
Overturning	4.34
Shear-friction	15.96
Flotation	5.40

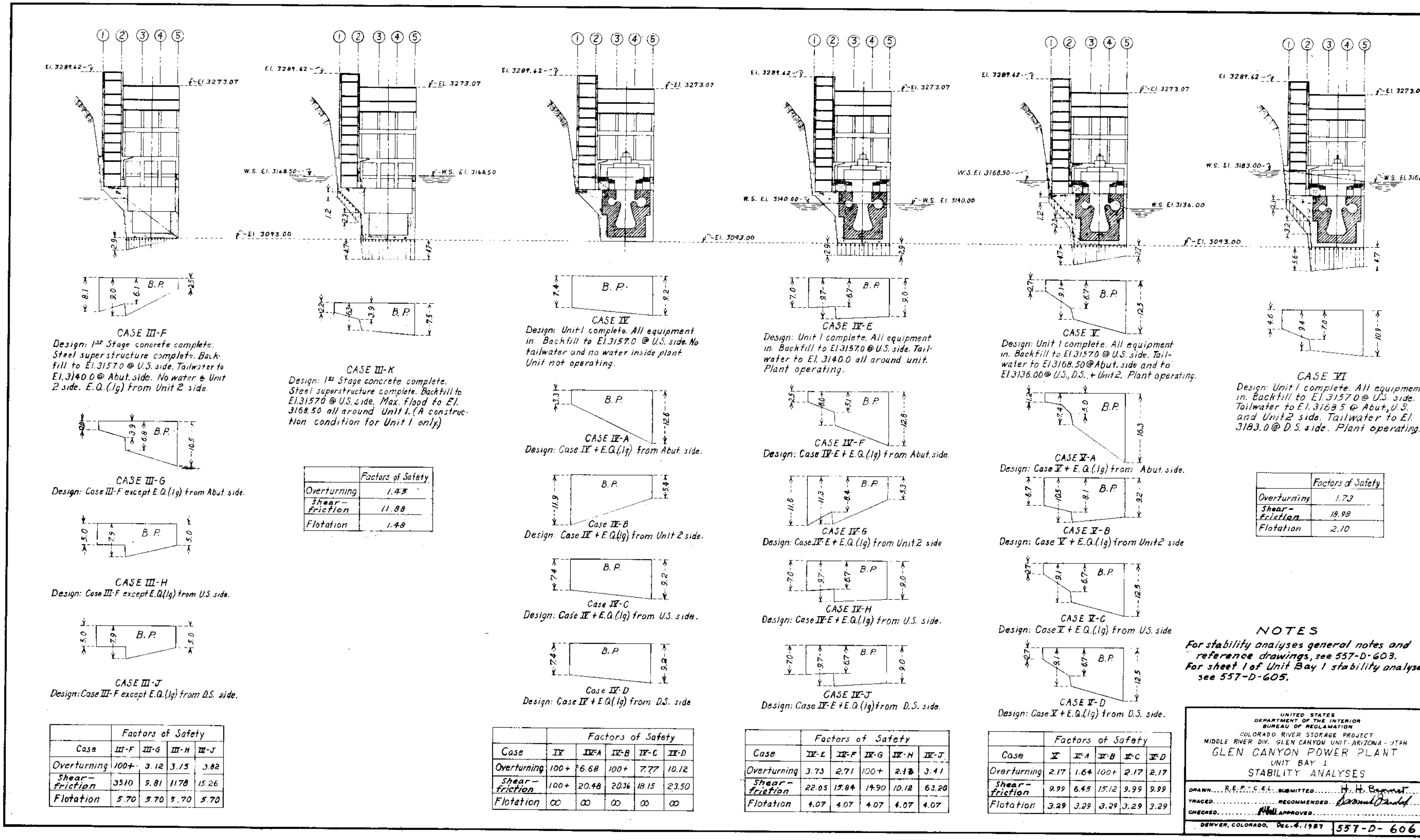
	Factors of Safety
Overturning	100+
Shear-friction	81.40
Flotation	2.85

Case	Factors of safety				
	III	III-A	III-B	III-C	III-D
Overturning	100+	100+	7.25	7.50	10.42
Shear-friction	100+	25.98	25.98	22.02	31.60
Flotation	∞	∞	∞	∞	∞

NOTES

For stability analyses general notes and reference drawings, see 557-D-603.
For sheet 2 of Unit Bay 1 stability analyses, see 557-D-606.

Figure 164.—Powerplant stability analysis—Unit bay 1. (Sheet 1 of 2.) From drawing No. 557-D-605.



	Factors of Safety			
Case	III-F	III-G	III-H	III-J
Overturning	100+	3.12	3.15	3.82
Shear-friction	35.10	9.81	11.78	15.26
Flotation	5.70	5.70	5.70	5.70

	Factors of Safety
Overturning	1.45
Shear-friction	11.88
Flotation	1.48

	Factors of Safety				
Case	IV	IV-A	IV-B	IV-C	IV-D
Overturning	100+	6.68	100+	7.77	10.12
Shear-friction	100+	20.48	20.36	18.15	23.50
Flotation	∞	∞	∞	∞	∞

	Factors of Safety				
Case	IV-E	IV-F	IV-G	IV-H	IV-J
Overturning	3.73	2.71	100+	2.18	3.41
Shear-friction	22.05	15.84	14.90	10.12	63.20
Flotation	4.07	4.07	4.07	4.07	4.07

	Factors of Safety				
Case	V	V-A	V-B	V-C	V-D
Overturning	2.17	1.64	100+	2.17	2.17
Shear-friction	9.99	6.45	15.12	9.99	9.99
Flotation	3.29	3.29	3.29	3.29	3.29

	Factors of Safety
Overturning	1.73
Shear-friction	18.98
Flotation	2.10

Figure 164.—Powerplant stability analysis—Unit bay 1. (Sheet 2 of 2.)

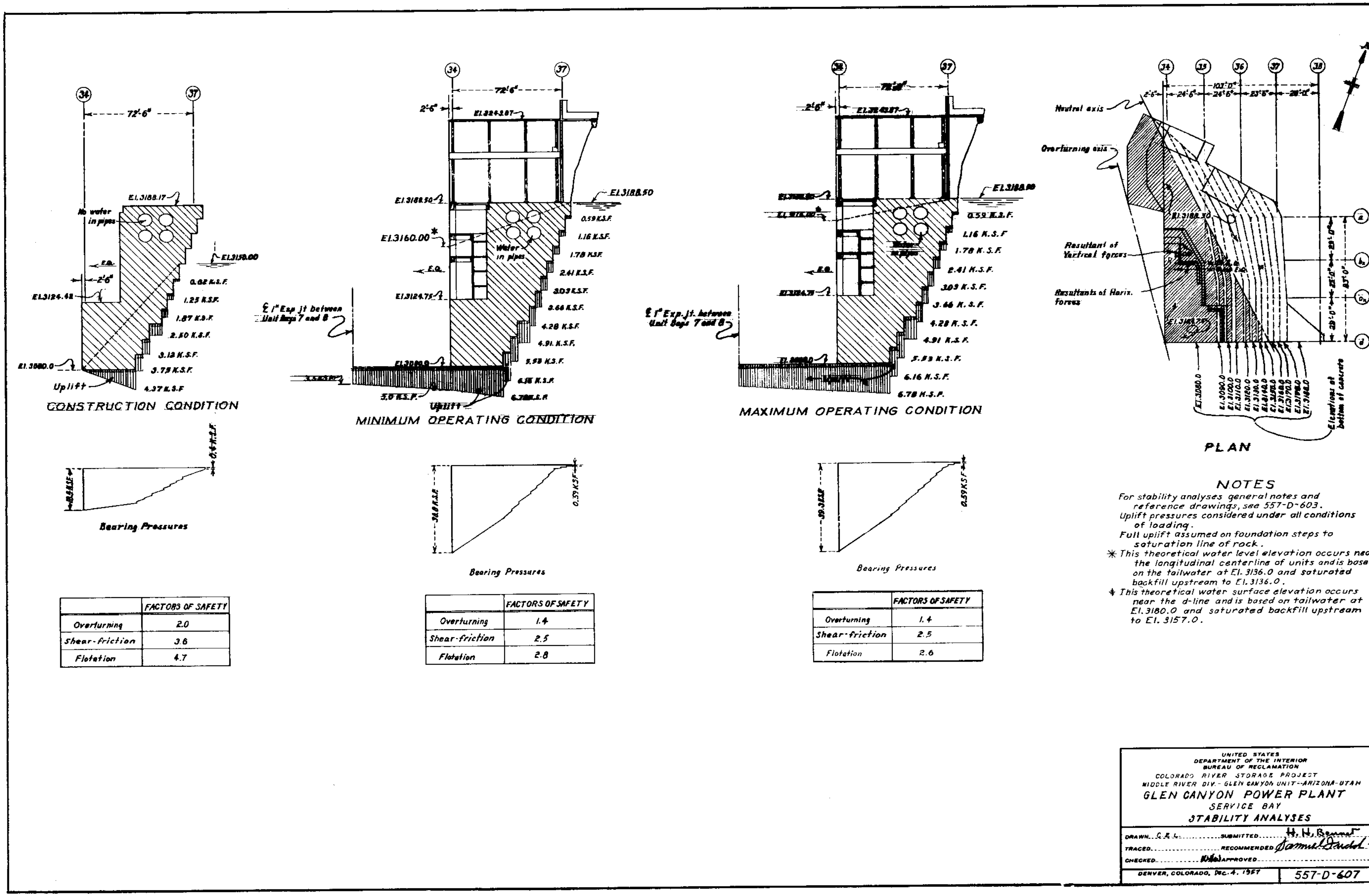


Figure 165.—Powerplant stability analysis—Service bay.

(d) *Machine Shop Bay.*—The stability analysis for the machine shop bay was similar to the service bay analysis. The same conditions were investigated; namely, (1) a construction condition, (2) the minimum operating condition, and (3) the maximum operating condition. Grouted 1-3/8-inch anchor bars were used to improve the stability of the structure. For the loading conditions and computed factors of safety, see figure 166.

74. DESIGN CONSIDERATIONS. (a) *General.*—The principal data pertaining to the structural design for the unit bays, the control area, the service bay and the machine shop bay are shown on the design data sheets (see figs. 167, 168, and 169). These sheets show the live loads and special loads for each floor, loading conditions for design, miscellaneous loads, assumptions for stability analyses, allowable increase of unit stresses, minimum requirements for temperature reinforcement, and estimated sizes and weights of necessary equipment.

The powerplant was designed for concrete having an ultimate compressive strength of 3,000 pounds per square inch at 28 days and an allowable working stress in the reinforcement of 20,000 pounds per square inch tension in flexure and 16,000 pounds per square inch tension in web reinforcement.

The original design was based on estimated weights and sizes of equipment. When the final data submitted by the manufacturers were in excess of the estimated values, the design was reviewed and revised accordingly. The structural arrangement drawings for the powerplant show the framing, member sizes, mass concrete, first-stage and second-stage concrete, and the embedded equipment (see figs. 170 through 177).

(b) *Mass Concrete.*—The final location and design of the river outlet pipes under the service bay and machine shop bay dictated the use of mass concrete from sound rock in the riverbed to the top floors of the machine shop and service bay. The mass concrete was placed against the left canyon wall to form the back wall and floors for the service bay and machine shop bay. A temperature control study of the mass concrete was made to determine what magnitude of volume change the foundation or mass concrete would undergo and through what period of time.

Based on a detailed analysis of this temperature study, the following decisions were reached:

(1) The mass concrete placed beneath all powerplant structures would have a strength of 3,000 pounds per square inch at 180 days except the top 10 feet of exposed surfaces where the mass concrete would have a strength of 3,000 pounds per square inch at 28 days.

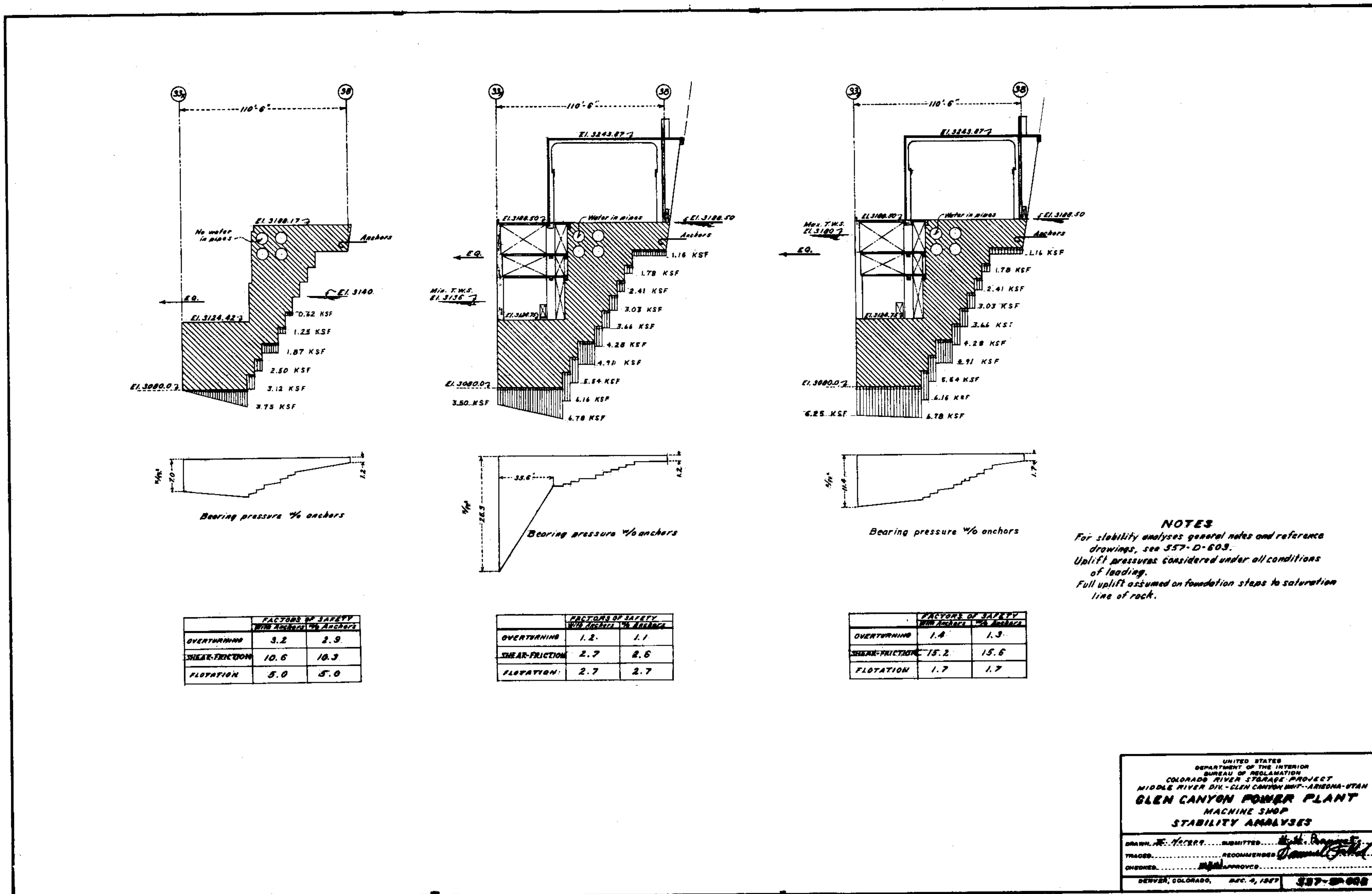
(2) The mass concrete beneath the machine shop bay and the service bay would be cooled while the mass concrete beneath units 1 through 8 would not be cooled.

(3) The initial placing temperature of the concrete would be maintained at 50° F. by precooling the aggregate and water if necessary. The concrete would be placed in 5-foot lifts at 3-day (minimum) intervals. By using cooling coils at the top of each lift and running 4 gallons per minute of water at 35° F. through the pipes, the maximum temperature of the concrete could be limited to 72.5° F. at 8 days. The temperature would then start dropping until it reached the mean annual temperature of 62° F. at about 26 days after placing. Since the cooling coils would be used and concrete placed in the machine shop river wall any time after 8 days, the foundation of mass concrete would be contracting by cooling and thereby putting the river wall in compression and assuring a more watertight structure.

(4) An investigation of the contraction or expansion of the mass concrete showed an expected value of about one-eighth of an inch. It was felt that this amount would not open any vaults for access to any joints in the steel outlet pipes.

(5) The decision was also made that a corkboard joint was not required between the machine shop wall and the gravity wall at the m-line and that there would be contraction joints at the d-line and m-line and these joints would not be grouted. It was further decided that with artificial cooling of the mass concrete, no expansion joint would be necessary in the machine shop structure.

To make the mass concrete and the main units act as a single structure, 1-3/8-inch dowels were used to engage the block of mass concrete between the a- and b-lines of the main units. The number, size, and length of dowels required to engage the mass concrete block between the a- and b-lines were designed from the maximum stresses determined from the various stability analyses. The maximum tensile stresses were computed on the assumption that the base would take no tension and that full uplift would be developed over the cracked area. Keys were also used to tie the mass concrete to the powerplant concrete.



	FACTORS OF SAFETY	
	With Anchors	W/o Anchors
OVERTURNING	3.2	2.9
SHEAR-FRICTION	10.6	10.3
FLOTATION	5.0	5.0

	FACTORS OF SAFETY	
	With Anchors	W/o Anchors
OVERTURNING	1.2	1.1
SHEAR-FRICTION	2.7	2.6
FLOTATION	2.7	2.7

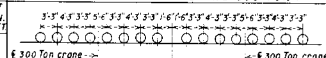
	FACTORS OF SAFETY	
	With Anchors	W/o Anchors
OVERTURNING	1.4	1.3
SHEAR-FRICTION	15.2	15.6
FLOTATION	1.7	1.7

Figure 166.—Powerplant stability analysis—Machine shop.

LOADS

ELEV.	LOCATION	LIVE LOAD LB./SQ. FT.	SPECIAL LOADS	FLOOR FINISH OR SURFACING	WEIGHT FL. FIN. LB/SQ. FT.
	CABLE AND ELEVATOR TOWER				
3289.62	Roof	50		4-Ply roofing	
3279.12	Elevator equipt. floor	250		6" Conc. surfacing	75
3267.87	Storage	150			
3262.87	Toilet and janitor room	100			
3257.25	Elevator lobby	200			
3257.25	Transformer room	200			
3250.37	Toilet and janitor room	100			
3244.24	Elevator lobby	200			
3238.96	Cable shaft - Cable duct	200	Cables		
3239.12	Toilet and janitor room	100			
3233.50	Elevator lobby	100			
3227.98	Office and storage	100			
3222.25	Elevator lobby	100			
3216.63	Telephone room and storage	150			
3211.00	Elevator lobby	100			
3205.38	Office, storage and cable shaft	100			
3199.75	Elevator lobby	100			
3194.13	Toilet and janitor room	100			
	CONTROL AREA				
3276.18	Roof (Unit Bays 1 and 2)	50		3" Cover slab	22
3257.25	Unit Bay 1	100			
3257.25	Unit Bay 2	70			
3245.50	Toilet (Unit Bay 1)	100			
3244.24	Unit Bays 1 and 2				
	Conc floor slabs & steel beams	200	D		
	Struct. steel building frame	150			
3244.12	Observation deck (Unit Bay 3)	200		3 1/2" Cover slab	45
	UNIT BAYS				
3244.06	Roof for Units 4 thru 8 (19)-(33)	50		3" Cover slab	22
3188.50	Transfer deck (e)-(f)	500	B,C,F	6" Cover slab	75
3188.50	Transformer deck (d)-(e)	500	C,F	6" Cover slab	75
3188.50	Balcony (33)-(33)	1,000	E,F,G,J	4" Conc. surfacing	50
3188.50	Balcony - all others	200		4" Conc. surfacing	50
3183.50	Toilet and janitor room (1)-(2)	100			
3168.50	Generator floor (a)-(d)	1,000	D,E,G	4" Conc surfacing	50
3168.50	Electrical gallery (d)-(e)	500*	D,H	4" Conc. surfacing	50
3168.50	Toilet and janitor room (1)-(2)	100			
3153.50	Governor gallery (d)-(e)	500	D	4" Conc. surfacing	50
3153.50	Service gallery (a)-(b)	300		Monolithic	
3153.50	Sewage collection and storage (1)-(2)	100			
3138.50	Pipe gallery (d)-(e)	300		Monolithic	
3124.75	Unwatering gallery (d)-(e)	200		Monolithic	
3124.75	Penstock gallery (a)-(b)	Unlimited		Monolithic	
	Stairs - below El. 3153.50	100		Monolithic	
	Stairs - above El. 3153.50	100		2" Bonded conc.	25

A - POWER HOUSE CRANE - TWO 300 TON ‡



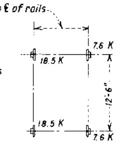
104 Kips per wheel max. + 10.4 Kips per wheel impact.
47 Kips per wheel min. + 4.7 Kips per wheel impact.
Longit. and lateral thrust 10.4 Kips per wheel one side and 4.7 Kips per wheel opposite side.

Wt. of crane = 505 Kips

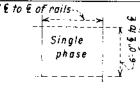
B - GANTRY CRANE - 10 TON

9'-0" ℄ to ℄ of rails

Crane wt. 21 Kips

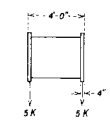


C - POWER TRANSFORMER ‡

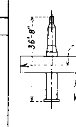


Kips per wheel incl. oil 65 K
Add 10 % impact
Wt. (incl. oil) 265 Kips

D - CABLE REEL ‡

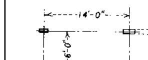


E - ROTOR AND SHAFT ‡



Rotor and shaft
Wt. = 1,150 Kips

F - H-20 TRUCK

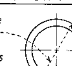


AXLE LOADS
Standard A.A.S.H.O.
Add 10 % for impact

G - STATOR ‡

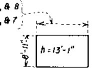
Stator outside dia = 35'-5"

Wt. = 1,000 Kips



H - AIR CIRCUIT BREAKERS ‡

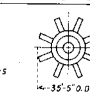
6'-8" Units 2, 4, 6, & 8
9'-8" Units 1, 3, 5, & 7



Static wt., 19 Kips
Imp. load, 9.8 Kips

J - UPPER BEARING BRACKET ‡

Wt. 218 Kips



EQUIPMENT WEIGHT

12" H.P. Twin strainer .... 4.9 K.

LOADING CONDITIONS FOR DESIGN

1. Dead loads.
2. Live loads on various floors and roof.
3. Wind loads.
4. Crane loads and thrusts.
5. Earthquake forces.
6. Temperature variations.
7. Backfill loads.
8. Maximum tailwater.
9. Normal tailwater.
10. Minimum tailwater.

ASSUMPTIONS FOR STABILITY ANALYSIS

Case I. All first stage concrete and superstructure completed; draft tube unwatered; normal tailwater; backfill upstream side to El. 3125.00 and assumed dry above El. 3020.00 with and without full uplift over entire base.

Required factor of safety against flotation = 1.10.

Case II, IIA, IIB, IIC, III, IIIA, IV, IVA, & V
For loads see Dwg. 557-D-603 & 604

UNIT DATA

Generator rating .... 112,500 KW
Volts 13,800
Speed .... 150 rpm
Turbine rating .... 155,500 hp
Rated head .... 450 ft.
Turbine discharge .... 3,580 cfs

MISCELLANEOUS LOADS

Temperature variation, ±40°F on exterior surf. ±20°F on interior surf.
Wind: 20 lbs. per sq. ft. of projected area.
Backfill: (horizontal pressure).
Saturated earth: 85 lbs./c.f. equiv. fluid pressure.
Dry earth: 40 lbs./c.f. equiv. fluid pressure.
Earthquake: Acceleration = 0.10 g.
Maximum tailwater, El. 3180.00
Normal tailwater, El. 3143.00 ‡
Minimum tailwater, El. 3137.00 ‡
Maximum water pressure in spiral case under water hammer conditions = 300 psi.
Pressure in spiral case during placing of conc. = 225 psi.
Hydraulic thrust plus wgt. turbine rot. pts. = 975 K. ‡
Thrust from each servo-motor = 275 Kips. ‡
Max. torque to be transmitted to the generator foundation = 73,784,382 ft-lbs.

ALLOWABLE INCREASE OF UNIT STRESSES

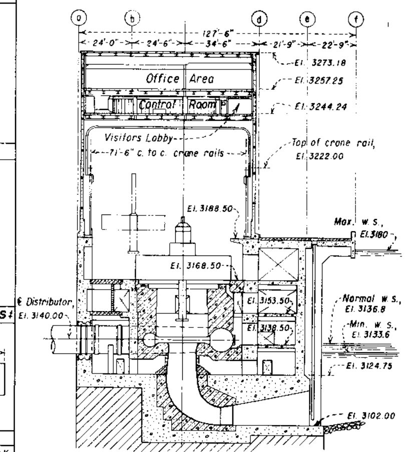
Dead, live, wind or normal water .... 0 %
Dead, live, wind, and max. annual water .... 25 %
Dead, live, and temperature .... 25 %
Dead, live, normal water, and earthquake .... 33 %
Dead, live, normal water, earthquake, and temp. .... 50 %
Temporary and construction loads .... 33 %

STRESSES

Reference drawing 103-D-563
Concrete: 3000 psi (ultimate at 28 days)
Reinf.: f_s = 20,000 psi in flexure
f_s = 16,000 psi web reinf.

TEMPERATURE REINFORCEMENT

Use: .005 Min. for roof and deck.
.004 Min. for superstructure walls.
.003 Min. for exterior members.
.002 Min. for interior members.
Increase percent of reinforcement adjacent to areas of restraint.



TRANSVERSE SECTION THRU UNITS

2-15-62 D W.D.B.	REARRANGED ROOMS AND LIVE LOADS FOR FALLOUT PROTECTION.
8-8-61 D E.G.M.	REVISED LOADS TO INCLUDE CONTROL AREA AND CABLE AND ELEVATOR TOWER.
5-15-62 D W.D.B.	REVISED POWER TRANSFORMER DIMENSIONS AND WEIGHTS.

DWG. 557-D-149 SUPERSEDES DWG. 557-D-148

NOTES

‡ Estimated
* Includes allowance for parts storage and handling governor

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZONA-UTAH

GLEN CANYON POWER PLANT
UNIT BAYS
STRUCTURAL DESIGN DATA

DRAWN G.L.S.-W.D.B. SUBMITTED [signature]
TRACED J.A.S. RECOMMENDED [signature]
CHECKED [signature] APPROVED [signature] ACTG. CHIEF DESIGN. ENGR.

DENVER, COLORADO, JULY 1, 1957 | 557-D-149

Figure 167.—Powerplant structural design data—Unit bays.

LOADS

ELEV.	LOCATION	LIVE LOAD LBS/SQ.FT.	SPECIAL LOAD	FLOOR FINISH OR SURFACING	WT. FL.FIN. LBS/SQ.FT.
3243.9±	Service Bay roof (34) - (37)	300		1" Insulation, 4 ply roof, tar, gravel.	8
3231.69	Service Bay roof (37) - Rock	400		5'-0" of 1½" to 3" Coarse aggregate	600
3188.50	Service floor	1,000	B,C,D,F,G,H	4" Conc. surf.	50
3177.50	Platform	500		Monolithic	
3168.50	Passageway and service area	500		4" Conc. surf.	50
3153.50	Passageway and storage area	300		Monolithic	
3124.75	Passageway and storage area	Unlimited	On rock	4" Conc. surf.	50
	Sewage collection tank area	Unlimited	On rock	4" Conc. surf.	50
	Stair runs and landings	100		Monolithic below El. 3153.50; 2" finish above El. 3153.50	25

LOADING CONDITIONS FOR DESIGN

1. Dead loads.
2. Live loads on various floors on roof.
3. Wind loads.
4. Crane loads and thrusts.
5. Backfill loads.
6. Uplift.
7. Earthquake forces.
8. Temperature variations.

ASSUMPTIONS FOR STABILITY ANALYSES

Constr. condition: See drawing ( )
Min. operation condition: Structure complete; no live load; uplift based on a waterline gradient of El. 3136.00 at the upstream and downstream faces of the main unit structure, at the main unit edge of the block, to El. 3188.50 at all steps on the abutment side. Factor against flotation desired = 1.25.
Max. operation condition: same except for waterline gradient of El. 3157.00 at the upstr. face to El. 3180.00 at the downstream face of the main unit edge of mass conc. block. Factor against flotation desired = 1.1.
See Stability Analyses Dwg. 557-D-

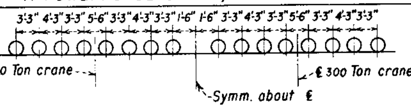
MISCELLANEOUS LOADS

Wind: 20 lbs. per sq. ft. of projected area.
Earthquake: 0.10g.
Uplift: Full uplift over entire base area including foundation steps based on water line gradients described under "Assumptions for stability analyses".
Backfill: (horizontal pressure)
Dry = 40 lbs./cu. ft. equiv. fluid press.
Saturated = 85 lbs./cu. ft. equiv. fl. press.
Temperature variations:
± 30° F. on exterior horizontal surfaces
± 10° F. on interior horizontal surfaces.

ALLOWABLE INCREASE OF UNIT STRESSES

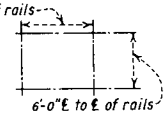
Dead, live, wind or normal water	0%
Dead, live, wind and max. annual water	25%
Dead, live and temperature	25%
Dead, live, normal water and earthquake	33%
Dead, live, normal water, earthquake and temperature	50%
Dead and temporary, or construction loads	33%

A-POWERHOUSE CRANE, TWO 300 TON ‡



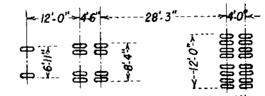
104 Kips per wheel max. + 10.4 Kips per wheel impact.
47 Kips per wheel min. + 4.7 Kips per wheel impact.
Longitudinal and lateral thrust 10.4 Kips per wheel one side and 4.7 Kips per wheel opposite side.
Wt. of crane = 505 Kips.

B-POWER TRANSFORMER ‡



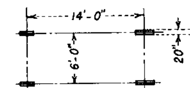
Kips per wheel incl. oil 65K
Add 10% impact
Wt. (incl. oil) 265 Kips

C-100 TON TRAILER



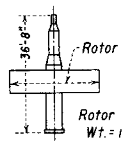
14 Kips 53 Kips ea. 72.5 Kips ea.
Add 10% for impact

D-H-20 TRUCK



8 Kips 32 Kips
AXLE LOADS
Standard A.A.S.H.O.
Add 10 % for impact.

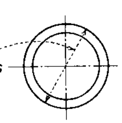
E-ROTOR AND SHAFT ‡



Rotor and shaft
Wt. = 1,150 Kips

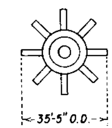
F-STATOR ‡

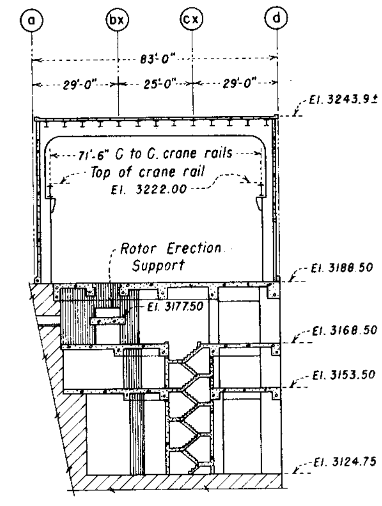
Stator outside dia. = 35'-5"
Wt. = 1,000 Kips



G-UPPER BEARING BRACKET ‡

Weight = 218 Kips





TRANSVERSE SECTION
THRU SERVICE BAY

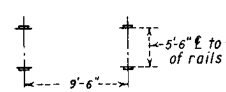
TEMPERATURE REINFORCEMENT

Use:
.005 Min. for deck
.004 Min. for superstructure walls
.003 Min. for exterior members
.002 Min. for interior members
Increase reinf. adjacent to area of restraint.

STRESSES

Reference drawing 103-D-563
Concrete: 3000 psi (ultimate @ 28 days) ✱
Reinf.: f_s = 20,000 psi in flexure
f_s = 16,000 psi web reinf.

H-TRANSFER CAR ‡



1.6 Kips per wheel
Add 10% for impact.

NOTES

‡ Estimated.
✱ Mass Concrete below El. 3115.00 : 3000 psi (ultimate @ 180 days).

11-19-62 D W.D.B.	REVISED B AND H LOADS.

DWG. 557-D-476 SUPERSEDES DWG. 557-D-150

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZONA-UTAH
GLEN CANYON POWER PLANT
SERVICE BAY
STRUCTURAL DESIGN DATA

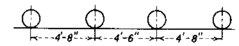
DRAWN W.C.S. SUBMITTED
TRACED J.A.S. RECOMMENDED
CHECKED R.G.M. APPROVED
CHIEF DESIGN. ENGR.

DENVER, COLORADO, AUG. 8, 1961 | 557-D-476

Figure 168.—Powerplant structural design data—Service bay.

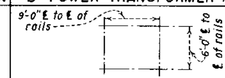
LOADS					
ELEV.	LOCATION	LIVE LOAD LBS/SQ.FT.	SPECIAL LOADS	FLOOR FINISH OR SURFACING	WT. FL. FIN. LB/SQ.FT.
3243.9	Roof (35)-(38)	300		1" Insulation, 4 ply roof, tar, gravel	8
3231.69	Roof (38)-Rock	400		5'-0" of 1½" to 3" coarse aggregate	600
3209.25	Office roof	200		Monolithic	–
3198.50	Office floor	200		Monolithic	–
3188.50	Machine shop	750	C, F, G, H	4" Conc. surf.	50
	Deck	500	B, D, E	8" Cover slab	100
	Tool room	100		4" Conc. surf.	50
	Toilet	100		4" Conc. surf.	50
3168.50	Electrical equipment	500	K	4" Conc. surf.	50
	Toilet	100		4" Conc. surf.	50
3153.50	Compressor room	300	J	Monolithic	–
	Sump pump motor room	300		Monolithic	–
3124.75	Sump pump room	300		4" Conc. surf.	50
	Oil purifier room	Unlimited	On mass	4" Conc. surf.	50
	Oil storage rooms	Unlimited	On mass	4" Conc. surf.	50
3091.50	Sump	Unlimited	On mass	Monolithic	–
	Stair runs and landings	100		Monolithic below El. 3153.50-2" Finish above El. 3153.50	25

A-MACHINE SHOP CRANE-ONE 75 TON



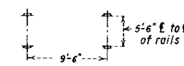
Wheel loads – Kips per wheel
64.0 Maximum + 6.4 impact
20.7 Minimum + 2.1 impact
Longitudinal and lateral thrusts: 6.4 under max. wheel, 2.1 under minimum wheel.
Crane weight: 165 Kips

B-POWER TRANSFORMER †



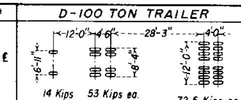
Kips per wheel incl. oil 65 K
Add 10% impact
Wt. (incl. oil) 265 Kips

C-TRANSFER CAR †



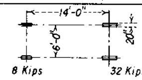
1.6 Kips per wheel
Add 10% for impact

D-100 TON TRAILER



Add 10% for impact

E-H-20 TRUCK



AXLE LOADS
STANDARD A.A.S.H.O.
Add 10% for impact

F-42" ENGINE LATHE

G-RADIAL DRILL

(Future)

H-16 Ft. BORING MILL

300 Kips

LOADING CONDITIONS FOR DESIGN

1. Dead loads
2. Live loads on floors and roofs
3. Wind loads
4. Crane loads and thrusts
5. Backfill loads
6. Earthquake forces
7. Hydrostatic loads on walls
8. Temperature variations
9. Uplift

ASSUMPTIONS FOR STABILITY

Constr. condition: Mass concrete complete, no struct. concrete. Rock saturated to El. 3140.00, no tailwater. Uplift over entire base.
Min. operation condition: Structure complete, no live loads. Rock saturated to El. 3188.50, minimum T.W. surface El. 3136.00. Uplift over entire base.

Max. operation condition: Structure complete, no live loads. Rock saturated to El. 3188.50, maximum T.W. surface El. 3180.00. Uplift over entire base.

See Dwg 557-D-608

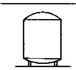
J-500 CFM COMPRESSOR

11 Kips-See Mfr. dwgs.

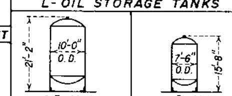
K-SAND FILTERS

(Future)

Operating wt. = 24 Kips



L-OIL STORAGE TANKS



2 Req'd.
10,000 Gal. Trans. Oil
Wt. of tank = 12.3 K
Wt. of oil = 75 K

2 Req'd.
4,000 Gal. Lub. Oil
Wt. of tank = 5.4 K
Wt. of oil = 30.4 K

TEMPERATURE REINFORCEMENT

Use:
.005 Min. for deck
.004 Min. for superstr. walls
.003 Min. for exterior members
.002 Min. for interior members
Increase reinf. adjacent to area of restraint.

MISCELLANEOUS LOADS

Wind: 20 lbs. per sq. ft. of projected area.
Earthquake: 0.10 g.
Backfill (horizontal pressure)
Dry = 40 lbs. per cu. ft. equiv. fluid press.
Saturated = 85 lbs. per cu. ft. equiv. fluid press.
Maximum tailwater, El. 3180.00
Normal tailwater, El. 3143.00
Minimum tailwater, El. 3136.00
Temperature variations
±30° F. on exterior hor. surfaces.
±10° F. on interior hor. surfaces.
Full uplift over entire base area, including foundation steps, based on water line gradients.

ALLOWABLE INCREASE OF UNIT STRESSES

Dead, live, wind or normal water ____ 0%
Dead, live, wind, and max. annual water __ 25%
Dead, live, and temperature ________ 25%
Dead, live, normal water and earthquake ____________ 33%
Dead, live, normal water, earthquake and temperature ______ 50%
Dead and temporary, or construction loads ____________ 33%

STRESSES

Reference drawing ____ 103-D-563
Concrete: 3000 psi (ultimate at 28 days)
Reinf: f_s=20,000 psi in flexure
f_s=16,000 psi web reinf.

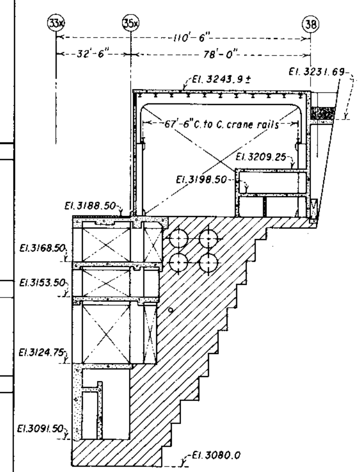
EQUIPMENT WEIGHT

Water supply pump ________ 8.4 K

NOTES

† Estimated

ALWAYS THINK SAFETY



11-19-62 D. W.D.B.	REVISED B, C, D, H, J, AND L LOADS.

DWG. 557-D-477 SUPERSEDES DWG. 557-D-151

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZONA-UTAH
GLEN CANYON POWER PLANT
MACHINE SHOP BAY
STRUCTURAL DESIGN DATA

DRAWN R.C.M. SUBMITTED [signature]
TRACED H.T.J. RECOMMENDED [signature]
CHECKED W.C.S. APPROVED [signature]
CHIEF DESIGN ENG'R.

DENVER, COLORADO, AUG. 8, 1961 — 557-D-477

Figure 169.—Powerplant structural design data—Machine shop bay.

(c) *Foundation Analysis.*—Since the base slab for units 3 through 8 would be on mass concrete, the base pressure used for design on these units was the uplift pressure only.

For units 1 and 2, the base slab would be on rock; therefore, a check was made to see what type of loading should be placed on the base slab. Hetenyi's[2] "Beams on Elastic Foundations" was used to determine this value. Also used in the foundation analysis was a report by the Board of Consultants,[3] dated May 7, 1957. The values from this report that were used are listed below:

Modulus of elasticity of rock, p.s.i.	500,000
Poisson's ratio for rock	0.06
Modulus of elasticity of concrete, p.s.i.	3,000,000
Poisson's ratio for concrete	0.20

The base slab and draft tube pit slab for units 1 and 2 were designed using the elastic foundation method mentioned above. The powerplant loads were distributed down through the plant to the rock and deflections determined at the required points. Using these deflections and the proper end conditions, the resulting moments and shears were computed.

(d) *Substructure.*—The substructure is that part of the powerplant below elevation 3124.75. The substructure concrete for the unit bays is the massive concrete below elevation 3124.75 and forms the base slab and draft tube openings. The substructure concrete of units 1 and 2 sets on rock and on mass concrete for units 3 through 8. It is designed to carry the loads from the superstructure, intermediate structure, plant equipment, and second-stage concrete down to the foundation.

(e) *Intermediate Structure.*—The intermediate structure is that part of the powerplant from elevation 3124.75 to elevation 3188.17. The design of the intermediate structure concrete was divided into four sections; namely, unit bays 2 through 8, unit bay 1, the service bay, and the machine shop bay.

(1) *Unit bays 2 through 8* were designed as three parts—the upstream gallery, the downstream gallery, and the cross walls.

The upstream gallery was designed as a two-floor bent or frame with various loads from the superstructure columns, floor loads and saturated backfill on the upstream wall combined. The beams in the upstream galleries at elevations 3153.50 and 3168.50 presented an unusually difficult design problem. A large tensional force, in addition to moment and shear forces, was transmitted to the beams from the maximum loading condition. The combination of tensional and shear forces resulted in a cracked section. Therefore, it was necessary to design the beam stirrups and longitudinal reinforcement to handle these stresses in addition to the moment stresses.

The downstream gallery was designed as a four-floor bent or frame with a cantilever deck. Various loads from the superstructure columns, transformer deck, floor loads, tailwater loads, etc., were combined. The transformer deck was designed for a transformer inplace and another transformer moving on the transfer rails.

The designs for the upstream gallery, the transformer deck, and the downstream gallery were solved by the trial-load method using an electronic digital computer. Spot checks of the machine results were made to verify the validity of the solutions.

Since the unit bay system is used for the powerplant, two cross walls are required at each expansion joint. Adjacent cross walls are separated by a 1-inch expansion joint; so calculations were made for the moments, shears, deflections, and necessary reinforcement for a temperature rise of 20° F., using the Bureau's Design Standards No. 9.[4] The cross walls were also designed to carry the load from the second-stage concrete to the base slab.

(2) *Unit bay 1* was designed similar to units 2 through 8 with the addition of a comprehensive analysis of the effect of the cable and elevator tower for shear, foundation pressures, and earthquake. The visitor's walkway or balcony also created very special design problems.

[2] Hetenyi, Miklos I., "Beams on Elastic Foundations," 1946.

[3] "Report on Foundation Adequacy and Design Considerations of Glen Canyon Dam," Julian Hinds, Chairman, John J. Hammond, Raymond E. Davis, Edward B. Burwell, Jr., John W. Vanderwilt, Denver, Colo., May 7, 1957.

[4] "Design Standards No. 9, Buildings—Chapter 3, Concrete Details—Figure 19, Design of Walls Adjacent to 1-Inch Expansion Joints," Bureau of Reclamation, October 27, 1959.

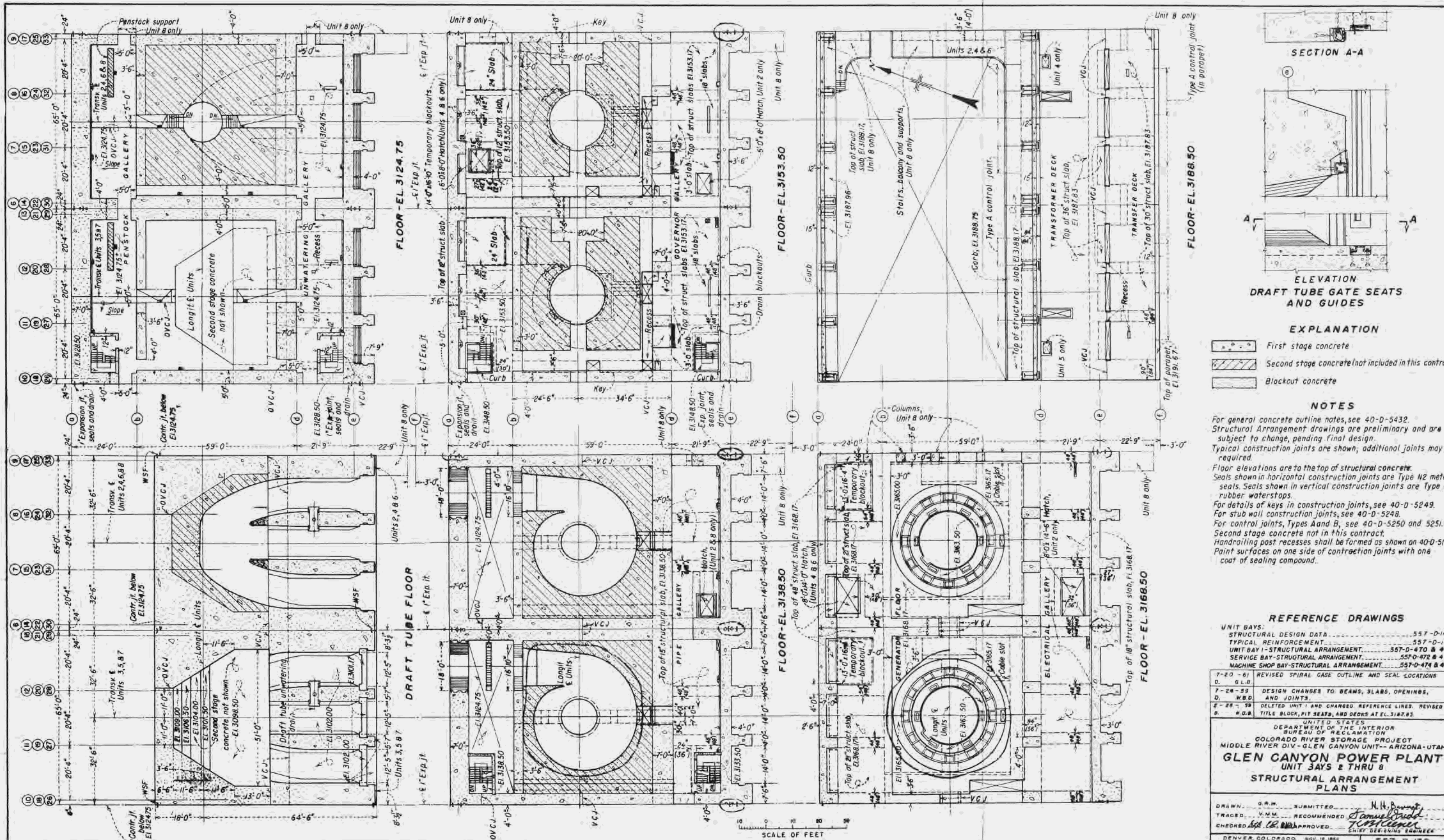


Figure 170.—Powerplant structural arrangement plans—Unit bays 2 through 8.

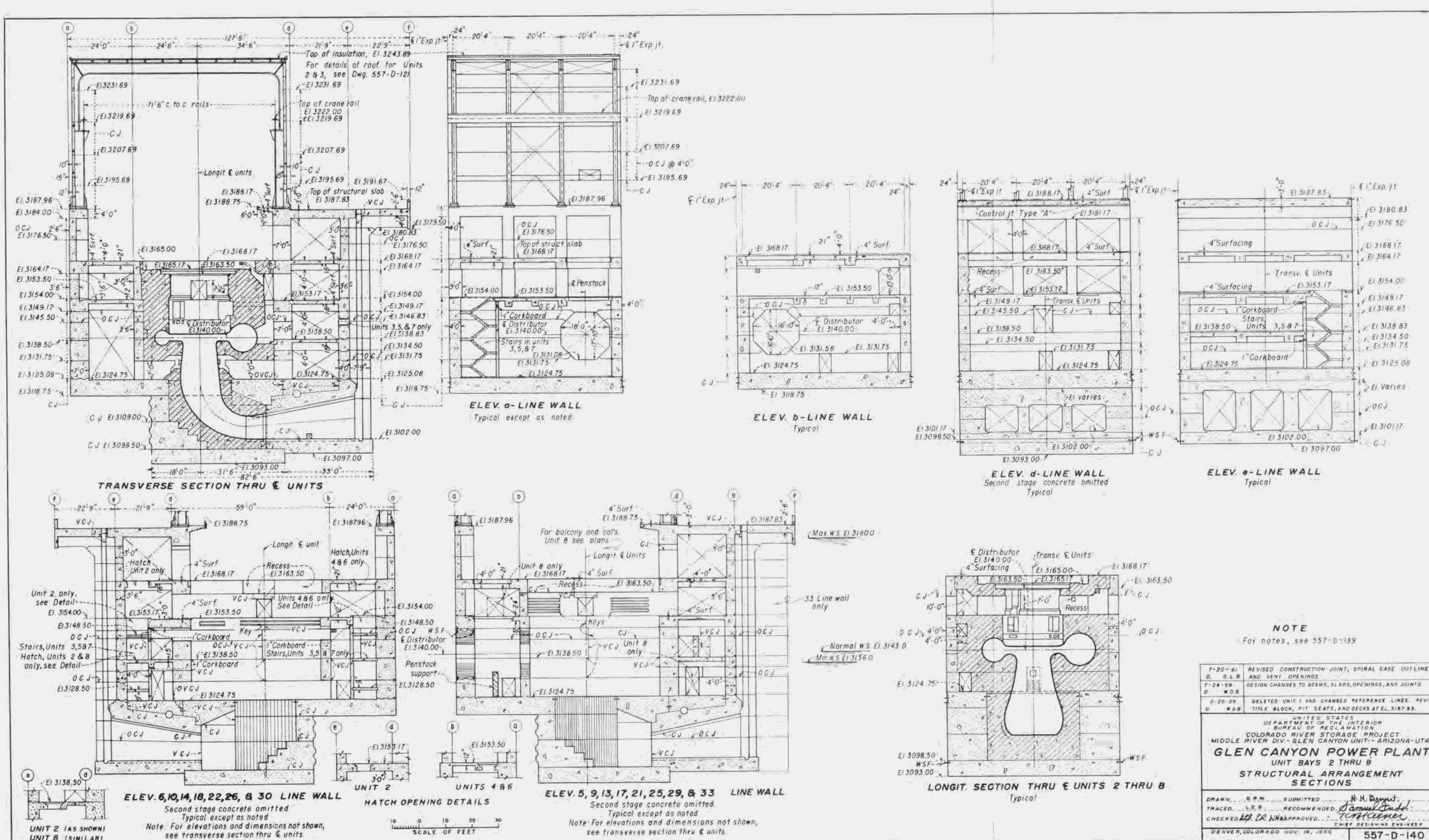


Figure 171.—Powerplant structural arrangement sections—Unit bays 2 through 8.

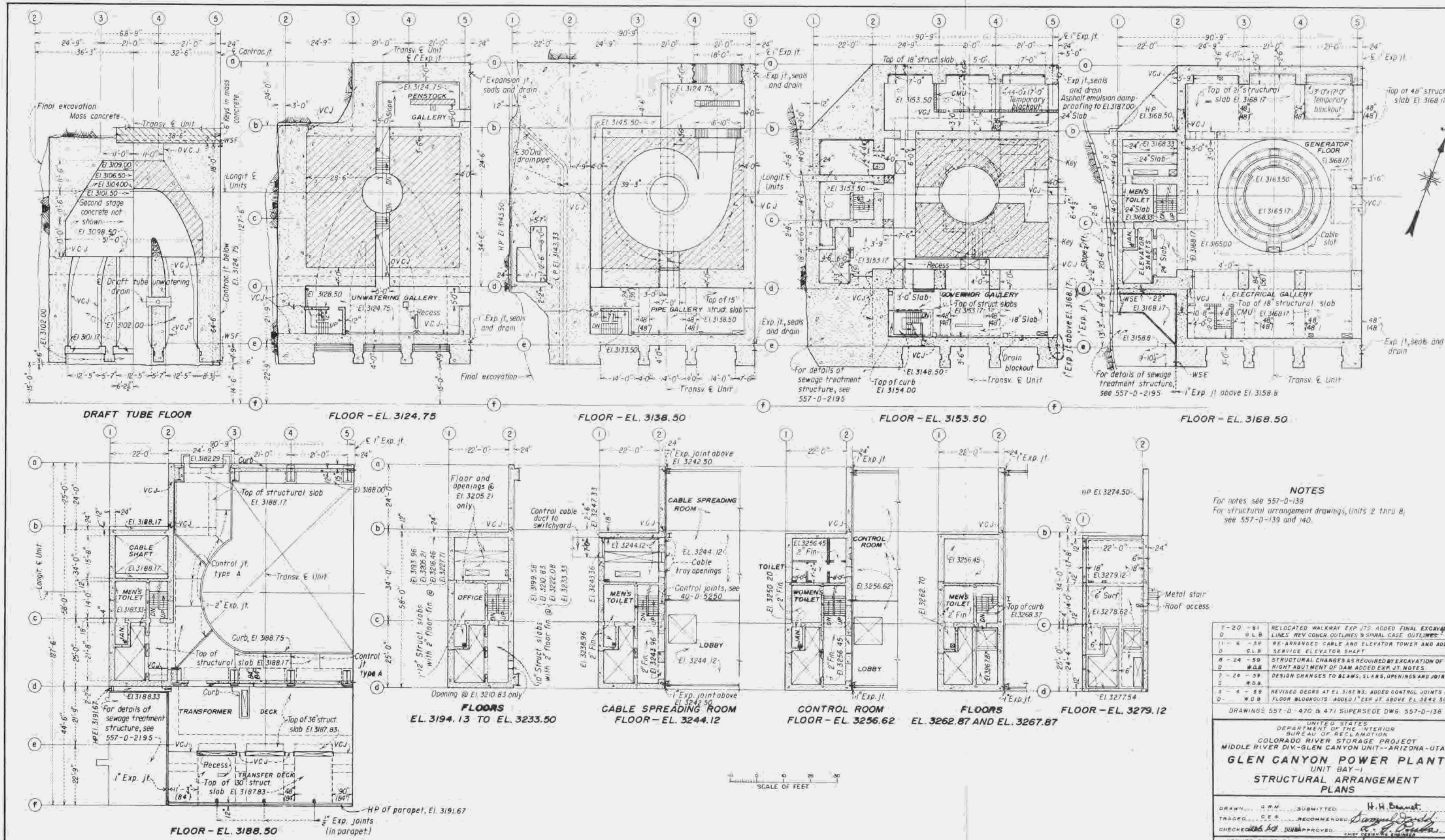


Figure 172.—Powerplant structural arrangement plans—Unit bay 1.

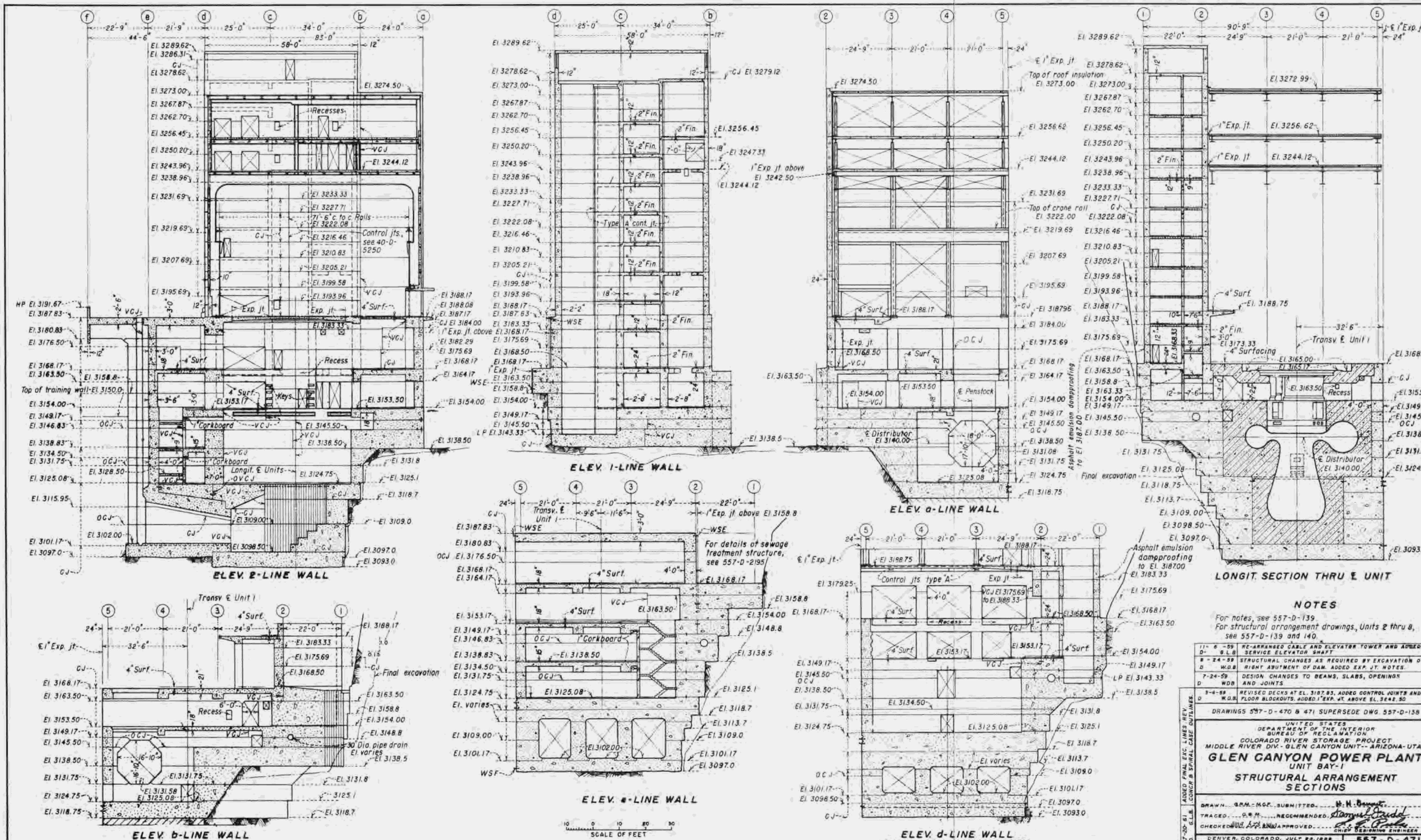


Figure 173.–Powerplant structural arrangement sections–Unit bay 1.

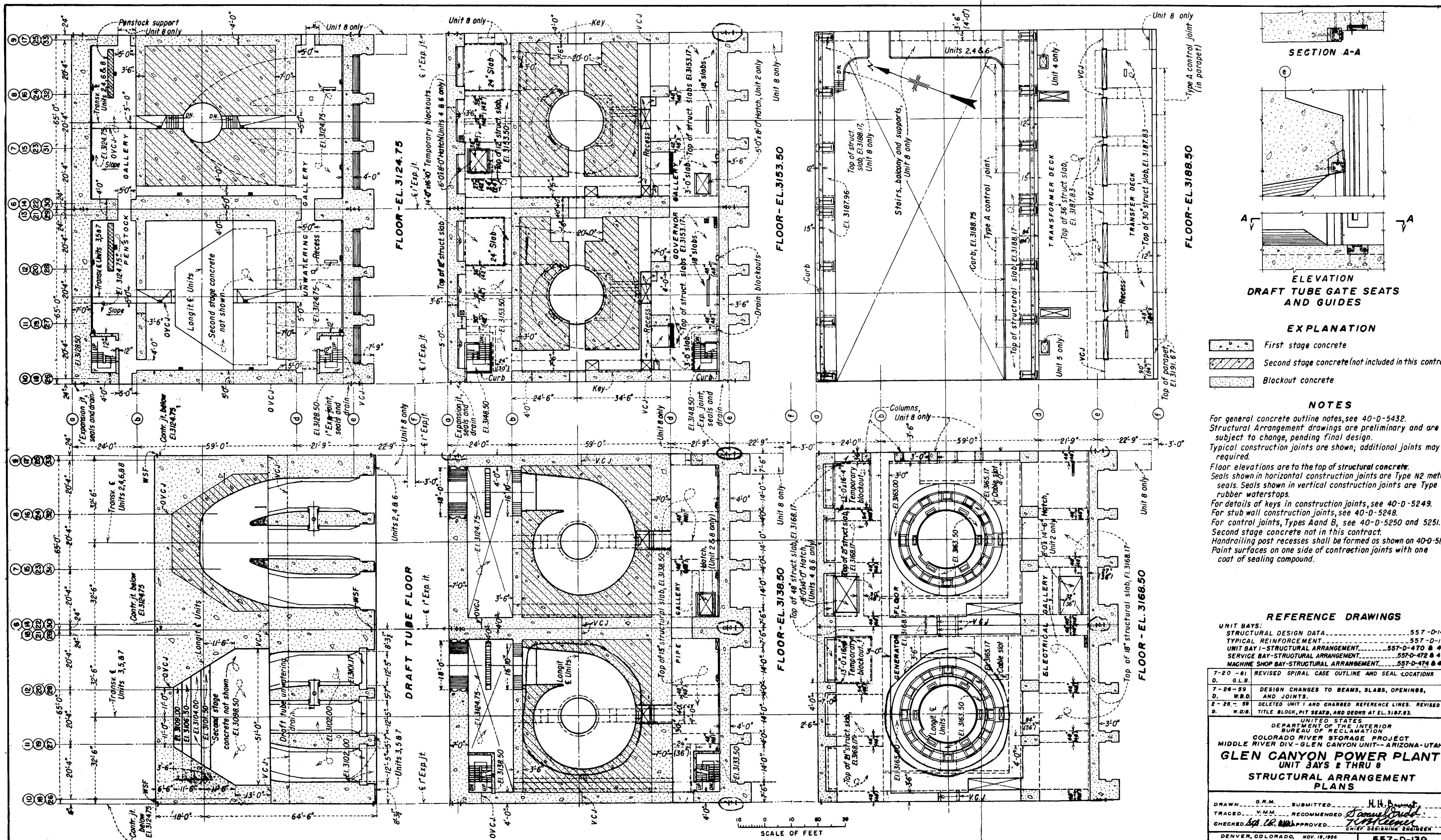


Figure 170.—Powerplant structural arrangement plans—Unit bays 2 through 8.

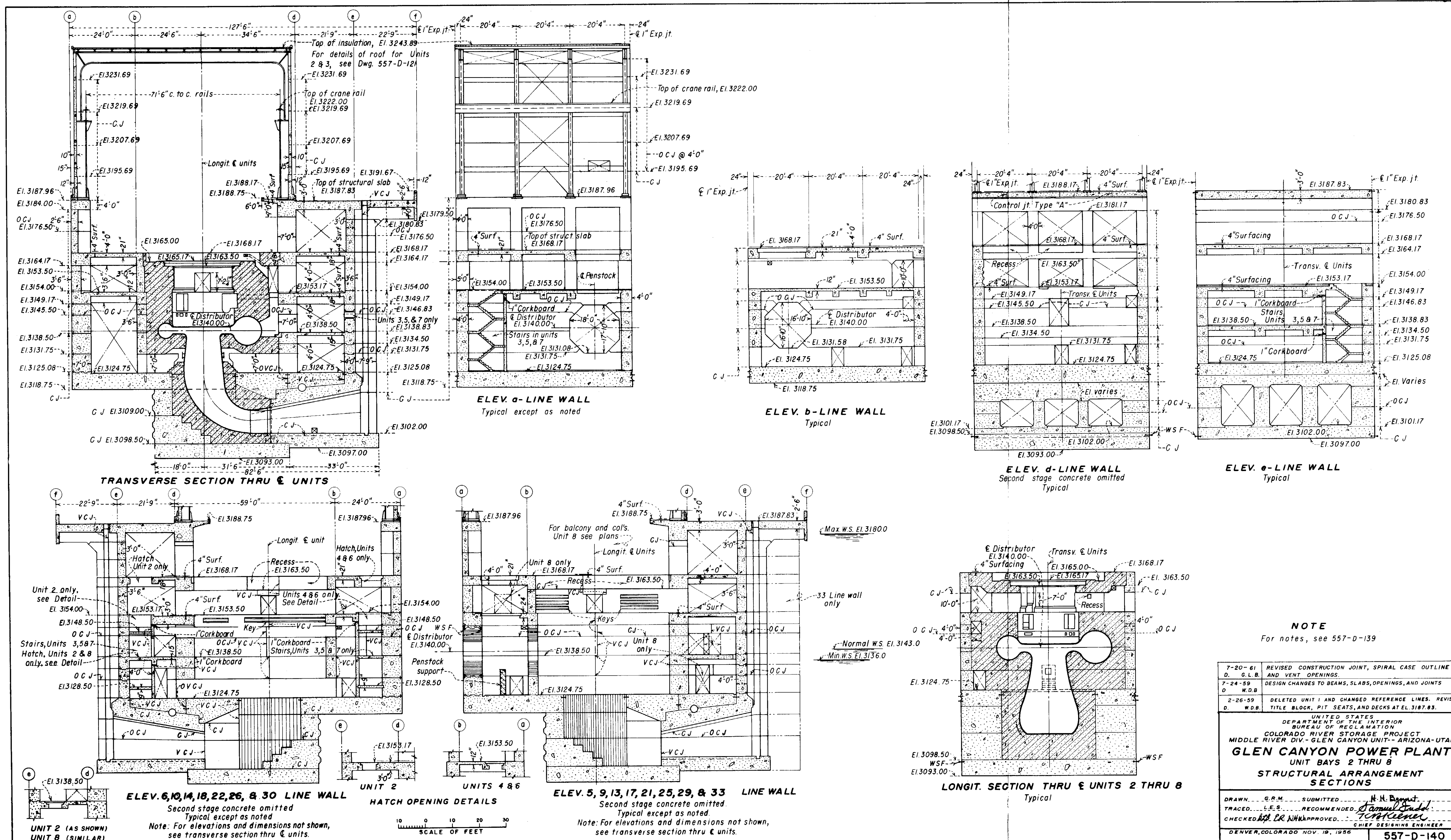


Figure 171.—Powerplant structural arrangement sections—Unit bays 2 through 8.

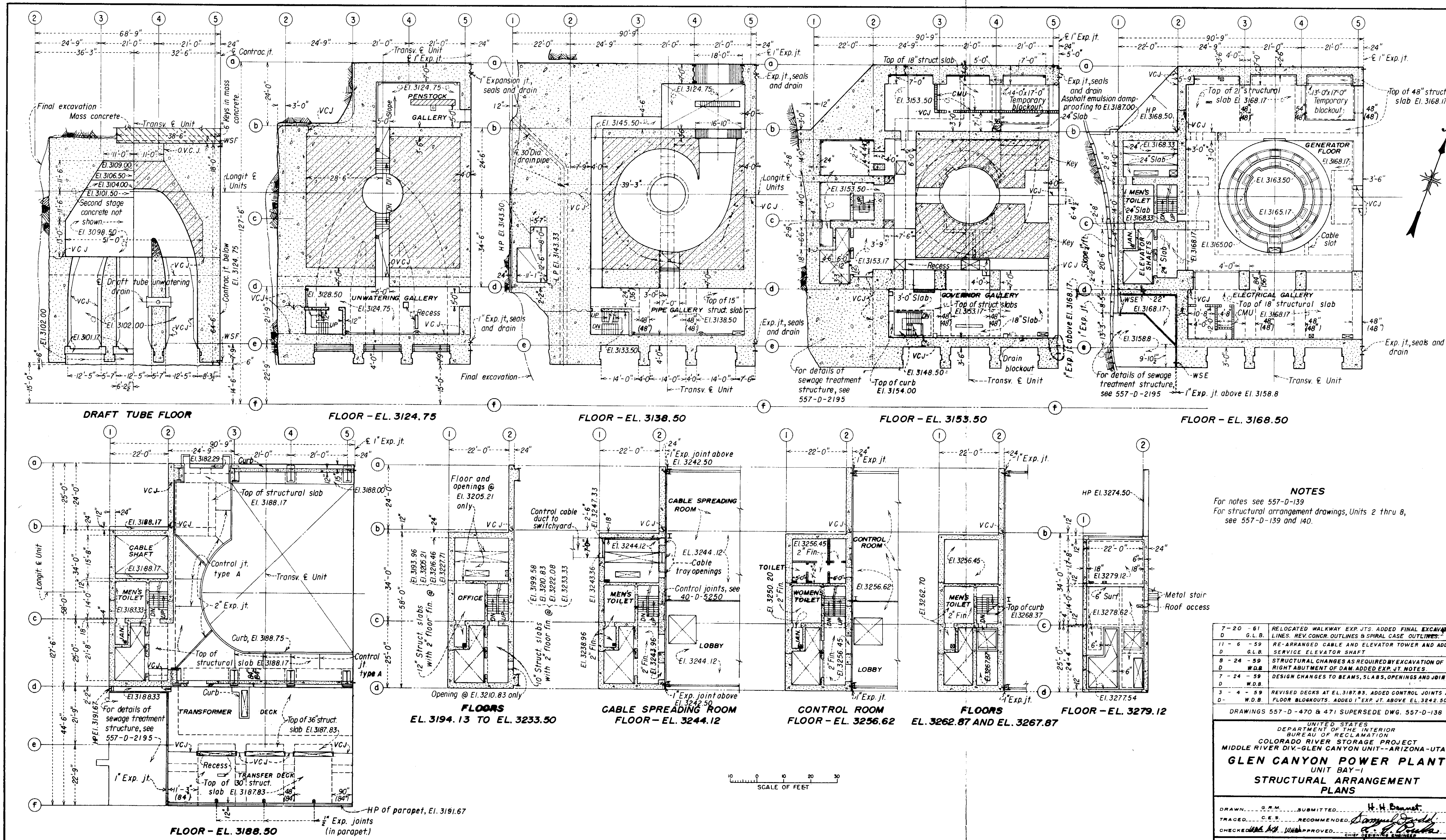


Figure 172.—Powerplant structural arrangement plans—Unit bay 1.

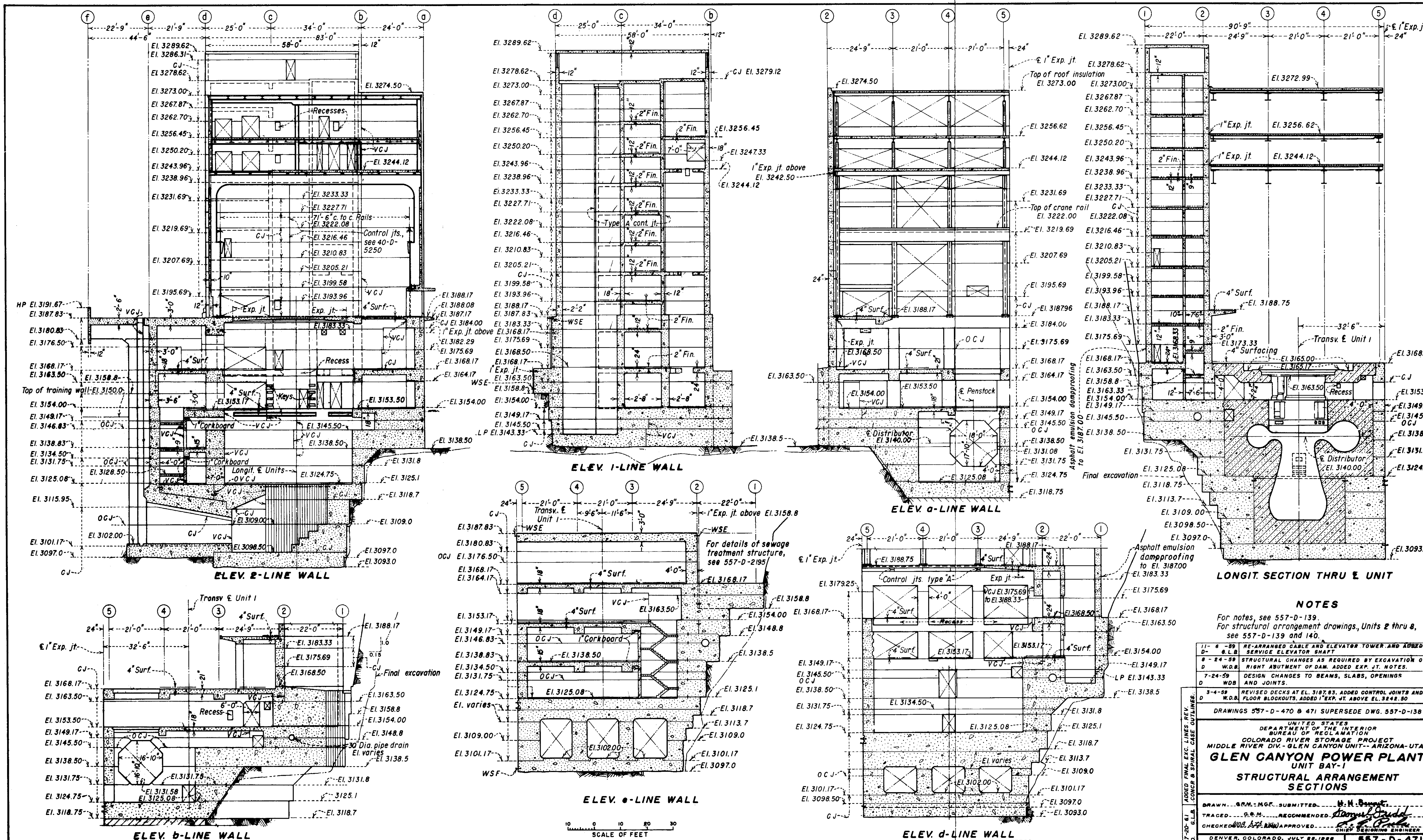


Figure 173.—Powerplant structural arrangement sections—Unit bay 1.

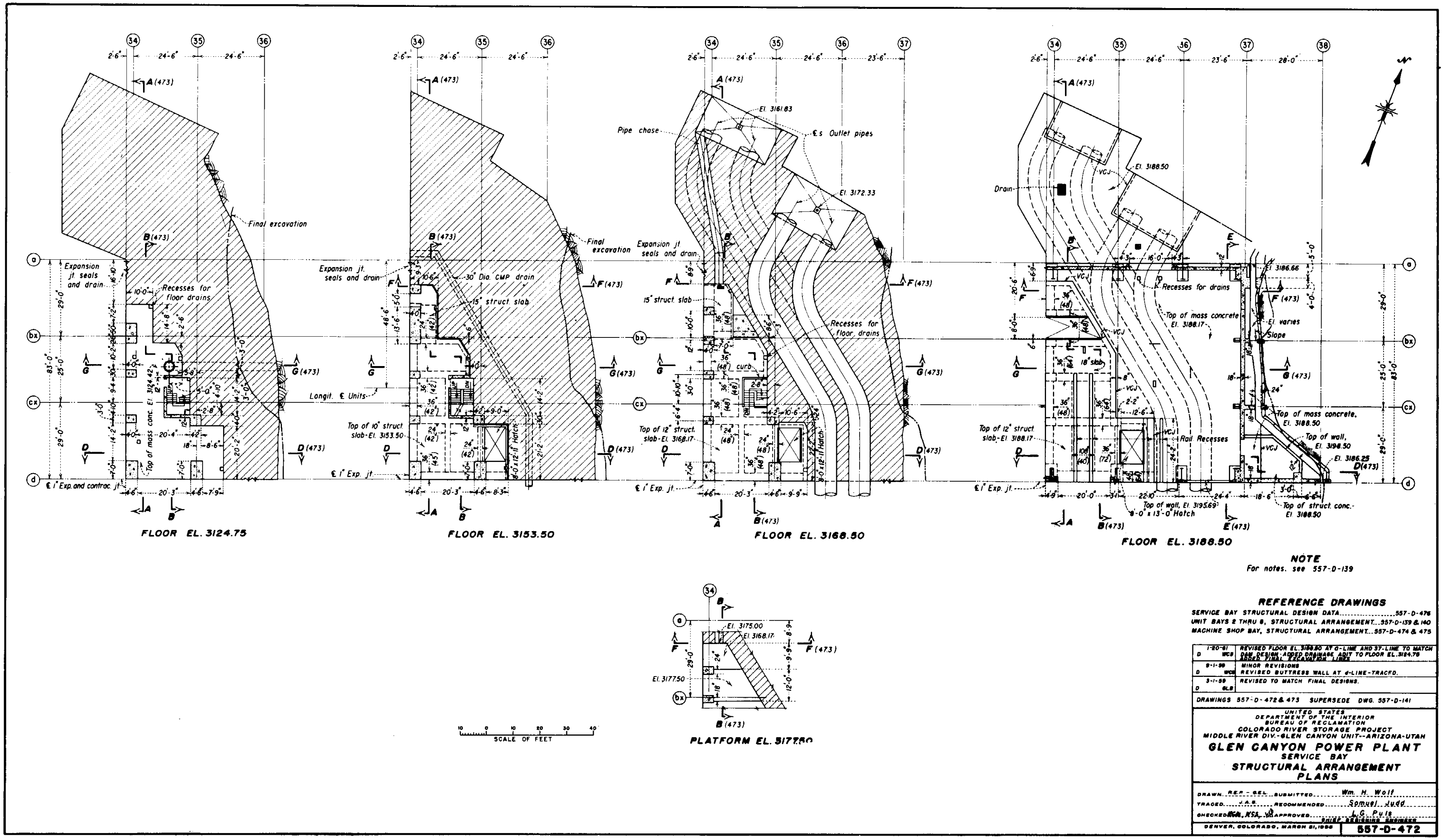


Figure 174.—Powerplant structural arrangement plans—Service bay.

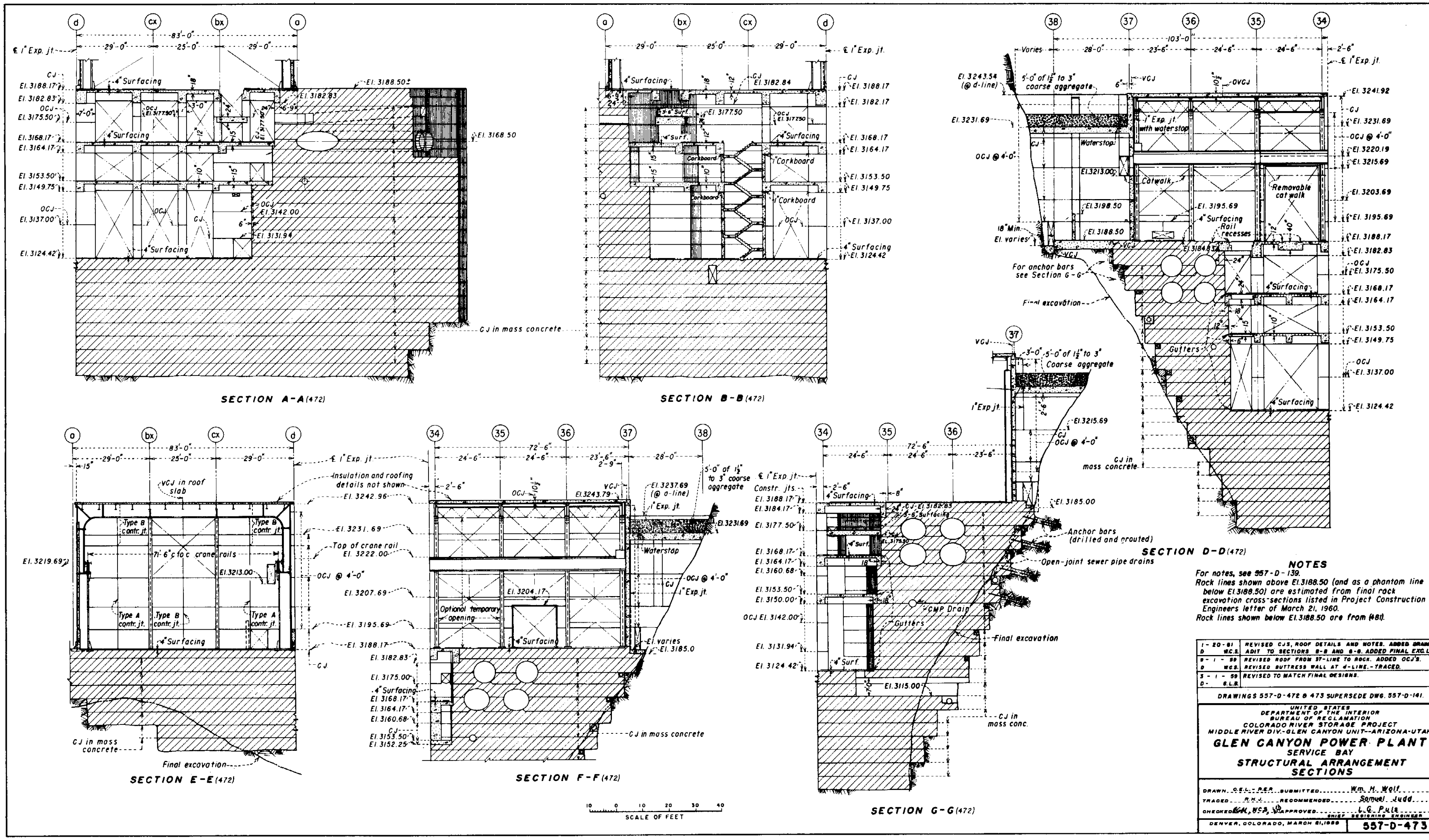


Figure 175.—Powerplant structural arrangement sections—Service bay.

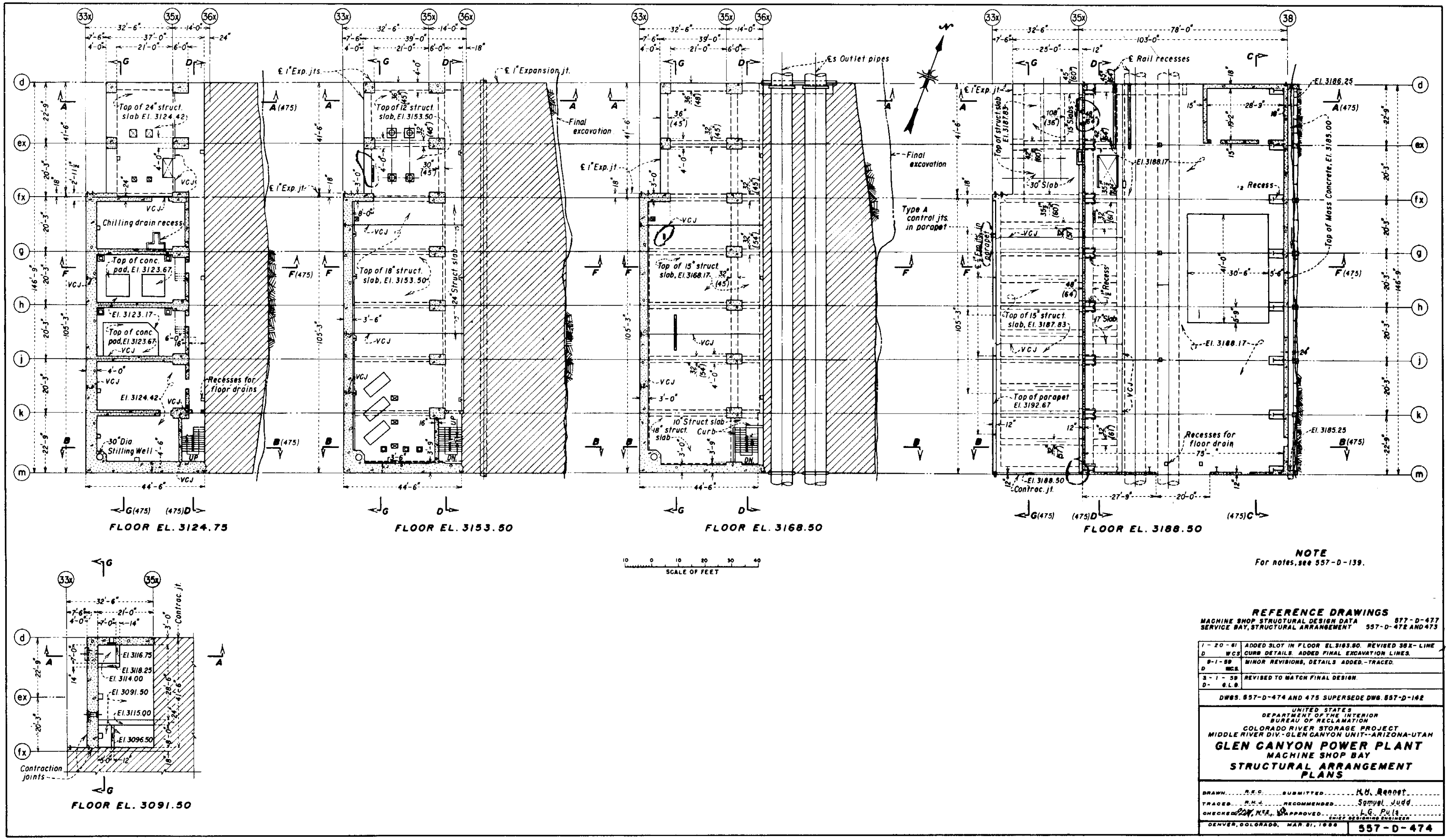


Figure 176.—Powerplant structural arrangement plans—Machine shop bay.

(3) *The service bay* intermediate structure was designed as a beam and slab system framing into the mass concrete anchor block for the river outlet pipes. The unusually heavy floor loads expected on the elevation 3188.50 floor were due to the concentration of the rotor weight on three sides of the slot provided in the floor to handle the erection of the rotor.

(4) *The machine shop bay* intermediate structure was also designed as a beam-and-slab system framing into the mass concrete block. The principal design problems for this system came from the expected high tailwater on the river wall side. Cross walls were used between the river wall and the mass concrete to act as rib stiffeners between elevations 3124.75 and 3153.50. The top deck was designed to carry heavy equipment to the service bay and for the expected temperature variations on the exposed deck.

Expansion and contraction of deck slabs have important effects on the design of the structural framework of the powerplant. A study was made of the heat distribution for a concrete slab held at a constant temperature on one side and subject to a daily variation of temperature on the other. The slab under consideration was the deck of the machine shop bay at elevation 3188.50. The transformer deck slab of the unit bays and the service bay is a similar type structure and the temperature study would apply to all decks at elevation 3188.50.

From the annual reports from the weather station at Lee Ferry, a maximum mean daily temperature of 104° F. was selected and a corresponding minimum temperature of 73° F. Considering the solar radiation on a horizontal slab, the temperature would increase about 16° F., making the maximum temperature 120° F. The temperature on the top of the slab was assumed to vary daily as a sinusoidal curve, and the temperature below the slab was assumed to hold at a constant 80° F.

The heat flow through the slab was analyzed according to Schmidt's method of conductivity. Two cases were studied and programed on the electronic digital computer. These were a hot and cold cycle for a slab 36 inches thick. The maximum and minimum values of the temperature 6 inches below the surface of the slab and the average temperature of the slab were desired. The computation program was set up to compute temperatures at 2-inch increments of depth through the slab and at time intervals of 15 minutes. At each time, the temperature 6 inches below the surface and the average temperature of the entire slab were recorded and printed for a 6-day cycle. During the sixth day, the average temperature of the slab varied from 95° to 100° F. The temperature at a point 6 inches below the exposed surface, or approximately the bottom of the cover slab, was 114° F. Since the major portion of the heat is in the top 6 or 8 inches, it is reasonable to assume that the temperature at the center of the 36-inch structural slab would be less than the 100° F. average.

The design of the exposed deck was for a temperature rise or drop of 20° F. based on the 80° F. at the bottom of the slab and the 100° F. average temperature at the center of the slab. This is assumed to be a conservative approach for the climatic conditions encountered at the powerplant.

The location of the large air compressors on the elevation 3153.50 floor created unusual foundation design problems to eliminate vibration or reduce it to a minimum. Special drawings and details were required for the installation of a 16-foot boring mill in the machine shop.

(f) *Superstructure.*—The superstructure is that part of the powerplant above elevation 3188.50. The design of the superstructure was separated into five different areas for continuity of design. These design areas were the cable and elevator tower, the control area, the superstructure walls, the cast-in-place roof slabs for the service bay and machine shop bay, and the buttress walls against the left canyon wall.

(1) *The cable and elevator tower* is located on the right side of unit bay 1 from elevation 3153.50 to elevation 3289.62. This tower is a tall, narrow, rectangular structure. The major design problems encountered here were associated with the shears due to the unusual shape of the structure and the earthquake design.

In the shear investigation for the cable and elevator tower walls, the problem was to determine the magnitude and direction of the principal stresses as computed by the use of Mohr's circle.[5] The shear

[5] Timoshenko, S., and Goodier, J. N., "Theory of Elasticity" McGraw-Hill Book Co., 1951.

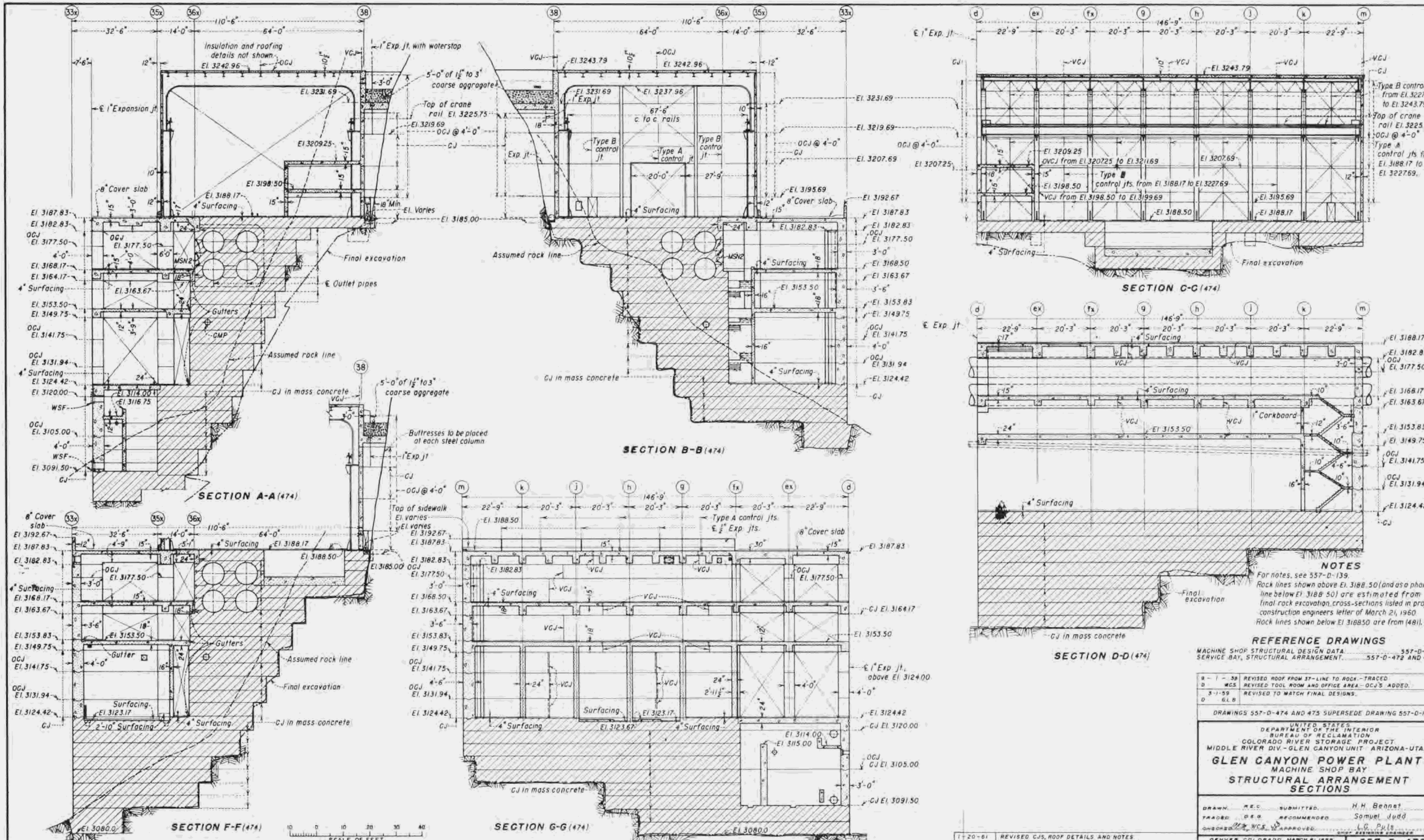


Figure 177.—Powerplant structural arrangement sections—Machine shop bay.

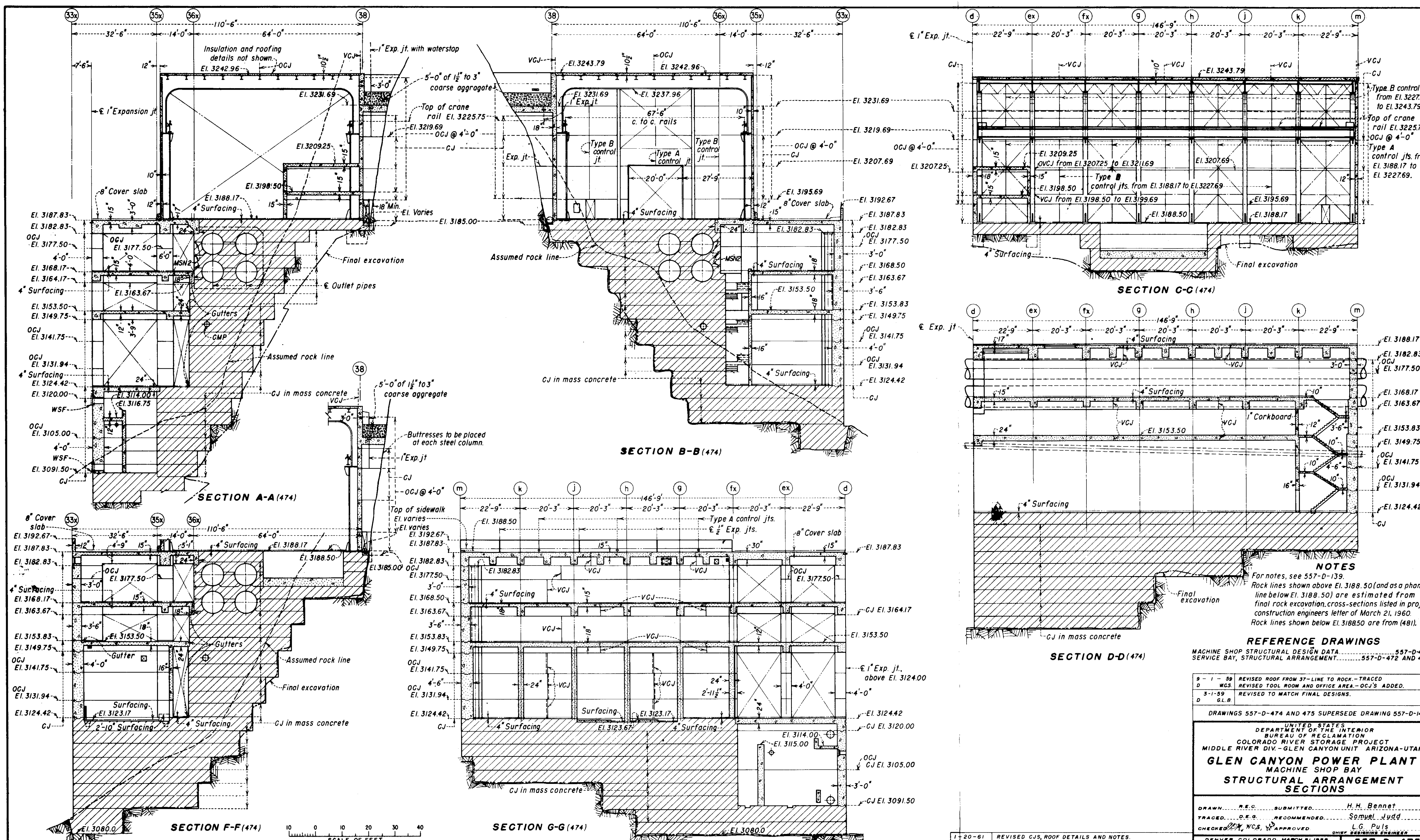


Figure 177.—Powerplant structural arrangement sections—Machine shop bay.

stresses were to be determined by assuming the tower hangs up, as implied from the stability analysis, and the total weight of the tower is taken through the cross walls below elevation 3188.50. The problem was solved in the following manner:

The shear diagram and then the shearing stress were determined by the formula $v = \frac{VQ}{It}$ where V = the total shear, Q = the static moment about the neutral axis of the section considered, I = the moment of inertia of the entire section, and t = the effective width of the section at the point considered.

The normal stress was determined for the cross walls at the section considered using the formula for bending and direct stress $f = \frac{P}{A} \pm \frac{Mc}{I}$. To determine M, full uplift was assumed for tailwater to elevation 3168.50 and water in the right canyon wall or abutment to elevation 3168.50.

The magnitude and direction of the principal stresses were then determined by combining the normal stress and the shearing stress at each point in question by the use of Mohr's circle.

The earthquake design, deflections were computed for the unit bay 1 superstructure wall and the cable and elevator tower, using an earthquake load equivalent to an assumed acceleration of 0.1 gravity. The frequency of vibration for each structure was also computed. A comparison of these values prompted a decision to tie the two structures together at the elevation 3244.24 floor with rigid steel connections. These connections will make the structures act together and minimize the destructive effect of two structures "slapping" together during an earthquake.

(2) *The control area*—That portion of the powerplant called the control area refers to the two-floor structure located on top of the unit bays 1 and 2 superstructure. The floors are located at elevations 3244.24 and 3257.25. The floor at elevation 3244.24 is a lightweight reinforced concrete slab on steel reinforcing floor forms. The maximum allowable live load is 200 pounds per square foot. The floor at elevation 3257.25 is a 12-1/2-inch-thick regular reinforced concrete floor slab. The purpose of this unusually thick concrete slab is to provide fallout protection for the operating personnel on the floor below.

(3) *Superstructure walls.*—The superstructure of the service bay, the machine shop bay, and the main units was designed as a structural steel frame with concrete curtain walls. The curtain walls are simply supported between the webs of the steel columns and are designed for wind and earthquake loads. These walls are carried to the top of the steel frame in the service bay, unit bays 3 through 8, and on the tailrace wall of the machine shop bay. In unit bays 1 and 2, the superstructure walls are continued past the top of the main frame to the roof of the control area. These walls above the main frame are also of the curtain wall design and similar to the rest of the superstructure.

(4) *The service bay and machine shop bay roof.*—The roof system over the structural steel frame for the service bay and machine shop bay is a 10-1/2-inch cast-in-place reinforced concrete roof slab designed for a live load of 300 pounds per square foot. This slab is to protect personnel and equipment against small rocks. For protection against large rocks, a 5-foot cushion of coarse aggregate is placed on top of a 30-inch reinforced concrete slab. This cushioned area is placed between the left canyon wall and the superstructure walls of the machine shop bay and the service bay. A 1-inch expansion joint is placed in the slab to prevent the transfer of this load to the superstructure frame.

(5) *Buttress walls.*—To support the 30-inch concrete slab and 5-foot-deep gravel cushion located between the machine shop and service bay walls and the left canyon abutment, 18-inch-thick buttress walls were provided at each column line from the upstream wall of the service bay to the downstream wall of the machine shop bay. One-inch expansion joints were used in each buttress wall to prevent loads being transferred to the superstructure frame. Openings were located at the bottom of each buttress next to the structure to provide a walkway and a drainage gutter for the full length of the machine shop and service bay.

(g) *Structural Steel Framing.*—The superstructure of the powerplant has a structural steel frame with concrete panel walls between the columns, and with the outside flanges of the columns and the eave purlins exposed. The building has eight unit bays, a service bay, a machine shop bay, and a structure referred to as the control area which is supported on the roof of unit bays 1 and 2. In addition there is a concrete structure, housing the elevator shafts, stairs, and control cable shaft, adjacent to unit bay 1. Hinged connections are provided between this structure and unit bay 1 to prevent impacting of one building against the other in the event of an earthquake.

The unit bays and the service bay each have four rigid-frame bents built up of welded steel plates. Diagonal bracing and struts between the bents are structural steel members. Structural steel purlins support the roof. The crane girders for the two 300-ton cranes in the powerplant are built up of welded steel plates. The details are shown on figure 178.

The machine shop has the same construction as the unit bays, with eight rigid-frame bents. The crane girder for the 75-ton machine shop crane is composed of wide-flange beams with angles welded to the top flanges.

The control area is a two-story steel frame building located on the roof of unit bays 1 and 2. The frames of this building have welded rigid connections to the powerplant frames, and welded rigid corners at the roof. The reinforcing metal floor pans for the floors and the precast concrete roof slabs are supported on structural steel beams.

The rigid frames were designed by the column analogy method. All structural steel was designed in accordance with the provisions of the 1956 fifth edition of the "Steel Construction Manual" of the American Institute of Steel Construction. Welding is in accordance with the applicable provisions of the American Welding Society's "Code for Arc and Gas Welding in Building Construction." Field connections were made with high-strength bearing bolts.

(h) *Roof Drainage.*—Drainage water is picked up at the roof drains in the unit bays and the elevator tower and carried to the downstream (south) side of the structure where it is collected into one vertical pipe in each bay. Each pipe runs horizontally under the transformer deck and discharges into the tailrace at elevation 3182.78.

In the service and machine shop bays the drainage water is picked up at the roof drains and carried to the east end wall where vertical pipes discharge it into the drainage trench between the building and the canyon wall.

(i) *Handrailing.*—Handrailing in the powerplant above floor elevation 3168.50, in the elevator tower, and on the stairs on the exterior of the upstream (north) wall of the powerplant is welded aluminum pipe railing. Handrailing in the powerplant below floor elevation 3168.50 and in the machine shop bay is welded black steel pipe. All stairs have handrailing. The aluminum handrailing on the balcony at floor elevation 3188.50 is made of welded aluminum pipe and is fitted with panels of woven aluminum mesh. The visitors walkways between the powerplant and the dam have handrailing made up of aluminum pipe fitted with aluminum grills. The observation deck on the roof of unit bay 3 has a fence composed of 3- by 3-inch-square steel tubing one-fourth inch thick with a woven aluminum mesh.

(j) *Structural Details.*—In designing large reinforced concrete buildings such as the Glen Canyon Powerplant, joints must be appropriately placed in the concrete to facilitate construction and to prevent destructive or unsightly cracking. Joints are also needed to separate buildings or different parts of the same building if excessive movement or operating equipment might transmit harmful vibration stresses or thrusts to each other if not separated. In Glen Canyon Powerplant four types of joints were used; namely, expansion, construction, contraction, and control joints.

(1) *Expansion joints* allow expansion and contraction of large structures and isolate certain areas in a building. These joints prevent cracking as well as insure against misalinement of equipment that would otherwise result from building distortion.

To separate each unit bay, the service bay, and the machine shop bay, 1-inch expansion joints were used. These expansion joints were formed by 1-inch corkboard joint filler extending from the substructure to the roof of each bay. They were sealed against water leakage by the use of two rubber waterstops, an asphalt seal, and a formed drain as shown on figures 179 and 180. The asphalt seals are provided with two 1/2-inch standard steam pipes with necessary fittings to which steam can be admitted to heat and liquify the asphalt.

Two-inch expansion joints, filled with sponge rubber, are used in the visitors' balcony in unit 1. These joints are required to keep the balcony from cracking if the movement of the tower is greater than that of the balcony.

One-inch joints filled with sponge rubber, and 1/2- and 1/4-inch joints filled with corkboard were used to isolate various areas throughout the powerplant.

(2) *Construction joints* were used where it was necessary to interrupt continuous placement of concrete, for structural reasons, and to reduce the

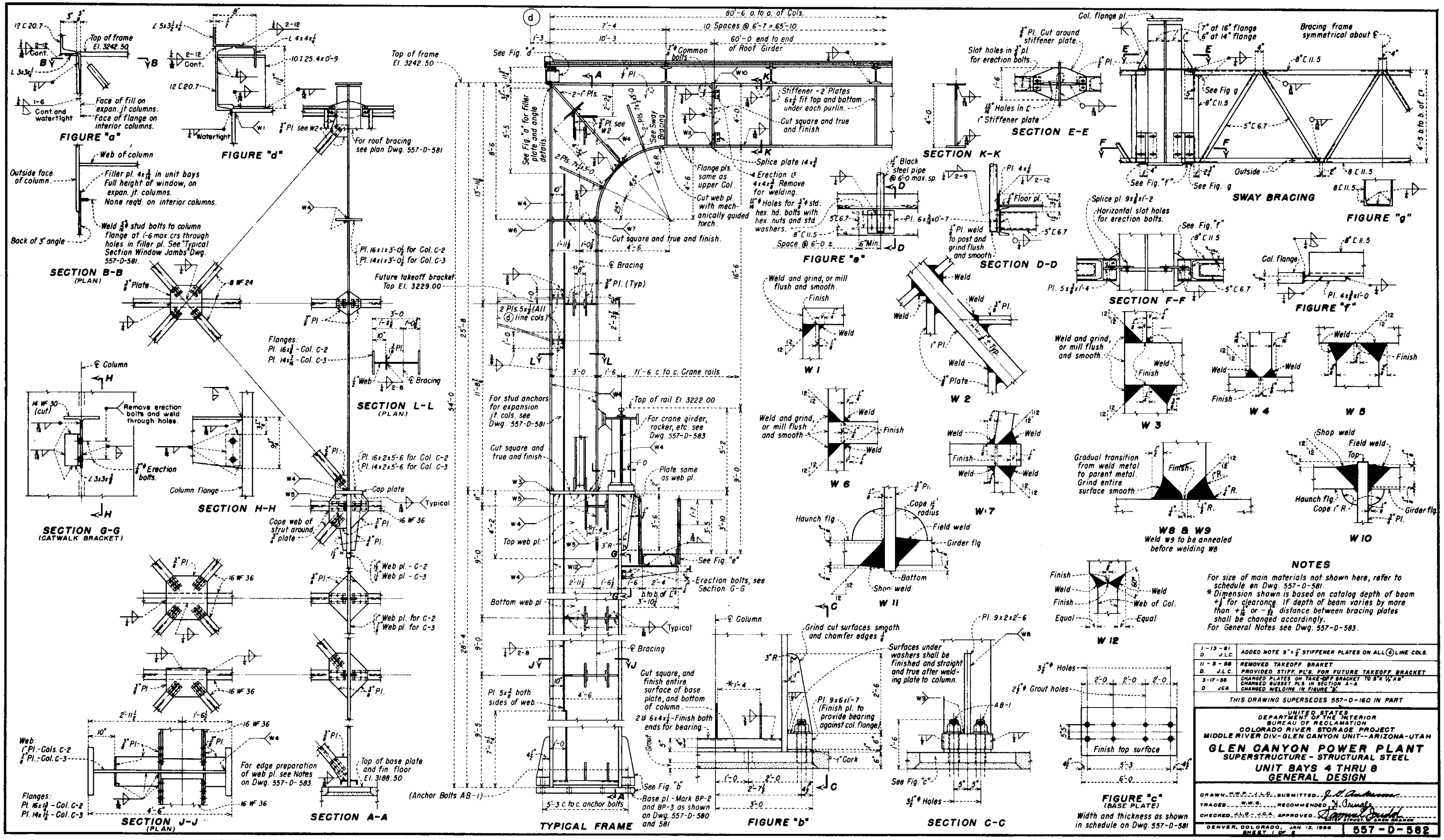


Figure 178.—Powerplant superstructure structural-steel general design—Unit bays 4 through 8.

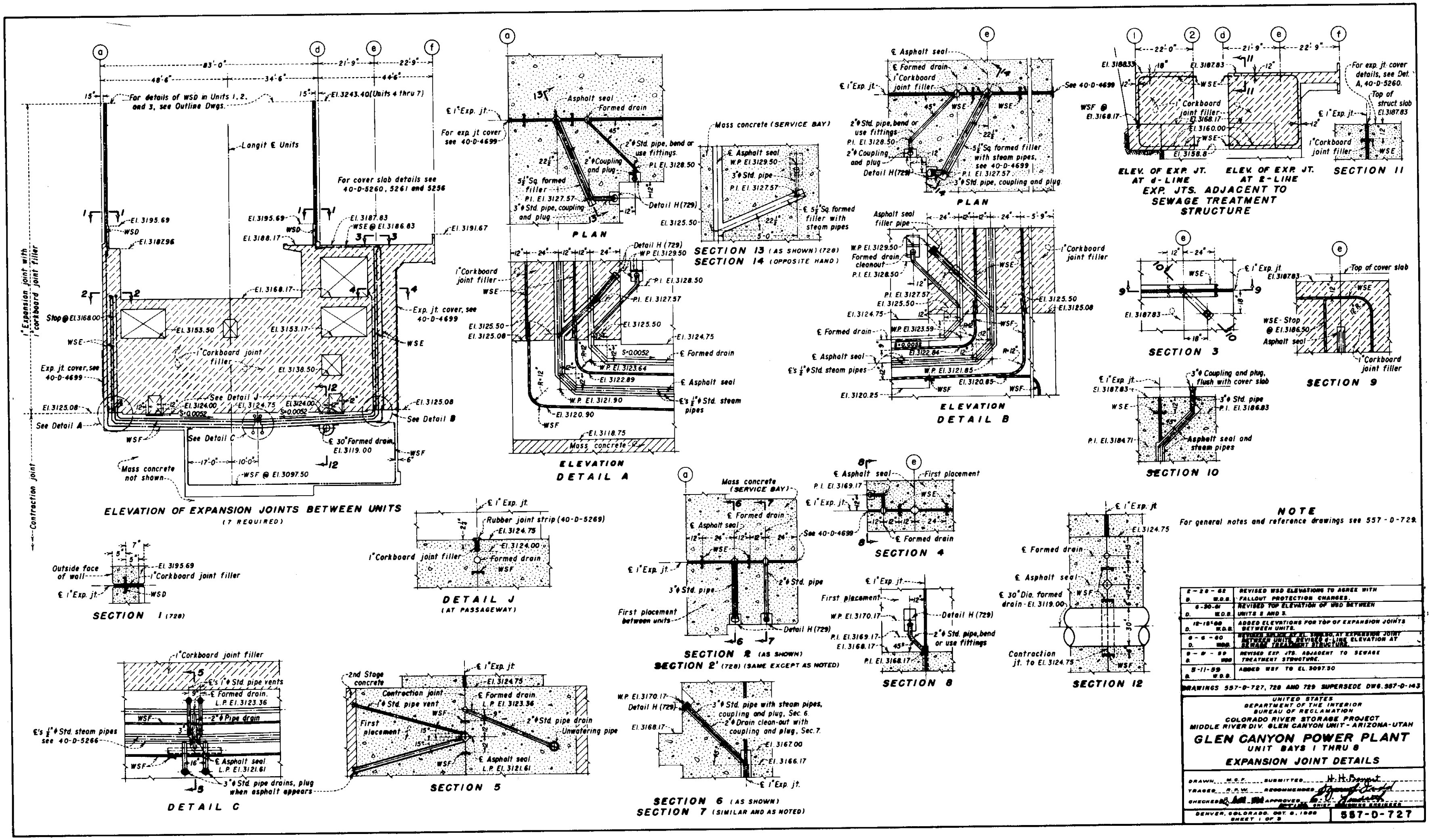


Figure 179.—Powerplant expansion joint details—Unit bays 1 through 8.

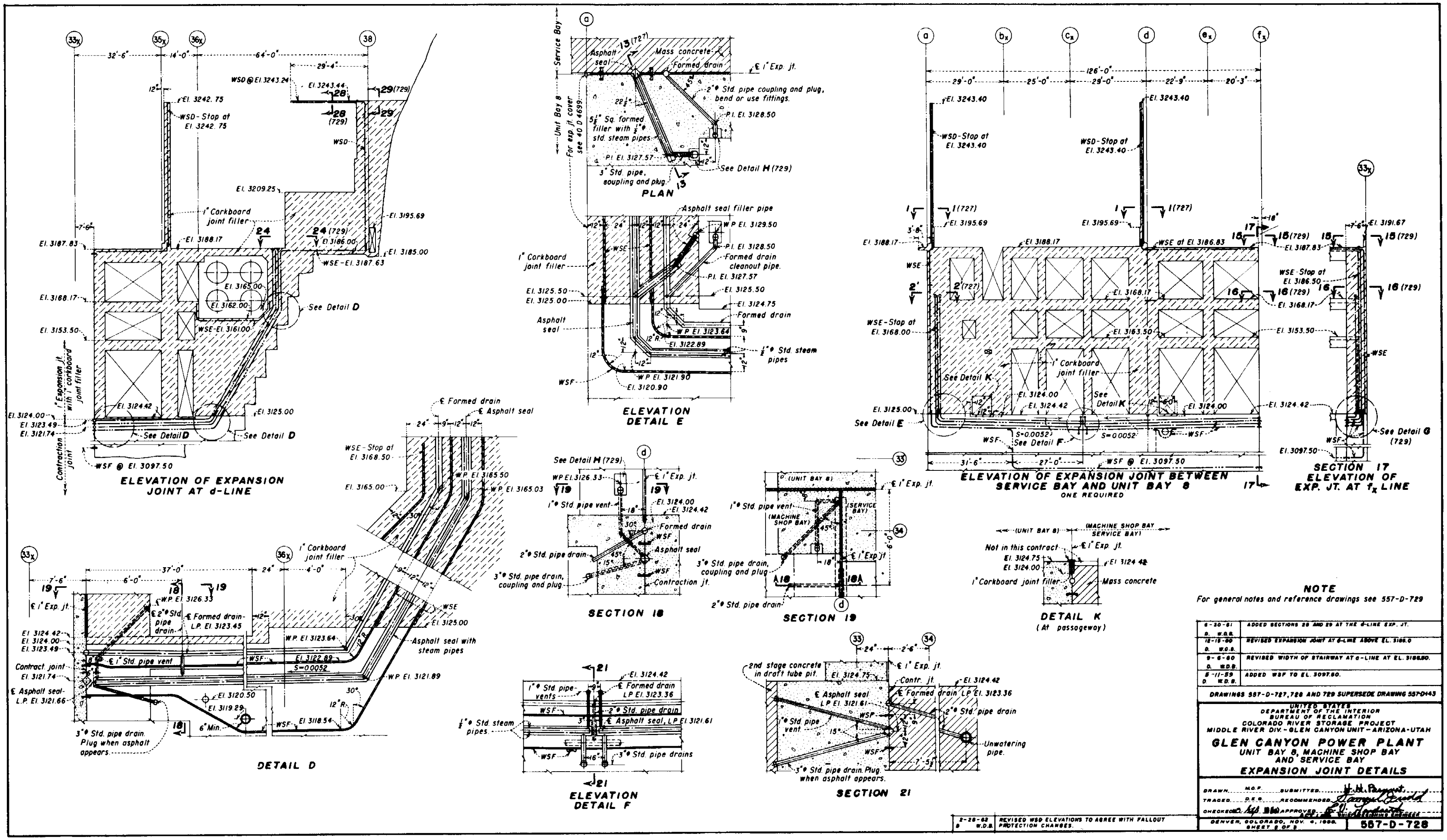


Figure 180.—Powerplant expansion joint details—Unit bay 8, machine shop bay, and service bay. (Sheet 1 of 2.)

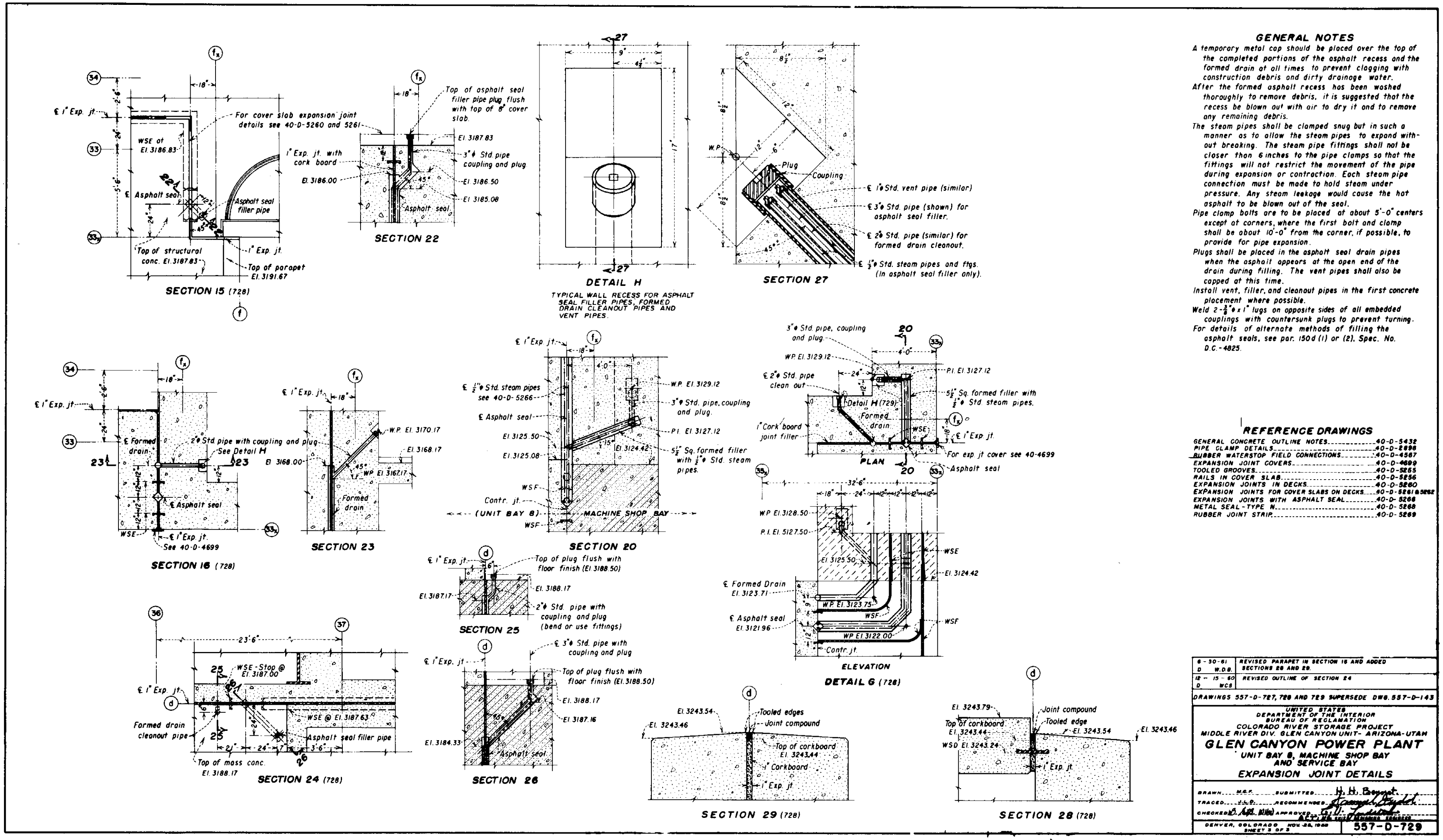


Figure 180.—Powerplant expansion joint details—Unit bay 8, machine shop bay, and service bay. (Sheet 2 of 2.)

effects of restraint and temperature rise of the concrete after placing. For an example of the sequence of concrete placement and location of construction joints in the powerplant, see figures 181 and 182. The reinforcement is continuous across the construction joints and care was taken to obtain good bond between adjacent placements of concrete. The joints were keyed when it was necessary to develop additional shearing resistance and to assure monolithic action of the structure. Rubber waterstops were used in the vertical construction joints and noncorrodible metal seals were used in the horizontal construction joints, where necessary, to prevent the flow of water through the joints.

(3) *Contraction joints* are used to relieve tensile stresses induced in a concrete structure by shrinkage. These joints are commonly used where temperature variations are small and the volume change in concrete is confined to shrinkage. Contraction joints differ from construction joints in that the reinforcement is discontinuous at the joint and means are used to prevent bond between the joint faces. A contraction joint may also serve as a construction joint.

In the powerplant, contraction joints were used in the mass concrete beneath the main unit bays, the machine shop bay, and the service bay. They were also used in the substructure concrete of the main units from the mass concrete up to elevation 3124.75. The machine shop bay and the downstream training wall were separated by a contraction joint at the m-line wall from the rock foundation to the top of the training wall.

(4) *Control joints* were used in the superstructure walls and the cable and elevator tower to prevent or control unsightly cracking. Two types of control joints were used, type A control joints and type B control joints. For the type A control joint, the continuity of the concrete surface is interrupted by interior and exterior grooves and a parting strip of sheet metal is placed in the joint. The reinforcement is continuous across the joint. The parting strip forms a weakened plane and induces the wall to crack at the grooves. When the control joint is continued past a floor, a floor blockout is used to prevent the floor from cracking. The floor blockout is filled after the initial shrinkage and cracking has taken place.

The type B control joint is a vertical keyed construction joint with grooves on the interior and exterior face of the wall. The first placement of concrete is painted with sealing compound to prevent bond and relieve the stress in the wall. The interior and exterior surface grooves are similar to the type A control joint grooves.

(k) *Second-Stage Concrete.*—The volume of second-stage concrete in each unit bay was 3,750 cubic yards. The contractor chose to use prepacked concrete around the upper and lower draft tube liners and up to the centerline of the spiral case and penstock at elevation 3140.00. He used regular cast-in-place concrete above elevation 3140.00 to the top of the generator floor at elevation 3168.50. The total volume of second-stage concrete used in the powerplant was 30,000 cubic yards.

The spiral case was designed to resist the difference between the pressure under water-hammer conditions and the spiral case installation pressure. To determine the required reinforcement around the spiral case, the embedded penstock, and the outside face of the concrete at the galleries and passageways, an analysis for a pipe shell with a cracked section was used. By inspection, the points of minimum second-stage concrete thickness were found and the necessary reinforcement determined for these areas. Each case was then analyzed as a hollow cylinder with many cracks in the concrete to a depth equal to the distance of the hoop reinforcement from the inner face and subject to uniform pressure on the inner and outer surfaces.

Each generator is supported on 12 stator foundation caps. The vertical load on the six foundation caps under the bearing bracket arms was 360,000 pounds per cap, and the vertical load on the six alternate foundation caps was 40,000 pounds per cap. The tangential force due to the single-phase short-circuit torque was approximately 220,000 pounds per cap. The concrete foundation was designed to withstand the maximum synchronizing out-of-phase torque of 6,150,000 foot-pounds per foundation cap, which is equivalent to 388,400 pounds of tangential force per cap. The bearing and shearing stresses on the concrete under the foundation caps were checked and found to be within the allowable limits. Stresses in the anchor bolts and soleplates were also checked and found to be within the allowable limits.

(l) *Surfacing.*—The following types of floor surfacing or finishes were used in the powerplant, dam adits, and dam elevator towers:

(1) A 6-inch concrete surfacing with reinforcement was used on the elevator machinery

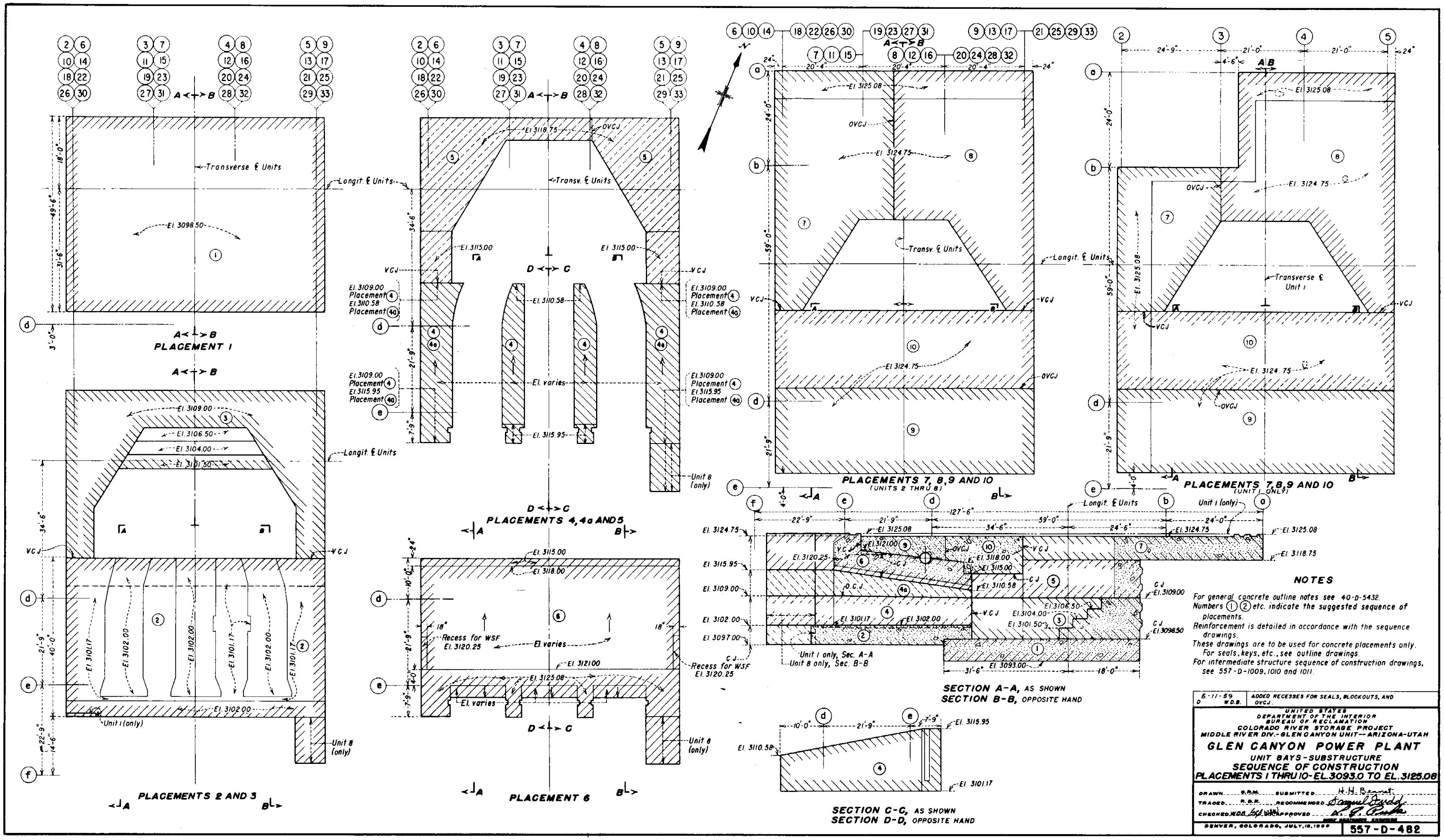


Figure 181.—Powerplant unit bays substructure sequence of construction placements 1 through 10—Elevation 3093.0 to elevation 3125.08.

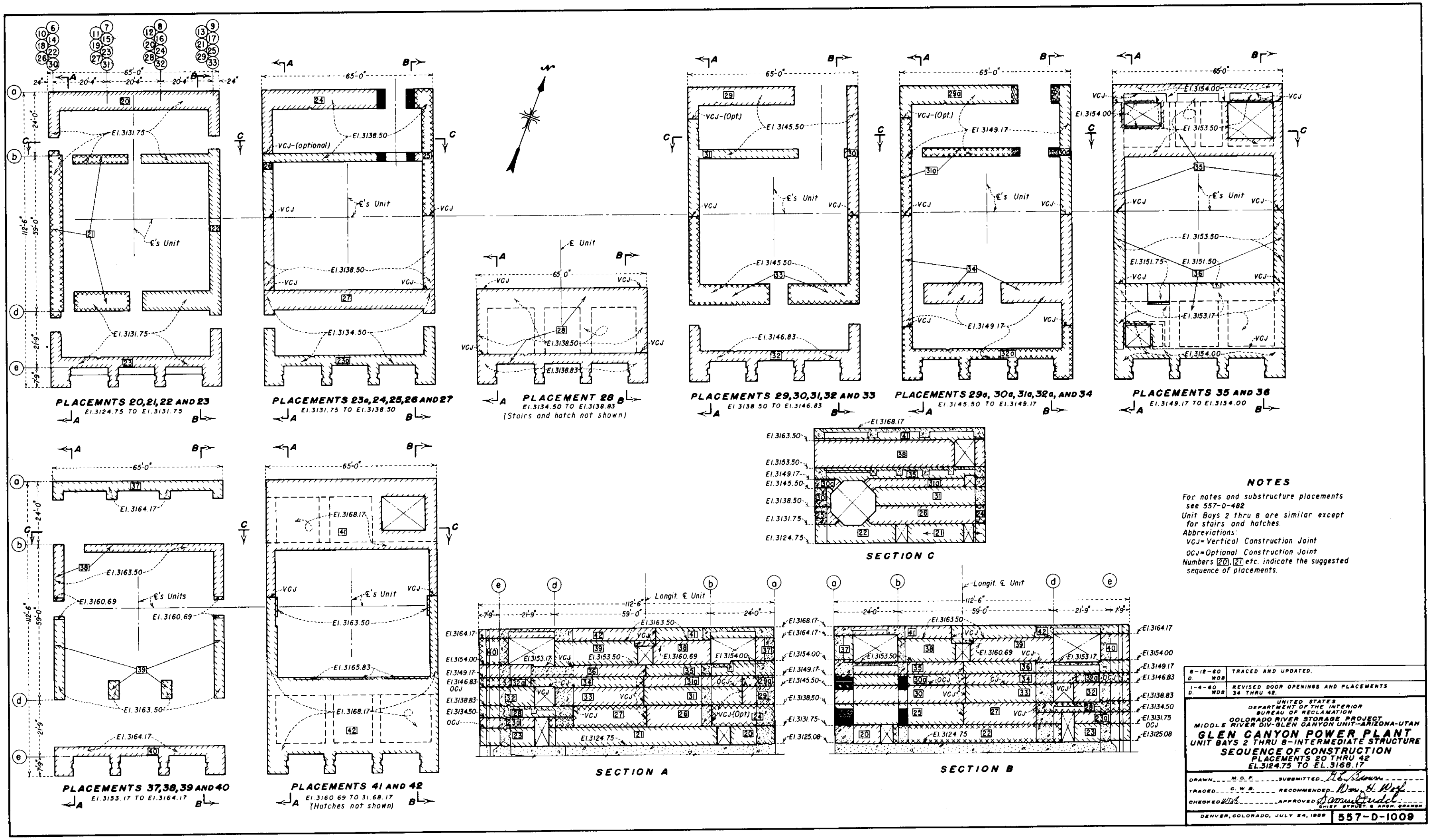


Figure 182.—Powerplant unit bays 2 through 8 intermediate structure sequence of construction for placements 20 through 42—Elevation 3124.75 to elevation 3168.17.

floor at elevation 3279.12 in the cable and elevator tower of the powerplant.

(2) A 4-inch concrete surfacing with reinforcement was used in the dam elevator towers, on the left abutment walkway between the dam and powerplant, on parts of the right abutment walkway, in the machine shop and service bay on the floors at elevations 3124.75, 3168.50 and 3188.50, and in the downstream gallery at elevation 3153.50 in unit bays 1 through 8.

(3) A 1/2-inch terrazzo finish on a 3-1/2-inch concrete base or underbed with reinforcement was used on the powerplant visitors walkway at elevation 3188.50 in unit bays 1 through 8, on the generator floor and electrical gallery at elevation 3168.50 in unit bays 1 through 8, in the toilet areas in the machine shop bay, in the dam elevator towers, in the dam adits, and on the concrete covers over the pipe chases in the dam adits.

(4) A 1/2-inch terrazzo finish on a 1-1/2-inch concrete underbed with an epoxy bonding agent was used in the toilet areas of the cable and elevator tower and on stairs S-2 through S-9 in the powerplant.

(5) A 2-inch-thick bonded concrete finish was used on the floors of rooms in the cable and elevator tower and on certain stair treads, risers, and landings in the powerplant.

(6) A 1-inch-thick bonded concrete finish was used on the deck of the right abutment walkway.

The 6- and 4-inch concrete surfacing, the 3-1/2- and 1-1/2-inch concrete underbed, and the bonded concrete floor finishes were placed as a part of the completion contract. The one-half-inch terrazzo finish was placed as a separate contract after all of the equipment was installed.

75. ARCHITECTURE. The architecture of the dam is of basic functional form, depending upon its tremendous mass for its emotional impact upon the observer. Its detail is bold and simple in keeping with the gargantuan proportions of both the natural and manmade features at the site.

The powerplant exterior, continuing this philosophy of sheer functionalism, is composed of exposed steel frame and concrete (placed in forms) filler panels (fig. 183). The building is windowless and designed in critical areas to withstand radiation resulting from radioactive fallout.

The interior of the powerplant is largely unadorned. Such finishes as are provided are confined largely to the following:

(1) Terrazzo floors in generator room area, toilets and visitors' adits.

(2) Tile walls in toilets.

(3) Metal movable partitions, acoustical ceilings, and vinyl floor covering in control room and offices.

(4) Paint in a selection of colors for the principal areas only.

C. BUILDING FACILITIES

76. SEWAGE TREATMENT PLANT. The sewage treatment plant was designed for a maximum daily flow of 15,300 gallons. Sewage from the powerplant, dam, and visitor center building flows by gravity either to the treatment plant or to the collection tanks in the powerplant. From the collection tanks, the sewage is pumped to the treatment plant. Gravity flow through the treatment plant terminates at the tailrace. (See fig. 184.)

Sewage is treated by the activated sludge process, extended-aeration-type plant. Design of the treatment plant was based on the following criteria:

(1) Design population:

Visitors—4,000 per maximum day
Operating personnel—190 per maximum day

(2) Sewage flow:

4,000 visitors at 3 gallons each = 12,000 gallons per day
190 operators at 17 gallons each = 3,230 gallons per day

Total—15,230 gallons per maximum day (compared with 10,500 gallons per day average during 1958 for June, July, and August at Hoover Powerplant)

Maximum 4-hour rate of flow based on most probable use of available facilities = 32 gallons per minute (same as actual sewage flow at Hoover Powerplant)

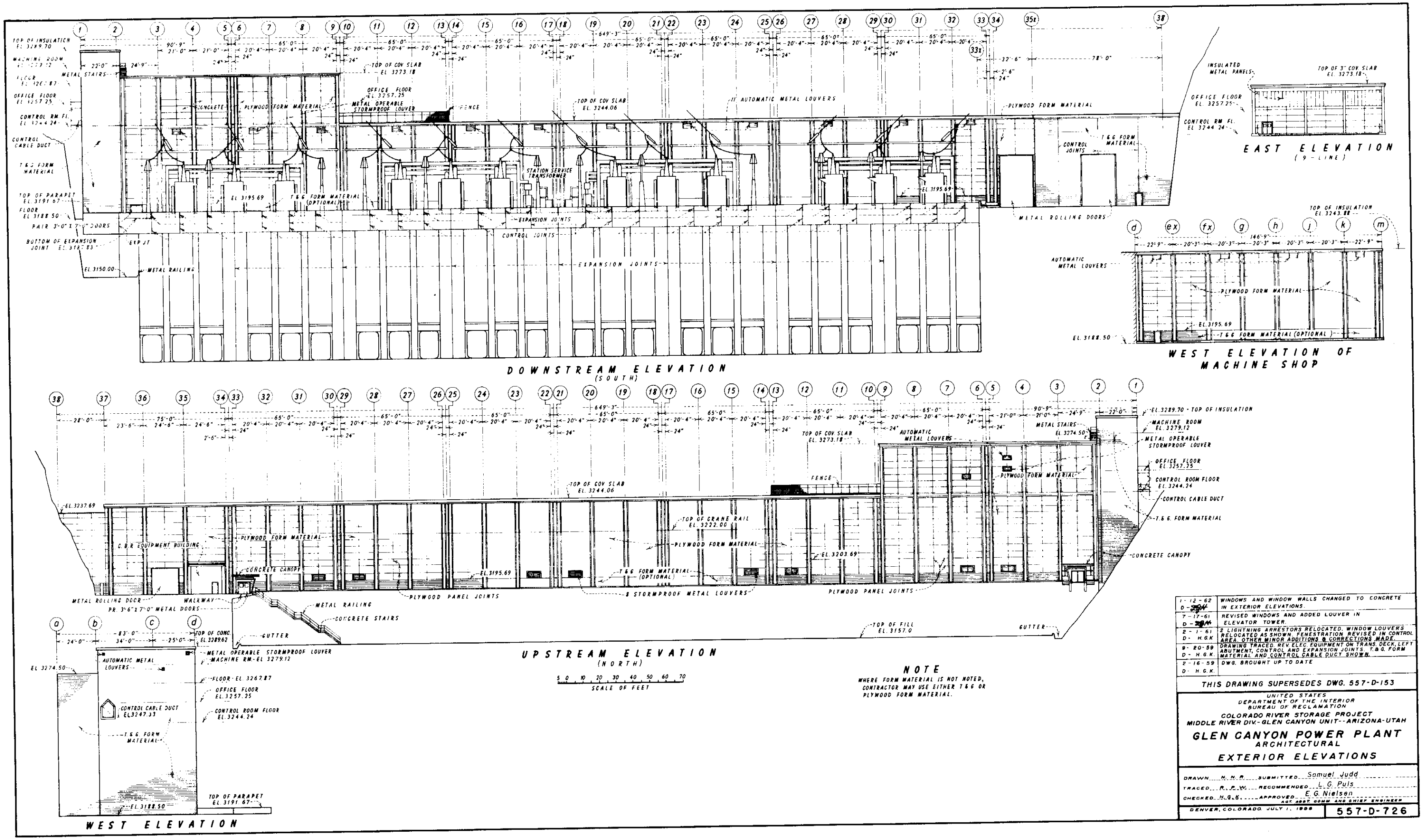


Figure 183.—Powerplant exterior elevations—Architectural treatment.

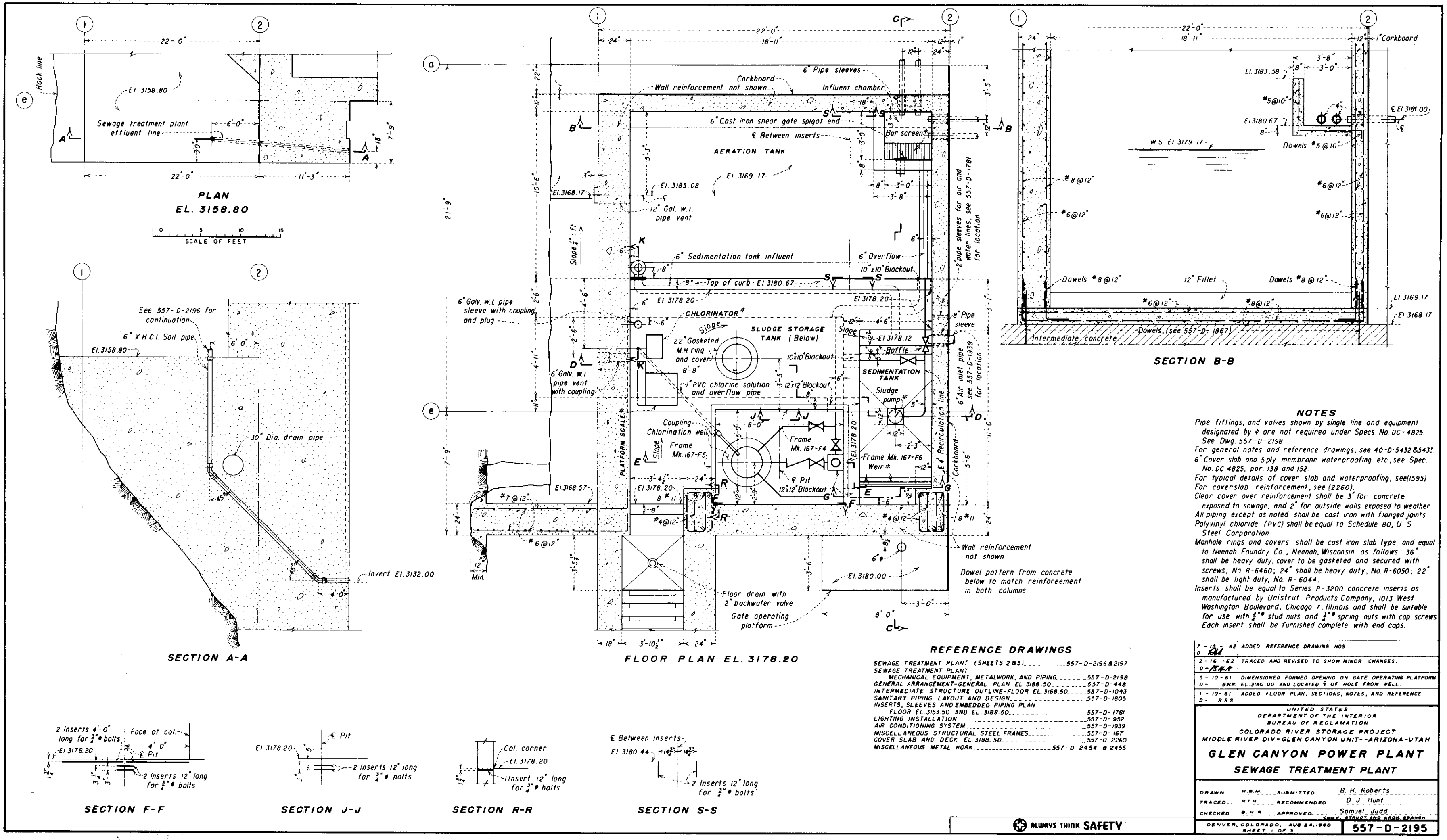


Figure 184.—Powerplant sewage treatment plant.

(3) Sewage biochemical oxygen demand (BOD):

4,000 visitors at 0.01 pound each = 40 pounds BOD
190 operating personnel at 0.06 pound each = 11.4 pounds BOD
Total = 51.4 pounds BOD per maximum day

(4) Aeration tank:

Detention time = 24 hours for maximum day
Air supply = 2,400 cubic feet of air per pound of BOD removed

(5) Sedimentation tank:

Detention time = 1.5 hours for maximum 4-hour rate of flow

(6) Chlorination:

Detention time = 15 minutes for maximum 4-hour rate of flow
Residual chlorine = 1.0 part per million

The U.S. Department of Health, Education, and Welfare fully approved the sewage treatment plant.

77. SANITARY SEWER. Sewerlines from the powerplant and elevator towers in the dam deliver sewage to three collecting tanks. One tank serves the elevator tower in block 17 of the dam, the control room, and office areas. Another tank serves the central portion of the powerplant. A third tank serves the elevator tower in block 8 of the dam, the service bay, and the machine shop. The sewage is pumped from these tanks to the sewage disposal plant for treatment. Floor drains throughout the building are not connected to the sewage lines but have their own system discharging into the building sump.

78. HEATING. Heating of the powerplant is provided for the protection of equipment and personnel and for the comfort of the personnel. The heating load for the powerplant was based on an outside design temperature of 0° F. and an inside design temperature of 45° F. in all spaces except those listed below. Special higher inside design temperatures were used in a few areas as follows: office and control room 72° F.; machine shop bay and toilets 65° F. A lower inside design temperature of 37° F. was used in the elevators and cable shafts.

The major heating capacity is furnished by water-to-water heat pumps which supply hot water to heat-exchanger coils installed in heating and ventilating units and air-conditioning units located throughout the powerplant. When the outdoor air temperature is below 65° F., the water-to-water heat pumps are energized. Additional heat is supplied where needed by electric natural convection and duct heaters.

79. VENTILATION. Ventilation is provided for the comfort and protection of personnel, the distribution and removal of heat, the relief of dampness, and the disposal of contaminated air. Approximately four air changes per hour of fresh air are furnished to the powerplant as a whole.

Exhaust fans are located in the building for removal of contaminated air from toilets, welding room, battery room, oil purifier room, and oil storage room. Excess ventilating air is relieved from the building through automatic louvers set to open under a static gage pressure of 0.125 inch of water.

80. COOLING. Cooling of the offices and control room is provided for protection of equipment and comfort of personnel. Cooling of the electrical gallery is provided for the protection of equipment. The cooling loads for the offices and control room were calculated using an outside condition of 95° F. dry bulb and 65° F. wet bulb, and an inside condition of 75° F. dry bulb and 59° F. wet bulb. The cooling load for the electrical gallery was calculated using an estimated powerload from equipment installed in the gallery.

The cooling is furnished by a water-to-water chiller which supplies cold water to heat exchanger coils in the air-conditioning unit. When the outdoor air temperature is above 65° F. the water-to-water chiller is energized.

D. MAJOR HYDRAULIC EQUIPMENT

81. TURBINE. On the basis of economic studies, the plant nameplate capacity was set at 900,000 kilowatts, utilizing eight 112,500-kilowatt generating units. The turbines were sized to match this rating at 450 feet net head, making the nameplate rating available for firm power to the power system about 90 percent of the time. The design (best efficiency) head

for the turbines was set at 510 feet, which is near the weighted-average operating head.

The turbine speed and setting were established jointly since they are mutually related. Some of the variables considered, evaluated, and judged were operating heads, tailwater elevations, cavitation factor (sigma), size of unit as affected by speed, building size, excavation required, and foundation conditions. From the studies it was determined that a unit speed of 150 revolutions per minute and a distributor centerline setting at elevation 3140.00 were optimum for the Glen Canyon installation.

The turbines were designed by the manufacturer, but the design was controlled by detailed Bureau specifications. Principal specified operating requirements or conditions are as follows:

Rated output---155,500 horsepower at 450 feet effective head
Best efficiency---90 percent or higher at 510 feet effective head
Range of effective head---341 feet to 560 feet
Specified minimum tailwater---elevation 3137.00 initial and elevation 3134.00 after degradation
Casing design pressure---300 pounds per square inch

A summation of turbine statistics, controlling dimensions, and predicted operating characteristics, is shown on the hydraulic turbine data sheet, figure 185. Figure 186 shows a sectional elevation of the turbine.

Design and construction details are given in invitation No. DS-5234, and shown on the drawings of the manufacturer, the Baldwin-Lima-Hamilton Corp. Materials for the parts were specified, and unit stresses and fabrication methods for main parts were controlled by reference to the ASME Boiler and Pressure Vessel Code. Principal construction features include the following items:

Spiral case—Plate steel with field welded radial joints
Draft tube liner—Plate steel elbow type with two intermediate piers
Runner—Cast steel with stainless steel coatings on surfaces most susceptible to cavitation
Head cover—Cast steel in half sections with bolted joint
Guide bearing—Babbit-lined, pressure-feed lubricated with external heat exchanger
Wearing rings—Stainless steel, removable and renewable, with replaceable inserts in stationary rings
Shaft—Forged steel with integrally forged flanged couplings
Shaft sleeve—Stainless, removable, and renewable
Wicket gates—Cast steel with stainless steel coatings over critical areas
Facing plates—Stainless steel, removable, and renewable
Stay ring—Cast steel with integrally cast stay vanes
Grease system—Automatic unit centralized

The turbine design was carefully checked for correctness, completeness, and compliance with the specifications. Stress checks were made on the major parts and on doubtful stress-carrying items to insure conformity with the specified stress limitations. Some of the items found to be overstressed and requiring revision were stay ring bolts, draft tube pier nose, draft tube stiffening ribs and tie rods, operating ring retaining clips, spiral case jacks, spiral case mandoor, and gate lever key.

There were five orders for changes issued on the turbine contract. One of these was for changing the painting requirements; one was for revising the walkways, platforms, stairways, and handrailing in unit 1 to include visitor accommodations; one was for an additional stay ring erection spider to accelerate installation; and the other two pertained to furnishing centralized automatic lubricating grease systems in lieu of the originally specified manual grease systems.

A shop assembly of the nonembedded stationary parts and the rotating parts was made in Belgium at the shops of a Baldwin-Lima-Hamilton subcontractor, Cockerill-Ougree, where these parts were manufactured. The stay rings, discharge rings, and throat rings were manufactured in Pennsylvania by Baldwin-Lima-Hamilton, the prime contractor for the turbines, and shipped to Salt Lake City, Utah, for fitting to the pit liners, spiral cases, and draft tube liners, which were constructed by Chicago Bridge and Iron, another subcontractor for Baldwin-Lima-Hamilton (B-L-H). Final balancing of the runners and alinement of the combined turbine and generator shafts were done by B-L-H at their shops in Eddystone, Pa. Because of the shop and field welded construction of the spiral case, pressure tests of the spiral case and stay ring assemblies were made in the field.

Installation of the turbines was performed under the supervision of the turbine contractor's erecting engineer. The major portion of the turbine installation was done by the completion contractor, Ets-Hokin Corp. However, the turbine contractor's subcontractor, SHEET NO. 70

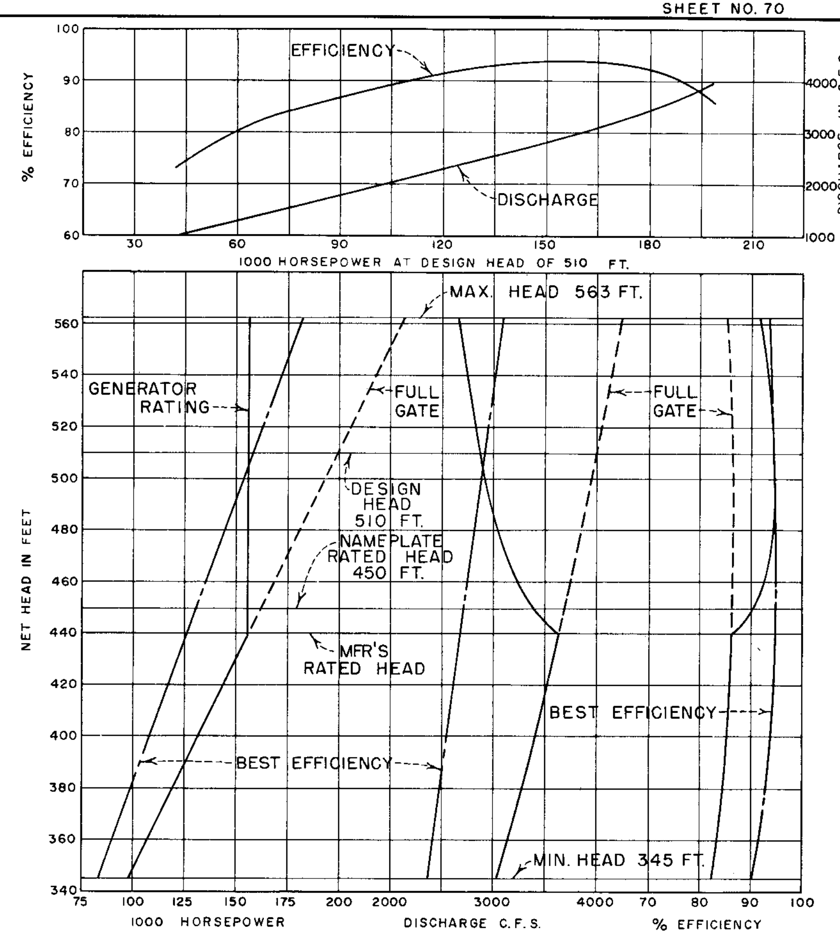
GLEN CANYON POWER PLANT ------ COLORADO RIVER STORAGE -- PROJECT.
SPECIFICATIONS NO: DS-5234 ------- UNITS No. 1 to 8 -------- DATE Nov. 24, 1959 ---------.
TURBINE NAMEPLATE RATING: H.P. 155,500 ---; HEAD 450 -- FT; --- SPEED 150 --- R.P.M.
GENERATOR RATING IN KV-A --- 125,000 ------------ POWER FACTOR 90 -- PERCENT.

Turbine mfr. Baldwin-Lima-Hamilton Corp. ------- Type --- Francis ------------.
Cost per unit f.o.b. factory -- $799,000 -------- Weight ---- 1,220,000 ------ lbs.
Cost per h.p. ------- $4.00 ------------------ Weight per h.p. ----- 6.16 ---- lbs.
Type of scroll case Welded plate steel spiral ---. Weight heaviest part 91,500 lbs.
Type of draft tube Elbow - Two pier, with plate steel liner ---------------.
Weight of runner 91,500 -- lbs. Weight of rotating parts ---- 185,000 -------- lbs.
Weight of turbine parts including hydraulic thrust to be carried by generator thrust bearing 650,000 lbs. New; 950,000 lbs. Worn rings.
Governor capacity in foot-lbs. ------ 441,700 -------- Pipe size -- 6 -- inches.
Gov. mfr. Pelton Div., B-L-H. Corp. ------------- Time element -- 5 -- seconds.
Cost per unit f.o.b. factory ---- $47,406.25 ---------- Weight --- 31,800 --- lbs.
Generator mfr. --- General Electric Co. -----------------------------.
Generator WR^2 ---- 72,100,000 ------ lbs. at one foot radius.
Turbine WR^2 ------- 3,350,000 ------ lbs. at one foot radius.
Regulating constant of unit ($\overline{R.P.M.}^2 \times WR^2 \div$ Design H.P.) ------ 8,560,000 ---------.
n_s of runner -- 24.6 --- at 510 --- ft. design head when delivering 157,500 --- h.p. (Best eff. gate).
N_s of runner -- 27.6 --- at 510 --- ft. design head when delivering 198,200 -- h.p. (Full gate).
H.P. at 510 ft. (Design head) ----- 198,200 --- ; at 100 -- percent of design head; 3985 c.f.s.
H.P. at 563 ft. (Max. head) ------ 229,800 ---; at 110.4 percent of design head; 4230 c.f.s.
H.P. at 345 ft. (Min. head) ------- 98,500 ---; at 67.6 percent of design head; 3035 c.f.s.
H.P. at 440 ft. (Mfrs. Rated Hd.) -- 155,500 --- ; at 86.3 percent of design head; 3630 c.f.s.
H.P. at best efficiency equals ---- 79.5 ----- percent of h.p. at full gate.
Runaway speed at 560 -- ft. hd. 277 -- r.p.m. equals 184.7 -- percent of normal speed.

DIMENSIONS OF TURBINE:

Unit spacing -- 65 -- ft.	Dia. of shaft 40 -- inches.
Max. dia. of runner 16.56 ft.	Dia. of cover plate 20.96 ft.
Dia. of gate circle 18.33 ft.	Number of wicket gates -- 24 ---.
Height of distributor case 2.375 ft.	Number of stay vanes -- 23 ----.
Dia. of scroll case inlet [redacted] 14.0 ft.	Dia. at top of draft tube 12.67 Ft=D_3.
Ouside radii of stay vanes 11.73 to 12.10 ft.	Distributor ℄ Elev. ---- 3140 ----- Ft.

Distance from center line of distributor to top of draft tube -- 4.625 -- ft.
Depth of draft tube -- 38.00 ft. equals 300.0 percent of dia. D_3.
Length of draft tube 58.83 ft. equals 464.4 percent of dia. D_3.
Width of draft tube 48.42 ft. equals 382.2 percent of dia. D_3.
Distance from center line of turbine to center line of scroll case inlet 18.5 -- ft.
Distance from center line of distributor to minimum tailwater, (Elev. 3133.64 -- ft.) (One unit operating at full load) -- 6.3 -- ft.
Pressure regulator mfr. ----- None ------------ Type ------------ Size ------ inches.
Cost per unit f.o.b. factory ---------------------- Weight ------------ lbs.
REMARKS:
Placed in operation --



PREDICTED CHARACTERISTIC CURVES
FROM MANUFACTURER'S DATA
CURVE NO. 6942 11-17-59

3-20-62

HYDRAULIC TURBINE DATA
GLEN CANYON POWER PLANT
U.S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Figure 185.—Hydraulic turbine data sheet.

110.4% = 33,840 cfs
100% = 31,880 cfs
67.6% = 24,280 cfs
86.3% = 29,040 cfs

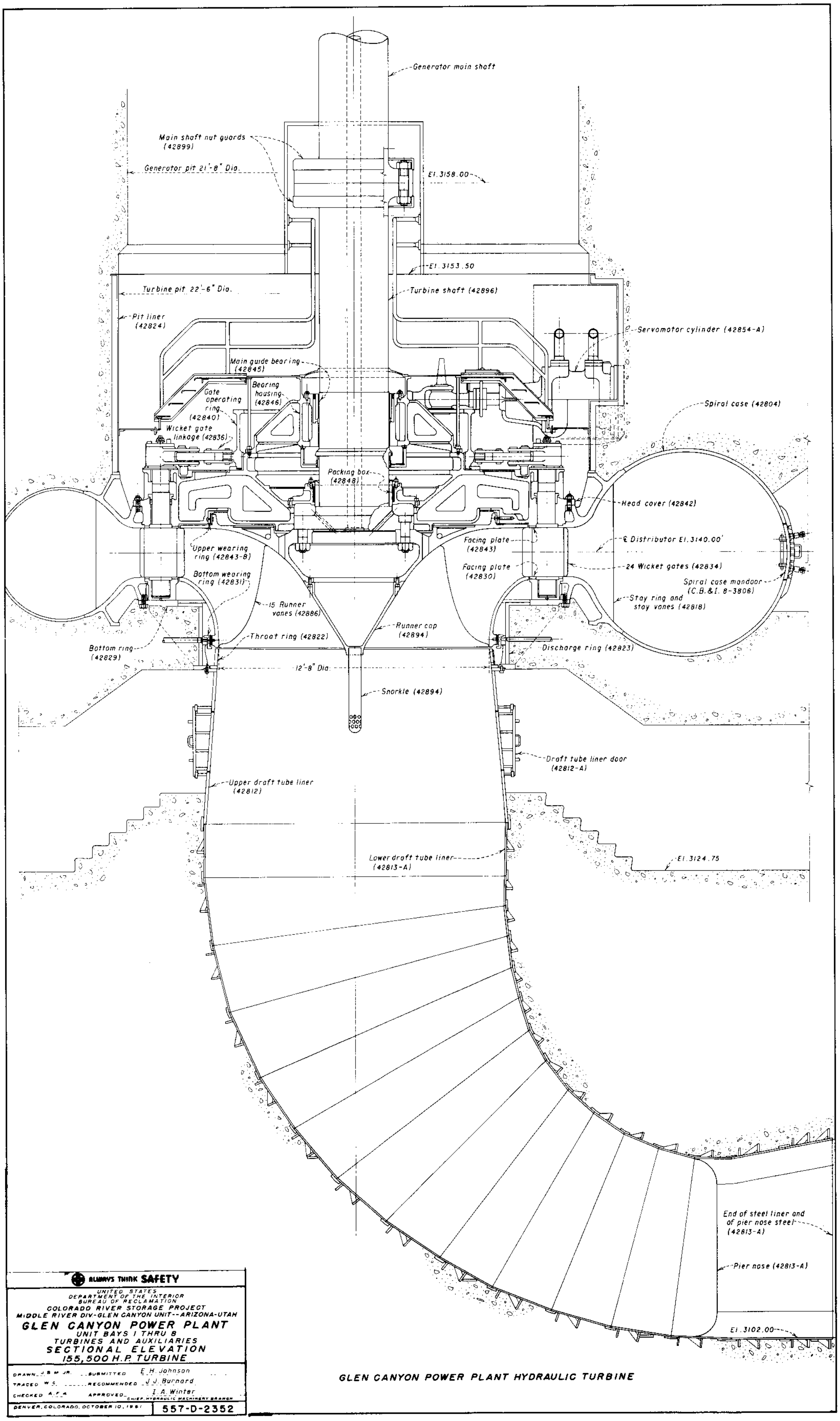


Figure 186.—Powerplant sectional elevation of turbines and auxiliaries—Unit bays 1 through 8.

Chicago Bridge and Iron, assembled, leveled, and alined the spiral case and stay ring sections, and tested the spiral case under a hydrostatic pressure of 450 pounds per square inch. Field erection, pressure tests, pressure embedment at 225 pounds per square inch, water passage calibrations, and run-in tests were successfully completed in accordance with the usual practice for these features.

During initial operation, the units were found to operate extremely quiet and smooth under the low power head available at this time.

Some of the deficiencies found during initial operation which required correction by the turbine contractor were inadequate securing of the throat ring which permitted loosening and rotation, insufficient thickness of the stainless steel coating on some areas of the runners (detected when checking minor cavitation, rusting, and pitting), brass shear pins that deformed rather than breaking cleanly when excessively loaded, set screws made of soft material that failed to properly hold the gate adjustment, and improper fitting of the throat ring at the runner discharge which caused cavitation of the throat ring. Also, during the initial operation period, the turbine contractor changed the full gate servomotor stroke from the machine capability of 15 inches to a limited 13-1/2-inch maximum stroke.

One turbine was to be tested after installation to determine whether the turbines met the efficiency and capacity warranties. Piezometer connections, installed on the turbines to permit future index tests of performance and future installation of flowmeters, were to be calibrated at that time.

82. GOVERNOR. The governors for turbine regulation are the oil-pressure cabinet-actuator type with electrically driven speed-responsive elements. Designing of the governors was done by the manufacturer, the Pelton Division of Baldwin-Lima-Hamilton Corp., but was controlled by detailed specification under invitation No. DS-5562. Details of the design and construction, including the various control and indicating features, may be found in the invitation and on the manufacturer's drawings. Basic design requirements for the governors were the foot-pound capacity and the minimum time for full gate stroke. The required foot-pound capacity was established at 441,700 foot-pounds by the turbine contractor. A 5-second minimum time for full gate stroke was set on the basis of the calculated speed rise for full load rejection and the corresponding water-hammer pressure created at the turbine. The 5-second minimum setting gives a calculated speed rise of 40 percent above normal for full load rejection and a water-hammer pressure of 682 feet at the turbine. The governors were set for a 5-second full gate stroke after tests determined this to be a safe operating condition.

The governor oil pressure is supplied by two 138-g.p.m., 30-horsepower, oil pumping units to a pressure tank where the operating oil pressure and level is continuously maintained for instant response to unit regulating requirements. The normal operating pressure at the governor pressure tank is 270 to 300 pounds per square inch. The tank has a total usable volume of 230 cubic feet which includes a reserve capacity below the alarm level corresponding to 265 pounds per square inch for three complete servomotor strokes with the oil pumps inoperative. This provision is a safety feature to provide sufficient energy for a limited operation and closure of the wicket gates in event of loss of power or pumping unit malfunction. Except for the servomotors which were a part of the turbine contract, all parts of the governor oil pressure system, including all interconnecting piping, were furnished by the governor contractor.

The governor control, indicating and protective features, as established to be essential and required by the invitation, include the following devices:

(1) *Gate limit mechanism.*—Operated manually at the actuator or electrically from the main control board.

(2) *Speed changer.*—Operated manually at the actuator or electrically from the main control board.

(3) *Speed droop adjustment mechanism.*—Adjustable manually at the actuator or electrically from the main control board for 0 to 5 percent speed droop.

(4) *Normal shutdown mechanism.*—Solenoid operated for automatic starting and normal shutdown of the unit.

(5) *Complete shutdown and lockout mechanism.*—Solenoid operated for normal-rate closure of the wicket gates, operated manually at the actuator or electrically by remote control.

(6) *Hydraulic-type hand-control device.*—For manual control of the wicket gates at the actuator.

(7) *Generator brake valve for controlling the operation of the generator air brakes.*—Normally solenoid operated but with provisions for emergency hand operation.

(8) *Gate position and gate limit indicators dual type.*--One mounted on the actuator and one for the main control board.

(9) *Speed changer adjustment indicator.*—Mechanical indicator mounted on the actuator and a similar electrical indicator for the main control board.

(10) *Speed droop adjustment indicator.*—Mechanical indicator mounted on the actuator.

(11) *Tachometers electrically operated, for turbine speed indication.*—One mounted on the actuator and one on the main control board.

(12) *Air-pressure gage.*—Duplex gage mounted on the actuator for indicating status of generator brake air supply.

(13) *Oil-pressure gage.*—Mounted on the actuator to indicate the pressure in the governor oil pressure system.

(14) *Dead stop and break-away indication.*—Flashing red lights to indicate when the turbine starts and when it comes to a complete stop.

(15) *Main overspeed switch.*—Mounted on and forming a part of the governor drive generator with adjustable contacts that can be set to operate at any speed from 125 to 200 percent of normal for use in control and protective circuits.

(16) *Auxiliary overspeed switch.*—Similar to main overspeed switch but to operate at speeds from 110 to 120 percent of normal.

(17) *Low speed switches.*—Mounted on and forming a part of the governor drive generator, one switch with adjustable contacts that can be set to operate at any speed from 15 to 35 percent of normal and one switch with contacts to operate between 40 and 50 percent of normal speed, for use in control and indicating circuits.

(18) *Oil-pressure alarm switch.*—Set to operate when the governor oil pressure drops to 265 pounds per square inch.

(19) *Oil-level alarm switch.*—Set to operate when pressure tank oil level drops to a level corresponding to 265 pounds per square inch pressure.

(20) *Oil-pressure shutdown switch.*—Operates when oil pressure drops to pressure corresponding to shutdown oil level.

(21) *Oil shutdown switch.*—Operates when oil in pressure tank drops to a level leaving only one servomotor volume before float valve closure.

(22) *Air-pressure failure switch.*—Operates when pressure in generator brake air supply drops to a predetermined level.

(23) *Gate position switches.*—Operate at various gate positions for use in the generator brake valve, auxiliary dashpot bypass, and turbine air admission circuits.

(24) *Indicating lamps for turbine air admission system.*--Mounted on the governor instrument panel for indication of "power supply on" and the setting of the unit master control for the turbine air admission system.

(25) *Auxiliary dashpot bypass.*—For providing additional bypass area in the dashpot of the compensating mechanism for increased flexibility of operation.

The governors and the associated auxiliary equipment were assembled and tested in the manufacturer's shop. Field installation and operational tests were performed by the completion contractor, Ets-Hokin Corp., under the supervision of the governor contractor's erecting engineer.

E. TURBINE AND GOVERNOR UNIT AUXILIARIES

83. AUXILIARY AND SERVICE SYSTEMS. (a) *General.*—Schematic diagrams of the auxiliary and service systems were prepared to aid in the layout of the piping and auxiliary equipment and for a convenient method of studying system functions and assuring correctness and completeness of design. These diagrams are listed below:

System	Figure No.
Gravity drainage	187
Pressure drainage and unwatering	188
Transformer oil	189
Lubricating and governor oil	190
Fire protection water	191
Service water	192
Compressed air	193
Carbon dioxide fire extinguishing	194
Unit cooling water	195

For the installation of these systems see sections 230 through 234. In general, the systems were designed to meet requirements for safe and convenient operation of the powerplant. Performance requirements, such as rates of flow, capacities, and operating times, were based mainly on the general established Bureau guide and on stated requirements of the major equipment manufacturers.

Principal types of piping were cast iron soil piping for gravity drains, cast iron bell and spigot piping for embedded pressure drains, screwed wrought iron galvanized piping for exposed lines under 2-1/2-inch diameter, welded wrought iron galvanized piping for exposed lines 2-1/2-inch through 8-inch diameter, and welded black steel extra strong piping for exposed lines 10 inches and larger.

The associated auxiliary equipment and instruments such as pumps, compressors, strainers, regulating and relief valves, gages, pressure and float switches, oil purifiers and drying oven, carbon dioxide equipment, and sewage ejection units, which form a part of the auxiliary and service systems and were not to be supplied by the major equipment manufacturers, were specified and purchased by separate invitations. (See Appendix A.)

(b) *Gravity Drainage Systems.*—Gravity drains were designed to convey water from the various sources of leakage or discharge to the plant drainage sump for collection and evacuation to the tailrace. Water collected in the sump is automatically pumped to the tailrace by two 1,000-g.p.m. deep-well turbine pumps under float control. Principal water sources considered in the design of the system were leakage from submerged powerplant walls, foundation drains, and mandoors; cooling water discharge from turbine packing boxes and air compressors; blowoff from fire lines, compressed air lines, strainers, and relief valves; oil room sink drain; discharge from sprinkler systems; and backwash drain from possible future domestic water filters. Special features include gravel-filled chilling basins for oil room drains to extinguish possible burning oil, and open discharge from the generator pit drains for visual detection of leakage. Four 24-inch-diameter emergency drainage openings with shutoff and flap valves are provided in the downstream wall of the powerplant to limit flooding to the lower levels in event of a serious plant flooding condition.

The drainage sump is provided with a manually controlled, water-operated eductor for complete evacuation of the sump to facilitate sump cleaning and equipment maintenance. The drainage and unwatering sumps share a common vent to the outside for maintaining atmospheric pressure in the sumps.

(c) *Unwatering System.*—The purpose of the unwatering system is to drain the draft tube and residual water in the spiral case and horizontal portion of the penstock for inspection or repair of the turbine. The penstock and spiral case will be drained down to tailwater level through the turbine before the unwatering system is used. Pipelines from the spiral case and draft tube conduct the water to the unwatering sump, where it is pumped to the tailrace by two 5,000-g.p.m. deep-well turbine pumps. The design objective is to accomplish the unwatering promptly and with minimum hazard of flooding through misoperation. Principal factors affecting design are tailwater level to be used, volume of the draft tube, volume of the spiral case and penstock below the tailwater level, sump elevations and volume, pump capacity, pipeline size, time to unwater, gate leakage, and conditions existing if the spiral case drain valve is opened by mistake with the penstock under full pressure. The spiral case drain valve should never be opened with the penstock under pressure.

The adopted design provides for unwatering of a unit from tailwater level with the bulkhead gates properly seated and normal leakage occurring. The spiral case drain header is normally closed to the sump

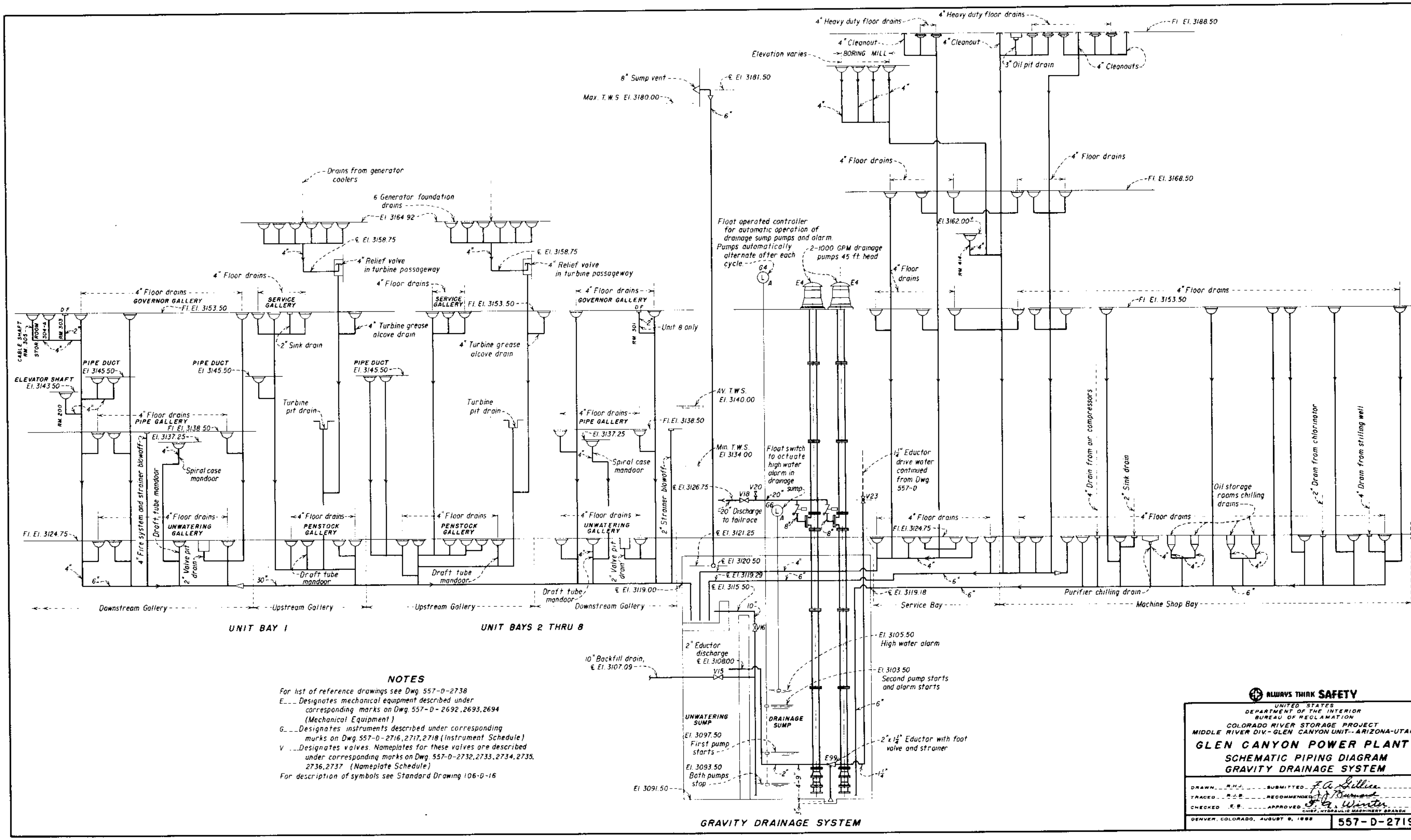
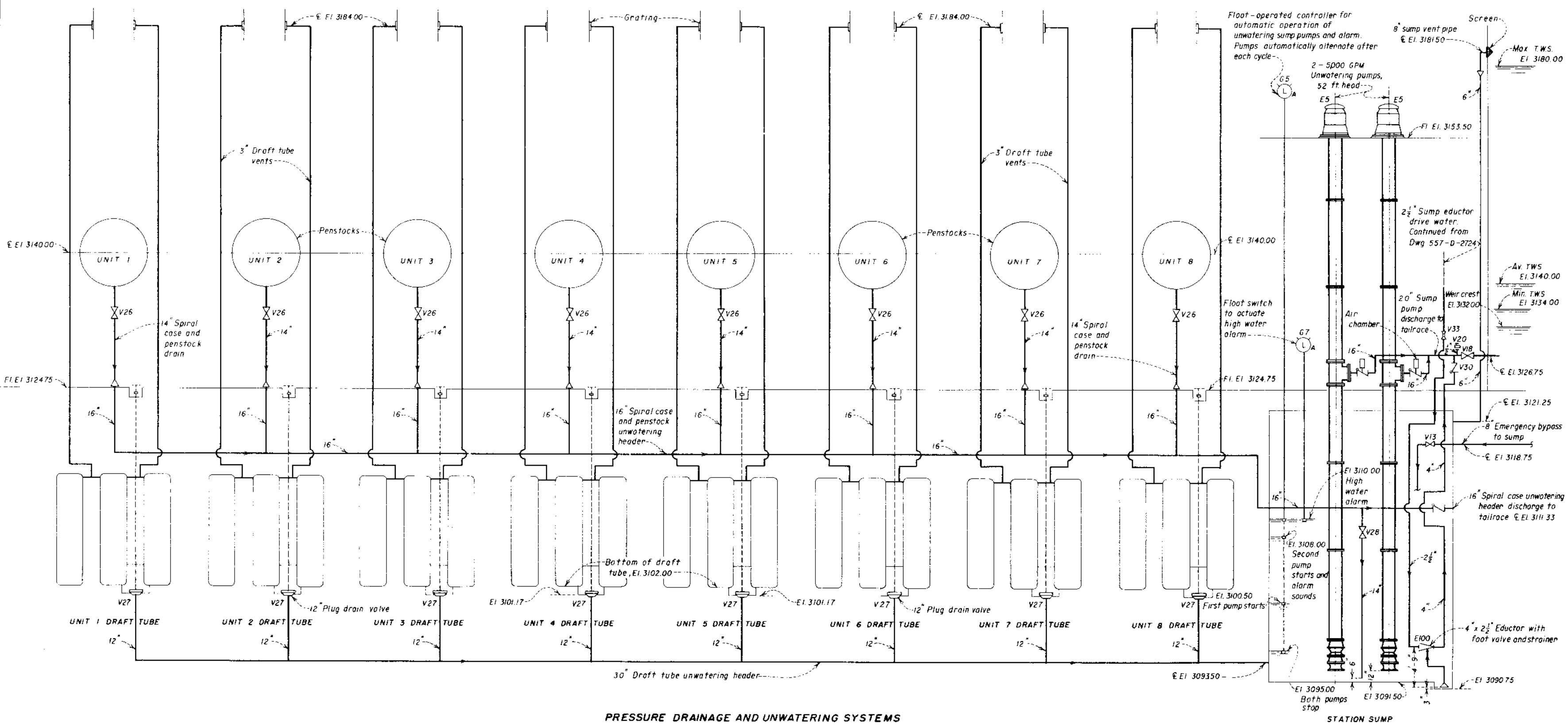


Figure 187.—Powerplant schematic piping diagram of gravity drainage system.



NOTES

For list of reference drawings see Dwg. 557-D-2738

E____Designates mechanical equipment described under corresponding marks on Dwg 557-D-2692, 2693, 2694 (Mechanical Equipment)

G____Designates instruments described under corresponding marks on Dwg. 557-D-2716, 2717, 2718 (Instrument Schedule)

V____Designates valves Nameplates for these valves are described under corresponding marks on Dwg. 557-D-2732, 2733, 2734, 2735, 2736, 2737 (Nameplate Schedule)

For description of symbols see Standard Drawing 106-D-16

ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.-GLEN CANYON UNIT-ARIZONA-UTAH

GLEN CANYON POWER PLANT

SCHEMATIC PIPING DIAGRAM
PRESSURE DRAINAGE AND UNWATERING SYSTEMS

DRAWN R.H.J. SUBMITTED F.A. Gillin
TRACED L.A.A. RECOMMENDED J.J. Burnand
CHECKED E.B. APPROVED L.A. Winter
CHIEF, HYDRAULIC MACHINERY BRANCH

DENVER, COLORADO, AUGUST 6, 1962 | 557-D-2720

Figure 188.—Powerplant schematic piping diagram of pressure drainage and unwatering systems.

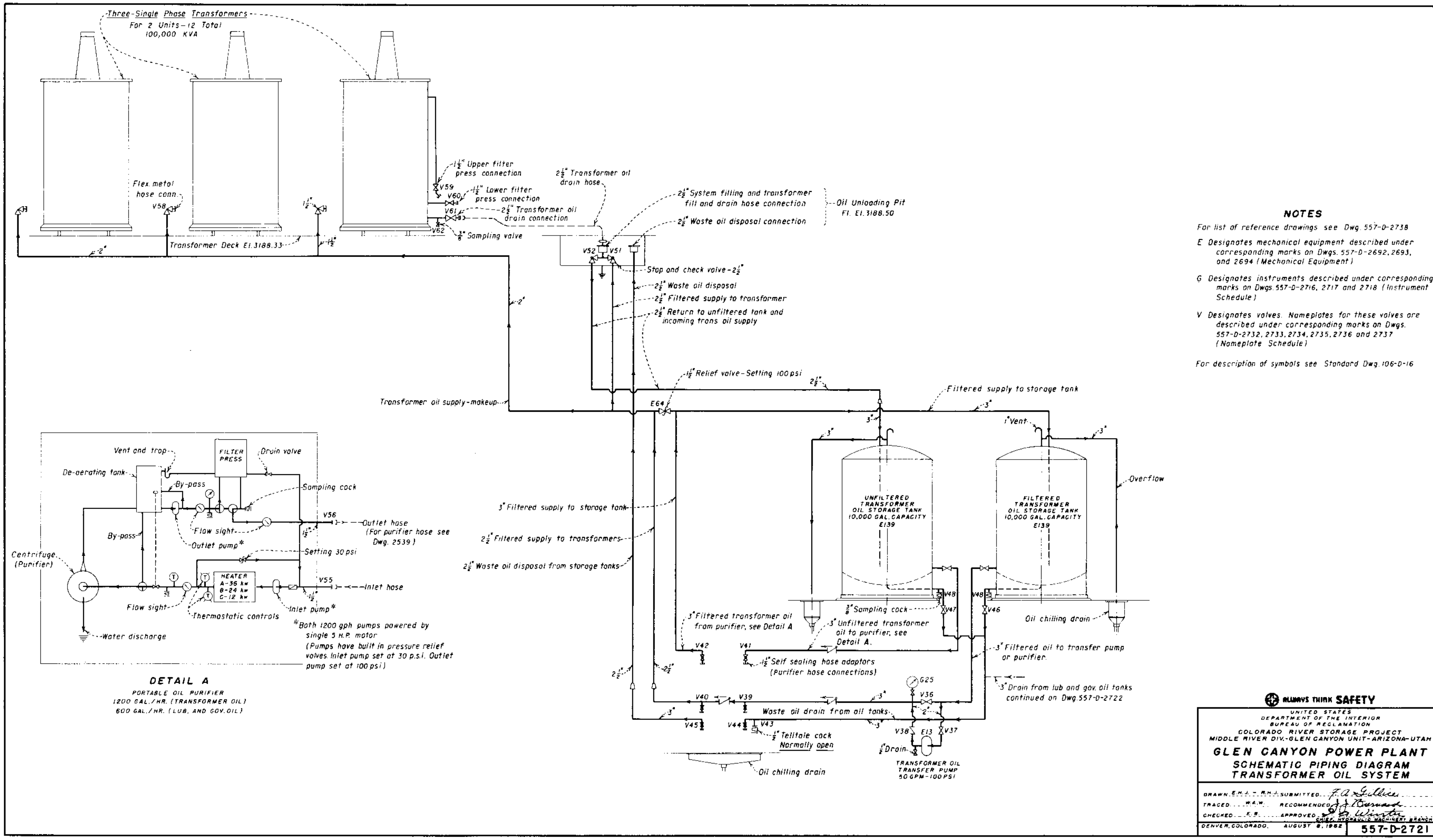


Figure 189.—Powerplant schematic piping diagram of transformer oil system.

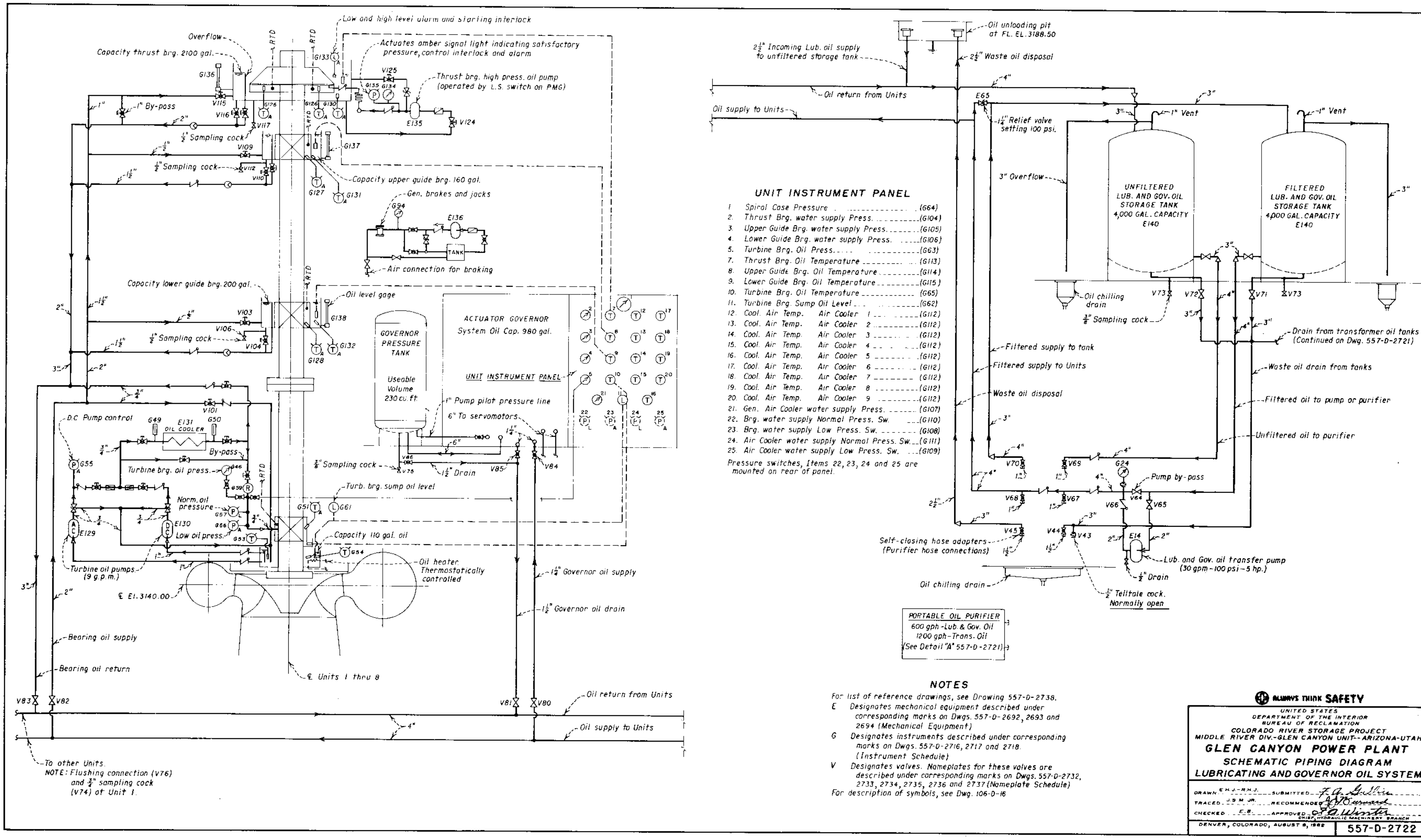


Figure 190.—Powerplant schematic piping diagram of lubricating and governor oil system.

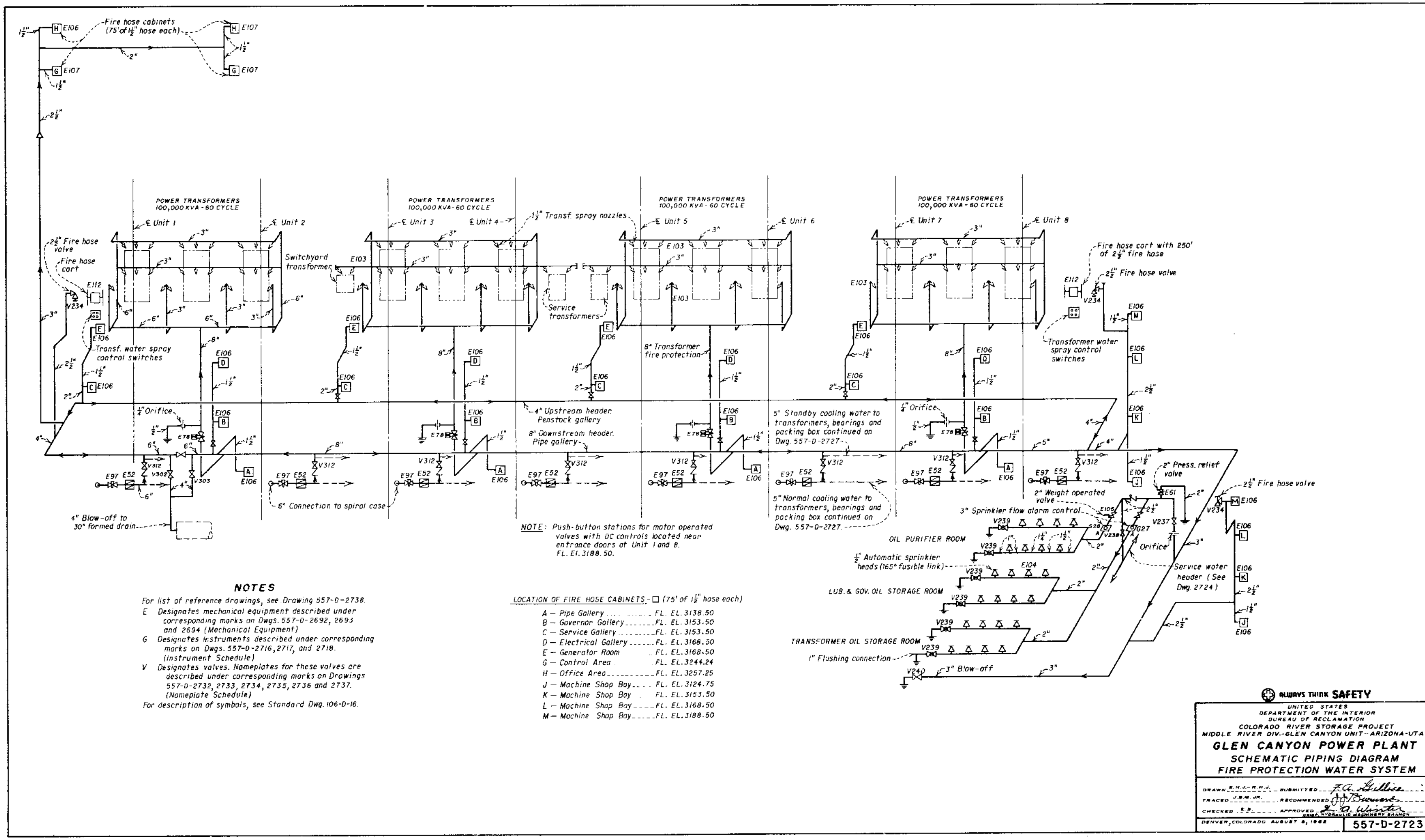


Figure 191.—Powerplant schematic piping diagram of fire protection water system.

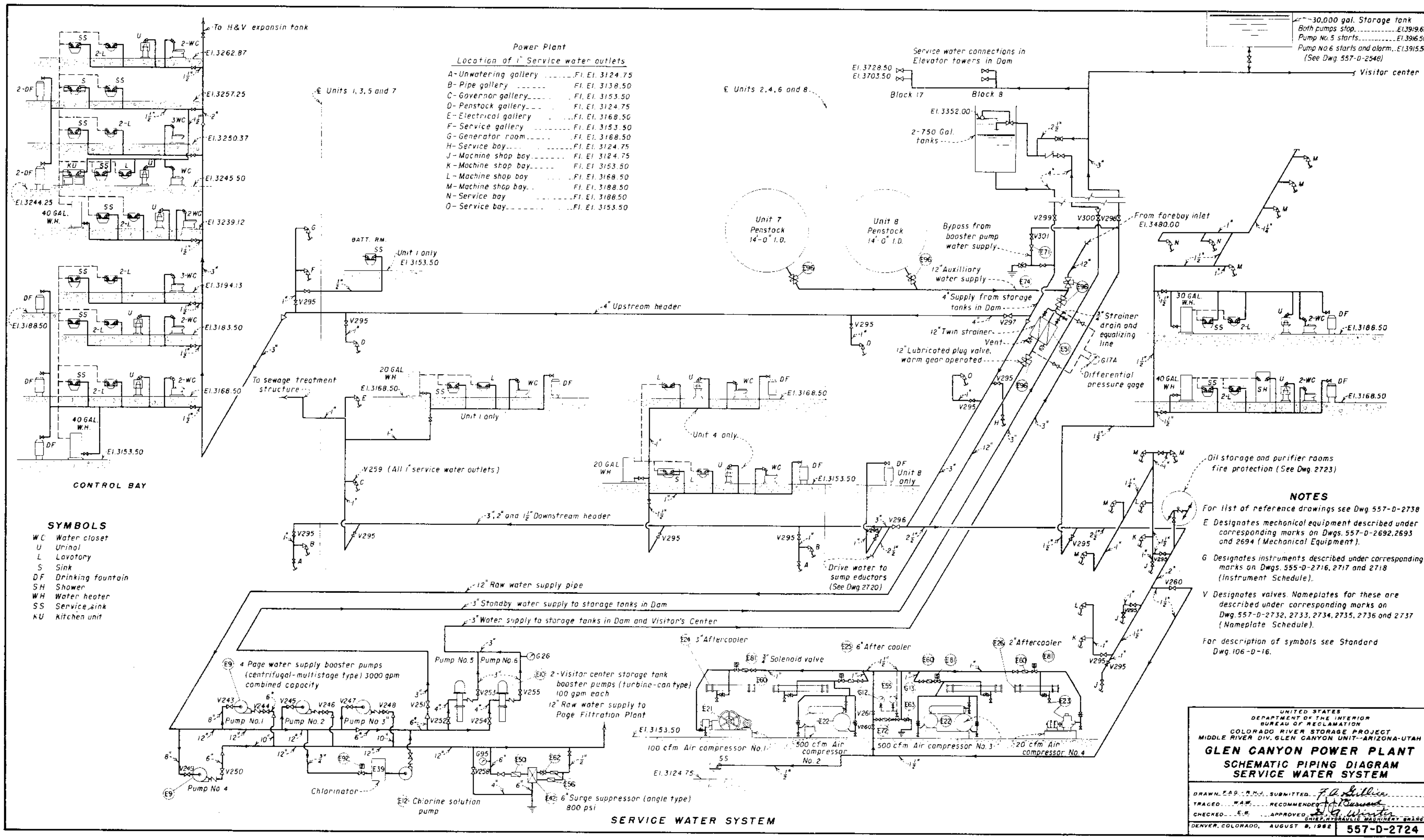


Figure 192.—Powerplant schematic piping diagram of the service water system.

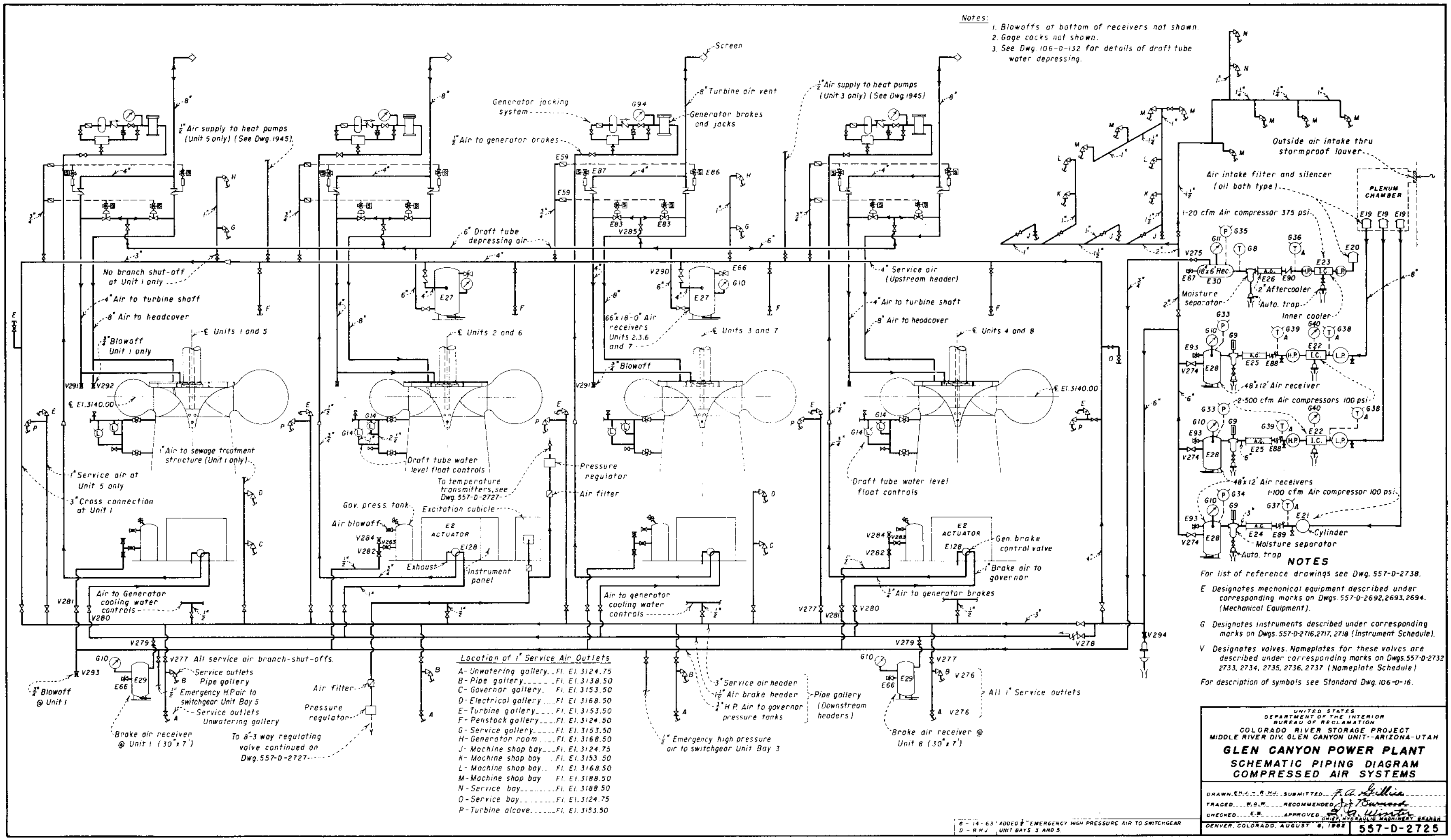


Figure 193.—Powerplant schematic piping diagram of compressed air systems.

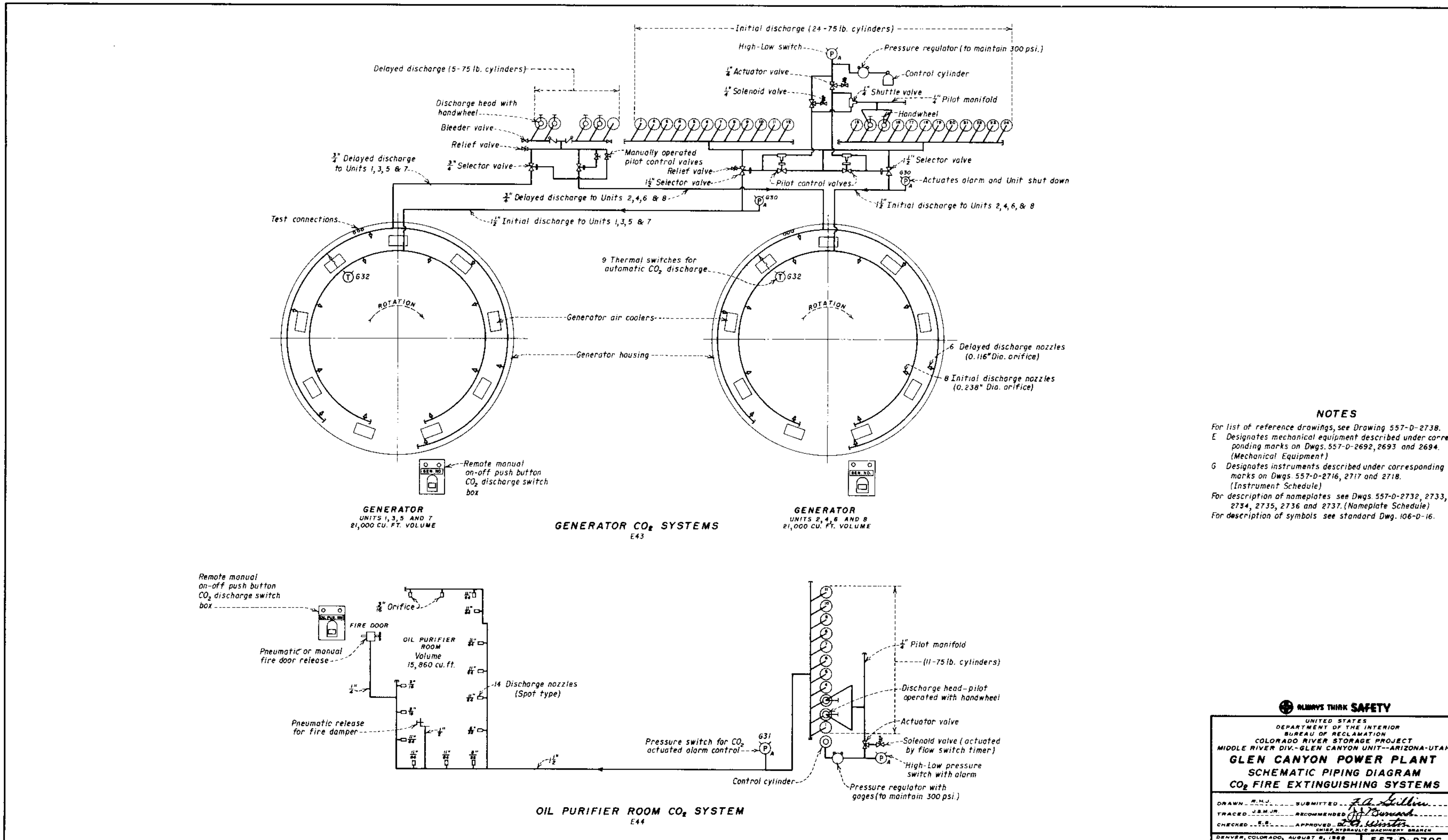


Figure 194.—Powerplant schematic piping diagram for carbon dioxide extinguishing systems.

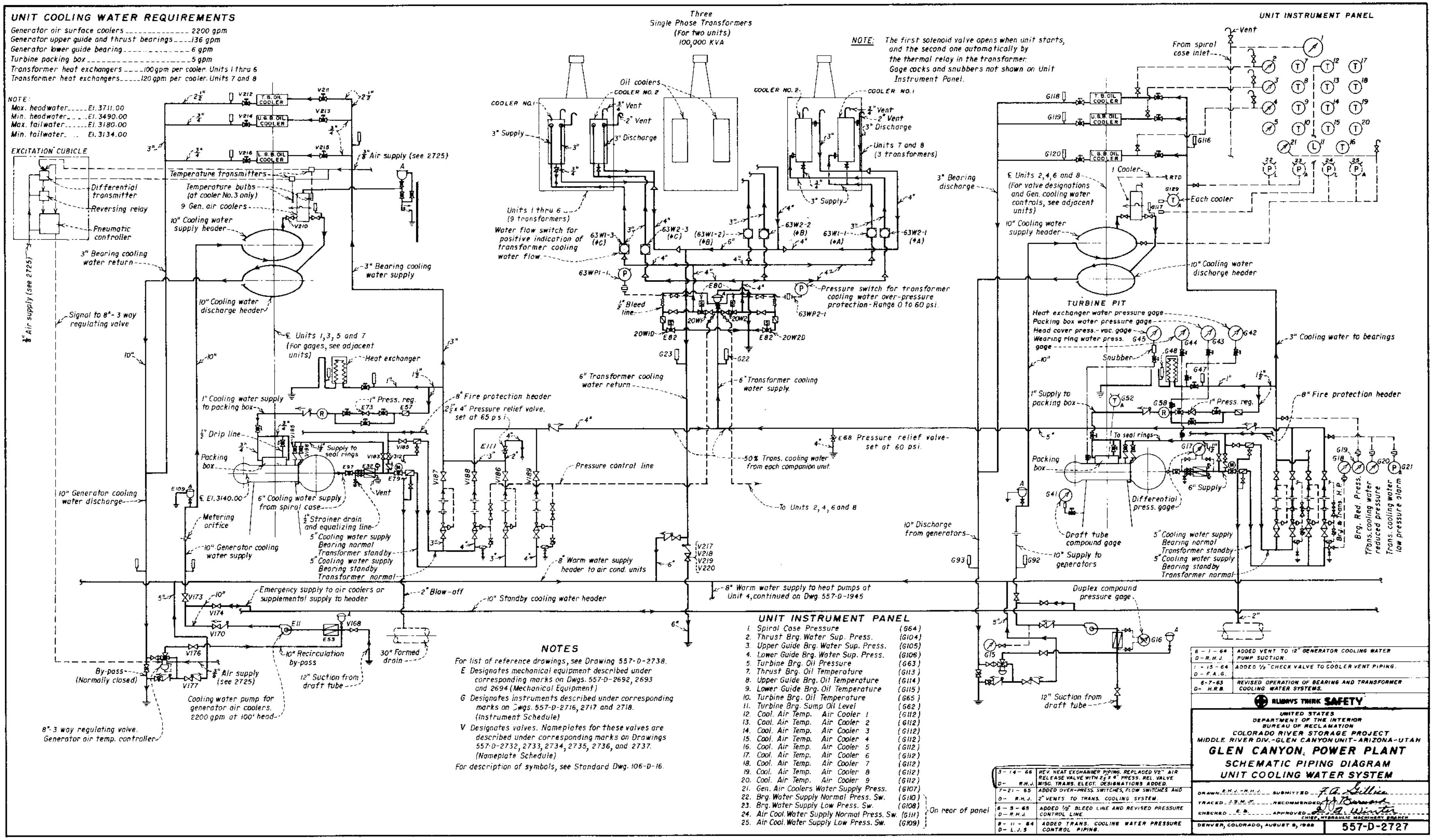


Figure 195.—Powerplant schematic piping diagram of unit cooling water system.

and is opened only for final unwatering of the spiral case and penstock. The drain header is provided with a bypass to the tailrace. This bypass is for the purpose of reducing high-velocity flow into the sump, in the event that a spiral case drain valve and the drain valve at the sump are both opened by mistake when the penstock is under pressure. A check valve in the bypass line prevents backflow from the tailrace to the sump. The unwatering sump pumps are normally controlled automatically by float switches, but may be controlled manually when desired. There are two independent provisions for high sump water level alarm.

Both the penstock and draft tube are provided with vents which will admit atmospheric air when a unit is being unwatered.

(d) *Transformer Oil Handling System.*—The transformer oil handling system is designed to provide for receiving, storing, and purifying the oil and for transferring it to and from the transformers. The system provides for receiving oil, either from the transformers or from a new supply, at an unloading pit near the service bay entrance, and conveying it by gravity to an unfiltered oil storage tank. From the unfiltered storage tank the oil is processed through a purifier to the transformers or to a filtered oil tank where it can be maintained in readiness for use in the transformers. The purifier can process transformer oil at a rate of 1,200 gallons per hour and can be used either for purifying oil in storage or the oil in the transformers with the transformers in position. Oil in the filtered oil storage tank can be transferred to the transformers by a 50-g.p.m. oil transfer pump or by use of the purifier. To be drained or filled, a transformer must be moved into position near the oil unloading pit. An oil makeup line with connections near each transformer is provided to supply small quantities of makeup oil when needed by the transformers.

The normal design criteria for sizing oil pump and pipelines is to fill or drain a transformer in 3 hours or less, at a minimum design oil temperature of 40° F. The system design was based on estimates prior to the purchase of the transformers. Because the transformers were somewhat smaller than anticipated, the system has a capacity capable of draining or filling a transformer in approximately 2-1/2 hours.

Each transformer oil storage tank has a capacity of 10,000 gallons, which is adequate for storing one transformer volume of used oil and one transformer volume of clean oil. The maximum oil capacity of a transformer is approximately 7,650 gallons. Waste oil from the storage tanks can be disposed of by use of the purifier pump.

(e) *Lubricating and Governor Oil Handling System.*—The lubricating and governor oil handling system was designed to provide for handling, storing, and purifying the lubricating and governor oil in a manner similar to that described above for the transformer oil. The system capacity is based on the requirements for one generating unit. The combined volume of oil required for a generator, turbine, and governor is approximately 3,550 gallons. The oil storage tanks have a capacity of 4,000 gallons each. The transfer pump was sized for 30 gallons per minute. The purifier, which is used for both the transformer oil and the lubrication and governor oil, can process lubrication and governor oil at a rate of 600 gallons per hour. Clean oil is pumped from the filtered oil storage tank to the units, and dirty oil is conveyed by gravity from the units to the unfiltered oil storage tank.

The normal design criteria for sizing the pump and pipelines is to fill the largest sump or reservoir in 1 hour with oil at a minimum design temperature of 40° F. Because the generator thrust bearing oil reservoir was slightly larger than anticipated, the filling time for it will be approximately 1 hour and 10 minutes. Overflow lines connected to the drainpiping are provided to reduce the danger of accidental overfilling of the oil reservoirs.

(f) *Service and Domestic Water System.*—The design of the service and domestic water system provides for supply, treatment, and distribution of service and domestic water for the powerplant, dam, and visitor center, and for supply only to the city of Page, Ariz. The design was based on established design standards for domestic water for housing and community facilities, and the powerplant service water requirements. Required water quantities and facilities were determined and the related pipelines, pumps, and storage tanks were sized to meet the present requirements and with reserve capacities to care for projected future demands.

The raw water supply is taken from the forebay and conveyed by gravity through a 12-inch pipeline to a pumping station in the powerplant. A 12-inch standby supply is provided from units 7 and 8 penstocks. The raw water supply passes through a strainer which removes any debris from the water.

The service and domestic water pumping station consists of four 920-g.p.m., centrifugal, multistage-type booster pumps, with a combined capacity of 3,000 gallons per minute, for supplying water to the city of Page, and two 100-g.p.m., turbine, can-type booster pumps for supplying the powerplant, dam, and visitor center.

The water supply for the city of Page is pumped through a 12-inch pipeline to the Page filtration plant where it is processed and distributed by the city.

The service and domestic water for the powerplant, dam, and visitor center is chlorinated at the powerplant by a gas chlorinator which automatically injects the chlorine solution into the suction side of the two 100-g.p.m. booster pumps. The water is pumped to a 30,000-gallon storage tank located near the visitor center building. The booster pumps and chlorinator are automatically controlled by float switches in this tank.

The elevation of the 30,000-gallon storage tank provides sufficient water pressure for the facilities at the visitor center and dam. Water is conveyed from the storage tank by gravity to these areas and distributed to the various facilities.

Two 750-gallon storage tanks are provided in the dam for supplying service and domestic water to the powerplant distribution system. These tanks are located at an elevation suitable for supplying water by gravity at the proper pressure to the powerplant facilities. The water level in these tanks is controlled by a float-operated valve and the tanks can receive water either by gravity from the 30,000-gallon storage tank or directly from the 100-g.p.m. booster pumps in event the pumps are in operation when the tanks need filling. A standby water supply line to the storage tanks in the dam permitted filling these tanks by gravity from the forebay with unchlorinated water during initial powerplant operation prior to completion of the 30,000-gallon storage tank near the visitor center. This standby supply can also be used in the event of future emergencies. To permit servicing of the tanks in the dam, provisions were made for bypassing them by means of a pressure reducing valve connected between the high-pressure gravity supply from the storage tank near the visitor center and the powerplant distribution system.

System demands and facilities that can be isolated for service use only are small relative to the overall powerplant service and domestic water system requirements. For this reason, an economical design required servicing the powerplant auxiliary equipment from the treated water supply, thus permitting combining the service and domestic supplies into one system.

(g) *Unit Cooling Water System.*—The design of the generating equipment required the use of water as a cooling medium for the transformers, generators, turbine and generator bearings, turbine packing box, and turbine wearing rings. Cooling is accomplished by means of oil-to-water heat exchangers for the transformers and the turbine and generator bearings. Primary cooling of the generators is accomplished by air-to-water heat exchangers. Cooling of the turbine packing box and wearing rings is by direct application of water, which also acts as a lubricant for these items. Cooling water for the wearing rings is required only when a unit is operating as a synchronous condenser with the water depressed below the runner. The heat exchangers were furnished by the equipment manufacturers as an integral part of the respective equipment.

The basic design requirement for the unit cooling water system is to furnish cooling water to the equipment in quantities as recommended by the equipment manufacturers, and within the pressure limits as established by the equipment specifications. The cooling water requirements, as estimated by the equipment manufacturers, are listed below:

Equipment	Flow, gallons per minute
Generator air coolers	2,200 per unit
Transformers (three single-phase transformers per two generators)	
Units 1 through 6 (two coolers per transformer)	100 per cooler
Units 7 and 8 (two coolers per transformer)	120 per cooler
Generator thrust bearing	130 per unit
Generator upper guide bearing	6 per unit
Generator lower guide bearing	6 per unit
Turbine guide bearing	10 per unit
Turbine packing box	5 per unit
Turbine wearing rings	40 per unit

The unit cooling water system can be divided into four subsystems, each designed to meet the supply and regulation requirements for a particular demand. One subsystem is provided for the generator air coolers, one for the transformers, one for the turbine and generator bearings, and one for the turbine wearing rings.

The cooling water system for the generator air coolers is designed to provide constant waterflow through the coolers, with temperature regulation being accomplished by varying the amounts of cooling water that is recirculated. The cooling water supply is taken from the draft tube of each unit, passed through a strainer for debris removal, and pumped through the coolers. The water discharging from the coolers passes through an air-operated three-way diverting valve which routes the water in varying proportions to the pump suction for recirculation, or to the tailrace. The system will recirculate all, part, or none of the cooling water, depending on the signal conveyed to the diverting valve from the heat-sensing elements in the generator, to maintain substantially constant temperature in the generator. In event the control signal or operating air to the diverting valve is lost, the valve will go to the "full discharge to tailrace" position and provide maximum cooling and thus "fail safe" operation. A standby line connects the pump discharge of all eight units. In the event of pump failure at one unit, the remaining seven units can provide emergency cooling water, or a pump from another unit not in service can be utilized while the defective pump is being repaired. Pressure switches are installed in the system to prevent unit operation in the event of failure of the generator air cooler water supply system.

The cooling water for the transformers, bearings, packing box, and turbine wearing rings is taken from a common high-pressure connection at the spiral case of each unit. This water passes through a strainer for debris removal before being used in the cooling systems.

The design of the transformer cooling water system departs slightly from the unit concept, in that the cooling water is taken jointly from the two units which are served by a bank of three single-phase transformers. The supply provisions at each unit are similar in design, are connected in parallel, and each has a capacity adequate for supplying total transformer cooling in the event the other unit is shutdown with no cooling water available from its respective spiral case. The high-pressure cooling water supply at each unit is reduced to a low pressure suitable for the transformers by means of a pressure-reducing valve connected in series with two orifice plates. An identical standby pressure-reducing system is provided at each unit to permit servicing the pressure-reducing valves. The combined cooling water supply is piped to an elevation just below the transformer deck for distribution to the transformer heat exchangers. Each of the three transformers in one bank is equipped with two heat exchangers. One heat exchanger on each transformer is used for primary cooling and operates continually when a transformer is in service. The other heat exchanger operates intermittently in response to additional cooling requirements as directed by a heat-sensing element within the transformer. Final distribution of water to the heat exchangers is accomplished by means of two headers. Each header supplies one heat exchanger on each of the three transformers. One header is used to supply water for primary cooling and the other is used for intermittent cooling. Solenoid valves installed in these supply headers control the flow of water to the heat exchangers. Discharge water from the heat exchangers is conveyed by gravity to the tailrace. The heat exchangers are provided with vents to prevent accumulation of air, to prevent build up of pressure, and to serve as vacuum breakers for the water discharge. Other safety features include a pressure relief valve, flow switches, over-pressure switches, and low-pressure switches.

Initial operation showed a need for the following principal modifications to the transformer cooling system:

(1) Replacement of oil-to-water and water-to-water differential pressure switches for the transformer coolers, with flow and water pressure switches.

(2) Increase in vent size to prevent intermittent negative pressure and surging at the cooling water discharge connection.

(3) Addition of bleed lines to offset leakage through the pressure regulators, with resultant "popping" of relief valves and water hammer in the piping, when a transformer bank is shutdown.

Proper adjustment of valves and control lines required considerable study and operational trial runs.

The high-pressure cooling water supply for the turbine and generator bearings, and the turbine packing box, is reduced to the desired pressure in a manner similar to that described above for the transformers. The water is piped to the bearings and packing box where final control is made by hand adjustment of valves. The discharge water from the bearing heat exchangers is piped to the tailrace. Discharge from the packing box is conveyed by gravity to the powerplant drainage sump where it is collected and pumped to the tailrace. The unit bearing and packing box cooling water system utilizes a motor-operated valve for "On-Off" control.

The high-pressure water supply from the spiral case is piped directly to the turbine wearing rings to provide cooling and lubrication when a unit is operating as a synchronous condenser. Control valves in the wearing ring cooling water supply are operated manually.

(h) *Air Compressor Cooling Water System.*—The air compressor cooling water system is shown schematically as a part of the piping diagram for the service and domestic water system (see fig. 192). This system was designed to automatically supply low-pressure cooling water in quantities as recommended by the air compressor manufacturers. A pressure-reducing valve reduces the plant service water from approximately 86 pounds per square inch to the required 20 pounds per square inch supply for the air compressors and aftercoolers. A bypass line, with a manually operated valve, is provided for emergency operation when the pressure-reducing valve is out of service. A pressure relief valve and a low-pressure switch installed in the low-pressure supply assures proper system pressure. Cooling water is automatically admitted to the compressors and aftercoolers by solenoid valves which open when any compressor is operated. The cooling water discharge drains by gravity to the powerplant drainage sump. Open sight flow tunnels are provided in the cooling water discharge lines for flow detection and to aid in flow adjustment. Final adjustment of cooling water flow is made manually by means of needle valves for each separate application.

(i) *Fire Protection Water System.*—The design of the fire protection water system supplies water to, and provides for, the following fire protection facilities:

(1) Thirty-two firehose cabinets located strategically throughout the powerplant. These cabinets are equipped with 1-1/2-inch firehose valves, adjustable pressure restricting valves, 75 feet of firehose, and spray nozzles adjustable from solid stream to fog. This equipment is suitable for one-man use.

(2) Three 2-1/2-inch firehose valves, two located just inside the main doors at each end of the transformer deck, and one just inside the main door to the machine shop. Two-wheeled firehose carts with 250 feet of hose and fixed fog nozzles are provided for use with the 2-1/2-inch firehose valves. The primary purpose of this equipment is for use on or near the transformer deck, but it may be used in other areas within hose range. This equipment is designed for two-man operation.

(3) Automatic sprinkler systems for the oil storage and oil purifier rooms. These systems utilize fusible-link sprinkler heads for automatic operation in event of fire in the oil rooms. Flow switches in the supply lines provide for alarm in event of fire at either location. In addition to alarm, the flow switch for the purifier room also starts a 5-minute timer. If not interrupted, the timer will initiate water shutoff and carbon dioxide discharge for the purifier room. Flow of water to the oil storage room will continue until shut off manually.

(4) Transformer water spray system. Each of the four banks of three transformers is provided with a fire protection water spray system. This system utilizes fixed spray nozzles for complete water spray coverage of the transformers. The "On-Off" control is by means of motor-operated valves, operated by pushbuttons at control stations just inside the doors at each end of the transformer deck.

The water supply for the fire protection system is taken from the high-pressure connection at each spiral case. A header interconnects the supply from each sprial case to assure an adequate supply of fire protection water at all times. However, due to the low pressure ratings available for the fusible-link sprinkler heads in the oil rooms, these systems are supplied from the service and domestic water system. A connection through a manually operated valve to the fire protection water system was provided for the oil storage room to insure against running out of water from the 1,500-gallon storage tank prior to installation of the 30,000-gallon storage tank. This connection should not need to be used after installation of the larger tank.

Blowoffs are provided on all dead-end firelines and on the main headers in the powerplant for periodic purging of the fire protection waterlines. The blowoffs in the oil rooms are also used for testing the sprinkler systems alarms and controls. During blowdown of the firelines in the oil purifier room, the electric controls for the purifier room carbon dioxide system must be turned off to prevent carbon dioxide discharge.

(j) *Carbon Dioxide Fire Extinguishing Systems.*—The carbon dioxide fire extinguishing systems are designed to provide automatic fire protection for the generators and oil purifier room. The design of the systems was based on guides established by the National Board of Fire Underwriters and Bureau guides. In accordance with these standards, the following quantities of high-pressure carbon dioxide were provided:

Generator initial discharge—Twenty-four 75-pound cylinders
Generator delayed discharge—Five 75-pound cylinders
Oil purifier room—Eleven 75-pound cylinders

There are four separate systems for the generators. Each system protects two generators. Automatic valves route the carbon dioxide to the proper generator in event of fire. One system protects the oil purifier room.

The systems include cylinder banks, routing valves, piping to hazards, discharge nozzles, sensing devices, test circuits, switches and trips for alarms, fire doors, and dampers, and all other equipment necessary to make a complete installation.

Carbon dioxide discharge to the hazards can be initiated by the following methods:

Generator initial discharge:

(1) Automatically by thermoswitches or differential relays of the affected generator.

(2) Manually by a remote control switch adjacent to each generator.

(3) Manually by a remote control switch on the unit auxiliary control panel.

(4) Manually by handwheel operation at the cylinder bank.

Generator delayed discharge:

Manually by handwheel operation at the cylinder bank (discharge time intervals to be determined by field tests——20 minutes will be used prior to tests).

Oil purifier rooms:

(1) Automatically by an electrical timer after 5-minute water sprinkler operation.

(2) Manually by remote control switch located outside the door.

(3) Manually by handwheel operation at the cylinder bank.

A bank of 29 spare carbon dioxide cylinders provides for immediate replacement of discharged cylinders.

(k) *Portable Carbon Dioxide Fire Extinguishing Equipment.*—Portable carbon dioxide fire extinguishers are located throughout the powerplant for first aid use on any type of fire. Extinguishers of the 5- and 15-pound sizes are paired, and are wall mounted at suitable locations. Wheeled portable 100-pound units are available for larger fires, and are located near areas of greater hazard.

The following portable carbon dioxide equipment is provided:

Two 100-pound wheeled-portable carbon dioxide carts with two 50-pound cylinders and 40-inch-long hose and discharge horn
Fifty-three 15-pound carbon dioxide cylinders with squeeze-type valve and 3-foot-long hose and discharge horn
Fifty-three 5-pound carbon dioxide cylinders with squeeze-type valve and discharge horn

(l) *Compressed Air System.*—The air compressor installation conforms to usual standards for plants of this size, as follows:

Quantity	Compressor rating	Use
1	100 c.f.m., 100 p.s.i.	Service air
2	500 c.f.m., 100 p.s.i.	Service air
1	20 c.f.m., 375 p.s.i.	Governor pressure tanks

The 100-c.f.m. compressor supplies normal small demands from the system, including supplying two separate receivers for the generator airbrakes. The 500-c.f.m. compressors are available for large demands including maintenance, repair, and construction, and for depressing the tailwater in the draft tube for synchronous condenser operation. The 20-c.f.m. compressor is used for pressurizing the governor pressure tanks and as a standby supply for the air circuit breakers. All compressors are equipped with aftercoolers and moisture separators.

The compressed air system provides storage capacity adequate for depressing two units simultaneously. The distribution system includes 1-inch service connections throughout the powerplant, depressing connections to each turbine draft tube, brake air to each governor and generator, supply to air-operated valves, high-pressure connections to governor pressure tanks, and provisions for supplying other miscellaneous demands.

The system includes protective devices, gages, filters, blowoff connections, and control equipment, necessary to provide safe and convenient operation. The compressors in the 100-p.s.i. service air system are equipped for either automatic or manual control. The compressor for restoring the high-pressure air cushion in the governor pressure tank is equipped for manual control only.

(m) *Tailwater Depressing System.*—A tailwater depressing system is provided to reduce power consumption when a unit is motoring. This system unwaters the turbine runner by releasing stored compressed air into the draft tube through holes in the head cover, runner, and shaft. The system is designed to depress the water to about 3 feet below the runner and keep it near that level while a unit is motoring. The supply of stored compressed air at a pressure of 85 to 105 pounds per square inch is adequate for unwatering two units simultaneously in about 30 seconds. Unwatering one unit can be accomplished somewhat faster. System pressure can be restored in approximately 5 minutes after depressing one unit and approximately 10 minutes after depressing two units, after which the 100-c.f.m. compressor should be able to maintain the depressed level.

All necessary operations for the depressing system are automatic, under control of a master switch. A tandem float switch in the draft tube controls admission of air. Piping to the floats is specially arranged to minimize misoperation caused by slugs of air and water.

(n) *Sewage Pumping Units.*—The sewage pumping units are provided to transfer the domestic waste drainage from the powerplant to a sewage treatment unit outside the plant. Package units consisting of steel receiving tanks, centrifugal pumps, float switches, and the necessary controls for pump operation, were selected for this purpose. Each unit contains two pumps, each pump sized to meet the expected demand, thus permitting one pump to be removed from service for maintenance. Three units are provided—one with a capacity of 250 gallons per minute, one with a capacity of 150 gallons per minute, and one with a capacity of 75 gallons per minute.

(o) *Water Level Gages.*—Indicating and recording water level gages for the forebay and afterbay are located in the main control room. The afterbay stilling well, with float-operated level indicator and transmitter, is located in the machine shop bay of the powerplant. The forebay stilling well, with float-operated level indicator and transmitter, is located in the dam.

F. MAJOR ELECTRICAL EQUIPMENT

84. GENERATORS. (a) *Ratings and Characteristics.*—Each of the eight generators is rated 125,000 kv.-a, *90-percent power factor, 3-phase 60 cycles, 13,800 volts, and 150 revolutions per minute. The generator rating and capacity were selected to match the turbine output at 450-foot head and full turbine gate output of 155,500 horsepower. The effective head on the turbine for power may range between 341 and 560 feet.

The power factor of the generators was selected on the basis of supplying the required kilovolt-amperes to the power system. The generator voltage of 13,800 volts was selected on the basis of being the most economical from an overall cost comparison and using equipment of standard manufacture.

Generator characteristics were specified which are normal for generators of this type, except for the low ratio of X_q'' to X_d'' of 1.2 and the exciter speed of response of 1.5. Both of these characteristics were specified because of anticipated transmission system needs due to the remote location of the plant from load centers. The generators were designed to withstand a runaway speed of 277 revolutions per minute.

An efficiency of 97.35 percent at 100-percent rated output was guaranteed by the manufacturer. It was specified that the price of each generator would be reduced by $6,700 for each 0.01 of one percent that the actual efficiency, as determined by test, is less than the warranted efficiency at 100-percent rated output and 90-percent power factor.

The following technical data on characteristics of the generator were calculated by the manufacturer:

(1) The losses in kilowatts at *90-percent power factor are:

	Full load	3/4 load	1/2 load	1/4 load
Friction and windage	792	792	792	792
Core	880	880	880	880
Armature I^2R	290	163	72	18
Stray load	290	163	72	18
Field I^2R	388	305	231	168
Total losses	2,640	2,303	2,047	1,876

* After acceptance tests, the generators were rerated at 95-percent power factor; however, all losses and other data shown herein pertain to the original 90-percent power factor rating.

The losses in kilowatts at 100-percent power factor are:

	Full load	3/4 load	1/2 load	1/4 load
Friction and windage	792	792	792	792
Core	880	880	880	880
Armature I^2R	290	163	72	18
Stray load	290	163	72	18
Field I^2R	234	200	168	140
Total losses	2,486	2,198	1,984	1,848

(2) The resistance of the armature windings at 75° C. is 0.00354 ohm per phase.

The resistance of the field winding at 75° C. is 0.379 ohm.

(3) The total capacitance of one phase of the armature windings to ground is 0.9 microfarads

(4) The deviation factor of wave form is less than 10 percent.

(5) The direct-axis synchronous reactance is 105 percent.

The quadrature-axis synchronous reactance is 68.6 percent.

(6) The direct-axis, rated-current, transient reactance is 33 percent.

(7) The direct-axis, rated-voltage, transient reactance is 30 percent.

(8) The direct-axis, rated-voltage subtransient reactance is 20.5 percent.

The quadrature-axis, rated-voltage subtransient reactance is 22.1 percent.

(9) The zero sequence reactance is 15.0 percent.

(10) The negative sequence reactance, at rated voltage is 21.1 percent.

(11) The negative sequence reactance is 0.8 percent.

(12) The short-circuit ratio is 1.155 percent.

(13) The direct-axis, open-circuit time constant is 6.17 seconds.

(14) The direct-axis, short-circuit time constant is 1.94 seconds.

(15) The initial, root-mean-square, symmetrical, 3-phase, short-circuit current* is 488 percent.

(16) The initial, root-mean-square, symmetrical, single-phase, short-circuit current* is 415 percent.

(17) The initial, root-mean-square, symmetrical, phase-to-neutral, short-circuit current* is 528 percent.

(18) The sustained, root-mean-square, 3-phase, short-circuit current* is 115.3 percent.

(19) The sustained, root-mean-square, single-phase, short-circuit current* is 166.7 percent.

(20) The sustained, root-mean-square, phase-to-neutral, short-circuit current* is 258 percent.

(21) The maximum value of no-load, balanced, telephone-influence factor is 50.

(22) The maximum value of no-load, residual, telephone-influence factor is 30.

(23) The calculated regulation, in percent of rated voltage, is 25 percent.

(24) The field current and collector-ring voltage required for rated kv.-a. output (125,000) at rated voltage (13,800) and power factor (0.90) are 1,010 amperes and 383 volts, respectively.

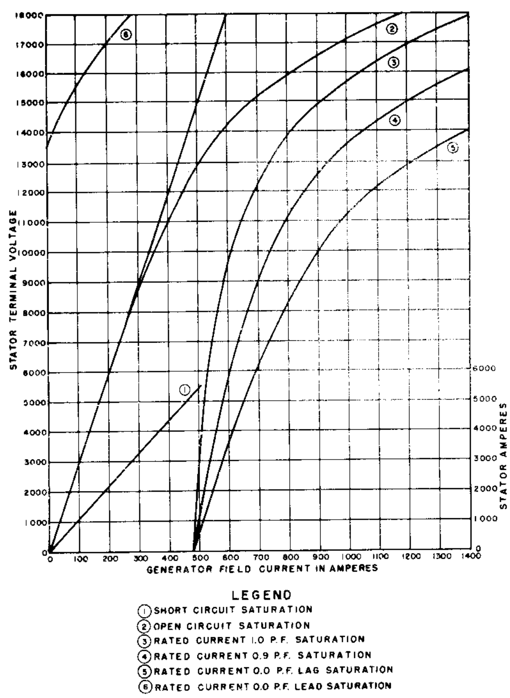
(25) The field current at 143,750-kv.-a. output at 14,490 volts and 0.90 power factor is 1,190 amperes.

(26) For characteristic curves, see figure 196.

(27) The approximate maximum operating temperature at rated output for:

Armature winding (by embedded detector) is 100° C.
Field winding (by resistance) is 100° C.

*With initial field current as required for rated voltage at rated speed and open circuit.



CALCULATED CHARACTERISTICS
GLEN CANYON GENERATORS

Figure 196.—Generator calculated characteristics.

Collector rings (by thermometer) is 105° C.
Cores and mechanical parts (by thermometer) is 90° C.

(28) The maximum line-charging capacity of each generator, neglecting heating, without the generator becoming completely self-excited, when operating at rated voltage, rated frequency, and when connected to a transmission circuit or circuits, open circuited at receiving end is 115,500 kv.-a.

(29) The burden on voltage regulator potential circuit, per phase, is 400 volt-amperes, 1.0 power factor. The burden on voltage regulator current circuit is 225 volt-amperes, 0.7 power factor.

(30) The test voltage to be used for induced dielectric test is 6,000 volts per turn.

(31) The integrated product capability, I_2^2T, is 40.

(32) The moment of inertia of the rotating parts of one generator and exciter, in pound-feet squared at a radius of 1 foot is 72,100,000.

(33) The amount that the rotor must be raised to dismantle the bearings is 0.25 inch.

(34) The quantity of oil, for one generator, required to fill:

The thrust bearing is 2,100 gallons.
The upper guide bearing is 160 gallons.
The lower guide bearing is 200 gallons

(35) The approximate quantity of water, for one generator operating at 115 percent rated kv.-a., at 25° C. required to cool:

The surface coolers is 2,200 gallons per minute.
The thrust bearing is 130 gallons per minute.
The upper guide bearing is 6 gallons per minute.
The lower guide bearing is 6 gallons per minute.

(36) The net volume of air within the housing and ducts of one generator, for which carbon dioxide for fire protection must be provided, is 21,000 cubic feet.

(37) The weight of:

One complete generator, including direct-connected exciter, is 1,550,400 pounds.
Rotating parts, including direct-connected exciter, is 740,000 pounds.
Heaviest individual part as disassembled for shipment is 146,000 pounds.
Heaviest part that must be lifted by crane during assembly or disassembly is 704,000 pounds.

(b) *Construction Details.*—The eight generators were manufactured under invitation No. DS-5522 by General Electric Co. Each generator is a vertical-shaft synchronous machine with a thrust bearing and a guide bearing above the rotor and a guide bearing below the rotor. The thrust bearing supports the entire weight of the rotating parts of the generator, exciter, and turbine. The upper bearing bracket supports the thrust bearing and its load as well as the stationary parts of the exciter and transmits this entire load to the stator frame. The stator frame is supported on a concrete foundation by 12 foundation caps. Each of the six caps under the upper bearing bracket arms will support a vertical load of 360,000 pounds and the six alternate caps between the bracket arms will each support a vertical load of 40,000 pounds. Each cap will support a maximum tangential force of 220,000 pounds based on single-phase, short-circuit conditions.

The lower guide bearings and the combination airbrakes and lifting jacks are supported by the lower bearing bracket. The lower bearing bracket is supported by six soleplates, each of which will support a vertical load of 164,000 pounds during jacking, and a tangential force due to braking of 2,800 pounds.

The thrust bearing was made by the General Electric Co., and consists of 10 babbitted stationary segments and a cast iron runner. The cast iron runner is secured to the rotor shaft by means of the thrust collar, and the stationary shoes are mounted on a flexible support of precompressed springs. The thrust bearings and guide bearings are insulated to prevent circulating currents from passing through the bearing surfaces. Test terminals are provided for use in connection with testing of the bearing insulation. The upper guide bearing, the thrust bearing, and the lower guide bearing are each located in separate oil reservoirs. Heat from the bearings is removed by water-cooled cooling coils provided in each oil reservoir. The bearings are self-lubricated except during starting or stopping operations when high-pressure oil is forced between the bearing surfaces of the thrust bearing to help maintain an oil film.

The stator winding is wye connected. The armature windings are provided with ASA class B insulation. The armature windings were designed to withstand the maximum 3-phase, short-circuit current of the generator. Twelve resistance temperature detectors were provided in the stator windings for temperature detection and indication.

The field poles also are provided with ASA class B insulation. The field poles were provided with amortisseur windings of the low-resistance type to improve stability under fault conditions and to reduce voltage distortion under conditions of single-phase fault.

Devices for the following purposes are furnished with each generator:

For indication on the generator instrument panel:

(1) Thrust bearing oil temperature.

(2) Upper-guide bearing oil temperature.

(3) Lower-guide bearing oil temperature.

(4) Cooling air temperature, each cooler.

(5) Thrust bearing cooling water supply pressure.

(6) Upper-guide bearing cooling water supply pressure.

(7) Lower-guide bearing cooling water supply pressure.

(8) Air cooler water supply header pressure.

(9) Thrust bearing high-pressure oil system pressure.

(10) Two air cooler water-supply header pressure switches (separately adjustable).

(11) Two bearing cooling water-supply pressure switches (separately adjustable).

For indication at the apparatus:

(1) Cooling water supply temperature located in the air cooler water supply header.

(2) Bearing cooling water supply temperature located in water supply line.

(3) Cooling water return temperature located in the water outlet from each surface cooler.

(4) Thrust bearing water return temperature located in thrust bearing water outlet.

(5) Upper-guide bearing water return temperature located in upper-guide bearing water outlet.

(6) Lower-guide bearing water return temperature located in lower-guide bearing water outlet.

(7) Oil level sight gage at each reservoir.

The following devices are furnished with the generator for remote operation of alarm for excessive temperature or loss of pressure, control of bearing cooling water, and starting control interlock:

(1) One thrust bearing oil level float switch for low- and high-level alarm and starting interlock.

(2) Two adjustable temperature devices in the thrust bearing metal.

(3) One adjustable temperature device in the upper-guide bearing metal.

(4) One adjustable temperature device in the lower-guide bearing metal.

(5) One temperature device at each surface cooler (nine).

(6) One temperature device in the thrust bearing oil reservoir.

(7) One temperature device in the upper-guide bearing oil reservoir.

(8) One temperature device in the lower-guide bearing oil reservoir.

(9) One pressure device in the main cooling water supply header (pressure failure).

(10) One pressure device in the bearing cooling water supply header (pressure failure).

(11) One pressure device in main cooling water supply header (normal operating pressure) for starting control interlock.

(12) One pressure device in the bearing cooling water supply header (normal operating pressure) for starting control interlock.

The following resistance temperature detectors are furnished for detection at the apparatus and remote indication:

(1) Thrust bearing—two detectors.

(2) Upper guide bearing—one detector.

(3) Lower guide bearing—one detector.

(4) Armature winding—twelve detectors.

(5) Generator ambient temperature—one detector.

Each generator is provided with an enclosed cooling system, complete with metal housing, air passages, and nine water-cooled heat exchangers spaced around the periphery of the stator. Radial blades attached to the upper and lower ends of rim segments on the rotor spider serve as blowers for the generator. The cooling system with one heat exchanger out of service should maintain a satisfactory air temperature when the generator is operated at rated output. The water requirement for the heat exchangers is 2,200 gallons per minute. A small rotor fan with radial blades is installed under the main exciter for circulating cooling air through the exciter.

The housings are practically airtight to insure effective operation of the automatic carbon dioxide fire-extinguishing system. The units are provided with pressure relief doors on the side of the housings for relief of excessive carbon dioxide pressure. Each generator is equipped with two independent ring headers supported near the stator winding and above the rotor, one for the initial discharge and one for delayed discharge from the high-pressure carbon dioxide gas supply. The initial discharge header has eight nozzles located at even intervals around the periphery of the housing, and the delayed discharge header has six nozzles located at even intervals around the periphery of the housing. A thermo-switch is located in the hot air passage ahead of each cooler. The thermo-switches are of the single-pole, fixed-temperature, self-resetting tubular-shell type rated at 0.5 ampere and designed to close a 125-volt, direct-current, ungrounded circuit when the surrounding air reaches a temperature of 185^{o} F., thereby effecting the release of the carbon dioxide. Release of carbon dioxide may also be initiated by a break-glass station located on the a-line wall of the generator floor.

Four electrical strip heaters, each rated 4,500 watts, 3-phase, 480 volts, are used to prevent condensation within the windings when the temperature approaches the ambient air temperature. The heaters are symmetrically spaced below the stator winding, and the heater contactor is controlled by a differential temperature control device which operates the heaters to maintain the temperature inside the generator at a minimum of 10^{o} F, above the outside temperature.

A braking system consisting of air-operated brakes mounted on each of the lower bearing bracket arms is provided for each generator. The brake shoes are applied to the brake ring on the lower side of the rotor and are designed with sufficient capacity to bring the rotating parts to a stop from one-half normal operating speed within 7-1/2 minutes after the brakes are applied. The brakes are also designed for use as hydraulic jacks to lift the generator and turbine rotating parts to permit removal or adjustment of the thrust bearings. A 440-volt, 3-phase, 60-cycle electric-motor-operated high-pressure oil pump is used to operate the jacks.

(c) *Excitation System.*—A direct-connected, direct-current, vertical-shaft, shunt-wound type exciter, mounted on top of each generator, was selected to provide generator excitation. The main exciter is connected directly to the generator field through a shunt with no main field rheostat. The field current of the alternating-current generator is controlled by varying the field excitation of the main exciter. Voltage regulators acting in conjunction with the rotating magnetic amplifier automatically control the exciter field current.

Following any load rejection up to 115 percent of rated output on the generator, the voltage regulating equipment is designed to restore generator terminal voltage to within 5 percent of the voltage being held before load rejection. The regulators will maintain average generator voltage within plus or minus 0.5 percent when operating under steady load conditions for any load or excitation within operating range of the generator. Under steady speed conditions for any overspeed up to 150 percent of normal, the voltage regulators will maintain generator voltage within 5 percent of the value the voltage regulators were holding before overspeed. For an overspeed between 150 and 175 percent of normal, the voltage regulators will maintain generator voltage within 10 percent of the value the voltage regulators were holding before overspeed.

A permanent magnet generator and housing are mounted on each exciter bracket and shaft.

The following data were calculated by the manufacturer as applicable to the exciters:

(1) The exciter rating at 150 revolutions per minute is 655 kilowatts, 500 volts, and 1,310 amperes.

(2) The field current of the exciter at rated output as per (1) above is 79.6 amperes.

(3) The exciter response ratio is 2.0.

(4) The maximum voltage when delivering rated current is 750 volts.

(5) The maximum temperature rise at rated output for:

Armature winding (by thermometer) is 40° C.
Field winding (by resistance) is 60° C.
Commutator (by thermometer) is 55° C.
Core and mechanical parts (by thermometer) is 40° C.

(6) The weight of:

A complete exciter is 38,100 pounds
Rotating parts is 12,700 pounds

(7) The direct-current requirements of excitation control system is 125 volts and 0.98 ampere.

(8) Additional data:

The number of poles is 12.
The current density in the brushes at rated load is 78.6 amperes per square inch.
The maximum reduction in commutator depth, measured on the radius, permissible for turning down is 0.5 inch.

(9) The voltage regulator voltage time response is 0.03 second.

(10) The excitation system voltage response ratio is 1.7.

(11) The continuous capacity in amperes of rotating and magnetic amplifiers and associated equipment supplying excitation current to the main exciter is 80 amperes.

85. GENERATOR ASSOCIATED EQUIPMENT. (a) *Generator Surge Protective Equipment.*—The surge protective equipment for each generator is mounted in a separate assembly. Surge protection from external high-voltage surges is provided by grounding each main lead of the generator through a lightning arrester and capacitors connected in parallel. The lightning arresters are 15-kilovolt, station-type arresters which are used to limit the amplitude of lightning impulse waves. The capacitors are single-pole, station type, having an electrostatic capacity of 0.25 microfarad and are connected two in parallel to each phase to increase the capacity to the necessary 0.5 microfarad per phase. The capacitors reduce the steepness of the wave front of a surge so as to decrease the turn-to-turn and coil-to-coil voltage stresses in the generator windings.

(b) *Generator Neutral Grounding Equipment.*—The stator winding of each generator is wye-connected and the neutral is connected with a cable to one terminal of the high side of a distribution transformer. The distribution transformer is rated 167-kv.-a., 12,000 to 120/240-volt, single-phase, 60-cycle. The other high-voltage terminal of the transformer is connected to the station grounding system. The secondary terminals of the distribution transformer are connected to a 0.12-ohm grid resistor, having a 30-second rating of 1,320 amperes, and a ground protective relay. The distribution transformer and resistor neutral grounding was used because of the lower cost in comparison with reactor grounding and because it eliminates most of the disadvantages of an ungrounded system. The sizes of the transformer and resistor were based on the charging current in case of a line-to-ground fault, which depends on the capacitance to ground of the interconnected generator voltage equipment. The neutral grounding equipment was furnished by General Electric Co., under invitation No. (D) 90,657-B.

(c) *Generator Voltage Switchgear.*—Each generator is connected to a bank of three single-phase power transformers through an air circuit breaker which is a part of the indoor station-type switchgear assembly. The air circuit breakers are of the compressed air (air blast), pneumatically operated type and are rated as follows:

Rated volts	14,400
Maximum design volts	15,500
Minimum volts for rated interrupting megavolt-amperes	12,000
Low-frequency withstand test, root-mean-square, volts	50,000
Impulse crest, volts	110,000
Frequency, cycles per second	60
Rated continuous current-carrying capacity, amperes	7,000
Rated continuous current-carrying capacity without benefit of supplemental cooling, amperes	5,000
Momentary current-carrying capacity, amperes	190,000
Four-second current rating, amperes	120,000
Interrupting rating, 3-phase capacity, kilovolt-amperes	2,500,000
Rated interrupting current, maximum, amperes	120,000
Interrupting time rating, 60-cycle basis, cycles	8

Each circuit breaker is normally pneumatically operated, but provision has been made for an emergency manual trip operation. The circuit breakers are trip-free in all positions and can be manually closed for test or maintenance. Visual indication of the circuit-breaker contact position from the front of the cubicle has been provided. Each air circuit breaker has been provided with interlocks to prevent opening or closing of the disconnecting switch unless the circuit breaker is open, to prevent access to the interior of the housing or cubicle unless the disconnecting switch is open, and to prevent closing of the disconnecting switch when the interior of the housing or cubicle is open.

The switchgear is provided with extensions of the main bus for connection to the generators through isolated phase bus at the bottom, and for connection to the power transformer low-side delta network through isolated phase bus at the top of the cubicle.

The switchgear has been provided with three current transformers on the generator side of the breakers and six on the transformer side of the breakers. The line side transformers have an ampere ratio of 7,000/5 and are used for generator differential relaying. The generator side transformers have an ampere ratio of 7,000/5 and are used for transformer differential relaying. The differential relays are located on the main control board. The switchgear assemblies were furnished by General Electric Co. under invitation No. DS-5828.

(d) *Main Buses.*—The generator-voltage bus structures are completely metal enclosed, providing phase-isolated arrangement of the conductors and all associated circuit equipment. Phase isolation is obtained by placing each conductor in an individual metal enclosure and separating each phase enclosure so that an insulating air space exists between them. This assures maximum protection against phase-to-phase faults. The enclosures are suitable for installation either indoors or outdoors as is required by this particular installation. No special arrangements are necessary for indoor-outdoor transitions except that isolating barriers are used where the bus structures pass through the transformer deck. The conductors are aluminum and are jointed to the conductors of the adjacent section by flexible connectors designed to compensate for movement caused by temperature changes or uneven settling of the supporting structure. Flexible connectors are also used for bus connections to the generators, switchgear, and transformers. The conductors are designed to carry their full rated current without exceeding a temperature rise of 50° C. above an external ambient temperature of 40° C. One insulator, designed to withstand short circuit stresses, is mounted at each supporting ring to support the conductor inside the enclosure. Each bus structure is connected to a ground bus so that there can be no circulating ground currents through the bus structure. The ground bus is then connected to the station ground system.

Each bus structure is complete with a grounding switch for protection of personnel during maintenance procedures. The grounding switch is a 3-pole, single-throw, gang-operated switch with a momentary current-carrying capacity of 120,000 amperes. Each switch can be opened or closed with an operating mechanism by one man. The grounding switch is equipped with six single-pole, control circuit, auxiliary switches which can be readily changed to circuit opening or circuit closing as required, and so constructed that false indication cannot be given if the mechanism fails to make a complete stroke of the grounding blade. Each switch is located adjacent to the even-numbered switchgear cubicles.

Each bus structure is equipped with an air compressor to maintain the internal portion of each bus at a positive pressure of 1 inch of water with respect to atmospheric pressure. This feature was included to aid in keeping dust and water vapor from entering the bus enclosure. Check valves are installed in the piping supplying each separate section of each bus structure to prevent migration of air from one section of bus to another. The air compressor for each bus structure is located beneath the grounding switch adjacent to the even-numbered generator switchgear cubicles.

The bus structures and associated equipment mentioned above were furnished by Westinghouse Electric Corp. under invitation No. DS-5828.

86. POWER TRANSFORMERS. The power transformer installation is located on the transformer deck of the powerplant, and consists of four banks of three single-phase transformers each. Single-phase transformers were selected because of physical size and weight limitations imposed by shipping restrictions. Each bank is connected delta on the low-voltage side and solidly grounded wye on the high-voltage side.

The three power transformer banks for generating units 1 and 2, 3 and 4, 5 and 6, are rated at 345 kilovolts on the high side. These transformers were furnished by Pennsylvania Transformer Division of McGraw-Edison Co. under schedule No. 1 of invitation No. DS-5780. Each of these transformers is an outdoor, oil-immersed unit, designed for operation with a temperature rise of 55° C. above a 25° C. ambient. The transformers are forced-oil-water cooled with the following ratings:

Kilovolt-ampere rating	100,000
High voltage (rated), 3-phase bank	345 kilovolts
High-voltage basic impulse level	1,175 kilovolts
Taps (high voltage)	2-1/2 percent (two above and two below 345 kilovolts)
Low voltage (rated)	13.7 kilovolts
Low-voltage basic impulse level	110 kilovolts
Taps (low voltage)	None
High-voltage neutral basic impulse level	110 kilovolts

Each transformer is equipped with an inert gas pressure oil-preservation system which maintains a cushion of dry nitrogen gas at positive pressure over the top oil surface in the transformer tank, and a cooling system consisting of two heat exchangers and two motor-driven oil pumps.

The power transformer bank for generating units 7 and 8 are rated at 230 kilovolts on the high side. These transformers were furnished by Westinghouse Electric Corp. under schedule No. 2 of invitation No. DS-5780. Each of these transformers is an outdoor, oil-immersed unit, designed for continuous operation with a temperature rise of 65° C. above a 25° C. ambient. The transformers are forced-oil-water cooled with the following ratings:

Kilovolt-ampere rating	100,000
High voltage (rated), 3-phase bank	230 kilovolts
High-voltage basic impulse level	825 kilovolts
Taps (high voltage)	2-1/2 percent taps two above and two below 230 kilovolts)
Low voltage (rated)	13.8 kilovolts
Low-voltage basic impulse level	110 kilovolts
Taps (low voltage)	None
High-voltage neutral basic impulse level	110 kilovolts

Each transformer is equipped with an inert gas pressure oil-preservation system which maintains a cushion of dry nitrogen gas at positive pressure over the top oil surface in the transformer tank, and a cooling system consisting of two heat exchangers and two motor-driven oil pumps.

Each transformer is equipped with the following accessories:

(1) Dial-type liquid thermometers with alarm contacts.

(2) Oil level gage with low-level alarm contact.

(3) Winding temperature relay with three electrically independent sequence contacts. The winding temperature relays are responsive to current in the low-voltage winding and oil temperature.

a. *345-kilovolt power transformers.*—The sequence contacts for the 345-kilovolt power transformers (generating units 1 and 2, 3 and 4, 5 and 6) are set as follows:

First contact (to turn on second heat exchanger oil pump)	55° C.
Second contact (alarm)	80° C.
Third contact (shutdown)	90° C.

b. *230-kilovolt power transformers.*—The sequence contacts for the 230-kilovolt power transformers (generating units 7 and 8) are set as follows:

First contact (to turn on second heat exchanger oil pump)	55° C.
Second contact (alarm)	90° C.
Third contact (shutdown)	100° C.

(4) A winding hot spot temperature detector of the resistance type, 10 ohms at 25° C., together with necessary accessories for remote indication for use with the temperature recorder.

(5) Fault pressure relay.

(6) Pressure relief device with alarm contacts.

(7) Oil inlet valve.

(8) Oil outlet valve.

(9) Oil sampling device.

(10) Upper and lower filter press connections.

(11) Wheels, lifting lugs, and jacking pad for lifting and moving the transformer horizontally.

(12) Two grounding pads on each transformer tank with a terminal connector for one 500,000-circular mil stranded copper cable on each grounding pad.

(13) Diagrammatic nameplate.

(14) Oil drain and water valves.

(15) Oil flow indicator, with alarm contacts, for each pump assembly.

(16) Differential pressure switches which are activated by water pressure differential across each cooler.

(17) Differential pressure switches which are activated by the difference in heat exchanger oil and water pressure.

(18) Pressure-vacuum gage, for indication of gas pressure in the transformer tank, with alarm contacts.

(19) High-pressure gage, for indication of gas cylinder pressure, with alarm contacts.

87. SHUNT REACTORS. The shunt reactor installation was of temporary duration at Glen Canyon Powerplant. The shunt reactor installation was located on the transformer deck of the powerplant and was divided into two groups, with one group located on each side of the 230-kilovolt power transformer installation for units 7 and 8. The reactors were removed from the powerplant after the fifth generating unit was placed in service, and were then placed in service at the Pinnacle Peak Substation. The purpose of their installation at Glen Canyon Powerplant was to provide the reactive kilovolt-amperes required for line voltage correction until adequate reactive capability was available from the generating units.

Each group of reactors was rated at 48,000 kv.-a. and was comprised of two banks of three reactors each. All banks were connected in delta. Suitable bus bar connections were provided to permit removal from service of any one 24,000-kv.-a. bank. A temporary timber enclosure was provided for each reactor group to protect personnel from accidental contact with the reactors, and to protect the reactors. The reactor groups were connected to the units 7 and 8 isolated phase bus structure by means of temporary extensions of the isolated phase bus. The temporary extensions were removed with the reactors. Permanent covers have been furnished for covering the openings in the bus to connect the temporary bus extensions, after removal of the extensions. Each individual reactor was rated as follows:

Kilovolt-amperes	8,000
Rated voltage (nameplate)	13,200 volts
Suitable for operation at	13,860 volts
Insulation class	25,000 volts
Basic impulse level	150,000 volts
Rated continuous current at rated voltage	606 amperes
Cooling class	AA (dry-type, self-cooled)
Temperature rise over an ambient air temperature of 40° C. at 105 percent of rated voltage	80° C.

The reactors were furnished by General Electric Co. under invitation No. DS-5828.

G. MAIN CONTROL AND STATION-SERVICE SYSTEM

88. GENERAL. Glen Canyon Powerplant was designed to be operated as an attended plant, to produce power for the use of the Upper Colorado River Basin States, and to regulate the flow of water to the Lower Colorado River Basin States. The plant was designed on the basis of two operators in attendance, one in the control room and the other in the governor gallery or other location throughout the plant. There is sufficient control, indication, and annunciation equipment on the main control board for one operator in the control room to have adequate supervision over the entire plant.

Automatic load and frequency control of the eight generating units from the Montrose Power Operations Center and breaker position scanning equipment for map board indication at the Montrose Power Operations Center are features of this design. The station-service power is normally supplied from the unit buses. Station-service power can also be supplied from the 25-kilovolt switchyard. In the event a total blackout of the powerplant and switchyard occurs, a generating unit can be started, without the unit auxiliaries operating, to regain station-service power. Normal unit control requires the starting of essential unit auxiliaries. The control system is designed to permit local-manual or local-automatic starting, voltage adjusting, synchronizing, and loading of the units from the main control board. Some provisions have been made for future remote-automatic control from a remote control station. Normal shutdown is ordinarily performed manually or automatically at the control board, and emergency shutdown is manually performed at the control board or automatically by the operation of protective relays. The major electrical equipment to be controlled is shown on the main single-line diagram, figures 197, 198, and 199.

89. MAIN CONTROL ROOM. The control room (fig. 200) is located directly above generating units 1 and 2, 75.74 feet above the generator floor. In this room are located the unit control boards, the generator and unit transformer relay boards, the line relay and control boards, the capacitor-potential device adjusting units, the main alternating- and direct-current distribution boards, carrier and microwave tone equipment boards, load and frequency control equipment racks and unit controller turret, sequential operation recorder equipment boards and automatic printer, and a desk for each of two operators. Space was provided for a dispatching room; however, because the dispatching station was relocated to Montrose Power Operations Center the area is now spare space.

The control and relay boards are of the rigid, self-supporting, sheet-steel, dead-front type with auxiliary swinging panels mounted on the rear. This type of board was selected, as it provides adequate instrument, meter, and relay space on a compact board of simple construction having ready access for maintenance. Control switches, indicating and recording instruments, annunciators, synchronizing equipment, and mimic bus to simulate major electrical connections are located on the front of the line and unit control board panels. Protective relays having targets or other requirements for operating accessibility are semiflush mounted on the front of the relay board panels. All major protective relays are of drawout construction for convenient testing and maintenance. All other control relays and molded-case breakers which supply direct-current control power are located behind panels or on auxiliary swinging panels.

90. GOVERNOR GALLERY. The governor gallery is located directly below the generator floor. In the gallery are located the battery room, the battery chargers, the voltage regulators and excitation control boards, the actuator cabinets, the auxiliary control

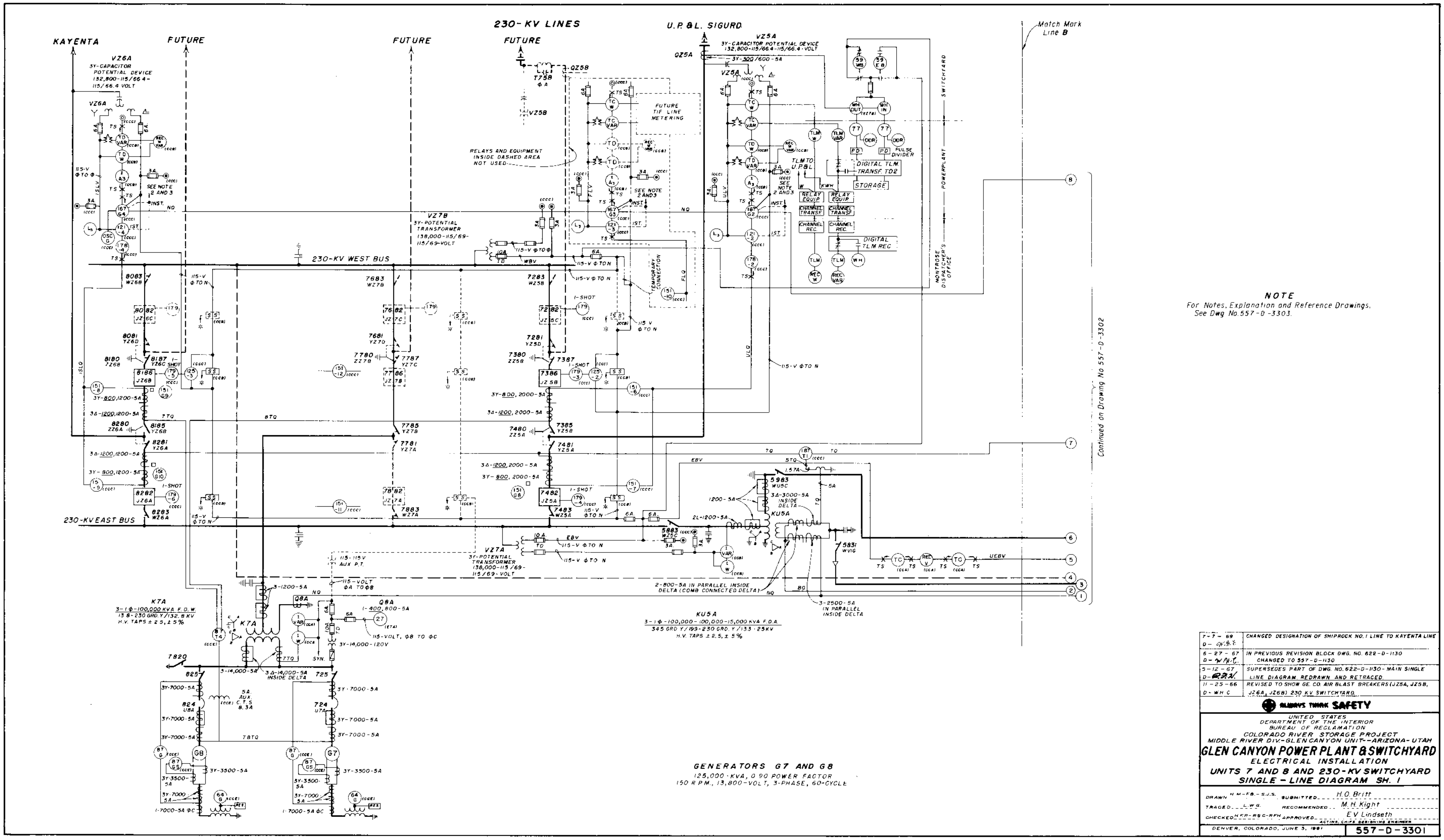


Figure 197.—Powerplant and switchyard electrical installation, single-line diagram—Units 7 and 8 and 230-kilovolt switchyard.

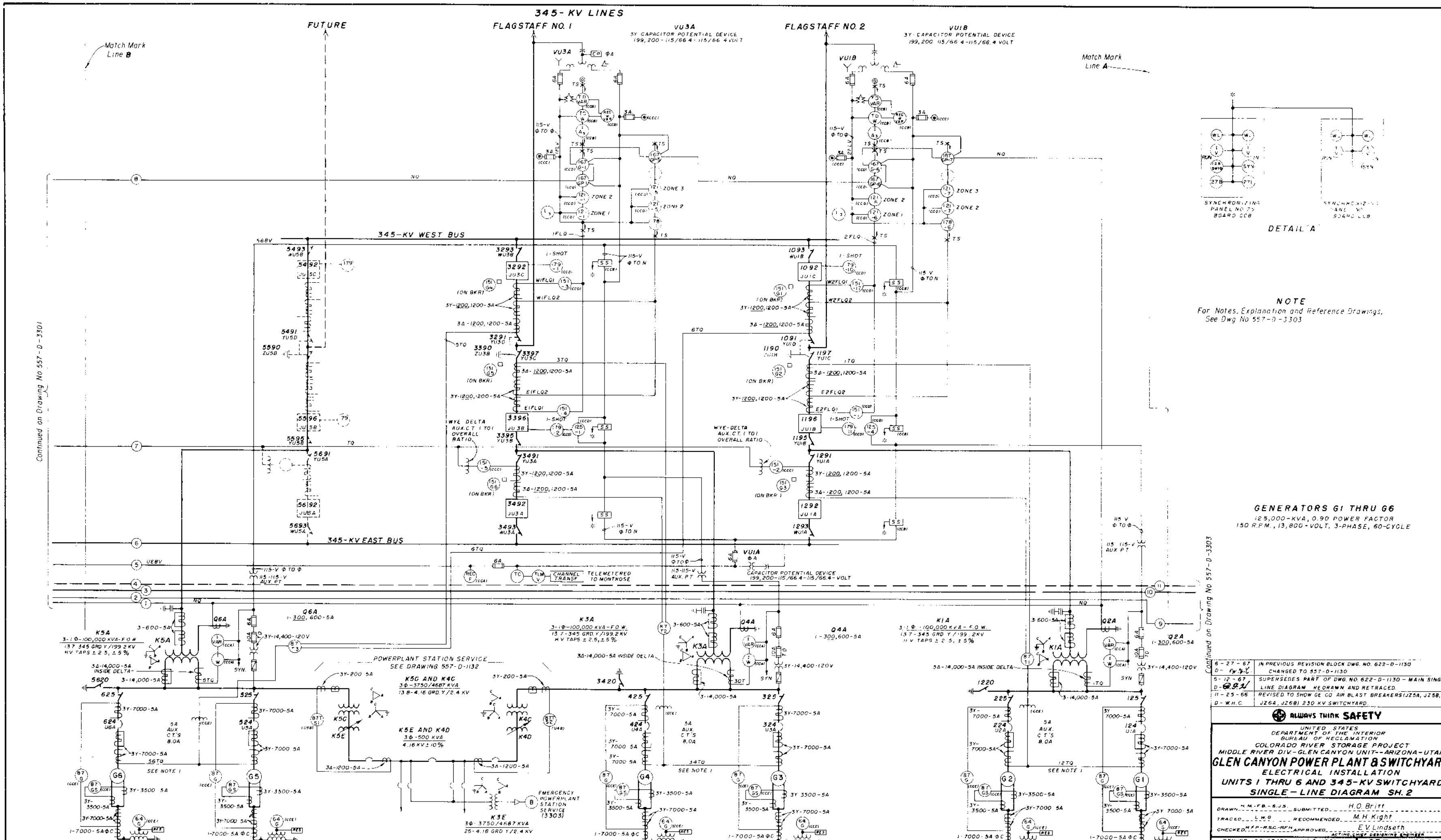


Figure 198.—Powerplant and switchyard electrical installation, single-line diagram—Units 1 through 6 and 345-kilovolt switchyard.

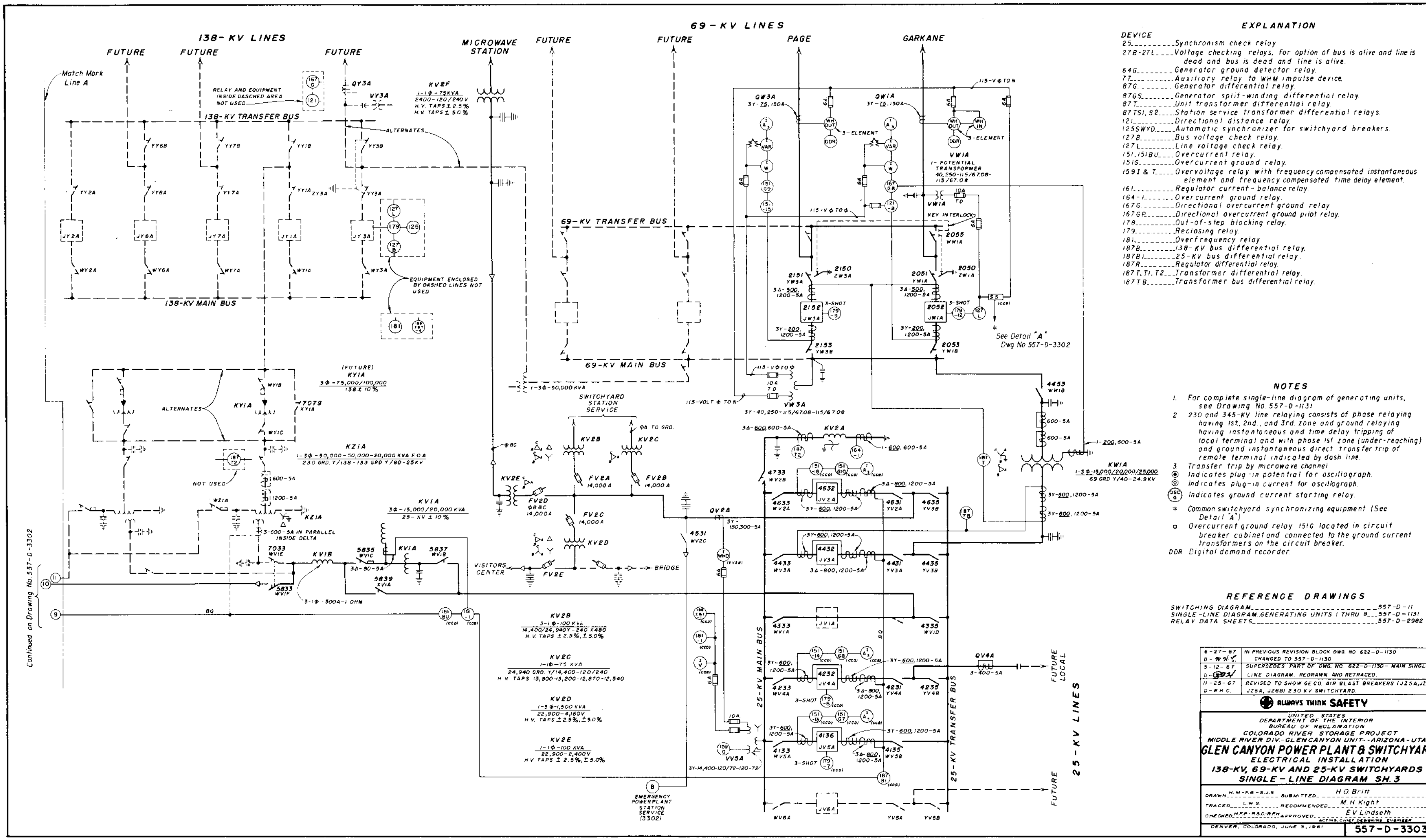


Figure 199.—Powerplant and switchyard electrical installation, single-line installation—138-, 69-, and 25-kilovolt switchyards.

Figure 200.—Interior of control room at Glen Canyon Powerplant. P-557-420-11773, January 8, 1966.

panels, the generator carbon dioxide fire protection systems, and the motor control centers for the auxiliaries power distribution.

91. CONTROL EQUIPMENT. (a) *General.*—The design of the plant and controls is such that the operator can easily operate the eight units and control the switchyard from the main control room. The plant is provided with automatic load and frequency control, breaker position scanning equipment, and a sequential operations recorder. Some provisions have been made for future supervisory control from a remote control station.

(b) *Unit Control Board CCA.*—The control equipment mounted on the front of the unit control board (CCA) is for local-manual and local-automatic control of the powerplant. The following controls and indications have been provided.

(1) Synchronizing lamps, synchroscope, synchronizing switches, voltmeters for incoming and running potentials, and frequency meter.

(2) Control switches and indicating lamps for auxiliaries control, generator brakes, generator breaker, exciter field rheostat, exciter field breaker, water depressing, and station-service breakers.

(3) Control switches for automatic starting and stopping of units, governor speed changer, wicket gate limit control, voltage regulator (manual, automatic), unit operation selector (local-manual, local-automatic, remote-automatic), dashpot bypass, and unit emergency shutdown.

(4) Indicating lamps for unit start-stop indication, unit symbol, unit shutdown and manual lockout, unit transformer symbol, and station-service transformer symbol.

(5) Indicating instruments for unit bus volts and amperes; unit watts and vars; unit speed, speed level and speed droop, voltage regulator volts, and field amperes; and station-service watts, volts, and amperes.

(c) *Line Control Board CCB.*—The control equipment mounted on the front of the line control board (CCB) is for manual control of the switchyard. The following controls and indications have been provided:

(1) Synchronizing lamps, synchroscope, synchronizing switches, and voltmeters for incoming and running potentials.

(2) Control switches and indicating lamps for all power circuit breakers.

(3) Control switches for reclosing cutoff and out-of-step blocking.

(4) Indicating lamps for hot line indication for all lines except the 69-kilovolt Page line.

(5) Indicating instruments for line amperes.

(6) Test switches for microwave transfer trip channels.

(d) *Additional Controls.*—The following additional controls have been furnished:

(1) *Penstock gate control.*—A penstock gate test switch with indicating lamps is provided on the unit control board (CCA) in the control room to test the penstock gate control circuitry. Normal control of the penstock gates is from the penstock gate control cabinet located at the top of the dam.

(2) *Governor controls.*—The governor controls located on the unit control board (CCA) in the control room are duplicated on the actuator panel in the governor gallery by hand-operated control knobs. These dual controls are advantageous since they permit starting the unit at the governor near the unit and near various unit temperature and pressure gages. After starting, the unit can be controlled and synchronized from the main control board.

(3) *25/69-kilovolt transformer (KWIA) bus differential relay cutout switch.*—The transformer bus differential relay cutout switch 187TBCS is located in line relay panel CCD6. This switch has been provided since the differential circuit must be taken out of service whenever the 25-kilovolt bus tie breaker 4432 (JV3A) is substituted for a station-service breaker or a line breaker.

92. PROTECTIVE RELAYING. (a) *General.*—The various protective relays for each line and switchyard transformer are semiflush mounted on the front of the line relay boards CCC and CCD, and for each generator and unit transformer are semiflush mounted on the front of the generator and transformer relay board CCE. The 345- and 230-kilovolt switchyards are provided with a breaker failure scheme to provide backup protection to trip local breakers in the event of primary breaker failure, since remote terminal relaying does not provide adequate protection.

The relaying scheme for the switchyard and transformers is shown on the main single-line diagram (figs. 197, 198, and 199), and for the generator on the unit single-line diagram (fig. 201).

(b) *345-Kilovolt Switchyard.*—Each of the two 345-kilovolt Flagstaff lines are protected by separate dual permissive overreaching transfer-trip schemes. This type of scheme is used to provide overlapping protection for phase-to-ground faults during certain minimum system conditions and to permit independent testing of line relays and associated control circuits while still retaining pilot protection of the line. The following line relaying has been provided for each line:

First set —

Device 121 (zone 1 and zone 2)	Directional distance relays for phase protection
Device 151	Instantaneous overcurrent relays for breaker-failure protection
Device 167G	Directional overcurrent ground relays for line ground protection
Device 167GP	Directional overcurrent ground pilot relays

Second set —

Device 121 (zone 2 and zone 3)	Directional distance relays for phase protection
Device 167GP	Directional controlled overcurrent ground pilot relays for line ground protection
Device 178	Out-of-step blocking relays for power circuit breaker protection

The 345-kilovolt switchyard utilizes a breaker-and-a-half scheme. Single-shot reclosing is provided for each line breaker. The reclosing on the center breaker for each line is interlocked with a synchronism check relay to prevent the breaker from closing when generators 1-2 and 3-4 are out of synchronism with the rest of the plant. It is still possible for generators 5-6 to be connected to the system out of synchronism if one of the two west bus line breakers are out of service when high-speed reclosing is in service. The operating personnel can safeguard against this situation by taking automatic reclosing out of service as necessary.

Space on the line control board CCB has been provided for one future 345-kilovolt line. Space on the line relay board CCD has not been provided for the future 345-kilovolt line.

The 345-kilovolt power circuit breakers have been furnished with ground overcurrent relays (151G) to provide for faults between the circuit breaker and the associated current transformers. This protection is required since the relaying current transformers are all located on one side of the circuit breakers, which causes small unprotected zones on the breakers between the current transformers and the breaker controls.

(c) *230-Kilovolt Switchyard.*—The 230-kilovolt Utah Power and Light Co.-Sigurd line and Kayenta line are protected by underreaching transfer-trip schemes. The scheme on the Kayenta line is of the permissive type using zone 2 and the directional element of the ground relay as the permissive interlock. The following line relaying has been provided for each line:

Device 121	Directional distance relays for phase protection
Device 151	Overcurrent relays for breaker failure protection
Device 167G	Directional overcurrent ground relay for line ground protection
Device 178	Out-of-step blocking relays for power circuit breaker protection

The 230-kilovolt switchyard utilizes a ring bus scheme. Provisions have been made to utilize a future breaker-and-a-half scheme. Single-shot reclosing is provided for each line breaker. The reclosing on the breaker between the lines and west bus is interlocked with a synchronism check relay to prevent the breakers from closing when generators 7-8 are out of synchronism with the rest of the plant.

Space on the line control board CCB and line relay board CCC has been provided for three future 230-kilovolt lines.

(d) *138-Kilovolt Switchyard.*—Space has been provided on the line control board CCB and line relay board CCD for a future 138-kilovolt switchyard.

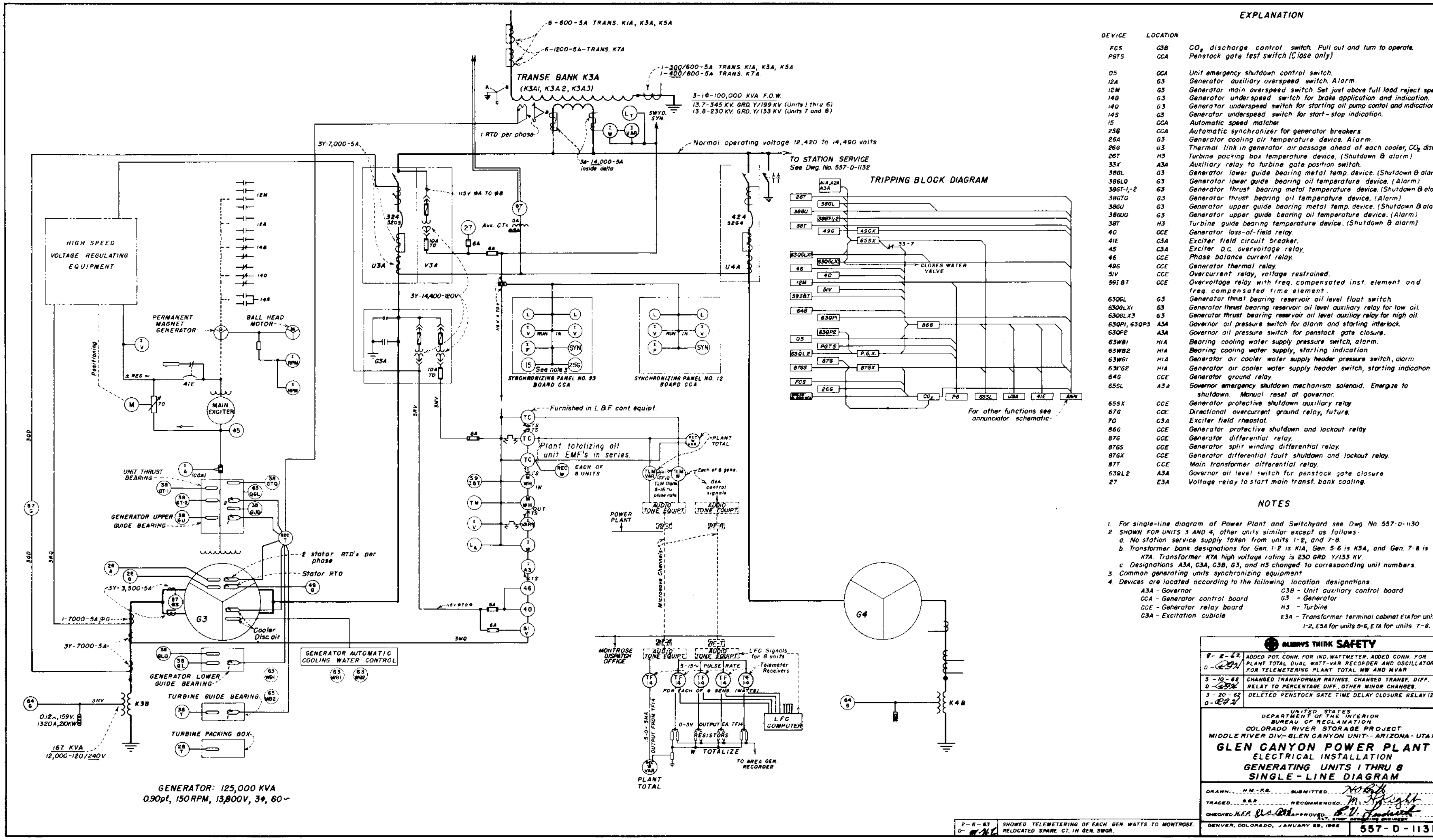


Figure 201.—Powerplant electrical installation single-line diagram—Generating units 1 through 8.

(e) *69-Kilovolt Switchyard.*—The 69-kilovolt Page line is protected by the following line relaying:

Device 151 . . . . Nondirectional overcurrent relays for phase protection
Device 151G . . . . . . Nondirectional overcurrent ground relay for line ground protection

The 69-kilovolt Garkane line is protected by the following line relaying:

Device 121 . . . . Directional distance relays for phase protection
Device 167G . . . . Directional overcurrent ground relay for line ground protection

The 69-kilovolt switchyard utilizes a main and transfer bus scheme. Three-shot reclosing is provided for each line breaker. The reclosing of the Garkane line breaker is interlocked with a deadline check relay to prevent reclosing on a hot line.

Space on the line relay board CCD has not been provided for the two future 69-kilovolt lines.

(f) *25-Kilovolt Switchyard.*—The following line overcurrent relaying has been provided:

Device 151 . . . . Nondirectional overcurrent relays for phase protection
Device 151G . . . . . . Nondirectional overcurrent ground relays for line ground protection

The 25-kilovolt switchyard utilizes a main and transfer bus scheme. Three-shot reclosing is provided for each line breaker.

Space on the line relay board CCD has been provided for a future 25-kilovolt line.

Devices 181 and 159 I&T, overfrequency and instantaneous and time overvoltage relays, respectively, are provided to protect the loads on the 25-kilovolt switchyard, including the 69-kilovolt switchyard, the 25-kilovolt lines, the grounding transformer KV2A, the station-service system, and the microwave station, from damage due to overfrequency or overvoltage resulting from load rejection on the 345- and 230-kilovolt lines. These relays are energized from the 25-kilovolt main bus potential transformers and operate to trip without initiating reclosing the 25-kilovolt breakers. The overvoltage relay 159 I&T is set to operate at a lower voltage than the generator overvoltage relay, device 59 I&T.

(g) *345- and 230-Kilovolt West Buses and Unit Transformers.*—Generator 1-2 unit transformer K1A is protected by a set of percentage-type differential relays. The protected zone includes from the 13.8-kilovolt generator side of the unit breakers 124 and 224, the unit transformer K1A, to the line side of breaker 1196, and to the unit transformer side of breaker 1292. Three winding differential relays are used to minimize tripping for system faults flowing through the switchyard.

Generator 3-4 unit transformer K3A is protected by a set of percentage-type differential relays. The protected zone includes from the 13.8-kilovolt generator side of the unit breakers 324 and 424, the unit transformer K3A, to the line side of breaker 3396, and to the unit transformer side of breaker 3492.

Generator 5-6 unit transformer K5A is protected by a set of percentage-type differential relays. The protected zone includes from the 13.8-kilovolt generator side of the unit breakers 524 and 624, the unit transformer K5A, the 345-kilovolt west bus, to the line side of breakers 1092 and 3292.

Generator 7-8 unit transformer K7A is protected by a set of percentage-type differential relays. The protected zone includes from the 13.8-kilovolt generator side of the unit breakers 724 and 824, the unit transformer K7A, the 230-kilovolt west bus, to the line side of breakers 7386 and 8186.

(h) *345- and 230-Kilovolt East Buses and Main Switchyard Transformer Bank KU5A.*—The main switchyard transformer KU5A is protected by a set of percentage-type differential relays. One zone of protection includes from the unit transformer side of the 345-kilovolt breakers 1292 and 3492, the 345-kilovolt east bus, and from the line side of the 230-kilovolt breakers 7482 and 8282, the 230-kilovolt east bus, to the tertiary winding of the main switchyard transformer KU5A.

(i) *25-Kilovolt Main Bus.*—The 25-kilovolt main bus is protected by a set of percentage-type differential relays and a set of overcurrent relays. The zone of protection includes from the 25-kilovolt line or transfer bus side of breakers 4136, 4232, and 4632, across the regulator KV1A and reactor KV1B, to the tertiary winding of the main switchyard transformer KU5A. Although the regulator KV1A is included in the above protected zone, more adequate protection of the regulator was provided by using a set of current-balance relays. The current-balance relay was selected to provide adequate protection required by the exciter winding of the regulator.

(j) *25-Kilovolt Transfer Bus.*—The 25-kilovolt transfer bus is protected by a set of phase overcurrent relays and an overcurrent ground relay associated with each breaker 4136, 4232, and 4632.

(k) *Zig-Zag Grounding Transformer KV2A.*—The grounding transformer KV2A is protected as follows:

(1) *Against overload.*—One long time delay overcurrent relay in the transformer neutral circuit.

(2) *Against transformer internal faults.*—A set of three short time delay overcurrent relays connected to the delta-connected secondary side of the three current transformers located near the high-voltage bushings to form a single sided differential circuit.

(l) *25-Kilovolt Circuit to Transformers KW1A, 25/69-Kilovolt Transformer KW1A, and 69-Kilovolt Main Bus.*—The 25-kilovolt circuit feeding transformer KW1A is protected by a set of overcurrent relays used in differentially connected current transformers between the main bus side of breaker 4432 to the transformer low-voltage bushings.

(m) *25-Kilovolt Bus to Transformer KW1A, 25/69-Kilovolt Transformer KW1A, and 69-Kilovolt Main Bus.*—The transformer KW1A is protected by percentage differential relaying (187T) which zone includes the low voltage side of transformer KW1A, and the 69-kilovolt main bus, and the line side of 69-kilovolt breakers 2052 and 2152. Another differential zone protected by relay 187TB was included so that the transformer differential could be in service at all times.

(n) *69-Kilovolt Transfer Bus.*—The 69-kilovolt transfer bus protection is normally incorporated in the 69-kilovolt Page line relaying. When the Page line circuit breaker is out of service and the Page line is energized using disconnect switch 2055, the Page line, the Garkane line, and the 69-kilovolt transfer bus are protected by the Garkane line relays.

(o) *Generator Relaying.*—Each generator is provided with the following relaying (fig. 201):

(1) Percentage-type differential current relays for phase-to-phase faults across the generator.

(2) A low-pickup voltage relay connected in parallel with the grounding resistor on the secondary of a distribution-type transformer in the generator neutral for phase-to-ground faults.

(3) Percentage-type differential current relays for phase-to-phase faults and turn faults in the split-windings.

(4) Voltage-restrained overcurrent relays for overcurrent backup protection. The voltage-restrained feature permits setting the relays close to full load current.

(5) Instantaneous and time delay frequency-compensated overvoltage relays for overvoltage protection.

(6) A negative-sequence-type phase current balance relay.

(7) A loss of field excitation relay of the A-C directional impedance type.

(8) Thermal relays for protection against excessively high temperatures.

Operation of either the generator differential relays or the operation of the thermal switches located in the generator air passage ahead of each cooler causes discharge of carbon dioxide gas into the generator housing. Complete shutdown of the unit is effected when carbon dioxide gas is discharged into the generator housing.

Overspeeding of the units due to sudden reduction of load will not cause shutdown of the units unless mechanical difficulties in the governing system permit the speed to go above the full-load rejection overspeed setting. In such an event, an overspeed switch will cause shutdown of the unit by operation of the generator lockout relay.

(p) *Station-Service Transformers.* Each station-service transformer is protected by a set of percentage-type differential overcurrent relays. These transformers are not included in any other differential relay protective zone, but due to the power connections a station-service differential relay operation will also trip out a main unit.

93. METERING AND INDICATION. (a) *General.*—All metering and indicating instruments are mounted on the front of the generator control board CCA and line control board

CCB. The oscillograph is mounted on the front of line relay board CCC. Instruments for the generators, station-service equipment and lines are provided on their respective panels. A mimic bus for the entire plant is mounted on the front of generator control board CCA and one for the switchyard is mounted on the front of line control board CCB. Lamp indications are as described for the control equipment in section 94.

(b) *Generator Control Panels.*—Each generating unit is provided with indicating instruments or meters on the respective generator control panels for indicating generator volts, three-phase amperes, watts, vars, field amperes, regulator volts, speed, turbine gate position and limit, speed droop and speed level. Speed, speed droop, speed changer position and turbine gate position and limit indicators are also available at the governor actuator.

Watt and var meters for the unit tie buses are located on the unit control panel of each respective odd-numbered unit. Watts and vars were provided in lieu of amperes, since potential was readily available.

An elapsed time meter for each generating unit is provided on the respective generating unit recorder panels to indicate running time of the units.

(c) *Station-Service Control Panels.*—Instruments for indicating the three phases of each station-service supply amperes and volts (with transfer switch) are provided on the generator control board CCA. Each station-service supply circuit has been provided with a wattmeter to indicate the station-service load, and each has been provided with an indicating demand watt-hour meter to register the energy consumption.

(d) *Switchyard Lines and Buses.*—The following indicating instruments are located on the front of line control board CCB:

(1) Indicating ammeters to indicate line amperes for each phase located in each respective line control panel.

(2) Indicating wattmeter and varmeter for the main switchyard transformer bank KU5A to indicate the load through the transformer bank.

(3) Indicating ammeters to indicate local and station-service bus amperes for each phase supplied from the 25-kilovolt switchyard.

(4) Indicating voltmeter for 25-kilovolt main bus voltage.

(5) Tap position indicator for regulator KV1A.

(6) Indicating ammeters to indicate the 25-kilovolt transfer breaker amperes for each phase.

(7) Indicating wattmeter and varmeter for each 69-kilovolt line located in the respective line control panel.

(e) *Recording Devices.*—The following recording devices for the powerplant are located on the front of the generator control board CCA:

(1) Generator watts and temperature recorders for each generating unit. The resistance temperature detector circuits for the main switchyard transformer KU5A are included in the generating unit 6 temperature recorder, and the resistance temperature detector circuits for the 25/69-kilovolt transformer KW1A are included in the generating unit 4 temperature recorder.

(2) For each generator, three-element "In" and "Out" watt-hour meters for registering the "in" and "out" energy consumption. Each meter has been provided with a detent to prevent registration on reverse flow of power.

(3) Forebay and afterbay water level recorders.

(4) Total generation recorders for plant total watts and vars, frequency and voltage.

The following recording devices for the switchyard are located on the front of the line control board CCB:

(1) "In" and "Out" digital demand recorders for the 230-kilovolt Utah Power and Light Co.—Sigurd line.

(2) "Out" digital demand recorders for the 69-kilovolt Garkane and Page lines.

(3) One each megawatt and megavar recorder for the 345-kilovolt Flagstaff No. 1 and No. 2 lines and the 230-kilovolt Utah Power and Light Co.—Sigurd line.

94. ANNUNCIATOR. Annunciator groups have been provided for the following:

(1) A 30-point annunciator for each generator and unit transformer with the respective window group located above each generator panel of main control board CCA. A duplicate lamp window group is located in the governor gallery and mounted on the front of the respective generator auxiliary control panel.

(2) One 30-point annunciator group for plant station-service faults, located on the front filler panel above station-service control panel 13 of the generator control board CCA.

(3) One 15-point annunciator group for each of the switchyards. They are located on the front of line control board CCB. The 345-kilovolt switchyard annunciator is located in the filler panel above line control panel CCB 16. The 230-kilovolt switchyard is located above line control panel CCB 13. The 69-kilovolt and future 138-kilovolt switchyards annunciator is located in the filler panel above line control panel CCB 22. The 25-kilovolt switchyard annunciator is located in line control panel CCB 19.

All group and common annunciator relays and associated equipment are located in each annunciator group except for the annunciator groups located in the governor gallery which are lamp groups only and have their lamps connected in parallel with the respective generator and unit transformer annunciators located in the control room.

A white and red indicating lamp is provided with each annunciator group. The white lamp indicates the control power is on, and the red lamp indicates that an annunciator point in that group has lighted.

Closing of a trouble contact will light the red group lamp, light an annunciator window, and sound an alarm. The switchyard annunciators are connected to sound one bell in the switchyard section of the control board, while the generator and unit transformers and station-service annunciators will sound another bell in the generator section of the control board. The generator and unit transformer annunciator also sounds two horns located in the governor gallery and two horns on the generator floor (one at each end). The switchyard annunciator bell can be silenced and the red group lamp reset by pushing the alarm reset pushbutton. The generator and unit transformer annunciator bell and horns can be silenced and the red group lamp reset by pushing the alarm reset pushbuttons located in the control room or the governor gallery. The station-service annunciator uses the generator annunciator bell for alarm. The alarm can be silenced and the red group lamp reset by pushing the alarm reset pushbutton. Release of the alarm reset pushbuttons will not cause the audible alarm to sound or the red group lamp to light; however, there will be an audible alarm and the red group lamp will light when an additional trouble contact closes.

The annunciator windows can be reset by pushing the lamp reset pushbuttons; however, if a trouble contact remains closed the associated annunciator window will remain lighted when the pushbutton is released. Each annunciator can be tested by pushing the test pushbutton. With this test, all annunciator windows of the annunciator group being tested should light. All related pushbuttons are located in the respective panels in which the annunciators are mounted.

The annunciator systems are designed so that there will not be an annunciation under normal operating conditions.

95. GROUNDING SYSTEM. The grounding system consists of a ground mat under the powerhouse foundation connected to a loop under the dam consisting of two 500,000-circular-mil conductors running parallel approximately 160 feet apart and connected to a ground mat in the switchyard. The powerplant and switchyard ground mats are connected by two 500-circular-mil cables running through the control and power cable tunnel located between the powerplant and the switchyard. The grounding system was designed to provide a ground resistance of 1 ohm or lower. Actual powerplant ground resistance as measured in the field was 0.05 ohms. The exposed surfaces of all electrical equipment and practically all exposed metal objects are grounded. The risers (500,000 circular mils in area) are arranged two in parallel in a multiple riser system around the perimeter of the powerplant to be capable of withstanding the maximum sustained short-circuit current for at least 10 seconds. The ground mat at the transformers is designed for an 18,000-ampere line-to-ground fault. Fault currents on the low-voltage side of the main power transformer and generator switchgear which are calculated to be approximately 100,000 amperes are confined within the powerplant and do not enter the ground mat; accordingly the ground mat and risers are not designed for this magnitude of current as the current is confined to the isolated phase bus.

96. LOAD AND FREQUENCY CONTROL EQUIPMENT. The load and frequency control equipment is used to provide remote control of the output of the generators from the Montrose Power Operations Center located south of Montrose, Colo. The desired megawatt output for each unit is held within the setting of the high and low limits as set by the Montrose dispatcher. The limit settings are compared with the actual unit generation. The difference between "desired" and "actual" unit generation develops the megawatt requirement for each unit, or the unit control error. This is then used by a computer located at the Montrose Power Operations Center to develop raise-lower impulses which will bring the actual unit generation into balance with the desired unit generation. The impulses developed by the computer provide a mandatory raise-lower control signal to the generator. The raise-lower impulses are transmitted to the powerplant by Government microwave equipment.

A unit controller turret has been provided at the powerplant. The unit controller turret is located on the operator's desk in the control room and provides manual "On-Off" load and frequency control of each unit. Remote control of the unit controller turret from the Montrose Power Operations Center has not been provided. The unit controller turret has the following control and indication features:

(1) Lighted pushbuttons for "On-Off" control of each unit. The "On" pushbutton lights red when the unit is on load and frequency control and the "Off" pushbutton lights green when the unit is off load and frequency control. In addition "On-Off" control indication for each unit has been provided at the Montrose Power Operations Center.

(2) Microwave signal failure white lamps indication for each unit.

(3) A short time duration audible alarm. Unit load and frequency control for the individual units will be automatically tripped off and an audible alarm sounded for the following:

A long-time pulse duration for raise or lower impulses to prevent misoperation of the generator due to faulty equipment or during extended periods of high noise level on the microwave channel
Microwave channel signal failure
Loss of direct-current control power
Abnormally high plant total frequency
Unit shut down to speed-no-load
Emergency and normal unit shut down

Whenever any unit is taken off load and frequency control, the unit must be manually returned to load and frequency control at the powerplant.

97. SEQUENTIAL OPERATIONS RECORDER. A 400-point sequential operations recorder has been provided to minimize the logging by the operator. This equipment logs, in the proper sequence, the opening and closing of the generator switchyard, and station-service annunciator trouble contacts, and also the closing and tripping of the generator and switchyard power circuit breakers. The date and time at which any of the above events occur are also logged. The sequential operations recorder equipment is located in the control room and consists of an equipment board designated CCF, and a printer. The equipment board contains the scanner and input gate section, the timing section, the sequential memory, and the decoder and printer drive section.

H. AUXILIARY ELECTRICAL EQUIPMENT AND INSTALLATION

98. ALTERNATING-CURRENT STATION-SERVICE SUPPLY SYSTEM. (a) *General.*—The alternating-current station-service supply system is supplied from two 3-phase, 3,750/4,687-kv.-a., 13,800 to 4,160-volt transformers. One transformer is connected to generators 3-4 common bus and the other transformer is connected to generators 5-6 common bus. A third emergency supply from the 25-kilovolt switchyard is available through a 3-phase, 3,750/4,687-kv.-a., 25,000 to 4,160-volt transformer. The two supplies from the two generator buses are normally used to carry the station-service loads with the switchyard feeder acting as a standby. Any one of these supplies will carry the entire station-service load, except that under some conditions nonessential loads may have to be curtailed (heating, lighting, machine shop, etc.). The 4,160-volt induction-type voltage regulators have plus and minus 10 percent voltage regulation capability and are used to regulate the generator supply sources to the station-service system. The entire station-service system is regulated, since the generator voltage varies approximately 20 percent from light to heavy load.

The station-service supply system consists of a 4,160-volt, double-ended intermediate voltage switchgear; three 460-volt, double-ended secondary unit substations; five 208Y/120-volt single-ended lighting unit substations; and various quantities of power boards and power distribution panelboards.

The 460-volt, double-ended secondary unit substations are supplied by two 3-phase, 750-kv.-a., 4,160 to 480-volt transformers located at each end.

Figures 202 through 205 show single-line diagrams for the 4,160-volt switchgear and the 460-volt distribution system.

(b) *4,160-Volt Intermediate Voltage Switchgear.*—The 4,160-volt intermediate voltage switchgear consists of a double-bus, double-ended unit substation with a supply breaker on each end, a bus-tie breaker, and a group of 4,160-volt feeder breakers. Since the unit substation is normally supplied from two different end sources, it could be possible to have unsynchronized sources connected to this switchgear. To avoid this possibility, the control to the two supply breakers and the tie-breaker is so arranged that only two of the three breakers can be closed simultaneously under manual or automatic mode of operation.

Each bus is protected by individual sets of bus differential, undervoltage, overcurrent, and overfrequency relays. Each feeder breaker is protected by individual sets of phase overcurrent and overcurrent ground relays.

Under normal operating conditions, the station-service load is divided and connected to each of the two end supplies. In the event that one end supply breaker trips by operation of the undervoltage or overfrequency relays, the tie breaker will close automatically and sound an alarm, provided the mode selector switches are in the "Automatic" position. The transfer is blocked when a supply breaker is tripped by its bus differential relay. When this transfer is made, the entire station-service load is connected to one end. If the transformer feeding this end is overloaded, an inverse time overcurrent relay will trip the end supply breaker. To restore station service due to this overloading, the 25-kilovolt switchyard supply can be used or some of the heating load can be dropped to reduce the overload condition. Restoration of this switchgear to normal operation is accomplished manually. When an end supply breaker and the bus-tie breaker are closed, the other end supply breaker can be closed manually through a synchronism check relay. Before this end supply breaker is closed, the bus-tie breaker will be tripped to prevent temporary paralleling of the two end supplies. The bus-tie breaker cannot be closed manually when both end supply breakers are closed.

The 25-kilovolt switchyard supply can be manually connected to one bus or the other bus through feeder breakers. The control of these feeder breakers is arranged so that only one breaker is closed at any one time. These breakers are interlocked with the bus differential relays and will be tripped on a bus differential and cannot be closed on the faulted bus. Since this operation is manual, the feeder breakers are not interlocked with the end supply or bus-tie breakers. The 25-kilovolt switchyard supply could be connected to a hot bus out of synchronism. Connection of this supply to the switchgear is left to the discretion of the operator.

(c) *460-Volt Secondary Unit Substations No. 1 and 2.*—The 460-volt secondary unit substations No. 1 and 2 consist of a double-bus, double-ended unit substation with a supply breaker on each end, a bus-tie breaker, and a group of 460-volt feeder breakers. Since the unit substation is normally supplied from two different end sources, it could be possible to have unsynchronized sources connected to this switchgear. To avoid this possibility, the control to the two supply breakers and the bus-tie breaker is so arranged that only two of the three breakers can be closed simultaneously under manual or automatic mode of operation.

Each bus is protected by individual undervoltage relays. The feeder lines are protected by the circuit breakers having static-type trip devices. The feeder breakers which feed the alternating-current unit auxiliary powerboards (D1A through D8A) are part of a selective trip feeder breaker control system to provide coordination with the feeder breakers on the auxiliary powerboards, so that a fault on the load side of the powerboard feeder breaker will trip the powerboard feeder breaker and not trip the respective secondary unit substation feeder breaker. The instantaneous elements on these breakers to the unit auxiliary powerboards have elements which can be set between 5 and 40 times the pickup setting, whereas the regular dual static overcurrent trip devices only have a range of 5 to 15 times that setting. The time delay characteristics of the selective static trip devices also reaches out to the 40 times. By setting the instantaneous elements higher, more time delay characteristics are used. At high multiples of the pickup current, the time delay trip time becomes faster. This feature is used to provide the selective trip characteristics.

Under normal automatic operating conditions, the secondary unit substation load is divided and connected to each of two end supplies. In the event that one end supply breaker trips by operation of the undervoltage relays, the end supply breaker will trip

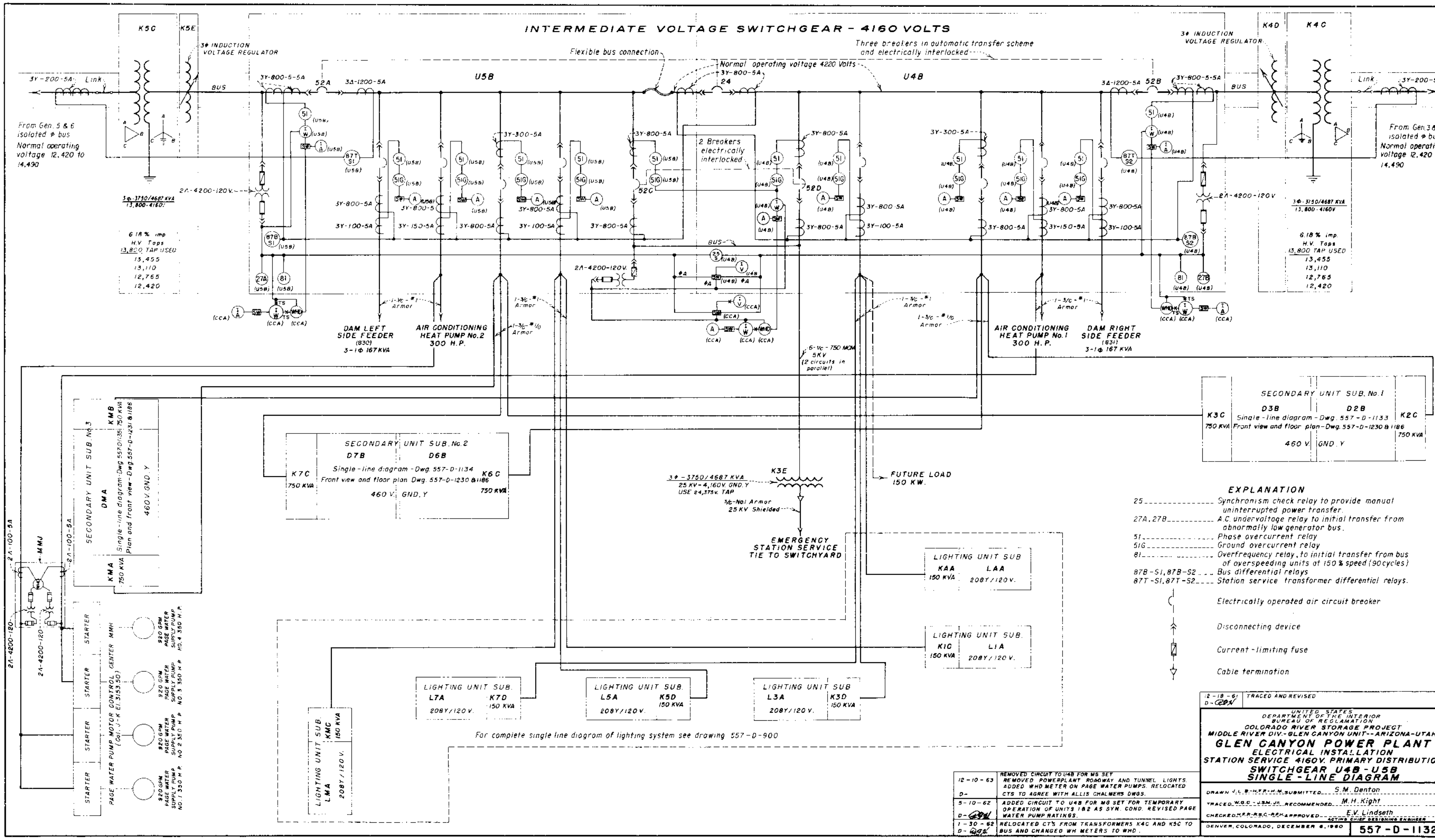


Figure 202.—Powerplant station-service 4,160-volt primary distribution single-line diagram—Switchgear U4B and U5B.

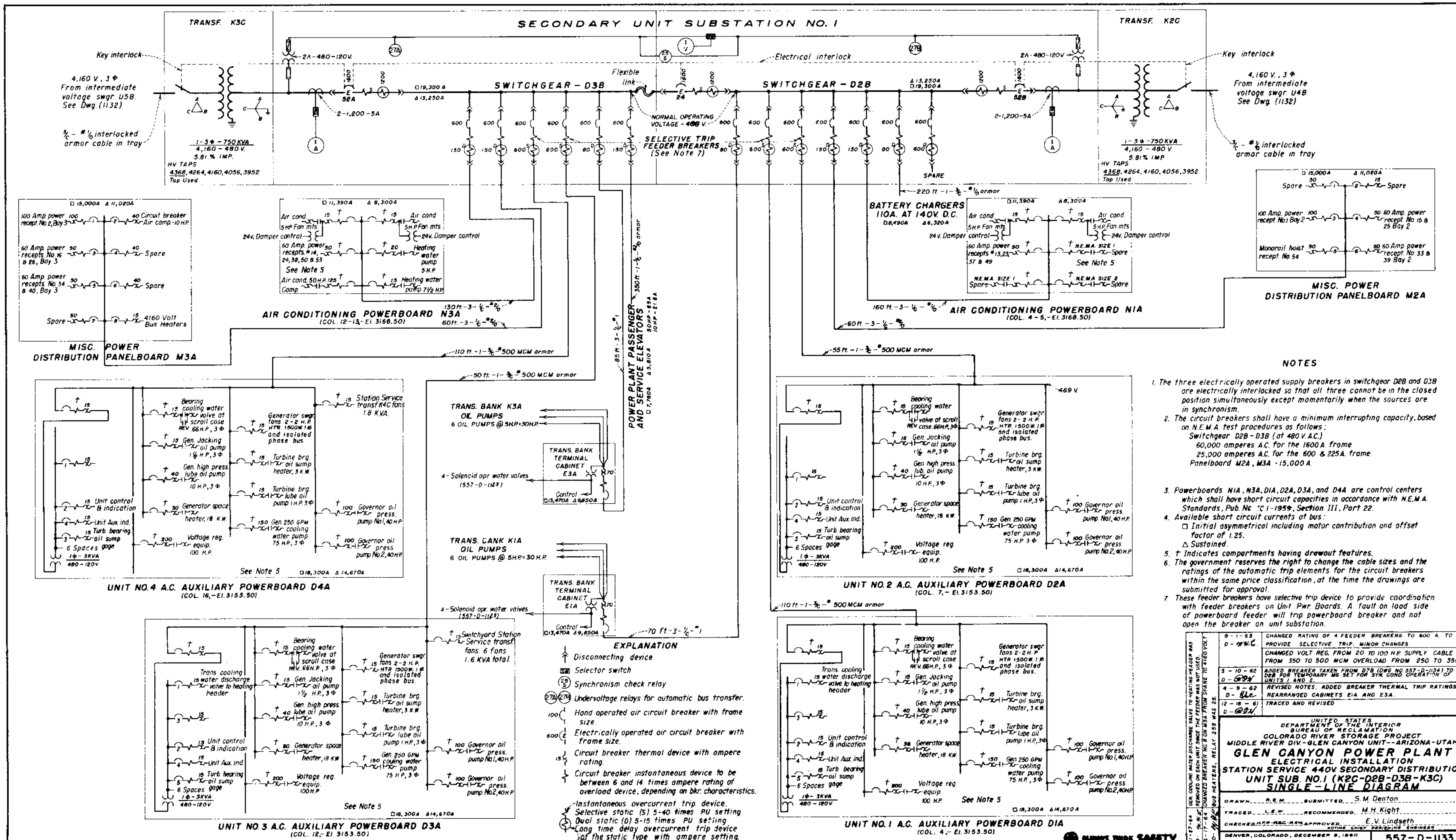


Figure 203.—Powerplant station-service 440-volt secondary distribution, unit substation No. 1—Single-line diagram.

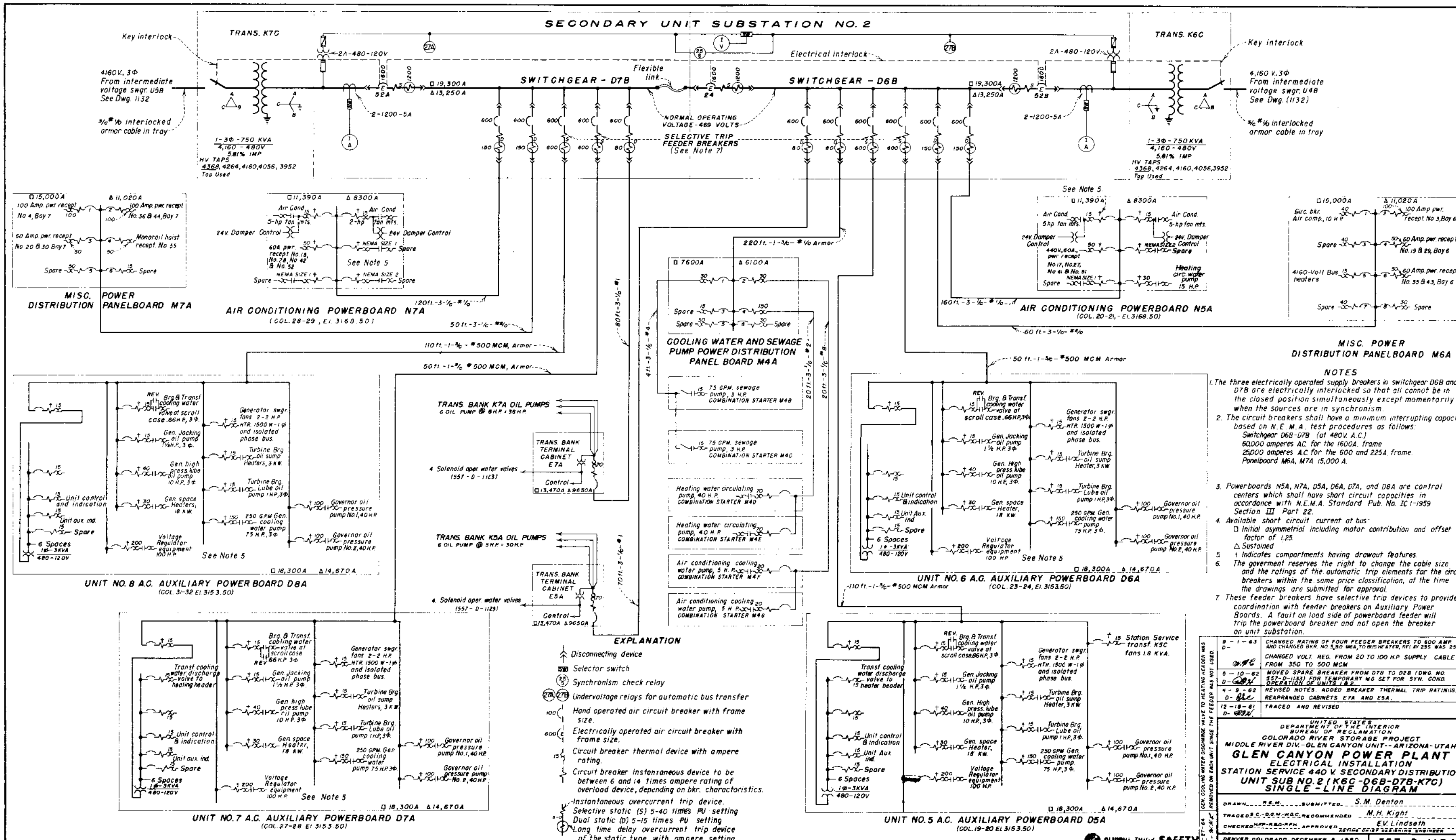


Figure 204.—Powerplant station-service 440-volt secondary distribution, unit substation No. 2—Single-line diagram.

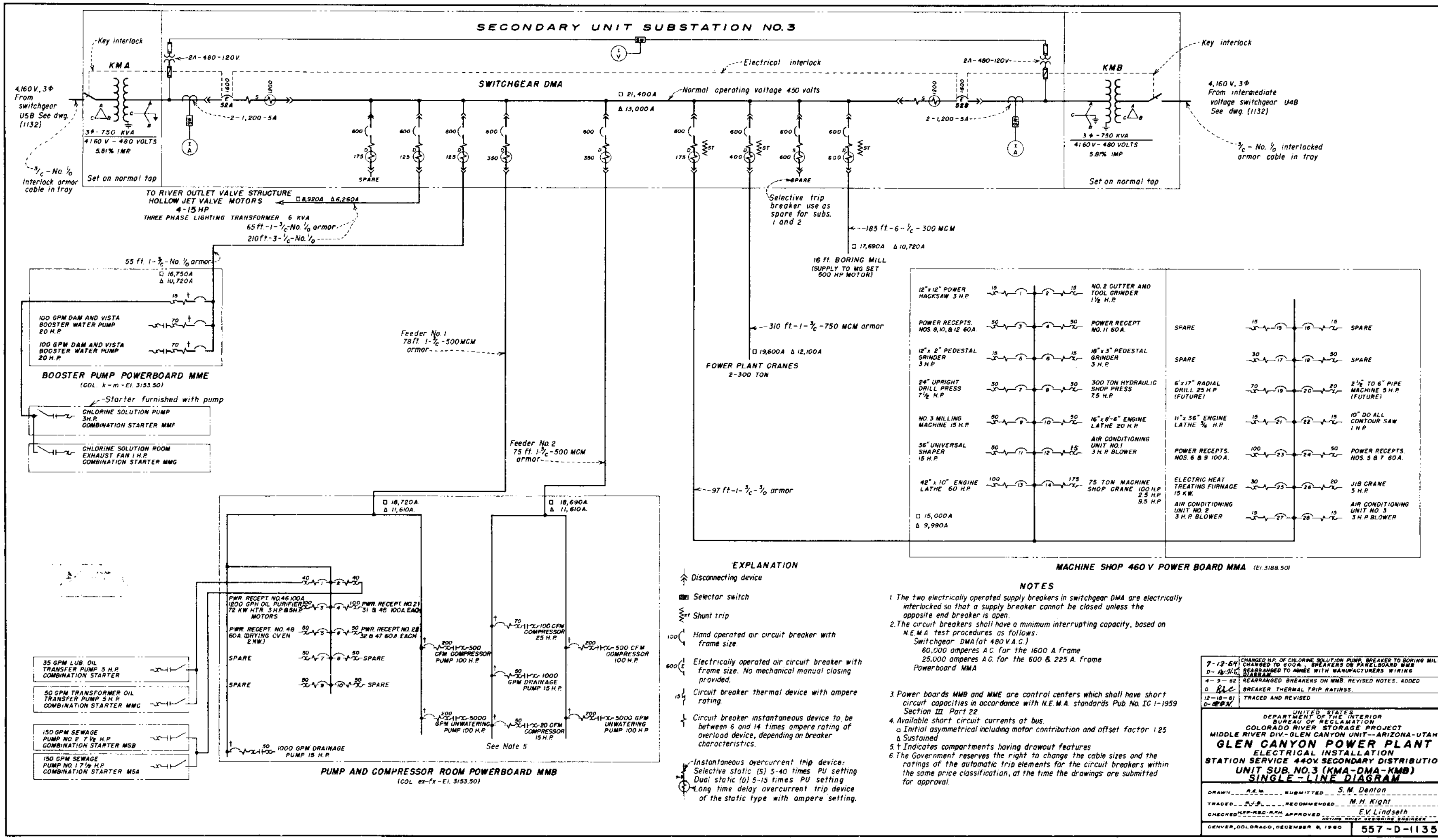


Figure 205.—Powerplant station-service 440-volt secondary distribution, unit substation No. 3—Single-line diagram.

and the bus-tie breaker will close and sound an alarm after a time delay. The time delay is necessary to allow the 4,160-volt intermediate voltage switchgear to transfer. The trip circuits on the end supply breakers are interconnected so that one end supply breaker will not trip unless the voltage to the other end supply breaker is normal. Restoration of these substations to normal operation is accomplished manually. When an end supply breaker and the bus-tie breaker are closed, the other end supply breaker can be closed manually through a synchronism check relay. Before this end supply breaker is closed, the bus-tie breaker will be tripped to prevent temporary paralleling of the two end supplies. The bus-tie breaker cannot be closed manually when both end supply breakers are closed.

(d) *460-Volt Secondary Unit Substation No. 3.*—The 460-volt secondary unit substation No. 3 consists of a single-bus, double-ended unit substation with a supply breaker on each end and a group of feeder breakers. The end supply breakers are electrically interlocked so that only one breaker can be closed at any one time. Automatic transfer has not been provided.

(e) *460-Volt Distribution System.*—The following alternating-current powerboards and power distribution panelboards have been provided throughout the powerplant.

(1) Unit auxiliary powerboards (D1A through D8A).

(2) Air-conditioning powerboards (N1A, N3A, N5A, and N7A).

(3) Miscellaneous power distribution panelboards (M2A, M3A, M6A, and M7A).

(4) Transfer bank terminal cabinets (E1A, E3A, E5A, and E7A).

(5) Cooling water and sewage pump power distribution panel board (M4A).

(6) Machine shop powerboard (MMA).

(7) Pump and compressor room powerboard (MMB).

(8) Booster pump powerboard (MME).

All of the above boards are located near the center of their loads to keep cable runs short and to be convenient for control purposes.

(f) *Battery Chargers.*—Two silicon-controlled rectifier-type battery chargers have been furnished and are located in room 308 at elevation 3153.50. Each charger is rated 110 amperes over its 125- to 140-volt range. Normally, one charger supplies the continuous direct-current load, and the station battery floats on the direct-current bus to supply momentary loads such as closing or simultaneous tripping of circuit breakers. The chargers can be arranged for parallel operation to supply the direct-current load and charge the battery at the maximum permissible charging rate when the battery is low. The chargers are also to be used for equalize charging of the battery. All charger controls are located on the chargers.

99. 125-VOLT DIRECT-CURRENT CONTROL AND DISTRIBUTION SYSTEMS. Battery charger control boards B1E and B1F have been furnished for direct-current distribution purposes. The following features have been included on the boards:

(1) Two circuit breakers and one disconnect switch on each board.

(2) System ground detector relay in board B1E.

(3) Two direct-current system ammeters in board B1F.

(4) One direct-current system voltmeter (with switch) on board B1F.

(5) Two direct-current system undervoltage relays on board B1F.

(6) Two direct-current time delay relays on board B1F.

The direct-current power is used for all important powerplant and switchyard control circuits and is distributed from various locations: the main distribution board B1G located near the battery chargers; the control area distribution board BCA located in line relay board CCC in the control room; the direct-current circuit breakers located on the back of the various generator and line control and relay panels in the control room; and the unit distribution boards B1A through B8A located in the governor gallery. Any direct-current circuit breaker supplying important circuits which do not have other means of annunciating direct-current power failure has been provided with undervoltage relays for alarm and annunciation.

I. MISCELLANEOUS CRANES AND HOISTS

100. POWERPLANT CRANES. (a) *General Description.*—Two 300-ton-capacity overhead traveling cranes are installed in the powerplant for installing and maintaining the turbines, generators, and other powerplant equipment. The cranes were manufactured according to the requirements of invitation No. DS-5260 and are indoor, cab-operated, overhead electric traveling cranes with twin trolleys each supporting a 150-ton main hoist and a 30-ton auxiliary hoist. The lifting capacity of the four main hoists may be combined through two equalizing beams and a lifting beam to give a total lifting capacity of 600 tons.

The main hoist blocks are suspended from 12 parts of 1-3/8-inch-diameter, 6 by 37, improved-plow-steel, fiber-core wire rope reeved 6 parts double. Each block is equipped with a sister-type hook bored for a horizontal lifting pin, which provides for easy attachment of the lifting beams and other lifting devices. Each hoist has a total lift of 80 feet, is powered by a 40-horsepower, 1,150-r.p.m., direct-current motor and operates at varying speeds up to 3.63 feet per minute hoisting and 5.80 feet per minute lowering with a 150-ton load.

Each auxiliary hoist block is suspended from four parts of l-inch-diameter, 6 by 37, improved-plow-steel, fiber-core wire rope reeved two parts double. Each block is equipped with a standard single hook. Each hoist has a total lift of 100 feet, is powered by a 40-horsepower, 1,150-r.p.m., direct-current motor and operates at varying speeds up to 19.1 feet per minute hoisting and 23.0 feet per minute lowering with a 30-ton load.

Individual motor-generator sets for each trolley are installed on the bridge of the cranes to supply direct-current power for both hoist motors on the trolleys. The hoist motor controls are electrically interlocked so that only one hoist on each trolley may be energized at a given time. Five-point speed control hoisting and lowering is obtained by varying the voltage output of the generator. Reversal of direction is accomplished by reversing the generator polarity. Each hoist is equipped with two direct-current magnetic brakes which are released when the hoist motor circuit is energized. The magnetic holding brakes are each capable of holding one and one-half times the rated load of the hoists. Block-actuated limit switches are installed on each hoist to limit the upper travel of the hook.

Each trolley is driven by a 5-horsepower, 1,100-r.p.m., wound-rotor induction motor at varying speeds up to 35.3 feet per minute. The motor is equipped with a motor-mounted, disk-type brake which sets when the trolley master switch is moved to the off position. A drift point is provided on the first point of the master switch to hold the brake open without energizing the motor.

Each bridge structure consists of two box girders supported by eight two-wheel trucks over a span of 71 feet 6 inches. Each bridge is driven by two 15-horsepower, 1,100-r.p.m., wound-rotor induction motors at varying speeds up to 110 feet per minute. Each bridge drive is equipped with a hydraulic brake which is foot operated from the operator's cab. Spring bumpers fixed to the bridge and trolley prevent damage to the cranes in case of collision with the runway or bridge girder stops. Steel angle conductors located below one of the main runway rails supply 3-phase, 460-volt, 60-cycle power to the cranes.

Full magnetic controls for each motion of the crane are located in the operator's cab on each crane. Each hoist has five speed control points for both lowering and raising; the bridges have five speed control points for each direction of travel; and the trolleys have four speed control points and a drift point for each direction of travel. The hoist controls are designed to limit the vertical movement of the main hooks to within 1/32 inch and the auxiliary hooks to within 1/16 inch with full rated load and when starting from a complete standstill. The trolley and bridge controls are designed to limit movement to 1/4 inch with full rated load and when starting from a complete standstill. Jogging buttons to bypass the drift point are provided on the trolley controls to aid in "inching" the trolley.

The cranes are equipped with a 115-volt permanent lighting system. Four 500-watt high-bay lighting units are installed on each crane to illuminate the working area under the crane. A 100-watt lighting unit and a convenience outlet are installed in the operator's cab and one convenience outlet is installed on each bridge walkway on each crane.

All ladders, platforms, and walkways necessary for access to the cabs, bridge drive machinery, trolleys, and other parts requiring attention and lubrication are provided on the cranes.

(b) *Design.*—The cranes were designed according to the Navy Department's standards of design for structural steel using a base stress of 14,000 pounds per square inch. Computation of stresses took into account all dead and live loads due to impact, acceleration, and deceleration. Allowable stresses due to combinations of these stresses were increased according to the Navy standard noted. A factor of safety of not less than five, based on the ultimate strength of the material and rated capacity of the cranes, was used in the design of all mechanical parts.

101. 75-TON MACHINE SHOP CRANE. (a) *General Description.*—A 75-ton-capacity overhead traveling crane is installed in the machine shop to handle materials and equipment. The crane was manufactured according to the requirements of invitation No. DS-5252 and is an indoor, cab-operated, overhead electric traveling crane, with a single trolley supporting a 75-ton main hoist and a 15-ton auxiliary hoist. The main hoist block is suspended from eight parts of 1-1/4-inch-diameter, 6 by 37, improved-plow-steel, fiber-core wire rope reeved four parts double. The block is equipped with a sister-type hook bored for a horizontal lifting pin, which provides for easy attachment of lifting devices. The hoist has a total lift of 40 feet, is powered by a 68.5-horsepower, 1,000-r.p.m., direct-current motor and operates at varying speeds up to 6.6 feet per minute with a 75-ton load.

The auxiliary hoist block is suspended from four parts of 0.785-inch-diameter, 6 by 37, improved-plow-steel, fiber-core wire rope reeved two parts double. The block is equipped with a standard single hook. The hoist has a total lift of 55 feet, is powered by a 58.5-horsepower, 1,000 r.p.m., direct-current motor and operates at varying speeds up to 36 feet per minute with a 15-ton load.

A single motor-generator set installed on the bridge of the crane supplies direct-current power for both hoist motors. The hoist motor controls are electrically interlocked so that only one hoist may be energized at a given time. Five-point speed control hoisting and lowering is obtained by varying the voltage output of the generator. Reversal of direction is accomplished by reversing the generator polarity. Each hoist is equipped with two direct-current magnetic brakes which are released when the hoist motor circuit is energized. These magnetic holding brakes are each capable of holding one and one-half times the rated load of the hoists. Block-actuated limit switches are installed on each hoist to limit the upper travel of the hook.

The trolley is driven by a 1.5-horsepower, 1,200-r.p.m., wound-rotor induction motor at varying speeds up to 30 feet per minute. The motor is equipped with a motor-mounted, disk-type brake which sets when the trolley master switch is moved to the off position. A drift point is provided on the first point of the master switch to hold the brake open without energizing the motor.

The bridge structure consists of two box girders supported by four two-wheel trucks over a span of 67 feet 6 inches. The bridge is driven by a 3-horsepower, 1,200-r.p.m., wound-rotor induction motor at varying speeds up to 85 feet per minute. The bridge motor is equipped with a hydraulic brake which is foot operated from the operator's cab. Spring bumpers fixed to the bridge and trolley prevent damage to the crane in case of collision with the runway or bridge girder stops. Steel angle conductors located below one of the main runway rails supply 3-phase, 460-volt, 60-cycle power to the crane.

Full magnetic controllers for each motion of the crane are located in the operator's cab. Each hoist has five speed control points for both lowering and raising; the bridge controller has five speed points for each direction of travel; and the trolley has four speed points and a drift point for each direction of travel. The hoist controls are designed to limit the vertical movement of the main hook to within 1/32 inch and the auxiliary hook to within 1/16 inch with full-rated load and when starting from a complete standstill. The trolley and bridge controls are designed to limit movement to 1/4 inch with full-rated load and when starting from a complete standstill. A jogging button to bypass the drift point is provided on the trolley master switch to aid in "inching" the trolley.

The crane is equipped with a 115-volt permanent lighting system. Two 500-watt high-bay lighting units are installed on the crane to illuminate the working area under the crane. A 100-watt lighting unit and a convenience outlet are installed in the operator's cab.

All ladders, platforms, and walkways necessary for access to the cab, bridge drive machinery, trolley, and other parts requiring attention and lubrication are provided on the crane.

(b) *Design.*—The crane was designed according to the Navy Department's standards of design for structural steel using a base stress of 14,000 pounds per square inch. Computation of stresses took into account all dead and live loads due to impact, acceleration, and deceleration. Allowable stresses due to combinations of these stresses were increased according to the Navy standard noted. A factor of safety of not less than five, based on the ultimate strength of the material and rated capacity of the crane, was used in the design of all mechanical parts.

102. 10-TON GANTRY CRANE. (a) *General Description.*—The 10-ton gantry crane is mounted on the powerplant transformer deck and is used for operation and maintenance of the turbine draft tube bulkhead gates. The 10-ton crane, which was manufactured under invitation No. DS-5398, is an outdoor, pushbutton-controlled, traveling-gantry type with base-mounted hoist. It travels on four single-flanged wheels with a 14-foot wheelbase, supported by 115-pound A.R.E.A. rails on 9-foot centers.

The hoist is reeved one part single with 1-1/8-inch-diameter, 6 by 37, improved-plow-steel, fiber-core wire rope, has a lift of 80 feet at a speed of 14 feet per minute hoisting or lowering with a 10-ton load and is powered by a 12-1/2-horsepower, 1,800-r.p.m., 440-volt, 3-phase, 60-cycle, totally enclosed, squirrel-cage motor.

The hoist motor is equipped with an automatic electric disk-type brake equally effective in both directions of movement and capable of overcoming the full-load torque of the motor. The automatic electric brake is equipped with a manual release which automatically resets when the motor is energized. A block-actuated limit switch is provided to limit upward motion of the hoist.

A heavy pattern, galvanized, open-end socket with a 2-1/2-inch-diameter pin and a connecting link is attached to the free end of the hoisting rope.

The gantry is driven by a 1-horsepower, 1,800-r.p.m. input, right-angle, worm motor-reducer using 440-volt, 3-phase, 60-cycle power and driving one wheel on each rail. Gantry travel is approximately 535 feet and the speed of the gantry in either direction is 35 feet per minute.

Pushbutton stations for hoist and gantry travel are located in a control box located inside the front of the unit. Control power is 120 volts obtained from a 440/120-volt transformer.

Power is supplied to the crane by a 75-foot-long flexible cable which connects a receptacle on the back of the crane with power outlets located at intervals along the downstream side of the deck. Anchor chains with snap fasteners are installed on each end of the power cable to prevent tension being taken by the plugs and receptacles. A lockable cabinet is provided for storage of the power cable.

Provision is made for storage of the lifting frame on the front of the crane, and handrailing is provided along the front of the deck between the gantry columns.

(b) *Design.*—The crane was designed according to the Navy Department's standard of design for structural steel using a base stress of 16,000 pounds per square inch. Computations of stress took into account windloads, all dead and live loads, and loads due to acceleration and deceleration. A factor of safety of not less than five, based on the ultimate strength of the material and rated capacity of the crane, was used in the design of all mechanical parts.

103. 75-TON TRANSFER CAR. (a) *General Description.*—The 75-ton transfer car is provided to transport turbine runners and other equipment between the powerplant and the machine shop. The car was manufactured under invitation No. (D) 90,582-A.

The transfer car has four single-flanged 16-inch-diameter wheels to fit 115-pound A.R.E.A. rails on 5-foot 6-inch centers. Each wheel is pressed on an axle which supports the transfer car through two roller-bearing pillow blocks mounted on the underside of the frame.

The welded structural steel frame supports a 7-foot-wide by 14-foot-long plate steel deck which is welded to the frame. There is a towing connection at each end of the frame, and two lifting eyes are on each side of the frame.

(b) *Design.*—The transfer car was designed using a base tensile stress of 16,000 pounds per square inch of net section, and a factor of safety of at least four with calculated stresses increased 10 percent to allow for shock.

CHAPTER IX. *Design*—SWITCHYARD

A. STRUCTURAL

104. GENERAL. The switchyard is located approximately 850 feet southwest of the right abutment of the dam, and approximately 170 feet higher than the top of the dam. The fill for embankment ranges up to 27 feet in depth and the cut for excavation ranges up to 24 feet in depth.

Since the embankment slopes, as constructed, consisted primarily of fine sand, a 12-inch layer of pit-run gravel was placed on the slopes for protection from wind and water erosion. As an additional protection against water erosion, a curb, gutter, and drain system was later constructed.

105. CONCRETE FOUNDATIONS. All pad and stem type foundations were designed for a gross toe pressure not to exceed 3,000 pounds per square foot and a factor of safety against uplift and overturning of not less than 1.5.

Rock-type foundations were extended a minimum 12 inches into rock to resist shear forces. Anchor bars were designed for a maximum tensile stress of 16,000 pounds per square inch and were embedded to a depth so that the rock shear stress does not exceed 300 pounds per square foot.

106. CONCRETE FOOTINGS FOR APPROACH TOWERS. The footings for towers C1-T2, C2-T2, C3-T2, and C4-T2 were designed as rock-type footings. To resist the shear forces, the footings were extended a minimum of 12 inches into rock. Anchor bars were designed for a maximum tensile stress of 16,000 pounds per square inch and were embedded to a depth so that the rock shear stress does not exceed 300 pounds per square foot.

The footings for rim towers C1-T1, C2-T1, C3-T1, and C4-T1 were designed as rock-type footings. To resist shear forces, the footings were extended a minimum of 24 inches into rock.

Anchor bars were designed for a maximum tensile stress of 16,000 pounds per square inch and were embedded a minimum of 25 feet into rock to attain maximum strength in the bedrock by spanning the bedding planes and stress relief joints which are present near the edge of the canyon rim.

107. CONTROL CABLE TUNNEL. The control cable tunnel extends approximately 1,280 feet from a lower portal near the powerplant to an entry structure in the switchyard. The control tunnel has concrete walls and a pneumatically applied mortar roof, except for approximately 100 feet at the portal end which has reinforced concrete walls and roof. To relieve any phreatic pressure, the floor of the tunnel was left unpaved.

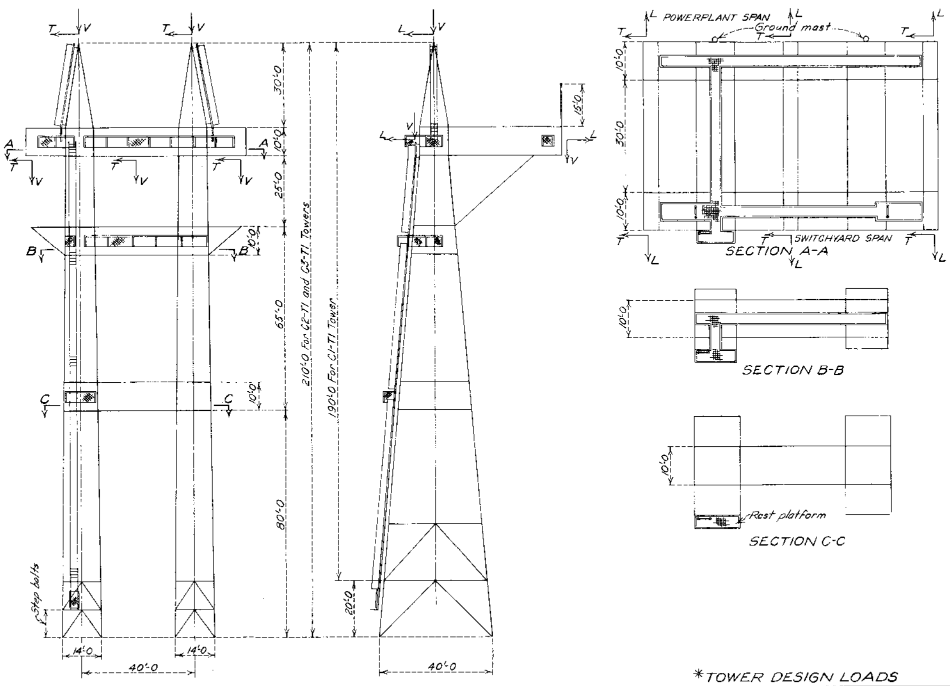
1. Transformer Circuits and Switchyards

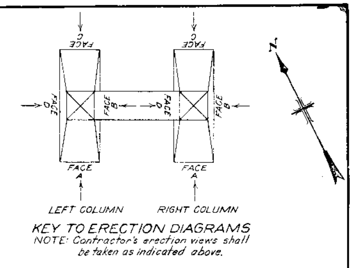
108. LOCATION. The four overhead transformer circuits, three at 345 kilovolts and one at 230 kilovolts, emanate from steel takeoff brackets on the powerplant wall at approximate elevation 3228.5. The three 345-kilovolt circuits pass under the Glen Canyon Bridge and rise to rim towers C1-T1, C2-T1, and C3-T1 (figs. 206, 207, and 208) on the west canyon rim. From these rim towers, the circuits proceed through backup towers C1-T2, C2-T2, and C3-T2 and then to the 345-kilovolt takeoff structure (fig. 209) in the switchyard. The 230-kilovolt circuit also passes under the bridge to rim tower C4-T1 on the east canyon rim, thence across the canyon to tower C4-T2 which accommodates the span to the 230-kilovolt takeoff structure in the switchyard (fig. 210).

The switchyard is located downstream from the dam, near the west canyon rim and south of the State highway and the bridge. This location provides for the most flexible arrangement for the several transmission line approach spans to the 345-, 230-, 69-, and 25-kilovolt yards and to the future 138-kilovolt yard.

109. STEEL STRUCTURES FOR TRANSFORMER CIRCUITS. (a) *Takeoff Brackets.*—A series of eight three-bay takeoff brackets were mounted on the exposed steel column flanges of the powerplant wall columns at a suitable elevation for required electrical clearances and conductor arrangement to the transformers and lightning arresters on the powerplant deck. This essentially provided a continuous run of line attachment beams, with the exception of breaks in continuity as imposed by expansion joints in the powerplant wall. With this takeoff bracket arrangement, sufficient line attachment spacing was realized for lines taking off at maximum 30^o horizontal and 37^o vertical line angles. The attachment spacing was correlated with phase spacing at the canyon rim towers to meet electrical requirements for midspan circuit spacing and phase separations.

Closely related to the takeoff bracket design was the problem of maintaining safe electrical clearances to





*FINAL WEIGHTS

TOWER TYPE	EXTRA HIGH STRENGTH STEEL 100,000 LB. YIELD	HIGH STRENGTH STEEL 50,000 LB. YIELD	STANDARD STRENGTH STEEL 36,000 LB. YIELD	HIGH STRENGTH BOLTS	ALUMINUM GRATING	TOTAL WEIGHTS
C1-T1	67,301	265,635	33,584	19,293	1,633	387,446
C2-T1	73,620	280,861	34,301	20,382	1,633	410,797
C3-T1	73,620	280,861	34,301	20,382	1,633	410,797
TOTAL WEIGHTS	214,541	827,357	102,186	60,057	4,899	1,209,040

* Weights shown in pounds and include galvanizing and details.

*TOWER DESIGN LOADS

CABLE	LOADING CONDITION	POWERPLANT SPAN			SWITCHYARD SPAN		
		L	T	V	L	T	V
CONDUCTOR	No ice, 22.3 lb. wind @ 60°F.	8,700	4,400	10,700	8,800	2,200	5,400
	1 inch rime @ 40 pcf., no wind, @ 30°F.	10,200	3,000	12,800	13,300	2,800	8,700
	¼ inch ice @ 57 pcf., 8 lb. wind, @ 15°F.	7,400	3,000	9,200	14,700	3,300	11,400
GROUND WIRE	No ice, 22.3 lb. wind, @ 60°F.				5,900	1,400	4,600
	1 inch rime @ 40 pcf., no wind, @ 30°F.				6,700	1,500	3,800
	¼ inch ice @ 57 pcf., 8 lb. wind, @ 15°F.				7,100	1,600	4,400

*Loads shown in pounds.
Overload Factor = 1.65 applied to the maximum member stress obtained from above loads.

REFERENCE DRAWINGS

LOCATION AND GENERAL ARRANGEMENT 557-D-2393
OUTLINE AND DESIGN .... 557-D-2590, -2591 & -2592
CONNECTIONS .... 557-D-2593, -2594 & -2595
CAGE, LADDER AND PLATFORMS 557-D-2596
STUBS 557-D-2587
CONNECTION BOLTS 557-D-2642
STEP BOLTS 40-D-5155

6-29-64 D-RJB	AS BUILT
3-13-63 D-DMR	REVISED ESTIMATED WEIGHTS.
3-1-63 D-DMR	GENERAL REVISIONS, DESIGN CHECK.
6-22-62 D-DMR	REVISED PLATFORMS AND ESTIMATED WEIGHTS. ADDED TOWER DESIGN LOADS.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
MIDDLE RIVER DIV.—GLEN CANYON UNIT—ARIZONA-UTAH

GLEN CANYON POWER PLANT
TRANSFORMER CIRCUITS-STEEL STRUCTURES
TOWERS NO. C1-T1, C2-T1 AND C3-T1
ASSEMBLIES

DRAWN R.D.M. SUBMITTED R.S. [signature]
TRACED ____ RECOMMENDED D. M. Robinson
CHECKED [initials] APPROVED Edwin [signature]
ACT. CHIEF, STRUCT. AND ARCH. BRANCH

DENVER, COLORADO, MARCH 1, 1962. 557-D-2589

Figure 206.—Transformer circuits steel structures—Assemblies of towers C1-T1, C2-T1, and C3-T1.

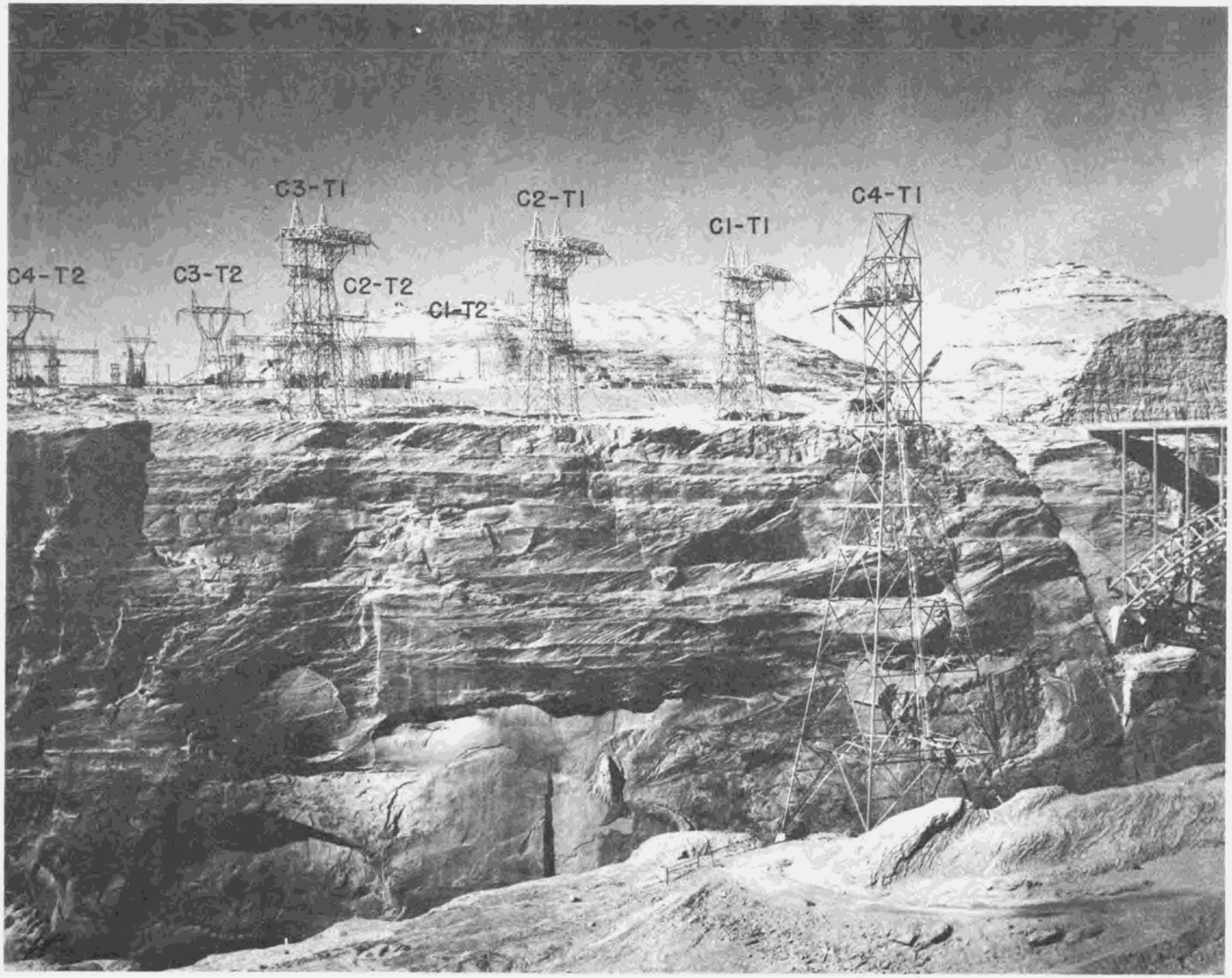


Figure 207.—Telephoto view of transformer circuit steel structures on canyon rim. P557-420-60090NA.

Figure 208.—Artist's conception of 345-kilovolt tower C1-T1 on canyon rim. P557-D-36377.

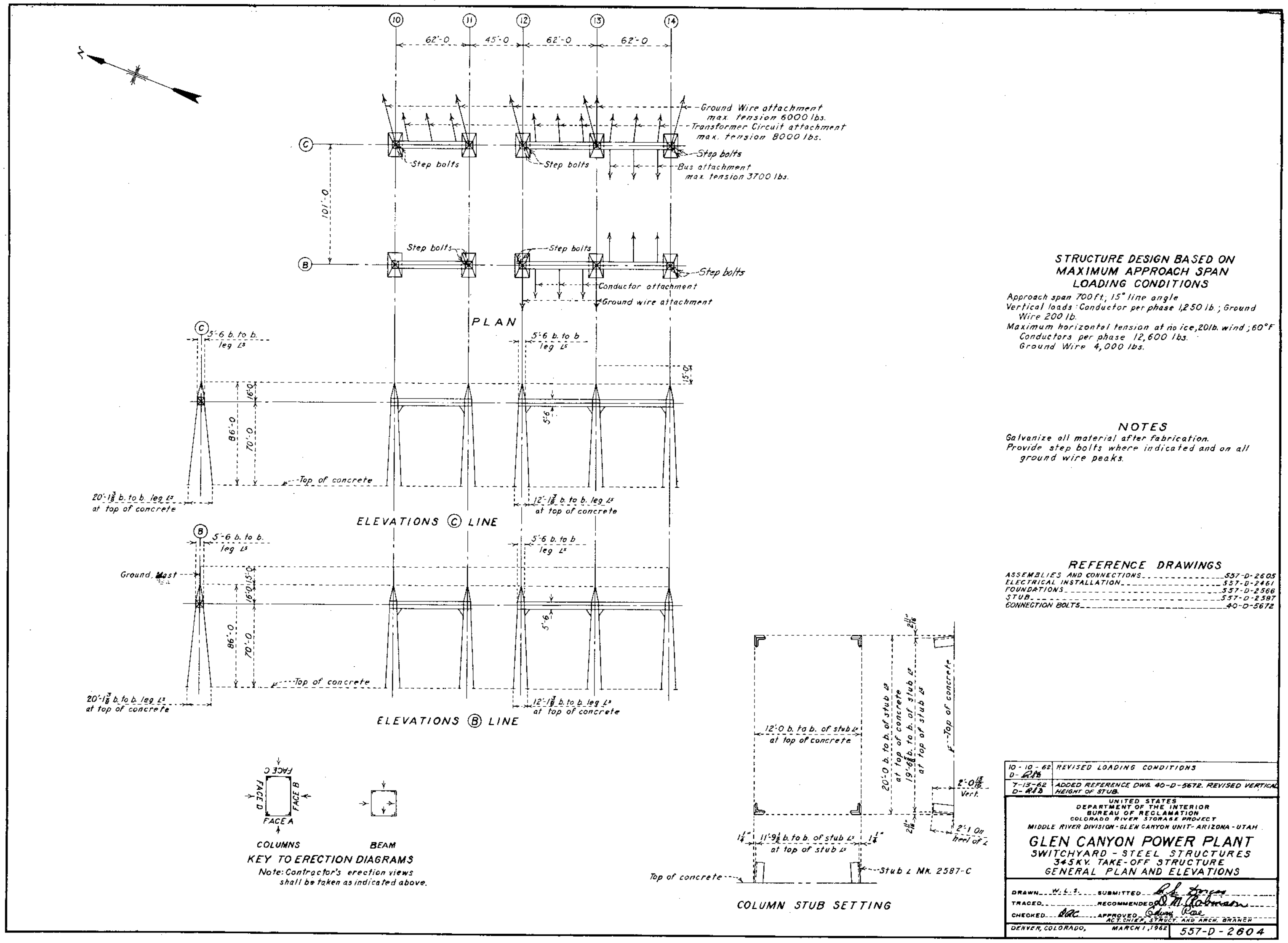


Figure 209.–Switchyard steel structures, 345-kilovolt takeoff structure–General plan and elevations.

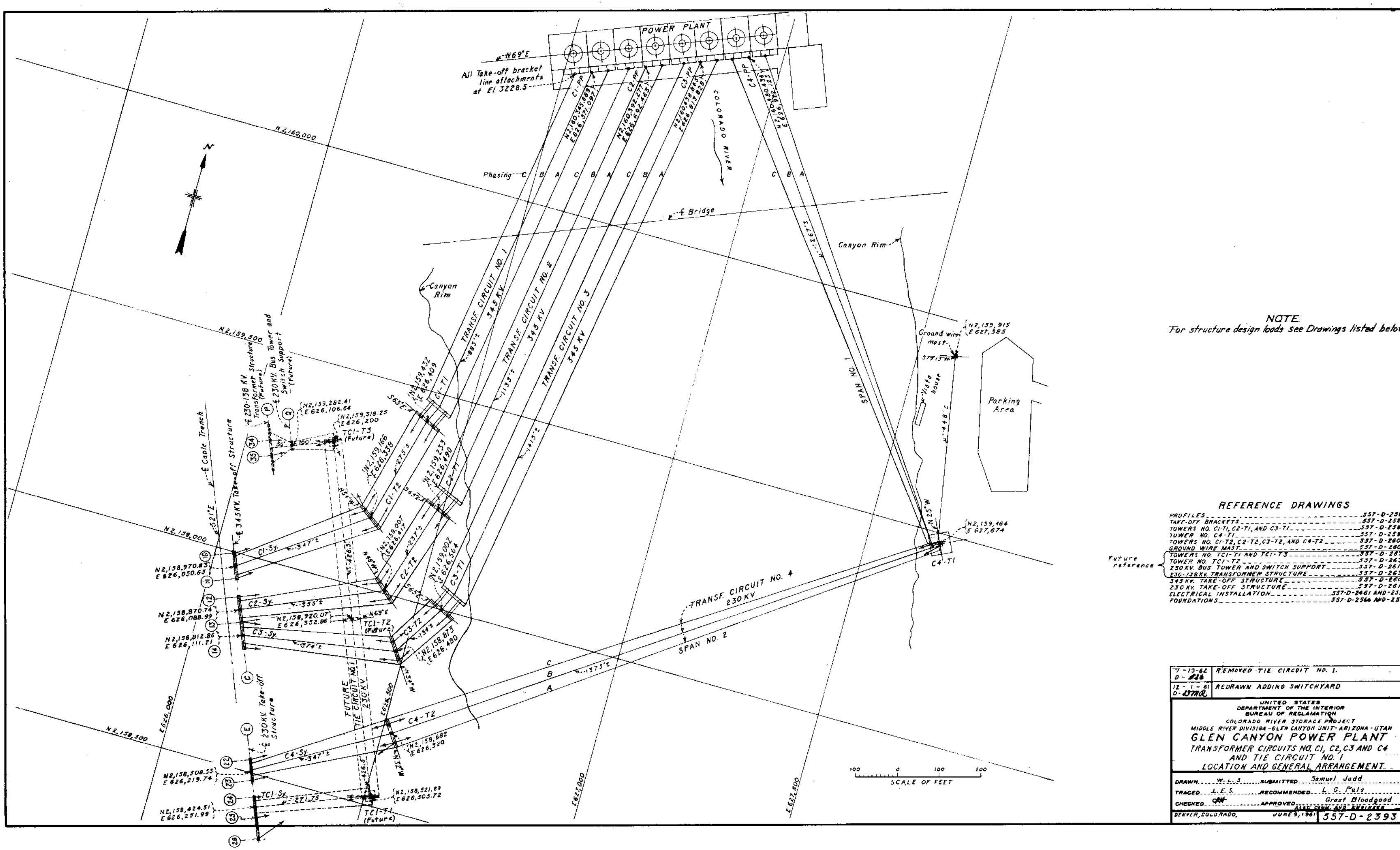


Figure 210.—Transformer circuits No. C1, C2, C3, and C4 and tie circuit No. 1—Location and general arrangement.

the downstream bridge arch, canyon walls, and canyon rims. Using the physical properties of a 2,167,000-circular-mil ACSR (Kiwi) conductor for the three 345-kilovolt circuits and 954,000-circular-mil ACSR (Cardinal) conductor for the one 230-kilovolt circuit, conditions of either sideswing from a 100-m.p.h. wind acting transversely, or the generally accepted elliptical movement in space from possible "galloping" were taken into account. These studies entailed careful positioning of the catenaries of conductors from the takeoff brackets to the rim towers. Also considered in the design was a maximum direct line tension of 16,000 pounds per conductor phase, with 1-inch radial ice at 40 pounds per cubic foot and no wind at 30° F. Because of these conductor problems at midspan, it was necessary to make concurrent designs on the powerplant takeoff brackets and canyon rim towers.

The beams of the takeoff brackets were designed to withstand bending stresses due to the 16,000-pound direct line tensions and torsional stresses induced by the vertical line loads acting in an offset position from centerline of the beam section. Using these line loads, coupled with transverse loads due to a 100-m.p.h. wind, a rigid-frame analysis was executed for designing the beam to bracket arm connections and the bracket arm to powerplant wall column connections.

(b) *345-Kilovolt Rim Towers C1-T1, C2-T1, and C3-T1.*—These towers (figs. 206 and 207) are the first in the United States using the new extra-high-strength structural steel shapes. The towers—two are 210 feet high and one is 190 feet high—incorporate three separate types of steel materials. These are heat-treated alloy steel of extra high strength, minimum yield point of 100,000 pounds per square inch; high-strength low-alloy steels, ASTM A 440, minimum yield point of 50,000 pounds per square inch; and standard-strength carbon steels, ASTM A 36, minimum yield point of 36,000 pounds per square inch. The legs of the two 210-foot towers from the ground to the 145-foot level and the legs of the 190-foot tower to the 125-foot level are of the extra-high-strength steel shapes; the remaining 65 feet of the legs of the structures and the entire web system and chords are the A 440 steel. The cage, ladders, and walkway platform supports are the A 36 steel. The walkway platforms are aluminum, in accordance with Federal Specifications RR-G-661a, type I, for a loading of 50 pounds per square foot.

Each tower supports a 345,000-volt transformer circuit emanating from the 900,000-kilowatt Glen Canyon Powerplant at the toe of the dam. The 3-phase transformer circuits, ranging from 900 to 1,400 feet in horizontal span length, pass under the Glen Canyon Bridge, a short distance downstream from the dam, and rise 800 feet from the powerplant to the rim towers.

Many unprecedented problems were encountered in the design of these rim towers. Studies were conducted to determine the special structure design characteristics required to overcome the many difficulties presented in reaching a satisfactory solution for transmitting electrical power out of the canyon, as mentioned in subsection (a) above.

The heights of the rim towers were determined by an acceptable conductor clearance under the bridge and over the canyon rim edge. The structures are located at safe distances from the canyon rim, consistent with foundation safety, adequate conductor spacing, and span length limitations. A minimum of 25 feet was permitted from the intercept of the canyon wall plane with the ground surface to the tower column legs. These requirements necessitated a cantilever design concept for the conductor attachment beams on the structures. From the resolution of these requisites, effective outlines of the structures were established with the most efficient use of both ground and air space.

The structures were also designed to withstand a 100-m.p.h. wind, plus an increase in wind pressure of 10 percent due to structure height, with an exposure factor of 1.7 applied simultaneously to both columns. The analysis was performed for all possible combinations of intact and broken wire assumptions, including full dead end on either side of the structures, plus maximum wind from any direction and with a factor of safety of 1.65.

Based on preliminary design computations, several types of built-up sections, using high-strength structural steel angles and plates bolted together, were tried and compared with the extra-high-strength steels. The use of the extra-high-strength steels resulted in a substantial savings in weight of material and fabrication and erection time.

The proper use of the three different strengths of steel resulted in far more economical structures than normally could be attained with one type of steel. Weight saved by the extra-high-strength steels was 50 percent in those portions of the structures where they were used. The weight reduction resulted in a 20 percent savings in the total cost of the structures.

The structures were designed for the tabulated loads shown on figure 206. The tower portion of the structures was analyzed as an indeterminate, three-story, single-bay portal frame. The cantilever portion was analyzed as indeterminate frames in both the plan and elevation views.

The reactions from the analysis of the cantilever portion of the structure were applied to the tower column as additional loads; also, the moment reaction from the cantilever frame in plan caused a torsional load on the tower columns.

Another very difficult and highly indeterminate problem was the torsional distribution at the top of the columns. After many studies were made on the torsional rigidity of the column versus the bending rigidity of the beam, a distribution of one-third of the torsion to the column and two-thirds to the beam was decided upon. The variable moment of inertia of the columns and the effect of sidesway were taken into account in the analysis of the tower and cantilever.

After careful consideration of the geological conditions in the area, it was decided to assume the base of the structure as being fully fixed. Using the results of the frame analysis and treating all the beams, columns, and cantilever parts as free bodies, all the forces on these free bodies were obtained for the resolution of stress diagrams.

The entire analysis up to and including the stress diagrams was performed for unit loads on the structures. This permitted the use of these analyses for all the various combinations of loads without repeating any of the analysis for the different cases. The beam-to-column and cantilever-to-tower connections are highly indeterminate and many solutions of these indeterminate connections were made using all possible combinations of loads.

The use of an extra-high-strength steel (100,000-p.s.i. yield point) required the development of a new column design equation. Using the secant formula a curve was developed to establish the relationship between slenderness ratio and ultimate stress, taking into account a reduction of 25 percent in yield point to 75,000 pounds per square inch.

To obtain the equation used for design, two straight-line segments were constructed tangent to the column design curve. The equations of these two straight lines, relating a range of ultimate stresses to a range of slenderness ratios, were used for the design equations. The cutoff point for the maximum ultimate stress was made either at ℓ/r equal to 50 or a stress of 54,800 pounds per square inch.

The design requirements for members of high-strength ASTM A 440 and A 36 materials are consistent with the requirements of the Bureau's Design Standards No. 10, Transmission Structures. A program developed by the Bureau for the electronic computer was available for use in determining the moments and shears for the numerous cases in the horizontal frame of the cantilever, and was used to check manual computations.

Because of the zero coefficient of friction between galvanized members, all structural joints were designed as shear connections using 7/8-inch-diameter ASTM A 325 bolts with recessed nuts tightened to a torque of 190 pound-feet to insure tight connections. The recessed nuts insured the exclusion of bolt threads from the shear plane and were locked in place with locknuts.

The allowable unit bolt stresses used were 40,000 pounds per square inch in shear and 80,000 pounds per square inch in bearing with an ultimate single shear load value of 22,600 pounds per bolt. Bearing values on materials were 12,100 pounds on a 3/16-inch thickness, 16,200 pounds on 1/4 inch, 20,300 pounds on 5/16 inch, and 24,300 pounds on 3/8 inch.

Hot-dipped galvanizing was used on the A 440 and A 36 materials, and coatings of zinc dust-zinc oxide paint on the extra-high-strength material were used to prevent corrosion, thereby extending structure life and minimizing structure maintenance.

For inspection and maintenance, an encaged ladder was provided on each tower for the full length of one of the columns and on both ground-wire peaks. A rest platform was provided at the 90-foot level, and inspection and maintenance platforms at the 135- and 170-foot levels. The platforms are completely encircled with a 3-1/2-foot-high wire fence for safety of personnel.

(c) *230-Kilovolt Rim Tower C4-T1.*—The same sag tension studies as mentioned for the conductors, in subsections (a) and (b) above were required for the 230-kilovolt circuit in order to maintain safe electrical clearances and to obtain the loads imposed on the tower.

This tower was designed to accommodate the conductors in a vertical configuration. This configuration served the dual purpose of providing reasonable sideswing clearances to the canyon wall, and providing conductor attachments such that it was possible to turn on a 90° line angle for the canyon crossing span without having to allow for electrical clearances through the structure, as was the case for the 345-kilovolt rim towers. Horizontal configuration would have required the use of two taller towers for accomplishing the same results.

The tower was designed for the tabulated loads shown on figure 211. Stress diagrams were drawn for each of the loads separately and combined for the various conditions to obtain the maximum stress in all members. This tower is unique in that the wire loads as applied to the tower imposes torsional loads at all three crossarm levels.

The design requirements for these members are consistent with the requirements of the Bureau's Design Standards No. 10, Transmission Structures. The leg angles are high-strength structural steel angles in accordance with ASTM A 440 and the web system and crossarms are standard strength A-36 material.

This tower was also provided with an encaged ladder and inspection and maintenance platforms as described under subsection (b) above.

(d) *Model Aids Design.*—A three-dimensional scale model of Glen Canyon features was built by the Bureau's engineering laboratories. The model projects the spatial relationships of major elements of the unit—powerplant, switchyard, bridge, and transformer circuit structures.

The model was used to check the planned solution to the difficult space problem of positioning the catenaries of conductors from the powerplant to the rim towers. A fine-link chain was suspended between the model rim towers and powerplant takeoff brackets to simulate the catenaries of the conductors and check all critical electrical clearances, tower heights, phase configurations, and rock trimming along the edges of canyon rims. Templates representing the maximum sideswing of conductors under transverse windloading were also used to check critical electrical clearances from the conductor sideswing positions to the canyon walls. For these checks to be valid, it was necessary that the model be of greater than normal accuracy with respect to topography and positioning of critical structures. Close checks between the model and calculations demonstrated that this effort was worthwhile.

110. STEEL SWITCHYARD STRUCTURES. The 345-kilovolt switchyard is the first Bureau switchyard which encompasses this high voltage. Therefore, all of the steel structures were newly designed for this first usage at Glen Canyon.

The several structure outlines were determined by the increased requirements for electrical clearances and heavy electrical equipment loads. These outlines are consistent with the existing structure outlines for previously designed 230- and 138-kilovolt switchyards.

The switchyard structures are of hot-dip galvanized structural steel, field assembled with high-strength bolted connections. The major structures were designed to withstand the loads imposed by conductors and overhead ground wires, wind and dead loads, and the loads imposed by the supported electrical equipment. The minor structures were designed for wind and dead loads and the weight of the supported equipment.

The takeoff structures and the transformer structures were designed for the tabulated loads shown on the drawings and also for the National Electrical Safety Code medium loading conditions of 1/4-inch radial ice at 57 pounds per cubic foot at 15° F. and a wind pressure of 8 pounds per square foot. The structures were analyzed as multiple-bay single-story portal frames in the transverse direction and as cantilevers in the longitudinal direction.

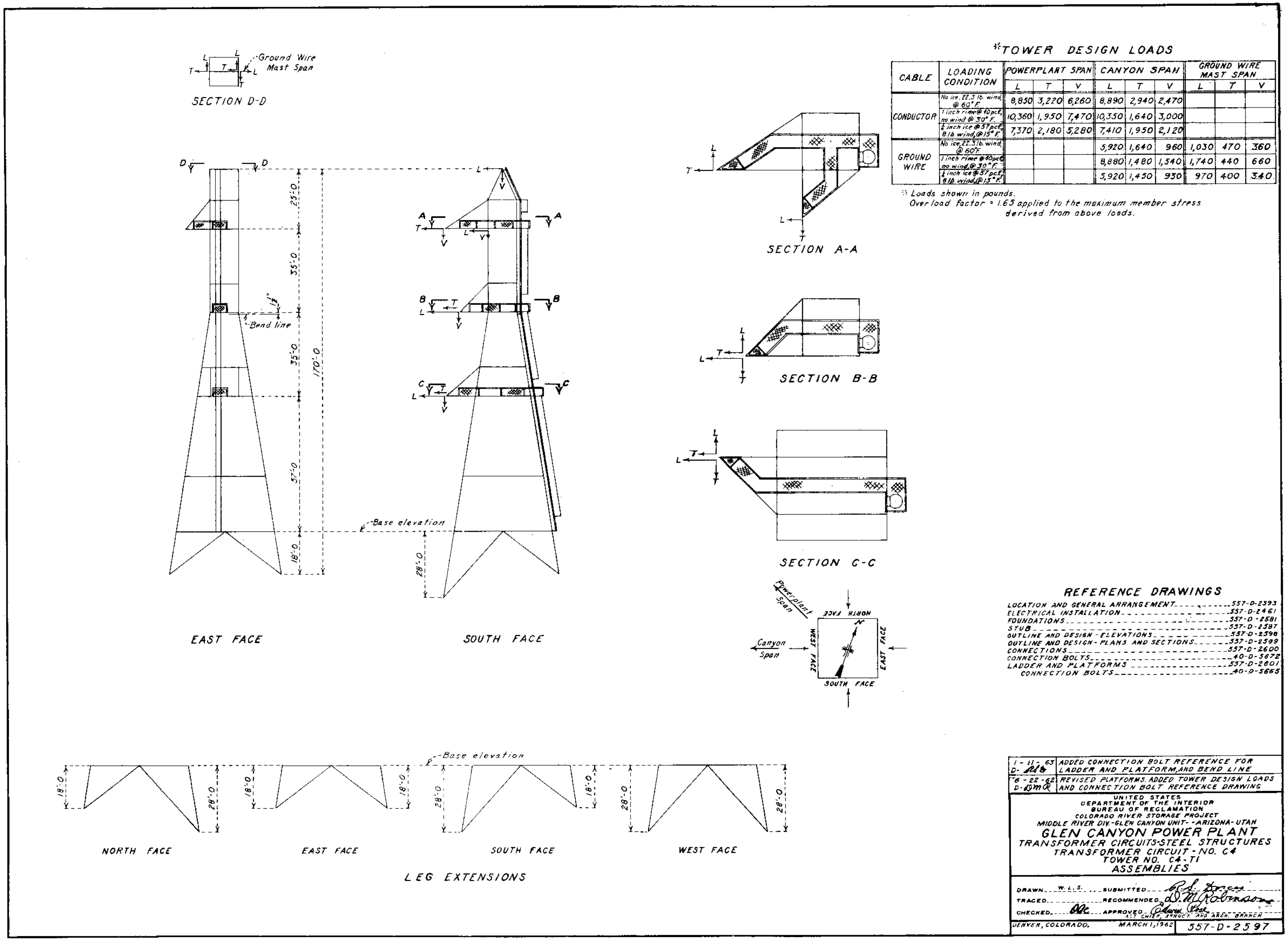
The maximum stresses in the individual space frame-type towers were obtained by combining unit stress diagrams for the various conditions of loadings including vertical loads, dead loads, and horizontal torsional shears produced by torsional moments of unbalanced longitudinal wires.

Wire tensions for conductors, ground wires, and tension buses were determined for the most severe condition of wind and temperatures as shown on figures 206, 209, 212, and 213. The wires of the approach spans of transmission lines and transformer circuits which anchor to the switchyard structures were slacked off to approximately one-half to two-thirds of the maximum tensions used throughout the lines in order to effect economies in the switchyard structures. This was permissible because the switchyard approach spans were not critical with respect to electrical clearances to the lower level of strain buses.

Stringing instructions for conductors and ground wires were calculated to insure that the design loads of the structures will not be exceeded (figs. 214 and 215). Tensions for conductors and ground wires within the switchyards were established by the following criteria:

(1) Strength of wires, insulators, and hardware.

(2) Limitation of tension at normal temperatures to avoid vibration in the wires.

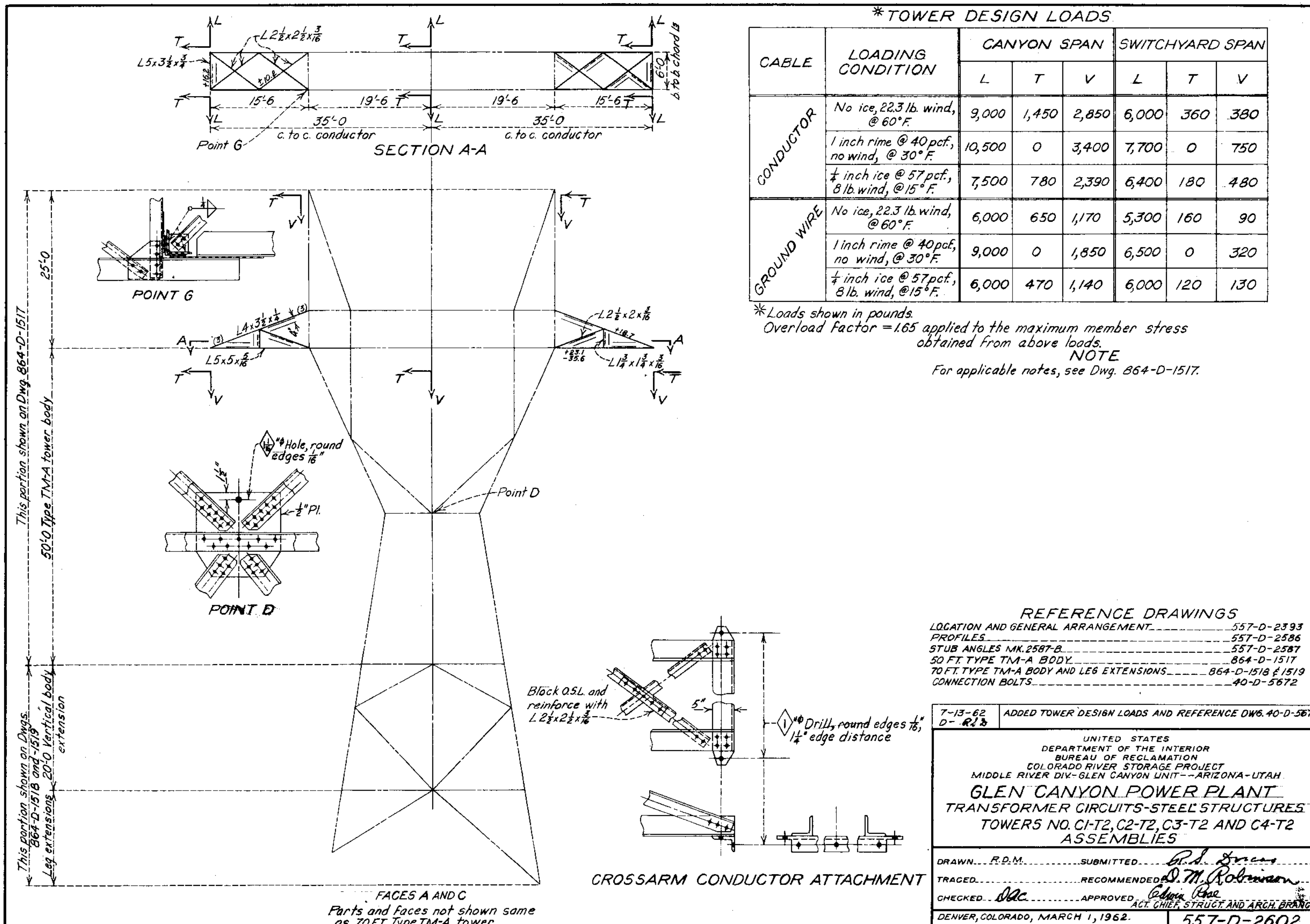


*TOWER DESIGN LOADS

CABLE	LOADING CONDITION	POWERPLANT SPAN			CANYON SPAN			GROUND WIRE MAST SPAN		
		L	T	V	L	T	V	L	T	V
CONDUCTOR	No ice, 22.3 lb. wind, @ 60° F.	8,850	3,220	6,260	8,890	2,940	2,470			
	1 inch rime @ 40 pcf, no wind @ 30° F.	10,360	1,950	7,470	10,350	1,640	3,000			
	¼ inch ice @ 57 pcf, 8 lb. wind, @ 15° F.	7,370	2,180	5,280	7,410	1,950	2,120			
GROUND WIRE	No ice, 22.3 lb. wind, @ 60° F.				5,920	1,640	960	1,030	470	360
	1 inch rime @ 40 pcf, no wind, @ 30° F.				8,880	1,480	1,540	1,740	440	660
	¼ inch ice @ 57 pcf, 8 lb. wind, @ 15° F.				5,920	1,450	930	970	400	340

* Loads shown in pounds.
Overload factor = 1.65 applied to the maximum member stress derived from above loads.

Figure 211.–Transformer circuits steel structures–Assemblies of tower No. C4-T1.



*TOWER DESIGN LOADS

CABLE	LOADING CONDITION	CANYON SPAN L	CANYON SPAN T	CANYON SPAN V	SWITCHYARD SPAN L	SWITCHYARD SPAN T	SWITCHYARD SPAN V
CONDUCTOR	No ice, 22.3 lb. wind, @ 60°F.	9,000	1,450	2,850	6,000	360	380
	1 inch rime @ 40 pcf., no wind, @ 30°F.	10,500	0	3,400	7,700	0	750
	¼ inch ice @ 57 pcf., 8 lb. wind, @ 15°F.	7,500	780	2,390	6,400	180	480
GROUND WIRE	No ice, 22.3 lb. wind, @ 60°F.	6,000	650	1,170	5,300	160	90
	1 inch rime @ 40 pcf., no wind, @ 30°F.	9,000	0	1,850	6,500	0	320
	¼ inch ice @ 57 pcf., 8 lb. wind, @ 15°F.	6,000	470	1,140	6,000	120	130

*Loads shown in pounds.
Overload factor = 1.65 applied to the maximum member stress obtained from above loads.

NOTE
For applicable notes, see Dwg. 864-D-1517.

Figure 212.—Transformer circuits steel structures—Assemblies of towers C1-T2, C2-T2, C3-T2, and C4-T2.

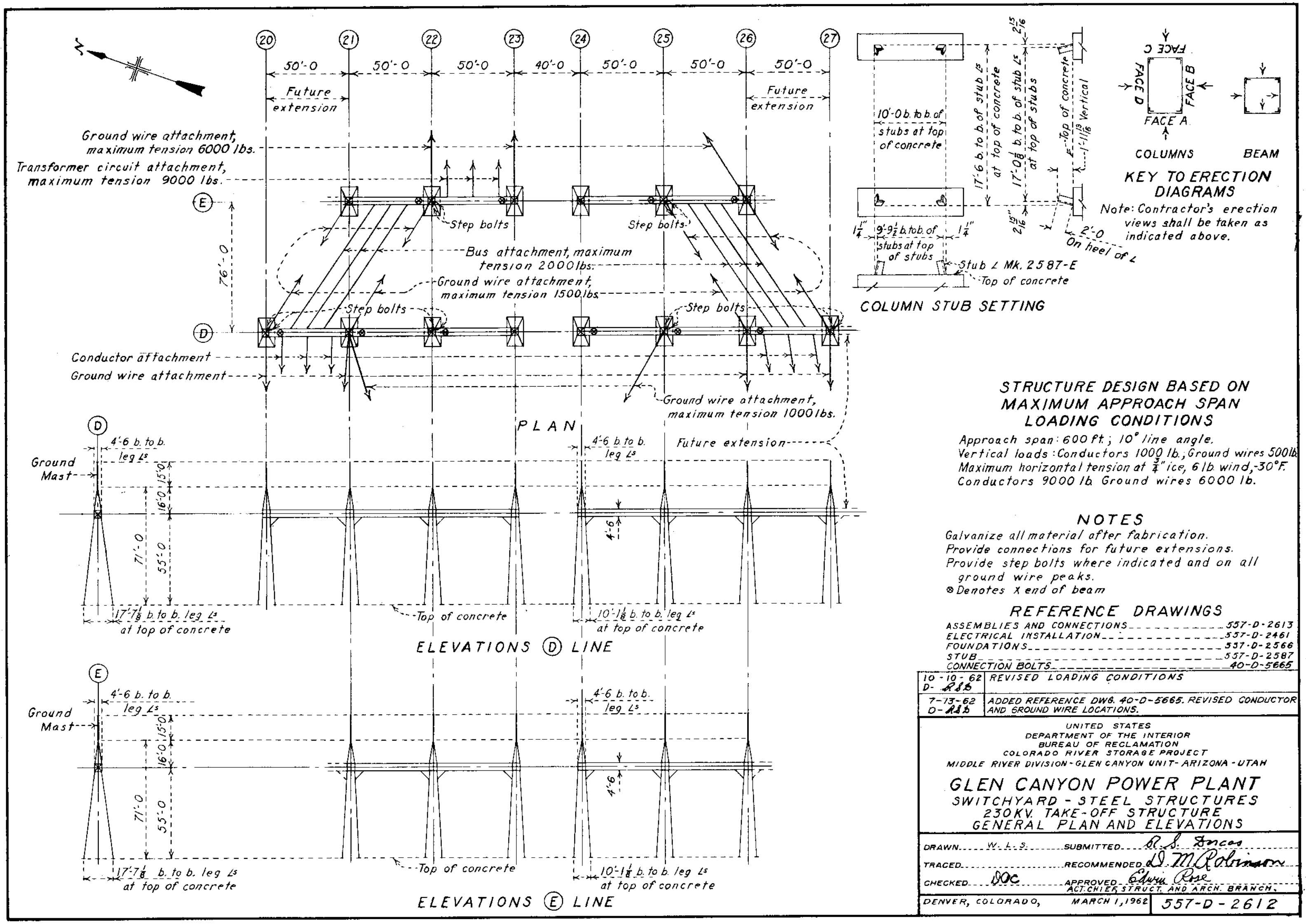


Figure 213.—Switchyard 230-kilovolt steel takeoff structure—General plan and elevations.

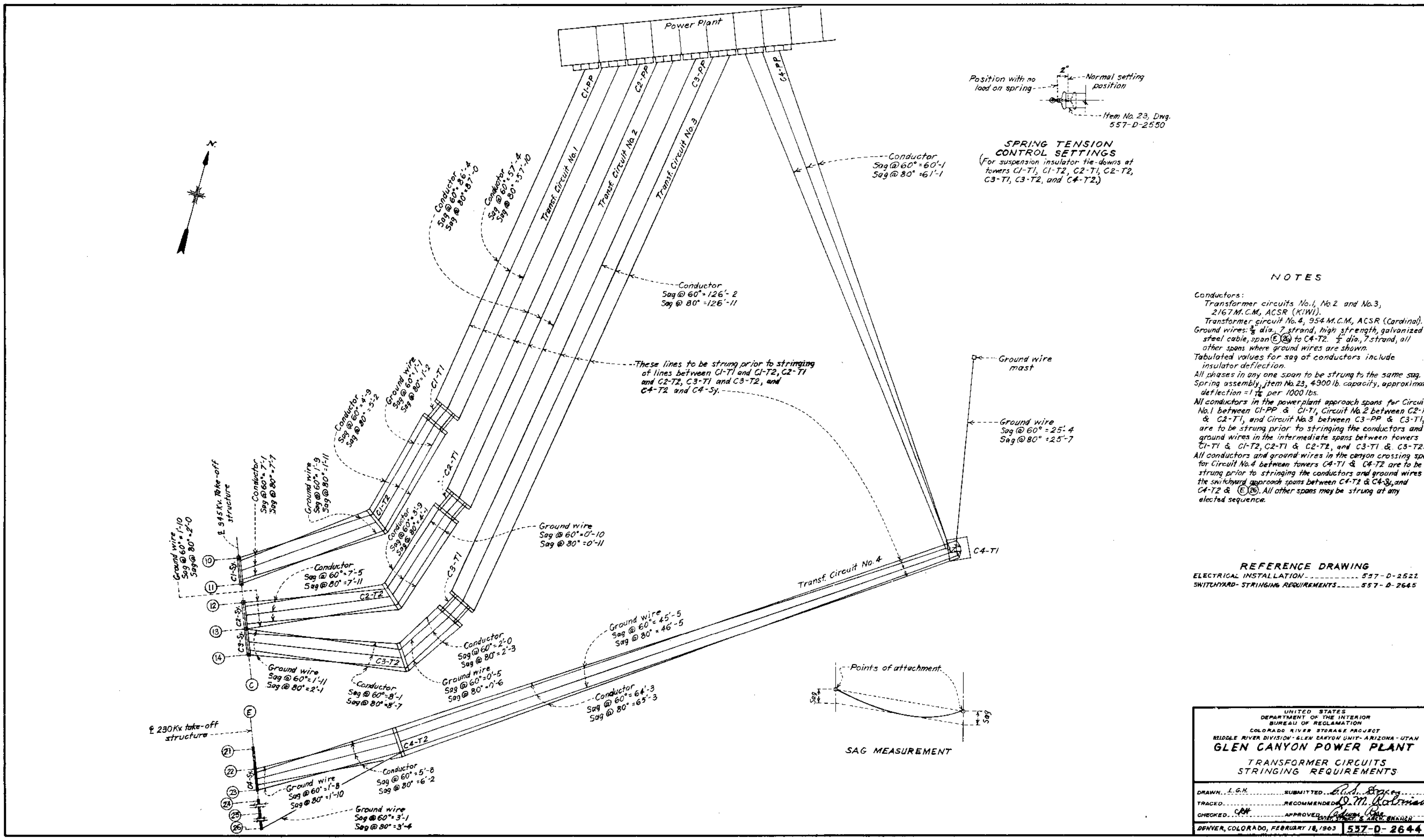


Figure 214.—Transformer circuits stringing requirements.

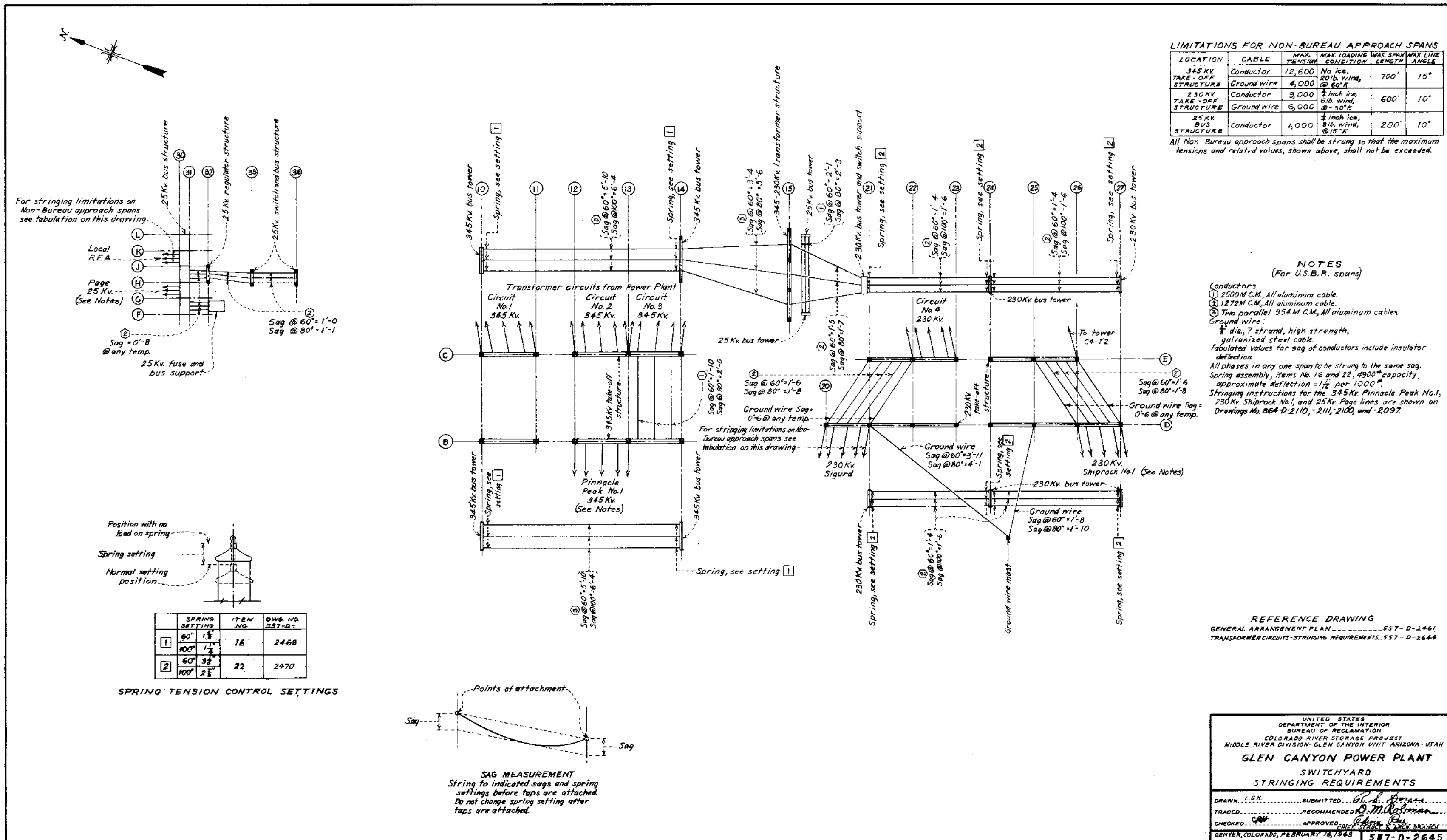


Figure 215.—Switchyard stringing requirements.

(3) Economy of the supporting structures.

(4) Electrical clearances.

(5) Avoidance of unsightly sags at maximum temperatures considering the weights of insulators and taps.

In the conductor stringing operation the powerplant spans were strung prior to stringing backup spans between the rim towers and switchyard approach towers. This eliminated any undue loads on the switchyard approach towers from possible rim tower deflections resulting from powerplant span loads.

Owing to changes in power commitments, the 138-kilovolt switchyard and corresponding transformer circuits were deleted from the specifications requirements.

B. ELECTRICAL

111. REQUIREMENTS AND GENERAL DESCRIPTION. The switchyard is composed of 345-, 230-, 69-, 24.9-, and 4.16-kilovolt structures and space for a future 138-kilovolt structure within a fenced area located on the canyon rim approximately 850 feet southwest of the right dam abutment. The elevation of the switchyard is approximately 3,885 feet above sea level. The transformer circuits extend from the transformer deck of the powerplant to the switchyard and consist of three 345- and one 230-kilovolt overhead lines. The general arrangement of the switchyard is shown on figure 216 and that of the transformer circuits on figure 217. The switching arrangement and principal equipment ratings and procurement data are shown on figure 218.

112. GENERAL DESIGN. The main buswork for the 345- and 230-kilovolt installations is of the strain-type design, providing for an ultimate breaker and one-half switching arrangement. Initial operation is essentially as ring buses. The typical general arrangement of these installations is shown on figures 219 and 220, and the arrangement of the main autotransformer bank installation is shown on figures 221 and 222.

The buswork for the 69- and 25-kilovolt installations is of the rigid-type design with main and transfer buses. The general arrangement of these installations including the 69/25-kilovolt power transformer installation and power supplies for station service and other auxiliaries is shown in plan on figures 223 and 224. Typical details of the takeoffs on the transformer deck and tower connections for the transformer circuits are shown on figures 225 and 226.

The control and low-voltage power cables are run in cable trenches through the switchyard. The location and details of the cable trenches are shown on figure 227. The cable trenches have removable aluminum plate covers throughout their entire length. The control cables and alternate station-service power supply run from the powerplant to the switchyard by means of the cable tunnel which joins the cable trench at the north end of the switchyard.

113. INSULATION, COORDINATION, AND LIGHTNING AND SWITCHING SURGE PROTECTION. The electrical insulation and protective devices were selected and coordinated to provide a safe margin of insulation strength above the maximum abnormal voltages permitted by the protective equipment during lightning, switching, and short-circuit surges.

All 345-, 230-, and 69-kilovolt lines and circuits are protected by two overhead ground wires terminated on peaks of the structures above the conductor takeoff levels, except the portion of the four transformer circuits between the powerplant and the canyon rim towers. This portion of the transformer circuits was considered to be adequately protected from lightning by the Glen Canyon Bridge, under which they pass, and the rim towers. Additional overhead ground wires have been provided to shield all buswork which would not otherwise be shielded.

Lightning arresters are mounted on or adjacent to all equipment having wound-type internal construction and all insulated cables having voltage ratings exceeding 600 volts, to dissipate lightning surges.

114. GROUNDING SYSTEM. The protective grounding system in the switchyard consists of 3/4-inch by 20-foot copperweld ground rods installed around the perimeter of the switchyard and around each lightning arrester installation. The ground rods are tied together by a network of 500,000-circular-mil A.W.G. bare copper cable buried 6 to 18 inches below grade. All equipment and steel structures, including the fence, are connected to the ground mat. A ground bus consisting of two 500,000-circular-mil A.W.G. copper cables extends through the cable tunnel to connect the switchyard ground mat with the powerplant ground mat. The switchyard key grounding plan is shown on figure 228.

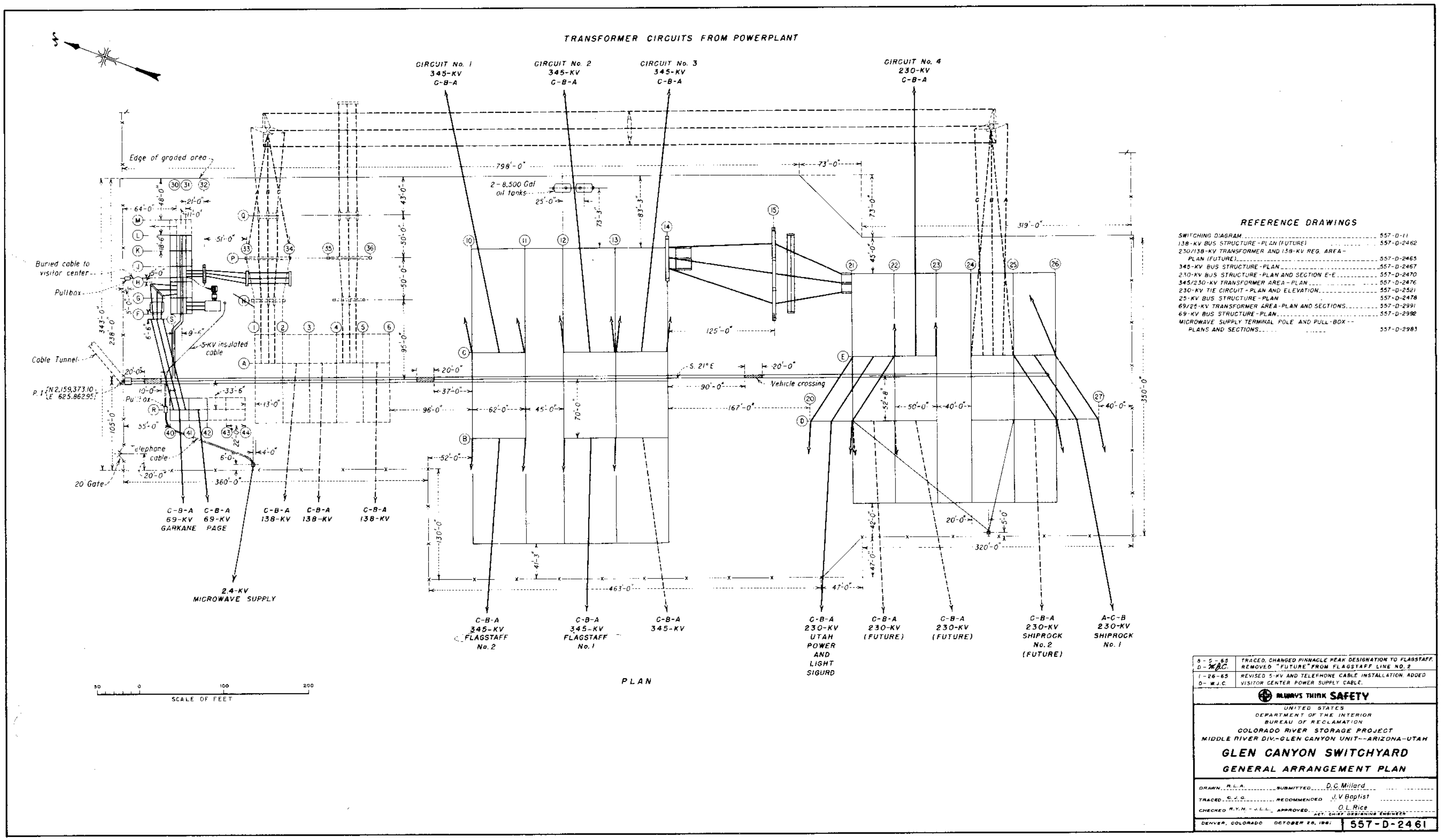


Figure 216.—Switchyard general arrangement—Plan.

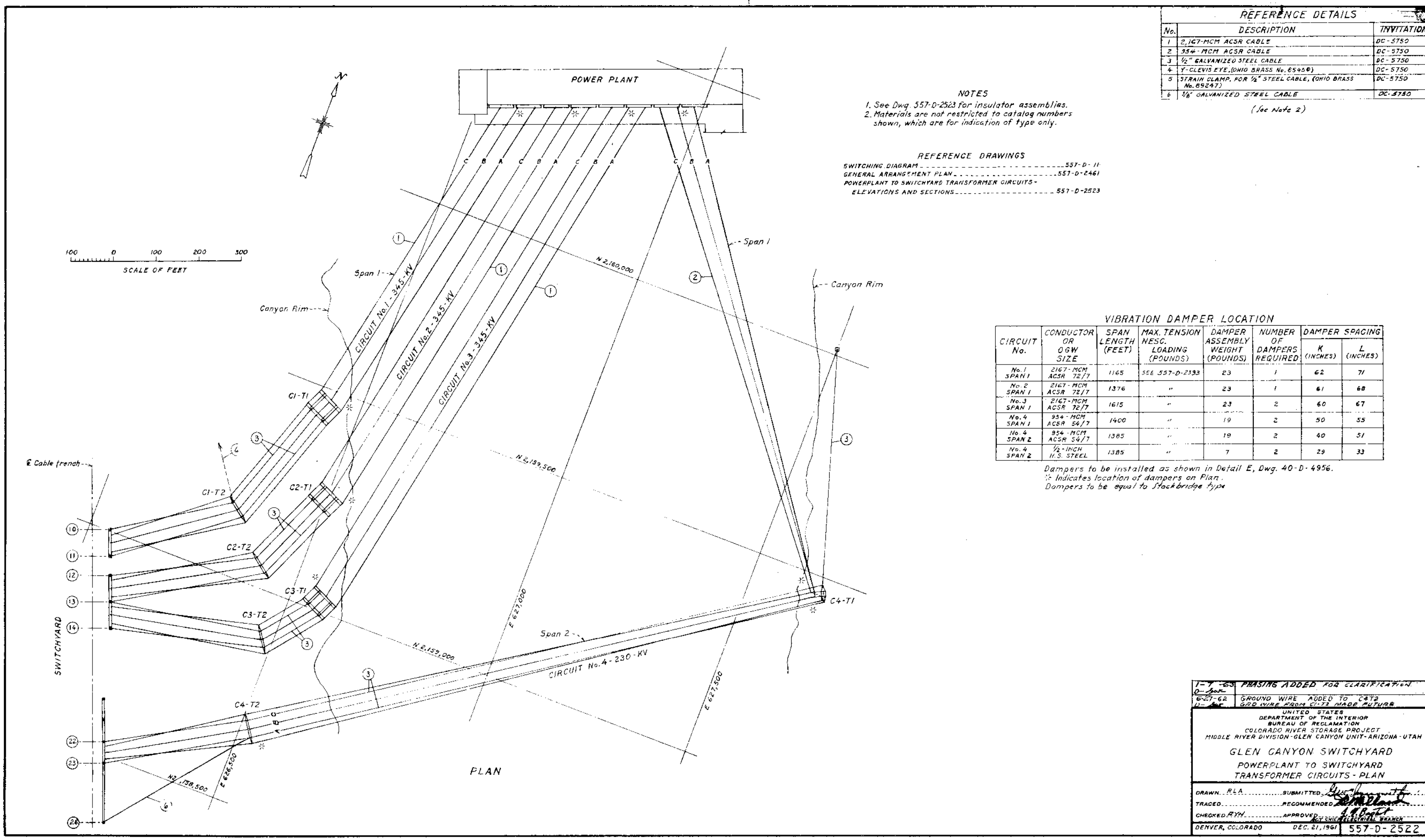


Figure 217.—Transformer circuits from powerplant to switchyard—Plan.

SWITCHES

DESIGNATION	AMP.	KV.	INVITATION NO.
WUIA, WUIB, WU3A, WU3B, YUIA, YUIB, YUID, YU3A, YU3B, YU3D,	1,600	345	DS-5860 (10)
YUIC - ZUIB YU3C - ZU3B	1,600	345	DS-5860 (9)
WU5C	1,600	345	DS-5860 (11)
WZ5A, WZ6A, YZ5A, YZ6A, YZ7B,	1,600	230	DS-5860 (7)
YZ5B - ZZ5A YZ5C - ZZ5B YZ6B - ZZ6A YZ6C - ZZ6B	1,600	230	DS-5860 (6)
WZ5C	1,600	230	DS-5860 (11)
WWIB	600	69	(D) 90,708-A (1)
WWIA, YWIB, YW3B,	600	69	(D) 90,708-A (2)
YWIA - ZWIA YW3A - ZW3A	600	69	(D) 90,708-A (3)
WVIA, WVID, WV2A, WV2C, WV3A, WV4A, WV5A, WV5B, YV2A, YV2B, YV3A, YV3B YV4A, YV4B,	1,200	23	DS-5860 (12)
WVIB	1,200	23	DS-5860 (3)
WVIC	1,200	23	DS-5860 (1)
WVIF, WVIG	2,000	23	DS-5860 (5)
WV2B	1,200	23	DS-5860 (4)
XVIA	1,200	23	DS-5860 (2)

TRANSFORMERS AND COUPLING CAPACITORS

DESIGNATION	KV.	INVITATION NO.
KIA, K3A, K5A	345	DS-5780 (1)
KU5A	345	DS-5784 (1), DS-6473(1)
VUIAI, VUIB2, VUIB3, VU3A2, VU3A3	345	(D) 90,635-A (2)
VUIBI, VU3AI	345	(D) 90,635-A (1)
K7A	230	DS-5780 (2)
QZ5A (300/600-5)	230	(D) 90,642-A (1)
VZ5AI, VZ6AI	230	(D) 90,635-A (3)
VZ5A2, VZ5A3, VZ6A2, VZ6A3	230	(D) 90,635-A (4)
VZ7A, VZ7B (138,000-115//69.0)	230	(D) 90,641-A (1)
KWIA	69	DS-6164 (1)
QWIA, QW3A (75/150-5)	69	(D) 90,720-A (2)
VWIAI, VW3A (40,250-115//67.08)	69	(D) 90,720-A (1)
KVIA	25	DS-5875 (2)
KVIB		(D) 90,646-A (1)
KV2A	25	DS-5875 (1)
KV2B	25	(D) 90,639-A (1)
KV2C	25	(D) 90,639-A (2)
KV2D	25	DS-6293 (1)
KV2E	25	(D) 90,706-A (1)
QV2A (150/300-5)	25	(D) 90,642-A (3)
QV3A, QV4A (200/400-5)	25	(D) 90,642-A (2)
VV5A (14,400-120//72)	25	(D) 90,641-A (1)

TRANSFORMERS AND COUPLING CAPACITORS

DESIGNATION	KV.	INVITATION NO.
Q2A, Q4A, Q6A (300/600-5)	15	(D) 90,642-A (5)
Q8A (400/800-5)	15	(D) 90,642-A (4)
KV2F	2.4	DC-5853

LIGHTNING ARRESTERS

(1) STATION TYPE	276	DS-5780 (1)
(2) STATION TYPE	276	DS-5784 (1)
(3) STATION TYPE	195	DS-5780 (2)
(4) STATION TYPE	195	DS-5784 (1)
(5) STATION TYPE	195	(D) 90,647-A (2)
(6) STATION TYPE	60	DS-6164 (1)
(7) INTERMEDIATE TYPE	60	(D) 90,715-A (1)
(8) INTERMEDIATE TYPE	60	(D) 90,723-A (1)
(9) STATION TYPE	30	DS-5875 (2)
(10) STATION TYPE	30	DS-5784 (1)
(11) STATION TYPE	30	(D) 90,647-A (1)
(12) STATION TYPE	30	DS-6164 (1)
(13) INTERMEDIATE TYPE	25	DS-5875 (1)
(14) INTERMEDIATE TYPE	3	DC-5853
(15) INTERMEDIATE TYPE	3	(D) 90,715-A (2)
(16) INTERMEDIATE TYPE	3	(D) 90,726-A (1)

WAVE TRAPS

DESIGNATION	AMP.	INVITATION NO.
TU3A	1,200	(D) 92,050-A

CIRCUIT BREAKERS AND POWER FUSES

DESIGNATION	INTERRUPTING CAPACITY	AMP.	KV.	INVITATION NO.
JUIA, JUIB JUIC, JU3A JU3B, JU3C	25,000 MVA	1,600	345	DS-5721 (1)
JZ5A, JZ5B JZ6A, JZ6B	20,000 MVA	1,600	230	DS-6287 (1)
JWIA, JW3A	2,500 MVA	1,200	69	(D) 90,700-A(1)
JV2A, JV3A, JV4A, JV5A	1,500 MVA	1,200	34.5	DS-5816 (2)
FV2A	14,000 AMP	25E	34.5	DS-5860 (13)
FV2B	14,000 AMP	20E	34.5	DS-5860 (14)
FV2C	14,000 AMP	100N	34.5	(D) 90,722-A(1)
FV2D2	14,000 AMP	7E	34.5	(D) 90,724-A(1)
FV2D3	14,000 AMP	7E	34.5	(D) 90,708-A-1(4)
FV2E		150N	7.2	(D) 90,722-A-1(2)

7-9-69 D-	CHANGED DESIGNATION
10-17-66 D-	RETRACED AND REPLACED JZ5A, 5B, 6A, AND 6B. CORRECTED KU5A DATA.
9-30-65 D-G.W.C.	CORRECTED KV2D RATING. COMPLETED INV. NO'S FOR TABLES.

ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT

GLEN CANYON POWER PLANT & SW'YD.

SWITCHING DIAGRAM

DRAWN J.L.L. SUBMITTED D.C. Millard
TRACED L.L.M. RECOMMENDED M.H. Kight
CHECKED G.W.C.-D.C. APPROVED K.B. Keener CHIEF DESIGNING ENG'R.

DENVER, COLORADO, MAY 18, 1956 | 557-D-11

Figure 218.—Glen Canyon Powerplant and Switchyard—Switching diagram. (Sheet 1 of 2.) From drawing No. 557-D-11.

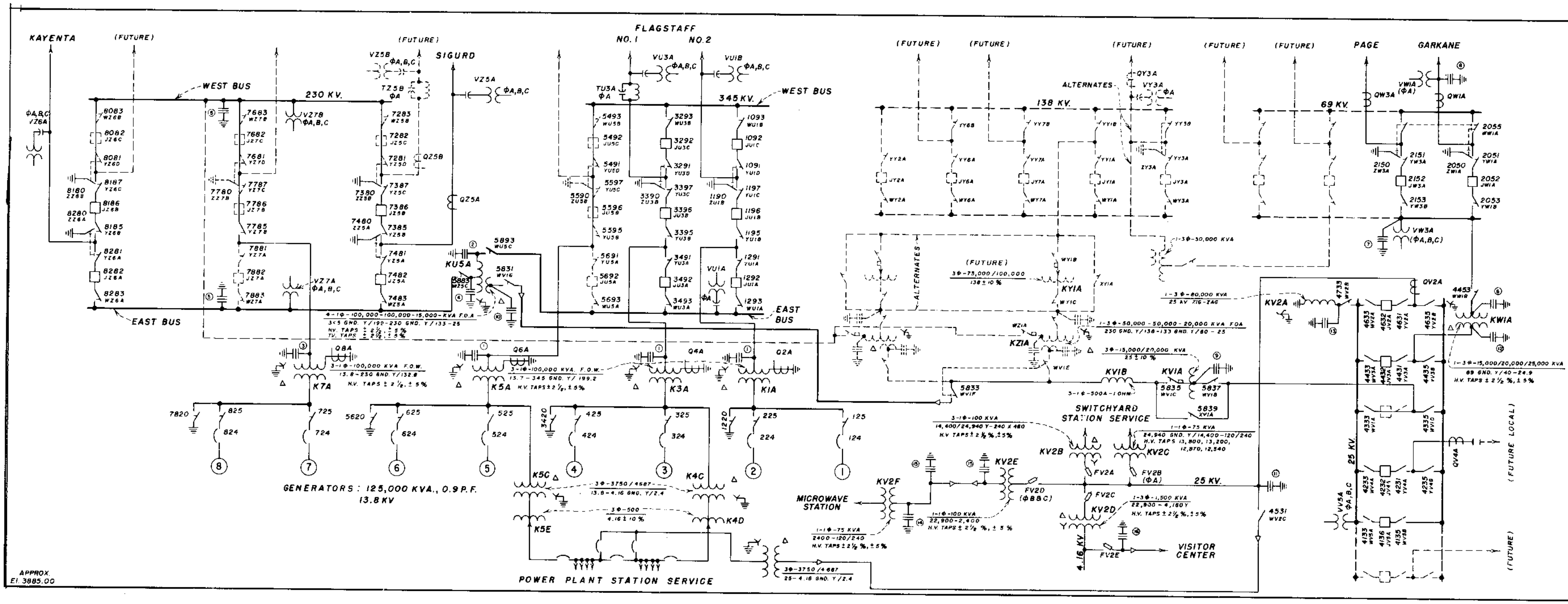


Figure 218.—Glen Canyon Powerplant and Switchyard—Switching diagram. (Sheet 2 of 2.) From drawing No. 557-D-11.

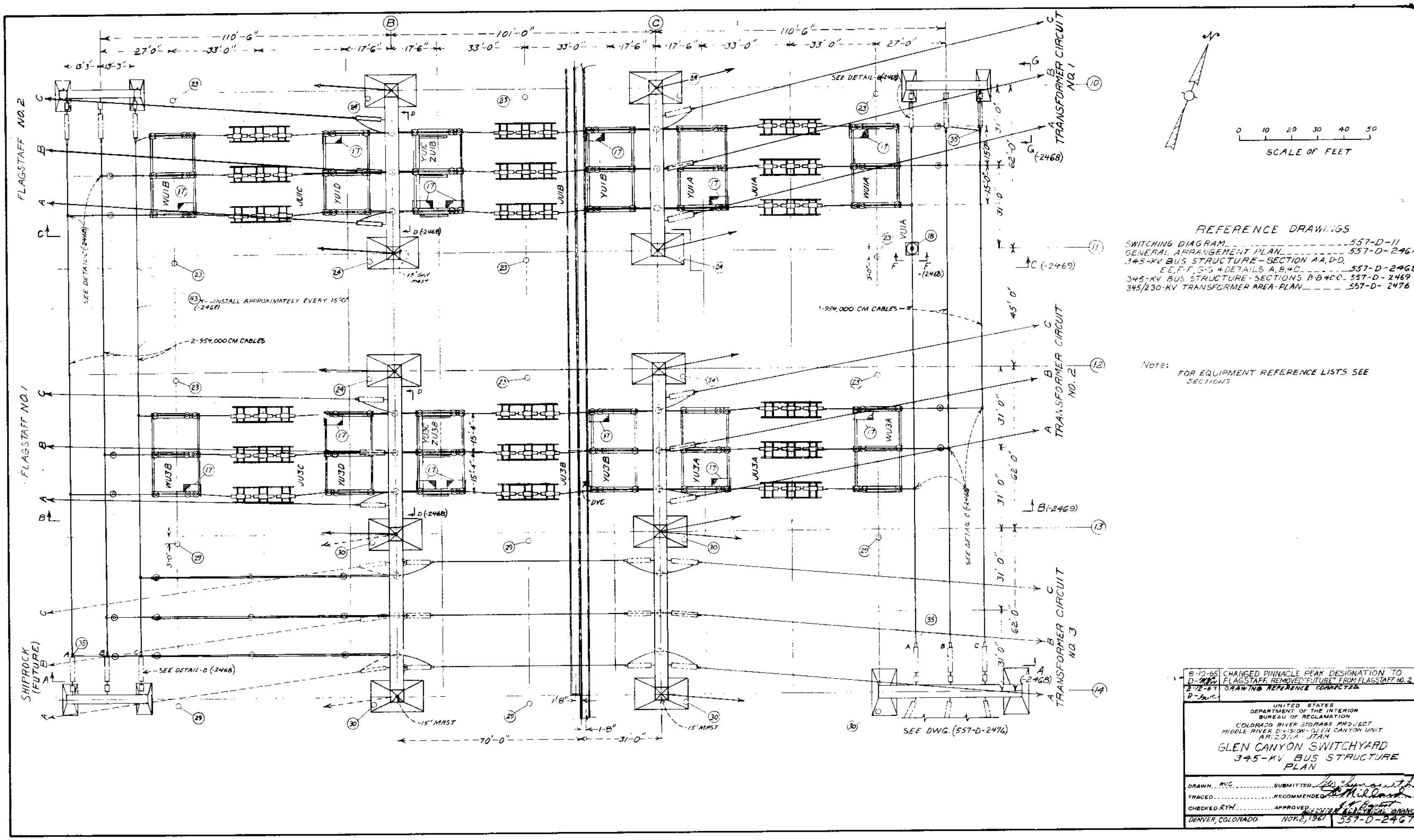


Figure 219.—Switchyard 345-kilovolt bus structure—Plan.

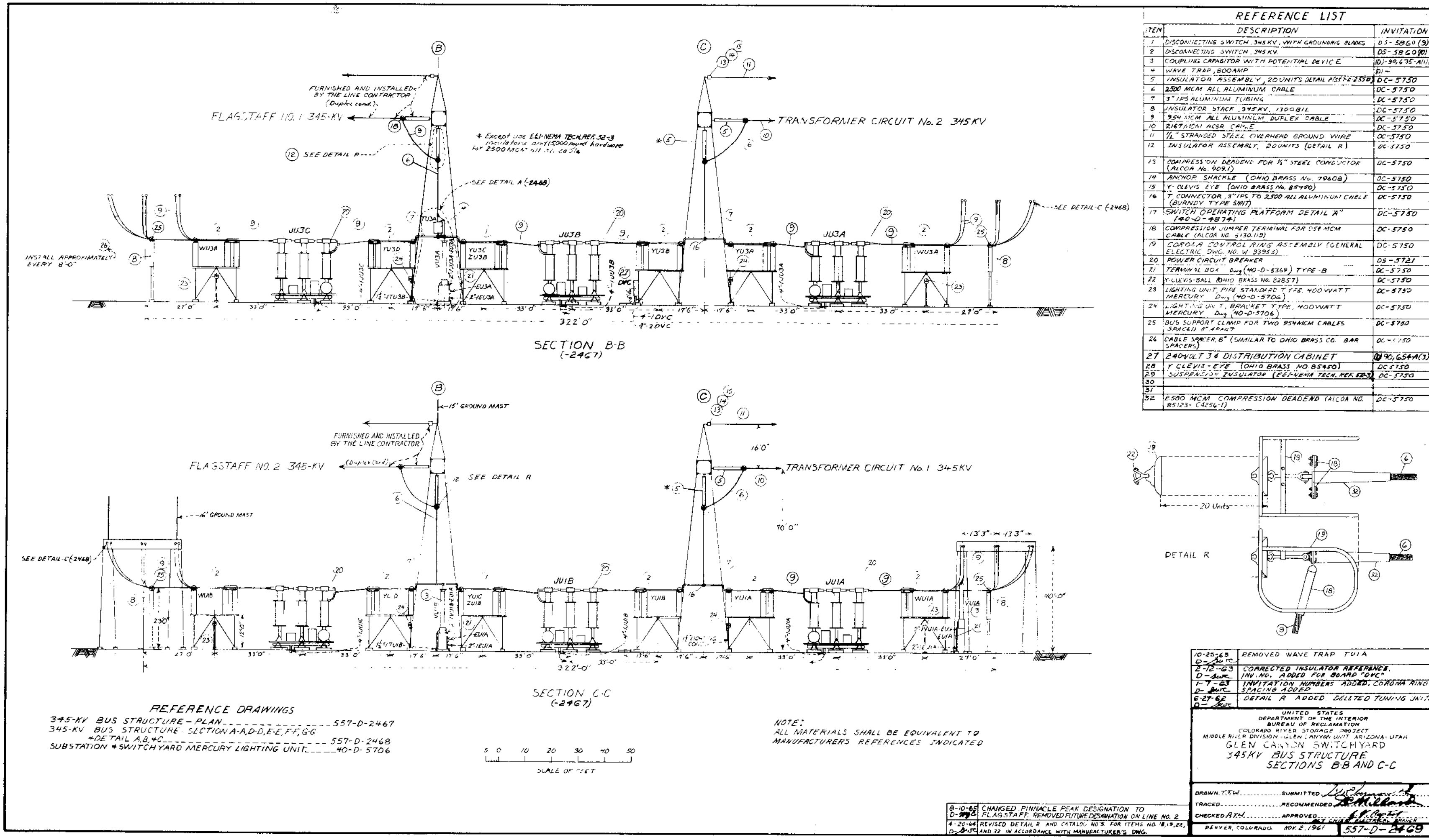


Figure 220.—Switchyard 345-kilovolt bus structure—Sections B-B and C-C.

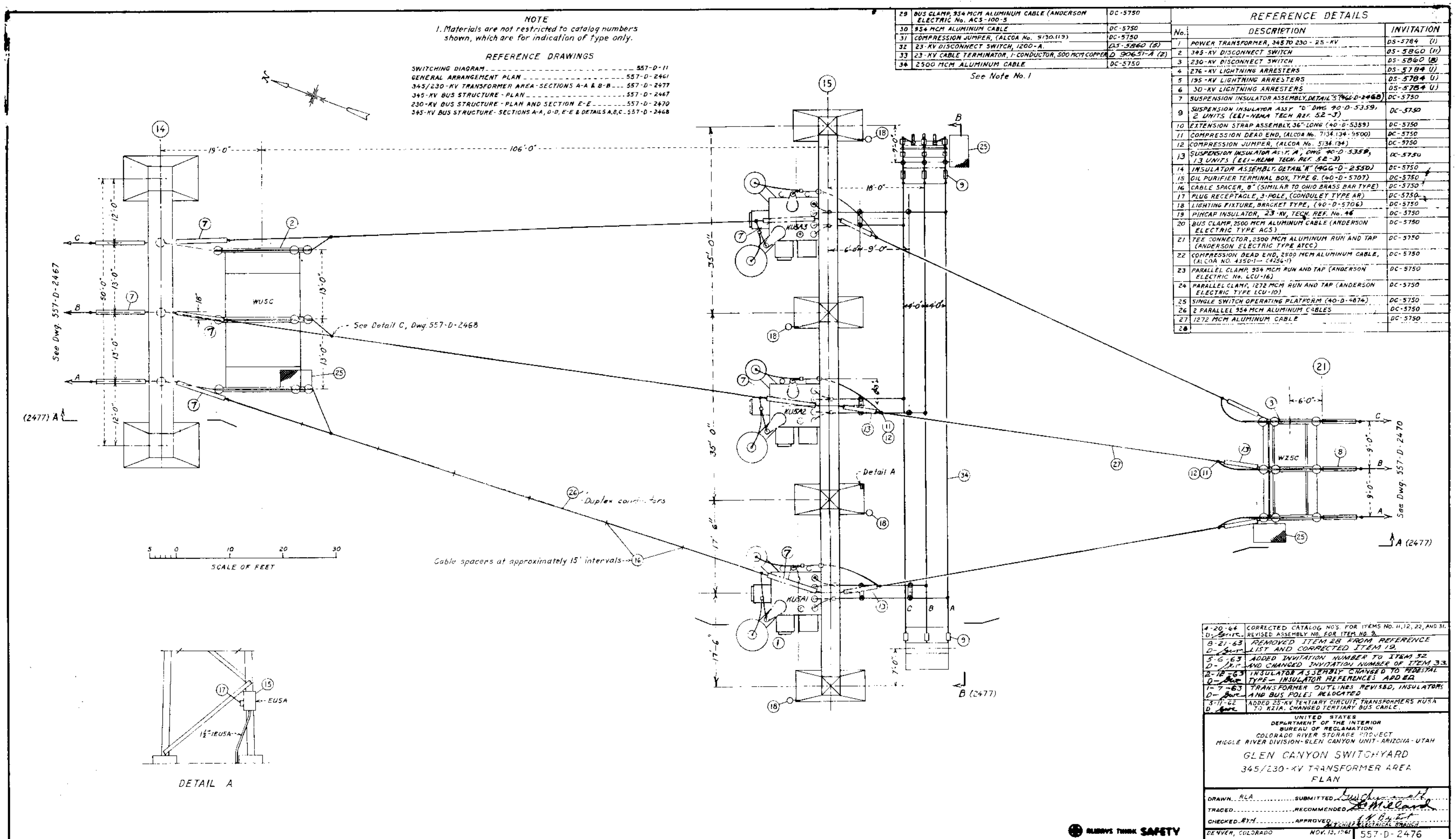


Figure 221.—Switchyard 345/230-kilovolt transformer area—Plan.

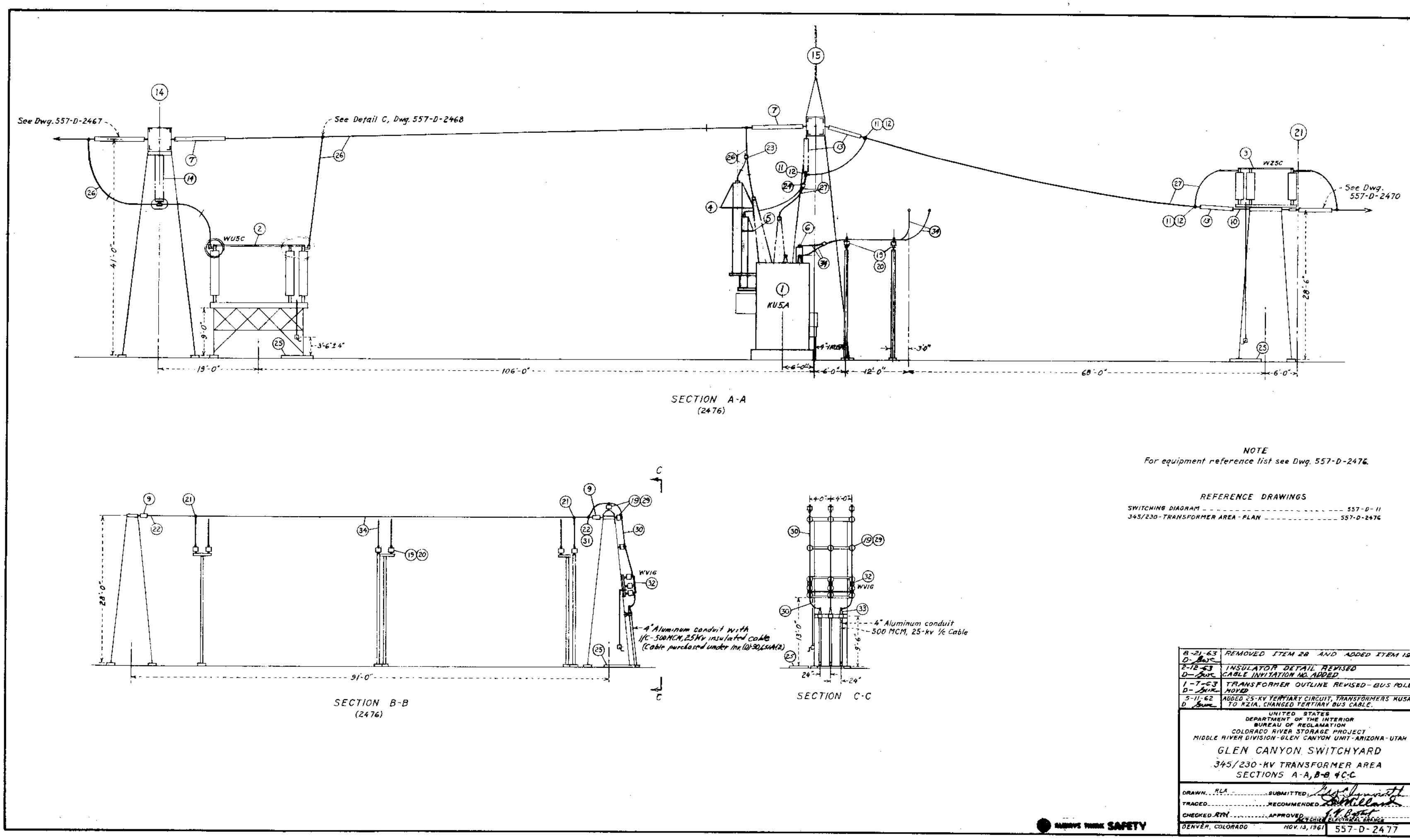


Figure 222.—Switchyard 345/230-kilovolt transformer area—Sections A-A, B-B, and C-C.

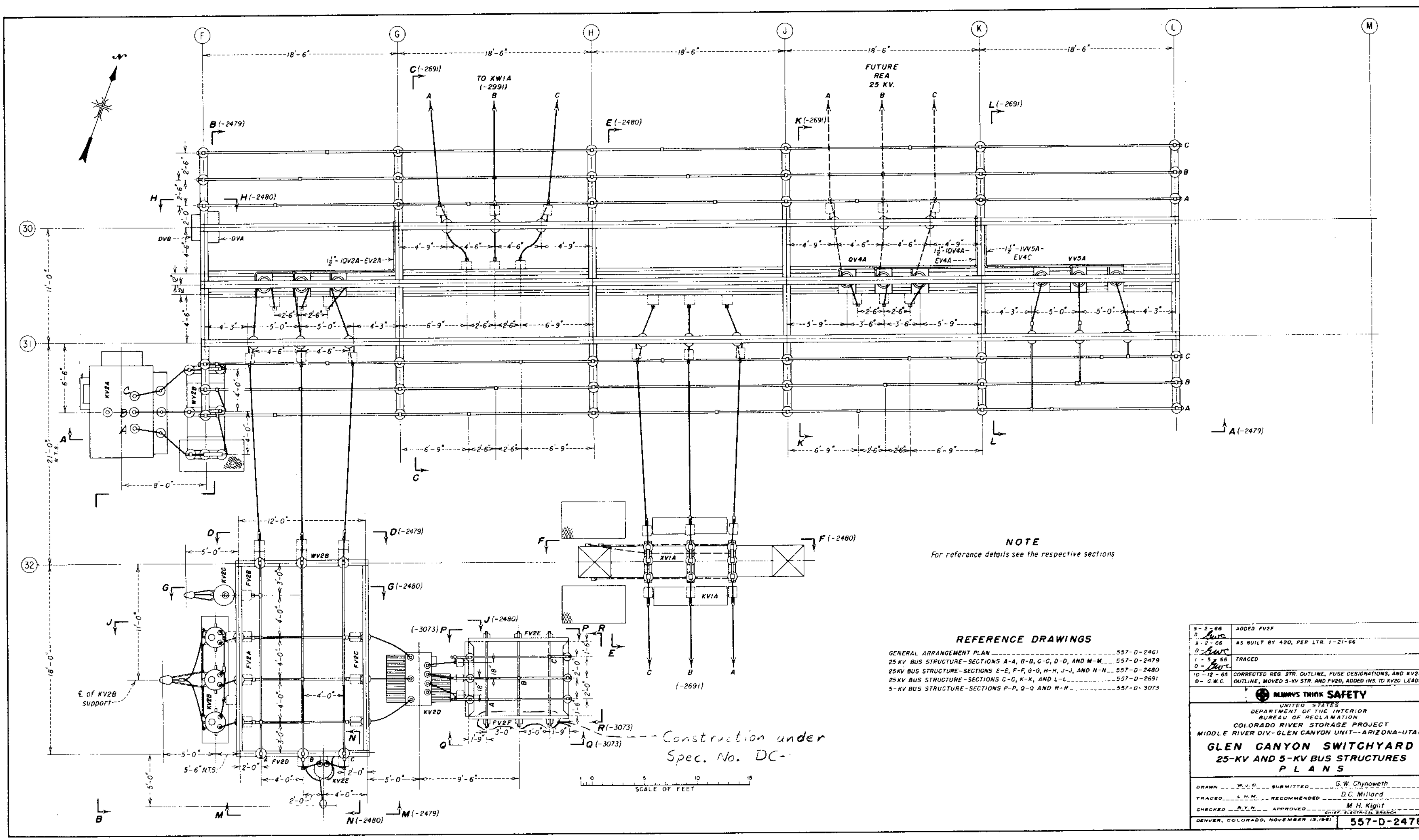


Figure 223.—Switchyard 25- and 5-kilovolt bus structures—Plans.

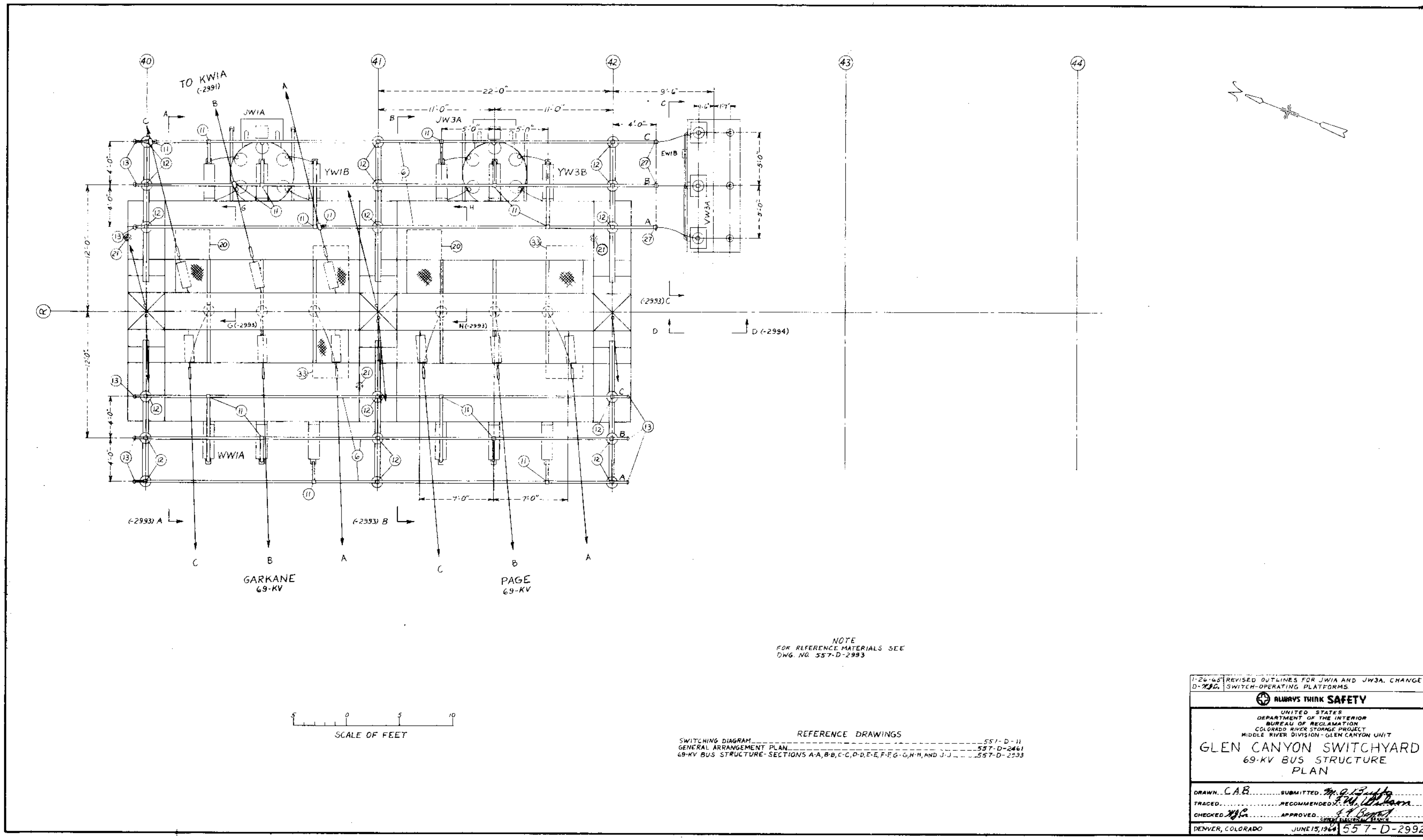


Figure 224.—Switchyard 69-kilovolt bus structure—Plan.

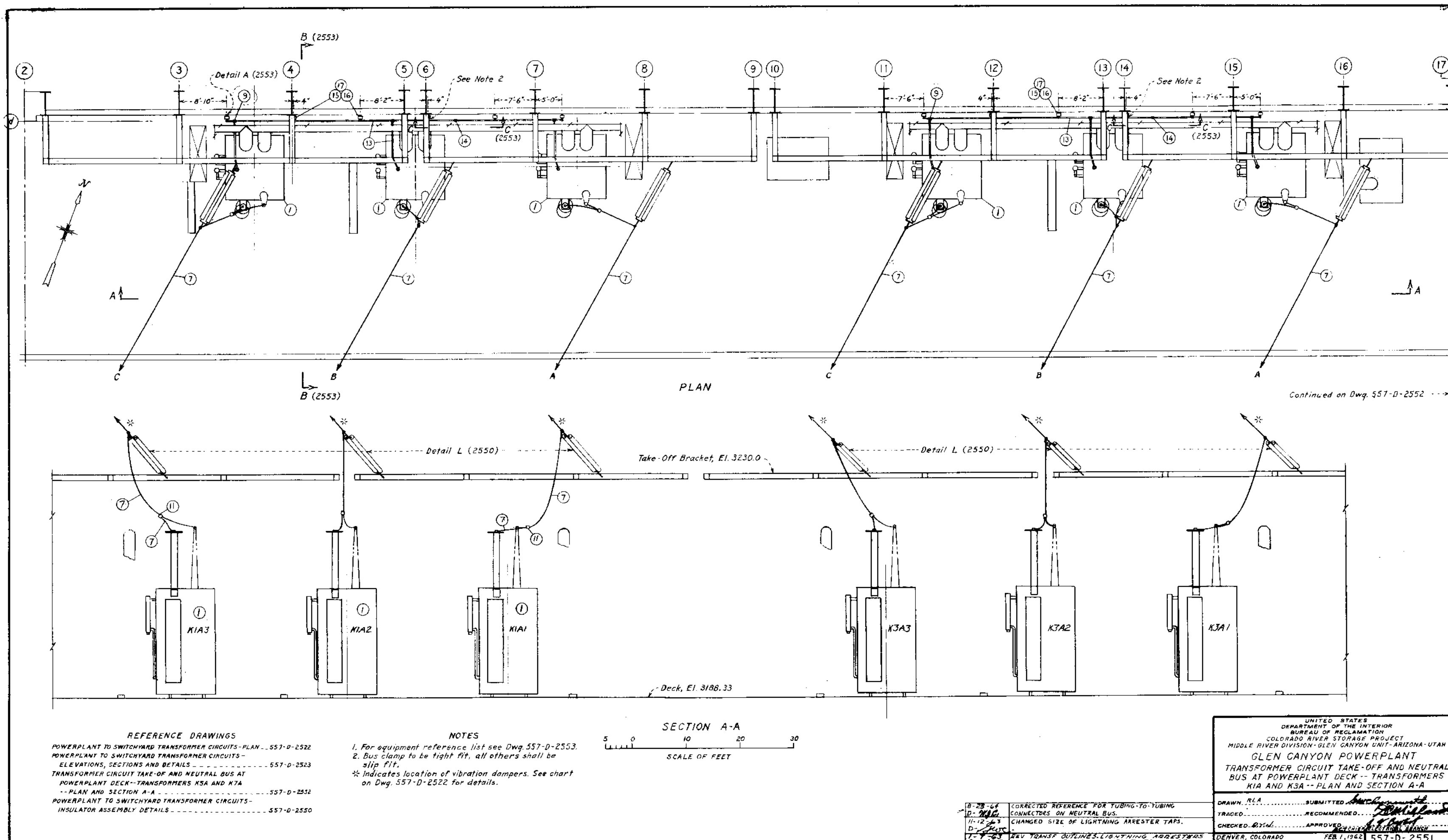


Figure 225.—Transformer circuit takeoff and neutral bus at powerplant deck, transformers K1A and K3A—Plan and section A-A.

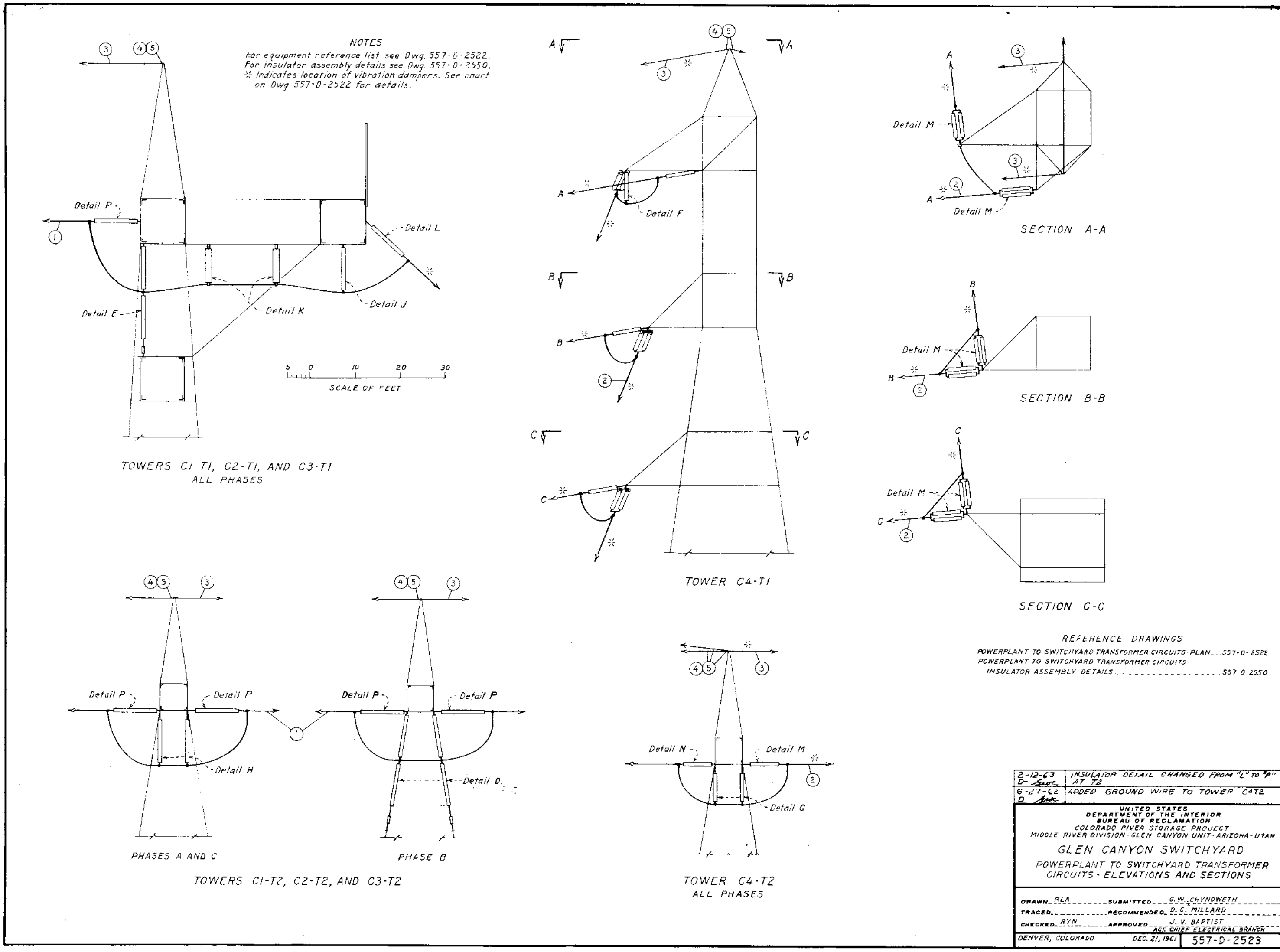


Figure 226.—Powerplant to switchyard transformer circuits—Elevations and sections.

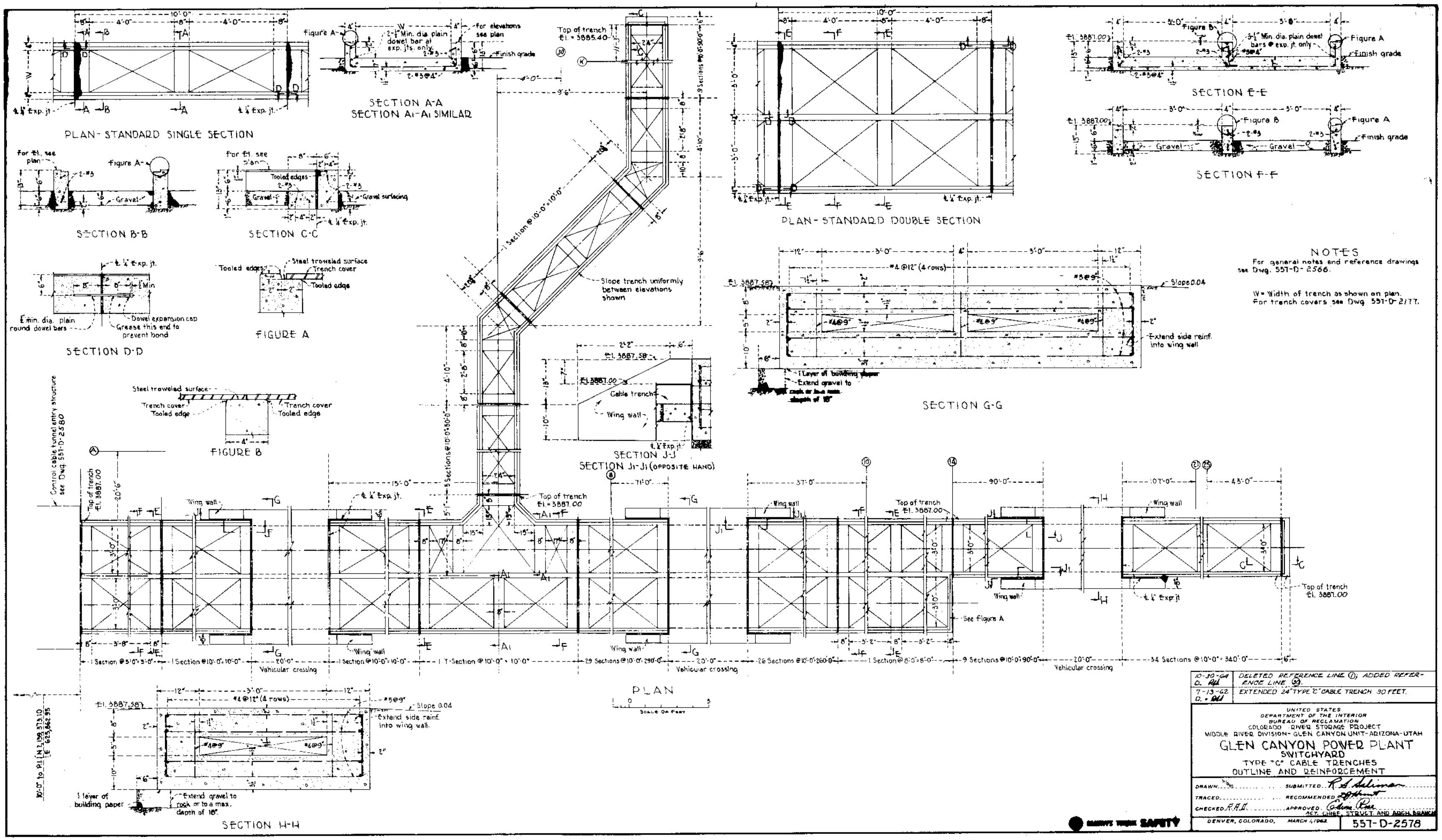


Figure 227.—Switchyard Type C cable trenches—Outline and reinforcement.

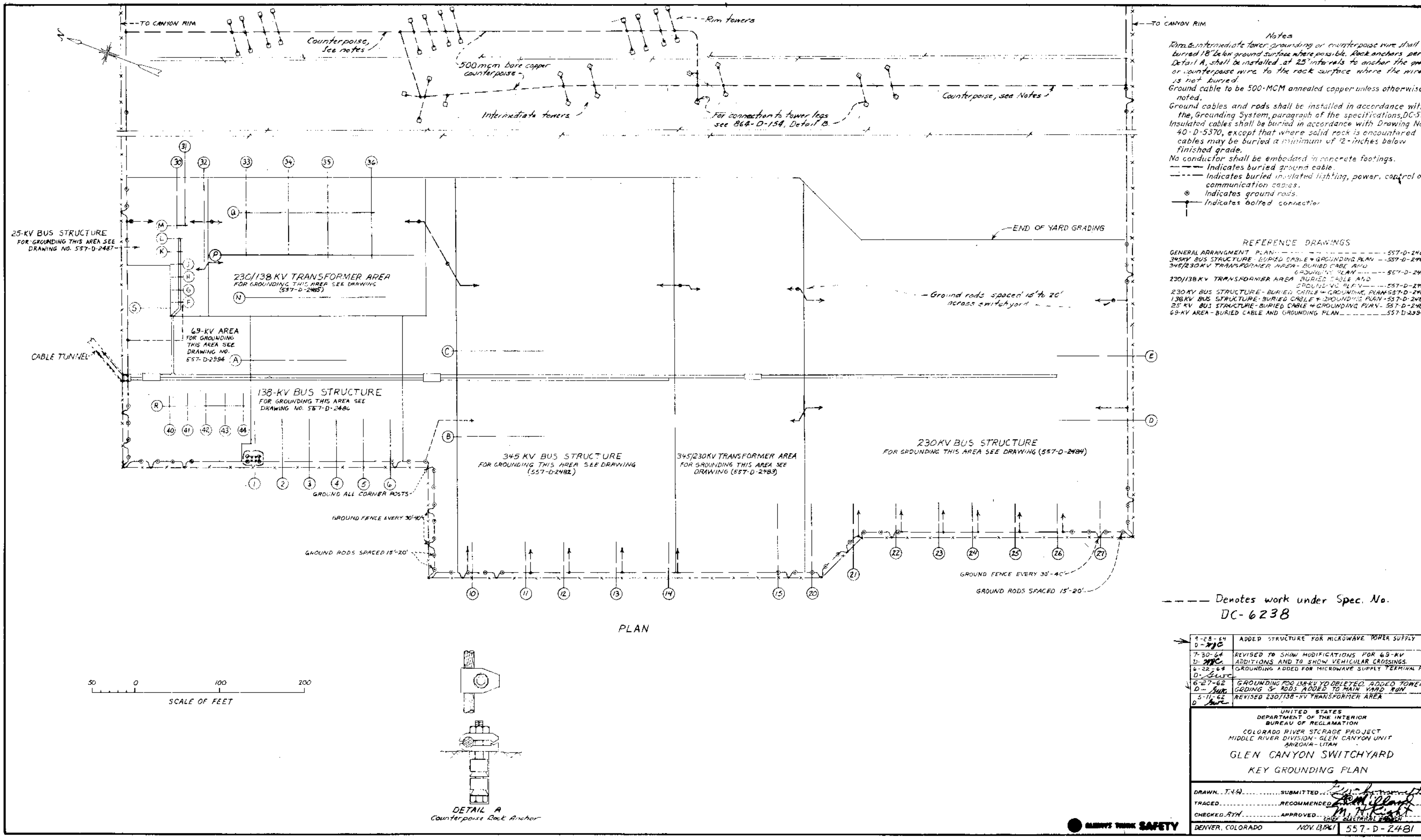


Figure 228.—Switchyard key grounding plan.

115. SERVICE FACILITIES. Since the switchyard is located comparatively near the powerplant, a service building was not provided in the switchyard.

116. SPECIAL PROBLEMS. The major problem in design of the switchyard was the lack of firm commitments for the power generated at Glen Canyon Powerplant at the time the completion contract had to be awarded. The specifications were issued with plans for a 230/138-kilovolt transformer with delivery to the city of Page and future delivery to Garkane Electric Association at 138 kilovolts. This was later modified to have Page supplied through Arizona Public Service Co. with delivery at 25 kilovolts. The 230/138-kilovolt transformer and the 138-kilovolt yard were deleted from initial installation and the 25-kilovolt switchyard enlarged. After the 25-kilovolt switchyard had been installed, both the Arizona Public Service and Garkane Electric Association decided to take delivery at 69 kilovolts. At this stage of construction, the most economical way to accomplish this delivery was to step up power from the 25-kilovolt bus. Later, if a requirement develops, a 230/138-kilovolt transformer with delivery at 138 kilovolts and a 138/69-kilovolt transformer may be provided to carry the additional load.

CHAPTER X. *Design*—CARL HAYDEN VISITOR CENTER

117. GENERAL. The visitor center at Glen Canyon Dam is a joint facility of the Bureau of Reclamation and the National Park Service and serves as a focal point for visitors to the Glen Canyon Dam and National Recreation Area. The center was dedicated by the Secretary of the Interior, Stuart L. Udall, on September 26, 1968.

The center was named the Carl Hayden Visitor Center in honor of the Arizona Senator who retired in 1968 at the end of the 90th Congress after 56 years of service as either a Representative or a Senator. While the name has been changed on the outside of the center and at similar locations, no effort has been made to make this name change on the numerous drawings associated with the center, and the numerous references to the Glen Canyon Visitor Center will continue on most official drawings.

118. ARCHITECTURE. The primary functional considerations for the design of the Carl Hayden Visitor Center included the following provisions:

(1) Central orientation center.

(2) Place of beginning for the visitor tour.

(3) Public restrooms.

(4) Public parking areas.

(5) Concession to dispense sundries and light refreshments.

(6) Space for displays provided by the National Park Service.

(7) View windows and terraces.

The building has a rectangular wing and a rotunda and is surrounded by entrance and view terraces (figs. 229 and 230). There is a part basement under the rectangular portion of the building. By cantilevering the 86-foot-diameter rotunda 10 feet over the canyon rim for about half of its diameter, a spectacular overlook was provided of the dam, powerplant, reservoir, highway bridge, and Glen Canyon and the Colorado River below the dam.

Elevators in the building convey the visitors down through the canyon wall to a rock cut tunnel which leads to the crest of the dam. From this point visitors may walk to the dam elevator which carries them down to the powerplant level where they can take a self-guided tour.

The exterior of the building expresses interior functions through the circular form of the exhibit-observation room, the raised block of the audiovisual room, and the projection of the elevator tower through the roof. Exterior surfaces (figs. 231 and 232) are of exposed aggregate precast concrete panels and columns in tan and white. Interior finishes were selected for appropriateness to their use and to provide long life, low maintenance, and pleasing appearance. An overall coordinated color design is provided through use of colored materials and painting.

Appropriate landscaping with an automatic sprinkler system has been provided in the building and parking areas.

119. SUBSTRUCTURE BUILDING DESIGN. The configuration of the visitor center building has been described in the previous section. The building foundation, basement, and first floor are of reinforced concrete. The reinforced concrete foundation and framing plans are shown on figures 233, 234, and 235. Except for a reinforced concrete elevator shaft and stairwell, the building superstructure is a structural steel frame with precast concrete wall panels.

(a) *Substructure.*—The rotunda was considered to act in two segments for design of the reinforced concrete foundation and frame. The semicircular portion farthest from the canyon rim consists of a circular perimeter foundation wall, intermediate beams and columns, and the circular core wall at the center of the rotunda. The portion adjacent to the canyon rim consists of beams extending radially from the circular core in the center to columns near the canyon rim and 10-foot cantilever beams beyond the columns.

Three radial stub walls in the rotunda were designed to carry the three main structural steel roof girders. Two of these walls extend into rock and the third wall supported from one of the cantilever beams. Structurally, the core wall in the center of the rotunda was the most efficient structural approach due to the many beams framing into a central vertical element. The circular wall also serves as a return air duct for the air-conditioning system.

All walls, pilasters, and columns (except those supporting the rotunda cantilever beams) extend a minimum of 6 inches into unweathered rock. Columns

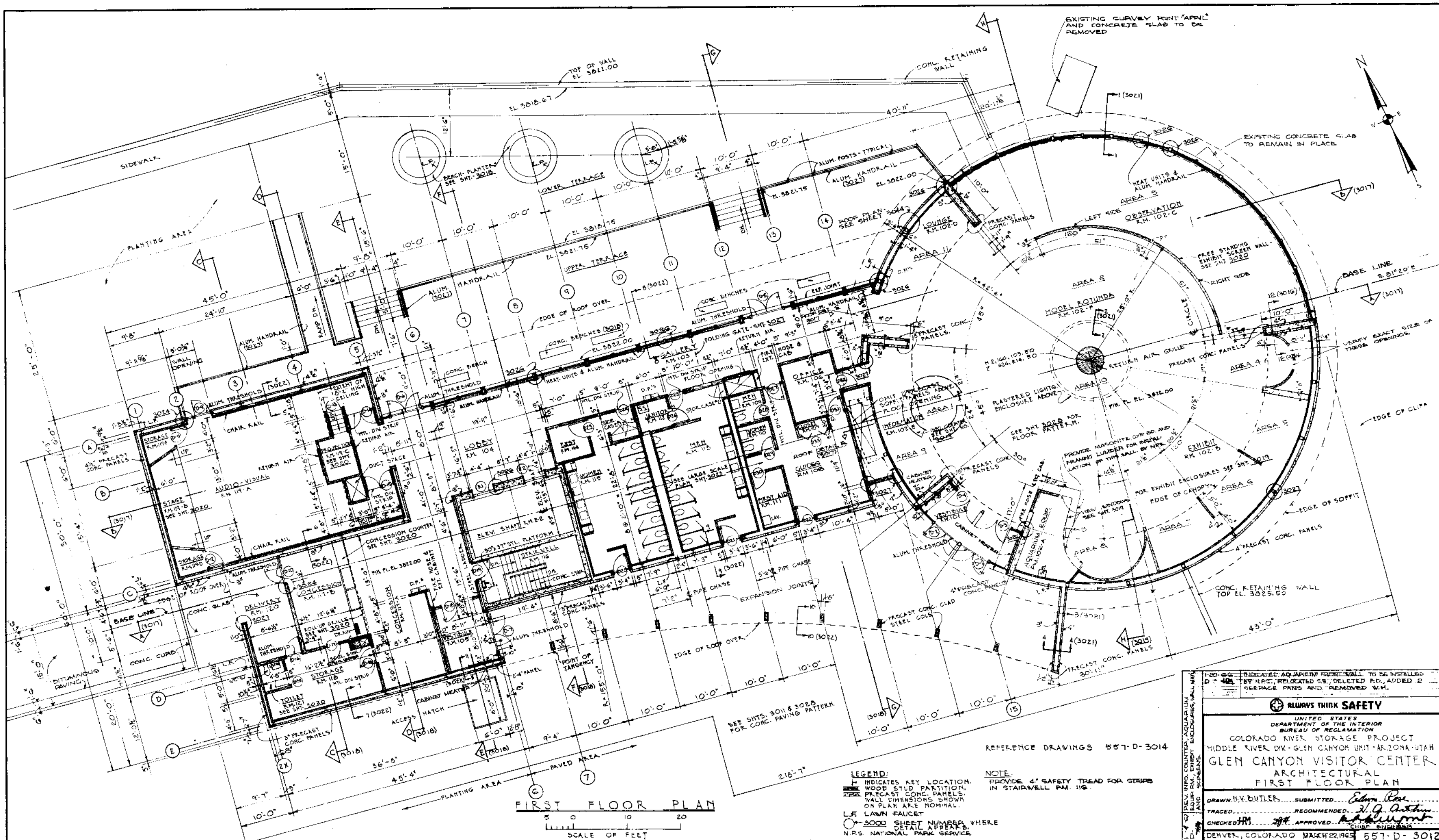


Figure 229.—Visitor center—First floor architectural plan.

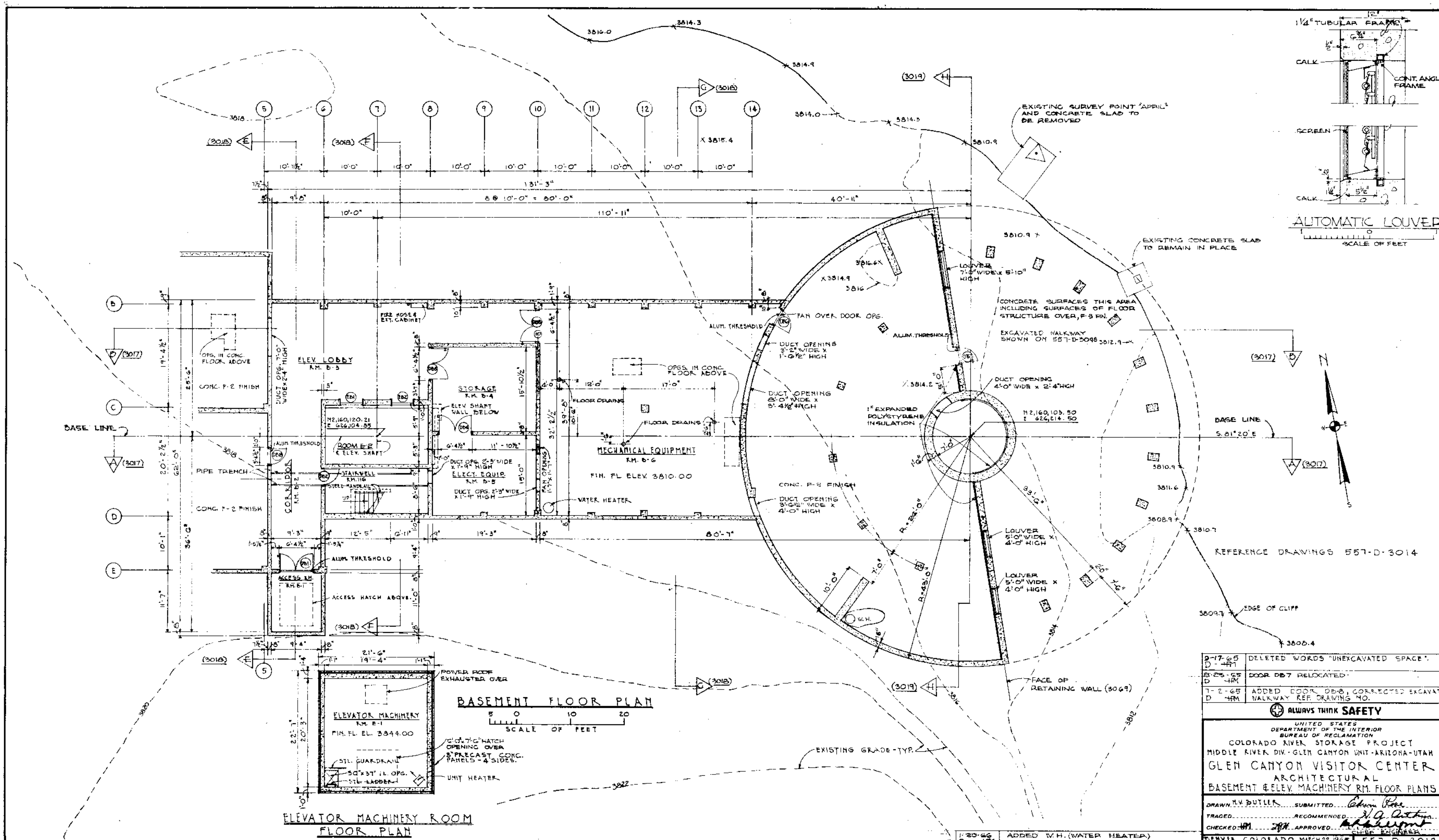


Figure 230.—Visitor center—Basement and machinery room architectural plans.

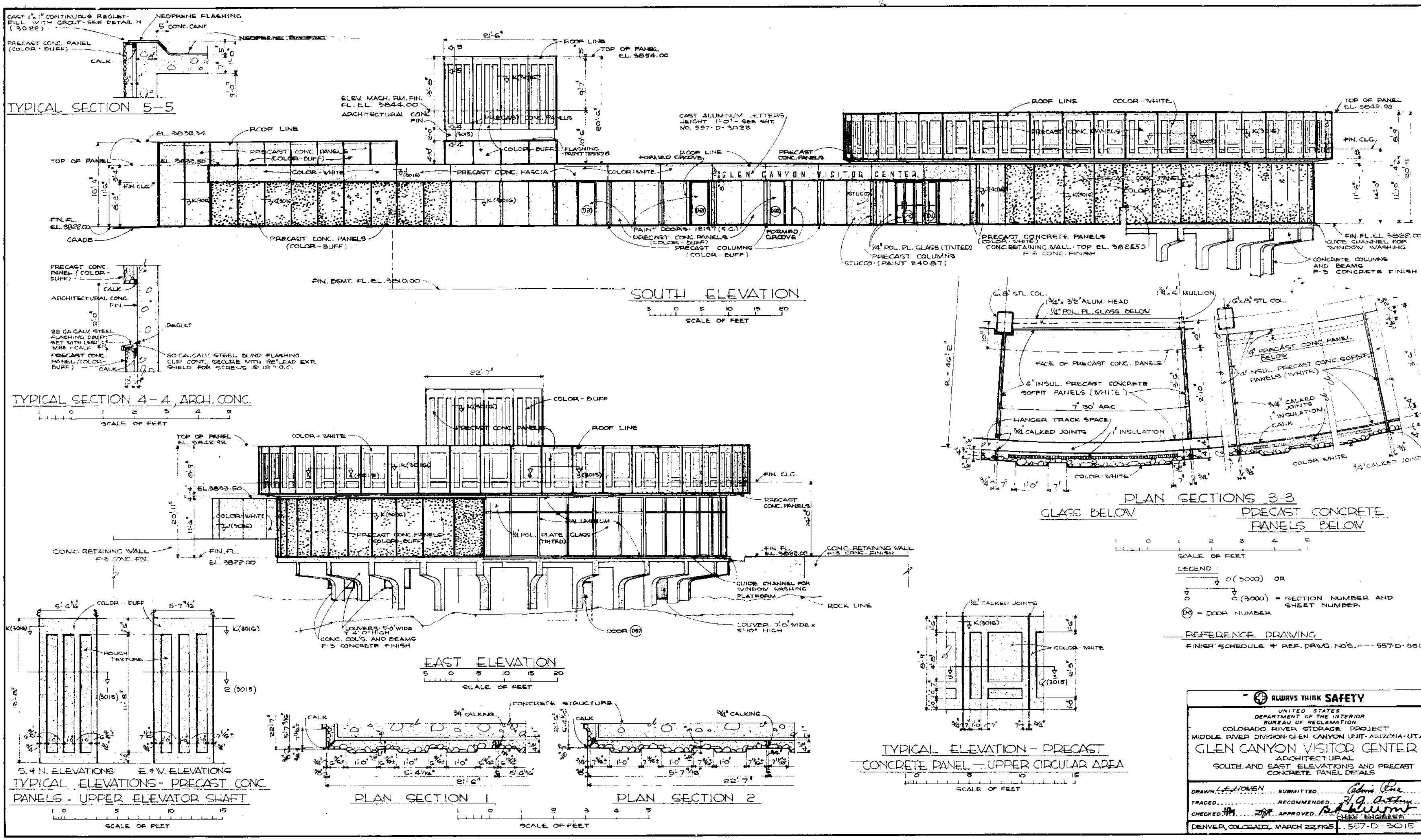


Figure 231.—Visitor center outside architecture—South and east elevations.

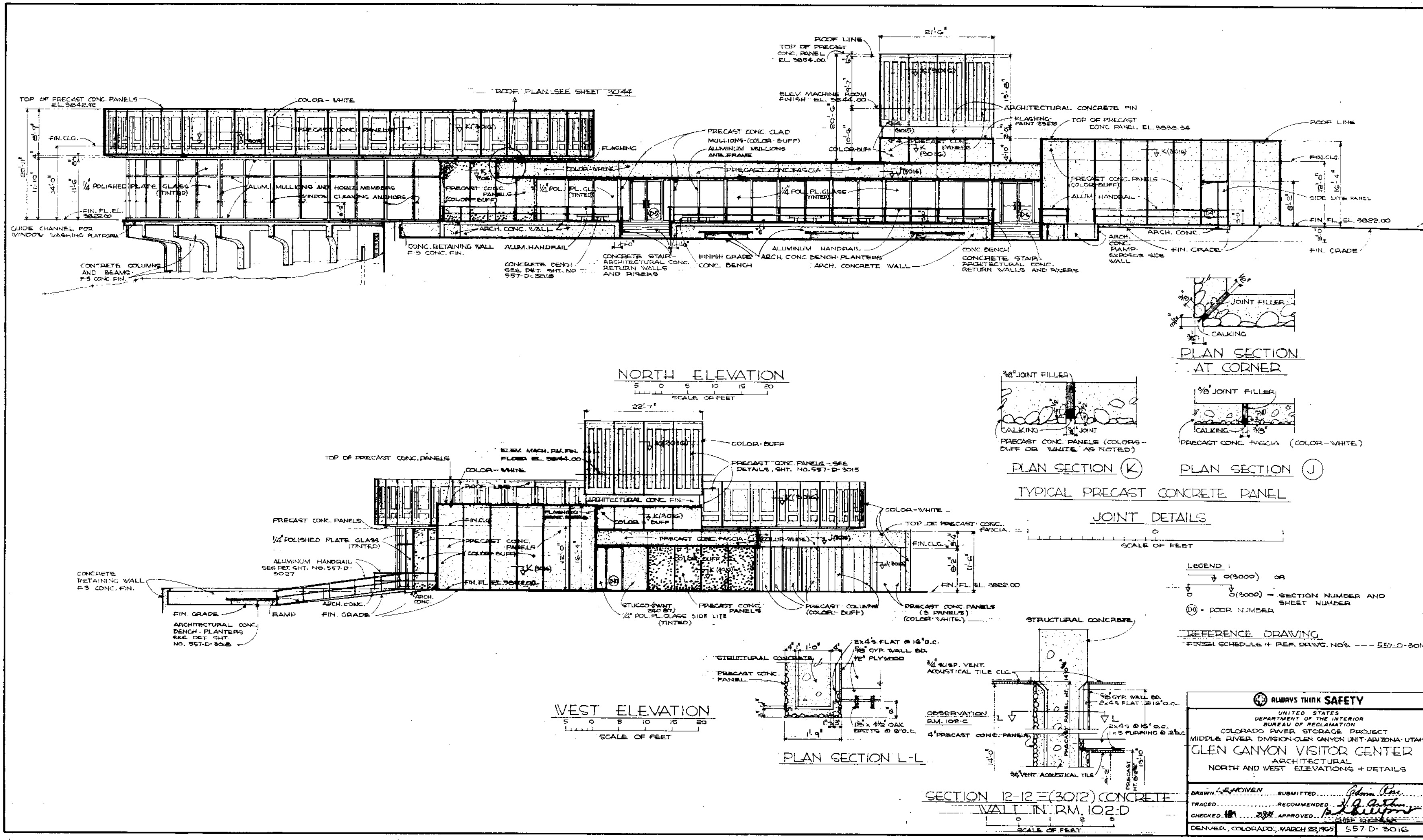


Figure 232.—Visitor center outside artchitecture--North and west elevations.

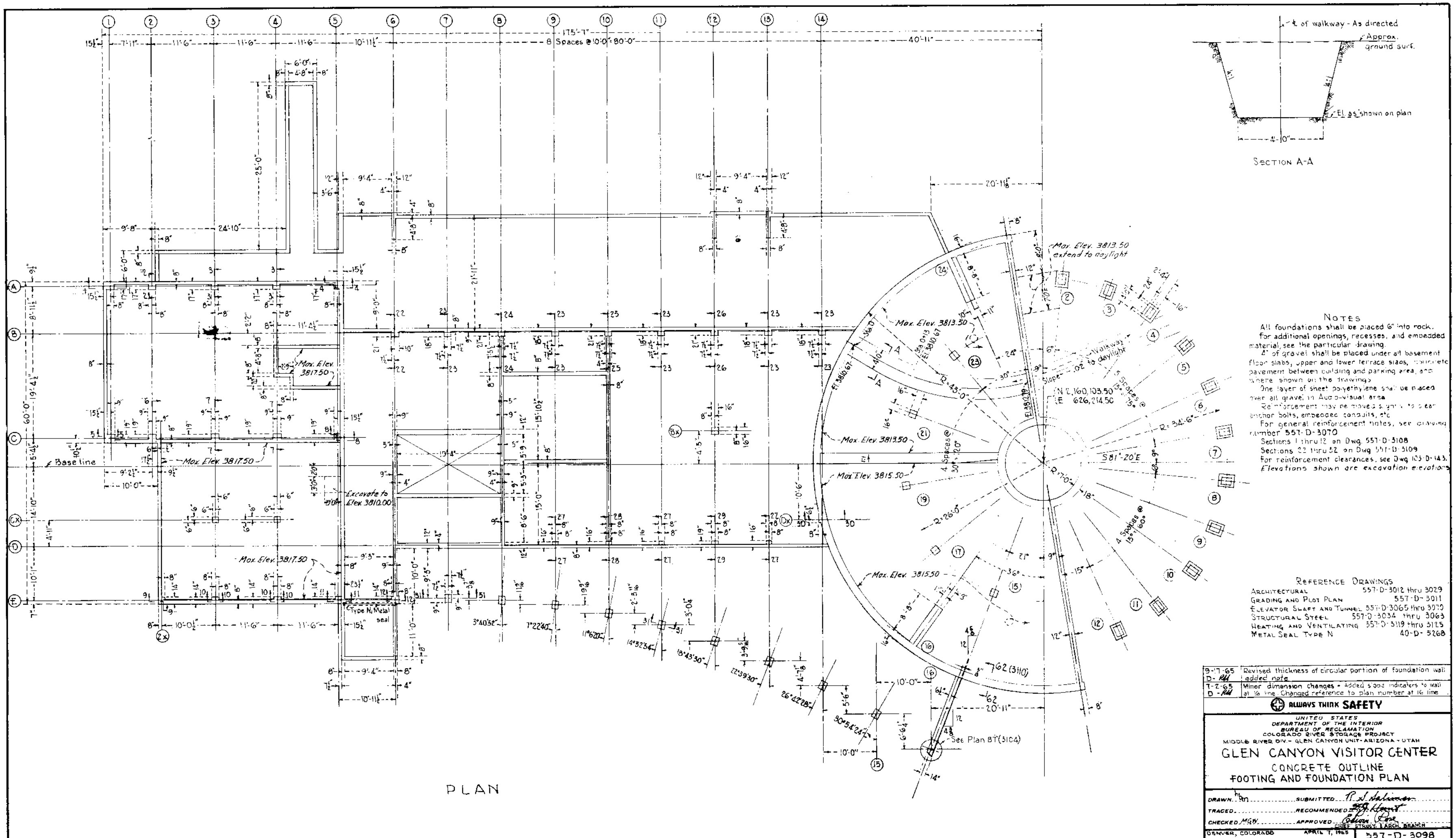


Figure 233.—Visitor center—Concrete outline, footing and foundation plan.

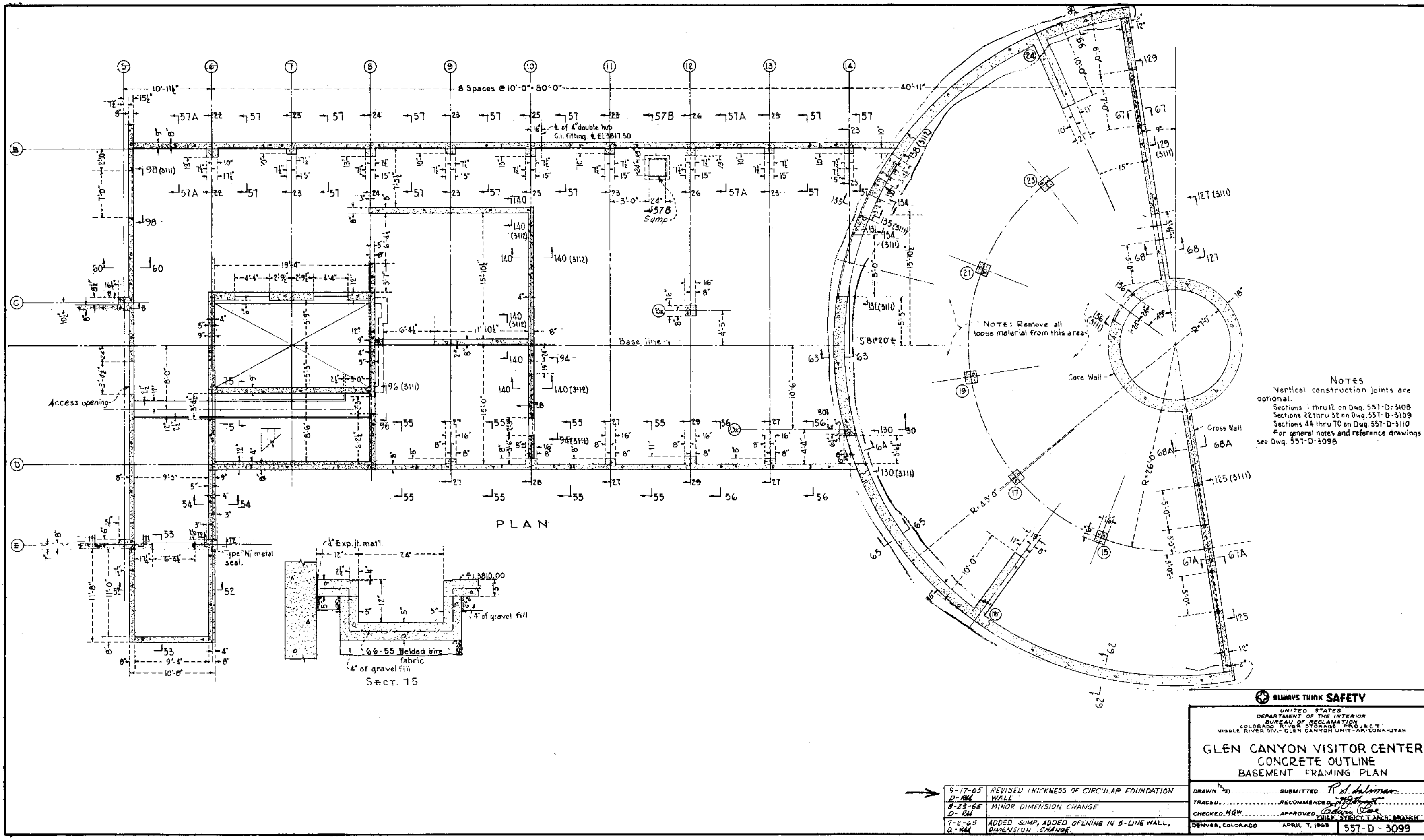


Figure 234.—Visitor center—Concrete outline, basement framing plan.

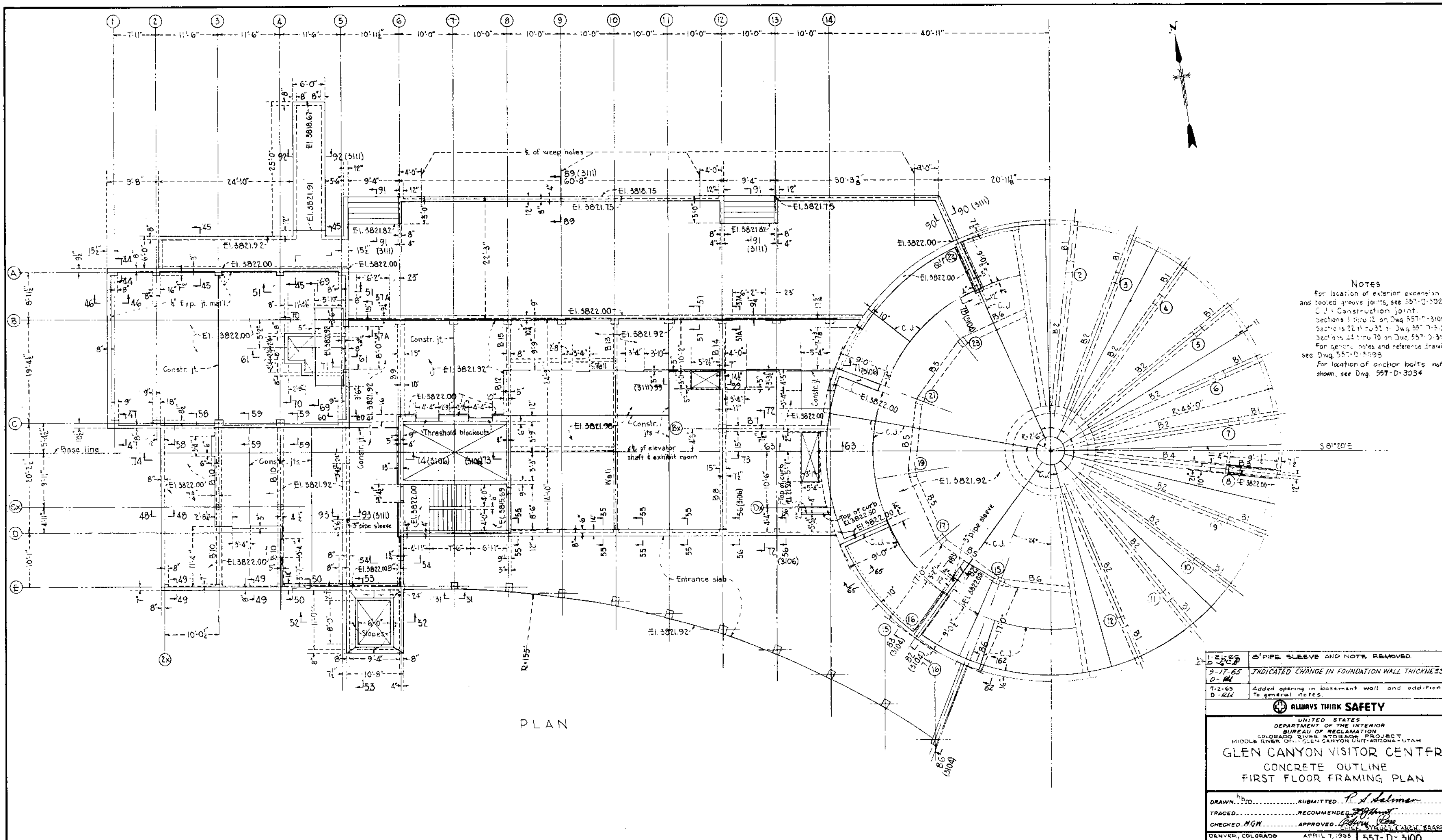


Figure 235.—Visitor center—Concrete outline, first floor framing plan.

supporting the cantilever beams were designed to rest on spread footings. These footings extend into rock a minimum of 12 inches. From each of these footings, four 1-3/8-inch reinforcing bars were embedded into rock a minimum distance of 8 feet for the two bars nearest the canyon rim and a minimum of 15 feet for the other two bars. These bars develop the maximum strength in the bedrock by "stitching" through possible fissures in the rock.

(b) *Structural Floors, Stairwell, and Shaft.*—The design of the structural floor was based on dead loads and the following live loads:

100 p.s.f. for rotunda, corridors and lobby, and concession and storage room.

50 p.s.f. for office, toilet rooms, first aid room, guides room, and stairwell.

Floors in the audiovisual room and in the basement were placed on grade.

The combined stairwell and elevator shaft were designed for dead and live loads, the weight of the elevator equipment, that portion of the room framing into the shaft, and seismic loadings.

(c) *Design Codes and Data.*—The following data and codes were used in designing the visitor center substructure:

(1) "Reinforced Concrete Design Data," Engineering Monograph No. 10, Bureau of Reclamation.

(2) "Moments and Reactions for Rectangular Plates," Engineering Monograph No. 27, Bureau of Reclamation.

(3) "Design Standards No. 9, Buildings," Bureau of Reclamation.

(4) "Design Standards No. 11, Housing and Community Facilities," Bureau of Reclamation.

(5) "Building Code Requirements for Reinforced Concrete," American Concrete Institute (ACI 318-56).

(6) "Influence Lines for Horizontally Curved Fixed-End Beams of Circular-Arc Plan," University of Missouri, Bulletin No. 35.

(d) *Allowable Unit Stresses.*—The visitor center substructure was designed for concrete having an ultimate compressive strength of 3,000 pounds per square inch at 28 days, and an allowable working stress in the reinforcement of 20,000 pounds per square inch in flexure and 16,000 pounds per square inch in web reinforcement.

120. SUPERSTRUCTURE BUILDING DESIGN. The building is a steel frame structure of welded and bolted construction, having in plan a 133- by 60-foot auditorium and utility wing, and an attached 92-foot-diameter rotunda (this diameter includes the exterior concrete panels). The auditorium and utility wing is a beam and column semirigid frame structure designed for vertical and lateral loads as noted below. Three intersecting welded plate roof girders, with clear spans of 81 feet 6 inches, form the main framework for the rotunda and were designed to carry all the vertical loads. The lateral loads are carried through a horizontal truss system into the three large concrete piers.

(a) *Loads.*—The auditorium and utility wing was designed for a roof dead load of 20 pounds per square foot and a roof live load of 30 pounds per square foot. The rotunda was designed for a live load of 35 pounds per square foot and a dead load of 35 pounds per square foot plus the weight of the overhanging precast concrete panels. Windloads used in design were a compression load of 20 pounds per square foot plus a tension of 10 pounds per square foot acting on exposed vertical surfaces; earthquake forces are 10 percent of gravity loads.

A 3-inch tongue-and-groove wood deck secured by nails and bolts to the supporting steel beams supports the roof loads and acts as a diaphragm for support and distribution of lateral loads.

(b) *Outside Platform.*—A platform which rides on an upper and lower track on the outside of the rotunda is provided for cleaning and servicing the windows overlooking the canyon rim. The designs provided that the wheels be inspected and maintained in free-running order before the platform is put to use, and that the tracks should be kept free of dirt and debris.

(c) *Codes.*—The code of the American Institute of Steel Construction Manual was used in the design of members and connections.

121. AIR-CONDITIONING SYSTEM. The air-conditioning system is a three-zone, constant-volume, variable-temperature, all-air system with supplemental electric convection heaters installed under windows and in vestibules to prevent cold drafts.

The central system consists of air-handling units with water-cooling coils, electric heaters, filters, outdoor and return air connections, electric reheaters, supply air ducts, outlets, ventilating ceiling, fans, and hermetic refrigeration machine. Air conditioning of the visitor center was provided for comfort and protection of personnel and visitors, the distribution and removal of heat, the relief of dampness, and the disposal of contaminated air.

The heating load design temperatures were an outside ambient temperature of 0° F. and an inside temperature of 72° F. The cooling load design conditions were an outside ambient temperature of 95° F. dry bulb and 65°F. wet bulb, and an inside temperature of 75° F. The heating and cooling loads were calculated in accordance with the methods outlined by the American Society of Heating, Refrigeration, and Air-Conditioning Engineers.[1]

The system is designed to automatically provide required ventilation, remove contaminated air, and maintain nominal room temperatures of 75°F. for cooling and 72°F. for heating.

122. TOP OF DAM AND ENTRANCE STRUCTURE. The entrance structure is located in block 26 at the top of the dam. A concrete slab was placed on top of the existing top of dam and parapets were placed with the slab. Epoxy-bonded concrete was used to connect the walkway slab to the existing walkways and top of dam. The entrance structure consists of canopy and roof for sheltered entrance to the tunnel, and end-of-dam parapets. A 4- by 4-foot shaft was provided to connect the pipe chase to the existing service adit in the top of the dam. The exposed canyon wall around the portal entrance was rock bolted prior to tunnel excavation. One row of 3/4-inch expansion-type anchor bolts at 5-foot centers was used to stabilize the rock and to prevent slabbing. The details and reinforcement of entrance structure are shown on figures 236 and 237.

(a) *Structural Design*.—The design was based on concrete having a compressive strength of 3,000 pounds per square inch at 28 days. The allowable working stresses are shown on figure 71.

Canopy and roof were designed for a live load of 40 pounds per square inch. Sidewalks and slabs were reinforced to control cracking——5/8-inch bars at 9-inch spacing each way were used. Parapets were designed for a load of 150 pounds per lineal foot acting at the top of the parapet. Horizontal temperature reinforcement (5/8-inch) bars at 6-inch centers) was used.

123. ELEVATOR SHAFT AND TUNNEL. An 8- by 8-foot 6-inch lined tunnel from the end of the dam in block 26 and a 10-foot 6-inch by 20-foot 4-inch lined elevator shaft connect the top of the dam with the visitor center complex. A pipe chase below the floor of the tunnel was provided to carry utility lines for the visitor center. Minimum thickness of lining for the tunnel and shaft was 9 inches. The details and reinforcement of elevator shaft and tunnel are shown on figures 238 and 239.

(a) *Structural Design*.—The design was based on concrete having a compressive strength of 3,000 pounds per square inch at 28 days. The allowable working stresses are shown on figure 71.

The elevator shaft was reinforced using 3/4-inch bars at 12-inch centers horizontally and 3/4-inch bars at 24-inch center vertically. One and one-eighth-inch anchor bars were installed on the centerline of the shaft on each side to stabilize the concrete lining. The anchor bars were spaced at 5-foot centers and embedded 8 feet into the rock. Three rows of 1-inch rock bolts spaced at 5-foot centers were installed at the junction of the elevator shaft and tunnel lobby. The rock bolts were embedded 10 feet into the rock.

The roof of the tunnel was designed as a simply supported beam with a 3-foot rock load. The tunnel was reinforced using 3/4-inch bars at 12-inch centers (transverse) and 5/8-inch bars at 18-inch centers (longitudinal). Rock bolts were installed in the roof of the tunnel as required.

124. RETAINING WALLS. The retaining walls were designed as gravity structures and located to conform to the existing rock surface along the canyon side of the visitor center. The base of each wall has a maximum slope of 5 to 1 with a 24-inch minimum bench. A 5-foot-wide walkway and a parapet were provided at the top of the retaining walls. The sidewalk has a minimum thickness of 18 inches. The parapet is 12 inches wide and 3 feet 4 inches high. The details of the retaining walls are shown on figure 240.

(a) *Structural Design*.—The design was based on concrete having a compressive strength of 3,000 pounds per square inch at 28 days. The allowable working stresses are shown on figure 71.

[1] "ASHRAE Guide and Data Book——Fundamentals and Equipment," American Society of Heating, Refrigerating, and Air-Conditioning Engineers, 1961.

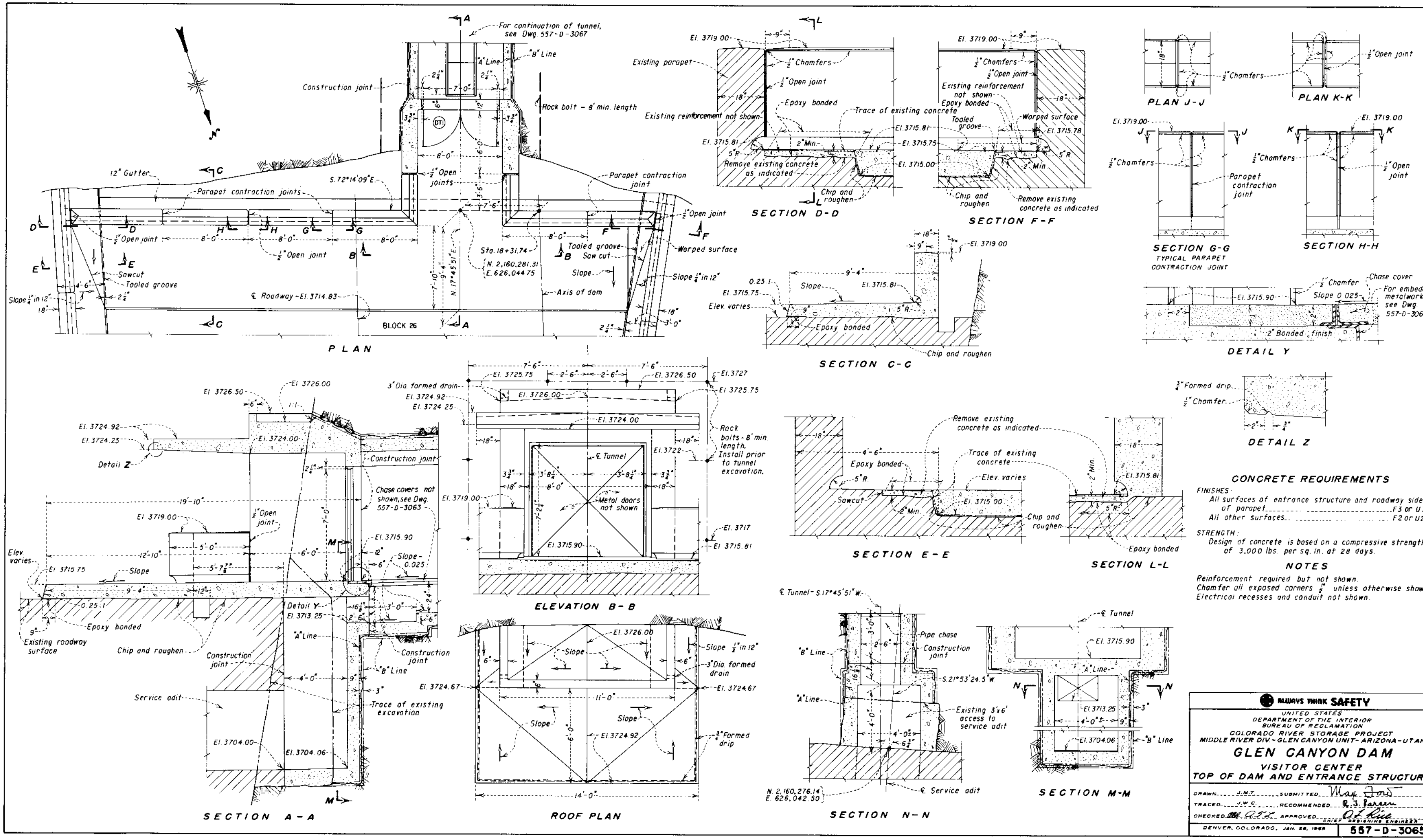


Figure 236.—Visitor center—Top of dam and entrance structure.

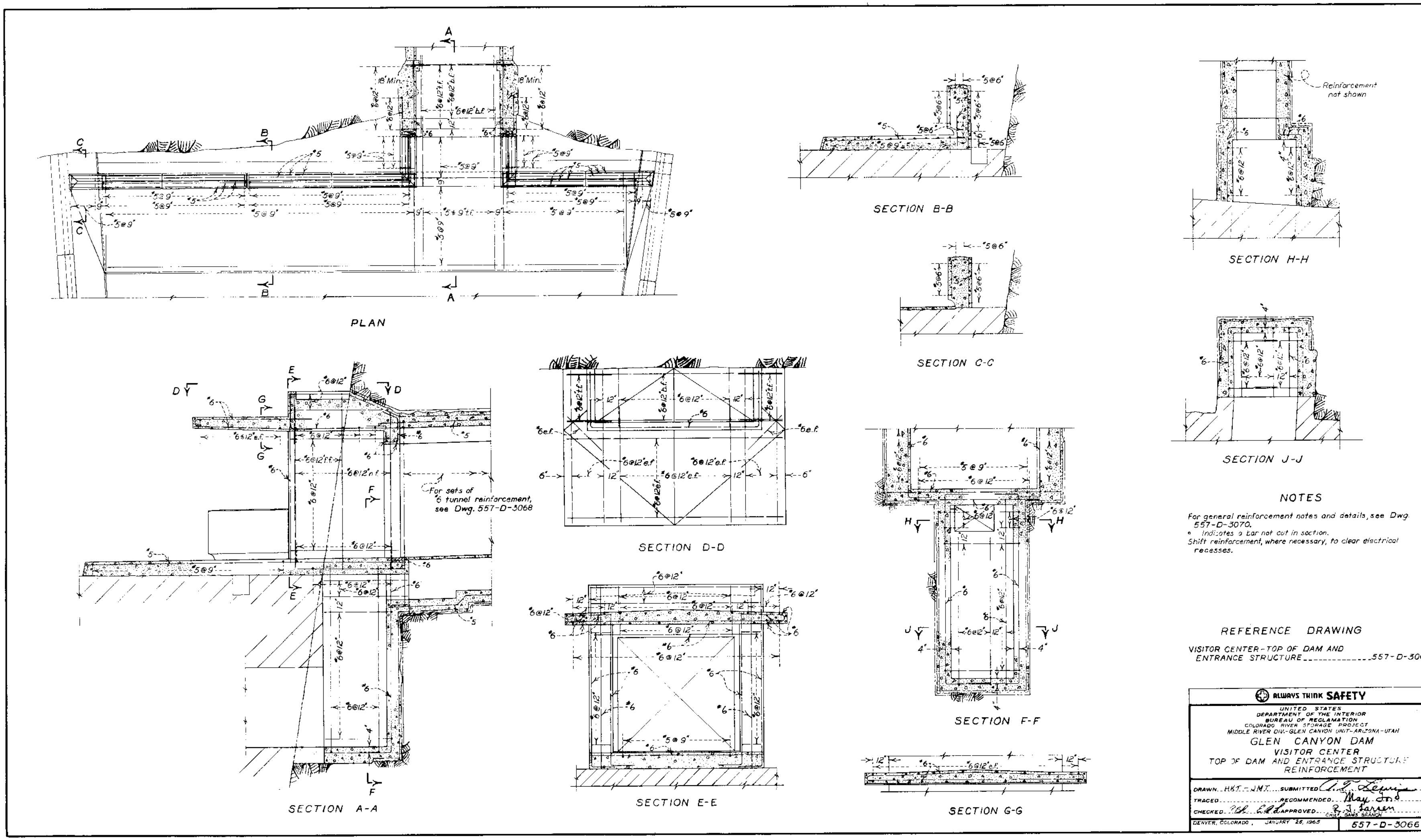


Figure 237.—Visitor center—Reinforcement for top of dam and entrance structure.

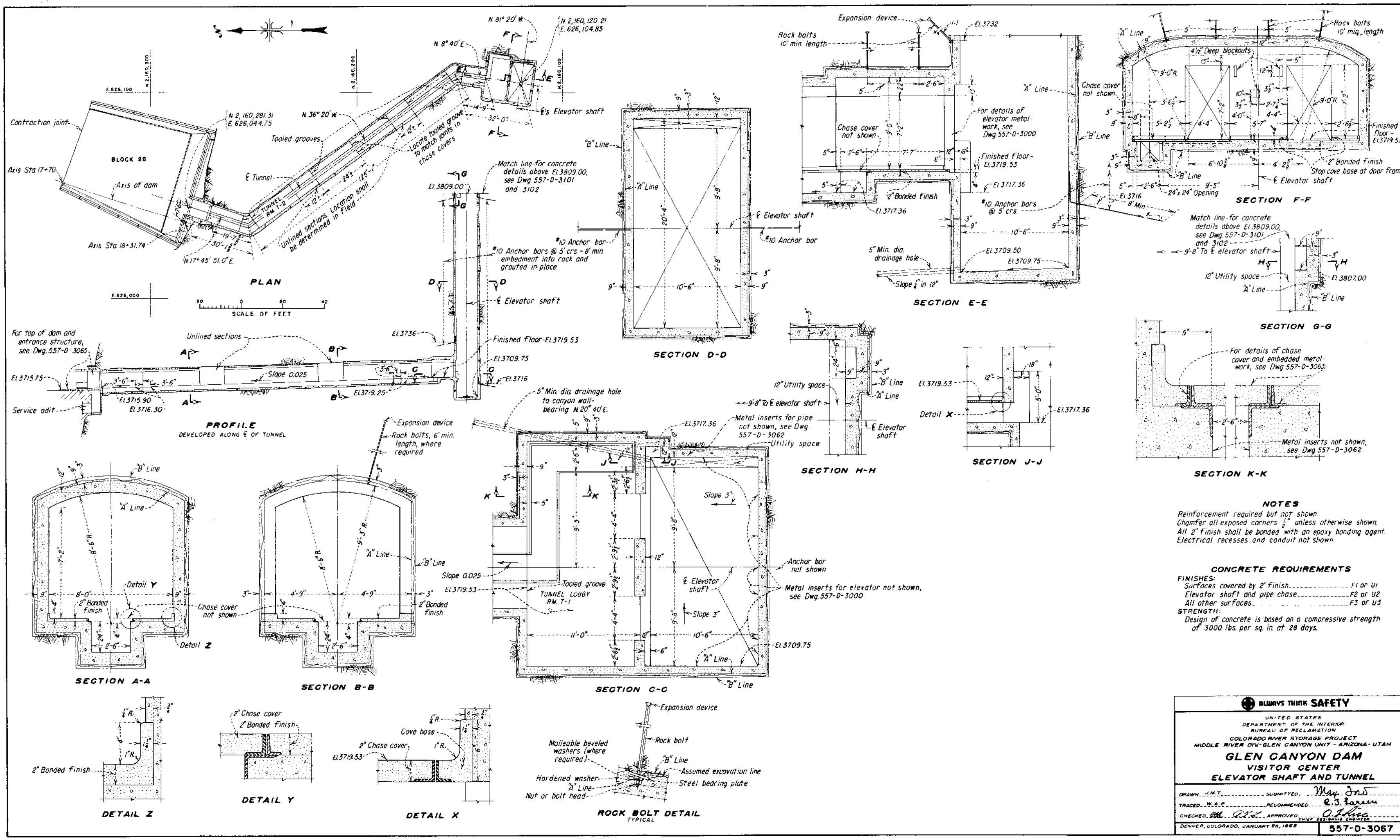


Figure 238.—Visitor center—Elevator shaft and tunnel.

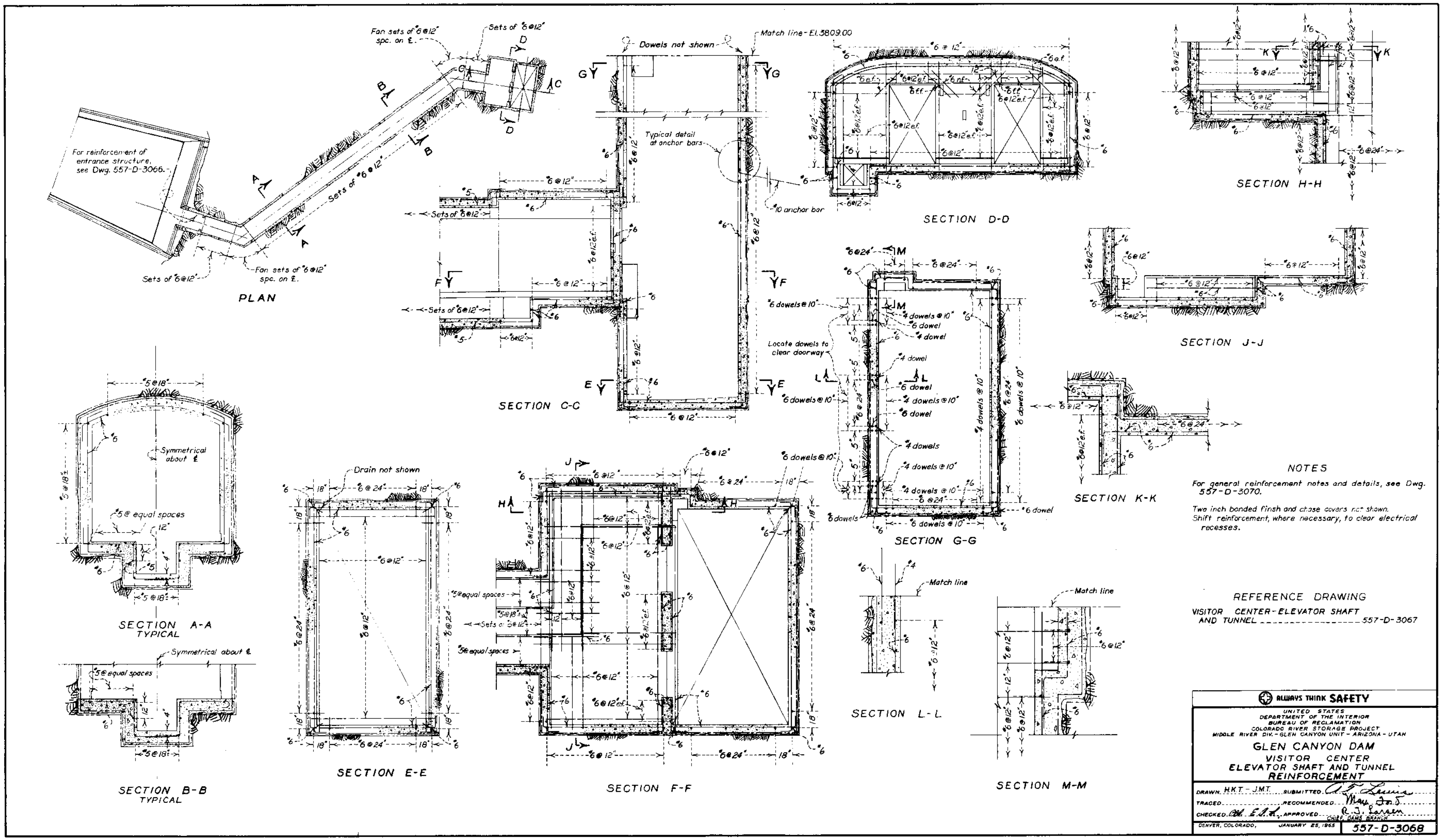


Figure 239.—Visitor center—Reinforcement for elevator shaft and tunnel.

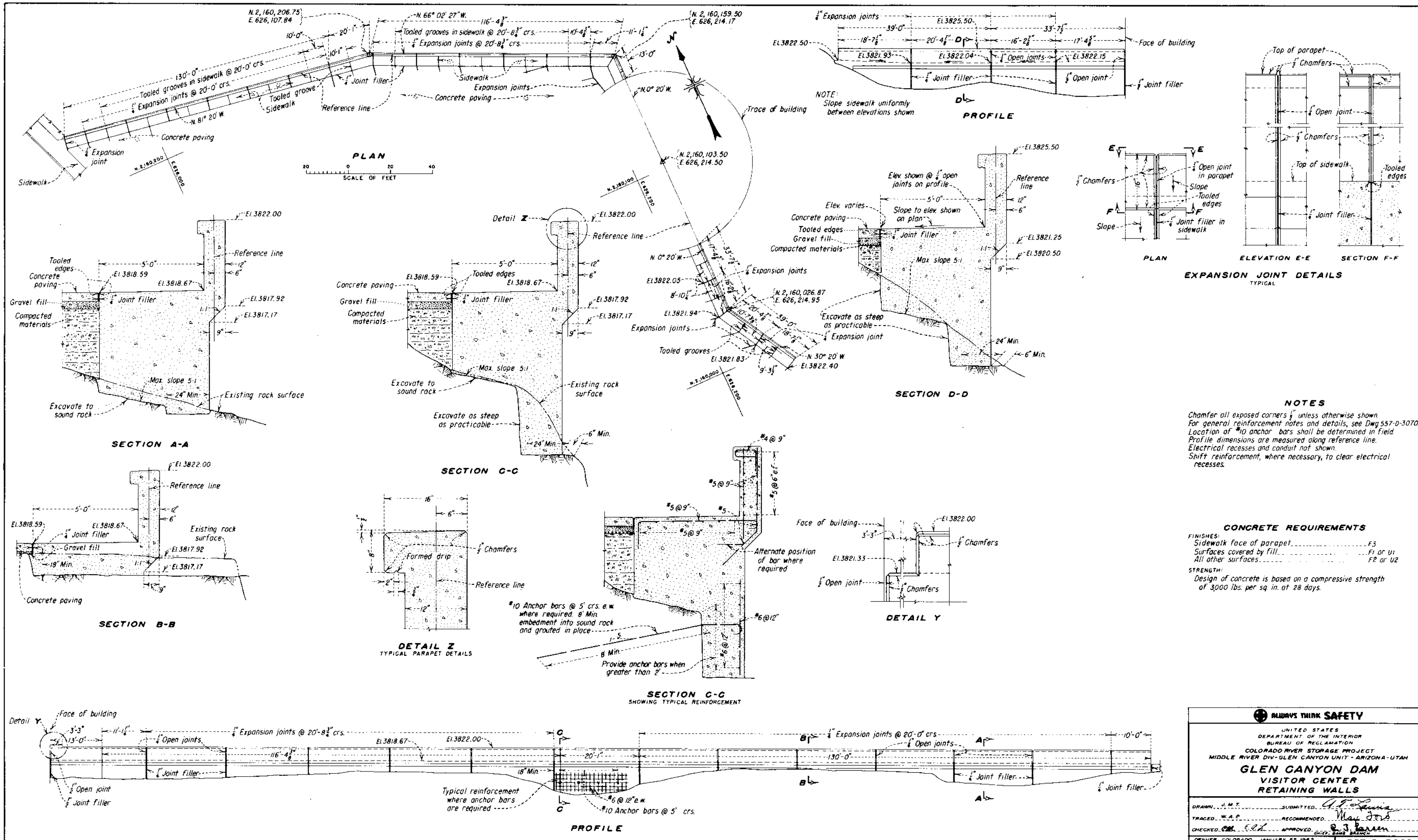


Figure 240.—Visitor center—Retaining walls.

A lateral load of 150 pounds per lineal foot was assumed acting at the top of the parapet. Horizontal temperature reinforcement (5/8-inch bars at 6-inch centers) and expansion joints were provided to minimize cracking in the parapet. The walkway was also reinforced to minimize cracking using 5/8-inch bars at 9-inch centers each way. One and one-eighth-inch anchor bars at 5-foot centers each way were required when the depth of the bench was greater than 2 feet.

125. PASSENGER ELEVATORS. Two passenger elevators provide a means of vertical transportation for visitors between the first floor lobby and the tunnel lobby, and vertical transportation for operation and maintenance personnel from the first floor lobby or tunnel lobby to the equipment rooms at elevation 3810 (landing B). The two elevators were manufactured by Montgomery Elevator Co., San Diego, Calif., and furnished under invitation No. DS-6276.

(a) *Description.*—Each elevator conforms to the following specifications:

Type	Passenger
Capacity	7,000 pounds
Speed	350 feet per minute
Machine	Geared traction
Landings	Three
Openings	Three
Travel in floors	Three
Travel in feet	102 feet 6 inches
Control	Generator field control with leveling device
Operation	Duplex, free car, selective collective, with independent service. (Keyed landing switch, see explanation below.)
Signals	Electric car position indicator. Electric corridor position indicators at each opening.
Call buttons	Up and down type, illuminated direction indication.
Car platform	7 feet 11 inches by 9 feet
Car doorway	4 by 7 feet
Car doors (power operated)	Single-speed center opening with photoelectric cells.
Car accessories	Exhaust blower and switch, 120 volts.
Car heater and heater switch	2,000 watts, 208 volts, single-phase
Car fixtures and fittings	Stainless steel handrails, padhooks, and entrance column returns, recessed telephone cabinet complete with dial telephone. Full area of ceiling is of luminous plasticgrid. (For lighting system see explanation below.)
Carfloor covering	1/8-inch thick, homogeneous vinyl tile (9 by 9 inches)
Car and counterweight guides	Manufacturer's standard, with roller guides.
Hoistway door operation	Power operated.
Compensation	Chain
Power Supply	Power: 440 volts, 3 phase 60 cycles Lights: 120 volts, single phase, 60 cycles Heater: 208 volts, single phase, 60 cycles.

An additional keyed switch is installed on each car control panel that will prevent the cars from stopping at landing B when the key is removed from the switch. This additional keyed switch does not prevent calling cars from landing B call button.

Lighting systems employed in the cars are the manufacturer's standard system and are adequate to provide a uniform horizontal illumination level of not less than 20 foot-candles at a height of 30 inches above the car floors. The system employed facilitates easy replacement of lamps. In addition to the normal lighting system in the elevator cars, an auxiliary automatic, battery-operated-type, emergency lighting unit is installed in each car. The emergency lighting provides approximately 5 foot-candles of illumination and is automatically energized upon failure of normal lighting service and deenergized upon restoration of normal lighting service. The type of battery furnished is readily available from normal commercial sources and is located in a readily accessible position.

The car operates between machined steel guides and is equipped with safety devices, operating unit, and lights. Hoisting machinery and control panels are installed in the machine room. Control pushbuttons are on the cars and at each landing.

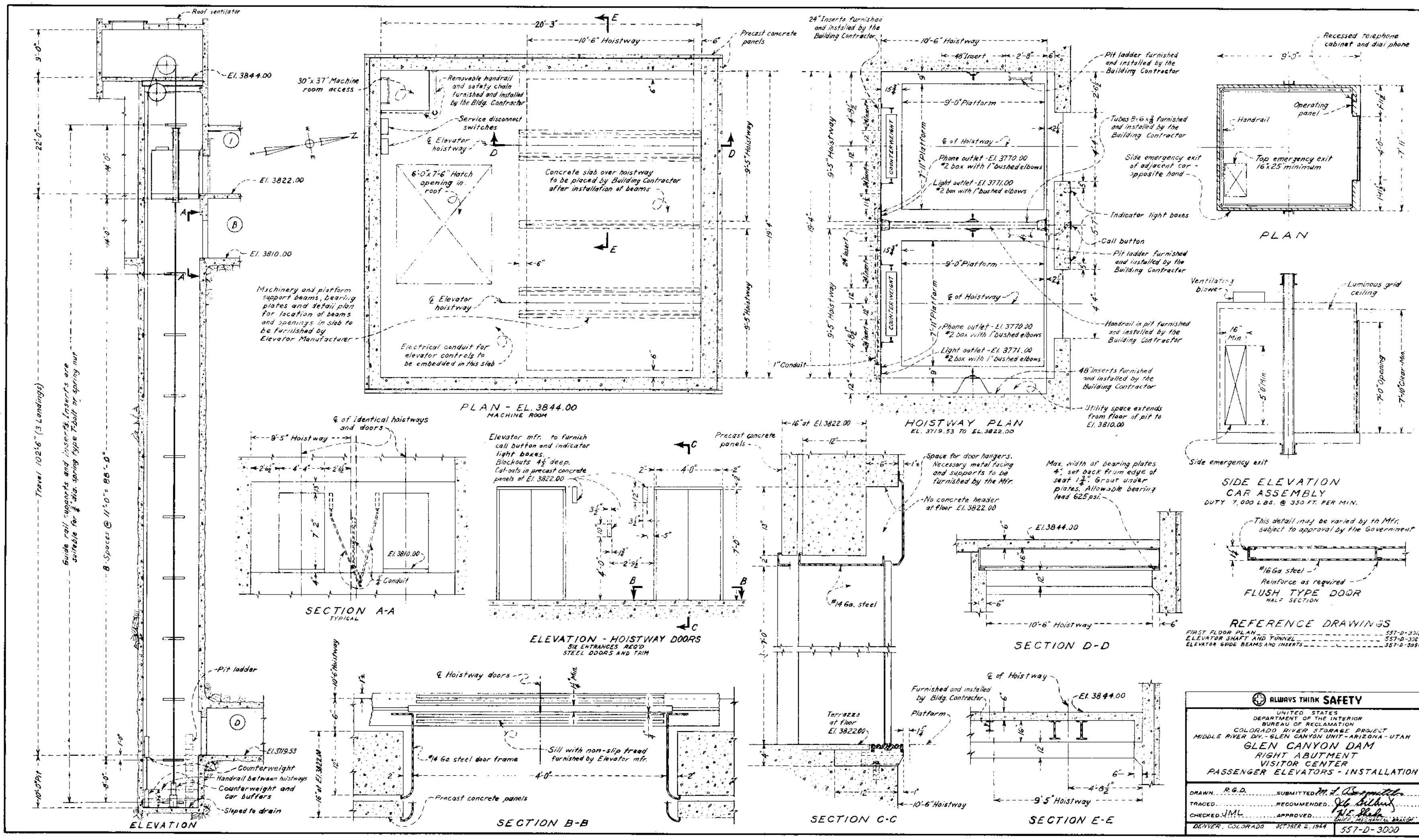


Figure 241.—Visitor center and right abutment passenger elevators—Installation.

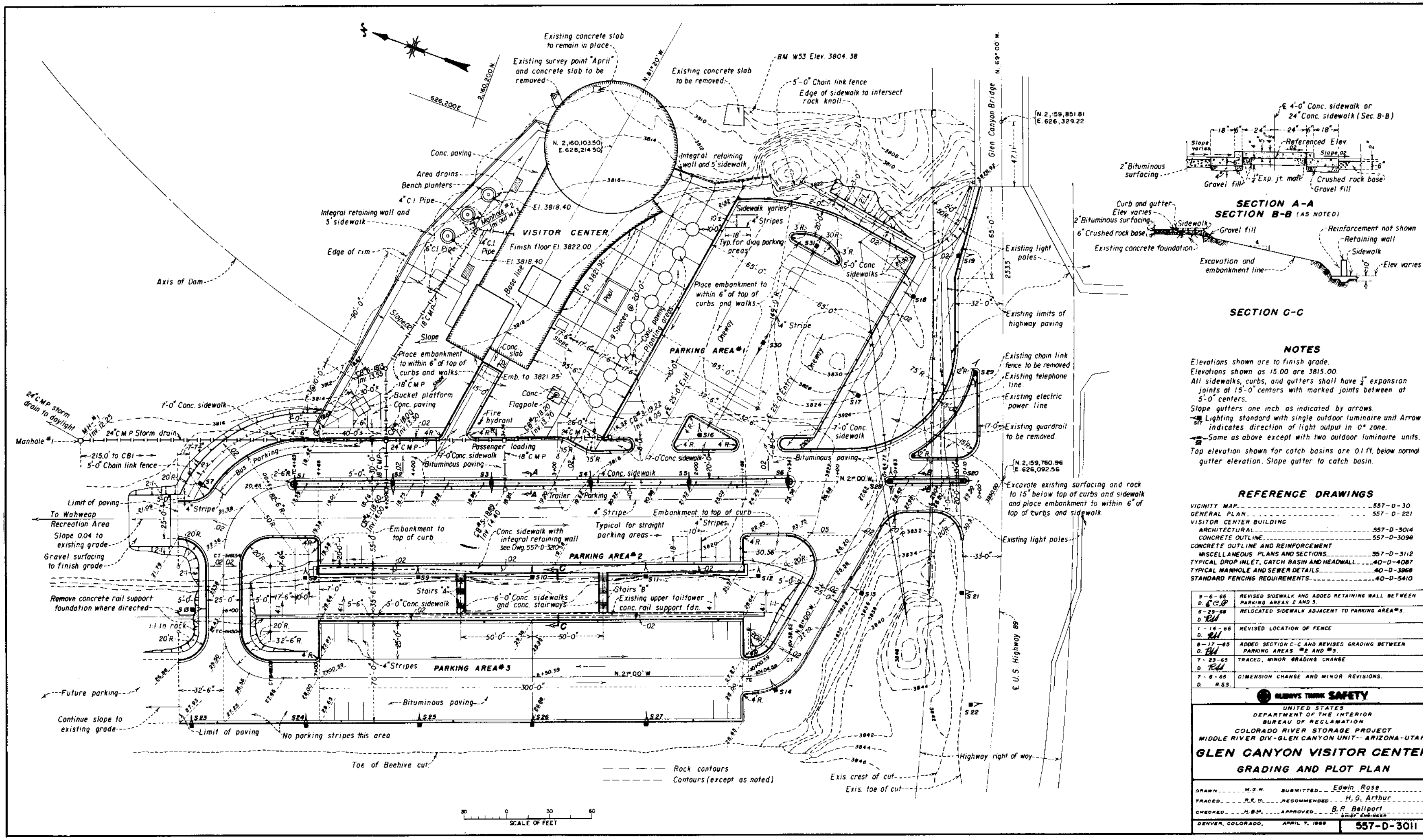


Figure 242.—Visitor center—Grading and plot plan.

(b) *Elevator Design.*—The elevators were designed, equipped, and manufactured in accordance with the latest edition of the "American Standard Safety Code for Elevators, Dumbwaiters, Escalators, and Moving Walks," published by the American Society of Mechanical Engineers. They were installed in accordance with figure 241.

126. PROMENADE, LANDSCAPING, AND PARKING AREAS. These areas are shown on figure 242. They are large concrete-paved promenades on both the canyon rim side and parking area side of the visitor center building. The promenades include planting areas, concrete benches, a lighting installation, and a fountain pool. Areas adjacent to the building and parking areas are landscaped. There is an irrigation system with automatic controls for the landscape areas and for most of the promenade planting areas.

Parking area is available for 150 cars, 8 trailers, and 5 buses adjacent to the visitor center. The parking area is surfaced and provided with drainage installations, curbs and gutters, sidewalks, and lighting standards for illumination. Sufficient area is available for future additional parking, if required. U.S. Highway No. 89 is adjacent to the parking areas and at the access road intersection. Divided traffic lanes are provided to facilitate access to and from the highway.

127. BUILDING WATER SUPPLY SYSTEM, SANITARY SYSTEM, AND DRAIN SYSTEM. Water for domestic use, service use, irrigation, and fire protection is pumped from the Glen Canyon Powerplant domestic water installation. The water treatment includes chlorination only. The water is pumped to an exposed 30,000-gallon steel storage tank on a rock bench near the visitor center building and about 85 feet higher in elevation.

Sewage from the visitor center building flows by gravity to the sewage treatment plant located in the powerplant. This is an activated-sludge, extended aeration-type plant designed for a maximum daily flow of 15,300 gallons, of which 12,000 gallons are from the visitor building. The plant also treats sewage from the powerplant and dam.

The drainlines from the roof drains, floor drains, condenser in the mechanical equipment room, and fountain pool floor drains connect to the storm water drainage installation for the promenade and parking areas.

128. MAIN HIGHWAY. The main road to Glen Canyon Dam and Page, Ariz. is discussed in subsection 4(h).

129. SERVICE ROADS. Specifications No. DC-4825 provided for the construction of approximately 2 miles of service road and access highway, excavation of approximately 9,000 feet of road tunnel and its approaches, and construction of the highway approach embankments for the Glen Canyon Bridge. The bridge is described in the report "Glen Canyon Bridge—Technical Record of Design and Construction." The powerplant service road and the right and left abutment service roads were built 30 feet wide shoulder to shoulder with a 4-inch crushed-rock base and a 24-foot-wide bituminous surface treatment. The powerplant service road tunnel was constructed on a maximum 8 percent grade with a 20-foot wall-to-wall roadway. The road has a 6-inch crushed-rock base and a bituminous surface treatment. Part of this service road was designed to be in open cut traversing a talus area between two tunnel sections. Owing to the unstable condition of the rock face in this area, it was decided to realine the road farther into the rock cliff and make it a continuous tunnel road. Adits 12 feet wide were constructed normal to the centerline of the tunnel on approximately 500-foot centers and extended from the tunnel to the face of the canyon wall. The left abutment service road was connected directly to the end of the dam. The right abutment service road was connected to the dam by a short steel girder bridge 30 feet wide designed for a H20-S16 loading in accordance with AASHO, 1957 edition. Short service roads were also constructed to the spillway intakes.

130. ACCESS ROADS. The access highway joining Glen Canyon damsite with U.S. Highway No. 89 was constructed in two reaches. Under specifications No. DC-4730 the first 4.5 miles of access highway, beginning at U.S. Highway No. 89, was constructed to the roadway subgrade which is 42 feet wide (fig. 242). At Echo Cliffs an unusually heavy cut approximately 280 feet deep was required. The original design at Echo Cliffs called for an 800-foot-long tunnel, but the open cut was finally selected as more desirable. The surfacing and guardrail for the entire access highway was built under specifications No. DC-4887.

The second reach of access highway, ending at the Glen Canyon bridge approach, was constructed under specifications No. DC-4756. The work consisted of constructing approximately 20 miles of roadway to a 42-foot-wide subgrade. A 140-foot reinforced concrete bridge was constructed across Waterholes Canyon. The bridge is a reinforced concrete rigid-frame structure with inclined legs. The design was based on H20-S16 loading in accordance with "Standard Specifications for Highway Bridges," of the American Association of State Highway Officials, 1953 edition; and for H20-S16 loading plus a 75-ton trailer load at normal unit stresses and H20-S16 loading plus a 100-ton trailer load at 120 percent of normal unit stresses.

Specifications No. DC-4887 provided for the surfacing and guardrail for the entire access highway from U.S. Highway No. 89 to the Glen Canyon bridge, a distance of approximately 25 miles. A 6-inch crushed-rock base was placed on the subgrade prepared under the other specifications, and 4 inches of plant-mix bituminous surfacing was then added. The finished road surface width is 34 feet. Beam-type guardrail was erected where necessary. Plant-mix bituminous curbs and spillways were placed on the roadway where required. Through supplemental notice No. 1, specifications No. DC-4887 was revised to include the construction of Manson Mesa airstrip. The runway is 4,500 feet long and 500 feet wide shoulder to shoulder. The 150-foot surfaced landing strip consists of 3 inches of plant-mix bituminous surfacing placed on a 6-inch crushed-rock base. An airplane parking area and 50-foot taxiways were also constructed using the same surfacing section.

Specifications No. DC-4896 provided for the construction of access roads to the community of Page, Ariz. (fig. 243). The connecting road, community to dam, approximately 1.7 miles long, was constructed with a 30-foot width shoulder-to-shoulder and with 1-1/2 inches of plant-mix bituminous surfacing over a 6-inch crushed-rock base. The north community access road, approximately 0.5 mile long, was built using the same roadway section as the connecting road. Both roads had a stone-chip seal coat. The south community access road was also of similar roadway section, except the bituminous surfacing was not required. This road is approximately 1.1 miles in length.

Under specifications No. 400C-83 an existing road was reshaped, a new road and a parking area were constructed, and all were surface treated. This construction provided access from the Arizona-Utah State line to the vista point and parking area at the right abutment of the Glen Canyon damsite. Approximately 7 miles of 6-inch crushed-rock surfacing 20 feet wide was reshaped by scarifying, blading, and rolling. The 0.75 mile of new road, with a 24-foot roadway and 3-foot shoulders, and the 4,200-square-yard parking area have 6-inch crushed-rock bases. To the 7.75 miles of road and the parking area was applied a modified penetration treatment of RC-5 cutback asphalt.

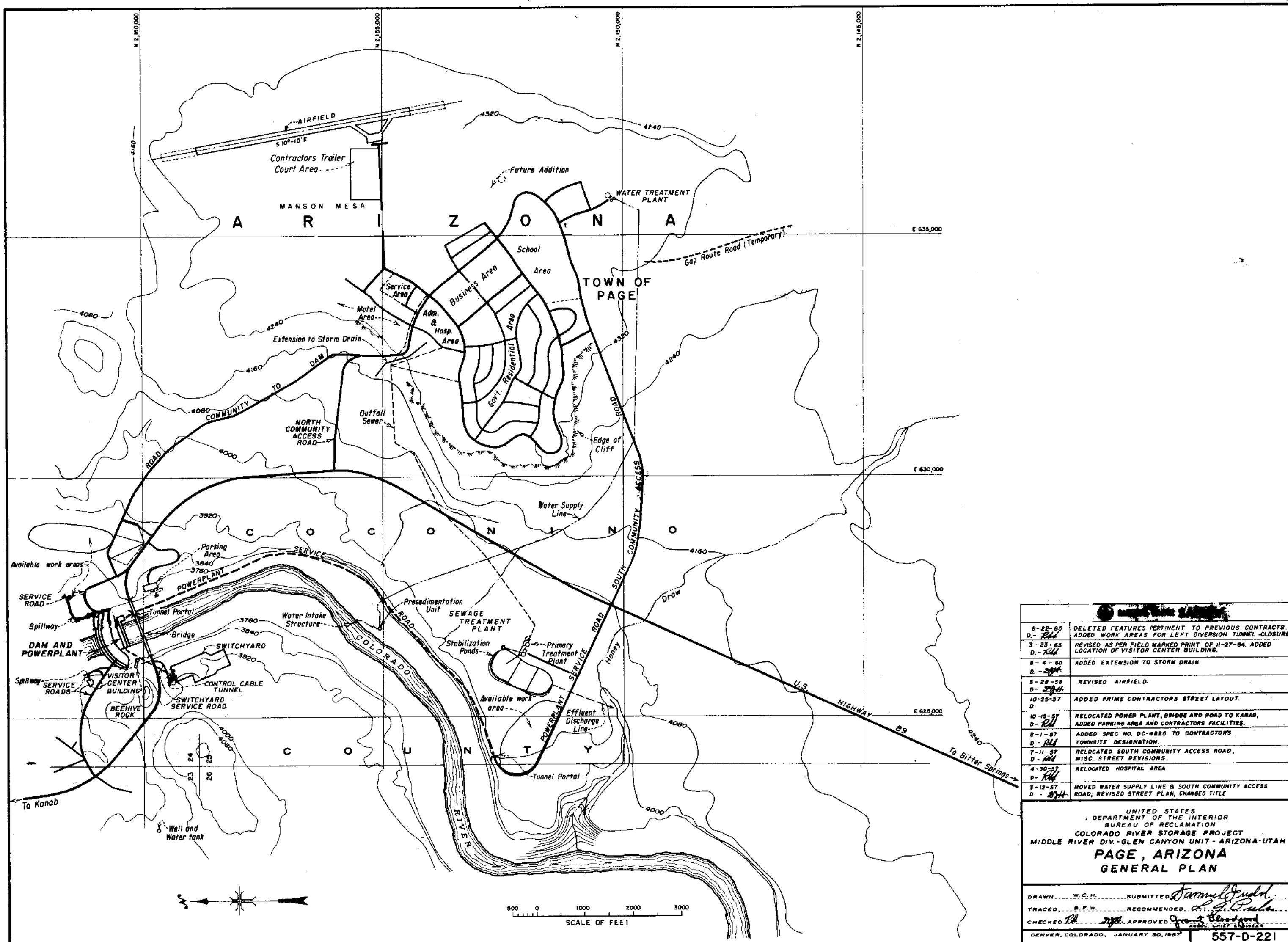
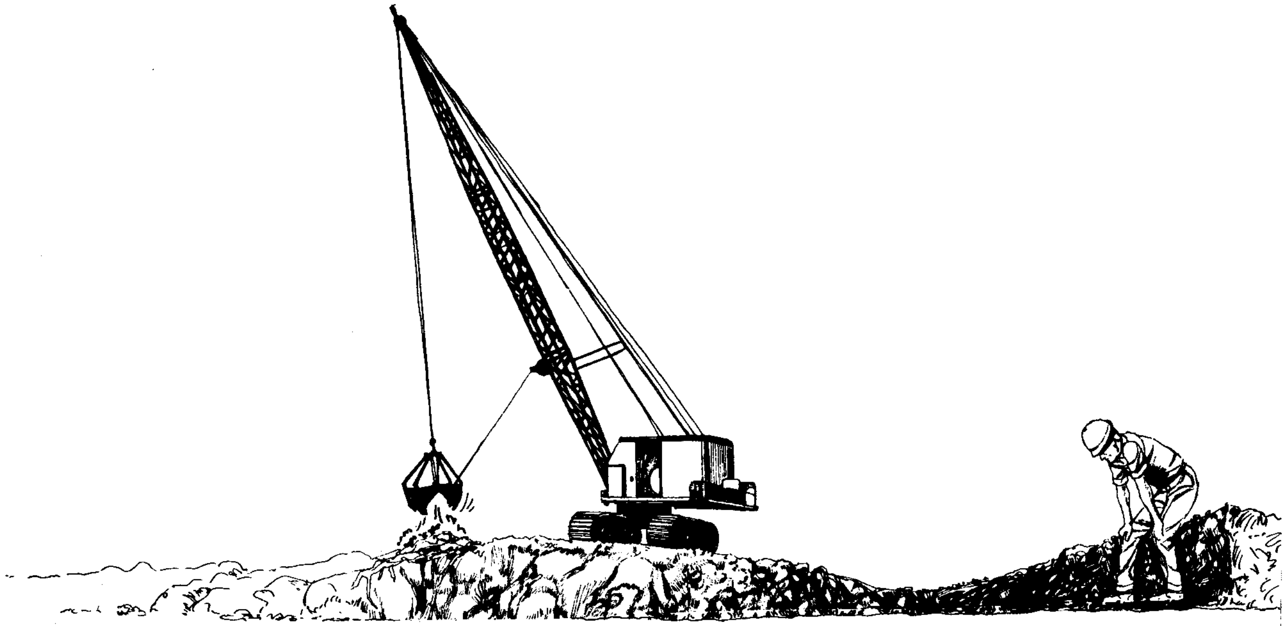


Figure 243.—Town of Page, Ariz.—General plan.

PART III



CONSTRUCTION

PART III--CONSTRUCTION

CHAPTER XII. *Construction*—CONTRACT ADMINISTRATION

131. SUMMARY. (a) *Dam, Powerplant, and Town.*—Owing to the critical need for access to the site of the future dam in Glen Canyon, the first major construction contract was awarded to the Strong Company of Springville, Utah, for work under specifications No. DC-4730 for earthwork and culvert construction for a portion of the Bitter Springs Highway. The work involved construction of subgrade and base course for about 4.6 miles of highway north from an intersection on U.S. Highway No. 89 near Bitter Springs, Ariz., and included deep cuts in rock through the Echo Cliffs monocline. Notice to proceed was received by the contractor on September 24, 1956, and the work was completed on October 1, 1957.

The contract for excavation of the right diversion tunnel was awarded early under specifications No. DC-4747 to provide for river diversion when needed for excavation of the foundation of Glen Canyon Dam and Powerplant. Awarded to Mountain States Construction Co. of Denver, Colo., the contract provided for excavation of a 2,749-foot-long circular diversion tunnel, 43.5 feet in diameter at the "A" line in the upstream portion. The diameter was increased by 3 feet for the downstream portion, which also served as the downstream portion of the spillway tunnel, and concrete lining was not required under these specifications. The contractor received the notice to proceed on October 2, 1956, and the work was completed on March 13, 1958.

Minor construction contracts awarded during this period included; work for exploratory drilling under specifications No. 400C-63; invitation No. GC-420-57-1 providing equipment for more detailed investigations of the Wahweap aggregate deposit with a rotary drill rig; invitations No. 400-424 and 400-428 for operated equipment; and specifications No. 400C-68, which provided for temporary access roads to the west side of the canyon.

W. W. Clyde and Co. of Springville, Utah, was awarded the contract under specifications No. DC-4756 for construction of earthwork and structures for another portion of the Bitter Springs Highway. This work included subgrade and base course construction for about 20.8 miles of highway from a point 4.6 miles north of Bitter Springs, Ariz., to near the east abutment of the Glen Canyon Bridge. Construction of a reinforced concrete bridge at Waterholes Çanyon was included in this contract work. The contractor received the notice to proceed on November 5, 1956, and the work was accepted as substantially complete on July 12, 1957.

Surfacing and guardrail construction for the Glen Canyon Dam and access highway and construction of Manson Mesa airstrip was done under specifications No. DC-4887 by Alexander Construction Co., Inc. Notice to proceed was received by the contractor May 31, 1957, and all work was completed and accepted March 3, 1958. Major items included placement of crushed-rock base, plant mix bituminous surfacing and seal coat and constructing guardrail for approximately 25 miles of access road, as well as furnishing and stockpiling additional surfacing aggregate and construction and surfacing of the airport.

The Glen Canyon Bridge was constructed under specifications No. DC-4800 by Keiwit-Judson Pacific Murphy, a joint venture, to provide the connecting link across the canyon between the two sections of the highways leading to the site of the dam and powerplant. The work involved construction of a steel-arch-type bridge with a single span 1,028 feet in length. The overall length of 1,271 feet included abutments, and the concrete roadway was 30 feet wide with a sidewalk 4 feet in width on each side. Notice to proceed with the work was received by the contractor on January 28, 1957, and the last part of the work was completed on January 18, 1959. The bridge was dedicated and opened to the public on February 20, 1959. See appendix E for additional cost and engineering data.

The main contract for construction of Glen Canyon Dam, Powerplant, and appurtenant works was accomplished under specifications No. DC-4825 by Merritt-Chapmen and Scott Corp. of New York, N.Y., and is referred to as the prime contract in this publication. The notice to proceed with the first part of the work was received by the contractor on May 2, 1957, and the last part of the work was completed on October 2, 1964. Part A of the contract schedule called for construction of the dam, powerplant, and related works, including the spillways and diversion tunnels (except certain work under specifications No. DC-4747). This work included rock and common excavation for these features, line drilling, rock bolting, backfilling, drilling and grouting, diversion and care of the river; mass, first-stage, and structural concrete; reinforcing bars, concrete tunnel lining, outlet penstock pipe installation, structural steel, concrete cooling systems, embedded metalwork, piping, gates and related equipment. Part B of the schedule included the excavation and surfacing of a section of highway from Bitter Springs to the left abutment of the bridge. Part C of the schedule included construction of the right and left abutment service roads to the dam and spillways, the powerplant service tunnel and roadway, and the switchyard service road.

On September 6, 1957, W. W. Clyde and Co. of Springville, Utah, received notice to proceed with construction of access roads, streets, and utilities for Page, Ariz., under specifications No. DC-4896. The contract provided for construction of a 1.7-mile-long connecting road from Page to the service road to the dam; a 0.5-mile-long access road from the Bitter Springs road to the connecting road; a 1.1-mile-long connecting road from the south edge of Page to the Bitter Springs road; and 8.44 miles of streets and 13.66 miles of curbs, gutters, and sidewalks in the town of Page. Grading of the townsite and construction of the storm sewer system, the sanitary sewer collection system, and the water distribution system for Page was also included in these specifications. All work was completed and accepted on December 19, 1958.

Minor construction contracts under specifications No. 400C-83 and 400C-98 were awarded during this period for earthwork, structures and roadway surfacing for temporary public use shelters and parking areas on each side of the canyon, just downstream from the construction site. Contracts were also awarded for construction of the foundations for 50 demountable houses under specifications No. DC-4886, and for the 150,000-gallon elevated storage tank and the 3,000,000-gallon storage reservoir under specifications No. DC-4865.

The sewage treatment facilities for the town of Page were constructed by W. W. Clyde and Co. of Springville, Utah, under specifications No. DC-4912. Notice to proceed was received September 6, 1957, and all work was completed and accepted May 2, 1958. Under this contract, an access road was constructed from the powerplant access road to the sewage treatment plant and an outfall sewer pipeline and a water service line, both about 7,000 feet long, were constructed from the town of Page to the treatment plant. Work on the treatment plant proper consisted of construction of concrete structures to house sewage treatment equipment, installing equipment, and building a compartmented stabilization pond.

Specifications No. DC-4924 provided for the construction of a laboratory building, a Government warehouse and a municipal building. Notice to proceed was received by Security Construction Co. of Salt Lake City, Utah, on September 13, 1957, and all work was completed and accepted on July 2, 1958. The laboratory was a wood frame building 39 by 64 feet to provide office space and house normal field concrete control equipment. The warehouse for housing Government material and equipment was a rigid steel frame building 320 by 100 feet with insulated metal wall and roof panels. The municipal building was a wood frame building with a main wing 40 feet 6 inches wide by 114 feet long with a wing measuring 24 by 75 feet. Initially providing general Government office space at Page, this building was later utilized as offices for a number of municipal services.

Southern Engineering Co., Inc., of Long Beach, Calif., constructed the water treatment plant for Page under specifications No. DC-4933. Notice to proceed was received on September 13, 1957, and all work was completed on November 8, 1958. The specifications called for water to be pumped from the Colorado River at a point about a mile downstream from the dam to a pretreatment plant, where initial flocculation and primary removal of sediments were performed. Water was pumped from there through a 12,000-foot-long pipeline to the treatment and storage area, where filtering, final treatment, and storage were carried out.

Permanent residences for Government employees were constructed by The Mobilehome Corp. of Bakersfield, Calif., and others in a joint venture, under specifications No. DC-4989. Notice to proceed was received January 9, 1958, and all work was completed and accepted March 20, 1959. The specifications provided for all on-lot work necessary to construct 200 concrete masonry block residences with concrete slab floors and wood-truss roofs.

Sierra Construction Corp. of Las Vegas, Nev., constructed the administration building and the garage, fire station, and police building under specifications No. DC-5115. The administration building was a one-story steel frame building 256 by 48 feet with concrete masonry and metal window exterior wall construction and a sloping built-up asphalt roof with white stone surfacing. The garage, fire station, and police building was divided into two adjoining parts. The garage and fire station portion had a steel frame, steel roof decking, and sheet metal siding; and the police station portion was of concrete block masonry with a built-up asphalt roof. Notice to proceed was received by the contractor December 10, 1958, and all work was completed and accepted August 20, 1959.

Minor construction contracts awarded during this period included the lighting installations for the airstrip under specifications No. DC-5066, a control cable installation for the municipal water supply system under specifications No. DC-5090, a 50-ton motor truck scale and scale house under specifications No. DC-5123, a seismograph vault and access road under specifications No. DC-5163, soil stabilization and landscaping for Government facilities at Page under specifications No. 400C-140, parking area and utilities for block 17 at Page under specifications No. DC-5206, temporary tourist center under specifications No. 400C-161, erection of column struts for Glen Canyon Bridge under specifications No. DC-5607, and certain modifications to the Page water supply system under specifications No. 400C-189.

The second main contract for construction of the dam and powerplant proper was the completion contract under specifications No. DC-5750 by the Ets-Hokin Corp. of San Francisco, Calif. Completion work included second-stage blockout and surfacing concrete; partition walls; architectural finishes (except terrazzo floors); installation of turbines, governors, and accessory equipment; piping systems; heating, ventilating and air-conditioning systems; installing power transformers, bus structures, and accessory electrical equipment; electrical control, lighting and distribution systems; switchyard earthwork and surfacing completion, steel structures, and bus and electrical equipment installations in the switchyard; completion of the control cable tunnel and the powerplant transformer-to-switchyard circuits. The notice to proceed was received by the contractor on July 19, 1962, and the last part of the work was completed on October 25, 1966.

The eight generators were furnished and installed by separate contract under invitation No. DS-5522 by General Electric Co., and the turbines were furnished by Baldwin-Lima-Hamilton Corp. under invitation No. DS-5234.

The permanent visitor center at Glen Canyon Dam was constructed by Allen M. Campbell Co., Inc., of Tyler, Tex., under specifications No. DC-6274. The contractor received the notice to proceed on July 12, 1965, and the work was completed on August 17, 1967. The building consists of a steel-framed one-story wing with basement, a steel-framed rotunda, and a concrete-lined elevator shaft. The center will be operated by the National Park Service, who will also furnish the exhibits for the rotunda display.

Completion of the left diversion tunnel plug and spillway elbow was performed under specifications No. DC-6317 by S. S. Mullen Co., Inc., of Seattle, Wash. The work includes final excavation of the lower end of the spillway shaft, placing backfill and lining concrete, drilling and grouting, and epoxy repairs for the existing tunnel lining. Notice to proceed with the work was received by the contractor on August 23, 1965, and the work was accepted as substantially complete on March 7, 1967.

A list of all construction contracts, as well as all significant supply contracts, in the order of issue of specifications is included as appendix A. Some of the above contracts are discussed in detail in the following sections.

In the subsequent sections direct and indirect reference is often made to extra work orders, orders for changes, and purchase orders, which are not presented complete in this publication; therefore, it is suggested that if more detailed information on each type of order is desired, the final construction report should be consulted (see bibliography).

(b) *Transmission Lines and Substations.*—The Glen Canyon-Shiprock 230-kilovolt transmission line was constructed by Electrical Constructors of Columbus, Ohio, under specifications No. DC-5610. The transmission line is a 3-phase, 60-cycle, single-circuit line approximately 175 miles long. The main features of the work were right-of-way clearing, construction of footings, erection of steel towers, and stringing two 1/2-inch steel overhead ground wires and three 1,272,000-circular-mil ACSR (aluminum conductor, steel reinforced) conductors. Notice to proceed was received by the contractor on October 5, 1961, and work was completed on January 28, 1964. A 7-mile section of the Glen Canyon-Shiprock Line, near Kayenta, Ariz., utilizing experimental towers was constructed under another contract.

A contract was awarded to Irby Construction Co. under specifications No. DC-5710, for the construction of 42 miles of 230-kilovolt 3-phase, 60-cycle single-circuit transmission line from the Shiprock Substation to a point east of Cortez, Colo. The work performed included right-of-way clearing, furnishing and installing gates in right-of-way fences, furnishing tower leg grounds, constructing tower footings, furnishing and erecting steel towers, and furnishing and attaching tower signs. Notice to proceed was received by the contractor on May 14, 1962, and work was initiated at the site on July 5, 1962. All work under the contract was accepted as complete on September 5, 1963.

The construction of the Cortez-Curecanti 230-kilovolt transmission line and stringing conductors and overhead ground wires for the Shiprock-Cortez 230-kilovolt transmission line under specifications No. DC-5753, was awarded in two schedules.

Schedule 1 was awarded to Electrical Constructors of Columbus, Ohio, on April 23, 1962, on their bid of $4,224,514.00. The work consisted of constructing reinforced concrete footings with embedded stub angles, furnishing and erecting steel towers, and stringing conductors and overhead ground wires for approximately 101 miles of 230-kilovolt transmission line. This schedule was completed on February 28, 1964.

Schedule 2 consisted primarily of furnishing and stringing conductors and overhead ground wires for approximately 42 miles of 230-kilovolt transmission line for which towers had been previously erected under another contract. Malcolm W. Larson Contracting Co. of Denver, Colo., was awarded the contract for this schedule on April 23, 1962. Work was completed on September 5, 1963.

The Glen Canyon-Flagstaff-Pinnacle Peak 345-kilovolt transmission line is a 3-phase, 60-cycle single-circuit line approximately 240 miles in length and consists of two sections. The Glen Canyon-Flagstaff section, approximately 123 miles in length, extends from Glen Canyon Switchyard to the Flagstaff Substation. The Flagstaff-Pinnacle Peak section extends from the Flagstaff Substation to the Pinnacle Peak Substation, a distance of approximately 117 miles. Ets-Hokin and Galvan Corp. of San Francsico, Calif., was awarded the contract for construction of the entire line on June 22, 1962. This firm later changed their name to Ets-Hokin Corp. Construction of the line was divided into four schedules. Schedules 1 and 2, from Glen Canyon to Flagstaff, were under the administration of the Project Construction Engineer at Page, Ariz. Administration of schedules 3 and 4, the section from Flagstaff to Pinnacle Peak, was the responsibility of the Project Manager at Phoenix, Ariz. The work for the Glen Canyon-Flagstaff section consisted of right-of-way clearing, footing construction, steel tower erection, stringing three double-bundle conductors, three single-bundle conductors at river crossings, and two overhead ground wires. All work under schedules 1 and 2 was accepted as substantially complete on November 12, 1964.

The transmission lines in the vicinity of Shiprock Substation were constructed by Reynolds Electrical and Engineering Co., Inc., of Santa Fe, N. Mex., under specifications No. DC-5850. These transmission lines were extensions of the Glen Canyon-Shiprock line, the Shiprock-Cortez line, and the Shiprock-Arizona Public Service Plant line. Land clearing, footing construction, furnishing and erecting steel towers, and furnishing and stringing conductor and overhead ground wires were the primary items of work under the specifications. Notice to proceed was received by the contractor on March 1, 1963. First work was initiated at the site on April 8, 1963, and all work was completed by June 13, 1964.

An experimental section of the Glen Canyon-Shiprock 230-kilovolt transmission line approximately 8.2 miles long near Kayenta, Ariz., was constructed by Construction Helicopters, Inc. of Grand Junction, Colo., under specifications No. DC-5953. The work mainly involved constructing structure footings and guy anchors; erecting 28 special structures and 4 standard-type SM towers; and stringing conductors and overhead ground wires. The specifications required that four of the special structures and two of the standard-type SM towers be erected by helicopter and the remainder by conventional erection methods. All structures and stringing materials were furnished by the Government. The contractor received notice to proceed on July 22, 1963, and all the work was completed by March 25, 1964.

The contract for construction of the Shiprock Substation, stage 01, was awarded to Reynolds Electrical and Engineering Co., Inc., of Santa Fe, N. Mex., under specifications No. DC-5907 on April 17, 1963. Construction of the substation consisted of six principal features of work. These features were earthwork, culverts, drains, parking area surfacing and access roads; transporting the Government-furnished autotransformer; constructing a masonry garage and a masonry service building; constructing reinforced concrete foundations for steel structures and equipment and erecting steel structures; electrical installations; and installing three 230-kilovolt approach spans. The contractor received notice to proceed on June 21, 1963, and all work was completed and accepted by June 5, 1964.

Construction Helicopters, Inc., of Grand Junction, Colo., constructed the Glen Canyon-Page 138-kilovolt transmission line under specifications No. DC-6030. The work involved construction of 138-kilovolt, 3-phase, H-frame wood-pole and steel tower, single-circuit transmission line, approximately 2 miles in length, extending from the Glen Canyon Switchyard to the city of Page, Ariz. Notice to proceed was received by the contractor on January 3, 1964, and work at the site started on January 23. All work had been completed by August 7, 1964. This line was initially energized on February 4, 1966, at 69 kilovolts.

Ets-Hokin Corp. of San Francisco, Calif., was awarded the contract for construction of the Glen Canyon-Flagstaff 345-kilovolt transmission line No. 2 under specifications No. DC-6100. This line is approximately 124 miles long and extends from the Glen Canyon Switchyard to the Flagstaff Substation. The work mainly consisted of furnishing and installing gates, cattle guards, and culverts; right-of-way clearing;

constructing tower footings; testing, furnishing and erecting steel towers; furnishing and performing RIV and corona tests on insulator assemblies and hardware; and furnishing and stringing conductor and overhead ground wires complete with all accessories. Notice to proceed was received by Ets-Hokin Corp. on July 14, 1964, and all work had been completed by April 24, 1966.

The Flagstaff Substation, located near Winona, Ariz., was constructed under specifications No. DC-6220 by The Howard P. Foley Co. of Tucson, Ariz. The principal features of the work performed under these specifications were earthwork for the substation site and access road; constructing a service building, constructing concrete foundations for structures and equipment; furnishing and erecting steel structures; electrical installation work; and installing Government-furnished materials and furnishing and installing contractor-furnished materials for the approach spans for the Glen Canyon-Flagstaff Transmission Lines No. 1 and 2 and the Flagstaff-Pinnacle Peak Transmission lines No. 1 and 2. Work at the substation site began on April 4, 1965. The overhead conductors were first energized through the substation on June 29, prior to the completion of all work on December 9, 1966.

The actual construction of the features contained in the contracts briefly outlined in this subparagraph (b) are not discussed in this publication.

A. MAJOR CONTRACTS

132. SPECIFICATIONS NO. DC-4825 (PRIME CONTRACT)—CONSTRUCTION OF GLEN CANYON DAM AND POWERPLANT. The contract awarded for work under specifications No. DC-4825 was the largest of all contracts dealing with the construction of the Glen Canyon Dam and Powerplant. It was also and still is (June 1969) the largest single contract ever awarded by the Bureau of Reclamation.

In general, the contract required construction of an arch-type concrete dam, construction of a powerplant structure, construction of two tunnel-type spillways, earthwork for the switchyard and control cable tunnel, and construction of service roads and parking areas. In addition, the contract included care and diversion of the river during construction.

The contract required all excavation work for the dam, drilling and grouting of the foundation, and drilling of drainage holes; placement of concrete, including the construction of galleries and shafts; installation of all embedded materials in the dam; and the installation of gates and other major equipment and metalwork, including penstocks and outlet works.

For the two spillways, the contractor was to perform all excavation, except the downstream portion of the right spillway, perform the required drilling and grouting, place reinforced concrete lining, and install the radial gates and hoists at the intake structure. The left diversion tunnel, the downstream portion of which became part of the completed spillway, was also constructed under this contract.

The contract included excavation for the construction of the entire powerplant structure, but, it did not include the placement of second-stage concrete, installation of turbines, generators, and electrical and hydraulic apparatus, installation of floor finishes and other finish items or the installation of piping and other metalwork that was to be performed later under the completion contract. The installation of powerplant cranes and the draft tube bulkhead gates was included.

Other general items included the construction of foundation piers for the penstocks, installation of penstocks between the dam and powerplant, installation of outlet pipes downstream of the dam and the mass concrete for these, installation of hollow-jet valves, excavation of the 10,056-foot access roadway tunnel, erection of service bridges at the top of the dam, and construction of the approaches to the Glen Canyon Bridge.

Prior to opening of bids, 13 supplemental notices were issued to clarify and supplement the original specifications.

(a) *Summary of Bids, Award of Contract, and Execution of the Work.*—Bids were opened at Kanab, Utah, on April 11, 1957, for Glen Canyon Dam and Powerplant, under specifications No. DC-4825. Four bids were received. Merritt-Chapman and Scott Corp. was the low bidder. The three lowest bids received and the engineer's estimate are listed below:

(1) Merritt-Chapman
and Scott Corp.
260 Madison Ave.
New York 16, N.Y. $107,955,522.00

(2) Glen Canyon Contractors
10 West Orange Ave.
South San Francisco, Calif. $118,336,476.00

(3) Morrison-Knudsen Co.,
Inc., et al
Boise, Idaho $120,178,853.00

Engineer's estimate $135,608,170.00

The contract, No. 14-06-D-2403, was awarded to Merritt-Chapman and Scott Corp. on April 29, 1957. The time for completion allowed in the specifications was as follows:

Part (1) All work to complete the powerplant structure, including required equipment installations; all work to complete the powerplant access road and to complete the parking area between the powerplant structure and the river-outlets valve structure; all work to complete switchyard grading, switchyard access road and cable tunnel excavation—1,760 days.

Part (2) All work to complete the access highway within 60 days after receipt of written notice from the contracting officer that work on the Glen Canyon Bridge, under specifications No. DC-4800, has been completed to the extent necessary to allow work under these specifications to proceed.

Part (3) The remainder of the work—2,500 days.

Notice to proceed was received by the contractor May 2, 1957, thereby establishing the completion dates as follows:

Part (1) February 25, 1962
Part (2) Determined by specifications No. DC-4800
Part (3) March 6, 1964

Construction started June 12, 1957, with the delivering of construction equipment to the jobsite.

(b) *Orders for Changes.*—There were 25 orders for changes under this contract totaling approximately 8 million dollars. For a brief description of the extra work performed and the resulting cost adjustments, see appendix B. A full description of this work may be found in the final construction report (see bibliography).

(c) *Special Agreements.*—There were four special agreements under this contract pertaining to construction power requirements and expediting certain work to avoid delaying turbine installations. A full discussion of this work is contained in the final construction report (see bibliography).

(d) *Findings of Fact on Requests for Extension of Contract Time.*—Findings of fact by the contracting officer, dated May 22, 1959, extended part 2 a total of 38 days. This delay was due to the Government's directive that the contractor postpone work on the surface treatment of the highway approaches to the Glen Canyon Bridge, due to low ground temperatures.

(e) *Claims Against the Government.*—A major claim developed early in the contract in connection with a contract provision for adjustment for increases in labor costs in which the Government was to pay 85 percent of the cost of wage rate increases. The controversy arose in late 1959, when a new labor agreement was negotiated with the five basic crafts (carpenters, cement masons, laborers, teamsters, and operating engineers). The new agreement provided for wage increases which were 50 cents an hour higher than wage increases for these crafts in the Arizona Master Labor Agreement. The Government refused to allow adjustment on this portion of the wage increases on the basis that this amount represented a subsistence payment which was specifically exempted by the specifications from the adjustment.

After considerable controversy involving the contractor, the Bureau, and the Board of Contract Appeals, the Government paid a portion of the contractor's claim, or $289,567.67, in full settlement of the claim. Further discussion of this matter is given in section 141 and in the final construction report (see bibliography).

133. SPECIFICATIONS NO. DC-5750—COMPLETION OF GLEN CANYON POWERPLANT, SWITCHYARD, DAM, AND APPURTENANT WORKS. Completion of the Glen Canyon Dam and appurtenant works consisted of completing architectural finishes, furnishing and installing plumbing fixtures, doors, partitions, handrails, heating and ventilating systems, and electrical installations. Completion of Glen Canyon Switchyard included earthwork, grading and surfacing, concrete foundations, furnishing and erecting steel structures, fence, and electrical installations. The work also included four transmission circuits from the powerplant transformer deck to the switchyard.

The completion of the Glen Canyon Powerplant consisted of placing second-stage concrete for embedding turbines and support for generators; placing blockout concrete and miscellaneous concrete at various locations throughout the plant; constructing partition walls and completing architectural finishes; installing penstock makeup pieces, hydraulic turbines governors for hydraulic turbines, accessory hydraulic equipment and machine tools; installing or furnishing and installing fire protection equipment, piping systems, air-conditioning, heating and ventilating systems, sewage treating equipment, plumbing fixtures, metalwork, electrical conduit, wires, cables, lighting fixtures, and other electrical equipment and accessories except the generators which were installed under a separate contract.

Four supplemental notices were issued prior to the opening of bids. These notices made changes and additions in the specifications in order to correct, supplement, and clarify the specification provisions and to include revised drawings.

(a) *Summary of Bids, Award of Contract, and Execution of the Work.*—Bids were opened at Page, Ariz. on June 5, 1962, for completion of Glen Canyon Powerplant, Switchyard, Dam, and appurtenant works, under specifications No. DC-5750. Seven bids were received; Ets-Hokin and Galvan, Inc., was the low bidder. The three lowest bids received and the engineer's estimate are listed below:

(1) Ets-Hokin and Galvan, Inc. 551 Mission Street San Francisco 5, Calif.	$7,891,271.70
(2) Morrison-Knudsen Co., Inc. 319 Broadway Boise, Idaho	$7,985,494.00
(3) Gunther-Shirley-Lane 4560 Sherman Oaks Ave. Sherman Oaks, Calif.	$8,248,980.00
Engineer's estimate	$7,736,602.00

Contract No. 14-06-D-4429 was awarded to Ets-Hokin and Galvan, Inc., on June 25, 1962. A period of 1,350 days was allowed by supplemental notice to the specifications for completion of all work under the contract. The work was divided into nine parts and, where applicable, each unit had a completion time within that part. Notice to proceed was received by the contractor July 19, 1962, thereby establishing March 30, 1966, as the date for completion. Work at the site was started on July 30, 1962. A strike, delays in delivery of Government-furnished turbine parts, and drawing approval were causes of delays subsequently approved by findings of fact and by contracting officer's letter. An amendatory agreement to expedite the work (subsection (d)) revised the completion dates for some parts. The work was accepted as substantially complete on October 25, 1966, within the adjusted time allowed.

(b) *Orders for Changes.*—There were 13 orders for changes under this contract totaling about $320,000.00. For a brief description of the extra work performed and the resulting cost adjustments see appendix C. A full description of this work may be found in the final construction report (see bibliography).

(c) *Findings of Fact.*—Findings of fact dated January 15, 1964, considered a delay in completion of the contract due to a strike by the Operating Engineers Local 428. As a result of the findings, the completion of all parts of the contract was extended 22 calendar days.

Findings of fact dated September 9, 1964, considered delays due to late delivery of Government-furnished turbine parts for units 1, 2, and 3 installed under part (1) of paragraph 16a of the specifications. As a result of the findings, the completion of part (1) of the contract was extended 224 calendar days for unit 1, 157 calendar days for unit 2, and 73 calendar days for unit 3.

By letter of February 28, 1964, the contracting officer allowed extensions of time for the various parts of the contract for delays in approval of drawings by his office.

(d) *Amendatory Agreement.*—An amendatory agreement dated January 6, 1964, established earlier completion dates from those in the specifications so that power production could begin sooner and allowed additional compensation to the contractor for the expenses incurred in speeding up his operation to meet the new dates. The net increase in the amount due under the contract for this work was $575,000.00.

B. MINOR CONTRACTS

134. SPECIFICATIONS NO. DC-4747—RIGHT DIVERSION TUNNEL. The diversion tunnel consists of 1,818 linear feet of unlined tunnel 43 feet 6 inches inside diameter, and 950 linear feet of unlined tunnel 46 feet 6 inches inside diameter. The principal features of the work for construction of the diversion tunnel were excavation, in open cut, at the upstream and downstream portals, and excavation for the diversion tunnel including furnishing and installing permanent tunnel supports.

(a) *Summary of Bids, Award of Contract, and Execution of the Work.*—Seven bids were received at Kanab, Utah, and were opened on September 11,

1956. The three lowest bids and the engineer's estimate are listed below:

(1) Mountain States Construction Co. Denver, Colo.	$2,452,340.00
(2) Morrison-Knudsen Co., Inc., and Henry J. Kaiser Co., a joint venture Los Angeles, Calif.	$2,961,800.00
(3) Coker Construction Co., Peter Kiewit Sons' Co., and Condon-Cunningham Co., a joint venture Omaha, Nebr.	$3,334,100.00
Engineer's estimate	$3,845,100.00

The contract, No. 14-06-D-2045, was awarded to Mountain States Construction Co. of Denver, Colo., on October 1, 1956. A period of 450 days was allowed in the specifications for the completion of the contract. Notice to proceed was received October 2, 1956, thereby establishing the completion date as December 26, 1957. The contractor began moving in equipment to the jobsite on October 8, 1956.

All work was completed and accepted March 13, 1958. The net contract amounted to $2,278,575.62.

(b) *Orders for Changes.*—There were three orders for changes under this contract. A brief description of this work may be found in the final construction report (see bibliography).

135. SPECIFICATIONS NO. DC-4800—GLEN CANYON BRIDGE. Glen Canyon Bridge consists of a steel arch with a single span of 1,028 feet and a total overall length of 1,271 feet including abutments. The bridge has a 30-foot-wide concrete road with sidewalk 4 feet wide on each side.

(a) *Summary of Bids, Award of Contract, and Execution of the Work.*—Two bids were received and opened in Kanab, Utah, on December 18, 1956. Kiewit-Judson Pacific Murphy of Emeryville, Calif., was the low bidder on the bid of $4,139,277.00. The second bidder was Bethlehem Pacific Coast Steel Corp. of San Francisco, Calif. Their bid was $4,408,944.00. The engineer's estimate was $2,944,750.00.

Contract No. 14-06-D-2240 was awarded to Keiwit-Judson Pacific Murphy on January 21, 1957, and notice to proceed was received on January 28, 1957. Completion of the work was divided into four parts. Part (1) provided for completion of the entire bridge, exclusive of painting below the concrete bridge deck, within 750 days. Painting below the concrete deck, under part (2), was to be completed within 870 days. Removal of equipment and facilities between lines 80 feet each side of the centerline of the east approach and 40 feet each side of the centerline of the west roadway approach within 720 days was required under part (3). Part (4) provided for the installation of metal railings for the abutments within 720 days. The established dates for completion of the four parts are as follows: Part (1), February 17, 1959; part (2), June 17, 1959; part (3), January 18, 1959; and part (4), January 18, 1959. All work under the contract was accepted as substantially complete on March 25, 1959. The contract amounted to a total of $4,658,497.57.

(b) *Orders for Changes.*—There were four orders for changes under this contract totaling approximately $186,000.00.[1]

(c) *Reference.*—See appendix E for a brief discussion of the Glen Canyon Bridge as well as a presentation of cost data. More detailed information is given in the technical record of design and construction on the feature.[1]

136. SPECIFICATIONS NO. DC-6274—VISITOR CENTER COMPLEX AT GLEN CANYON DAM. The main features of the building are a steel-framed, one-story wing with basement, a steel-framed rotunda and concrete-lined elevator shaft. Precast panels with facings of exposed aggregate were used for the walls of the building wing and rotunda. A concrete-lined tunnel was constructed to connect the elevator shaft to the top of the dam. Other work included the construction of parking areas, roadways, water supply systems, and miscellaneous switchyard work in connection with the center electrical control and power systems.

(a) *Summary of Bids, Award of Contract, and Execution of the Work.*—The bid opening was held in Page, Ariz., on June 8, 1965, with six bids being received. The three lowest bids and the engineer's estimate are listed below:

(1) Allen M. Campbell Co. General Contractors, Inc. Tyler, Tex.	$1,123,000.00

[1] "Glen Canyon Bridge," Technical Record of Design and Construction, Bureau of Reclamation, 1959.

(2) Lembke Construction Co., Inc. Albuquerque, N. Mex.	$1,187,341.00
(3) The Brezina Construction Co., Inc. Rapid City, S. Dak.	$1,224,440.00
Engineer's estimate	$ 900,330.00

Award of contract No. 14-06-D-5634 was made to Allen M. Campbell Co. on June 30, 1965. The specifications provided for the completion of the work in two parts. Part (1) was for all work to complete the elevator lobby and shaft to permit installation of the elevator by the elevator contractor within 800 days. Part (2) was for the remainder of the work and was to be completed within 480 days. Notice to proceed was received by the contractor on July 12, 1965, and completion dates for parts (1) and (2) were established as April 26 and October 23, 1966, respectively. At the time notice to proceed was received, a statewide construction strike was in progress. This strike was not settled until about August 15 when a 5-year agreement was signed by the operating engineers. Construction began on August 16, 1965, with the erection of the contractor's field office.

The time allowed for completion of the contract was extended 41 days under part (1) and 53 days under part (2) due to delays in returning approval drawings. A findings of fact dated April 22, 1966, extended the time allowed for completion of parts (1) and (2) by 45 days. The time for completion of part (2) was also extended 56 days by a findings of fact dated April 27, 1967. All work was accepted as substantially complete on August 17, 1967.

(b) *Orders for Changes.*—There were four orders for changes under this contract totaling about $75,000. A full description of this work may be found in the final construction report (see bibliography).

137. SPECIFICATIONS NO. DC-6317—COMPLETION OF LEFT DIVERSION TUNNEL PLUG AND SPILLWAY ELBOW. The principal features of the work under these specifications included final excavation of the lower end of the spillway shaft, placing backfill concrete in adits, conduit, gate chamber and tunnel plug sections; placing spillway lining; drilling and grouting; and epoxy repairs to the existing lining.

(a) *Summary of Bids, Award of Contract, and Execution of the Work.*—Five bids were received. The bids were opened at Page, Ariz., on July 29, 1965. The three lowest bids and the engineer's estimate are listed below:

(1) S. S. Mullen, Inc. Seattle, Wash.	$1,535,333.13
(2) Martin K. Eby Construction Co., Inc. Phoenix, Ariz.	$1,985,016.52
(3) The Eagle Construction Corp. Loveland, Colo.	$2,066,298.13
Engineer's estimate	$1,638,423.13

Contract No. 14-06-D-5670 was awarded to S. S. Mullen, Inc., on August 6, 1965. The specifications allowed 300 days for completion of the work. Notice to proceed was received by the contractor on August 23, 1965, thereby establishing June 19, 1966, as the completion date. Order for changes No. 2 extended the contract completion time 95 days. A findings of fact, considering several miscellaneous delays, extended the time 76 additional days. The net earnings under the contract were $1,741,098.03.

(b) *Orders for Changes.*—There were two orders for changes under this contract. A brief description of this work may be found in the final construction report (see bibliography).

C. GOVERNMENT AND CONTRACTOR'S ORGANIZATION

138. CONTRACTOR'S ORGANIZATION. Major construction on Glen Canyon Dam, Powerplant, and Switchyard was completed by two main contractors, which have been designated as the prime contractor and the completion contractor. The prime contract was awarded to the Merritt-Chapman and Scott Corp. of New York City for work under specifications No. DC-4825 and the completion contract to Ets-Hokin and Galvan, Inc., of San Francsico under specifications No. DC-5750. The name of Ets-Hokin and Galvan, Inc., was later changed to the Ets-Hokin Corp.

(a) *Prime Contractor (Specifications No. DC-4825).*—The prime contractor's principal staff included a project manager, a project engineer, an office manager, a general superintendent, and an administrative assistant.

With the office manager were the chief timekeeper, the job accountant, and the chief warehouseman. Reporting to the general superintendent were his assistant, the shift superintendents, the rigging, carpenter, aggregate plant, concrete, batching plant, pipe, excavation and electrical superintendents, a master mechanic and an electrical engineer. A chief field engineer and an office engineer reported to the project engineer. The chief field engineer supervised a computer, a subcontract inspector, three survey parties, and an excavation man. The office engineer was in charge of a quantity engineer, materials engineer, an estimates man, two laboratory technicians, a mechanical and hydraulic engineer, a chief structural engineer, four assistant engineers, and a files and reproduction man.

Under the administrative assistant were a safety engineer, a cost engineer, a cost coder, a personnel and labor relations man, and the security guards which organization was under subcontract.

The maximum and average employment by the prime contractor for each year of construction follows:

Year	Maximum	Average
1957	744	453
1958	1,724	1,128
1959	1,458	572
1960	2,093	1,274
1961	2,401	1,939
1962	1,865	1,600
1963	1,636	995
1964	506	211

In addition, Merritt-Chapman and Scott Corp. had a large portion of the work performed by subcontractors. A partial list of these subcontractors and the work they performed is presented in the final construction report (see bibliography).

(b) *Completion Contract (Specifications No. DC-5750).*—The completion contractor, Ets-Hokin Corp., subcontracted the majority of its contract work to specialized contractors, performing only about 17 percent of the work with their own forces. Ets-Hokin's supervisory and administrative staff at the peak of their construction work consisted of a project manager, an office engineer, a superintendent, three assistant superintendents, an office manager, two accountants, two clerks, and two typists. Total employment under the completion contract, including subcontractors, during each year of the contract is shown in the following tabulation:

Year	Maximum	Average
1962	88	49
1963	350	170
1964	276	175
1965	82	45
1966	40	26

A partial list of Ets-Hokin Corp. subcontractors and the work they performed is presented in the final construction report (see bibliography).

139. GOVERNMENT ORGANIZATION. On July 1, 1956, a project construction engineer and one engineer were assigned to the Glen Canyon unit. Headquarters were established in the old high school building in Kanab, Utah, about 71 miles west of the damsite. The number of Government employees assigned to the project work totaled 144 by December 31, 1956. A geologist was initially detailed from the Bureau's Region 3 office at Boulder City, and he was later assigned to the project.

In the fall of 1958, after the municipal building and an adequate number of residences were completed to house employees, the project headquarters were moved to Page. Temporarily occupying the municipal building and two employee residences across the street from it, project headquarters were moved to the administration building a few months later in August 1959, when that building was completed. Procurement, property management, and materials control personnel moved into the warehouse offices at the time the headquarters moved to Page and remained there throughout construction.

During the succeeding years, a full scale construction organization was established as the expanding program demanded. This organization ultimately consisted of seven divisions, which were subdivided into branches and sections assigned to specific phases of the work. The seven divisions were: field engineering division, office engineering division, administrative division, city management division, special services division, safety management division, and on February 8, 1961, the transmission line division.

On March 25, 1964, the Glen Canyon unit construction office was consolidated with the Glen Canyon field division of the CRSP power operations office. As construction continued to be a dominant activity for the next 2 years, a project construction engineer remained at the head of the division, under the administrative direction of the Regional Director. He was the authorized representative of the Chief Engineer on construction matters, was responsible to the Project Power Manager at Montrose, Colo., on power operations and maintenance matters, and was responsible to the Regional Director concerning Page city management matters.

Construction functions and personnel were incorporated in a new construction branch which included a field engineering section, office engineering section, and transmission line section. The divisional structure of the former unit office was changed to branch designations composed of the following: construction, plant maintenance, plant operations, city management, and administrative services branches.

In early February 1967, all office personnel except those in city management, property management, and procurement moved to the permanent offices on the ninth floor of the powerplant.

On February 27, 1967, the Glen Canyon field division was reorganized for operations and maintenance status under the immediate direction of a division chief who was under the general direction of the Project Power Manager at Montrose, Colo. The chief of the construction branch, served under the general administrative direction of the chief of the Glen Canyon field division, and was responsible for the remaining construction of Glen Canyon Dam, Powerplant, and appurtenant facilities. He also served as authorized representative of the contracting officer on contracts for construction and for repair, installation, and modification of facilities in construction status.

During the peak of construction in 1962, the construction organization had as major components: field engineering, office engineering, administrative services and city management divisions. A maximum of 359 Government employees was reported at the year's end in 1962.

(a) *Field Engineering Division.*—The field engineering division, with the field engineer at the head, consisted of a dam inspection branch, a powerplant inspection branch, an electrical engineering branch, a concrete control branch, a mechanical engineering branch, and a surveys branch.

(1) *The dam inspection branch* consisted of a general inspection section, a grouting inspection section, and a cooling section. The general inspection section, consisting of 9 engineers and 21 inspectors, was responsible for inspection of all concrete placing in the dam and for inspection of related work under any other contracts on the project. The grouting inspection section, which consisted of two engineers and five inspectors, was responsible for inspection of all grouting in the dam foundation, the dam, and the diversion tunnels and spillways. The cooling section, consisting of two engineers and one technician, was responsible for inspection of the proper installation of the cooling system and for maintaining a check to insure that proper temperatures were maintained in the mass concrete that required cooling.

(2) *The electrical engineering branch* consisted of an engineer, a technician, two inspectors and one engineering aid, as well as the technical instruments section. The technical instruments section, consisting of seven technicians and four engineering aids, was responsible for the placement and reading all technical instruments in the dam. These instruments consisted of deformation meters, joint meters, stress meters, strain meters, no-stress strain meters, and resistance thermometers.

(3) *The powerplant inspection branch,* consisting of two engineers and nine inspectors, was responsible for inspection of all concrete placements and related work by the prime contractor on the powerplant.

(4) *The concrete control branch,* headed by an engineer and an assistant, consisted of 40 engineers, technicians, and inspectors of various grades and experience during the peak of construction. The branch chief and assistant were assisted by an office and laboratory chief technician and one supervisory engineer on each shift who coordinated and directed inspection and testing procedures. The branch was responsible for the investigation and selection of suitable concrete aggregate and soil materials; the design and inspection control of all aggregate and concrete production; sampling and testing for acceptance, all pozzolan and cement as delivered to the project; sampling and testing of other construction materials, including soils, used during construction of the dam and appurtenant structures; and preparation of all progress reports for the concrete control branch and preparation of data necessary for progress payment estimates with respect to concrete, soils, cement, and pozzolan.

(5) *The mechanical engineering branch* consisted of three engineers and one inspector reporting to an engineer who was chief of the branch. The branch was responsible for the installation of all mechanical equipment such as

penstocks, outlet gates, ring-follower gates, hollow-jet valves, radial gates in the spillway, spiral and inclined stairs, etc. One engineer in the branch was stationed at the Vinnell Steel fabrication plant at Flagstaff, Ariz., from August 10, 1959, to December 1, 1960, to supervise the fabrications and welding of the penstocks. The work consisted of inspection of stress relieving of the penstock sections, X-ray inspection of the welds, hydrostatic pressure tests, and expansion joint jacking tests.

(6) *The surveys branch* consisted of an engineer as chief of branch, six technicians, four survey aids, and one highscaler. A survey party of four began locating the west side access road from the Arizona-Utah State line to the damsite on May 29, 1956. The branch was responsible for the triangulation, precise level net at the damsite, making cross sections of canyon walls for dam and appurtenant works, setting lines and grades for forms, lot layout in commercial and residential areas of the townsite, and structural behavior measurements.

(b) *Office Engineering Division.*—The office engineering division consisted of an office engineer, a contract administration branch, a design and cost estimates branch, and a materials control branch.

(1) *The contract administration branch,* at the head of which was one engineer (with one clerk-typist), included a contract adjustment section and a cost summaries section. The contract adjustment section consisted, during the peak of construction, of two engineers, one engineering technician, and one general clerk and was responsible for negotiation with all contractors on all claims for extra compensation and delays and for the drafting of all orders for changes and findings of fact. The section was also responsible for bid openings and bid reports, for transmittal of all construction drawings and instructions to the contractor, and for furnishing drawings to the Government field forces. The contractors' and subcontractors' payrolls were checked for conformance to labor standards provisions. The Contractors' compliance with nondiscrimination requirements was also the responsibility of the contract adjustment section. A check on labor escalation payments claimed by the contractor was made after May 31, 1959.

The contract summaries section consisted of two engineers and two engineering technicians, with additional temporarily assigned personnel to meet peak workload demands, and was responsible for the preparation of all contract summaries and vouchers for monthly partial payments and final payments to construction contractors. The contract summaries section also checked the contractors' shop drawings, concrete placement drawings and reinforcement drawings, and reviewed the contractors' construction schedules for proper coordination of the work.

(2) *The design and cost estimates branch,* with an engineer in charge, included a computation and estimates section, a design section, and a drafting section. The computation and estimates section, consisting of five engineers and two engineering technicians, was responsible for computation of quantities for monthly and final pay estimates for all items of work concerning earthwork and concrete. The section was also responsible for all other computations relating to the project, reduction of all field survey data, and marking of prints relating to work in the section. The design section, consisting of two or three engineers, was responsible for all project layout work, preparation of grout pipe and drainhole drawings, and cooling pipe drawings, town lots, street grades, sewer and waterline alinement and profile, and the storm sewer alinement and profile. A number of drawings were revised after the Denver office had prepared the design in a general way or if a need for revision was critical, such as for excavation of the keyways. The right and left abutment duct bank design and the uplift pressure pipes layouts were also made in the design section. Preliminary design drawings were also made by the section and transmitted to the office of the contracting officer. The drafting section consisted of one drafting supervisor, one to three draftsmen, one clerk-typist, and was responsible for all project drafting, filing of all drawings, and furnishing of prints as needed on the project.

(3) *The materials control branch,* at the head of which was an engineer, consisted of one engineer, two engineering technicians, one laborer, and one clerk-typist. The branch was responsible for inspection and checking of all Government and contractor-furnished equipment received for all construction contracts. Where inspection of equipment was waived at the shipping point, inspection was made by the branch at the railhead or at Page if shipped by truck. This required the presence of one or more employees at the unloading point to check for damages, shortage, reloading if necessary, and for proper storage. A record was kept of all materials received, installed, and returned to the Government. In case of a shortage after the contractor had signed for equipment received, a charge would be made against the contractor.

Material was checked for conformance to Federal specifications or to manufacturers' catalogs when required. Checking of equipment, including all parts, against the drawing was especially important so that a construction contractor would not be delayed due to shortage of parts.

(c) *Administrative Services Division.*—The staff of the administrative services division, under the direction of an administrative officer, was responsible for all clerical and administrative functions in connection with project activities. The work included the procurement of a variety of Government-furnished supplies and equipment, and management, storing and accounting for both expendable and nonexpendable property; the final coordination of the project's function for budget, costs, time, leave and payroll, travel, collections for construction activities, and other fiscal functions; managing special tours, accommodations and local itineraries of foreign officials and dignitaries and special scientific groups; office services functions such as mail, files, stenographic services, telephone and teletype facilities, office building maintenance, records management and disposal; the personnel management program with delegated appointing authority and position classification through GS-11 and all wageboard positions; the preparation of budget documents for the various project features; the preparation of program schedules to reflect work accomplished and work contemplated; and the preparation of various technical and descriptive reports of the entire construction activity.

(d) *City Management Division.*—This division headed by a city administrator and assistant, consisted of a city engineering branch, under which was a city engineering and inspection section and an operation and maintenance section; a ranger branch; a fire control branch; and a city services branch. Page, Ariz., was established in connection with the construction of Glen Canyon Dam and Powerplant to provide necessary housing and community facilities and services for Government and contractors' employees engaged in the construction and operation of the dam, reservoir, and powerplant. Under the supervision of the city administrator, the division was responsible for representing the Government in the management affairs of the Federal Municipality of Page; the conduct of municipal affairs under applicable county and state laws and Federal regulations and statutes; development of specific programs for each phase of local affairs, including the encouragement of private business enterprises and residential development by facilitating the sale of commercial and residential building sites; devising and administering procedures for accomplishing eventual self-government under Arizona laws, such as zoning and disposal of lands and real property, tax structure and source of revenue for municipal financing under laws of incorporation, transfer of utilities, shops, and other public properties from Federal to municipal ownership, and articles of incorporation and charters and correlating economic and social interests of various citizen groups.

(e) *Organization.*—An organization chart as of December 31, 1961, is presented in figure 244. The maximum number of Government employees during the principal years of construction were as follows:

Year	Employees
1956	150
1957	211
1958	219
1959	229
1960	279
1961	324
1962	359
1963	355
1964	293

140. SAFETY. The accident prevention program was included as a specific part of the contractural obligation under specifications No. DC-4825. By reference, the Bureau of Reclamation manual, "Safety Requirements for Construction by Contract, Second Edition" was included in the specifications. During the tenure of the contract, a third edition of the manual was issued, which was readily accepted by the contractor. The "Manual of Accident Prevention in Construction," published by the Associated General Contractors of America, was also used by the contractor as a guide to establish safe practices in the work.

The project safety program was initiated in November of 1956, prior to initiation of work under the prime contract, continued throughout the life of the prime contract in late 1964, and then until completion of project work. Prior to construction, a conference was held with key representatives of the prime contractor and the Bureau to insure a clear understanding of the safety standards required and to eliminate any areas of procedural misunderstandings and probable conflict. In particular, the need for constant emphasis on accident prevention by the superintendents and foremen was stressed, responsibility for each area of work was established, and the necessity of a constant program of workman education was emphasized.

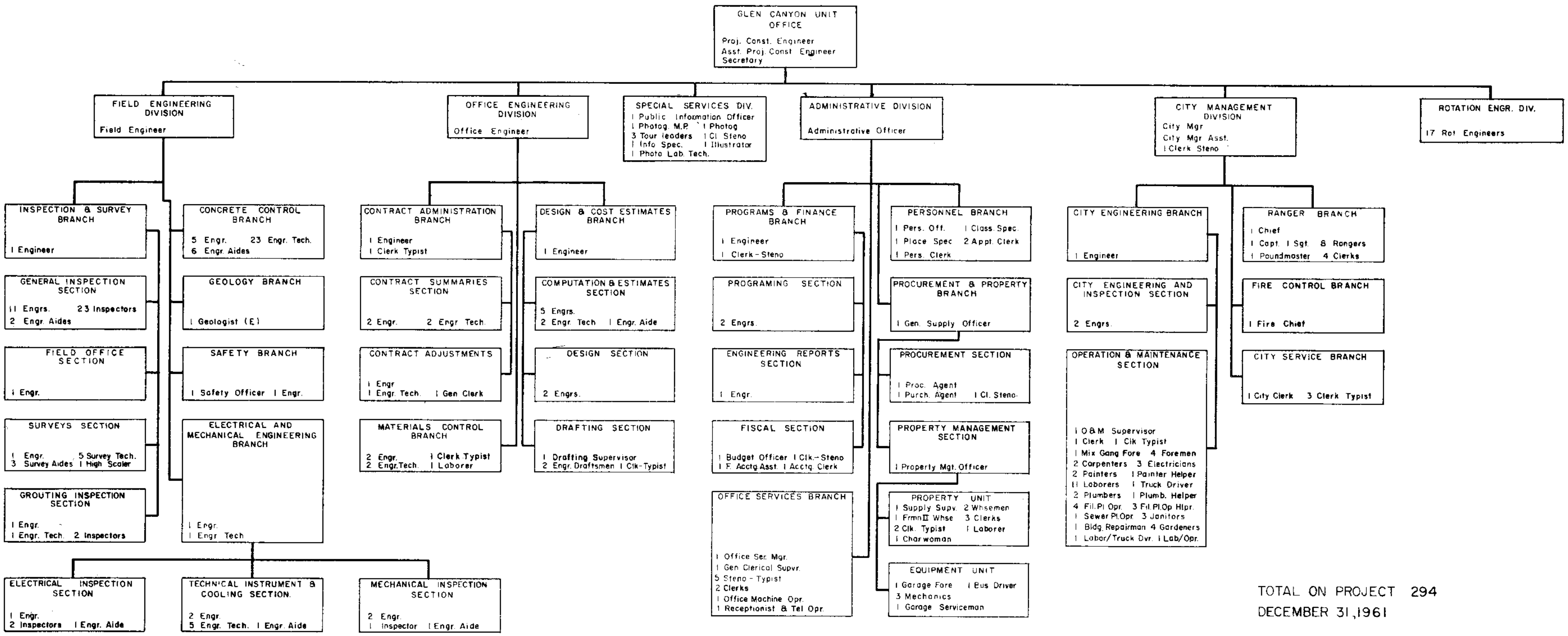


Figure 244.—Organization chart for Glen Canyon Unit as of December 31, 1961.

Details of the contractor's safety program were formulated by the full-time safety engineer employed by the contractor. Basically, the safety program recognized that most accidents were caused by human failure rather than mechanical failure, which was reflected in a continuing education and information program for the workmen. In addition, mandatory requirements were established to insure safe working environment, including the use of protective equipment furnished by the contractor for use by the individual employees.

Another basic part of the accident prevention, with respect to the individual employee, was an initial medical screening program instituted by the contractor. Performed prior to employment, the screening included a physical examination when appropriate depending upon the position or past history of the individual. There was a wide-spread acceptance of wearing the customary hardhats, hard-toed shoes, and safety goggles or glasses. This attitude was reflected in the general acceptance of the safety program by the employees as well as the individual good records. Use of measures to protect the skin from cement dust and other irritants was only gradually accepted, as a matter of routine course, but persistence by the foreman in briefing of new employees proved effective. There were very few cases in which employees had to be reminded that failure to follow standard safety procedures could result in termination of employment.

A joint safety policy committee was established to discuss problems of mutual interest involved in accident prevention. Meeting monthly, the committee was composed of key supervisory personnel from each of the active contractors at the site and Government representatives. Representing the largest employer with over 2,000 employees at peak construction, the prime contractor's safety engineer was a leader in the effectiveness of this committee. Policies or procedures established, as well as changes in past regulations, were also disseminated in this manner. At the workmen's level, the crew foreman, the key link in safety indoctrination, would inform the individual workmen of these new regulations and describe the implementation required. In turn, the workmen were encouraged to bring any potentially unsafe practices to the attention of the foreman, and the matter was forwarded for presentation to the safety committee when a general change in procedures or policy might be required. Review of subsequent accident reports also provided a source of information needed to avoid duplication of the same accident environment or condition. Visual aids were also used, including posters, signs, pamphlets, and films.

First aid kits were installed in close proximity to each area of work. All vehicles and pieces of major equipment were equipped with the first aid kits. Training and certification of foreman in the American Red Cross or Bureau of Mines first aid courses produced a capable group of workmen who assisted in immediate job evacuation of injured employees and in assuring early medical attention of injuries. Two fully equipped mobile trailers, along with two ambulances, were located immediately on either side of the job and were staffed by registered nurses on a 24-hour basis under the general supervision of a physician. A first aid skip was also provided for the cableways to evacuate severely injured employees from inaccessible areas on the damsite. Prompt and efficient treatment of minor wounds and injuries prevented development of more serious problems and also reduced the number of lost time injuries.

The specifications also provided that the contractor construct and staff a small hospital near the town of Page, as soon as possible after the award of the contract. This permanent-type hospital was to have a minimum capacity of 25 beds. Medical personnel, including doctors, nurses and technicians, were recruited and retained for operation of the hospital. Early opening of the hospital eliminated the occurrence of complications from delays in reaching distant medical facilities. The hospital was also open to the general public on a space available basis, who would pay for the services they received. Airport facilities were also utilized in a plan for rapid evacuation by air, when more complicated problems required the attention of the medical centers in the larger cities.

It is interesting to note that the contractor's accident program included orientation on offsite accidents. Recognizing that two offsite accidents happen for every onsite accident, the contractor also stressed home and highway safety in his basic program of accident prevention.

The greatest hazards in the work were related to the height differentials involved in construction. As a result, great emphasis was placed upon protection from falling rock and other objects, and upon precautions to prevent falls by the employees themselves. Although the accident prevention program was highly successful in this regard, as compared with other projects of this nature, injuries from these two causes accounted for over one-half of the lost-time accidents. Safety precautions for electrical powerlines and equipment were exceptionally vigorous, and injuries and accidents from this source were relatively small in number and minor in severity. Adequate ventilation was provided during tunnel excavation and tests for toxic gases were conducted at frequent intervals. Extensive onsite equipment and machinery repair facilities, coupled with the regular program of inspection and maintenance, considerably reduced accidents and injuries due to equipment failure. Adequate facilities for storage and handling of explosives were provided and maintained properly. Good housekeeping, the indicator of efficient working conditions and good safety habits, was continually stressed by the contractor with generally successful results. The aggregate haul road from the Wahweap screening plant to the stockpiles adjacent to the damsite was well designed and maintained by the prime contractor. As the haul road was 6 miles in length and along the canyon rim, only two well-regulated crossroads were necessary for cross traffic, one near each end of the road.

The safety programs of other contractors generally paralleled the program described for the prime contractor. The contracts were usually smaller and for more specialized work and the personnel responded readily to the weekly toolbox safety discussions. Little difficulty was experienced in maintaining adequate safety programs for these contracts, except for the inconvenience of the long travel distances involved in the transmission line contracts.

As a result of this program of accident prevention, contractors' employees experienced an overall average accident-frequency rate of 17.3 disabling injuries per million man-hours of exposure during the project construction period. A total of 14 fatalities occurred on the prime contract during the 7 years of tenure, nearly all of which resulted from errors in human judgment or freak accidents. There was one fatal accident among contractor employees during construction of the right diversion tunnel and two fatal accidents during construction of transmission lines under other separate contracts. In addition, there were two Bureau fatalities; however, one of these was caused by an act of God and was not chargeable against the project safety record. There was a total of 18 fatal accidents during construction of the entire project. During the construction of the bridge, there were no fatalities even though the bridge is the highest steel arch bridge of known record. In summary, there were 343 lost-time on-the-job injuries to all contractors' employees while performing 19,789,000 man-hours of hazardous construction.

In addition to the contractors' safety efforts, a program of accident prevention was pursued by the Government employees. Although particular emphasis was placed on vehicular safety and first aid training, all contractors' safety programs were observed and encouraged by engineering and inspection personnel as a matter of good practice, as well as setting an example. Maintenance employees followed a program of accident prevention similar to those of the contractor, including weekly toolbox safety meetings. There was a total of 22 lost-time injuries to Government employees during the 5,105,000 man-hours worked, or an accident frequency rate of 4.3 disabling injuries per million working hours.

The contractor's and the Government employees lost-time accident summaries are shown in tables 1 and 2, respectively.

TABLE 1.—*Lost-time accident summary for contractor employees.*

Year	Man-hour exposure	Number of injuries		Days lost	Frequency rate	Severity rate
		Disabling	Fatal			
1957	1,044,753	18	3	18,403	20.1	17,614
1958	2,805,346	43	2	16,970	16.0	6,049
1959	1,113,625	13	1	6,315	12.6	5,670
1960	2,422,059	39	4	25,493	17.7	10,112
1961	3,771,901	63	3	21,105	17.5	5,595
1962	3,356,333	68	1	8,911	20.3	2,655
1963	3,259,369	50	2	13,653	15.3	4,189
1964	1,222,632	24	0	1,386	19.6	1,133
1965	387,548	4	1	6,059	12.9	15,634
1966	405,681	4	0	188	9.9	463
Totals	19,789,247	326	17	118,483	17.3	5,987

TABLE 2.—*Lost-time accident summary for Government employees.*

Year	Man-hour exposure	Number of injuries		Days lost	Frequency rate	Severity rate
		Disabling	Fatal			
1957	354,585	2	0	98	5.6	276
1958	426,978	4	0	78	9.4	183
1959	433,072	2	0	44	4.6	101
1960	514,320	2	1	6,014	5.8	11,693
1961	650,600	5	0	57	7.7	88
1962	705,966	4	0	794	5.7	1,125
1963	706,022	1	0	230	1.4	326
1964	527,012	2	0	39	3.8	74
1965	447,989	0	0	0	0	0
1966	338,020	0	0	0	0	0
Totals	5,104,564	22	1	7,354	4.5	1,441

141. LABOR RELATIONS AND CONTRACTOR'S WAGE RATES. (a) *Labor.*—Although the Glen Canyon unit is in an isolated and remote location, approximately 135 miles from the nearest Arizona town of any size, an adequate supply of labor was generally available for the construction work. Recruitment of construction labor by the contractors was, in general, through the union halls of the various crafts, either in Flagstaff or Phoenix. The heavy demand for construction labor at Glen Canyon had a significant effect on the labor situation in Arizona and undoubtedly contributed to the acceleration of wage rate increases and fringe benefits which occurred in Arizona during the period of construction at Glen Canyon.

Wage rates more than doubled during this period as can be seen by table 3 which shows the rates paid each craft in 1956 and in 1964. Approximately 80 million dollars in gross wages were paid by contractors working at the site during the construction period. Of this total, Merritt-Chapman and Scott Corp., the prime contractor on the dam and powerplant, and its subcontractors paid a total of 68.3 million dollars in gross wages, or approximately one-half of the final contract value. Total employment under this contract by year is shown in section 138.

At the time work began on the Glen Canyon unit, most of the construction crafts received subsistence payments when working at distant locations from the union halls. Typical of the working agreements in effect at the time work started was the statewide Master Labor Agreement between the Associated General Contractors and the unions representing the five basic crafts (carpenters, teamsters, operating engineers, cement masons, and laborers). This agreement contained a "Remote Projects" provision which required that when a project was declared remote, as determined by a special Remote Projects Committee made up of members of both parties, the contractor would have the option of providing a camp at the jobsite or of providing free transportation from the nearest town where suitable living accommodations were available. If a camp was provided, no charge was to be made to employees for the board and room facilities provided. If free transportation was provided, travel time was to be paid at the regular straight time rate. The Remote Projects Committee established under this provision designated the Glen Canyon project as remote on December 12, 1956, and advised that the project would remain remote until 30 days after the committee found that residential housing facilities were available for those employees who wished to avail themselves of these facilities.

Early contractors at Glen Canyon operated under the Master Labor Agreement and complied with the requirement of the Remote Projects provision. Free board and room was provided, usually through a concessioner.

The prime contractor for the dam and powerplant, Merritt-Chapman and Scott Corp., arrived in late spring of 1957, and established temporary headquarters at Kanab, Utah, some 75 miles west of the site of the work. Merritt-Chapman and Scott expected to operate under the terms of its national agreement with the various crafts and did not anticipate becoming a signatory party to the existing statewide Master Labor Agreements. But when Merritt-Chapman and Scott contacted the local unions to make arrangements for a supply of labor for the job, the contractor was required to sign the Master Labor Agreements as a condition to the union's furnishing labor. In order to comply with the Remote Projects provision, the contractor immediately began construction of barracks building, a messhall and a trailer court, and multiple-unit type dwellings. Construction was also started on the hospital that was required by the construction contract with the Government. Hoping to be relieved of the requirement for furnishing free board and room, the contractor requested the Remote Projects Committee, late in 1957, to declare the project no longer remote. Because of a tie vote on the question in the committee the matter was submitted to arbitration. The arbitrator's decision was that the contractor had not made adequate residential facilities available. After the contractor had taken steps to correct the deficiencies found by the arbitrator, and had completed the hospital, he again sought to have the project removed from the remote classification. After the arbitrator ruled, on January 15, 1959, that adequate residential facilities were available the contractor advised his employees that free room and board would cease on January 25, 1959. Members of the five basic crafts walked off the job immediately, shutting down the work for several days. The workers voluntarily returned to work, a few at a time, until full operations were resumed, even though free board and room were no longer furnished.

The Master Labor Agreement, in effect at the time, expired on May 31, 1959. The new statewide agreement, in addition to providing wage increases of about 20 cents per hour for each of the next 3 years, discontinued the Remote Projects provision and required payment of expense allowances for work performed at locations a certain distance from several established basing points. The expense allowance varied by zones, Glen Canyon being in a zone requiring payment of $6.00 per day. The contractor agreed to pay the new basic wage rates established in the agreement, but rejected the requirement for payment of the expense allowance, and attempted to negotiate a separate agreement for Glen Canyon. Members of the crafts involved continued to work without payment of any subsistence or expense allowance until July 6, 1959, when union pickets were established and the workers did not report for work. Except for work that was necessary for maintenance, work under the contract was at a standstill the remainder of the year. The contractor's principal objection to the new statewide agreement was the requirement for payment of a subsistence allowance. The escalation clause in its contract with the Government provided for reimbursement to the contractor of 85 percent of the cost of wage increases above the basic rates established in the Davis-Bacon decision applicable to the work, but specifically excluded reimbursement of costs for subsistence paid to employees. Eventually a separate job agreement for Glen Canyon was accepted by the union and the contractor, and work under the contract was resumed on January 4, 1960. In the new agreement, wages of the five basic crafts were established as 50 cents an hour higher than required under the statewide agreement and no expense or subsistence was required. Higher rates at Glen Canyon thus became established and continued throughout most of the remaining construction period at Glen Canyon. The electricians agreement, which prior to June 1958 had required payment of $7.00 per day subsistence in areas away from town, had already dropped the requirement for subsistence but at the same time called for substantial increases in wages. Other crafts, after a separate agreement was made for Glen Canyon for the five basic crafts, soon followed the pattern of converting the subsistence payments to wages.

The Government refused to pay the increases in labor costs which were attributed to the wage increases paid in lieu of subsistence. The contractor submitted a claim for these amounts on which reimbursement was not allowed. By date of April 20, 1960, the contracting officer issued a findings of fact and a decision which denied the contractor's claim for additional labor costs in connection with the five basic crafts and electricians.

A similar findings and decision was issued by the contracting officer, under date of January 9, 1963, to cover wage increases on painters, plumbers, pipefitters, sheetmetal workers and ironworkers. The contractor appealed both of these decisions to the Board of Contract Appeals who eventually, after lengthy hearings and considerations, reversed the contracting officer's decisions and granted the contractor's claim on the wages paid to the five basic crafts in its entirety, and allowed a portion of that claimed on other crafts.

The total of all escalation paid to the contractor for increased wages paid by him and his subcontractors, including the amounts allowed under the claims, was $11,553,565.53 (see appendix B).

There are a number of features in the labor agreement between the prime contractor, Merritt-Chapman and Scott Corp., and the unions of the five basic crafts which are noteworthy:

(1) All workmen would be requisitioned through the local union halls, in Phoenix, Ariz., for most crafts, but if the union was unable to furnish workmen within 48 hours the contractor could obtain them from other sources, provided

arrangements were made promptly for proper referral of workmen after they are hired. However, the agreement did permit the hiring of key or specialized personnel without regard to the above requirements where such hiring was agreed to in a conference between the contractor and union.

(2) When a workman was dispatched from the union hall the contractor was required to pay travel expenses from the union hall, or the employee's home if closer, to the jobsite and back to the beginning point after his termination. This amounted to 5 cents per mile plus wages depending on the distance traveled. Payment of these travel expenses was also required when the workman was rejected when arriving at the worksite. However, if an employee quit before working 5 days, no travel time pay was required to be paid.

(3) All work performed in excess of 8 hours per day or 40 hours per week was to be paid at one and one-half times the straight time rate. Time and one-half was also to be paid for Saturday work and for time worked before and after the established starting and ending times for the shift. Work on Sundays or holidays was to be paid for at twice the straight time rate.

(4) When two shifts were employed the second shift received 8 hours pay for 7-1/2 hours worked. On three-shift operations the third shift received 8 hours pay for 7 hours of work. Otherwise, no pay differential was to be made on shift work. Tunnel workers on any shift, however, were required to work only 7 hours for 8 hours of pay.

(5) The agreement specified the rates to be charged for food and living quarters as follows:

- Cafeteria—$21.50—minimum of 5 days per week at $4.30 per day
- Lodging in dormitory—$7 per week
- Apartment units—$75 per month for one-bedroom units, $80 per month for two-bedroom units, $100 per month for three-bedroom units
- Company trailers (rental or rental purchase) $60 per month by payroll deduction (a $25 deposit required)
- Trailer space—$40 per month

The rates for trailer space were to be paid in advance and included electricity, water, sewage, and garbage disposal.

(6) A workman who reported for work when job conditions prevented working received 2 hours show-up pay unless notified not to report before the end of his last shift worked. If work was provided the reporting employee was to receive at least 4 hours pay, and if more than 4 hours were worked he was to receive at least 8 hours pay unless prevented from working by reasons beyond the control of the contractor, such as inclement weather, completion of the job, or breakdown of operations.

The agreement was to remain in effect for 5 years, but provided that if either party desired to change that party should give written notice to such effect 60 days prior to June 15, 1962. However, since the agreement established rates for only 3 years, a new agreement was made on June 16, 1962, which established new and higher rates for the next 3 years. No other significant changes were made.

By the time the agreement expired in June 1965, the contractor had completed his work and no further special job agreements were established for Glen Canyon. For most crafts, work was subsequently performed under the statewide agreements which by this time had nearly all established separate rates for northern Arizona or had zone rates determined by the distance from local union halls.

Other than the 6-month strike in 1959, the only other significant delays due to strikes during the construction at Glen Canyon occurred during 1965 when failure to reach new statewide agreements in Arizona delayed the start of work on the visitor center and the completion of the left spillway for several weeks. The work on the completion contract was nearly completed by this time and was not affected. Work on the Flagstaff Substation was also delayed by the strike, and the start of footing construction for the second Glen Canyon-Flagstaff 345-kilovolt transmission line was also delayed. Although jurisdictional disputes were fairly common, they actually stopped the work on only a few occasions and then for only 1 or 2 days at a time.

There were no significant problems in enforcement of labor standards provisions of the contracts, particularly with the large contractors who had prior experience on Government contracts. Minor difficulties with misclassification, unauthorized deductions, failure to pay overtime, and failure to include labor standards provisions in subcontracts, were encountered with some of the smaller contractors who had little previous experience on Government work.

On all contracts, labor standards provisions compliance was emphasized during preconstruction conferences, labor classification checks were made weekly in the field, and all payrolls were checked for compliance and accuracy. In addition, each contractor was urged by letter to hire the local Navajo Indian people as much as possible, and substantial numbers of them were employed in the work on the dam and powerplant and the transmission line work. Most of them worked in the various laborer classifications, but some were employed as teamsters, cement finishers, and ironworkers.

(b) *Contractor's Wage Rates*—The contractor's wage rates for 1956 and 1964 are presented in table 3.

TABLE 3.—*Classification and wage rates for contractor personnel.*

Classification	Rate 1956	Rate 1964
Carpenter	3.03	5.05
Carpenter-welder	3.155	5.175
Saw filer	3.03	5.05
Saw operator	3.28	5.30
Millwright	3.28	5.10 (1963)[1]
Carpenter helper	2.18	2.36 (1958)
Ironworker, structural	3.25	5.80
Ironworker, splicer	3.50	6.05
Batch plant operator	2.995	5.07
Field equipment serviceman	2.675	4.64
Equipment serviceman helper	2.415	3.495 (1960)
Heavy duty repairman	2.995	5.07
Heavy duty repairman helper	2.415	4.30
Heavy duty mechanic	2.995	5.07
Heavy duty mechanic helper	2.415	3.525 (1960)
Air tool mechanic	2.995	5.07
Heavy duty welder	2.995	5.07
Heavy duty welder helper	2.415	3.495 (1960)
Heavy duty lathe operator	2.995	5.07
Shovel operator	3.17	5.32
Crane operator	3.17	5.32
Motor crane operator	3.17	5.57
Whirley crane operator	3.17	5.57
Whirley crane oiler	2.415	4.55
Oiler	2.415	4.30
Compressor operator	2.415	4.30
Bulldozer operator	2.995	5.07
Dragline operator	3.17	5.32
Pump operator	2.415	4.30
Pumpcrete operator	2.675	5.07
Electric hoist operator	2.835	4.86
Cableway operator (hiline), 20 tons or more	3.17	5.32
Cableway operator, less than 20 tons	3.17	5.32
Tugger operator	2.995	5.07
Drum and hoist operator	2.995	5.07
Motor grader operator	2.995	5.32
Pickup driver	2.29	4.11
Flatrack driver, 20 tons or more	2.625	4.475
Dump truck operator, 16 tons or more	2.625	4.475
Bus driver	2.29	4.11
Flatrack truck driver, less than 10 tons	2.29	4.11
Flatrack truck driver, 15-20 tons	2.415	4.245
Dump truck driver, less than 16 tons	2.475	4.30
Forklift operator	2.675	4.64
Skip loader operator	3.17	5.32
Dump truck driver, 16 cu.yd. or over	2.265	4.475
Dumpster driver	2.39	2.59 (1958)
Semitruck driver	2.625	2.845 (1958)
Lowboy driver	2.625	2.845 (1958)
Machinist	2.995	5.32
Cement mason	2.96	4.90
Cement finisher tender	2.265	4.075
Wagon driller	2.70	4.55
Air track driller	2.70	4.55
Chuck tender	2.34	4.16
Jackhammer man	2.50	4.33
Scaler driller	2.785	4.78
Signalman	2.18	2.54 (1959)
Powderman	2.70	4.55
Powderman helper	2.34	4.16
Miner	2.60	3.865 (1960)
Laborer	2.18	3.99
Air tool operator	2.39	4.22
Vibrator operator	2.39	4.22
Nozzleman, air and water	2.39	4.33
Sandblaster nozzleman	2.70	5.02

[1] Where certain classifications were not employed in 1964, their last year on the job has been indicated with their hourly rate at that time.

Classification	Rate 1956	1964
Painter	2.80	5.00
Pot tender (sandblast)	2.34	4.24
Warehouseman	2.265	4.075
Conveyor operator	2.525	4.25 (1963)
Dinkey operator	2.675	4.44 (1963)
Tireman	2.34	4.16
Pipefitter	3.35	5.70
Plumber	3.35	5.70
Motor crane driver	2.675	4.89
Winch truck driver, less than 10 tons	2.29	3.86 (1962)
Roofer	2.76	3.80 (1962)
Electrician and lineman	3.325	6.20
Groundman	2.725	5.08

Apprentices were used throughout most of the job and they were paid in accordance with the apprentice program at rates required by the specifications or the State of Arizona, union rates or precentages of the mechanic's pay.

D. LAND AND LAND RIGHTS

142. LAND ACQUISITION. Land and land rights have all been acquired for Glen Canyon Dam and Reservoir at the time of this writing except four parcels for which condemnation proceedings have been initiated. Included in the properties acquired and reported herein are mining interests, withdrawn public land, state land privately owned land, and termination of grazing permits on public land.

The total acreage acquired from private interests is 763.54 acres representing a payment of $107,905. The area for which condemnation was initiated is 176.39 acres at an estimated combined value of $13,315. Land acquired from the State of Utah, by court action, comprised 10,039 acres at a cost of $125,000. There were 1,177,610 acres withdrawn from public lands which include property acquired under Public Law 85-868 (72 Stat. 1686) (51,606.78 acres) from the Navajo Indians as shown below.

The Bureau of Land Management (BLM), through agreement with the Bureau of Reclamation and on a reimbursable basis, investigated a total of 7,531 mining claims in the Glen Canyon withdrawal area. Of that number, 9 claims were patented and 4 others determined to be valid; these 13 claims were purchased by the Bureau. Four hundred ninety-three of the claims investigated were not taken to final validity determinations (although several of these were considered valid or ostensibly valid) since they were considerably above the reservoir basin and outside the area considered essential to the project from a Reclamation standpoint. A total of 7,025 of the claims were invalid either by acquiescence on the part of the owners or through contest hearing procedures. Total cost to the Bureau for the BLM investigation work in the Glen Canyon withdrawal area was $384,207.00.

Under Public Law 85-868 (72 Stat. 1686), an Act "to provide for the exchange of lands between the United States and Navajo Tribe," 51,606.78 acres were acquired from the Navajo Indians in exchange for which the Navajos received virtually the same acreage on McCracken Mesa in Utah. As provided by said Act, these exchanged lands have the status of ordinary Reclamation-withdrawn public lands. In connection with the exchange, the grazing permittees and owners of range improvements on the McCracken Mesa public lands were compensated by the Bureau for their permits which were canceled to accommodate the exchange at a total cost of $144,000.00. In addition, the BLM, through agreement with the Bureau of Reclamation and on a reimbursable basis, investigated the mining claims on the Mesa; a considerable number of claims were involved but all were finally determined to be invalid. The BLM was reimbursed $29,678.58 by the Bureau of Reclamation for this work. Approximately 30 individual Indian owners of improvements on the 51,606.78 acres of Tribal lands acquired under P. L. 85-868 were compensated for their improvements at a total cost to the Bureau of $11,840.00.

Altogether the acreages amount to 1,188,588.93 at a total acquisition cost of $1,252,721. The final total cost will change consonant with the outcome of the ensuing litigation.

143. LAND FOR ESTABLISHMENT OF THE CITY OF PAGE. Public Law 85-868, September 2, 1958, transferred to the United States all the right, title, and interest of the Navajo Tribe in and to certain lands required for the Colorado River Storage project, including the land required for the establishment of a city for construction and operation and maintenance purposes, named Page, Ariz.

Prior to the enactment of P. L. 85-868, land at Page could be made available for private use only on a lease basis under Section 10 of the Reclamation Project Act of 1939. Public Law 85-868 authorized the Secretary to dispose of lots in townsites established on the lands transferred under P. L. 85-868 under such terms and conditions as he determined appropriate, but at not less than the current fair market value, except for dedication of lands for public purposes. Subsequent to the enactment of P. L. 85-868, lots at Page, Ariz., have been sold at appraised values determined by FHA and

Bureau appraisers. Under the same authority, Government-owned houses are sold at FHA appraised values.

144. ARCHAEOLOGICAL INVESTIGATIONS. Archaeological investigation and salvage programs were conducted in the Glen Canyon Reservoir (Lake Powell) area and environs under two arrangements:

(1) The antiquities act of June 8, 1906, (34 Stat. 225) and the accompanying "Uniform Rules and Regulations", prescribed by the Secretaries of the Interior, Agriculture, and War. This Act authorizes the Secretaries to issue archaeological survey and salvage permits to reputable museums, universities, colleges, or other recognized scientific or educational institutions or their duly authorized agents. The Secretary of the Interior has delegated the issuing of these permits to the Chief Clerk of the Department.

(2) The Historic Sites Act of August 21, 1935, (49 Stat. 666). This Act provides that the Secretary of the Interior, through the National Park Service, shall conduct a program for the preservation of historic American sites, buildings, objects and antiquities of national significance.

In 1955, the Chief Clerk of the Department of Interior issued a permit to the Museum of Northern Arizona for archaeological investigations of the San Juan River arm of Glen Canyon Reservoir. In 1956, a permit was issued to the University of Utah for archaeological surveys along the Colorado River and the tributary stream basins that would be affected by future storage in Glen Canyon Reservoir.

During 1956 the National Park Service entered into memoranda of agreement with the Museum of Northern Arizona and the University of Utah for continuing archaeological surveys and salvage operations throughout the Glen Canyon Reservoir area and environs. One of the memoranda of agreement provided for the Museum to carry on an emergency salvage operation at station 371 of the Bitter Springs Glen Canyon access road.

Staff members of the Museum of Northern Arizona and the Department of Anthropology of the University of Utah have prepared numerous articles and publications presenting the results of their studies in the Glen Canyon area. A partial listing of this material is presented in appendix H.

E. CONSTRUCTION SUPPORT FACILITIES

145. REFERENCE. The construction support facilities for construction of the Glen Canyon complex were many and varied. Certain of these facilities, such as service, municipal, housing, and public use, will not be discussed in this publication. Information on the above listed facilities is included in the final construction report, an unpublished internal document (see appendix H).

146. CONSTRUCTION PLANT—GENERAL. The prime contractor, Merritt-Chapman and Scott Corp., obtained a large amount of equipment for a plant to construct Glen Canyon Dam and Powerplant. The largest portion of this plant was for concrete production facilities, which included an aggregate processing plant, a concrete mixing plant, a cooling plant, and cement and pozzolan storage. Some of the major items of other construction support facilities were traveling cableways, a temporary powerplant, monkey slides and footbridges, and carpentry, reinforcing steel, and office and equipment repair shop areas. In addition, the contractor provided housing facilities for employees, such as trailer courts, barracks, apartments and houses, as well as concessions and messhalls.

Some of the points of special interest in the contractor's construction plant were 12-cubic-yard concrete placing buckets transported by high-speed, 50-ton traveling cableways, a vertically arranged batching plant 217 feet high, a large refrigeration plant for concrete and aggregate cooling, and an aggregate processing plant which incorporated heavy media separation. Operations and further details of the major plant items are included in the description of the construction of the appropriate project features in this publication.

In accordance with contract provisions, payment in the amount of $4,500,000 for preparatory work was advanced to the contractor for the special plant and equipment necessary for processing, conveying and storing of aggregates; storing cement and pozzolan; batching, mixing, and transporting concrete; concrete and aggregate cooling plants; cableway structures and cableways for river crossings, or other equipment employed to place concrete. Subsequent repayment of the above amount to the Government was made by deducting $1.25 per cubic yard from payments made to the contractor for mass concrete in the dam.

Figure 245.—Aerial view of aggregate processing plant on ledge above Wahweap Creek. P557-420-5568, December 14, 1960.

147. AGGREGATE PROCESSING PLANT AND AGGREGATE STORAGE. The aggregate processing plant (fig. 245) was located on a ledge above Wahweap Creek about 6 miles from the damsite on the lower end of a natural deposit some 7 miles long and averaging about one-fourth of a mile in width. The basic components of the plant consisted of the following items:

Quantity	Item	Size or capacity
1	Truck unloading hopper	100 cu. yd.
1	Scalping grizzly	Plus 6-inch
1	Vibrating screen	Double deck, consisting of 3-1/2- and 1-3/4-inch mesh
1	Vibrating screen	Triple deck, 7/8-inch, No. 4, and 1/8-inch mesh
	Hydraulic sand sizers	–
1	Vibrating screen	No. 8 mesh
1	Dewatering screw	Twin-spiral
1	Heavy media separation unit	No. 4 to No. 8 sand
1	Vibrating screen	Twin deck, 3/4-inch to No. 4 mesh
–	Conveyors	
–	Truck loading tunnel	
–	Water supply and distribution system	

Finished stockpiles at the aggregate plant consisted of eight stockpiles of the following maximum sizes; 6-inch, 3-inch, 1-1/2-inch, 3/4-inch, 3/4-inch heavy-media-treated, 1-1/2-inch heavy-media-treated, graded natural sand, and graded heavy-media-treated sand. Larger stockpiles in three finished sizes were also provided at the construction site, and the material was delivered to the batching plant by way of conveyor belts in a reclaiming tunnel underneath the piles.

148. CONCRETE BATCHING AND MIXING PLANT. The main concrete batching and mixing plant (fig. 246) for construction of Glen Canyon Dam and Powerplant was designed by the Noble Co. of Oakland, Calif. This plant was designed to produce concrete at an average rate of 420 cubic yards per hour, or a maximum rate of 480 yards per hour for short periods of time, using automation as far as possible. The octagon-shaped mixing plant had six main floors, served by an exterior man-lift, and was 217 feet high. Large 3,000-ton-capacity aggregate storage bins, containing eight bin compartments for two types of sand and the six sizes of coarse aggregate, were located in the middle of the plant. The screening tower and chutes were located in the portion of the plant above the storage bin, and the batching and concrete mixing floors were located below the storage bin. On the mixer floors, six tilting mixers were installed in a circular pattern, each having a capacity of 4 cubic yards. The mixers were discharged into three separate holding hoppers, which in turn were loaded into 12-cubic yard ladle buckets on electric-driven transfer trains on a trestle below the plant. The primary economic reason for locating the mixing plant on the bench at elevation 3540 was that over 80 percent of all concrete would be placed by cableways to areas below this elevation.

Figure 246.—Concrete batching plant being set up on west rim of Glen Canyon. P557-420-4781, April 29, 1960.

149. OTHER CONCRETE PRODUCTION COMPONENTS. On a ledge above and just downstream of the batching plant, a refrigeration plant was constructed to provide cold air, cold water, and crushed ice for use in lowering the temperature of the plastic concrete. This plant also served in cooling the placed concrete during hydration and post-cooling of the main concrete for contraction joint grouting. More than 4,000 tons of refrigeration with a power installation of 6,275 horsepower was required. The plant included a battery of 18 ammonia compressors, eight 48-inch condensors, and a cooling tower, which were supplemented by 10 shell and tube chillers. Fourteen of the 18 compressors were arranged for multipurpose use as follows: eight 600-horsepower, one 450-horsepower, two 150-horsepower, two 100-horsepower, and a 25-horsepower compressor.

The other four 125-horsepower compressors were used for the 22 "shaved" ice-making machines. Two of the shell and tube chillers were 32 inches in diameter and were used to chill the mixing water, three 52-inch units chilled water for cooling the coarse aggregate on the conveyor belt, and the other five cooled circulating water for the cooling pipes and the dam and were 38 inches in diameter.

Seven silos were provided to store bulk cement and pozzolan and were located on the canyon rim, above the batching plant. Each of the four cement silos had a capacity of 10,000 barrels of cement and each of three pozzolan silos held 1,000 tons, a capacity of 40,000 cubic feet. Both cement and pozzolan were delivered to the site in special trailers, which were bottom dumped into a screw conveyor which lifted the materials to air slides and transported it to the silos. The mixing plant received both cement and pozzolan by screw conveyors from the bottom of the silos, and the cementing materials were transported by air slides to balancing storage at the batching plant. Bulk storage tanks for calcium chloride solution, air entraining agent, and liquid water reducing agent were located on the canyon rim adjacent to the plant, and were piped for gravity flow to the batching floor.

150. CABLEWAYS. Two 50-ton-capacity cableways were used for placing concrete in the dam. The cableways were equipped with moving head and tail towers set on two sets of parallel tracks. One of the cableways had higher towers to enable it to cross over the other cableway so that each could be operated independently and simultaneously. This cableway system was designed for the contractor by Lidgerwood Industries and was the largest of this type ever used in dam construction. A used, low 25-ton cableway was erected and placed in operation later, primarily for powerplant use in concrete placement, and subsequently for use as a general service crane.

The higher of the two 50-ton cableways had a cable span of 2,050 feet between the head tower and the tail tower. The lower 50-ton cableway had a span of 1,800 feet between towers, both load-carrying cables being 4-inch lock coils. These two cableways were designed for a normal capacity of 31 tons at a normal operating speed and condition, and up to 50 tons each at a reduced speed and greater main cable sag. The vertical velocity of the fall block with a hook load of 31 tons was 600 feet per minute, and about half this velocity at full load. Further details of the cableways, as well as operations, are included in section 174.

151. TEMPORARY POWERPLANT. Under a supplemental agreement, the prime contractor furnished electrical energy for construction and for the townsite. A temporary powerplant was built on the right abutment which initially housed four 1,360-kilowatt electric generators. Ten diesel powered generators of 1,100-kilowatt capacity each were added later, for a total rated plant capacity of 16,440 kilowatts.

Under the specifications, the Government was to furnish electric power for construction purposes to the contractor at the rate of 15 mills per kilowatt-hour. Under the supplemental agreement, the contractor provided power for his own construction purposes and would also furnish up to 3,000 kilowatts to the Government at the rate of 13.5 mills per kilowatt-hour, subject to adjustment, and the Government advanced $2,100,000 to the contractor which was to be repaid by deduction of 50 cents per cubic yard for mass concrete placed until the full amount of the advancement had been returned. By an amendatory agreement to the supplemental agreement, the power was delivered in the service area of Page, Ariz., rather than at the contractor's generator-plant switchyard adjacent to the temporary generator plant. The Government paid $33,125.46 or 51.546 percent of the cost of the transmission line and other facilities.

Under invitation No. 400S-97, Arizona Public Service Co. constructed, owned, and maintained all of the distribution facilities, including a street lighting system, required for the purpose of furnishing electric service in the Government areas of Page, Ariz. Power and energy required for such electric service was furnished by the Government through purchase from the prime contractor. Arizona Public Service Co. constructed a 69-kilovolt transmission line from Cameron, Ariz., to Page and the line was energized on the weekend of April 22, 1961. Electric energy for both the townsite and the construction work was thereafter supplied through this line. At the end of construction of the Glen Canyon unit the contractor dismantled the temporary powerplant.

152. OTHER CONSTRUCTION SUPPORT FACILITIES. The contractor also constructed a number of other buildings for use as office, shop, and storage facilities for the various construction needs. These buildings were generally made of the "Butler-type" construction with steel panels on structural steel bents and concrete floors. These buildings were used for the office space, equipment repair shops, and service shops, which were located on the west side of the river; and for the warehousing, electrical, plumbing, carpentry, and reinforcing steel bending shops on the east side of the river. Many miles of access roadways and large areas were graded and gravel surfaced for transporting, handling, and storage of equipment and materials on both sides of the canyon. In addition, the contractor constructed a large number of small buildings, usually corrugated metal on wood frames, for use as small field offices and tool rooms. These buildings were usually constructed so that they could be moved by cableway or mobile crane to various locations about the work. Small powder shelters were also provided a safe distance from the construction site on the west side of the canyon. First aid facilities were provided at appropriate locations, including fully equipped trailers on each side of the worksite, as discussed further under the section of this report on safety matters.

A steel-cable suspension-type footbridge (fig. 247) was constructed in October 1957, which spanned the canyon from the top of the canyon walls upstream

Figure 247.—View of prime contractor's highline cableway being used to move trailers from west side to east side of the Colorado River. Footbridge is in foreground of trailer being taken across. P557-420-1721, February 14, 1958.

from the damsite. Two temporary wood catwalks on cables were also built across the canyon, about 30 feet above the river, one near the downstream portals of the diversion tunnels and the other near the upstream portals. These lower catwalks were in use until the time of the diversion of the river, after which they were removed.

Two electric-powered monkey slides were erected on the east side of the canyon. One was located upstream of the left abutment of the highway bridge and the other just downstream of the upstream portal of the left diversion tunnel. The location of the monkey slides was later changed; one pair was placed in the left keyway to provide access to the dam and another pair was located over the machine shop bay to provide access to the powerplant. As the top of the dam narrowed and the downstream face moved further upstream, one of the monkey slides was removed and access to the dam was provided by catwalks at different levels from the downstream slide.

A stiff-legged derrick of 50-ton capacity was erected on the canyon rim above the upstream cofferdam. A 25-ton highline cableway was also erected just upstream of the upstream portal of the left diversion tunnel. The stiff-legged derrick was later moved to the rim above the intake structure of the left spillway tunnel and was also used on the lining of the left spillway tunnel. Still later, the stiff-legged derrick was moved back upstream to just above the upstream portal of the left diversion tunnel. A large revolving (whirley) crane was erected on tracks set just downstream of and parallel to the powerplant, primarily for yarding.

Six water wells were drilled 6 miles north of the site to provide a water supply for the aggregate screening and washing plant. These wells were drilled from 625 to 800 feet in depth and had a total rated capacity of 7,750 gallons per minute. In addition, four other wells were drilled to serve the contractor's needs on the west side of the construction site.

A compressor station, containing two large compressors each having a capacity of 3,350 cubic feet per minute, was constructed on the east side of the canyon in March 1959. Permanent-type air piping was erected from the compressor station to the bottom of the canyon to supply job air requirements. Four compressors, also for job air, were set in the refrigeration plant building, each with a capacity of 1,860 cubic feet per minute.

The contractor also provided housing facilities for employees which included trailer courts, concessions, barracks, and messhalls which were discussed previously in section 146.

153. CONTRACTOR'S CONSTRUCTION EQUIPMENT. The following is a list of the major construction equipment used by the prime contractor and the subcontractors under specifications No. DC-4825:

Quantity	Item	Size or capacity[1]
2	Truck cranes	20 ton
1	Crawler-mounted power shovel	2-1/2 cu. yd.
3	Truck cranes	30 ton
3	Crawler-mounted power shovels	4 cu. yd.
2	Clamshell buckets	3/4 cu. yd.
1	Clamshell bucket	3/8 cu. yd.
1	Crawler-mounted dragline	6-1/2 cu. yd.
2	Dragline buckets	2 cu. yd.
2	Dragline buckets	6-1/2 cu. yd.
1	Dragline bucket	3-1/2 cu. yd.
2	Dragline buckets	7-1/2 cu. yd.
1	Power scraper	60-inch
1	Power scraper	34-inch
1	Power crane	7-1/2 ton
1	Power crane—whirley	100 ton
2	Crawler-loaders	2 cu. yd.
4	Crawler-loaders	2-1/2 cu. yd.
1	Forklift truck	6,000 pound
1	Forklift truck	9,000 pound
1	Diesel locomotive	30 ton
2	Concrete trailer cars	12 cu. yd.
3	Concrete cars, selfpropelled	12 cu. yd.
4	Manlift elevators, monkey slide	30 man
1	Radial stacker	—
1	Truck-mounted jumbo	6 drill
2	Wheel loaders	—
7	Crawler tractors	175 drawbar hp.
3	Power graders	125 flywheel hp.
1	Crawler tractor	95 drawbar hp.
1	Crawler tractor	200 drawbar hp.
1	Crawler shovel loader	115 flywheel hp.
3	Semitrailers—low slung	2 cu. yd.
3	Truck tractors	—
3	Semitrailers	—
2	Buses	—
17	Rear-dump trucks	20 ton
2	Bottom-dump haulers	27 cu. yd.
14	Truck tractors	—
3	Water wagons	—
14	Aggregate trailers	50 ton
3	Special dump trucks	5 cu. yd.
2	Motorized scrapers	25 cu. yd.
2	Ambulances	300 hp.
69	Trucks, pickup and flatbed	Assorted sizes
10	Automobiles	—
4	Stationary air compressors	1,700 c.f.m.
2	Stationary air compressors	3,350 c.f.m.
3	Rear-dump trucks	—
4	Grease trucks	—
4	Mechanic's trucks	2-1/2 ton
8	Hoists, pneumatic	2,000 pound
2	Hoists, 2 drum	—
2	Hoists, 1 drum	12 ton
3	Hoists, 3 drum	40 ton
1	Hoist, 2 drum	—
1	Hoist	15 ton
1	Hoist, 1 drum	5 ton
2	Hoists, 2 drum	31 ton
2	Hoists	Assorted
1	Winch, air	1 ton
1	Hoist, mine, electric	—
4	Deepwell pumps	5,000 g.p.m.
1	Deepwell pump	1,500 g.p.m.
1	Deepwell pump	500 g.p.m.
1	Suction pump	8,700 g.p.m. 1,000 hp.
5	Centrifugal pumps	4,000 g.p.m., 4 inch
1	Pump, water	8,000 g.p.m., 8 inch
2	Pumps, submersible	43 g.p.m.
76	Pumps, water	Assorted

Quantity	Item	Size or capacity[1]
1	Pump, sand	20 hp.
3	Pumps, grout	—
3	Pile drivers	—
8	Air tractors, deep hole	—
3	Drills	—
1	Drifter	—
26	Jackhammers	—
18	Vibrators	—
5	Diggers, pneumatic	—
7	Pavement breakers	—
6	Chipping hammers	—
1	Drill, pneumatic	—
2	Sandblasters	—
2	Sandblast pots	—
8	Drills, air tractors	—
2	Pumpcrete machines	—
1	Grout mixer	3 cu. yd.
7	Mixers, concrete	4 cu. yd.
1	Mixer, plaster	—
1	Derrick, stiff-leg	50 ton
5	Concrete buckets	12 cu. yd.

Quantity	Item	Size or capacity[1]
4	Concrete buckets	12 cu. yd.
2	Concrete buckets	2 cu. yd.
5	Concrete buckets	4 cu. yd.
2	Concrete buckets	1 cu. yd.
6	Concrete buckets	8 cu. yd.
1	Laydown bucket	1 cu. yd.
1	Bucket, cement finish	—
8	Air compressors	600 c.f.m.
1	Air compressor	25 c.f.m.
2	Air compressors	80 c.f.m.
2	Air compressors	113 c.f.m.
1	Air compressor	125 c.f.m.
1	Air compressor	210 c.f.m.
1	Refrigerator plant	6,275 hp., 20 tons flake ice per hour
177	Transformers, electric	Ranging from 5 to 1,000 kv.-a.
3	Electric generating plants	—
8	Cages for cableways	30 man
2	Cableways	50 ton
1	Cableway	25 ton

[1]Where the type and size or capacity of a particular make and model of equipment were not stated in the original report, these have been obtained from current construction equipment catalogs. As specifications for a particular model may vary from year to year, some minor discrepancies may exist between the reported size or capacity and that of the equipment actually used.

F. CITY ADMINISTRATION FOR PAGE, ARIZ.

154. GENERAL DESCRIPTION AND LOCATION. The proposed site of Glen Canyon Dam was 75 miles from and without direct highway access to the nearest community of Kanab, Utah. The nearest city and railhead was Flagstaff, Ariz., which was about 135 miles south of the site of construction. In general, this arid, high plateau area is very rugged and prior access was available only over the most primitive trails.

To construct a project of this magnitude, a large number of construction employees would be required, as well as those in indirect support of the work. Some facilities for permanent operating and maintenance personnel would also be required, and designed facilities invariably prove the most economical when considering initial costs and operating and maintenance costs. Prior experience had shown that temporary construction support accommodations create severe problems during and after construction, both in respect to morale and economic considerations. In addition, large projects have historically generated small permanent communities with little regard to long-range planning. The demand for additional facilities to support public recreation usually becomes a major factor with the formation of a reservoir of this size.

The actual site of Glen Canyon Dam was established 4 miles south of the Arizona-Utah State line and 12 river miles downstream on the main stem of the Colorado River. Public lands lie on the west side of Glen Canyon and Navajo Tribal lands to the east. Small mesas, surrounded by blowsand areas, are near the construction site on both sides of the canyon. The mild, arid climate is typical to the high desert country.

After full consideration of the various possible locations, Manson Mesa on the east side of the river was chosen as the most advantageous for a potential townsite. Conveniently located about 2 miles from the site of the dam, this mesa is about 4,300 feet above mean sea level and offered optimum conditions for housing, businesses, and municipal facilities. Being on Navajo Tribal lands, an area of 24.3 square miles was exchanged for other lands adjacent to the Reservation in the Navajo Exchange Act of September 2, 1958, Public Law 85-868. A townsite plat was laid out for a portion of this area and the city was named after Mr. John C. Page, a former Commissioner of Reclamation (fig. 248.).

Basically, Page was designed to provide a permanent townsite to serve those residents engaged in the operation and maintenance of the dam and power facilities, recreation and tourist services, personal services and general business enterprises, and the Federal, State and municipal activities and services. Municipal facilities and services were designed primarily to serve the permanent segment of this population and, secondly, to support the peaking population during construction. That portion of the permanent system which was necessary to build for the temporary population was designed to provide a margin for reserve and for future expansion. Fully temporary facilities were located adjacent to the initial permanent development on Manson Mesa, and the contractor was required to remove them on completion of his construction work. A congressional appropriations limitation of $11,800,000 was placed upon the various items of municipal and construction support facilities at the city of Page, and this limitation was not exceeded.

The Government constructed 200 permanent houses for employees, which was the original estimate of employee family units required for operation and maintenance of the project. Additional temporary housing for construction employees was composed of 50 demountable "transa-house" housing units, and a small trailer court with about 40 Government-owned

Figure 248.—Aerial view of the town of Page, Ariz., looking north. Glen Canyon Dam is in background. P557-420-8316, May 15, 1963.

trailers and several spaces for employee-owned trailers. Five permanent houses were also constructed by the National Park Service for their personnel. There were also 225 lots left for private homesites in a developed area adjacent to the houses constructed by the Government.

The prime contractor constructed nine permanent-type houses in the residential area and 24 quadriplex and triplex apartment houses with 93 units for key employees. A large temporary trailer court was designed with 700 spaces for the construction employees. It was expanded more than once during construction, and there were eventually more than 1,000 house trailers in this one court managed by a concessioner. The prime contractor also allowed businessmen and other project-connected personnel to occupy trailer spaces when available. Upon completion of the work, the temporary court facilities were leveled and the site was restored to as near original condition as practical.

Temporary housing for teachers consisted of six apartment houses containing 40 units. The Department of Health, Education and Welfare provided funds to aid in their construction.

The contract for construction of the 200 Bureau houses also provided for optional construction of an additional 150 houses for sale to the general public. The contractor did not exercise this option. Subsequently, individual financing was obtained for construction of housing and, at the end of construction, 44 single-family dwelling units, three duplexes, and one two-story quadriplex had been completed. A permanent commercial trailer park, consisting of 174 rental spaces, was also constructed and mobile home subdivision was under development at the end of construction.

155. MUNICIPAL SERVICES AND UTILITIES. Streets constructed in Page, included 4-1/2 miles of 42-foot-wide streets and almost 4 miles of 70-foot-wide streets. Four inches of crushed rock base was surfaced with 1-1/2 inches of asphalt concrete sealed with a stone chip wearing coat, except for Seventh Avenue which had a 6-inch rock base. Concrete sidewalks with sloped curbs and gutters were also constructed on most of these streets. As a result of this high-quality, permanent-type construction, maintenance costs have been relatively small and no major repairs have been necessary. The only significant maintenance problem encountered involved cleaning blowsand from the streets, which was significantly reduced by installation of lath drift fences, covered with burlap, at strategic locations. Conversely, the north and south community access roads had minimal curb or shoulder treatment and have required constant routine maintenance to retain shoulder shaping in the blowsand areas.

By designing the sewage treatment system for the peak construction population, no unusual difficulties were experienced due to treatment capacity. Early difficulties with blowing sand were reduced as the general area became more stabilized, and the area immediately adjacent to the plant was protected with landscaping and tree windbreaks. Flooding from a thunderstorm on August 30, 1963, caused considerable work to repair damage to the outfall sewerline and access roads. As was the case with the city streets, quality construction of the sewage system has contributed significantly to the reduction of maintenance costs.

The water treatment plant, storage facilities, and distribution systems were also constructed to meet the peak population demands during the construction period. However, some expansion of the plant was necessary to meet a higher demand than originally anticipated. Early difficulties and high operational costs experienced with pretreatment of the muddy river water was eliminated when a permanent connection was made. The raw intake is now located on the upstream face of the dam in the outlet works trashracks and receives clear water from Lake Powell. A small, but separate, service charge is made to the consumers for operational costs of the waterplant.

A regular program of refuse collection and disposal is performed for the city under contract. This contract provides for the collection of garbage, both private and commercial, and the charge is billed directly to the individuals under a rate schedule established by the competitive bidding. This contract includes the collection of normal refuse and trash, grass, tree and shrub cuttings. Trimmings from the grass and tree cuttings are handled separately and the contractor is paid on the basis of an hourly rate under the supervision of city employees. Disposal of the garbage, refuse, trash, and other debris is made at a cut-and-fill type sanitary disposal pit at a site furnished by the Bureau well outside of the townsite limits.

Electrical service for Page is provided under contract by the Arizona Public Service Co. This service includes the operation and maintenance of the distribution system and the street lighting systems in the community. Billing is made directly to the consumers by the Arizona Public Service Co. on a rate schedule subject to the Arizona State Corporation Commission.

Gas for domestic use is furnished by the Petrolane Gas Service in the form of liquified petroleum gas. Large tanks are located strategically, each with a small distribution system which serves a group of houses or businesses. Initially, the rate schedule was established under contract with the Government. Subsequent to the expiration of the original contract, the rate schedule was no longer controlled for private consumers; however, the gas service company unilaterally continued to furnish gas service to private consumers at the same rate set in the contract for furnishing gas to the Government facilities.

Telephone service is provided by the Mountain States Telephone and Telegraph Co. A telephone exchange and office building, a maintenance yard, and a microwave terminal facility were established in the town of Page. Normal private and commercial service is available subject to the standard rate schedule. Installation of the service was started in October 1957 and was completed in early 1958.

A microwave television service with a cable distribution system is available. Four Phoenix network channels are available at a monthly service rate of $6.50 after an initial connection charge of $90.00. For short-time subscribers, an alternate rate of $10.00 per month is available without the payment of the $90.00 connection charge.

In addition to constructing the water treatment plant, the sewage disposal system, and the streets and surface drains, it was necessary to construct other municipal facilities to serve the city of Page; these included a municipal building, a police and fire station, an operation and maintenance shop, an airstrip, and a hospital.

The municipal building primarily serves to house offices of the city manager. During the peak of construction, this involved the services of the city manager, a city engineer and staff, and a city services section, supported by clerks and assistants. This building also provided office space for the services of an assistant County Assessor and an elected Justice of the Peace. Facilities are also provided for drivers license examinations conducted by the Arizona State Examiner on a monthly basis. Periodic requests for office spaces from other Governmental agencies are also honored.

The police and fire station facilities are actually adjunct to the Bureau garage, which serves the automotive repair needs of the project. The police portion contains office space, a dispatcher's room, and a jail section. The jail contains four detention rooms for men, two for women and juveniles, and a drunk tank. The dispatching facility includes radio equipment for communications with the local police cars, as well as interconnections with the Coconino County Sheriff's office and the State Highway Patrols of Arizona and Utah. Three radio-equipped vehicles are operated from this facility for police services. Personnel at the peak of construction consisted of a chief ranger, 10 full-time rangers, four dispatchers, and one dog catcher. The rangers were also deputized by Coconino County and the Navajo Tribe. The chief ranger is also deputized in adjoining San Juan County, Utah.

As would be expected, the management of city operations involved a complex of cooperative functions despite centralization of responsibility on the Bureau representative. For instance, round-the-clock police protection was furnished by the Bureau Rangers but enforcement of the laws of Arizona began with the Justice of the Peace, an elective office in Coconino County. The assessment of property taxes and licenses was established by the Deputy County Assessor, the evaluation also being the basis of the municipal service charge. Similar interrelation of functions were also present in regard to public health conditions, civil defense activities, elections, and similar community affairs. County and State representatives were cooperative, despite the many inconveniences of distance, and much of the successful operation of the city was due to their efforts.

The fire station is a partitioned-off section of the garage building and houses two 750-g.p.m. pumpers which were obtained through Government surplus. A 1,000-pound direct-chemical unit was mounted on a pickup and a 500-gallon water tank on a trailer is held in reserve at another location. Communication facilities are coordinated with the ranger communication system and alarm equipment is activated in the ranger office. The fire detachment consists of a full-time chief and 15 volunteers, all of whom are employed and trained by the Bureau. The majority of the volunteers are employed in buildings and shops immediately adjacent to the fire station.

Maintenance shops for city operations are located in a Butler-type metal building adjacent to the Bureau's warehousing facilities, primarily to utilize an adjacent, improved storage area. The shops also serve for maintenance of the other Bureau project supporting facilities in the city of Page, including offices and housing. These shops include areas for carpentry, plumbing, electrical work, painting, offices, and storage of spare parts. Major equipment repairs are performed in the garage facilities. Adequate staffing by members of the various crafts was maintained to support the municipal and construction facilities during the construction period. Operational equipment serviced included the street sweeper, grader, dozer, lawn maintenance equipment, and pest and weed control equipment.

A blanket service charge was made for municipal services furnished to private individuals and businesses. These services included police and fire protection and the operation and maintenance of streets, parks, street lighting, and water and sewer utilities. Charges for these services were based on a mill levy rate, established by the Bureau of Reclamation, applied to the assessed value of the land and building, plus assessable property, established by the Coconino County Assessor. Land use and zoning regulations provided that the levy rate could be adjusted by the Bureau, but that it would not exceed the tax rate in Flagstaff, Ariz. Except those exempt by State law, every property owner was required to pay for the municipal services furnished by the Bureau. The whole amount of the charge could be paid in November of each year or, at the option of the property owner, could be made in semiannual payments in November and May. A City Services Section in the City Management Division prepared bills and received payment for utility services and deposits, leases, land sale, building and peddler permits, and municipal service charges.

Under the provisions of the prime contract for construction of Glen Canyon Dam, contractor Merritt-Chapman and Scott Corp. constructed and operated a 25-bed hospital during the term of their contract. When the hospital was turned over to the U.S. Government on January 1, 1965, a contract was made with the Hospital Sisters of the Third Order of St. Francis to continue operation of the hospital facilities. The prime contractor was able to attract competent staffing for the hospital and these arrangements have proved highly satisfactory for a community hospital facility.

In addition to these municipal service facilities, some construction and operational support facilities were also constructed in the city of Page for convenience. These facilities included an administration building, a large warehouse with fenced storage yard, a concrete testing laboratory, and a garage with fenced parking area. A post office building was also constructed by the Postal Department in the shopping center.

Owing to the general remoteness of the damsite, it was necessary to construct an airstrip in order to provide air service for the community. The main runway is paved with asphaltic concrete and is 150 feet wide by 4,500 feet in length. A lesser, graveled runway is also available for light planes. The airport facilities furnished by the Government include navigational beacons, guidance and runway lights, and fencing around the airport. Regular passenger service was inaugurated on April 26, 1959, by Bonanza Airlines with flights to and from Phoenix, Ariz., and Salt Lake City, Utah. Airport management is provided under a contract between the Government and Page Aviation. Other services available are a private charter plane service, mechanical repair, flight instructions, and automobile rentals, all of which are furnished by the commercial flight service operation. All terminal and repair facilities have been constructed by the airport operator. Space has also been provided for future expansion on the main runway and additional parking areas. Another transportation means, bus service, was established by Continental Trailways on May 15, 1959, placing Page on a route between Kanab, Utah, and Flagstaff, Ariz.

CHAPTER XIII. *Construction*—DAM, POWERPLANT, AND APPURTENANT STRUCTURES

A. DIVERSION AND CARE OF RIVER

156. GENERAL. Diversion of the Colorado River during construction of Glen Canyon Dam and Powerplant was accomplished through two concrete-lined tunnels, one located in each abutment. The inside tunnel diameter of 41 feet was determined from routing studies made of the 25-year frequency flood. The length and location of the tunnels were laid out to allow ample space for disposal of all foundation excavation material between the upstream face of the dam and the upstream cofferdam. Space for the downstream cofferdam was also considered in the location of the tunnel outlets.

The diversion tunnel in the right abutment (fig. 28) was 2,749 feet long with an invert elevation of 3137.37 feet at its entrance, which was essentially river level. The left abutment tunnel was 3,011 feet long, but the entrance invert elevation was at 3170.67 feet, or 33.30 feet higher than the right tunnel. The difference in entrance elevations was made to allow for installation of outlet gates in the left tunnel during a low-flow season without benefit of an entrance closure structure. The upstream cofferdam, designed by the contractor, had a top elevation of 3300 and the downstream cofferdam had a top elevation of 3165.

The downstream portions of the diversion tunnels also served as the lower, horizontal section of the spillway tunnels. Final closure of the right diversion tunnel was made by installing a 150-foot-long plug in three sections. A closure structure was provided at the entrance of this tunnel to facilitate construction of the plug section. The closure structure consisted of three structural steel slide gates, designed for 90 feet of head, and a temporary concrete plug immediately downstream from the gates, designed to withstand 200 feet of head.

Features of the left abutment tunnel were similar to those of the right, except that high-pressure outlet gates were installed in the plug section and a concrete trashrack structure was installed at the tunnel entrance. Installed in the two upstream sections of the tunnel plug, the outlet works consisted of three steel-lined conduits through the plug section, two 7- by 10.5-foot high-pressure gates in tandem in each conduit, and an operating chamber (fig. 31). The outlet works was necessary to meet downstream water requirements during the period between closure of the right tunnel and availability of the permanent waterways in the dam and powerplant.

The right diversion tunnel was excavated by an early separate contract under specifications No. DC-4747 to provide lead time for establishment of the major construction facilities by the prime contractor for the dam and powerplant. Except for final closure on the left diversion tunnel, the remainder of work related to care and diversion of the river was performed under the prime contract, specifications No. DC-4825. Closure of the left tunnel and completion of the spillway elbow was made by separate contract after the downstream water requirements were being met through the power facilities.

157. EXCAVATION OF RIGHT DIVERSION TUNNEL. Mountain States Construction Co., of Denver, Colo., started construction operations on October 8, 1956, for excavation of the right diversion tunnel under specifications No. DC-4747, by moving equipment to the jobsite and setting up a temporary camp. On October 15, 1956, construction formally began by setting off the initial blast at the downstream portal of the tunnel at a signal transmitted by President Eisenhower in connection with the formal Glen Canyon ground breaking ceremonies. A large slab of rock directly over the lower portal was blasted from the canyon wall.

Initial work consisted of blasting and scaling overhanging rock from the canyon wall over the downstream portal. Construction of a warehouse, mess hall, bunkhouses, and other necessary camp buildings was completed in December.

A barge was constructed in November 1956, to ferry equipment from the mouth of Wahweap Creek to the upstream tunnel portal. A crawler tractor, hoist, and miscellaneous tools were lost in the river when the barge sank and this scheme was abandoned. All equipment was recovered except some miscellaneous tools. A highline cableway was under construction, but because of many delays in its completion, an access road was pioneered from the mouth of Wahweap Creek to the upstream tunnel portal.

A drill and associated equipment were lowered over the canyon wall with a temporary hoist early in December, and drilling in opencut was started upstream of the upstream portal. On December 21, 1956, the highline was completed and put in operation and a crawler tractor equipped with bulldozer was lowered down the canyon wall and set on the talus near the downstream portal. The bulldozer began construction of an access road along the talus between the two portals.

Excavation in opencut progressed at both portals, and on February 13, drilling of the tunnel began at the upstream portal at station 7+80. The drilling was subcontracted to Northwood, Inc., of Seattle, Wash. Moving the muck from the tunnel to the disposal area was subcontracted to Theo Wood Construction Co., of Salt Lake City, Utah.

Drilling was done from a jumbo designed and built by Mountain States Construction Co. The jumbo was not too efficient, as it had to be moved to and from the heading of the tunnel by a crawler tractor and numerous breakdowns occurred due to inadequate bracing. The jumbo was 31.5 feet high and 40 feet wide and had four platforms from which drilling was accomplished. The lower two of the four platforms were split in the middle and were raised by two air tuggers and held vertical by cables during mucking operations to permit equipment to pass through the 16-foot square opening. Mucking equipment consisted of end-dump trucks and a crawler-mounted tractor-shovel loader. Trucks were equipped with exhaust scrubbing equipment. The tunnel was lighted by 75-watt bulbs spaced 35 feet apart and also with floodlights on the jumbo. Ventilating air was provided at 70,000 cubic feet per minute from a 48-inch fan with 42-inch vent lines into the tunnel.

An average of 14 feet of drilling per day was accomplished and approximately 750 cubic yards of rock was removed per round. About 1,140 pounds of 35 percent No. 3 powder with 9 delays was used for each round. An average of 144 holes, 1-1/2-inch-diameter starters, were drilled on approximately 3-foot 4-inch centers. Twelve main V-cuts and 32 lifters were drilled 18 feet in depth. The remainder were 16 feet deep except for the set of 8-foot center pilot, or baby cut, holes.

Some difficulties were experienced because of the roof of the tunnel slabbing off along the left side of the drift. This was probably caused by the fact that the tunnel was apparently being driven along, or close to, a strike of extremely steep dipping crossbeds, complicated by a localized tendency towards exfoliation. The dip of the crossbedding along the left side of the tunnel roughly parallels the tunnel wall, whereas the dip intersects the right wall of the tunnel at a steep angle. The slabbing off, or caving, along the left side of the tunnel roof required the use of many additional roof support bolts. Mine roof ties of steel-ribbed 2- by 6-inch by 13-foot channels, fastened by 6- to 8-foot bolts, were used in lieu of chain link fabric for tunnel roof support.

During April, May, and June 1957, the water rose in the river and considerable trouble was caused by water seeping through the canyon wall into the tunnel. This problem was resolved by trenching to sumps and dewatering with 4-inch pumps. The peak flow of the Colorado River during the drilling period was 124,000 cubic feet per second on June 12, 1957.

Opencut excavation in the downstream outlet channel area continued until September 1957 when this work was discontinued. The remaining opencut excavation was deleted from the contract and was later completed by the prime contractor on the dam. The common excavation deleted from the contract amounted to 47,431 cubic yards and the rock excavation amounted to 46,846 cubic yards.

Drilling of the tunnel continued at the rate of about 340 feet per month until October 30, 1957, when the tunnel was holed out at station 35+20. A portable jumbo was constructed by Northwood and used behind the drilling jumbo for barring loose rock and for placing roof bolts. The invert of the tunnel was left about 6 feet above grade and the removal of this invert section, as well as removal of radial tights and installation of roof bolts, was completed on March 13, 1958, which was also the date all work under the contract was completed and accepted.

The excavated diameter of the tunnel to the A-line was 43.5 feet from the upstream portal to station 25+70. From station 26+11.72 to the downstream portal, the diameter was 46.5 feet with a tapered transition section between these stations. The difference in diameter was due to the difference in thickness of concrete lining of the two sections. The downstream, or spillway section of the tunnel, was lined with 2 feet 9 inches of concrete and the upstream or diversion tunnel section was lined with 1 foot 3 inches.

Overbreak amounted to 3,096.58 cubic yards for which payment to the Government was made at the rate of $15.00 per cubic yard for a total of $46,448.70. The total contract less liquidated damages and payment for overbreak amounted to about $2,280,000.00.

158. COMPLETION OF RIGHT DIVERSION TUNNEL AND CONSTRUCTION OF LEFT DIVERSION TUNNEL. The right diversion tunnel was being excavated under specifications No. DC-4747 by Mountain States Construction Co. at the time the prime contract was awarded. The tunnel under this previous contract was completed March 13, 1958, and

was not lined. Excavation of the keyways in the plug section and lining of the tunnel were parts of the prime contract under specifications No. DC-4825. Excavation for the keys in the plug section was started in May and completed in August of 1958.

Under order for changes No. 1, the contractor was directed to perform all opencut excavation for the outlet channel of the right diversion tunnel above the grade line. This work was deleted from the contract of Mountain States Construction Co. for the driving of the tunnel. Excavation at the downstream portal of the tunnel started in April 1958. Numerous rock bolts were placed on the canyon wall. Excavation at the downstream portal was completed early in February 1959, just before the diversion of the river and after the lining of the tunnel had been completed.

Opencut excavation at the upstream portal of the left diversion tunnel was completed in November 1957, and driving of the tunnel started. All tunnel excavation work was subcontracted to Frazier-Davis Construction Co. of St. Louis, Mo. Drilling of the tunnel was carried out on a three-shift, 6-day-week basis and progressed at a rate of from 22 to 30 feet per day. A round was shot and mucked on each shift. The drilling scheme and blasting procedures were essentially the same as on the right tunnel. The invert of the tunnel was also left about 6 feet high to provide a roadway for trucks and the truck-mounted jumbo. Drilling of the tunnel plug section was completed in April 1958. On June 10, 1958, a 12- by 15-foot pilot hole was completed a distance of 100 feet to daylight the tunnel in the opencut section. The invert of the tunnel was drilled, shot and mucked out, tights were removed, and additional roof bolts placed where needed.

Because of rock conditions at and downstream from theoretical portal station 36+52, it was decided to extend the left tunnel. The tunnel was extended 42 feet, and the downstream portal was located at station 36+94 with the concrete face later extending to station 36+96. A slide occurred above the downstream portal of the left tunnel in the opencut section between tunnel stations 36+60 and 39+20 on August 5, 1958. The fallen material was removed and the wall scaled.

Excavation of the invert was completed the week of October 19, 1958. Excavation of the gate chamber was started in June 1958 and completed, except for removal of tights, in the latter part of July. An 8- by 10-foot pilot hole was driven from the tunnel for the left spillway raise for a distance of 550 feet and was then completed from the top.

Lining of the right tunnel (fig. 249) was started in May 1958 with the placing of concrete curbs, and lining of a 70° invert arc began in June. The remaining 290° of arch concrete was made in a second placement series. Concrete was furnished from the temporary mixing plant on the right abutment, located about one-fourth of a mile from the highline cableway. A water-reducing, set-retarding agent, with a calcium salt of lignosulfonic acid as the active ingredient, was used in the concrete at the rate of 0.25 pound per sack of cement. Concrete was trucked from the batching plant to the highline in 4-cubic-yard buckets and lowered to the canyon bottom. It was then discharged into a specially constructed concrete hopper mounted on a truck chassis and delivered to the tunnel. After loading into a 1/2-cubic-yard bottom-dump concrete bucket, which was lifted by crane, the concrete was discharged into the forms and was consolidated with electric, immersion-type vibrators. After the initial placement in the invert, a pumpcrete machine was used and concrete was delivered to it in the same manner as noted above. Concrete was pumped a distance as great as 900 feet. The placing schedule called for three 60-foot-arch placements per week and the schedule was generally met.

Figure 249.—View of north portal of the right diversion tunnel showing concrete lining in place. P557-420-2818, September 17, 1958.

The lining of the plug section of the right tunnel was placed in a different sequence. The invert, an arc slightly over 11 feet in length, was placed last to allow for additional clearance. The arch was lined in six placement of 6 feet of arc each to a point just over 3 feet above the springline. These placements were formed with wooden-truss, rib-type forms. The remaining arch section was lined in one placement. On this placement above the springline, Blaw-Knox steel forms were used, upon which a wooden form was superimposed to develop the wedge shape of the keyway. Lining of the right tunnel was completed early in January 1959, as was the concrete for the intake closure structure at the upstream portal.

Lining of the left diversion tunnel was started the last week of August 1958 with the placing of rail concrete from station 17+00 to station 20+82. Lining of the invert and arch sections followed in the same manner as in the right tunnel, except for the plug section where the invert was placed first. Two more lifts brought the lining to a level just above the springline. The remaining arch was placed in one single placement as in the right diversion tunnel. Lining of the left diversion tunnel was completed in March 1959.

Rail concrete consisted of a curb or rail on each side of the tunnel to provide a base for carrying the lining forms. The curbs were formed with wood forms and the concrete was placed by a conveyor or bucket from a specially equipped dump truck. The usual dump body was replaced by a 4-cubic-yard hopper fitted with a bottom-discharge chute and clam gates. The concrete was discharged into a short inclined conveyor and was carried up to the curb forms. In some cases, the concrete was discharged from the hopper into a laydown bucket hoisted to the forms by a tractor with a swing boom.

The invert placement was made by use of an invert screed car riding on rails bolted to the top of the curbs. The car was propelled by a cable anchored into the curb ahead and attached to a 15-ton winch. An air wrench actuated the winch. As the screed car moved ahead, it shaped the invert and struck off the concrete to a smooth surface.

Closure of the right diversion tunnel was started January 21, 1963, and completed January 23, and the river was diverted through the left diversion tunnel with the outlet gates fully open. The lower sections of the right diversion tunnel plug were placed early in February 1963, to elevation 3142.25. The contractor's 21-foot plug, adjacent to the intake closure gates was placed February 11. This 21-foot plug was part of the design of the contractor-furnished intake closure gates. The plug sections were completed about the middle of March, and the contraction joint, periphery, and cooling pipe grouting were completed after the concrete had cooled sufficiently. The 18-inch bypass pipe in the plug section was plugged in June. The backfill section of the right diversion tunnel was placed during June and July 1963, and the contraction joints and periphery grouted.

Placements 1 through 6 of the left diversion tunnel were made in September 1962 to elevation 3147.75 and the upstream sections of conduits and outlet gates were set in place in October. Plug section sequence placements were made to embed these sections. Installation of the outlet gates was completed and the six gates were operative early in January 1963. Placement of the two upstream sections of the left diversion tunnel plug was completed in December 1962.

The six outlet gates were supplied by Yuba Consolidated Industries, Inc., of Benicia, Calif., under invitation No. DS-5216. The 7- by 10.5-foot bulkhead gate is of welded and cast steel construction, and consists of a body, a leaf, and a hydraulic hoist which is an integral part of the gate. The main portions of the gates were received in Flagstaff, Ariz., starting on October 5, 1960, and deliveries were substantially complete on March 1, 1961. It was noted that the supplier had experienced difficulties in application of the CA-50 paint and field repair was required after materials were at the jobsite. The control cabinet was supplied by Kendo Equipment Co., of Denver, Colo., under invitation No. DS-5364.

The 7- by 10.5-foot bulkhead gates were installed by the prime contractor under specifications No. DC-4825. Owing to the location of the bulkhead gates, the contractor's concrete placement schedule in the left diversion tunnel, the embedded structural steel supports, and leveling jacks provided by the Bureau installation plan were not used. The contractor placed the concrete in the tunnel plug invert to about elevation 3147. Rows of 150-pound railroad rails were embedded in a vertical position with the top of the exposed rails higher than the elevation required to support the gates. When the gates were to be placed in position, the rails were cut to exact elevations and capped with a steel plate. The gate bodies were moved into position on cribbing, and final alinement and level were obtained with embedded turnbuckles and metal wedges on the rail supports. The procedure produced good results, and the allowable tolerance between the gate leaves and their body seats was obtained without reworking the seats. The remaining parts of the gates were assembled, the controls installed, and the system was filled with oil without unusual difficulties. Servicing and testing were completed and closure was made in March of 1963.

Installation of the left plug and completion of the spillway elbow were originally required under the prime contract. Because of downstream water requirements, however, this work was deleted from the contract and temporarily deferred. Following the desired service during reservoir filling, the high-pressure gate conduits and gate chamber were later plugged with concrete during completion of the left plug and spillway elbow (figs. 250 and 251) under specifications No. DC-6317.

159. COFFERDAMS AND CARE OF RIVER. Construction of the upstream and downstream cofferdams under the prime contract, specifications No. DC-4825, began in October 1958, with the dumping of materials excavated from the right keyway. Both cofferdams were extended from the west side. The material was wetted and compacted with tamping rollers as it was placed. Placing of earth and rockfill continued until February 8, 1959, when closure of the upstream cofferdam was started. The dike in front of the right diversion tunnel was removed. Steel tunnel liner forms, structural steel ramps, and boulders were used to complete the closure, and the river was first diverted through the right diversion tunnel, at 7:30 a.m., February 11, 1959.

Fine material was placed at the upstream face of the cofferdam and the dam was widened. The dam was then at approximate elevation 3183 and was widened to 50 feet. A cutoff trench was constructed just

Figure 250.—View looking upstream to plug section in left diversion tunnel. Sand and rock are falling through holes from elbow excavation above. P-557-420-11896, Feburary 9, 1966.

Figure 251.—Looking up in elbow section of left spillway. Drillers at work in foreground. P557-420-11982, Feburary 23, 1966.

downstream of the 50-foot section, and sheet steel piling 60 feet in length was driven into the trench. Sand and bentonite were mixed and placed around the piling. The downstream cofferdam was closed during the week of February 22, 1959, but was reopened to permit seepage water to flow through. The average elevation of the top of the sheet piling at the upstream cofferdam was 3158. The mixture of the sand and bentonite was placed throughout the cutoff trench as the dam rose. Three electric-driven pumping units with a total capacity of 8,000 gallons per minute were installed on rafts and placed on the pond between the two cofferdams.

River diversion was through the river level reached about elevation 3171, at which time the left diversion tunnel began to accept a portion of the 20,000 c.f.s. river discharge.

Free-draining material from the river bottom was stockpiled on the downstream cofferdam for later use between the dam and powerplant and in the tailrace. In February of 1960, a buttress-type concrete cutoff or retaining wall was erected at the upstream toe of the downstream cofferdam to prevent the free-draining material from sliding into the downstream sump. A concrete sump was constructed in the river bottom on the upstream side of the retaining wall for controlling the seepage water by pumping. During the first 2 weeks of June, some undercutting occurred at the downstream edge of the cofferdam at a riverflow of about 40,000 cubic feet per second. When the haul road through the dam and powerplant areas was removed, part of the excavated material was hauled to the downstream cofferdam and dumped on the right side.

During 1960, a concrete cutoff wall was placed at the downstream edge of the upstream cofferdam and just upstream of the upstream sump. Two 10-foot lifts were placed to complete this wall. Water which formed behind the wall from seepage through the cofferdam was pumped from a sump behind the wall. Maximum flow of the Colorado River recorded at Lee Ferry during 1960, was 46,300 cubic feet per second on June 9.

During 1961, a considerable amount of free-draining material was removed from the upstream side of the downstream cofferdam for use as backfill between the dam and powerplant. Other material from the cofferdam was used as backfill over the outlet works mass concrete and over the machine shop and service bays. The upstream side of the downstream cofferdam was changed considerably while excavating and grading for the tailrace slab. Later some of this material was used as backfill on the tailrace slab from the crest of the slab to its downstream edge. Maximum flow of the Colorado River in 196l was 39,200 cubic feet per second and occurred on June 4 and 5; maximum flow in 1962 occurred on May 16 with a flow of 84,450 cubic feet per second.

The high flow of the river during the runoff season deposited a large amount of driftwood and trash around the upstream portals of both diversion tunnels. A bulldozer was transported to the upstream cofferdam by an LCM (landing craft, medium) boat. Both were lowered to the pool just above the dam. Driftwood was removed from the left diversion tunnel trashrack structure by the bulldozer. The driftwood around the upstream portal of the right diversion tunnel was burned.

The lower center beam of the left diversion tunnel trashrack structure was installed in August. This beam had been left out to permit the passage of trucks into the tunnel during installation of the outlet gates, but it was decided to haul in the gates and concrete from the outlet end of the tunnel. The metal trashrack, except for four sections, was installed on the concrete trashrack structure and gravel placed on top of the structure.

A wood and canvas bulkhead was placed around the trashrack structure to about elevation 3210, or 40 feet above the base of the trashrack, to protect the tunnel plug concrete and conduit in case of a sudden rise in the river. During the rise of the river on October 21 and 22, the pool upstream of the upstream cofferdam rose to approximate elevation 3175, which is slightly higher than the base of the structure. The bulkhead leaked, but by using additional pumps, the water was prevented from rising to the level of the river outlet conduits.

Closure of the intake gates in the right diversion tunnel began January 21, 1963, and was completed on January 23. The river was diverted through the left diversion tunnel, which had an entrance invert elevation 33.30 feet higher than the right tunnel. The outlet gates in the left diversion tunnel were kept fully open until March 13, 1963, when the initial storage began in Lake Powell. The reservoir was at elevation 3203.3 at this time.

At 2:00 p.m., March 13, outlet gates No. 1 and 3 in the left diversion tunnel were fully closed and gate No. 2 was lowered to within 4 feet, 2-5/8 inches of closing. The discharge was kept at 1,000 cubic feet per second or slightly above until June 3. Numerous adjustments to outlet gate No. 2 were necessary to maintain this discharge as the lake rose; and at 8 a.m., on June 3, 1963, the gate was set at a 1-foot 10-inch opening with 1,000 cubic feet per second being discharged. At 9:13 a.m., on June 3, outlet gate No. 2 was set at 4 feet 4-1/2 inches to discharge 2,500 cubic feet per second. With 1,000 cubic feet per second discharging for 9 months of the year and 2,500 cubic feet per second discharging the remaining 3 months, 1,000,000 acre-feet would be passed downstream as required by the filling criteria. At 8:21 a.m., on July 10, 1963, outlet gate No. 2 was set at 1 foot 8 inches to discharge 1,000 cubic feet per second.

The spring runoff in 1963 was the lowest recorded for a number of years and, therefore, the lake did not rise as rapidly as expected. On June 15, 1963, the reservoir was at elevation 3360.9 and was rising at the rate of about 0.5 foot a day. During the week of July 6-12, 1963, the lake elevation remained at 3371.0, and after the outlet gate was closed on July 10 to discharge only 1,000 cubic feet per second the rise amounted to only about 0.1 foot a day.

It was thought that Lake Powell would reach elevation 3390, on or before November 1, 1963, to permit discharge of 1,000 cubic feet per second through the four 96-inch-diameter outlet pipes and permit closing of the left diversion tunnel. The reservoir did reach elevation 3390 on September 16,

1963. However, the filling criteria stated that "until elevation 3490 is first reached, any water stored in Lake Powell shall be available to maintain rated head on Hoover Powerplant." The risk of failing to maintain an elevation of 1123 in Lake Mead appeared to be too great to justify plugging the left diversion tunnel until turbine capacity was available and with discharge available through Glen Canyon Dam only by the four river outlet pipes. Therefore, the prime contractor was relieved of the work of plugging the left diversion tunnel and completing the excavation and lining of the lower elbow section of the left spillway tunnel.

Discharge from Lake Powell was kept at or near 1,000 cubic feet per second until January 29, 1964, when outlet gate No. 2 was set to discharge 2,500 cubic feet per second. On January 30, 1964, all three gates in the left diversion tunnel were opened to 2 feet to discharge 4,000 cubic feet per second. A release of near that amount was held until March 26, when instructions were received from the office of the Secretary of the Interior to increase the release as it was necessary to maintain generator operating head at Lake Mead of elevation 1123. Releases from March 26 through March 30 increased from 10,000 to 18,000 cubic feet per second and on March 31, the release was cut to 16,000 cubic feet per second. On April 3, the release was lowered to 12,500 cubic feet per second and on April 17 it was cut to 11,000. From April 27 through May 10, releases were held to near 12,500 cubic feet per second. On May 11, because of a more favorable estimate of the April–July runoff, the gates were again set to release 1,000 cubic feet per second with the object of filling Lake Powell to elevation 3490 in order to start generating in September 1964.

From the period March 26 through May 11, the level of Lake Powell dropped from elevation 3414.9 to elevation 3394.5, or a distance of 20.4 feet. The downstream cofferdam was removed in May 1964. The reservoir at elevation 3394.5 on May 11, 1964, contained 2,560,000 acre-feet. Elevation 3490 was attained in the reservoir on August 17, 1964, and it contained 6,124,000 acre-feet at that height. On this date, the release was increased to 6,000 cubic feet per second as Lake Mead had dropped to elevation 1100.96.

Scheduled generation of power from unit 1 started at 11:30 p.m. on September 4, 1964, and at 6:55 p.m. on that date the outlet gates were fully closed and discharge was through the generating units until December 28. Discharge varied between 2,700 and 4,000 cubic feet per second from September 10 through November 5, 1964, was at about 6,000 cubic feet per second until December 28, 1964, and was 13,150, 12,410, and 8,890 cubic feet per second on December 29, 30, and 31, respectively.

160. LEFT DIVERSION TUNNEL PLUG AND SPILLWAY ELBOW LINING SECTION. (a) *Preparatory Work.*—As indicated previously, construction of the left plug and spillway elbow, originally provided under the prime contract, was deferred. Following the desired service of the high-pressure gates during reservoir filling, specifications were issued for completion of this work.

Preparation work to provide access to the left diversion tunnel for prospective bidders under specifications No. DC-6317 consisted of installation of a hoist and a mancar at the upper end of the spillway and a pump to dewater the downstream part of the diversion tunnel.

A single-drum 440-volt electric hoist was leased from the Sunfeldt Equipment Co., of Long Beach, Calif. The hoist had a capacity of 17,000 pounds direct line pull at 40 feet per minute with a reversible worm gear drive. Three-quarter-inch wire rope was installed on the drum and connected to the mancar previously used in the spillway by Merritt-Chapman and Scott Corp.

Access to the waterway from the spillway bridge was by means of a mancage on a truck crane situated on the bridge. This truck crane was also used to lower the hoist down to the waterway ground level. This truck crane was the property of the Bureau of Reclamation and was operated by a Government employee. The electric power source was the spillway gate panelboard.

Access to the downstream end of the diversion tunnel was by boat until such time as a trail and walkway from the powerplant parking lot was constructed by operation and maintenance laborers and carpenters. The pump used to unwater the diversion tunnel was a 12-inch vertical propeller type, 2,400-g.p.m. pump, with a 25-horsepower, 3-phase, 220/440 volt, electric motor. The power source for this pump was the power panel at the hollow-jet valves.

The installation of this pump was accomplished by lowering it and the required discharge pipe down the wall of the canyon from the east rim, a vertical distance of approximately 700 feet. This was done by use of an air-operated hoist mounted on the back of a hydraulic truck crane. A two-part 3/8-inch wire rope line was used which fed through a pulley on the boom hook and dead ended at the end of the boom. A second air hoist was set up beside the truck crane with an A-frame on the canyon rim to lower and raise the high scalers. Air to operate these hoists was furnished by a 600-c.f.m. diesel-powered air compressor.

During the lowering of the pump, a highscaler was needed to keep the pump from being snagged by rock bolts or projecting rock. This same procedure also had to be followed for each section of the discharge pipe and hose. The pump was suspended vertically on chain hoists which were hung on anchors above the tunnel portal headwall. A flexible rubber hose was connected between the discharge pipe and the pump so the pump could be lowered with the water level. Continuous pumping for approximately 57 hours was required to pump out the diversion tunnel. To keep the water level down required pumping approximately 50 percent of the time.

Prospective bidders were taken into the tunnel from the powerplant parking area by a walkway along the left canyon wall to the flip bucket, down a ladder into the diversion tunnel, and along the invert of the tunnel on foot to the plug section. A small boat was used to cross the plug section to a ladder up to the 7- by 10.5-foot gates. Access to the gate chamber was through the gate chamber adit in the dam at elevation 3157.5. Access to the spillway was by cage and crane from the spillway bridge down to the waterway level, then by ladder and walkway, and by hoist and mancar down to the elbow form in the spillway. From where the jumbo form was suspended, access to the bottom of the spillway excavation was by way of ladders. Prospective bidders were also shown available work areas and were taken to the aggregate storage area.

(b) *Award of Contract.*—The contract was awarded to the low bidder, S. S. Mullen, Inc., of Seattle, Wash., and the notice to proceed was issued on August 23, 1965. The contractor rented a vacant store building in Page and on September 13, 1965, started building partitions for offices in the building. On the same date a truck arrived with a 600-c.f.m. deisel-powered portable air compressor, an air receiver tank, an air-track drill, and an arc welder.

(c) *Diversion Tunnel Plug.*—Preparatory work started on the following day consisting of pumping the water out of the tunnel, building an approach road to the access adit along the left wall of the canyon downstream from the powerhouse parking area, building a stairway with metal commercial scaffolding from the spillway bridge to the ogee section, and drilling and blasting out the concrete plug in the adit tunnel for access to the diversion tunnel at the lower end. The contractor assumed the lease on the hoist at the upstream end of the spillway and rented the pump in the deflection bucket from the Bureau.

From September 13 to October 13, the work performed by the contractor consisted of blasting the plug out of the access adit, installing a 12-inch drainline in the diversion tunnel, running air and powerlines into the work area from both the top and bottom of the tunnel, cleaning out the debris and waste concrete in the bottom of the spillway shaft, providing man access to the spillway and through the form jumbo to the bottom of the shaft, and moving in construction equipment. A two-shift work schedule was initiated on October 4, and the first pay item work started with the installation of rock bolts and wire mesh in the raise and mucking out loose material that had been left in the spillway. Mucking was done with a crawler-type tractor equipped with a front-end loader. This tractor was lowered down the spillway from the top and through the form jumbo on a timber ramp. It was diesel powered and equipped with an exhaust scrubber.

All material excavated in the spillway shaftway was dumped through the pilot shaft into the diversion tunnel below, loaded in dump trucks, and hauled outside and dumped downstream of the access adit along the left wall of the canyon.

Three 3-inch holes were drilled with an air track drill on the right side of the invert at the downstream end of the existing spillway lining. These holes were drilled vertically down through the lining of the diversion tunnel. Pipes were calked into the lower end of these holes and connected to the 12-inch drainpipe leading out of the tunnel. The water from the drain holes in the existing lining was then diverted into these holes. Later three more holes were drilled approximately 30 feet downstream and used to collect water seeping out of the rock. Some 3-inch drain holes were drilled 25 feet into the rock to relieve water pressure, but this was discontinued as it was observed that the rock bolt holes were acting as drain holes.

The excavation was shot out in 5-foot lifts with the contractor's engineers and the Bureau survey crew setting control for alternate lifts.

During the time excavation was underway, preparations were being made to place concrete in the access adit from the dam to the spillway. This work consisted of placing metal panning in the adit for water control, with grout systems installed on each side behind the panning and a vent system in the arch to facilitate void grouting after concrete placement. Also during this time the batching plant was being erected near the upper portal of the powerplant access tunnel. This was a mobile plant with a 3-1/2-cubic-yard turbine-type mixer. On November 19, 1965, the first concrete was placed in the diversion tunnel plug section to elevation 3140.25. This was followed by a 5-foot lift on November 24, 1965.

Prior to placing concrete in the plug section of the diversion tunnel, it was necessary to control the water leaking around the upstream 7- by 10.5-foot high-pressure gates. This was done by cutting 8- by 8-inch timbers to fit the bottom of the downstream gate seats. These were wedged in watertight, with 1-1/2-inch pipes through holes in the timbers. The water was carried through these pipes to a point downstream of the backfill concrete. The downstream gates were then lowered and closed tight on the timbers. After the second lift of concrete in the plug section was placed, a 7-foot lift was placed in each of the diversion conduits downstream of the high-pressure gates.

During the time the above concrete work was being done, the contractor removed the gate control equipment and electrical equipment from the gate chamber and gate chamber adit with the exception of the hoist cylinders. It was decided to leave the hoist cylinders in place with a reduction in contract price of $5,000.00 or one-half the bid item price.

Concrete was pumped from the pumpcrete machine, located at approximately station 26+20 in the diversion tunnel, up to and through the 7- by 5-foot adit, to the gate chamber and then through the dam gate chamber adit to the downstream end of the 90-foot plug section. Placing of concrete was started on December 3, 1965. With approximately 50 percent of the concrete in place, pumping was delayed while the contractor pulled the slickline back; and the pipe was plugged when this operation was completed. As it was impossible to start the flow of concrete again, the pumpcrete pipe was broken down and cleaned out with the exception of a 240-foot section that had set up too hard to clean out. This section was discarded. New pipe was installed and the placement of concrete in the adit plug was resumed and completed on December 8, 1965, with no further trouble.

On succeeding days the placement of concrete was completed in the three diversion conduits. The metal seals in the top downstream end of the conduits was in good condition and required only bending out and welding. The grout supply and vent systems were also found to be clear and required only the installation of grout box covers.

Concrete placing in the diversion tunnel plug section continued while waiting for time lapse before grouting the plug in the dam-to-gate-chamber adit. Also, grout holes were drilled out of the gate chamber upstream of the dam-to-gate-chamber adit and out of the 7- by 5-foot adit downstream of the gate chamber to the joint between the plug and the tunnel lining.

On December 27, 1965, the holes out of the 7- by 5-foot adit were pressured with thin grout, but there was no take on any of the eight holes. On December 29, 1965, the void grouting in the plug section in the dam to gate chamber adit was completed. Because of the seepage of water out of the rock, it was not possible to get a return of thick grout from the arch vent system. Final filling was accomplished by hooking the vent system and forcing the water back into the rock until there was no further take of grout. This required 440 sacks of cement. Grout was pumped with a grout pump rented from the Bureau.

On December 14, 1965, the contractor suspended excavation in the spillway raise due to conflict with concrete operations in the diversion tunnel below the pilot shaft which was being used as a muck chute. The invert of the tunnel was mucked out and backfill concrete was placed up to elevation 3145 to eliminate water problems in the working area. This concrete was placed using a 16-inch concrete conveyor suspended from a hydraulic crane.

The pumpcrete machine, an 8-inch duplex pump, was moved back into the diversion tunnel and set up on top of the backfill concrete at the downstream end of the plug section. Pipe was run into the dam to the gate chamber adit and on December 30, 1965, the balance of the adit was backfilled. The contractor elected to do this with regular concrete rather than sand and gravel. This was at no cost to the Government for cement, and made it possible to place the material by pumping.

Concrete was transported from the batching plant to the tunnel in five 7-1/2-cubic-yard transit mixers mounted on diesel trucks equipped with exhaust scrubbers. The trucks dumped onto a conveyor belt which in turn dumped into the pumpcrete hopper.

The existing grout system in the gate chamber was found to be usable except for the vents to the top. Grout covers were installed and new vent lines were run into the dome. The access doors to both adits were bolted in place with a manhole in the downstream door. A length of 36-inch corrugated split pipe was installed on the end against the downstream wall of the gate chamber for access during placing of concrete. The plastic drainpiping, a 6-inch drain from the dam-to-gate-chamber adit doorway and five 1-1/2-inch pipes from the grout holes drilled out of the gate chamber were extended through the downstream door. The pumpcrete pipe was installed through a hole in the door and extended to the dome of the chamber. A swiveled deflector was installed on top of the pipe which dumped into chutes that distributed the concrete fairly uniformly over the area of the chamber. Steel scaffolds were built on the gate hoist cylinders for vibrator crews to work on. A thermocouple was installed near the center of the gate chamber and 12 feet above the bottom, and the wires were brought out through the downstream bulkhead.

Pumping of concrete in the gate chamber was started on January 4, 1966, and was completed on the following day. Vibrator crews worked in the chamber until the concrete reached the top of the corrugated pipe manway. Men and tools were brought out and pumping continued to refusal. Electric switches installed in each end of the chamber dome closed and turned on indicator lights outside just before the pump stalled.

All grout, vent, and drainpiping, and thermocouple wires were extended to the downstream end of the 7- by 5-foot adit. Grout covers were installed on the existing grout and vent systems and the adit was pumped full of concrete. Steel baffles were installed across the adit to help hold the concrete in place and prevent excessive voids in the top due to sliding.

Placing of concrete was resumed in the plug and backfill sections of the diversion tunnel. This was done in 5-foot lifts with cooling pipe installed between lifts. Seep water from the rock was used for cooling until such time as the water in the tailrace reached a temperature below 52° F. which was the temperature of the seep water. This occurred on January 5, 1966.

The concrete was placed with a conveyor system consisting of an inclined conveyor discharging onto a horizontal conveyor. The horizontal conveyor was of the side-delivery type and was mounted on rails. The upstream rail was supported on brackets bolted to the existing concrete, and the downstream rail was bolted on top of the form the top of which was 2-1/2 feet above concrete grade. The horizontal conveyor was hydraulically self-propelled laterally and the side-delivery chute moved longitudinally on a hydraulically operated chain. This system gave good coverage over the placement area.

On January 12, 1966, the 6 inch drain from the dam-to-gate-chamber adit and the five grout holes drilled out of the gate chamber were grouted.

The grout systems for the diversion conduits and the drains for the gate chamber were piped above elevation 3175 and backfill concrete was placed as shown in the following tabulation:

Station		To
From	To	elevation
23+28.2	23+78.2	3175.00
23+78.2	24+35.2	3170.50
24+35.2	24+92.2	3160.50
24+92.2	25+46.8	3155.00

Several places in the existing metal seal at station 23+77 were badly damaged and broken by the flow of water during diversion and required extensive repairs.

On February 2, 1966, concrete work was suspended and excavation was resumed in the spillway shaft and existing diversion tunnel lining.

On February 7, 1966, the arch voids in the three diversion conduits were grouted using a total of 101 sacks of cement. The following day the voids in the gate chamber and the 7- by 5-foot adit and the plastic drainage system in the gate chamber were grouted using a total of 255 sacks of cement. The concrete temperatures at the time of grouting were as follows:

Conduit No. 1—58° F.
Conduit No. 2—60° F.
Conduit No. 3—59° F.
Gate chamber—80° F.

By March 10, 1966, excavation of rock and concrete lining was substantially completed and the placing of backfill concrete was resumed. Pumpcrete was used to fill under the arch of the existing lining and conveyors on the remainder. The concrete was placed on the downstream slopes without the use of forms. Placing of backfill concrete was completed March 26, 1966. While the backfill concrete was being cooled the contractor worked at removing tight rock from the ribs and arch of the tunnel. During the cooling process the water in the tailrace ranged in temperature from 44° to 48° F. depending on the amount of discharge through the powerplant.

All backfill concrete was down to the required maximum temperature of 60° F. and joint and void grouting was started on April 20, 1966. On this date the joints at stations 23+28.2 and 23+78.2 and the arch voids from station 23+28.2 to 24+14 were grouted. The following day the station 24+35.2 joint, the station 24+92.2 joint, the cooling coils, and the drainpipes between the high-pressure gates were grouted.

After grouting was completed the contractor finished the excavation which consisted of removal of the invert of the old concrete lining between the backfill concrete and station 26+11.72.

(d) *Spillway Elbow Lining Section.*—It was decided to place the entire length of the invert of the spillway elbow lining by the slipform screed method. Placing of invert concrete was started on May 12, 1966, using a sliding screed pulled with two air-operated winches. The concrete was trowel finished by hand behind the screed. Water was kept off the fresh concrete by a roof suspended over the area on cables; the roof was made of fiberglass-reinforced sheet plastic. Seepage water was controlled by a gravel blanket and perforated pipes draining under the concrete to the downstream end and into a sump. The concrete was delivered from the trucks to the screed by belt conveyors. Placing of invert concrete was completed on May 28, 1966. The form jumbo which had been suspended in the existing portion of the spillway was rigged with eight-part lines each on two 50-ton air-operated winches. Wheels were mounted under the shortened needle beams and the form was lowered into the downstream end of the elbow. This had to be done in two steps as the winch drums did not hold enough line to lower the form all the way to station 26+11.72. An extra set of anchors was set at approximately station 24+40 when invert concrete was placed. The form was tied off to these and the pulling lines were brought down to the anchors to lower the form to final position. The form required a complete new plywood skin, and part of the sheathing under the skin had to be replaced. The bottom wood buildup was removed from the form, leaving a lapover onto each edge of the invert concrete.

Preparations for lining the elbow are shown in figure 252. The first concrete placement in the arch of the elbow lining was made on July 11, 1966, using two lines off the pumpcrete machine with one going to either side of the form. After a number of delays the placements were completed and closure was made at station 23+82 on October 7, 1966. The major problem encountered during arch placements was excess water at invert construction joints. Approximately 75 percent of the entire surface was covered with metal sheets or polyethylene shield to divert the water into a drainage system provided through the invert concrete which drained into a sump at the lower end.

Figure 252.—Installing reinforcing bars in elbow section of left spillway. Slipform is in background. P557-420-12375, June 14, 1966.

A subcontractor, Lynch Brothers Drilling Co., started grouting the drainage system from the invert sump at about station 26+00 on October 28, 1966, working 21-hour shifts. Grouting of the drainage system and arch voids was completed on November 7, 1966. The total amount of cement used was 2,713 sacks; 125 sacks were pumped into the 12-inch dewatering drainpipe.

On November 12, 1966, the subcontractor started drilling and grouting the radial grout holes from the first ring at station 25+89 and continued upstream to station 23+85. Drilling and grouting radial holes was completed on December 30, 1966, with a total of 106 holes 40 feet deep.

Drilling for the 3-inch drain holes, which were 25 feet into rock, started on December 19, 1966. Between stations 26+95 and 23+08, a total of 22 rings with 4 holes each were drilled, as well as 4 diagonal holes on the centerline invert. The drilling for drain holes was completed on January 19, 1967. All drain holes and 2-1/2-inch connecting pipe headers were then flushed open and clear. The lower drains were capped and the surface recess was dry-packed or filled with epoxy mortar.

Boyles Brothers subcontracted the work for placing epoxy mortar and bonded concrete invert repairs. The contract required repairs to existing invert concrete downstream from station 26+11, because of heavy erosion that had occurred during the diversion period. Technical assistance with the repairs and help in organizing crews for epoxy work were obtained from the Denver office. Water control was a significant problem. Construction joints and cracks in the lining had to be calked with lead throughout the repair area. Temporary hot patch dikes and tapped drain holes above the springline were necessary to divert water outside the repair area. Temperature control was provided by two air-jet heaters.

During cleanup preparations for epoxy-bonded concrete, a rock pocket extending from the surface to foundation rock, was discovered. The unsound concrete lining was repaired by the contractor with payment made as extra work. Epoxy-bonded concrete was placed from February 2, 1967, to February 9, 1967, on the invert to 11.75 feet each side of centerline. Epoxy-bonded concrete was placed on the left side in the rough, eroded surface to the proposed limits between stations 26+11 and 27+20 and was also the most practical solution for repairing another area between stations 26+32 and 26+98.

Epoxy mortar repairs started on February 16, 1967, and were completed on March 7, 1967. After completion, the surface was found to have several irregularities which required grinding to allowable tolerances. This work required two finishers to grind out surface irregularities from March 16, 1967, to April 7, 1967, between stations 26+11.72 and 27+84.

The finishing crew completed the work on repairs, grinding, and epoxy patching of the spillway raise invert and sidewalls to centerline between March 2 and 20, 1967. Using the drilling and grouting jumbo the crew consisted of a finisher foreman, three finishers, two laborers finisher helpers, two hoist operators and two ironworkers. Part of the repair work involved repairs to lining constructed by others for which payment was made as extra work. Finishers with the same crew continued on down through the elbow section, epoxy patching, and grinding abrupt and gradual irregularities to required tolerances. After this work was completed on April 20, 1967, the jumbo was dismantled and removed from the tunnel.

The construction access adit to tunnel station 36+02 was prepared for the concrete plug. The periphery grouting system and metal seals were installed, as well as two coils of 1-inch aluminum cooling pipe and one thermocouple.

Concrete was placed by pumping methods on May 8, 1967, and tailrace water at 48° F. was circulated for cooling concrete. The maximum temperature reached 140.6° F. and cooling was necessary for 15 days to bring the concrete to 69° F. On May 24, 1967, the plug arch, periphery, and cooling coils were grouted using 5 sacks of cement at 1:1 water-cement ratio. Final cleanup of the immediate worksite and other working and storage sites required about 2 weeks to complete.

B. CONSTRUCTION OF DAM

161. KEYWAY EXCAVATION. Excavation of the right keyway began in October 1957 and was started in the left keyway in December 1957. Work continued in the keyways until July 6, 1959, when a strike against the contractor started. At that time the left keyway had been excavated to elevation 3100 and the right keyway to elevation 3080. Figure 253 is a general view of the dam construction area on May 20, 1959.

Excavation material in the keyways was at first pushed over the side by bulldozers and removed from below by shovels and hauling equipment. A 3-1/2-cubic-yard shovel assisted in moving the excavated material toward the edge of the canyon. Most of this material was used in the cofferdams. Some of the material fell in the river and had to be removed by draglines. Keyways were excavated in 10- to 12-foot lifts, generally using several rounds to remove the entire level so that while a blasted area was being mucked out another round of blast holes were being drilled in another section of the level. The drill pattern and amount of charge varied considerably according to conditions, but usually from 3,000 to 5,000 pounds of 40 percent powder was loaded in from 200 to 400 vertically drilled holes and 3 to 5 delays were used. Drilling was accomplished with up to six wagon drills.

In November 1958, when the right keyway had been lowered to approximate elevation 3265, a haul road was constructed from the keyway to the canyon

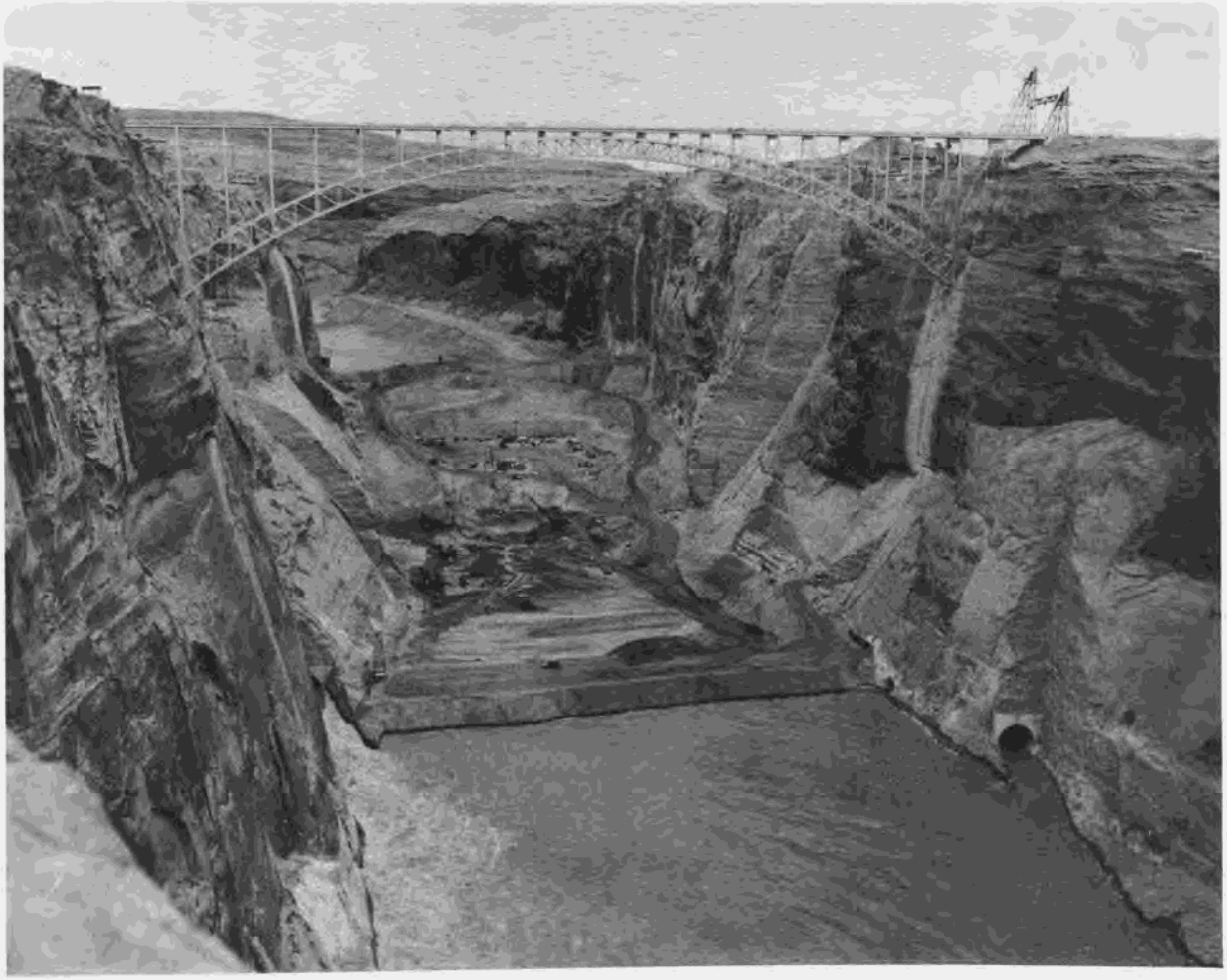


Figure 253.—Glen Canyon damsite as seen from progress photo point No. 1, approximately one-half mile downstream from the damsite on the west rim of the canyon. Note the heavy equipment working on the keyway foundation. P557-420-3817, May 20, 1959.

bottom and the excavated material was then moved over this road. In December 1958 a haul road for excavated material was built from the canyon bottom into the left keyway. The keyway had been lowered to approximate elevation 3280 at this time.

Three foundation tunnels were driven in each keyway. The foundation tunnels were designed with dimensions of 5 by 7 feet, but the contractor was permitted to excavate a l-foot arch in the roofs in order to use a crawler-mounted overshot loader. This extra excavation was at his request and at his expense. In the right keyway the tunnels were at elevations 3630, 3480, and 3247.5 and for lengths of 254, 317 and 565 feet, respectively. In the left keyway, the elevations were the same and the lengths were 250, 350 and 451 feet, respectively.

Because of poor rock conditions it became necessary to retrim the downstream toes of both keyways. This work began the week of April 19, 1959, and at that time the right keyway had been excavated to approximate elevation 3123 and the left to elevation 3131. The right keyway was retrimmed or deepened at the downstream toe from elevation 3465 to elevation 3280 and the left keyway from elevation 3300 to elevation 3175, when during the week of June 14, 1959, the work was suspended and excavation plans revised. On June 30, 1959, retrimming of the right keyway was resumed between elevations 3390 and 3200 to revised excavation lines.

On July 25, 1959, a fallout occurred at the upstream edge of the excavation for the left keyway between approximate elevations 3350 and 3510. During September 1959 an additional section of this rock fell out. The dangerous overhang above the fallout was removed in February 1960 and retrimming operations in the downstream section started from elevation 3300 down. The keyway was completed to elevation 3080 in April 1960 and the foundation was then sloped from elevation 3080 to elevation 3030 in the river bottom. Chain link mesh was anchored across a joint at the upstream edge of the left keyway excavation between approximate elevations 3250 and 3150.

From field studies early in August 1960, it was apparent that a rock slab on the left canyon wall, immediately downstream from the toe of the dam, needed anchoring. The area investigated was between stations 4+10 and 5+14 at about elevation 3170 and between stations 4+10 and 5+37 at about elevation 3100.

Six test holes drilled at approximate elevation 3150 indicated that the slab at that location was 8 to 9 feet thick with a 3/4- to 1-inch opening behind it. Seven test holes at about elevation 3115 indicated that at that level the slab was 6 to 6-1/2 feet thick with a 1/2- to 3/4-inch opening. Four other test holes, drilled at random locations between stations 4+10 and 4+45, indicated that between these stations the slab was 4 to 5 feet thick with a 1/2- to 3/4-inch opening between it and the solid rock of the canyon wall.

A price was negotiated with the contractor for extra work and seventy 20-foot anchor bars were placed to anchor the slab to the canyon wall. After the mass concrete was placed in this area, the opening between the slab and the solid rock was grouted by very low pressure grouting.

Two 5- by 7-foot drifts were driven into the right keyway in April and May 1960, both near the hill of the dam and at invert elevations 3112 and 3065, respectively. The purpose of the drifts was to observe seepage and to provide a seepage cutoff. The drift at elevation 3112 was stopped at 73 feet as very little seepage water was encountered. The lower drift was driven 173.7 feet. Considerable seepage occurred in this drift as it was driven along the weeping seam which extended into the abutment at elevation 3070. A total flow of approximately 50 gallons per minute seeped throughout most of the length of the drift, but for the last 25 to 50 feet only very light weeping occurred. Both drifts were backfilled with concrete and grouted. A french drain was installed in the lower drift, and after the arch void grouting was completed the drain was also grouted.

Work in the dam foundation area was started about April 1, 1959, and at the time of the strike had reached elevation 3078 in the upstream section. With the resumption of work in January 1960, mucking was started with draglines in the upstream and downstream sumps, and excavation followed in the area to the left of the plane of centers in the upstream section.

This area was gradually extended downstream to include the area between the dam and powerplant and into the powerplant area. The left side of the dam foundation was sloped from elevation 3080 at the base of the keyway to elevation 3030 in the upstream of "A" section of the blocks. The downstream or "B" sections were lower with the elevation on the left of block 10-B at elevation 3020 and sloping down to elevation 3004.7 at the left of block 11-B, lowest point of the dam foundation, and back to elevation 3020 in block 13-B. From this point the slope was relatively uniform to elevation 3110 at the base of the right keyway.

Figure 254 shows early concrete placement in the dam and powerplant.

1. Concrete Production and Placement

162. GENERAL DESCRIPTION. The bulk of the prime contractor's plant and equipment required for the work was directly related to the production and placement of concrete. The major components were the aggregate processing plant and the concrete mixing plant. Screening and washing of aggregate was performed at a plant located on a natural bench over Wahweap Creek some 6 miles from the worksite. The concrete batching and mixing plant was located on a bench excavated into the right canyon wall adjacent to the dam. In addition, a large refrigeration plant was erected near the concrete mixing plant, primarily to

cool the concrete, and two 50-ton and one 25-ton cableways were constructed to transport the concrete and other materials. A flow chart for the aggregate processing plant is presented in figure 255.

163. AGGREGATE PIT OPERATION. Before pit excavation operations were started, the Wahweap Creek channel was diverted and confined to protect excavation operations from the intermittent and silt-laden flows of Wahweap Creek. The contractor constructed this channel adjacent to the left bank of the deposit, by utilizing waste overburden. An embankment was constructed 10 to 15 feet in height with a berm width of about 12 feet, which maintained the creek width of about 20 feet at the same grade and elevation as the surface of the original channel. The Wahweap Creek was diverted at the upstream end of the operations, station 170+00, by an earth embankment constructed across the channel from bank to bank, approximately 20 feet in height and 2,000 feet in length. Self-loading scrapers and bulldozers were used for removing the overburden and depositing on the dike embankments. No special compaction was used for constructing these embankments, except that obtained by the earth-moving equipment.

At the beginning of excavation operations in the borrow area, no attempt was made to dewater the pit. However, it was found that the saturated pit-run material could not be elevated on the 18^{o} slope conveyor belts delivering material to the aggregate plant. The borrow areas were therefore dewatered by pumping and excavation proceeded to bedrock. The face of the pit was developed from the lower end of the deposit to take advantage of the natural drainage slope. Excavation in the pit was performed by the use

Figure 254.—View downstream from right spillway showing concrete blocks rising in dam and powerplant area. P557-420-5165, August 4, 1960.

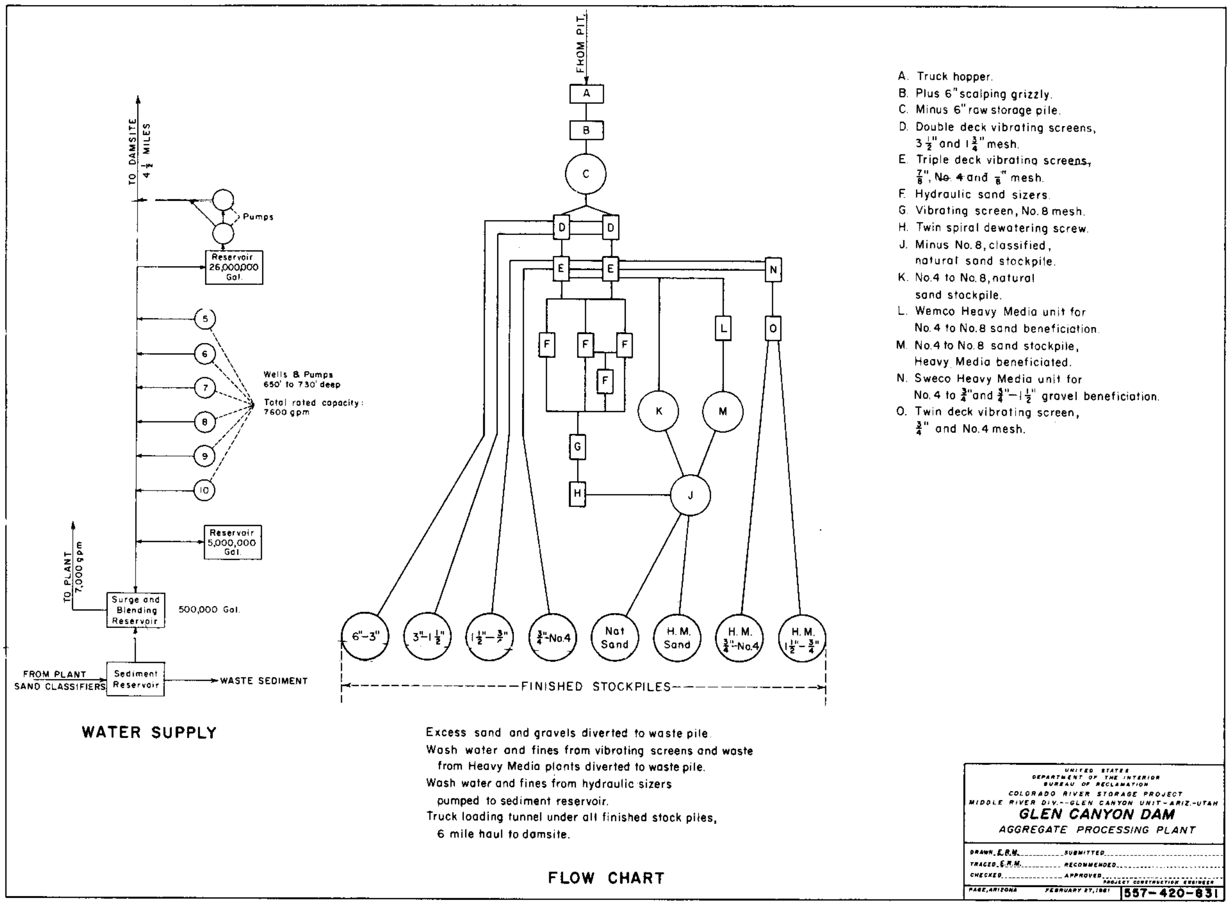


Figure 255.—Aggregate processing plant flow chart.

of two draglines with 6-1/2- and 7-cubic-yard capacities, usually loading into a 50-cubic-yard hopper. The contractor initially used twelve 35-cubic-yard-capacity bottom-dump trucks in the pit excavation. Under full production, six of these trucks were replaced with four 45-cubic-yard bottom-dump trailers, drawn in tandem, for use on the longer hauls.

Raw material delivered from the borrow areas was discharged into a 100-cubic-yard unloading hopper at the base of the canyon cut by Wahweap Creek. The pit-run material discharged from the hopper onto a vibrating scalping screen equipped with parallel bars with openings between the bars of 6 to 6-1/2 inches. Rock larger than 6-inch size, often chunks of clay or tabular sandstone, were wasted at this point. Material passing through the vibrating grizzly was fed through a belt feeder onto a 36-inch conveyor belt traveling 640 feet per minute on an 18^{o} slope. The material was conveyed to a balancing raw storage stockpile having a live storage capacity of approximately 5,000 tons, which provided uniform feed to the plant and at least 3 hours of storage margin. The material passed through a metal-lined cone or hopper at the base of the stockpile, where the amount of feed required was controlled by a manually operated gate above a belt feeder. The feed fell directly onto a 36-inch endless conveyor, 1,800 feet in length, also on an 18^{o} slope, which delivered the material to the top of the aggregate screening and washing plant. This conveyor was equipped with a recording-type weighing scale which recorded the instantaneous load and the total daily tonnage.

164. AGGREGATE SCREENING AND WASHING PLANT. The prime contractor, Merritt, Chapman and Scott Corp., designed and erected a screening and washing plant to supply the coarse and fine aggregate required for producing concrete at a sustained rate of 420 cubic yards per hour at the concrete batching and mixing plant. The plant was located on the right bank of the Wahweap Creek channel at elevation 3715,

opposite area B, and approximately 225 feet above the channel floor.

To obtain the required aggregate size gradation, the minus 6-inch raw material passed through a complex flow pattern. Water was introduced as the material was fed to the plant and was further washed with spray bars over all of the vibrating screens. Approximately 8,000 gallons per minute of water at a pressure of about 80 pounds per square inch was used while washing and screening. The first set of two sloping, vibrating screens, each having a double deck with screen meshes of 3-1/2 and 1-3/4 inches, removed the 3- to 6-inch and 1-1/2- to 3-inch coarse aggregate. Both of these size fractions were conveyed directly to the finished stockpiles located at the aggregate plant, without further processing.

Aggregate which passed through the 1-3/4-inch mesh screen was routed by gravity to two sloping vibrating screens, each having a triple deck with screen meshes of 7/8 inch, No. 4 size, and 1/8 inch. Aggregate retained on the 7/8-inch mesh screen was conveyed to the 3/4- to 1-1/2-inch finished stockpile, or a portion could be diverted for heavy-media processing to remove lightweight rock. Aggregate which was retained on the No. 4 and 1/8-inch screens was similarly handled going to the No. 4 to 3/4-inch and the No. 4 to No. 8 stockpiles or a portion of these could also be diverted for the heavy-media process.

The minus 1/8-inch sand was routed to three hydraulic sizers where the sand was separated into fractions. The excess fines, consisting of the No. 8, No. 100, and minus No. 100 were wasted and the remaining portion was diverted by gravity to two 30-inch spiral screw classifiers where it was further blended and dewatered before it was conveyed to the minus No. 8 finished stockpile. Initial sand production results in three initial stockpiles, one containing the No. 4 to No. 8 sand fraction, not heavy-media treated. One stockpile contains minus No. 8 sand and the third stockpile contains the No. 4 to No. 8 heavy-media treated fraction. Final sand production required blending approximately 20 percent of No. 4 to No. 8 untreated sand with 80 percent of the minus No. 8 sand to produce untreated natural sand, or 20 percent treated No. 4 to No. 8 fraction with 80 percent of the minus No. 8 fraction to produce heavy-media treated sand.

This completed the screening cycle of raw material from the borrow areas. It should be noted that an excess of any aggregate or sand fraction could be diverted to the waste pile on a separate conveyor without affecting the rest of the plant operation. During processing of coarse aggregate, only the No. 4 to 3/4 inch and the 3/4- to 1-1/2-inch material was in excess. Most of the excess No. 8 to No. 4 fraction of the fine aggregate was wasted in the hydraulic separators and diverted to the coarse aggregate waste pile, but a portion of this waste was used for the production of sandblast sand. The remaining waste fine aggregate was diverted with the plant wash water by gravity to the excavated area at the downstream end of area B where settling basins were constructed for separating the waste sand from water. A portion of this water was reused again, as discussed in section 166.

No serious problems during production of aggregate were encountered, although various minor difficulties were experienced. One problem involved the recovery of critical size fractions in sand, especially the No. 30 sand. The actual cause of this difficulty originated in the overloading of the lower deck of the triple-deck shakers containing the No. 8 screen. Consequently, some of the No. 16 and No. 30 sizes passed over and were not saved. Some relief of this condition was gained by replacing the No. 8 screen with a 1/8-inch screen, allowing the coarser material to pass on into the hydraulic sand separators. When leaving the hydraulic separators, it was removed by passing over a No. 10 screen, provided with water sprays. This further removed the smaller sizes of sand which fell directly in the sand dewatering screws. This operation allowed the contractor to salvage the No. 8 size fraction at a convenient location and divert some of the excess material to a rotary dryer for the production of sand blast sand for use in concrete construction joint cleanup. Approximately 100 tons of this sandblast sand was produced per day.

165. HEAVY-MEDIA SEPARATION. The raw material from all borrow areas contained variable amounts of lightweight, unsound, and deleterious rock. Preconstruction investigations determined that most of these deleterious materials were contained in the minus 1-1/2-inch aggregate, with most of the lightweight and undesirable material present in the No. 8 to No. 4 fraction. Tests also indicated that concrete of good quality could be produced when lightweight material below 2.50 specific gravity was removed. As a result, the specifications for Glen Canyon Dam and Powerplant required that all fine aggregate between the No. 8 and No. 4 sand size and all coarse aggregate between the No. 4 and 1-1/2-inch size would be subject

to heavy-media treatment when used for production of exposed, or exterior, concrete. This concrete included a minimum thickness of 5 feet on both faces and the top of the dam, the lining of the diversion and spillway tunnels, and all powerplant concrete, except the initial mass concrete, amounting to about 10 percent of all concrete. The specifications also provided that lightweight material having a specific gravity of 2.50 or less would not exceed 2.0 percent by weight of the sink in the above two fractions. As this process is somewhat unusual, considerable details follow in this publication.

For production of this aggregate, the contractor incorporated two heavy-media separation units (fig. 256) in conjunction with operation of the screening and washing plant to meet the requirements of the specifications. An early attempt was made to process a composite feed of No. 8 to 1-1/2-inch size range, which proved unsuccessful when using the spiral screw in an inclined tank type of classifier. The main difficulty appeared to be with weir openings. Being set for enough flow to permit passage of the larger size lightweight particles, many fine heavy particles were swept by the medium current over the weirs, and many were carried on top of larger lightweight particles. As much as 90 percent of No. 8 sand was lost with the float product in field trials. As a result, a separate heavy-media unit was installed consisting of a cone-type separator for treating the No. 8 to No. 4 sand fraction. The key elements of both types of separators are the separating vessel and the circulating medium. The medium contains 30 to 40 percent magnetite having a specific gravity of 4.7 and ground to 100 mesh, and 60 to 70 percent ferrosilicon having a specific gravity of 6.9 and ground to 65 mesh. These finely ground materials are proportioned and mixed with water to make a suspension having a specific gravity in the separating vessel which would be sufficient to separate the lighter particles from the heavy particles at a division point of 2.50 specific gravity. The media in suspension in the cone separator ranges from 2.35 specific gravity at the surface to 3.05 at the bottom. In the inclined-screw type classifier, the specific gravity of the media as it enters the system ranges between 2.47 and 2.50.

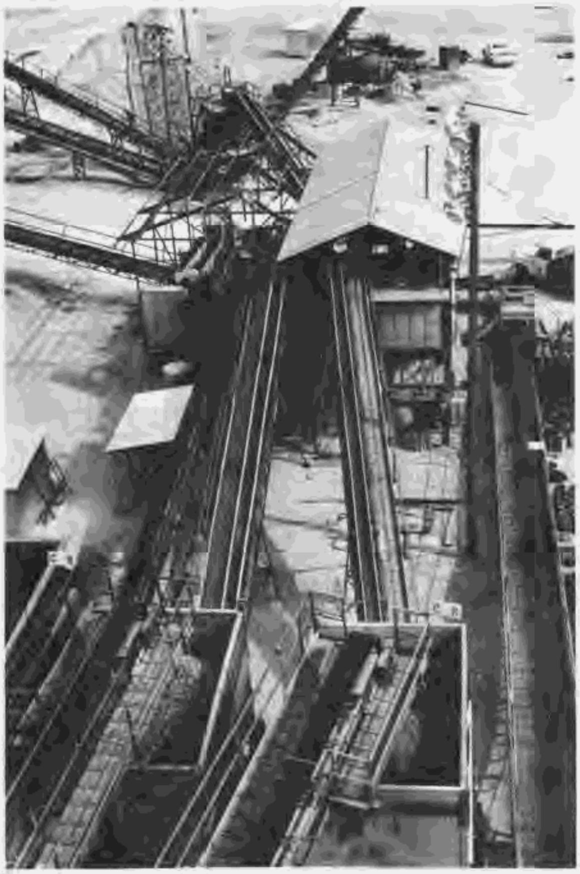


Figure 256.—Heavy-media separation plant (center) at Wahweap aggregate plant. Separator for No. 8 to No. 4 sand is on the left and separator for 3/4- to 1-1/2-inch aggregate and No. 4 to 3/4-inch aggregate is on the right. Raw feed bins are at bottom. Raw feed belts to plants are at center. Waste conveyor is at the right. At top left is screening plant for separation of No. 4 to 3/4-inch and 3/4- to 1-1/2-inch beneficiated aggregate. At top center rod mill is being erected for production of sandblast sand. P557-420-4609, March 25, 1960.

Aggregate to be heavy-media processed was diverted from the regular aggregate plant and conveyed to three 200-ton storage bins. Untreated aggregate was drawn from these bins and passed over dewatering screens before entering the heavy-media plant. These dewatering screens provided a more uniform moisture content and better control of the heavy-media specific gravity. Due care was taken to thoroughly wash the material, as dirty material affected the specific gravity of the medium, causing more frequent cleanup and recharging costs.

Aggregates were fed into the separation pool at the lower end of the tank. The coarser fraction, 3/4 to 1-1/2 inch, entered at the side of the separation pool while the finer fraction, 3/16 to 3/4 inch, was fed into the end of the pool where the depth was greatest and the material had a longer retention time. The heavy aggregate particles sank through the medium and were carried up the inclined tank by the rotating spiral. They were discharged from the upper end onto one side of a compartmented vibrating drainage screen. The light particles floated on or near the surface of the separation pool and were carried with the medium overflow through weirs located on the side of the tank opposite that of the aggregate feed. The medium, transporting the float particles, passed through a channel to the float side of the compartmented screen.

The first few feet of the compartmented vibrating screen were located directly over a medium drainage sump. The medium draining from the aggregate was collected in this sump and continuously pumped back to the separating vessel. Most of the medium leaving the separatory vessel was returned in this manner. As the aggregate products continued along the vibrating screen, they passed under washing sprays where the remaining medium clinging to the particles was removed. The products were then discharged separately at the end of the screen in a dewatered condition. The finished product was then conveyed to a rescreen plant where the two coarse fractions were separated and conveyed to the aggregate plant stockpiles.

The water and medium which were washed from the aggregate were collected in a wash sump. This diluted medium was pumped to a magnetic separator where the magnetite and ferrosilicon were concentrated into a semidewatered condition. Nonmagnetic solids and excess water were wasted. The magnetic concentrate recovered from the magnetic separator flowed into a spiral densifier. Here the reclaimed medium was dewatered and continuously fed back into the spiral separator. The rate of return of the medium was controlled by raising or lowering the densifier spiral. An electric coil was placed in the circuit between the densifier and the separator for the purpose of demagnetizing the solids as they were discharged, thus preventing flocculation of the medium within the separatory vessel.

A cone-type separatory vessel was used for processing No. 8 sand. The cone had a greater depth and volume than the spiral separator and was capable of retaining a given rate of feed for a longer period in the separation pool. It consisted of an inverted 7-foot-diameter cone equipped with paddles to agitate the heavy medium, and appurtenances similar to those used with the spiral separator for product drainage, medium reclamation, storage, etc. The heavy particles sank to the bottom of the cone where they were removed through a vertical pipe. The sink was carried upwards in this pipe by the circulation of the medium and the action of an air jet; it was then deposited at the upper end onto a drainage screen. The float material flowed over a weir at the top of the cone and was deposited on the draining and washing screen for medium removal. Treatment beyond this point was similar to that described for the spiral-type plant.

The cone separator was operated to obtain a large differential between specific gravities of the medium at the top and bottom of the separating pool. This was helpful in producing efficient separation, particularly when there was little difference between specific gravities of many sound and unsound aggregate particles in the feed. Tests showed that the larger the differential, the better the product obtained. The differential in medium specific gravity increased with increasing depth of pool, with increasing ratios of ferrosilicon to magnetite, and with reduced agitation and circulation. These factors also increased the possibility of plugging, in the circulatory system, with resultant delays and costly cleanout and restarting expenses. Therefore, it was important to frequently sample the medium near the top and bottom of the pool and maintain a differential that would provide efficient separation and smooth operation. Although the specific gravity of separation was 2.50 for the aggregate used in the dam, separation can be accomplished at any required specific gravity up to about 3.0. When it is desired to separate below a specific gravity of about 2.40, a slurry with magnetite alone may be used; for separation at a specific gravity of 2.80 or above, a slurry of ferrosilicon alone can be used.

The specific gravity differential diminished when the medium became contaminated with silt and clays appearing on the surface of the aggregates. These slimes also increased the viscosity of the medium and lowered the average specific gravity. It was important, therefore, to continually remove slimes even at the expense of losing a small amount of ferrosilicon and magnetite. This was done by regulating overflow weirs in the tank containing the magnetic separator as well as regulating the rotational speed of the magnet. Proper weir setting would assist in washing most of the slimes out of the system without excessive waste of magnetite and ferrosilicon. Average loss of the latter materials at the dam was estimated at about 1 pound per ton of sink product.

166. WATER SUPPLY. A substantial amount of water was required to thoroughly wash aggregates, manufacture sand by the wet process, remove water soluble sulfate and chloride salts, plus supply normal plant cleanup needs. The flow from Wahweap Creek was small, erratic, and the water was high in sulfate content. The silt content in the Colorado River, some 5 miles distant, was high even during normal flow, and the cost of desedimentation and pumping would be high. The contractor elected to drill six wells near the aggregate processing plant in order to provide nearly 8,000 gallons per minute needed for full operation. The six plant wells were drilled to depths ranging from 625 to almost 800 feet and had a combined rating capacity of 7,750 gallons per minute. Four other wells were drilled to provide domestic water for construction purposes, the contractor's camp, and shop areas on the west side of the Colorado River near the damsite.

Four earth reservoirs were provided: two for storage and holding 26,000,000 and 5,000,000 gallons; one for sediment settling, and one surge and blending reservoir for holding 500,000 gallons. The system was designed so that water could be pumped to the storage reservoirs, the screening plant, or damsite, or pumped from the reservoirs to the other features. Water used in the screening plant returned to the sediment reservoir. About 20 to 30 percent of the used water was mixed with 70 to 80 percent of fresh water in the surge and blending reservoir for reuse in the screening plant. Extent of water reuse was governed by the soluble sulfate in the finished sands; and the sediment reservoir was usually drained, cleaned and refilled with fresh water about once a week. Replacement of pump bowls was necessary about every 4 months, and occasional periods of short supply did occur when more than one pump was out of service for repairs. However, the water system proved adequate for job requirements and produced well-screened and clean aggregate. Settling basins in the Wahweap Creek were developed later and use of the basin at the aggregate plant was discontinued.

167. STOCKPILES OF AGGREGATE. Processed aggregate was stored in large stockpiles adjacent to the screening plant. The stockpiles were arranged in a single row (fig. 257) with one stockpile each for natural (or untreated) material of 6- to 3-inch, 3- to 1-1/2-inch, 1-1/2- to 3/4-inch, and 3/4-inch to No. 4 sizes; graded sand; heavy-media treated sand; heavy-media treated 3/4-inch to No. 4 aggregate; and heavy-media treated 1-1/2- to 3/4-inch aggregate (fig. 255). The heavy-media treated sand stockpile contained the No. 4 to No. 8 size fraction which was beneficiated and then recombined with the minus No. 8 natural sand (see sec. 164).

A large tunnel extended under the line of stockpiles where 30-ton bottom-dump trucks were loaded for the 6-mile haul to the concrete batching and mixing plant storage stockpiles. There were no steep grades on the gravel-surfaced haul road, and the only maintenance normally required was use of a water truck to keep down dust and a grader to maintain the roadbed.

As the six separated sizes of coarse aggregate were delivered to the damsite, they were stockpiled according to size. The bottom-dump trucks arriving from the aggregate plant discharged into an unloading hopper of sufficient capacity to hold two truckloads. The material was conveyed from the hopper to an overhead, shuttle conveyor which extended above the stockpiles. The elevated tracks on which the reversible shuttle was mounted ran the full length of the stockpiles; the shuttle conveyor was one-half the length of the tracks. When concrete operations got under full production, it was found that too much time was consumed moving the conveyor to the various stockpiles. A separate conveyor was therefore installed for stockpiling the 3- and 6-inch aggregate. Rock ladders were constructed for each coarse aggregate stockpile greater than the 3/4-inch size. Material was drawn from the bottom of the stockpiles from two different gates for each aggregate for delivery to the batching plant.

The two types of graded sand were stockpiled by the use of a radial stacker, drawing material from a common unloading hopper. The radial stacker traveled on tracks through an arc of 135^{o}, delivering the heavy-media-treated sand at one end of the arc and the natural sand at the other. Heavy-media- treated sand was drawn from the bottom of the stockpile through one of three different gates for delivery to the concrete batching plant, while the natural sand was drawn from one of seven different gates. Multiple gates were provided to assist in obtaining greater consistency in moisture content for production control purposes.

The contractor operated the stockpile area with five men on each shift; one operator for the coarse aggregates and two for the sands, one laborer in the reclaim tunnel, and one laborer for general work.

168. CONCRETE SPECIFICATIONS DATA SUMMARY. Specifications for the Glen Canyon Dam and Powerplant provided for the use of pozzolan for

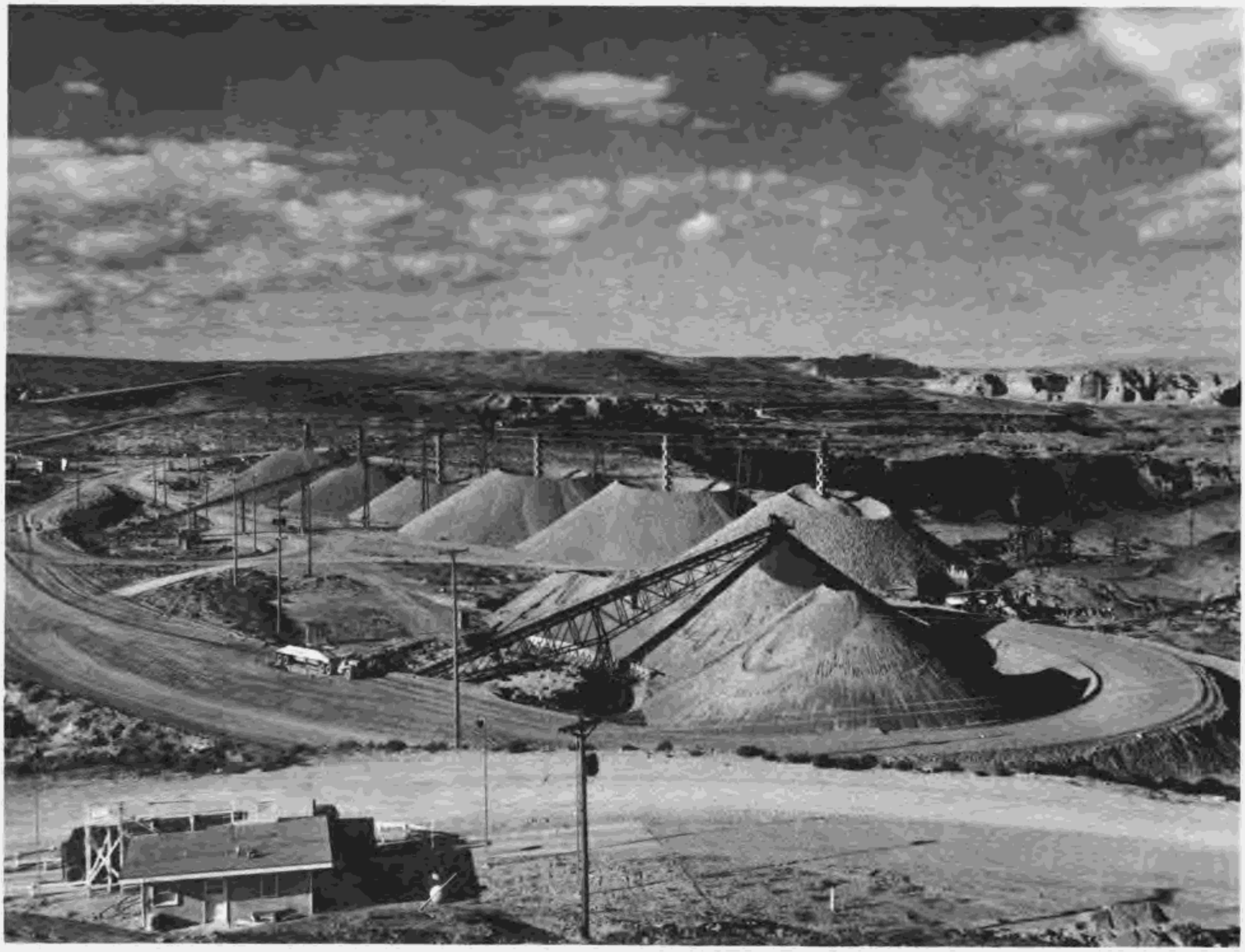


Figure 257.—View across the aggregate stockpiles near the west rim of Glen Canyon as seen from Little Beehive. P557-420-5487, November 14, 1960.

30 percent of the cementing material of concrete placed in the dam, mass concrete for powerplant and other massive concrete, as well as some interior or unexposed concrete.

The temperature of concrete placed in the right and left diversion tunnels and spillway tunnels was not to exceed 80° F. or be less than 40° F. The temperature of all other concrete, including concrete in the dam, was not to exceed 50° F. or be less than 40° F. Initial temperature restrictions aided in controlling the heat of hydration when the concrete was setting, which was supplemented by circulation of cold water through tubing embedded in the concrete. A water-reducing set-retarding agent, was used after concrete operations got underway, which was covered in orders for changes No. 2 and 12. The use of this agent also aided in holding heat of hydration temperatures down by further reducing the cement content.

The percent of air entrained in concrete varied according to the maximum size of coarse aggregate in the concrete in the following designed amounts:

Coarse aggregate maximum size, inches	Total air percent by volume
3/4	6 ± 1
1-1/2	5 ± 1
3	4 ± 1
6	3-1/2 ± 1

The slump of deposited concrete varied with the location and use, and these limits were followed:

(1) Two inches maximum slump: mass concrete containing 6-inch maximum coarse

aggregate; concrete placed in the tops of walls, piers, parapets and curbs; and concrete placed in tunnel inverts and horizontal, or nearly horizontal, slabs.

(2) Four inches maximum slump: concrete pumped or placed in side walls and arch of tunnel linings.

(3) Three inches maximum slump: all other concrete.

169. CONCRETE BATCHING AND MIXING PLANT. Before the large batching and mixing plant for full concrete production was available, the contractor erected a temporary plant to supply his plant construction needs and for supplying concrete for lining the diversion tunnels. This temporary plant was erected on the right side of the damsite and adjacent to the aggregate stockpiles. This plant was equipped with two 4-cubic-yard mixers, automatic weigh batchers, and automatic recording devices for batch weights and consistency. Final screening of aggregate was performed adjacent to the plant and cement was stored in three 2,000-barrel-capacity silos. Bulk cement from the Permanente Cement Co., Lucerne, Calif., was transported by truck and furnished by the contractor. Discharged into 4-cubic-yard cableway buckets, concrete was transported on flat-bed semitrailers to a cableway erected for the diversion tunnel construction. The plant was capable of producing about 80 cubic yards of concrete per hour. No significant difficulties were experienced with the operation of this plant.

Late in 1957, the Noble Co., Oakland, Calif., was given the task of designing a plant which could produce concrete on an average hourly rate of 420 cubic yards, and a maximum rate of 480 yards for short periods of time. The plant was to be as automated as possible and still maintain quality control to produce concrete in compliance with the specifications.

Usually called the mixing plant, the concrete batching and mixing plant was located on a bench excavated from the west canyon wall at elevation 3540 just upstream from the right abutment of the dam. The bench also served as a roadbed for the tracks of the electric-powered transfer trains which moved the concrete-loaded ladle buckets from the mixing plant and across a structural steel trestle for transfer to the cableway buckets.

Concrete footings for the plant were placed during March 1959, and erection of the supporting steel frame began in April. Steady progress was made erecting steel until July when labor difficulties stopped all work on the project until January 1960. Erection of the plant was then resumed and was essentially completed in April 1960. Tests and checking out of all electrical and mechanical components were made during May and the mixing plant began operation in June 1960.

The octagon-shaped mixing plant (fig. 258) was 217 feet in height and contained six main floors. A large 3,000-ton-capacity aggregate storage bin was located near the middle of the plant. This bin contained eight compartments for the two types of sand and the six different sizes of coarse aggregates required for concrete production. The portion of the plant above the storage bin was called the screening tower; the batching and concrete mixing floors were beneath the storage bin.

Sand and coarse aggregates were transported to and entered the plant by belt conveyors. At the top of the plant, coarse aggregates passed over sloping, vibrating screens (final screening) and dropped into proper bin compartments. Sand was delivered directly from the sand stockpiles by a separate conveyor system to the mixing plant storage bins. Sand and aggregates were weighed through batching scales, discharged into one of six 4-cubic-yard mixers, and combined with cement, water, and required admixtures into a batch of concrete. The concrete was discharged from the mixers into one of three holding hoppers for loading into the 12-cubic-yard ladle buckets on the transfer trains. Two of the holding hoppers had a capacity of 26 cubic yards and the other 12 cubic yards.

Location of the plant on the rock bench below the canyon rim allowed almost horizontal delivery of materials to the plant and retained the advantage of gravity flow during production. Another prime factor in this location of the plant was that about five-sixths of the almost 5 million yards of concrete required would be located below the bench at elevation 3540, resulting in a substantial savings in the cost of operating the cableways.

(a) *Arrangement.*—At the top of the mixing plant, on floor 6, sand and coarse aggregates entered by conveyor from the cooling chamber and dewatering screens. A 42-inch reversible transfer conveyor, 10 feet in length, transferred sand to the proper storage bin compartment for "natural" sand or for heavy-media treated sand. Coarse aggregates passed through a transfer chute with the flow divided between two 6- by 14-foot sloping vibrating screens, located below on floor 5, where the aggregate received the first step in final screening.

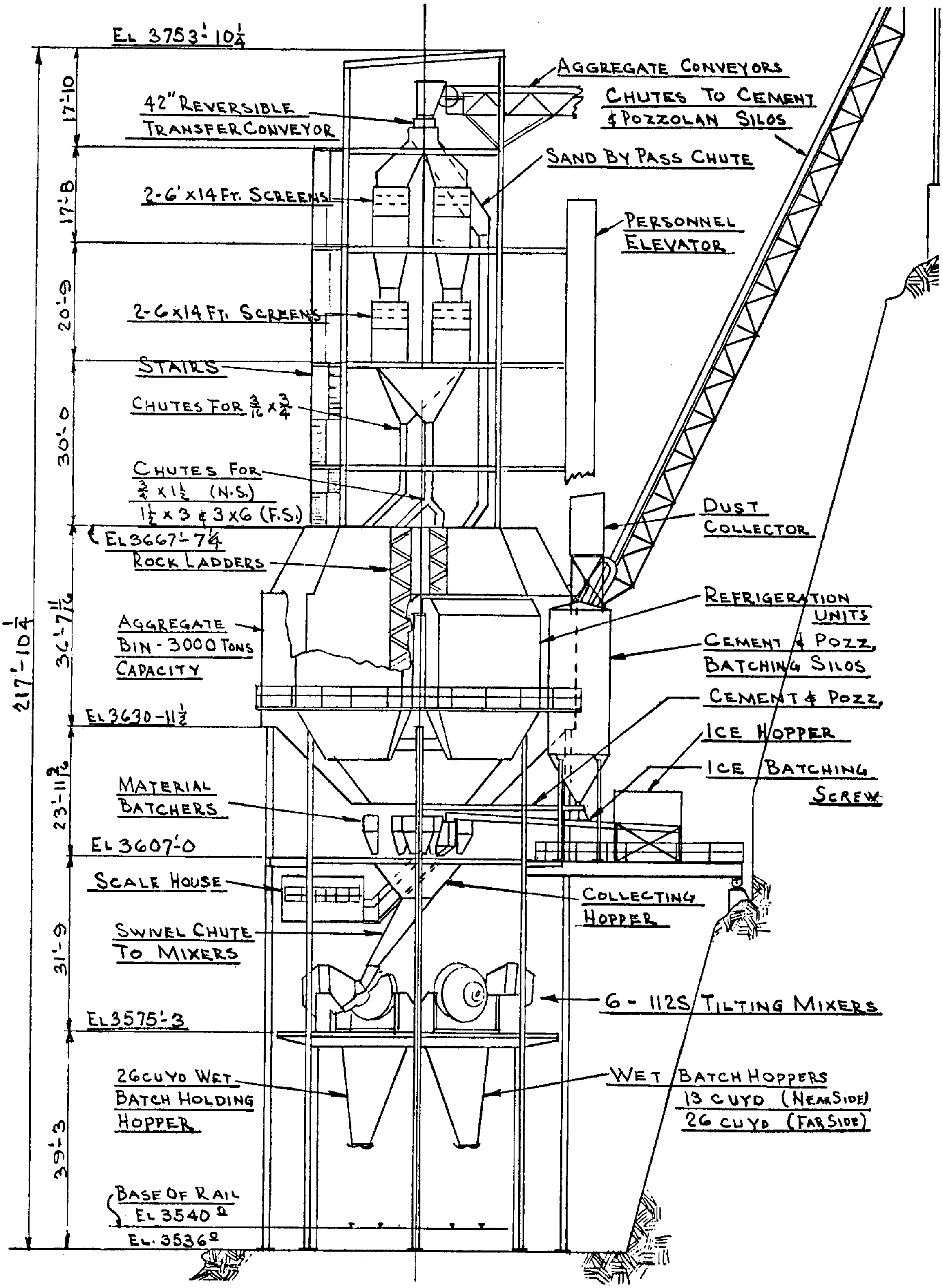


Figure 258.—Elevation of mixing plant.

Both of the two sloping vibrating screens on floor 5 were double decked. The top deck on each screen had a 3-1/2-inch mesh for final screening for the 3- to 6-inch coarse aggregate, which passed directly down a rock ladder to a bin compartment. The two lower decks had 1-3/4-inch mesh screens and gave final screening to the 1-1/2- to 3-inch coarse aggregate, which also passed by rock ladder to a bin compartment. Aggregate passing the 1-3/4-inch deck screens dropped below to sloping vibrating screens on floor 4.

There were two double-deck, 6- by 14-foot vibrating screens on floor 4, each with 7/8- and 3/16-inch mesh screens. The top deck on each screen removed the 3/4- to 1-1/2-inch coarse aggregate which passed by chute to a storage compartment. The lower screen decks gave final screening to the 3/16- to 3/4-inch coarse aggregate which also passed by chute to a storage compartment. Material passing each of the 3/16-inch mesh screens was wasted. When screening the 3/4- or 1-1/2-inch maximum size aggregates, there were flow arrangements available to divert either size of aggregate to one of two storage compartments depending on whether the material had received heavy-media treatment.

Floor 3 consisted of decking over the main storage bins and provided access to inspect the various bin compartments and repair worn chutes and rock ladders. Floor 2 was located below the 3,000-ton storage bin and was known as the batcher floor. Here the individual weigh hoppers were suspended below the storage bin compartments, along with others to weigh cement, pozzolan, water, ice, air-entraining agent, water-reducing agent, and calcium chloride solution. Gates to fill and empty each weigh hopper were operated electronically through an automatic sequence, or individually by manual control. The contractor's plant office, a tool and workshop room, and a materials laboratory were also located on this floor. The laboratory was maintained by the Bureau in order to check coarse aggregate and sand samples for grading and moisture content, and for control of liquid admixtures by specific gravity tests.

Adjacent to floor 2 and between the plant and right abutment wall two surge silos were located, one 750-barrel silo for bulk cement and one for 75 tons of pozzolan, and a 5-ton storage bin for crushed ice. Cement, pozzolan, and ice were delivered to their weigh hoppers, located inside the plant on floor 2, by remote-controlled screw conveyors. All weigh hoppers loaded to the predetermined amounts required for the mix of concrete being produced.

The six tilting mixers were installed on floor 1 in a circular pattern, being supported by a separate concrete foundation and structural members to reduce vibration in the rest of the plant. Each mixer had a capacity of 4 cubic yards and was charged from the front. The mixers were charged by a collecting hopper and a swivel chute, which is tapered to an offset angle with an opening of about 32 inches at the lower end. When the swivel chute was centered in front of a mixer, the lower end contacted a shorter length of chute, called the retractable chute, and the two sections provided a clear passage for charging that mixer with materials from the weigh hoppers on floor 2.

The control room at the top of the screening tower located above the 3,000-ton storage bin was equipped with pushbutton controls to enable the tower operator to draw sand and coarse aggregate from the storage stockpiles for delivery to the mixing plant on belt conveyors. A 36-inch-wide conveyor was used from the reclaim tunnel to the chilling chamber, a distance of 2,000 feet, having two shuffle points. A belt conveyor 42 inches in width and 250 feet in length was used through the chilling chamber. After the coarse aggregates passed through the chilling chamber, they passed through a vibrating double-deck dewatering screen by gravity, and then were conveyed to the top of the mixing plant, a distance of 150 feet. The plant storage bins were originally equipped with high and low bin indicators to show when the bins were nearly empty and when filled. This system was replaced with a closed-circuit television system which enabled the operator to see the actual bin level and to maintain better control on filling the compartments.

Coarse aggregates drawn from the storage stockpiles traveled through a covered section enclosing the conveyor called the chilling chamber. During hot weather, the aggregates were sprayed here with water chilled to 34^{o} to 40^{o} F. to lower their temperatures. Sand was delivered to the mixing plant on a separate conveyor system, but did not pass through the chilling chamber. The storage compartments for natural 1-1/2, 3-, and 6-inch maximum aggregates and the 1-1/2-inch maximum size heavy-media treated aggregate were insulated with exterior fiberglas blankets. Further chilling of these aggregates was provided by means of cold, refrigerated air, injected and circulated through the compartments.

The main plant control room was located above the mixers on a small mezzanine floor, adjacent to the downstream side of the plant. The control room was glass enclosed, giving a clear view of the mixers and swivel chute action, and was maintained under slight air pressure to reduce maintenance on electronic devices and meters due to dust. An open-mesh catwalk circled in front of and above the six mixers, enabling the operator and technicians to visually spot check the actual concrete mixing action in any one or all of the six mixers.

(b) *Operation.*—The control room operator could operate the mixing plant, either automatically or manually, by switches and pushbuttons conveniently arranged on a single console board. The operator had a choice of 12 different concrete mixes, which had been previously computed by a Bureau representative. A large panel of monitoring dial instruments were electrically connected to each of the weigh hoppers on floor 2 to show the progress of weighing materials used in a batch of concrete. The operator could readily see if the weigh hoppers were filled correctly or if any material batches were under or over the required weight desired. Underloads which if not immediately corrected disrupted the mixing cycle, and overloads greater than permissible specification limits would be corrected by addition or deletion of materials in proper amounts.

When the plant was set to operate on automatic production, a complete mixing cycle for each mixer took 3 minutes; one-half minute was allowed for filling and emptying the weigh hoppers and 2-1/2 minutes were used for the mixing time for each mixer. After any mixer was charged with the batch ingredients, the swivel chute automatically rotated clockwise to the next mixer. With the completion of the first charging cycle to all mixers, the plant continued to produce 24 cubic yards of concrete each 3 minutes until the charging and mixing cycle was stopped or the operator placed the plant on manual control.

During 2 years of maximum concrete production, an average of 335 cubic yards per hour on a 24-hour day, 5 or 6 days a week, was produced. During one 8-hour shift the plant produced the maximum plant capacity of 3,840 cubic yards, and the maximum production for a single 24-hour period was 10,127 cubic yards.

An automatic recording machine printed a graphic record of the weight of each material used in a batch of concrete. This record, or roll, was used to check the number of batches of each kind of concrete produced by shift, and to check on the amounts of materials weighed. Another recording machine, called a consistency meter, recorded the electrical current required to turn any mixer. Concrete having a "dry" consistency or low slump took more current to turn the mixer than a mix of a "wet" consistency or higher slump concrete. Both of these records became part of the permanent Government data and were retained for a period of time after completion of the work.

Communications throughout the various floors of the mixing plant and to other features of the jobsite, were facilitated by an elaborate system of telephones, intercoms, and two-way radios. Access throughout the plant was by stairways, ladders, and an outside elevator to all main floors.

(c) *Cement.*—Four silos were provided to store bulk cement, each having a capacity of 10,000 barrels. Three silos, each holding 1,000 tons, were provided to store pozzolan. Cement was delivered by truck from the Phoenix Portland Cement Co., Clarkdale, Ariz., a distance of 188 miles, under invitation No. DS-5023. Pozzolan was supplied under invitation No. DS-5053 by J. G. Shotwell from a plant located on pumice deposits about 25 miles north of Flagstaff, Ariz., and trucked 110 miles to the damsite.

Trucks containing cement or pozzolan were received at the Government scale house for weighing and then proceeded to the storage silos for unloading. The trucks bottom-dumped into a 14-inch screw conveyor which passed the cement or pozzolan to a bucket elevator where the material was lifted to air slides at the top of the silos and transported to the respective silos. Cement was transported to the mixing plant by screw conveyors from the bottom of the silos, then transported by air slides and gravity to the balancing storage silos, located adjacent to the plant at the batching floor elevation.

On the canyon rim adjacent to the plant, bulk storage tanks contained calcium chloride solution, air-entraining agent, and liquid water-reducing set-retarding agent, which were piped for gravity flow to the batching floor for dispensing.

(d) *Crew.*—The contractor operated the mixing plant with a crew of 12 men on each shift, with a general superintendent in charge of all work except the refrigeration plant. Each shift consisted of one foreman, one operator at the cement and pozzolan silos, one operator at the dewatering tower, one operator for rescreening operations at the top of the plant, one operator for batching and mixing, one dispatcher at the bottom of the plant for dispatching concrete, one signalman for routing trains, one electrician and one mechanic for maintenance and general repair, three or four general laborers for cleanup and general help.

(e) *Delays.*—During operation of the plant, three difficulties were encountered which slowed down concrete production at times. As previously mentioned, the final rescreening of coarse aggregate was performed at the top elevations of the plant, and plant storage was about 3,000 tons, or each aggregate bin had a capacity of 275 tons when full. However, it was not possible to maintain all the bins full all of the time and the operational level for each coarse aggregate was closer to 200 tons. In addition, the 36-inch conveyor delivering material from the stockpiles could not deliver more than three materials at one time. It was not possible to load 3- and 6-inch rock on the 36-inch conveyor belt at one time because all feeder gates were manually operated and required different settings for these fractions. Consequently, as these materials were alternated from one gate to the other, the storage bins had a tendency to become layered with the results that the aggregate grading in each bin was not uniform.

The rescreening plant had sufficient capacity if it had been possible to feed all the different materials uniformly at one time. This condition would have been improved by utilizing a wider conveyor belt between the reclaim tunnel and the chilling chamber and installing a belt feed in the reclaim tunnel for the 3- and 6-inch aggregate. When rescreening the two sizes of heavy-media-processed aggregate, only these two sizes of aggregate could be screened. Consequently, with sustained operation above 400 cubic yards of concrete per hour, the storage of aggregate was gradually depleted and separation increased within the bins. If a repair delay of more than 1 hour occurred, the supply of some size of aggregate was exhausted, usually the 3-inch size.

Another difficulty in the plant involved obtaining a suitable timing sequence for charging the various materials to each of the six mixers. A timing sequence that would be good for one or two mixers would not be satisfactory for the other mixers. This difficulty was related to the 35-foot distance of travel of materials between the weighing hoppers and the mixers, and the different angles of approach the swivel chute would have in relation to each weighing hopper and each mixer. This difference in travel time of materials for charging the mixer was improved by installation of two more timing devices. Best results were obtained, after considerable experimentation, by having three mixers on one sequence, two mixers on another, and one mixer on separate sequence.

An unusual problem was presented as the result of the lead time needed to allow a pause in production because of delays at the placing site. These delays frequently resulted when the cableway was used for yarding, leaving up to 136 cubic yards of concrete in storage between the placement and the mixers. This yardage was distributed as follows: 24 cubic yards in the mixers, 64 cubic yards in the three "gob" hoppers, and 48 cubic yards in the transfer cars. This naturally created a tendency for slump loss, especially during warmer weather, which further slowed down placing operations. This situation would have been somewhat improved by closer coordination of the placing and batching operations in providing lead times and deceleration of placements.

170. REFRIGERATION PLANT. A refrigeration plant was erected on the west side of the canyon on a bench above the mixing plant. The plant was assembled by the Lewis Refrigeration Co. of Seattle, Wash., and was nearly complete at the time of the strike, July 6, 1959. A cooling tower was erected on a bench above and just upstream from the refrigeration plant.

The primary purpose of the plant was to control concrete temperatures in three phases: precooling of components and mix, cooling during hydration, and postcooling the mass for contraction joint grouting. The refrigeration required totaled more than 4,000 tons with a power installation of 6,275 horsepower. Cost of the plant was approximately $2 million, with equipment alone valued at $1.25 million.

Construction specifications required that mass concrete be placed at a temperature between 40° F. and 50° F. Further cooling was accomplished through pipe coils embedded at the base of each 7.5-foot lift. A total of 4,873,408 cubic yards of concrete required cooling.

The contractor's system incorporated the following process stages:

(1) Cooling of coarse aggregate by chilled water.

(2) Air cooling of coarse aggregate in mixing plant bins.

(3) Chilling mixing water.

(4) Direct addition of ice to mix.

(5) Cooling placed concrete with water or brine through embedded pipes.

As used in sustained production, the first four items, involving reduction of the mix to below 50° F., required about 40 percent of the plant output. Twenty percent was used in initial cooling of the coarse aggregate prior to delivery to the batching plant.

Several features in the installation were of special interest. For example, refrigeration coils were installed beside each aggregate bin at the batching plant to chill the air that cooled the rock. At another location, one central bank of coils was installed with air ducts to the different bins. Temperature of the aggregates in the bins was controlled by thermostats to prevent freezing. In addition, the refrigeration coils were cleaned of frost and dirt accumulations by automatic water flushing systems operating on a timeclock.

Another interesting feature was the automatic operation of the ice batching system. Most ice systems have been semiautomatic, requiring a man in the ice storage bin to keep the ice friable and moving into screw conveyors. In this plant, however, the flake ice fell from the ice-making machine onto a fast-moving rubber belt which carried it some 50 feet to an 8-inch-diameter chute where it dropped 125 feet. It fell into a small storage bin immediately adjacent to the batching plant weigh hopper floor. This bin contained about a 30-minute supply of ice. To insure an adequate supply of ice at all times, 22 ice-making machines were installed. An automatic signal from the batching plant ice bin activated more or fewer of these ice-making machines as necessary to keep up with mixing operations.

The plant included a battery of 18 ammonia compressors most of which were arranged for multiple-purpose use. Four 125-horsepower compressors were used solely to provide refrigeration for the 22 ice-making machines which produced 20 tons of flake ice per hour. The other units were connected with appropriate controls so that they could be used effectively for varied load demands. These other units consisted of eight 600-, two 100-, one 450-, two 150- and one 25-horsepower compressors. The compressors were serviced by eight 48-inch condensers. Excess heat, peaking at more than 1,000,000 British thermal units per minute, was wasted to the air by a cooling tower installation. The tower recirculated water at 15,000 gallons per minute to the condensers.

There were 10 shell and tube chillers. Two of these, 32 inches in diameter, were used to chill the mixing water. Three, 52 inches in diameter, chilled water for cooling the coarse aggregate on the conveyor belts. The other five, 38 inches in diameter, cooled water for circulating through the cooling pipes embedded in the dam. These units were initially located at the base of the dam for cooling of the first 2 million cubic yards of concrete, but were moved to the right canyon rim for cooling of the remaining 2.9 million cubic yards.

The refrigeration plant was designed to meet a concrete placing rate of 480 cubic yards per hour. With a normal temperature of aggregates at 87.5° F., cement at 150° F., pozzolan at 120° F., and water at 80° F., the temperature of the resulting mix would be about 95° F. This is 45° F. higher for placing than the allowable 50° F.

To bring the concrete mix temperature to 50° F., the following four steps were used:

(1) Spray the aggregates. This step involved spraying the coarse aggregate (3/16 to 6 inches) on the conveyor belt as it moved from the stockpile area to the batching plant. This required 830 tons of refrigeration and involved the use of 2,200 gallons of water per minute at 35° F. The water was recovered for recirculation. Accumulated particles were settled out of the water before recirculation. The aggregate was passed over dewatering screens before proceeding to the batching plant. This process cooled the rock to 50° F. and would reduce the mix temperature to 72° F.

(2) Air cool the aggregate. The aggregate was stored at the batching plant in eight bins with a total capacity of 3,000 tons. Four of these bins were equipped for air cooling. These were the 3- to 6-inch, the 1-1/2- to 3-inch, and the two bins holding 3/4- to 1-1/2-inch aggregates. One of these latter bins held aggregate from which lightweight rock had been removed by the heavy-media process. No attempt was made to air cool aggregates smaller than the three-fourths inch.

Air cooling of the larger aggregates required 270 tons of refrigeration and reduced the rock temperature to 30° F. This action would further reduce the mix temperature to 64° F. As mentioned, each of the bins had its own individual air-cooling coils which were automatically defrosted and cleaned. Air circulation was on a closed circuit basis with air moved by blowers.

(3) Chill the mix water. In this step, the mix water was chilled from 80° to 35° F. This required

180 tons of refrigeration; the chilled water, together with the aggregate cooling, would reduce the temperature of the resulting mix to 59° F.

The chilled water, including the water in the surge tank at the batching plant, was continuously circulated. The chilled water was also used by the ice plant.

(4) Add flake ice to the mix. Flaked ice was fed direct to the surge tank at the batching plant without intermediate storage. This tank had capacity for 30 batches. Automatic operation started and stopped ice machines as necessary to keep the surge tank filled. The ice was constantly agitated in the tank to provide ready delivery to the weigh hopper. The 22 ice machines could provide about 20 tons of ice per hour which required 400 tons of refrigeration effort. By adding 300 pounds of ice per 4-cubic-yard batch, the mix was finally cooled to about 47° F. This is about 3° below the specifications requirement, but this margin was needed to insure that the final placing temperature at the forms was 50° F. or less.

Cooling of placed concrete was accomplished in two stages and required 1,600 tons of refrigeration effort. The first phase was for removing the heat of hydration and used water at 38° to 47° F. circulating through 1-inch aluminum tubing embedded at the base of each 7.5-foot lift. Specifications required that water be cooler than the concrete being placed. Circulation of this water was required for 12 days. The purpose of the first phase of the cooling was primarily to control potential cracking in the concrete. Secondary cooling was used to reduce the temperature of all concrete in the dam below elevation 3450 to 40° F. The required temperatures from 3450 to the crest elevation of 3715 varied from 40° to 50° F. Time for secondary cooling was about 52 days for the 7.5-foot lifts used. To get the dam concrete to 40° F., chilled water at 38° F. was used initially and was switched to brine at 35° to 28° F. in the final stages. Secondary cooling was used to open up the contraction joints which permitted effective grouting to make a monolithic structure of the dam. Instruments embedded in the dam provided temperature controls during construction and will provide structural behavior data under reservoir loading conditions.

171. CONCRETE CONTROL OPERATIONS. To perform the assigned duties of the branch, a complement of 40 engineers, technicians, and inspectors of various grades and experience were employed under the direction of the branch chief. A supervisory engineer was assigned to correlate and direct inspection and testing on the site for the three shifts of concrete production.

The contractor's operations at the Wahweap aggregate washing and screening plant were inspected on all shifts of operation. On day shift, two inspectors were scheduled and one each on the other shifts. Special tests, as well as routine tests were performed on the day shift, such as aggregate production tests. Although the specification requirements limited the acceptance of materials at the batching plant as batched, it was found more practical to perform routine tests, such as sand gradation, quantity of light-weight materials in heavy-media processed aggregate, and soluble sulphate content in the fine aggregate as produced at the production site. These same tests were performed, but not as frequently, on all the material as it was batched to assure that these materials had not been contaminated or mishandled during transit to the batching bins. This procedure proved satisfactory, and when materials did not meet specifications during production, it was more practical and economical to reject material at the source of production rather than waste large quantities in the various stockpiles. Gradation tests on coarse aggregate were not performed at the aggregate screening and washing plant because coarse aggregate was later screened at the concrete batching plant.

Owing to the nature of the concrete production facilities and the intended use, adequate quality control was necessary for each step in the manufacture of the component materials, the production of the concrete mixture, and the placement. Adequate inspection and production testing were therefore provided to insure acceptable concrete in accordance with the specifications. Being a key feature in the whole operation, a substantial part of the concrete control effort was expended at the batching and mixing plant.

The inspection force for each shift of operation consisted of an engineer and shift chief, who was assisted by four inspectors. On the day shift tour of duty, a supervisory engineer was assigned to the plant to coordinate the work and test procedures on all shifts, calibrate and maintain all testing equipment used at the plant, make mix adjustments for gradation, supervise the adjustments necessary for mixer charging sequences to obtain mixing efficiency, and supervise the checking of all batch weighing scales.

Duties of the plant inspectors for an average shift consisted of selecting and setting up the proper concrete mixes for all authorized concrete placements, performing tests and insuring conformance with the specifications. This involved observing and recording the scale weighing cycle for accuracy; observing the mixing action of each mixer visually and also checking the recorded record of concrete consistency for each batch; obtaining test batches of concrete on each major mix used on the shift; obtaining a breakdown from the test batches to show aggregate moisture and gradation of aggregate, slump, temperatures, entrained air content, unit weight; and casting at least four 6- by 12-inch test cylinders for later testing of compressive strength. During the shift, other routine tests were made to verify slump, temperatures, entrained air content, and unit weight. Two mixer efficiency tests were made on each shift per week, which constituted an efficiency test on each mixer for approximately every 6,500 cubic yards of concrete mixed.

The tests performed at the central laboratory required the use of more precise equipment and temperature and humidity controls. Among these tests were the physical properties, acceptance tests on pozzolan, the storage and testing of all test cylinders for compressive strengths (see fig. 259 for typical test results), the soluble sulphate tests on sands, and all other special tests.

The government furnished all cement and pozzolan to the contractor at his unloading facilities. This required that all cement and pozzolan be acceptance tested and weighed before delivery to the contractor. No unusual difficulties were experienced with the delivery of cement. During the winter months, heavy snowfalls near Flagstaff, Ariz., caused hazardous road conditions and shipments were delayed on several occasions, but the project supply was never depleted.

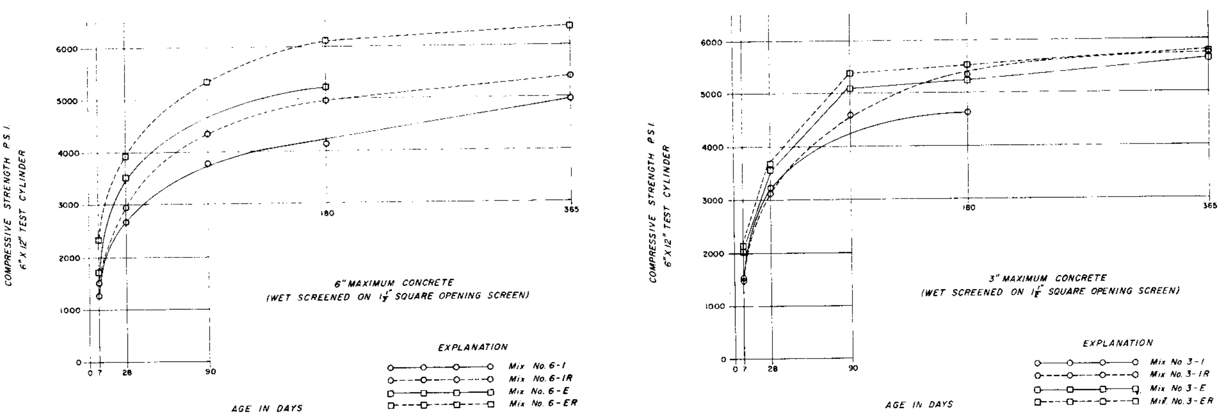


Figure 259.—Compressive strength of concrete test cylinders for dam.

For the performance of all tests at the mixing plant, two field laboratories were located within the plant, one on the batching floor and one on the mixer deck. All operation records and test data were compiled on proper report forms and turned in at the end of the shift to the concrete control branch office.

The central laboratory and control office was located in a separate building in Page, Ariz., and was the center of coordination between all concrete activities in the field and the main project office. Headed by a supervisory engineering technician and assisted by approximately nine engineers and technicians, their work alternated between the office and the laboratory special tests. At times they served as supervisory inspectors on construction contracts, such as on the transmission lines, the completion contract and other smaller contracts, and served as routine relief inspectors in the field.

Although even greater problems in supply were experienced with pozzolan, the project supply of this material was also never completely depleted.

The Government scale house was located on the west side of the canyon between the highway and the storage silos and operated with one technician on each shift. Aside from weighing cement and pozzolan, the technician obtained samples and performed site acceptance tests on both products. Acceptance tests on cement included one for temperature, which was not to be above 180° F. in the cement as delivered; and a false set test, which was not to exceed 17 millimeters between the initial and final readings. All other acceptance tests on cement were performed by the National Bureau of Standards at the cement plant. The project tests were performed on about every tenth truckload of cement (or 1,400 barrels) delivered under normal conditions. No difficulty was experienced with temperature. During the summer of 1962, some difficulty was experienced with false set in cement. For a short time thereafter, all truckloads of cement were tested before unloading and several truckloads were rejected. The cause of the condition was apparently corrected promptly, as no further difficulties were experienced with false set.

A temperature acceptance test on pozzolan was performed at the scale house. The upper allowable temperature limit for pozzolan was 170° F. and a 15-pound sample of pozzolan was obtained from each 150 tons, or every sixth truckload delivered. At least two truckloads were sampled each day for determining the moisture content of pozzolan. Truckloads were not accepted before the Blaine test had been completed, which generally consumed approximately 45 minutes of time. When the test failed on a truckload, tests were taken on each subsequent truckload until 10 consecutive truckloads met the specifications. On different occasions, several consecutive truckloads of pozzolan were rejected.

The delivery of pozzolan of acceptable quality and in sufficient quantities at the peak of construction caused some concern. At one time it was necessary to ship in several carloads of fly ash from the Chicago area, and on two other occasions pozzolan was trucked 250 miles from Panaca, Nev., in order to keep the prime contractor in supply. Although the project supply was never depleted, it was down to less than a 1-day supply on several occasions, and part of the pozzolan in the concrete was replaced by cement.

The concrete control office compiled all concrete data from the field and prepared its part of the daily and weekly progress reports and the monthly L-29 report. The daily, weekly, and monthly compilations of these data were valuable in analyzing the progress of the work, and aided the branch chief in making any necessary changes in mix design, test procedures, or the requirement for additional tests. Data were also used for the purpose of ordering cement and pozzolan for project needs in a regular and orderly manner on a weekly and monthly basis.

172. CEMENTING MATERIALS. The Government-furnished type II, low-alkali cement for the construction of Glen Canyon Dam and Powerplant was supplied under invitation No. DS-5023. This contract was awarded to the American Cement Corp. of Los Angeles, Calif., on April 3, 1958, on their low bid of $9,741,900 for the estimated quantity of 3 million barrels of cement. The Phoenix Cement Co., a subsidiary of the American Cement Corp., proceeded to construct a mill at Clarkdale, Ariz., which was completed and in operation late in 1959. The Clarkdale area was chosen because there were large limestone deposits in the immediate area, other necessary materials for making cement were readily available, and there were no other mills in northern Arizona.

Two types of limestone, Redwall limestone and Lakebed limestone, are found near Clarkdale, each being created in a different geologic age. The Redwall limestone is a high-quality white limestone laid down in Pennsylvania times about 100 million years ago. This limestone is found in a fractured state due to the movement of the earth since it was deposited and the pieces are coated with Supai sandstone, making them red in color, which was also deposited in the same era. The Lakebed limestone, also a high-quality white limestone, was laid down by a fresh water lake 750,000 years ago or in the Pleistocene age. The argillaceous materials needed for cement came from slag from the old Clemenceau smelters, located only 4 miles away from the mill, and from other nearby quarries. Gypsum was obtained from the Verde Gypsum Co., mined near Camp Verde about 20 miles from the Clarkdale mill.

Utilizing the dry-process manufacturing method, the Clarkdale plant was reportedly capable of producing 5,500 barrels of cement daily. It was estimated that the maximum monthly quantity of cement required at Glen Canyon would not exceed 120,000 barrels. The contractor was able to meet all cement requirements during the life of the contract. Inspection of the manufacturing process and plant testing of the cement was made at the plant by a resident inspector from the National Bureau of Standards. Maximum temperature and false set tests were performed by the Bureau's concrete control section at the construction site.

Phoenix Cement Co. issued a subcontract to Belyea Trucking Co. of Los Angeles for transporting the cement from the mill to Glen Canyon, a one-way distance of 188 miles. Twenty truck-tractors pulling two hopper-type trailers, specially designed to haul the maximum load allowed under Arizona State laws, were used to make the haul. Each tractor-trailer combination had an overall length of 59 feet and 11-1/2 inches and was capable of hauling 142 barrels or 27-3/4 tons of bulk cement.

Had the Clarkdale mill not been constructed, all of the cement would have had to have been hauled from southern Arizona or southern California which would have substantially increased the cost of the cement to the Government. A total of 3,046,441 barrels of cement were purchased for use by the prime contractor and 40,043 barrels for use by the completion contractor at a net cost to the Government of $10,649,706.

173. POZZOLAN. J. G. Shotwell of Mercer Island, Wash., was awarded the contract under invitation No. DS-5023 to supply pozzolan for use in the construction of Glen Canyon Dam and Powerplant.

The pozzolan plant is located about 30 miles north of Flagstaff, Ariz., and was constructed in 1959. Raw material was taken from an area known as the Bonner deposit which was located on both sides of U.S. Highway No. 89 and became the plant site. This natural ash deposit is located just north of the San Francisco Peaks in an area of relatively recent volcanic activity. When this deposit became nearly depleted in August 1962, another nearby source, referred to as the Sugar Loaf deposit and located about 12 miles from the plant, was investigated. Test samples from this deposit met the specifications requirements and the first pozzolan delivery was made during September 1962. The Bonner Deposit was finally exhausted in October 1962.

The initial excavation in the pit was done by a scraper which resulted in excessive contamination of the material from the overburden. As this condition caused lower compressive strength in the concrete using the pozzolan, it was necessary for the contractor to modify his stripping operations. This was accomplished by using a dragline for pit excavation, rather than the scraper.

A ring mill and a hammer mill were originally used for crushing the minus 3/8-inch material. This plant was not capable of producing material of sufficient fineness in the necessary quantity to satisfy the requirements of Glen Canyon. Major renovation of the plant took place throughout 1960. Four large ball mills and a hammer mill were erected followed by another large ball mill and hammer mill added in 1961. A 600-ton storage tank was constructed to store the pozzolan at the plant site. This amount of storage proved to be inadequate, and there were times when the temperature of the pozzolan was too high as it was delivered at the construction site.

The first delivery of pozzolan was made to the damsite on April 6, 1961. Deliveries of pozzolan during 1961 and 1962 were generally below the quantity required. Additional pozzolan was shipped to the project from a plant in Panaca, Nev., and approximately 662 tons of fly ash was shipped from Chicago. At times the insufficient rate of delivery reduced the amount of pozzolan in the design mix from 94 pounds per cubic yard of concrete to 80 pounds, and at times as low as 70 to 75 pounds per cubic yard. Although pozzolan production and delivery was low, the concrete placing operations were never suspended due to the lack of pozzolan.

A total of 204,367 tons of pozzolan was used by the prime contractor and completion contractor at a net cost to the Government of $2,510,235.

174. CABLEWAYS. Two 50-ton-capacity mobile cableways (fig. 260) were erected for use in placing concrete in the dam and one of these was in operation early in February 1959. The cableways were equipped with traveling head and tail towers, and the load-carrying cables were 4-inch lock coils, the largest ever spun. The high towers of the higher cableway enable it to cross over the lower cableway so that each cableway can be operated independently of the other. A 35-ton load test was satisfactorily made on the higher cableway on February 11, 1959. The lower No. 2 cableway was placed in operation early in March 1960. The 4-inch cables required replacement every 9 or 10 months; the armor coating on the cable would develop separations in individual strands and ravel to such an extent that replacement was necessary. The 4-inch cable for cableway No. 2 was replaced in March and August of 1961 and May of 1962. The cable on cableway No. 1 was replaced in June 1961, March 1962, and January 1963. A 25-ton-capacity mobile cableway was erected and placed in operation late in 1960.

The cableway system was designed for the contractor by Lidgerwood Industries, and was the largest ever used in dam construction.

Two of the cableways had a rated capacity of 50 tons with a combined capacity of 100 tons when teamed together by means of an equalizer beam. The size and capacity of the 50-ton cableways were determined by the necessity of handling a 12-cubic-yard bottom-dump concrete bucket that weighed approximately 30 tons when filled with concrete. Two cableways, capable of carrying 12-cubic-yard buckets were needed to handle the volume of concrete to be placed in each 24-hour period. This volume was set by the contractor's concrete placement schedule. The 12-cubic-yard bucket was selected over the usual 8-cubic-yard bucket because of the relatively long time required for one cableway to make a complete cycle from the concrete transfer car trestle to the placement area in the dam. At the start of concrete placements in the dam, the

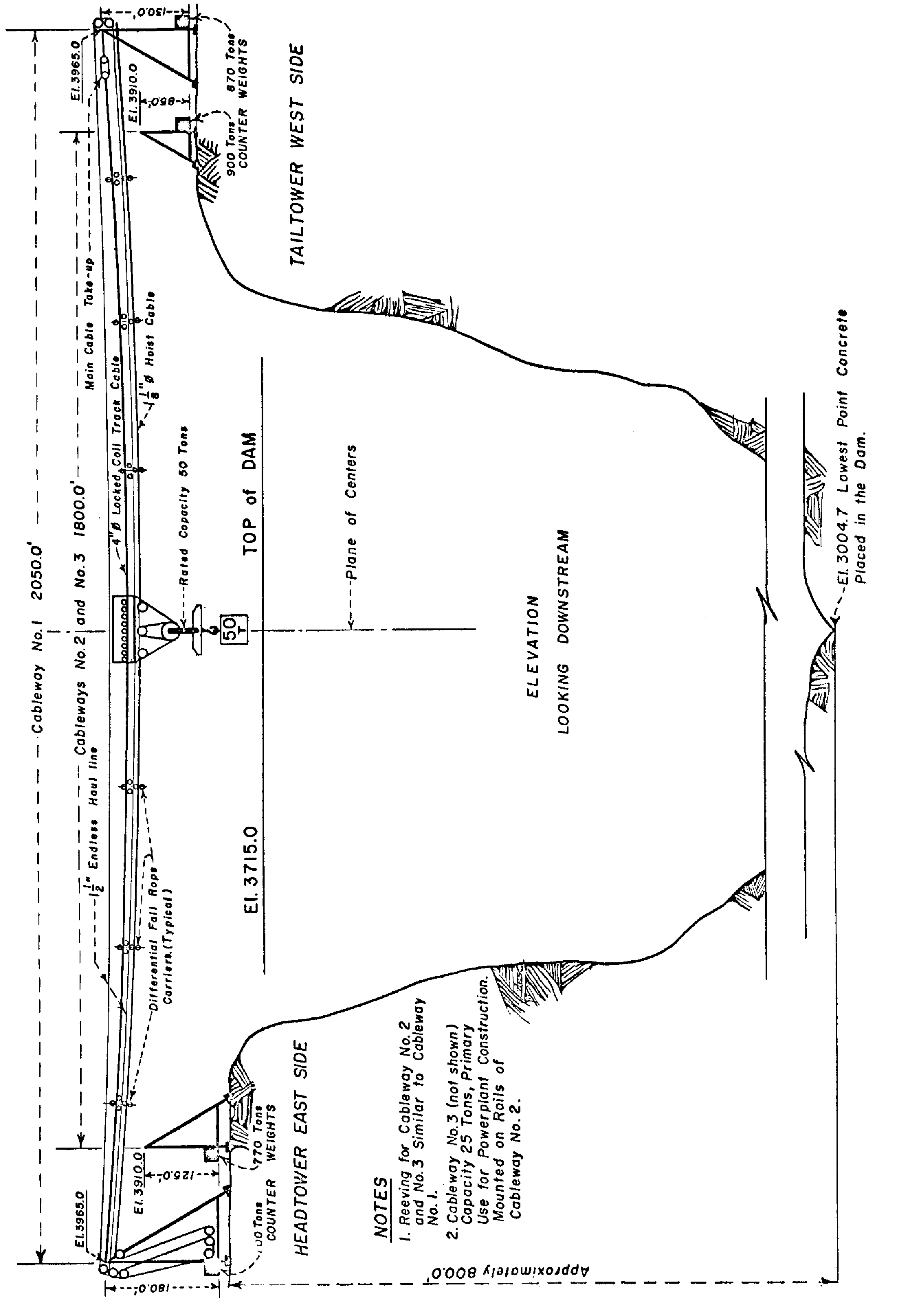


Figure 260.—Elevations of cableways.

bucket cycle time was about 4 minutes, but this decreased steadily to under 2 minutes as the dam rose to the elevation of the trestle. These two cableways were designed and constructed with a normal capacity of 31 tons each and up to 50 tons each under reduced speed and greater main cable sag.

The higher of the two 50-ton cableways had a span of 2,050 feet between the head tower and tail tower. The head tower was 180 feet high, the tail tower 130 feet high, with a lateral movement of 910 feet for each tower. The lower 50-ton cableway had a span of 1,800 feet between towers. The head tower was 125 feet high and the tail tower 85 feet high, with a lateral movement of 810 feet for both towers. The difference between head and tail tower heights is due to the difference in elevation of the tower rails, the rails on the west side being higher. The towers traveled on tracks of 175-pound rail.

To place tracks for the tail towers where the cableways would be the most advantageous, the contractor had to excavate a large portion of an existing rock landmark. This rock of solid sandstone, called the Beehive because of its shape, stands approximately 250 feet above the canyon rim on the west side. The excavation from the east face of the Beehive amounted to approximately 135,000 cubic yards.

The third cableway was added to take care of yarding and concrete placement in the powerplant so that the other two could be used for concrete placement in the dam without interruption. The track of the third cableway was later lengthened upstream, and it was also used on the dam. This cableway, with a track cable of 3-inch diameter and a rated capacity of 25 tons, was used primarily for handling materials and for carrying an 8-cubic-yard bottom-dump concrete bucket. This cableway moved on one of the track rails of the lower 50-ton cableway and one other rail. The use of the third cableway was discontinued in February of 1963 when it was disassembled and shipped to another project.

Except for local plate stiffening for concentrated loads, all members of the tower structures were fabricated from wide-flange beams. Plate stiffening was used at the head of each tower to accommodate the pull of the main cable. Welds and rivets were used in the shop for subassemblies, with high-tensile bolts used for field assembly. All structural steel for the towers for the two 50-ton cableways was furnished by Judson Pacific-Murphy Corp., Emeryville, Calif. (fig. 261).

Each tower was mounted on 4 identical 8-wheeled trucks, 1 under each tower leg, or 32 wheels in all. Each of the trucks was electrically driven by a 40-horsepower, alternating-current wound roto-gear motor with direct gearing to the axles. The trucks were furnished by Consolidated Western Steel.

Maximum travel speed of the towers was 120 feet per minute. To equalize the overturning moment of the towers, reinforced concrete counterweights, weighing up to 900 tons for the tail tower of the lower cableway, were placed just above the eight-wheeled trucks across the vertical legs of each tower.

The main cable or track cable for each 50-ton cableway weighed approximately 38 pounds per foot and was manufactured by American Steel and Wire Co., Division of U.S. Steel Corp. The tail towers were equipped with take-up winches for adjusting the sag of the track cable for either a 31- or 50-ton hook load. A 50-ton-capacity carriage assembly, mounted on two 10-wheel units, rode the track cable of each cableway and was pulled by a 1-1/2-inch haul line. The haul hoist consisted of two sheaves in tandem, one an idler. These sheaves handled the 1-1/2-inch wire rope which served as an endless conveyorline. The haul hoist system was driven by a single-horsepower direct-current drive motor. The carriage velocity was 1,200 feet per minute with a 31-ton hook load, and 600 feet per minute with a hook load of 50 tons.

The fallblock had two sheaves with four parts of a 1-1/8-inch hoist line running to an equalizer beam attached to the carriage assembly. This 1-1/8-inch wire rope was dead-ended at the tail tower and reeled on a single drum, grooved for 1-1/8-inch wire rope, in the head tower. The hoist drum was 106 inches in diameter with a face width of 96 inches. It could wind 2,180 feet of 1-1/8 inch wire rope on the first layer. The drum was operated by two 500-horsepower, direct-current drive motors. The vertical velocity of the fallblock with a hook load of 31 tons was 600 feet per minute, and 300 feet per minute with a hook load of 50 tons. Independent motors and gearing allowed combinations of horizontal, vertical, and lateral movements of the fallblock.

The three direct-current drive motors that operated the haul line sheaves and hoist drum of each 50-ton cableway had identical ratings. This made standardization of spare parts possible. The motors were 500-horsepower, 900-r.p.m., 480-volt units. Power for the direct-current drive motors was derived

Figure 261.—A 44-ton section of the crane for the powerplant is hoisted into the air by both of the 50-ton-capacity cableways. Both cableways were used because of the awkward length of the section. P557-420-6678, February 4, 1962.

from three generators mounted on a common motor-generator with a 1,250-horsepower alternating-current drive motor.

Housing for the mechanical and electrical equipment was located on the lower platform of each head tower, directly in front of the counterweight. The electrical equipment was furnished by General Electric Co. and the mechanical equipment by Lidge.wood Industries. Figure 262 is an interior view of the hoist room in the head tower of the mobile cableway.

The 1-1/2-inch haul line and the 1-1/8-inch hoist line were suspended from the track cable by six differential carriers. These carriers, three on each side of the carriage assembly, were used as moving hangers to prevent sagging of the lines. Use of the differential carriers limited the maximum unsupported length of the haul and hoist line to about one quarter of the cableway span. A platform on each side of the track cable was cantilevered from each tower to make possible maintenance on the carriage, fallblock, differential carriers, and take-up winches.

A monorail system was used to carry power and control wiring to the equipment in the head towers with a minimum of cable maintenance. Special flexible electrical cables were mounted on steel hangers and were sagged on the monorail parallel to each set of head tower tracks. Electrical power for the main hoist equipment was supplied through this monorail system at 4,160 volts. The power for propulsion motors was reduced to 440 volts.

Undesirable stress was introduced into the main track cable if the cableway towers were not kept directly opposite each other when in operation. Therefore, a unique application of closed circuit television was devised to keep the towers of the two 50-ton cableways directly opposite each other. In a housing on each tail tower, a television camera was mounted with a 20-inch telephoto lens aimed at a target on the head tower across the canyon. After some experimentation, the best target was found to be a 16-foot-long fluorescent tube which could be seen in any kind of weather. If the towers began to drift out of direct opposition, the divergence showed up on a receiver in the operator's control booth and could be corrected immediately. This receiver had a vertical reference line on the screen, and the operator had only to keep the image of the fluorescent tube matched to the reference line. The television system was originally installed to supplement a somewhat unreliable automatic electrical system. It worked so well that it was later considered to be a primary control. This closed circuit television system utilized transistorized cameras. The towers were kept directly opposite each other by the operator's use of a simple sighting device.

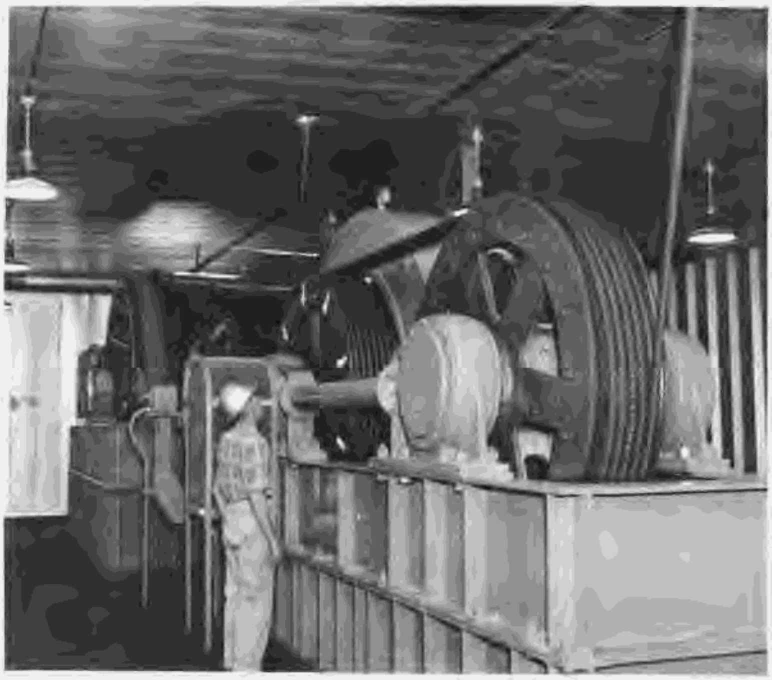


Figure 262.—Interior view of the hoist room in the huge head tower of the mobile cableway. P557-420-4818, May 1960.

Operation of each cableway was handled by one man. The operators for the two 50-ton cableways were located in a double-deck control booth cantilevered from the west canyon wall, level with the concrete transfer trestle tracks and within sight of the landing platform where concrete was transferred to the cableways. This location allowed the operator an unlimited view of the entire work area. On each operator's control panel were a series of levers, buttons, and switches that permitted handling of materials and the concrete buckets with ease. Since the distance from the operator's booth to the concrete placement in the forms was so great, a "bellboy" or spotter was used at the point of placement. The operator used a double system of two-way radio to confer with the spotter.

The operator's booth for the 25-ton cableway was located in the head tower on the east rim of the canyon. When this cableway was used for concrete placing, a spotter was needed on the concrete transfer trestle platform as well as at the point of placement. The spotter at the point of placement kept in touch with the cableway operator by carrying two lightweight radio transmitters and one transistorized receiver on a special belt. Total weight was only 6-1/2 pounds and the use of the radio gave him complete freedom and mobility and eliminated trailing wires.

In one hand, the spotter carried a microphone for talking directly to the cableway operator, and in the other hand he carried a signaling device, job-made from two hacksaw blades, by which he could cause 1,000-cycle "beeps" to sound in the operator's booth. When the concrete bucket was near the forms, both communication systems were used at the same time to speed up the exchange of information. The spotter also wore a small earphone through which he not only heard the operator's voice but his own voice and the beep signals sounding in the operator's booth. He, therefore, knew at all times whether both of his communication systems were working. Either system alone was sufficient to conduct this operation while the other system was being repaired. A red light would come on in the operator's room if anything went wrong with the base station transmitter. Difficulties with the system were negligible, due partly to the thorough servicing program carried out. All sets were taken to the shop for servicing at the end of each shift. This required two complete sets of equipment, but practically eliminated breakdowns. The radio communication system and the closed circuit television were designed and installed by Carl Hutton, owner of Kaibab Radio, Page, Ariz.

Preventive maintenance servicing of the cableways and inspection of the cables were made each week. A second set of controls was located in each head tower to permit routine maintenance operations. Inspection of the cableways was very important, not only from a safety standpoint but also to keep a close check on the condition of the track cable. The total concrete yardage handled by each cableway is the only means of evaluating the life of the track cable. In the first year and a half of placing concrete, the track cable for each of the 50-ton cableways had to be replaced twice. The life of the first track cable was approximately 400,000 cubic yards of concrete but the other 4-inch track cable handled only 300,000 cubic yards. The primary source of trouble was the armor facing of the cable which broke at numerous locations and eventually became entangled with the wheels of the carriage assembly. When this happened, the carriage assembly had to be freed by cutting the armor facing loose with a cutting torch and a new track cable installed.

175. MASS CONCRETE IN THE DAM. The first mass concrete was placed in block 10-B of the dam from elevation 3005 to elevation 3015 on June 17, 1960, when Fred A. Seaton, then Secretary of the Interior, pulled the lanyard to dump the first 12-cubic-yard bucket. The ceremony was attended by the Assistant Commissioner and Chief Engineer of the Bureau of Reclamation and some members of his staff, the Regional Director, Governors of Arizona, Colorado, and Utah and many other dignitaries. Several thousand interested spectators watched the ceremony from the Glen Canyon Bridge and the vista points. Figure 263 shows early dam concrete placement.

Following the ceremony, concrete placing operations continued and block 8-B was placed from rock to elevation 3052.5. The lowest point of the dam was at the downstream left side of block 11-B at elevation 3004.7. Concrete placing was slow for several months while the batching plant was undergoing considerable adjustments and placing crews were being trained for the work. On the weekend of August 20, 1960, the plant was shut down for installation of replacement equipment. Work in the bottom was also slow as the placing crews were new at their jobs and were not too well organized. With continued adjustments, production gradually increased and for the week ending December 23, 1960, total concrete placed amounted to 34,300 cubic yards. The highest weekly production was 49,581 cubic yards for the week ending November 17, 1961. In 1960, a total of 356,588 cubic yards of concrete was placed in the dam. In 1961, 1,800,759 cubic yards were placed; 2,050,658 cubic yards were placed in 1962; and 686,378 cubic yards in 1963. Six-thousand five-hundred and thirty-two cubic yards were placed in 1964 for a total of 4,900,915 cubic yards. Mass concrete alone in the dam amounted to 4,873,700 cubic yards. Figure 264 shows concrete placing operations using ladle cars and 12-cubic-yard bucket in 1960.

The dam was built in blocks formed by a system of transverse and longitudinal joints. The maximum size of a block was approximately 60 feet wide by 210 feet long. Initial placement was made with wood forms; but when the blocks were high enough for a full placement of 7-1/2 feet on each side, and with enough space at the bottom to attach the forms, steel forms were used.

Cleanup of bedrock was by water jets. Debris consisting of mud, small rock, trash, etc., was loaded into skips and removed. Cleanup of the construction joints before placing the next lift in any block was made by water jets and sandblasting. No. 4 to No. 8 beneficiated sand fraction was used for sandblast sand. Grout was placed by the concrete bucket and broomed over the surface before placing the first block on the foundation and on the construction joint before placing the next lift.

In accordance with paragraph 117 of the specifications, the contractor had the option of placing the dam blocks in 5- or 7-1/2-foot lifts and he elected to use 7-1/2-foot lifts. Concrete was placed in layers not to exceed 20 inches and was consolidated by immersion-type pneumatic vibrators. The contractor decided to place the downstream blocks before the upstream blocks. The maximum differential between adjacent blocks was 30 feet and the highest block in the dam could not be more than 52.5 feet above the lowest block. This was later extended at the contractor's request for construction reasons to 37.5 feet and 60 feet, respectively. These differentials were waived and no height differentials existed after odd-numbered low blocks 7 through 17 reached elevation 3375.0 where the first longitudinal joints were terminated. No concrete could be placed directly above the termination of a longitudinal joint until all concrete in the upstream and downstream blocks separated by such longitudinal joint had been cooled to 50° F. Twelve-inch half-round pipe was placed at the

Figure 263.—View looking down on the damsite from the canyon rim. Powerhouse is at left, dam at right, with penstock foundations between them at center. P557-420-5038, July 7, 1960.

termination of longitudinal joints in all blocks. The rate of placement in any one block was limited to not more than one lift in 72 hours.

At the end of 1960, A and B blocks had been placed from block 6 through block 18 with blocks 5-B, 19-B, and 20-B also placed. Blocks 8-B and 16-B were the highest blocks at elevation 3135, and blocks 9-A, 11-A, and 13-A were lowest at elevation 3090.0. During December 1960, temperatures fell below the freezing point most nights and remained below 40° F. on many days but did not hamper concrete placements significantly. Two snowstorms brought enough moisture to make dewatering operations necessary on placements.

After two consecutive days with mean temperatures below 40° F., concrete was maintained at a temperature of about 50° F. for the required 72 hours by means of fire barrels placed above and around the newly placed concrete. Scrap lumber was generally used as fuel, but at the end of December a few oil burning salamanders were used. Late in December, the contractor inaugurated a program of spraying the steel forms with an asphaltic base material containing a high percentage of ground cork. Total concrete placed in the dam in 1960 amounted to 356,588 cubic yards. Figure 265 shows the downstream face of the dam as of May 19, 1961.

At the end of 1961, A and B blocks had been placed from blocks 3 through 22, with block 23-B also placed. Block 16-B was the highest block at elevation 3352.5 and block 3-A was lowest at elevation 3292.5. A total of 1,800,759 cubic yards of concrete was placed during this year.

Figure 264.—Transfer ladle car dumping its load of concrete into a 12-cubic-yard bucket high over the Glen Canyon damsite. Note abutment of Glen Canyon Bridge in background. P557-420-5057, July 11, 1960.

Concrete placing was shut down on December 11, 1961, because of subfreezing temperatures and was not resumed until December 18. Minimum temperatures in the canyon bottom on December 12, 13, and 14 were 12°, 11°, and 12° F., respectively. From December 18 through 31, temperatures ranged from 18° to 28° F. and protective measures were necessary.

Two 60-horsepower boilers were used at the batching plant and the mixing water was heated to temperatures ranging from 70° to 95° F., depending on the temperature of the aggregate. The blocks were warmed with fire pots and other type heaters to receive the concrete, and each placement was covered with one-half inch mats of closed-cell butyl plastic. The mats maintained the surface temperature of the blocks within the required range. The steel forms were coated with insulating material as was done the previous winter. One percent of calcium chloride, by weight of cement, was used in the concrete mix from December 11, 1961, through February 7, 1962.

Early in June 1961, a change in the top echelon of the contractor's forces was made. The new project manager ,aid off slightly more than 500 employees and scheduled concrete placing for only the swing and graveyard shifts, 5 days per week, except that placements not completed on graveyard shift were completed on the day shift. The main work on the day shift then consisted of freighting reinforcing steel and other materials and equipment with the highlines, for setting and general maintenance. Heretofore, forms had been raised and lowered by jacking, which required quite a number of laborers and was rather slow. Several mobile cranes were acquired for use in raising and placing forms, and after the crews had been trained, some time was saved and the operation was less expensive.

Decreased concrete placing because of the changed schedule was reflected in July 1961, when only 135,800 cubic yards were placed in the dam. However, concrete placing was gradually accelerated, and in October concrete placed in the dam amounted to 177,774 cubic yards. For the first 6 months of that year, concrete placement in the dam amounted to 880,770 cubic yards, and for the last 6 months the total was 919,989 cubic yards. The one-millionth cubic yard of concrete was placed in the dam on May 8, 1961, and the two-millionth cubic yard was placed on November 17 of that year.

At the end of 1962, concrete had been placed in all 26 blocks. Blocks 2 and 4 were the highest at elevation 3622.5 and block 26 was lowest at elevation 3487.5. Only four B blocks remained in blocks 22 through 25, as the other B blocks had been topped out at various elevations beginning with odd blocks 7 through 17 at elevation 3375.0. In December of 1962 removal of the downstream portion of the concrete transfer trestle began so that concrete could be placed in blocks 24, 25, and 26, and in March 1963 the section of the trestle was removed from blocks 22 and 23.

The use of insulating protective mats on the exposed surface of newly placed block lifts was started December 26, 1962, because of the cold weather. These were the same mats used during the 1961-1962 winter concrete placing season and were used through most of February 1963 except for a short period of warm weather. During cold weather concrete placing, 1 percent of calcium chloride by weight of cement was used in the concrete mix.

Concrete placing early in 1963 proceeded at about the same rate, or only slightly less than previously,

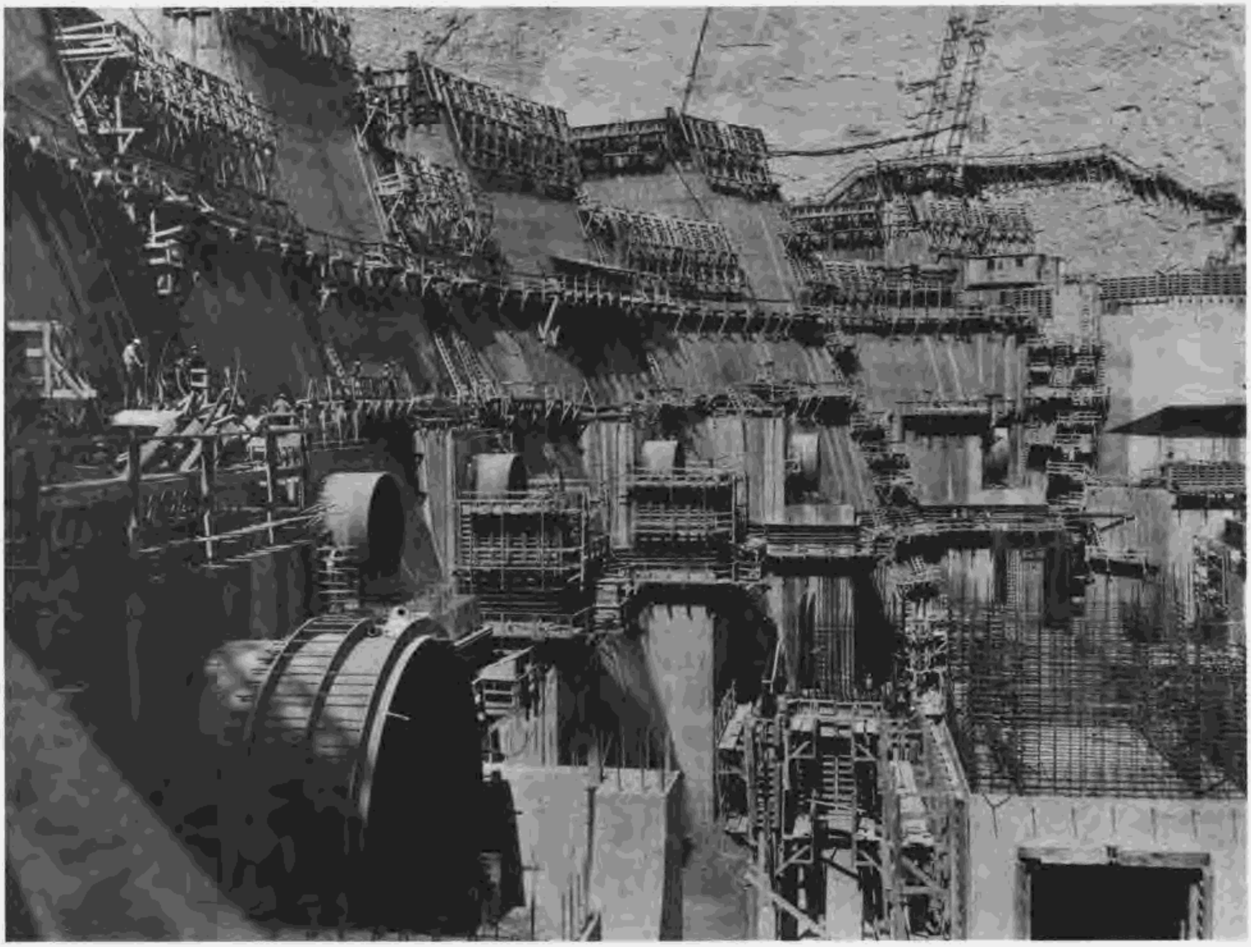


Figure 265.—View looking east along downstream face of dam, showing penstock pipes protruding from face of dam. Penstock support is at right foreground. P557-420-6001, May 19, 1961.

until a number of blocks got close to their ultimate height. Block 2 was topped out at elevation 3715.0 on March 30, 1963. Blocks 4 and 6 followed on April 10 and 19, respectively. Beginning with block 20 on May 8, other blocks were topped out at about 7-day intervals. The last two lifts from elevation 3705.0 to elevation 3710.0 and from elevation 3710.0 to elevation 3715.0 were 5-foot lifts. Block 25, the last block to be completed, was topped out at elevation 3715.0 on September 13, 1963. Placement of mass concrete in the dam took slightly more than 3 years, with the first placement being made on June 17, 1960, and the last placement on September 13, 1963.

Figure 266 shows the upstream face of the dam as of July 25, 1963.

2. Structural Behavior Installations

176. REFERENCE. Chapter IV presents a discussion on the structural behavior testing apparatus that is used in the dam. These are briefly described below.

177. EMBEDDED INSTRUMENTS. The embedded instruments include resistance thermometers for measuring the temperature of the concrete in the dam, strain meters for measuring the conditions of strain in the concrete, stress meters for measuring the stress in the concrete, joint meters for measuring the opening of the contraction joints, and deformation meters for measuring the deformation of the foundation and abutment rock. These instruments

Figure 266.—Upstream face of Glen Canyon Dam during construction as seen from the footbridge. P557-420-8625, July 25, 1963.

were installed in the dam as shown on the drawings. All of the meters were furnished by the Government and installed by the prime contractor. These instruments are of the elastic-wire type with the exception of the resistance-wire type thermometers. Embedded in the mass concrete of the dam, the instruments are connected by electrical cables to terminal boards located in the galleries of the dam where systematic readings are made with portable wheatstone bridges.

The several systems of 1,658 embedded instruments include 1,142 strain meters, 60 stress meters, 74 resistance thermometers, 264 joint meters, and 112 deformation meters.

All instruments are embedded in the mass concrete of the dam and are connected to convenient terminal boards by three or four conductor electrical cables. The cables are rubber jacketed, rubber insulated, No. 16 A.W.G., stranded, and of matched resistance. The terminal boards and outlet boxes are located in the galleries of the dam where readings are made using specially built, portable, wheatstone-bridge test sets. A total of 74 terminal boards were used; eleven 10-terminal boards, four 12-terminal, seventeen 20-terminal, twenty-two 25-terminal, and twenty 30-terminal boards. In addition, 116 outlet boxes were used for terminating meter cables in the dam. Two types of test sets are used, a wheatstone-bridge test set and a strain meter test set, which is a specially built wheatstone bridge. Padded carrying cases are used to protect the test sets.

Strain meters are installed in clusters of 12 instruments each at several points on radial lines that define section of arches and cantilevers in the dam, forming the major system of instruments. The radial lines of strain meter clusters are located near the base of the maximum dam section and in three arches, uniformly spaced between the base of the dam and about two-thirds of the elevation of the dam. In each arch, the lines of instrument clusters are in the maximum section, near each abutment and at applicable intermediate locations between the abutments and the maximum section. The lowest arch contains three lines of instruments, the second contains five lines of instruments, and the third arch contains seven lines of instruments. Two arches between the two-thirds elevation of the dam and the top of the dam are instrumented using series of stress meters installed on seven radial lines in each arch. These instruments determine stress only in the direction of arch thrust. Pairs of strain meters are installed under free-surface metal canister- like covers, one pair of meters with each elevation of instruments, to detect possible autogenous growth or shrinkage of the mass concrete. At three widely separated locations near the top of the dam, trios of three-dimensionally arranged strain meters are installed near the upstream and downstream faces of the dam to determine surface stress in the structure.

In conjunction with the installation of strain meters and stress meters throughout the dam, joint meters were placed on the radial contraction joints at the same elevations as the meter clusters. Where the dam's longitudinal joint crosses blocks containing strain meters or stress meters, joint meters were installed on the longitudinal joint near its intersection with the radial joints. Additional joint meters were installed on the longitudinal joint at intermediate elevations between arches containing the strain meters and stress meters. Patterns of joint meters were installed on each of the two radial joints nearest each dam abutment and in the upper 200 feet of the dam elevation. The joint meters were embedded so that they were bisected by the transverse or longitudinal contraction joints and were properly anchored.

An installation of three mutually perpendicular meters—two strain meters and one joint meter—is located in the control cable tunnel where a stress relief joint in the abutment rock, termed the A joint, crosses the tunnel to detect movement of the joint. Electrical cables from the meters extend to three outlet boxes located in the powerplant.

Deformation meters were installed to detect initial foundation deformation as the load is applied to the sandstone foundation, and the elastic deformation of the foundation after the initial deformation has stopped. The deformation meters were installed so that they were bisected by the plane of contact between the foundation or abutment rock and the concrete of the dam and were anchored in the rock and the concrete of the dam. These long-range joint meters have anchorages that are made approximately 20 feet beyond the foundation surface, and are installed vertically on several radial lines at the base portion of the dam and in horizontal and sloping patterns at several elevations on each abutment.

Resistance thermometers were installed in the maximum dam section in a vertical grid pattern, having an approximate 40- by 50-foot spacing for determining temperature in the section. The temperature readings from the thermometers were augmented by temperature readings from the strain meters and stress meters in the same section. Special-purpose installations of a series of thermometers were made near one penstock at two elevations, and near the downstream face of the dam at three widely separated locations to determine localized temperatures.

178. INSTALLATION OF EMBEDDED METERS. The meters were furnished and assembled in groups by the Bureau and were installed under the prime contract by an electrical subcontractor. The cables, ground wires, outlet boxes, terminal boards, and covers were also furnished by the Government. Drawings of the meter installation layout and cable routing were furnished to the contractor for each individual block installation.

Initially, the contractor tried a blockout procedure for meter group installation. However, this was rejected early for economic reasons and the meter groups were most efficiently installed in excavations in the fresh concrete. As the clusters were assembled on a metal template, or spider, installation was quickly made by leveling and plumbing a horizontal and vertical meter of the cluster. Cable trenches were excavated in the fresh concrete to provide for a minimum coverage of 6 inches. The cables were laid in the trenches and the trenches backfilled by hand. The locations of the buried cables and meters were marked with an iron oxide dust, which was worked into the concrete surface, to minimize damage to the installation by drilling of holes for form installations and other purposes. Meter outlet boxes were installed at the terminal points directly upon the forms, and the terminal boards and covers were installed later when the terminals were soldered. When terminal boards were below the meter installation, conduit was usually installed for routing the cable from the meters to the boards.

As individual unit pay items were provided for these installations, no unusual difficulties were experienced in minor changes in the quantity of items. Some difficulties occasionally occurred when inexperienced workmen were assigned to the meter installation work. The most efficient and best installations were made when a single experienced crew was responsible for the meter installation.

179. DEFLECTION MEASUREMENTS. In addition to the embedded instrument installations, two systems of measurements employing refined methods of surveying were provided for determining the manner in which the dam deflects during periods of reservoir filling and operation.

One system comprises five plumblines, each in a formed well (fig. 267) extending from the top of the dam to a point near the foundation. The wells are located in the maximum section and at points approximately one-third and two-thirds the distances between the maximum section and the abutments. The plumblines are located in blocks 4, 7, 12, 18 and 21 of the dam, and have a total of 19 reading stations. At each reading station, movement between the dam and plumbline is measured using a micrometer slide and microscope apparatus. The micrometer can be read in ten-thousandths of an inch. The reading stations are oriented in plan so that measurements of deformation are in planes which are radial and tangential to the dam's axis. By this expedient, the measurements require no trigonometric resolution to obtain deformation in the desired directions. Readings are made periodically and are tabulated on forms in a manner that computer card punching can be made directly from the data sheet without further transposition or trigonometric resolution. The second system for determining deflection of the dam consists of a grid system of 68 targets placed on the downstream face of the dam and 17 targets on the foundation along the abutments. Locations of the targets are charted periodically from primary theodolite stations on each abutment downstream of the dam and from secondary stations on the canyon rim, using precise triangulation surveying methods. This system is discussed in further detail under section 186 on surveys.

Figure 267.—Plumbline well reading station in block 4 of dam at elevation 3390.00. P557-420-9334, February 7, 1964.

180. UPLIFT PRESSURE PIPES. Hydrostatic uplift at the base of the dam is measured at 41 locations by pipes connecting to wells at the concrete-rock contact plane and terminating in the dam galleries. Located in blocks 2, 4, 5, 7, 11, 16, 19 and 25, the pipes are arranged in seven lines, each line being made up of from five to seven pipes. Permanent data showing the elevation at which the pipes are installed are recorded on data sheets and readings are made at appropriate intervals. Pressure is measured by means of a Bourdon-type pressure gage calibrated in feet of water, attached through a gage cock to the uplift pipe. When zero pressure is indicated in a pipe, the water level is determined by sounding. Continued zero pressure with water standing at the level of the pipe is investigated further by adding a transparent standpipe section to the pipe to observe the level to which the water rises.

181. DRAINAGE MEASURING DEVICES. Weirs have been installed to elevation 3480 in the gallery gutters to measure total drainage flows from the following areas.

(1) Each foundation tunnel, including the access tunnel to the left diversion tunnel.

(2) Each area between the above tunnels.

(3) Each area in the foundation gallery from the lowest of the above tunnels to the pump sump.

(4) Each area in the drainage gallery from right and left of the transverse adit in block 11 to the top of the drainage curtain.

182. SEISMOGRAPH STATION. A seismograph station, located approximately 11 miles northwest of the damsite, records earthquake shocks. Records from the station show the magnitude of any earthquake

tremors in the vicinity and also serve to determine any possible change in local seismic activity that may occur in the area due to the increased weight of the reservoir. The equipment from the Bureau's Hungry Horse project was initially installed, but was replaced by new equipment in September 1964 when replacement parts for the obsolete equipment could no longer be obtained. Installation was made by members of the U.S. Coast and Geodetic Survey who also trained Bureau personnel in operation of the station.

The seismograph vault was constructed under specifications No. DC-5163 by W. W. Clyde and Co. of Springville, Utah, in the summer of 1959. The vault is 10 by 22 feet with reinforced concrete floor, walls, roof, retaining walls, and instrument piers. The walls and roof were 9 inches thick, the floor was 4 inches thick and the retaining wing walls ranged from 9 to 6 inches in thickness. The roof sloped 2 feet in 22 feet toward the open end of the vault. Backfill was placed around three sides and over the roof. A complete electrical system was installed by the contractor and electric power was supplied by Arizona Public Service Co. over a line which they constructed.

Approximately 2 miles of 12-foot-wide unsurfaced road was constructed under specifications No. DC-5163 to provide access from U.S. Highway No. 89 to the vault.

3. Concrete Cooling and Grouting

183. CONCRETE COOLING. Initial cooling of the mass concrete in the dam was provided to remove the heat of hydration. This was accomplished by circulating chilled water for a period of 12 days through 1-inch aluminum tubing, placed on top of each 7.5-foot lift (fig. 63). Secondary cooling to shrink the blocks prior to grouting continued for about 40 days or until a temperature of 40° F. was reached in concrete below elevation 3450 and 50° F. in concrete for elevation 3450 to the top of the dam.

Cooling water first came from a package refrigeration unit located just downstream of block 11, with a similar unit being used to cool the mass concrete in the machine shop and service bays. This package unit was used to cool concrete in the dam until August 3, 1960, and then a chiller, supplied with liquid ammonia from the main refrigeration plant, was used.

Between January and April 1961, six chillers were installed to supply additional cooling water. Three chillers were installed upstream of unit bay 1 of the powerplant and were connected to 8-inch headers installed on brackets above the elevation 3120 catwalk across the downstream face of the dam. The fourth chiller supplied cooling water for concrete in the machine shop, the service bay, and the dam above elevation 3060. Two chillers were placed on the elevation 3180 catwalk for initial cooling above elevation 3180 while the other cooling system was used for secondary cooling of the elevation 3120 grout lift.

In January 1962, three chillers were moved to the downstream end of the refrigeration plant on the elevation 3715 bench. Gravity flow of cooling water through the cooling coils was used for both initial and secondary cooling. Pressure was reduced by valves located on the elevation 3240 catwalk on the downstream face of block 21. Two pumps, rated at 1,800 gallons per minute at 392 feet of head, were installed on the elevation 3240 catwalk to pump the water back to the chillers.

Pressure-reducing valves and return pumps were also installed on the elevation 3300 catwalk on the face of block 22 for initial cooling. These pumping stations were raised to higher stations as necessary. The cooling water was delivered through 8-inch horizontal headers installed across the face of the dam. Vertical 4-inch headers led from the 8-inch header to blocks being cooled, and 1-1/2-inch headers led from there to the 1-inch aluminum cooling pipe. As the blocks became small, near the top of the dam, the flow of cooling water had to be reduced below 4 gallons per minute to keep the lifts from cooling faster than 1-1/2° F. per day. Cooling water normally flowed at the rate of 4 gallons per minute, except during some cold periods when the rate of flow was increased to prevent freezing.

184. UNUSUAL STRESSES DURING CONSTRUCTION. Numerous horizontal and vertical cracks appeared in the dam blocks beginning in 1960. These cracks appeared especially during final cooling and grouting. They were plugged with lead wool before grouting of a grout lift and sealed by the contraction joint grouting. A number of cracks showed up in block 8-B. Two mats of reinforcing steel were placed in the elevation 3112.5 lift to stop a crack which extended from the 8-B—9-B block line at elevation 3067.5 and from the 7-B—8-B block line at elevation 3075 extending to the top and across the surface of the elevation 3097.5 lift and to the top of the elevation 3105 lift. The crack was not noted higher than elevation 3105.

It was thought that changing the intensity or pattern of the temperature stresses within each block by reducing the temperature gradient might be a solution to avoid this cracking. The cooling procedure was, therefore, changed. After grouting the elevation 3180 lift, the next area cooled was the 3240 to 3300 grout lift instead of the 3180 to 3240 grout lift. Since some of the concrete in the higher lift was relatively new when secondary cooling was started, the cooling was performed in the cooling coils at elevations 3277.5, 3285.0, 3292.5, and 3300.0 only until the temperature at thermocouple elevation 3288.75 was 60° F. At the same time cooling was performed in the cooling coils at elevations 3247.5, 3255.0, 3262.5, and 3270.0 until the temperature at thermocouple elevation 3258.75 was 55° F. Upon completion of the above-described partial secondary cooling for the 3240 to 3300 grout lift, the 3180 to 3240 lift was cooled to 40° F. This procedure took place in succeeding pairs of lifts. The top half of the upper lift was cooled to 60° F. while the lower half of this lift was being cooled to 55° F. The lower lift was then connected for the second time, cooled to 40° F. and grouted. This sequence was followed up to elevation 3420 where practicable. Some modification was necessary due to topping out of the longitudinal joints of the odd-numbered blocks 7 through 17 at elevation 3375. This change permitted the cooling of all concrete below the top of the longitudinal joints to the required 50° F. temperature after the 3180 to 3240 grout lift was cooled to its final temperature of 40° F. In a similar manner, the cooling in blocks 5, 6, 19, and 21 deviated from the two-step sequence of cooling operations. The cooling in these blocks at 50° F. was accomplished after the 3240 to 3300 grout lift had been cooled to 40° F.

In January 1962, a mat of reinforcing steel was placed in block 21-B in the 3300 to 3307.5 placement lift to prevent further extension of a longitudinal crack.

During 1962, the plan for final cooling in the dam was changed from that given in paragraph 140 of the specifications to the following:

Up to elevation 3300	40° F.
Elevation 3300 to elevation 3360	45° F.
Elevation 3360 to elevation 3600	50° F.
Elevation 3600 to top of dam	55° F.

The Board of Consultants[1] in their report of October 12, 1961, stated: "The cracks described which were not available for inspection by the Board, are undesirable but are not believed to endanger the safety of the structure. Their elimination at the expense of appreciably increasing abutment stresses above the present specified limit is not justified."

On August 1, 1962, permission was received from the Denver office to extend the initial cooling periods for those lifts at or below elevation 3420, provided that the drop in temperature did not exceed 1° F. per day. Further, the cooling of the upper and lower halves of the 3360 to 3420 grout lift to 60° and 55° F., respectively, was done simultaneously. Final cooling of the 3360 to 3420 grout lift to 50° F. coincided with the cooling of concrete to 50° F. in the high blocks having joints which terminated at elevation 3480.

On February 12, 1962, the Denver office requested that a special thermocouple installation be made in block 4-A, at the elevation 3397.5 lift. The thermocouples were located halfway between the elevation 3390 gate chamber gallery and the block 4-A—4-B longitudinal contraction joint, and midway between the transverse joints. The purpose of this installation was to provide additional temperature data on the temperature differentials within a lift in order to correlate temperature stress studies with 7-1/2-foot placement lifts.

The first thermocouple was placed on top of the lift at elevation 3390 and was taped to a cooling pipe. Thermocouple No. 2 was embedded vertically above thermocouple No. 1 at a distance of 6 inches, and the next seven thermocouples were embedded vertically above No. 2 at 1-foot intervals. Thermocouple No. 9 was placed at the top of the lift at elevation 3397.5. Thermocouples No. 10 through 15, inclusive, were placed at elevation 3390, adjacent to thermocouple No. 1. These five thermocouples were placed in a line normal to the cooling pipe at 6-inch intervals. The distance from the last of these, No. 14, to the next cooling pipe was measured and recorded.

Readings on all thermocouples were taken at the time of embedment. For the first 5 days of the initial cooling period, three readings were taken per 24-hour period at about 7- or 8-hour intervals. For the remainder of the initial cooling period, one reading per day was taken. No further readings were taken until final cooling was started in this area. At that time, readings were taken on a twice-a-day schedule for the first 5 days then twice a week until final cooling was completed. No readings were required thereafter.

[1] "Glen Canyon Dam Design and Construction Problems," R. E. Davis, J. W. Vanderwilt, J. J. Hammond, E. B. Burwell, Jr., Julian Hinds, October 12, 1961.

Other thermocouples were installed in the dam at 30-foot vertical intervals. During initial cooling, readings were taken twice daily for the first 5 days and once daily for the next 7 days. During the interval, until secondary cooling began, readings were taken twice a month. When secondary cooling was started in a grout lift, the thermocouple readings were taken once a week for 2 or 3 weeks and when the temperature in the grout lift approached 40° F. they were taken two or three times a week. About 800 thermocouples were installed in the dam during the placing of mass concrete.

Although considerable cracking occurred in the dam blocks throughout 1962, fewer cracks were noted than in previous years. The cracks ranged from 1/16 inch to 1/32 inch in width. Water emitted from most of the cracks, and the lifts were water tested and all cracks which made water were calked with lead wool before grouting.

Almost no cracks were noted during 1963. As the dam rose, the blocks became smaller which no doubt reduced the tendency toward cracking. However, in the fall of 1963, a crack was noted near the contact of the concrete and sandstone at the right abutment. The crack started in the rock upstream of the dam, entered the adit at the end of the utility gallery, came out of the adit and continued across the contact of the dam and the rock and continued down the concrete-rock contact to about elevation 3680.

Twenty-one holes were drilled from the top of the dam and the utility adit and gallery, elevation 3697.5, to intersect this crack above elevation 3680. The four downstream holes at elevation 3680 lost water near the contact. This crack was grouted early in June of 1964.

185. GROUTING. (a) *Diversion Tunnels and Spillways.*—Initial drilling and grouting activities consisted of drilling holes through the right diversion tunnel and spillway tunnel linings to grout the voids in the arch between the concrete lining and bedrock (fig. 105). Grouting of these voids was started in November 1958, in the right spillway downstream of station 26+11.

The general pattern for locating arch holes consisted, alternately at 10-foot centers, of two holes located 30° right and left of the centerline of the arch and one hole at the centerline of the arch. Some of the 1-1/2-inch pipes embedded in the concrete lining for radial grout holes were used for grouting arch voids, when they fitted the above pattern.

The grout was injected at a maximum pressure of 20 pounds per square inch, and no sand was used in the mix. Type II cement from paper bags was used with a water-cement ratio of 1:1 by volume. A circulating line from air-driven 6- by 3-1/2- by 6-inch and 7- by 3- by 10-inch duplex piston-type pumps was used to deliver the grout to the manifold at the collar of the hole where the injection pressure was controlled by valves. A maximum of five grout units were in operation at one time with the fifth unit containing a helical screw-type pump. Later, another screw-type pump was used to replace one of the duplex piston pumps.

The mixer was a horizontal cylinder tub with mixer blades rotating around a horizontal axis. The agitator sump was a vertical cylinder tub with baffles; and paddles were attached to a vertical shaft. Both were powered by small air motors. The water meter read to tenths of a cubic foot. Telephone communication was provided between the manifold and pump. Drilling equipment consisted of lightweight air-powered, power-vane diamond drills. Bits were EX size (1-5/16-inch) and both core and plug bits were used. Movable jumbos were used in the 41-foot-diameter tunnels for drilling and grouting operations.

Radial grouting was started in December 1958, and the french drain was grouted at the upstream portal. Arch void and radial grouting and drilling of drainage holes continued in the right diversion and spillway tunnels through January 1959, when work was suspended in anticipation of diversion of the river through the right diversion tunnel. Closure was completed and the river was diverted through the right diversion tunnel at 7:30 a.m. on February 11, 1959.

The following work remained to be done in the right spillway and diversion tunnel at that time:

(1) Drilling of 35 radial grout holes between stations 23+95 and 25+79.

(2) Drilling of 21 radial grout holes between stations 32+39 and 39+99.

(3) Drilling of a few holes to check the adequacy of arch void grouting from station 25+39 to station 24+37 due to omitting radial holes in this area.

(4) Drilling of drainage holes from station 27+11 to station 30+11.

Drilling of radial grout holes was started in the left diversion and spillway tunnel in February 1959. Grouting of arch voids and radial grouting continued in the left tunnel until April 7, 1959, when the work was suspended in anticipation of the left diversion tunnel carrying a part of the spring runoff. Pressure in the radial grouting varied from 10 to 100 pounds per square inch, and the water-cement ratio was 5:1 by volume. Arch void grouting was completed by April 1959, but some radial grouting remained to be done.

Grouting of voids under the liners and outlet gates in the left diversion tunnel was started in December 1962, and completed under the three liners and gates in January 1963. Drilling and grouting of the remaining radial grout holes were started in the left diversion and spillway tunnel on December 21, 1962, and completed on January 15, 1963. Contraction joints, periphery and cooling pipe in the left diversion tunnel plug section were grouted, and also the periphery of the contractor's access adit. Two drainage holes were drilled.

Final closure of the right diversion tunnel was made on January 23, 1963. The contractor's 16-foot plug adjacent to the closure gates was grouted in March. Radial grouting was completed in the backfill section of the right diversion tunnel and contraction joints, and periphery and cooling pipe were grouted in the plug section. The 18-inch bypass pipe through the plug section was plugged with concrete in June. All concrete had been placed in the backfill section of the tunnel by mid-July 1963. Arch void and radial grouting and drilling of 3-inch drainage holes were completed in the right spillway early in June 1964.1

Arch void grouting, radial grouting, and drilling of drain holes were started in August 1963, at the upper end of the left spillway tunnel behind station 21+40. The rch oid grouting as completed in September and the radial grouting nd drilling of drainage holes was completed in October 1963.

(b) *Stress Relief Joint.*—The report of the Board of Consultants,[2] dated May 5, 1960, recommended that suitable instrumentation should be provided for determining possible future movement of a wedge-shaped rock mass at the downstream side of the right abutment keyway below elevation 3370. The mass was separated from the canyon wall by a stress relief joint, and this joint was designated as joint A.

The A-joint was readily accessible for instrumentation in the powerhouse control cable tunnel and along the joint exposure at the surface of the right keyway. Carlson strain meters and a joint meter were installed in the control cable tunnel in February 1961. Small vertical and horizontal movements were recorded but they appeared to be the result of temperature changes. The Consulting Board in its report of October 12, 1961,[1] recommended that the measurements be continued.

Additional vents to the A-joint were drilled downstream of the A-line above elevation 3174 where there is no concrete to support or hold the rock on the right canyon wall. An NX hole (2-3/8-inch drill size) at powerplant station 4+14 at elevation 3172, drilled on the right canyon wall, showed a grout seam at a depth of 32.6 feet, which is the A-joint. Three additional holes at stations 4+62, 4+14, and 3+64 at about elevation 3196 indicated the depth to the A-joint ranges from 38 to 36 feet.

Beginning in February 1961, stress relief joints in the right abutment above elevation 3121.5 were grouted in 7.5-foot lifts as the concrete lifts were placed in the blocks, so as to maintain concrete against the rock being grouted. Grouting was accomplished through 1-inch risers with relief vents just above the top of the next lift to be placed. Vents were washed after being used for grout returns and then utilized as supply holes for ensuing lifts. These grout holes passed through the A-joint in most cases. In November 1961, the surface of the A-joint was exposed in the downstream area above elevation 3260 and, therefore, the stress relief holes no longer intercepted the A-joint with the exception of the vent 10 feet downstream of the dam. As the collars of the stress relief holes were on the surface of the A-joint, there was no grout travel downstream along the joint above elevation 3256. The stress relief joints were grouted to elevation 3496.5 when the final vents at this elevation were grouted in February 1963.

(c) *Dam Foundation.*—Preparations for drilling and grouting B-holes in the dam foundation began on May 12, 1960, when nipples were set between dam axis

[1]Op. cit. p. 454.

[2]"Foundation Adequacy and Design Considerations of Glen Canyon Dam," R. E. Davis, J. W. Vanderwilt, J. J. Hammond, E. B. Burwell, Jr., Julian Hinds, May, 1960.

stations 6+00 and 8+40 in the upstream portion of the dam foundation. Grout take in the area below elevation 3078 was relatively small.

Drilling equipment consisted of lightweight, air-powered, power-vane diamond drills. The drilling speed normally used was 50 revolutions per inch of forward movement. Standard EX size plug bits were used.

The grouting equipment included a helical screw-type pump electrically powered. Mechanical packers were largely used with pneumatic packers available for oversize holes. Cement used was type II in paper bags from the plant in Clarkdale, Ariz. Initial B-hole grouting was confined to blocks 6 and 7. Grouting of these holes, which intersected the bedding plane at elevation 3078, succeeded in confining water leakage to the vicinity of the heel with minor leakage about 200 feet downstream from the heel along the bedding plane.

Five holes on 20-foot centers were drilled and grouted downstream of line 8+93 on dam axis station 11+80 to locate the line of flow of water which was leaking on the elevation 3069 bedding plane joint, about 60 feet downstream of line 8+93. The grout in these holes intersected the water from the bedding plane joint as planned. The concentrated flow was then located on the same joint in block 15 some 200 feet downstream of the dam axis. The flow was piped to the drainage gallery in block 14. Progress in B-hole grouting depended on the rate of placement in adjacent low dam blocks. As the dam rose, B-hole grouting was continued in the right and left abutments. The holes averaged 25 feet in depth, the pressure was about 25 to 45 pounds per square inch, and the water-cement ratio 5:1. Drilling and grouting of B-holes in the left abutment was completed in April 1963 and in the right abutment in July 1963.

A-hole grouting from the foundation gallery was initiated in November 1961, in blocks 10, 12, and 14. Very little grout was taken at first. The A-holes made water which accumulated gradually as the hole was drilled deeper, indicating the permeability of the sandstone. Hole 45, drilled to a depth of 126 feet, made water at the rate of 8 gallons per minute and a maximum water pressure of 38 pounds per square inch. Water flowed from other holes in comparable quantities. Holes were drilled in 40-foot stages because deeper stages frequently caused oversized holes in which packers could not be seated. Grouted stages of incomplete holes were washed from 6 to 8 hours after grouting to prevent the necessity of redrilling. Holes were grouted in 20-foot stages from 0- to 100-foot depths, in 25- or 30-foot stages between 100 and 150 feet, and in 40-foot stages below 150 feet. Maximum pressures of stages were determined from the formula:

$$\text{Pressure (p.s.i.)} = 250 + 2d$$

where

d = depth of the packer in rock.

A water-cement ratio of 10:1 was used after trying thinner ratios up to 20:1 for deeper stages with no apparent increase in take.

Because of water flowing from drilled A-holes, the area between holes 29 and 45 was made a test area to check the feasibility of decreasing the flow of water in the upper stages of holes prior to drilling the lower stages. All holes were drilled and grouted in 40-foot stages before proceeding deeper in any one hole. The holes were drilled and grouted to the same depth according to the method-of-closure sequence, i.e., on 80-, 40-, 20-, and 10-foot centers, respectively. Flow quantities from each hole were measured after drilling and after washing the stages. It appeared when following out this procedure that none of the water was being sealed off by grouting and that the entire depth of the hole, in rock, was making water.

The grout curtain between holes 29 and 45 was completed in April 1962. Drainage holes near grout hole 30 were drilled in February 1962, and a measuring device installed with which the pressure and flow could be measured. The flow at first measured from 5 to 8 gallons per minute but dropped to 5 gallons or less within a few weeks. The A-hole grouting program was completed in July 1963.

(d) *Chemical Grouting Experiment.*—At this time the Denver office was formulating a test program of the feasibility and effectiveness of chemical grouting of foundations. Before starting actual insertion of chemical grout, as much information as possible was desired in connection with source of foundation flow, effectiveness of cement grouting, rate of flow, and similar data. Accordingly, the following procedures were suggested and followed:

Step 1. Complete A-hole drilling and grouting of the test area between holes 29 and 45, in the stages reported above.

Step 2. After completion of above, drill the drain holes in both the foundation and the drainage galleries through the embedded pipes closest to the radial lines through

A-holes 30, 37, and 44, to full depth, i.e., 85 feet plus or minus into rock, and record flows daily.

Step 3. About 2 weeks after completion of step 2, drill the drain holes in both the foundation and the drainage galleries downstream of A-holes 36 and 38 to full depth and record flows daily.

There was also some early concern regarding erosion behavior of the foundation drainage holes. Water flowing from the initially drilled A-holes deposited sand on the gallery floor and gutter when left uncapped for a period of several hours. Therefore, in addition to the water flow data for drainage holes as outlined in steps 2 and 3 above, the volume of sand that was washed or eroded from each of the drainage holes was recorded. All of the above records were reported in the weekly progress report. The sand deposit proved to be negligible. Step 2 was completed in March 1962, and step 3 started immediately.

The advisability of using a chemical grout to seal the Navajo sandstone had been considered previously, but inadequate data were available for proper evaluation. Further testing was therefore necessary to determine if chemical grouting would be effective and which chemicals would give the best results. The Bureau did not have the necessary grout injection equipment nor available personnel to supervise performance of the work. Navajo sandstone is composed of fine to medium sized quartz grains with minor amounts of feldspar, which are only poorly to moderately well cemented. Much of it is cemented only at the points of contact between the grains, leaving a fairly high percentage of open and connected voids through which water will pass slowly but through which cement grout will not pass.

Contract No. 14-06-D-4470 was negotiated with Halliburton Oil Well Cementing Co. of Duncan, Okla., who was considered the sole source of supply for the work to be performed. Under this contract the contractor furnished one grouting unit and all necessary related equipment, six chemical grouting materials, and the service of two engineers or technicians who were thoroughly familiar with the equipment and materials furnished by the contractor, and who were competent to advise the Bureau as to chemical grouting procedures.

The test site selected was the floor of the left spillway approach channel. Agreements were made with the prime contractor for preparing the site, drilling, furnishing operating personnel, water, electricity and air. Items not payable under specifications No. DC-4825 to Merritt-Chapman and Scott were included under an order for changes.

The test area was divided into five contiguous subareas, each 15 feet in diameter. The central hole in each subarea was preserved for injecting grout after saturating each area with several thousand gallons of water through the six holes spaced around the perimeter of the test area. The holes were 1-1/2 inches in diameter and 30 feet deep.

The tests included use of four chemical grouts, designated for convenience as follows: type A—a resin-type grout with a viscosity of 6 centipoise; type B—a lignin-type grout with a viscosity of 8 centipoise; type C—sodium silicate plus sodium chloride of very low viscosity; and type D—a combination of chemicals that polymerize to produce a stiff gel, viscosity 3 centipoise at 78° F.

Injected quantities of grout were negligible. The sandstone refused to accept the chemical grout in quantity. A new test area was therefore prepared and a different source of water was obtained, as it was felt that dissolved solids in the mixing and saturation water were reacting chemically with grout materials. The new water supply was obtained from Merritt-Chapman and Scott's domestic water supply which contained 70 to 80 percent less dissolved solids than the water used in the first series of tests. Water used in the first series was from the same source as that used for concrete.

Grouting operations in the new test area were more successful than those in the original area, the penetration into the sandstone of the three chemical grouts, types A, C, and D, being greatly increased. Acceptance by the sandstone of these grouts in decreasing order was type D (5 percent concentration), type A, and type C (10 percent and 11 percent solutions with or without addition of 2 or 2-1/2 percent, respectively, of sodium chloride). Injection of type B grout was not successful due to its immediate reaction with the chemicals of the sandstone which apparently plugged the voids.

As a result of these tests, further consideration of chemical grouting of Navajo sandstone at Glen Canyon was terminated because of the small quantities accepted and short travel of the four chemical grouts tried. The field tests were carried out between October 2 and October 30, 1962.

(e) *Rock Bolting.*—In its report of October 12, 1961,[1] the Board of Consultants, in view of stress relief joints that had developed, or might develop, in the abutment rock downstream of the dam, particularly in the right abutment, and the increase of rock stresses that would result from the arch thrust, recommended that extensive rock anchorage and drainage be provided to insure stability of these arch masses. The most critical areas appeared to be those below elevation 3450 in the right abutment and somewhat below elevation 3350 in the left abutment where the intrados of the arch at the abutment lies close to the surface and where relief jointing is most noticeable. The Board was of the opinion that the anchorage in the right abutment should extend at least 25 feet back of the A-joint.

Holes ranging from 40 to 80 feet in depth were drilled between elevations 3420 and 3150 and from station 5+00 at the A-line of the powerplant, downstream to station 3+00. Most of the holes were from 60 to 70 feet in depth and all extended more than 25 feet back of the A-joint.

The holes were cleaned thoroughly with air and water. The bottom 15 feet of each hole was sandblasted to remove grease, etc., to assure bonding with grout, and the top 5 feet of each bar was wrapped with waterproof building paper to prevent bonding. When a bar was lowered into the hole, 1/2-inch plastic tubing was attached to the bottom of the bar through which a measured quantity of 0.8:1 cement grout with 5 parts of aluminum powder to 100,000 parts of cement was injected into the bottom of the hole to assure that the bottom 12 feet of hole was filled with grout. The grout stop was then set at 5 feet from the top of the hole and the top section of the bar grouted in place.

The surface of the bearing plate was roughed out by bush-hammering, then finished with a grinding wheel with diamond inserts built especially for this operation. Each bar was loaded by a hydraulic ram, having a 40-ton capacity, to a tension of 63,000 pounds, after which the nut was tightened against the bearing plate with a torque of 3,500 inch-pounds. The combination of elongation of the bar and possible compression of the sandstone ranged from 5/16 inch to 9/10 inch.

Three-hundred and twenty-five bolts were installed and loaded on the right canyon wall and 189 were installed and loaded on the left canyon wall. The right canyon wall was completed in July 1963, and the left wall in September of that year. The anchor bars were loaded not less than 28 days after the bars were grouted in place. A total of 134 drain holes was drilled on the canyon walls. After the initial start, a rotary drill was used with lightweight drill rods and tricone roller rock-bits.

Under order for changes No. 24, the contractor furnished and installed regular rock bolts and 1-1/2-inch anchor bars in the west canyon rim in the vicinity of the skewback of the Glen Canyon Bridge. This work was done in September and October 1964, after a fallout occurred under the skewback. Ninety feet of regular rock bolts were installed and 1,225 feet of anchor bars were installed and grouted in place. The anchor bars were approximately 20 feet in length.

(f) *Grout Adits.*—Drilling and grouting from the right and left abutment grout adits was started in June 1962 in the elevation 3157.5 and elevation 3270 adits. Vent or pressure relief holes were drilled in the right abutment downstream of the dam on the powerplant A-line between elevations 3150 and 3250 to act as vents while grouting from the grout adits. Grouting in the left abutment took place in the grout adits at elevations 3315, 3367.5, 3427.5, 3480, 3547.5 and 3630 and was completed in May 1963. Grouting in the right abutment was performed from the grout adits at elevations 3157.5, 3217.5, 3270, 3322.5, 3367.5, 3427.5, 3480, 3547.5 and 3630 and this work was completed in July 1963. Holes ranged from 40 feet in depth to 100 feet. The pressure ranged from 100 to 125 pounds per square inch, with most of the holes being grouted at 125 pounds per square inch.

During the period from May 1961 through April 1964, eighty-eight 10-inch-diameter concrete cores were drilled in the dam. Ten cores were drilled in B-blocks 7, 8, 11, 14, 15 and 17. Sixty-one cores were drilled in the floor and walls of the galleries and adits of the dam at various elevations, and 17 cores were drilled in the face of the dam at various elevations. These cores were drilled in blocks 5, 6, 8, 11, 12, 14 and 26. Two cores were drilled vertically in the service bay at elevation 3188.

(g) *Contraction Joints.*—First contraction joint grouting was from bedrock to elevation 3060 and was completed in May 1961. The joints grouted included:

[1] Op. cit. p. 454.

(1) Four transverse joints between blocks 9-10, 10-11, 11-12, and 12-13. Each transverse joint was separated by metal seals into two separate areas. A small area, 4 feet 6 inches wide, was located near the upstream face of the dam.

(2) Five longitudinal joints in blocks 9 through 13.

This was the total of the contraction joints below elevation 3060. Temperatures of the joints were mostly lower than the 40° F. requirement and some reached a minimum of 29° F. in the final stages of secondary cooling. Joint meters indicated joint openings of 0.105 to 0.157 inch at elevation 3037.5; and at elevation 3060 openings ranged from 0.022 to 0.089 inch in the same blocks.

Backfill between the dam and the powerplant was placed to elevation 3060 at the downstream face of the dam at the time of the grouting. This required the contraction joint piping systems extruding from the face of the dam below elevation 3060 to be grouted through 1-1/2-inch risers extending to approximate elevation 3067.

Contraction joints in the elevation 3060 to elevation 3120 lift were grouted in October 1961. Thirteen transverse joints were grouted, extending from joint T6/7 to joint T18/19 and 13 longitudinal joints in blocks 6 through 18 were grouted.

Refrigerated brine was used during the latter part of secondary cooling, and all blocks were lowered to 40° F. except block 11 which was at 50° F. While testing the contraction joints and attempting to fill the transverse joints with water, numerous leaks appeared from cracks in the drainage and foundation galleries. The cracks were calked with lead wool by use of pneumatic chipping hammers and then grouted to seal the leaks.

Contraction joints in the 3120 to 3180 lift were grouted in January 1962 as follows: (1) Seventeen transverse joints extending from joint T4/5 through joint T20/21; (2) 15 longitudinal joints extending from block 5 through block 19; and (3) 13 special transverse joints extending from T6/7 through T18/19. Concrete temperatures were lowered to 40° or 45° F. prior to grouting. Leaks again developed and were calked with lead wool and sealed with grout.

Contraction joints in the 3180 to 3240 lift were grouted in May 1962, as follows: (1) Nineteen transverse joints extending from joint T3/4 through joint T21/22; (2) 18 longitudinal joints extending from block 4 through block 21; and (3) 15 special transverse joints extending from joint T5/6 through joint T19/20. Numerous leaks again appeared in cracks and these were calked and grouted.

The 3240 to 3300 grout lift was grouted in July 1962, and included the following contraction joints: (1) Twenty transverse joints extending from joint T3/4 through joint T22/23; (2) 20 longitudinal joints extending from block 3 through block 22; and (3) 70 transverse joints extending from joint T4/5 through T20/21. Some leaks were found in the galleries, adits and shafts, and these were calked and sealed with grout.

The 3300 to 3360 grout lift was grouted in November 1962 and included the following contraction joints: (1) Twenty-one transverse joints extending from joint T3/4 through joint T23/24; (2) 21 longitudinal joints extending from block 3 through block 23; (3) 19 special transverse joints extending from T3/4 through T21/22; and (4) 1 longitudinal perimeter joint in block 3. Fewer leaks occurred in this grout lift than in previous lifts and these were calked and grouted as before. Figure 65 presents some transverse contraction joint information.

The 3360 to 3420 grout lift was grouted in December 1962 and included the following contraction joints: (1) Twenty-one transverse joints extending from joint T3/4 through joint T23/24; (2) 21 longitudinal joints extending from block 3 through block 24; (3) 20 special transverse joints extending from T3/4 through T23/24 exclusive of T21/22, which was grouted in March 1963; and (4) 2 longitudinal perimeter joints, blocks 3 and 24. Only one leak which required calking was found in this lift.

The 3420 to 3480 grout lift was grouted in March 1963 and included the following contraction joints: (1) Twenty-one transverse joints extending from joint 3/4 through joint 23/24; (2) 22 longitudinal joints extending from block 3 through block 24; (3) 21 special transverse joints extending from T3/4T through T23/24T and T21/22T of the 3360-3420 grout lift which was omitted with that lift; and (4) 2 longitudinal perimeter joints, blocks 3 and 24. Some leaks were again found and were calked and sealed with grout.

On March 11, 1963 a check was made on transverse contraction joints 1/2 and 2/3 from bedrock to elevation 3480 to determine the extent of interconnections and leaks that could be anticipated while water loading these joints with a pressure of 100 pounds per square inch. On March 12, the same check was made on transverse joints 24/25 and 25/26 from bedrock to elevation 3480.

The procedure consisted of installing a 1-1/2-inch water supply line to each transverse joint. A three-way header was used with valves and a gage to determine the water pressure at the header before and during the injection of water into the joints. Gages were also installed on the vent system of the joint being tested in order to keep the pressure below 25 pounds per square inch, initially. Each transverse contraction joint was tested individually.

All of the four transverse contraction joints below elevation 3480 were found to be connected to their respective transverse perimeter and transverse face perimeter.

On March 28, 1963, the T2/3 and T1/2 joints were water loaded, but it was impossible to obtain a pressure of 100 pounds per square inch due to excessive leaks where the joints crossed galleries or adits. On March 29 the joints were grouted, each grout lift from bedrock to elevation 3480 being grouted individually. All transverse joints were interconnected with their respective perimeters, and in each case the face perimeter was grouted with the perimeter. All joints were sealed off under pressure. Beginning April 15, the same procedure was used on the right end joints, but it was not possible to obtain a pressure of 100 pounds per square inch on the joints.

The 24/25 joint between elevations 3420 and 3480 developed a leak where the joint crossed the grout adit at elevation 3427.5. The joints were grouted on April 16, each joint being grouted individually.

Duplex piston-type pumps, size 10 by 3 by 10 inches, were used at each location for grouting of the contraction joints. A circulating grout supply line, 1-1/2 inches in diameter, was laid on the catwalk most convenient to the lift being grouted. Type II cement in waterproof bags was used and the starting water-cement ratio was 1:1. This was reduced to 0.8:1 when the joints became half full.

The 3480 to 3540 grout lift was grouted in November 1963 after the reservoir had reached elevation 3395. The joints grouted were as follows: (1) Twenty-three transverse joints extending from joint 2/3 through joint 24/25; (2) 8 longitudinal joints, L-2, L-3, L-4, L-30, and L-22 through L-25; (3) 23 special transverse joints extending from T2-3T to T24 25T; and (4) 2 longitudinal perimeter joints, blocks 2 and 25. All joints that leaked were calked with lead wool and all joints were grouted under pressure except T5/6 and T5/6T which leaked into the elevation 3600 grout lift.

The 3540 to 3600 grout lift was also grouted in November 1963 and included the following contraction joints: (1) Twenty-three transverse joints extending from joint T2/3 through joint T24/25; (2) 2 longitudinal joints L-2 and L-24; (3) 23 special transverse joints extending from T2/3 through T24/25 and one longitudinal perimeter joint L-2. The joints were tested by filling with water, and those which leaked were calked with lead wool before grouting. All joints were grouted under pressure.

Grouting of the 3600 to 3660 grout lift was completed in December 1963 and included the following contraction joints: (1) Twenty-three transverse joints; and (2) 23 special transverse joints extending from the 2/3 joint through the 24/25 joint. Joint meters and dial indicator gages were read at time intervals during grouting to detect movement of blocks. The joints were filled with water prior to grouting and those which leaked were calked with lead wool. All joints were grouted under pressure.

Grouting of the final grout lift between elevations 3660 and 3715 was completed December 20, 1963. This included 23 transverse joints and 23 special transverse joints from the 2/3 joint through the 24/25 joint. Joint meters and dial indicator gages were read at time intervals during the grouting to detect movement of blocks. Prior to grouting, the joints were filled with water and those which leaked were calked with lead wool. All joints except four T and four TT joints were grouted under pressure and these joints would not hold pressure. Joint meters and dial indicator gages were read at time intervals before and during grouting to detect movement of the dam blocks.

Water loading and grouting of end contraction joints 1/2 and 25/26 from elevation 3480 to elevation 3660 and the top two lines of the reinjectable grout system below elevation 3480 was completed January 21, 1964. The 1/2 transverse joint was water loaded but due to extensive leaks only pressures between 21 and 40 pounds per square inch could be held. In the 25/26 joint, pressures between 40 and 60 pounds per square inch were held for 6 hours but it was impossible to obtain 100 pounds per square inch. An attempt was made to inject grout into the 3420 to 3480 grout lift through the reinjectable grout system but grout was refused.

A crack developed at the concrete-sandstone contact above elevation 3667 at the right abutment and this was grouted in June 1964 to complete the grouting program on the dam.

Figure 66 shows some of the longitudinal contraction joint grouting system.

4. Construction Surveys

186. GENERAL. Surveys for construction of Glen Canyon Dam were conveniently divided into two categories—control and construction. Control surveys refer to those directly related to geodetic principles; construction surveys are those loosely grouped under the localized application of plane surveys. Controls were further divided into the usual horizontal and vertical surveys. Vertical controls and all applications at the dam are those with elevations in feet above the mean sea level datum. The triangulation network possesses the more unique quality of horizontal control due to the local terrain and construction requirements. This network is a chain of quadrilaterals having upper and lower net portions with suitable ties and offsite references. Horizontal distances of the network were adjusted to datum plane elevation 3500 as an average elevation of the construction area.

187. CONTROL SURVEYS. Preliminary studies for feasibility of construction of the high dam at the Glen Canyon site were based on fourth order surveys and the U. S. Geological Survey quadrangle topographic map. More detailed topographic maps and some aerial photographs were made as investigations progressed. These were used to record the geological data obtained from diamond drill holes, exploratory tunnels, and surface examination of the site. When the overall suitability of the site became evident, higher order controls were desired for design studies and for later construction use.

Vertical controls from bench marks set along the Colorado River by the U. S. Coast and Geodetic Survey in 1921 were accepted. Through financial contribution by the Bureau of Reclamation, the Coast and Geodetic Survey brought in a second order triangulation network in 1947. This network consisted of six stations within a mile radius of the site. It included a measured baseline of more than 7,000 feet in length as a check, and provided basic data on a central zone of the Arizona State coordinate system. These stations, supplemented by secondary points at the site, were used to complete investigations and preliminary design work for the principal features of the project.

During 1956, the Bureau of Reclamation survey crews checked out the Coast and Geodetic Survey stations by turning angles with a European-made 1-second theodolite to confirm that the stations to be used had not been disturbed over the preceeding 9-year period. The Bureau then laid out the local primary control network with stations along each side of the canyon rim and incorporated one station each on the east and west sides of the river. The coordinates of these stations were adjusted to the mean elevations, and the adjusted distances and bearings between them were computed. This adjusted line was used in further computations for projecting the Bureau local net.

The initial observations were started in November 1956. The primary local network included an upper net along the canyon rim and a lower net along the floor of the canyon. There were 6 stations along each canyon rim on an approximate 1,000-foot grid pattern, 5 stations about 500 feet back from the canyon rim on each side, and 12 stations along each wall near the canyon floor. A 1-second theodolite was used for the observations, utilizing 12 positions of the circle. Many of the observations of angles were made at night. However, when weather and atmospheric conditions were favorable, observations were made during the daylight hours. Some difficulty was experienced in getting triangulation to close to second order accuracy because of the steep vertical angle. Under this condition, when the instrument plate was even minutely out of level, inaccuracies in the horizontal angle were substantially magnified. A striding level was affixed to the theodolite in making these observations, and all triangles were closed within 5 seconds of 180°.

After the positions of the network station had been observed, adjusted, and coordinates computed, an azimuth check was made to the Coast and Geodetic Survey stations. Excellent results were obtained as the average azimuth check from the top to the bottom was within approximately 1.5 seconds of the computed bearings. The completed network consisted of 28 second-order quadrangles, 12 third-order quadrangles and central point figures. A triangulation network was later established on top of Manson Mesa in conjunction with the layout of Page townsite. This consisted of five additional quadrangles and one central point figure. The triangles were closed to third-order accuracy.

The existing vertical control of the damsite is based upon a line of levels run up the Colorado River in 1921 by the U. S. Coast and Geodetic Survey. The local precise net established by the Bureau of Reclamation was begun at one of these bench marks on the canyon floor about 2-1/2 miles upstream from the damsite.

The initial line was run downstream through the canyon to make a tie to the first Coast and Geodetic Survey bench mark below the damsite. Upon tying into this bench mark, a large discrepancy was found due to the bench mark having been disturbed. The levels were then continued downstream another 5 miles to a third bench mark where satisfactory closure was made. The equipment used to run the level net consisted of precise tilting levels, invar rods graduated in yards to the nearest hundredth of a yard, and steel pins 1 inch in diameter by 18 inches long for turning points. The three-wire method was used in reading the rod.

The primary level loop was formed by running levels from an established bench mark on the canyon floor, up the side canyon to the rim, along the canyon rim to the damsite, across the canyon, downstream along the east rim to a joint in the wall, then down the canyon wall to the river, and then back up the river to the point of the beginning. The levels were run to the west canyon rim by a circuitous route utilizing a series of rock shelves and benches in a side canyon. A route was chosen in advance of the level party and the turning points were established with a hand level. Turning points were made from 3/8-inch bolts grouted into the canyon wall with heads protruding about 1 inch. The levels were then run along the top rim to the west abutment of the damsite. The river crossing was made over the 1,200-foot-wide canyons by reciprocal leveling methods developed by the Coast and Geodetic Survey, as described in their special publication No. 239,[3] Manual of Geodetic Leveling. This method employs the use of two tilting levels with graduated micrometer screws on the tilting device.

The total primary loop was 17.4 miles in length and the error of closure to the original bench mark was 0.027 foot, an error of less than 0.002 foot per mile. Nine secondary control loops were also run for a total level loop distance of 36 miles. Standard brass disks were used for bench marks and were conveniently located along the primary loop and at points most useful for construction. A number of these bench marks were also set above the maximum lake elevation and back for some distance from the lake area to preserve the datum and for use in future settlement studies.

188. CONSTRUCTION SURVEYS The primary control network was tailored to the design and construction requirements for the project. Construction layout was based on two principal control features of the dam—the plane of centers and the radius of curvature of the axis of the dam. From these major references, a complex construction layout was achieved for the three-dimensional curves of the arch structure. The plane of centers, a vertical plane down the approximate center of the canyon, provides the directional base of the project. This base line was targeted on the canyon wall both upstream and downstream from the site. A 900-foot radius of curvature of the axis of the dam has its center point on the plane of centers, giving a starting position suitable for use in the basic layout system. The radius point was established by triangulation astride the plane of centers.

Original and final topography of the large keyways posed a problem in surveys due to the 700-foot-high, nearly vertical canyon wall. The original cross sections for the rock excavation were taken about 10 feet apart, both horizontally and vertically. It was necessary to include an area sufficient to cover all excavation for each of the features, a linear distance of about 2,500 feet down the canyon length. Sections in the foundations and keyway excavations of the dam were taken in a more detailed form than in some of the other areas. Using offset base lines parallel to the plane of centers, where practical, a series of normal sections were established on stations 10 feet apart along the very edge of both canyon rims.

The triangulation group was comprised of three teams. The actual transit team was composed of an instrumentman, a recorder, and a high-scaler to serve as rodman. The three transits were placed on the canyon rim opposite the side where the sections were to be taken. The end transits were deployed along the offset baselines at a distance which would give horizontal angles of approximately 45°. The center transit was placed on the offset baseline at the station where section was being taken and used as a constant 90° horizontal angle. The center transit alined the high-scaler on the opposite canyon wall to mark and number vertical stations at intervals of approximately 10 feet, using a small pressurized paint dispenser. The end transitmen read both horizontal and vertical angles at each numbered position as the high-scaler desended the wall. The center transit also read the vertical angle of the numbered position.

[3] U.S. Department of Commerce, Coast and Geodetic Survey, "Manual of Geodetic Leveling," Howard S. Rappleye, Special Publication 239.

Approximately 300 of these sections were taken for the original topography of the canyon wall, the majority of these being made down the entire height of the canyon wall. There were approximately 20,000 points surveyed by this method. A total of 32 man-days was required for reduction and listing of the notes of the triangulation of all 20,000 points as compared to an estimated 500 man-days of manual computation. The original notes were punched on electronic computer cards in only 20 man-days, following 2 days of programming in the Denver office. Computer time actually required 6 days and reporting time was 4 days. The original program was maintained on file and was used in the final cross-section tabulation which doubled the benefits of the program.

189. STRUCTURAL BEHAVIOR MEASUREMENTS. Provision was made for obtaining data periodically on the structural behavior of the dam, both during construction and after construction, as the reservoir becomes impounded. Some of these data will be obtained electrically by the use of strain and stress meters embedded at selected locations within the dam, but others on movement or deformation will also be obtained by survey methods.

Five plumbline wells are located along and near the upstream face of the dam, extending from approximately the foundation to the top of the dam. They are provided to detect tilting. An optical plummet was used to project the plumbline from the top to the bottom of these wells. Three foundation deformation wells are located near the toe of the dam. These points are accurately located and coordinated with permanent triangulation points established from the horizontal control net. Differences between initial and successive coordinate position of the points furnish measurements of horizontal movement resulting from foundation deformation.

Another system for determining the dam deformation consists of a grid pattern of 70 targets on the downstream face of the dam and 18 targets on the foundation along the abutments. On the dam, the targets are ranged as five equally spaced elevations between the top and a plane at about one-fourth the elevation of the dam. The targets are alined in 11 vertical planes. The target locations are charted periodically by observations from a system of eight theodolite piers downstream from the dam and two piers near the base of the dam, using precise triangulation methods. In addition to the exposed targets, three targets in observation wells were installed in the foundation of the dam. These targets are used for determining horizontal foundation deformation.

Selection of the target face patterns suitable for observations at the dam presented a problem due to local conditions. After a number of trial designs and field observations, two types of the acceptable target were used, one having a full rim body for headon observation, and the other having a body with a cutaway rim for offside observation. The 3-1/4-inch outside-diameter bodies were finished in a baked-on low-gloss black enamel. Two types of sighting points were also chosen. One has a 3/8-inch-diameter cylinder with alternate red and white enamel bands, and the other a 3/8-inch-diameter white ball mounted on the end of a threaded stud. Both sighting points are attached to the target bodies by screw threads. The targets are attached to the dam and foundation rock by the threaded target body that screwed into pipe couplings attached to short lengths of 1-1/2-inch-diameter embedded pipe.

Following accumulation of initial data on the target location, observations made on the targets are anticipated at 50-foot increments of increasing reservoir elevation. After the reservoir elevation reaches about two-thirds the height of the dam, observations are anticipated on a quarterly basis until normal reservoir operations are achieved. After that time, the observations will probably be continued on a quarterly or semiannual basis for at least one or two cycles of reservoir operations. Data will include sets of observations in the summer and winter during periods of maximum and minimum monthly air temperature. The reduction of the observed data will be carried out by automatic data processing methods in the Denver office. Computer programs are available for reducing the target observation data and for adjusting the triangulation data pertaining to the primary and secondary theodolite stations.

In addition, there are invar reading stations in four abutment tunnels for measurement of foundation rock deformation (Subsec. 38(f)). One set is located each in the right and left abutment tunnels of the dam at elevations 3630 and 3480. Each tape set is 225 feet in length, measured from a zero station near the concrete abutment of the dam, with intermediate stations at 100, 75, and 50 feet. Periodic readings are taken, coordinated with water surface elevations, and reported to the Denver office for evaluation.

Further details on surveys at Glen Canyon Dam are included in the reference paper.[4]

[4] Fink, H. L., "Survey Control at Glen Canyon Dam," Journal of the Surveying and Mapping Division, Proc. A.S.C.E., vol. 88, No. SU1, November 1962.

C. OTHER FEATURES IN THE DAM

190. PARAPETS AND TRASHRACKS. Placement of parapets on the dam started shortly after the final placement of mass concrete in each block except for those left out on the upstream side while "monkey slides" were being used for trashrack structure placements. These sections were placed after the trashrack and gate hoist structures were completed. The concrete approach slab, with parapets, was placed on the left abutment adjacent to block 1 in October 1963.

The elevator shafts in blocks 8 and 17 were placed with the dam to elevation 3680.9 and the elevator towers were separate placements from that point to the roofs at elevation 3750.1. Stairs were placed in the elevator shafts.

Placement of the cantilever sections of the trashrack structures was started in May 1962, and placement of the trashracks followed. Each trashrack structure consisted of 19 placements, a marginal beam bracket, and a cover slab. Access to the work was at first by ladders and catwalks; and work was suspended January 15, 1963, because of ice on the catwalks causing hazardous working conditions. Work was resumed in June 1963, in blocks 6 and 12. Monkey slides were installed on the upstream face of the dam in each block which contained a trashrack structure and in blocks 5 and 6 for the outlet pipe trashrack structures. All trashrack structures including marginal beam brackets were completed in November 1963. The cover slabs and marginal beams were precast and placed on the trashracks.

Metal trashracks were placed on the river outlet trashrack structures in June 1963, and this work was completed for the penstock trashrack structures in June 1964.

Placement of the gate hoist structures for the penstock fixed-wheel gates started in November 1963, and was completed in February. The structures were made in 10 placements from elevation 3652.30 to elevation 3724.58. Trashrack guide extensions, stoplog guides, and fixed-wheel gate guides were in the eight blocks containing penstocks. Vista canopies were placed adjacent to the elevator towers in blocks 9 and 16.

191. SURFACE DRAINAGE GUTTER. Constructed early in 1963, a surface drainage gutter runs along the downstream face of the dam just below the compacted backfill which is at elevation 3157.0. The invert of the gutter is 2 feet wide and it runs left and right of a stilling basin in block 12. The invert of the stilling basin is at elevation 3153.00 and the gutter is at elevation 3155.50 at that point. A 30-inch corrugated metal pipe drain brings the water from the pump chamber to the stilling basin, and the left branch of the gutter carries it to block 7 and then along the service bay to a 30-inch corrugated metal pipe in a drop inlet at the A-line of the powerplant. From there, the water flows through the machine shop bay to a 'flap valve in the left training wall which discharges into the tailrace. The right branch of the gutter carries water along the face of the dam to block 21 and into a 30-inch corrugated-metal-pipe drain in a drop inlet near the intersection of the 1- and B-lines of the powerplant. The 30-inch drain carries the water through a flap valve in the right training wall to the tailrace. Behind each of the flap valves is a 30-inch gate valve. The flap valves are at elevation 3146.0 and, in case the tailrace becomes too high for surface drainage, the 30-inch gate valves can be closed and the water discharged into the powerplant sump through an 8-inch emergency bypass from a point in the 30-inch corrugated-metal pipe in the machine shop to the powerplant sump. An 8-inch gate valve is located on the emergency bypass line in the powerplant sump.

192. ELEVATORS IN DAM. The elevators were supplied and installed by Pacific Elevator and Equipment Co. of San Francisco, Calif., under specifications No. DS-8443. Installation was started on the elevator in block 8 as soon as the hoistway and machinery room were made available to the contractor on March 23, 1963, and work under the contract was accepted as substantially complete December 26, 1964. Completion date of the contract was extended by 177 days to cover delay to the contractor caused by failure of the Government to make the elevator hoistways and machinery rooms available as specified. Cost of furnishing and installing the two elevators in the dam was $284,919, which included $5,919 for extra work and materials provided by the contractor. Invitation No. DS-8443 also provided for furnishing and installing two elevators in the powerplant, which is discussed separately in section 217.

193. GANTRY CRANE. The 165-ton outdoor gantry crane operates across the top of the dam from the left abutment to near the right abutment. The crane is used for installation and maintenance of penstock gates, hoists, stoplogs, and river outlet bulkhead gates. After difficulties were experienced with the main hoist gearing on the 300-ton powerplant cranes (sec. 227), the supplier of the 165-ton crane was required to check and repair the main hoist gearing in the shop. This reduced the field repair at the

construction site and was the least costly to the suppliers of the equipment. The material was received in Flagstaff during April of 1963; it was found that some minor shipping damage had occurred, but this was repaired at no cost to the Government during the erection of the crane by the prime contractor.

The erection of the 165-ton crane was performed by Merritt-Chapman and Scott Corp. under specifications No. DC-4825; erection was started in March 1963 in the prime contractor's fabrication yard. Installation of the crane rails began in July 1963 in block 1 of the dam; all rail had been laid and blockout concrete placed in October. The crane was erected on block 3 of the dam.

The structural steel was erected by the ironworker crew of Merritt-Chapman and Scott Corp., and the cleanup, adjustments, and installation of mechanical equipment was performed by their crew of millwrights. The erection engineer arrived at the installation site on November 4, 1963. The erection engineer, working with a qualified crew, did not have any problems of note on the mechanical portion of the erection. Counterweights were placed, the crane was painted, and the crane moved to the left abutment for testing. The formal load tests were conducted starting on November 14 and were completed November 18, 1963.

194. SERVICE ROADS AND BRIDGE. Excavation for the right abutment service road to the dam was started in February 1964, and the right abutment service road bridge was completed in April 1964. Excavation for the left abutment service road to the dam was started in April 1964. The service roads and parking areas were completed, including bituminous surfacing, in September 1964. Installation of guard rail followed. Gates were installed by the contractor at the entrances to the spillway intake structures, and Government forces installed a gate near the entrance to each abutment service road.

195. TERRAZZO AND TILE. Areas that would be toured by the general public were designated to receive terrazzo floor finishes, structural wall tile, or ceramic wall tile in the dam. Marus Marble and Tile Co. of Greensboro, N.C., was awarded a contract under specifications No. DC-6057 to place 1-1/2-inch bonded concrete underbed on stairs and landings in the elevator towers of the dam between elevations 3697.77 and 3734.02; to place thin bonded terrazzo finish on the floors in the right abutment adits to the powerplant, including lobbies, and on the stairs, stair landings, lobbies, and restroom floors of the elevator towers in the right abutments adits to the powerplant, including lobbies, and on the stairs, stair landings, lobbies, and restroom floors of the elevator towers in lobbies; to install ceramic wall tile in the restrooms located in the elevator towers between elevations 3697.97 and 3734.02; and to place terrazzo floor and install ceramic tile at various locations in the powerplant. This last item is discussed separately under a section on the powerplant.

Start of work under specifications No. DC-6057 was delayed from April 21, 1964 (the date the contractor received notice to proceed) until September 28, 1964, when the contractor was able to start work. This delay was caused by failure of the Government to provide the floor areas to allow orderly performance of the required terrazzo work. The structural and ceramic wall tile was supported on the terrazzo base coves and could not be installed until after the terrazzo floors were placed. By a findings of fact, an extension of time of 227 days was allowed the contractor for completion of his contract. All work was completed April 23, 1965, under specifications No. DC-6057, with a cost of $53,500 allotted for the dam.

196. LOG BOOM. A 1,228-foot-long log boom, with a 15-foot boat passage near the middle, was constructed across the reservoir approximately 1,500 feet upstream from the dam. This log boom serves the purpose of intercepting floating debris before it reaches the dam and aids in control of boats on the reservoir. The boom consists of 40-foot sections constructed of pairs of logs fastened together side by side. The sections are fastened together with lengths of 1-inch wrought iron log chain, and the boom is anchored to the canyon wall at each end. Five anchor points were constructed in each canyon wall spaced between elevations 3520 and 3703. The anchors consist of a 1-1/4- by 12- by 24-inch iron plate, provided with a ring, mounted against a vertical concrete pad. The plate is secured to the canyon wall with two 1-inch-diameter by 10-foot-long bolts grouted into the rock.

The log boom was constructed at a cost of $34,874 by R. H. Jackson and Associates of Golden, Colo., under specifications No. 400C-291. Work was started June 23, 1965, and all work was completed October 25, 1965. Two orders for changes were processed which increased the amount of anchor chain, increased the thickness of the anchor plates, deleted the crack-free requirement on four logs in the boat passage section, and extended the time for completion of the contract by 28 calendar days.

197. PLAQUES. A contract was awarded to Alton Iron Works of Albertson, N. Y., under invitation No. (D) 45-1113-A, for furnishing and installing two plaques at Glen Canyon Dam, along with plaques at other locations. Both plaques were fabricated from sheet aluminum and were machined and polished at the contractor's Albertson, N. Y., plant and were anodized by Perma Plating Co., Brooklyn, N. Y. Both plaques were 7 feet 6 inches in diameter and were mounted one each on the two elevator towers. The one mounted on the block 17 tower was the Great Seal of the United States and the one mounted on the block 8 tower was the U.S. Department of the Interior seal. The plaques were installed on January 26, 1967, by Young Electric Co. of Salt Lake City, Utah, representing Alton Iron Works. Cost of the two plaques was $7,000.

D. SPILLWAYS

198. GENERAL. The spillways (fig. 101) are designed to discharge, at reservoir elevation 3711, 276,000 cubic feet per second. By passing an additional 15,000 cubic feet per second through the river outlets and 9,000 cubic feet per second through four units of the powerplant, a total flood release of 300,000 cubic feet per second can be obtained. The entrances for the spillways are located about 600 feet upstream from the dam, and each consists of an unlined approach channel and reinforced concrete crest structure with piers. Each crest is set at elevation 3648, and discharge through the tunnel is controlled by two 40- by 52.5-foot structural steel radial gates with counterweights.

The spillway tunnels for the greater part of their length are 41 feet in diameter. The transition section downstream from the intake structure changes from a flat-arch-roof section 89 feet wide by 52 feet high to a circular section 48 feet 3 inches in diameter. From this point there is further transition of the circular section to the 41-foot-diameter tunnel. The tunnels are designed to flow partially full or at never more than 0.7 of the height.

199. EXCAVATION. Drilling equipment was moved to the right spillway intake area in July 1957, and excavation began. Excavation in the left spillway intake was started in October 1957. Excavation in opencut in both spillway intakes was completed in April 1958, and excavation for the spillway tunnels was started from the top.

At the same time, spillway raises were started in the diversion tunnels. The right spillway raise was a 7- by 14-foot pilot hole which was holed through at 585 feet on July 1, 1958. The left spillway raise, an 8- by 9-foot pilot hole, was pushed from the bottom for 550 feet and then holed through from the top. The length of this pilot hole was 567 feet. Excavated material from spillway tunnel excavation was then dropped through the raises into hoppers and discharged directly into trucks. Excavation at the upper portals of the spillway tunnels was completed to the point where both were ready for arch steel in May 1958.

The heading of the right tunnel reached elevation 3476 during the second week of January 1959, when work was suspended because of interference with diversion through the right tunnel. Forty-two sets of arch steel had been placed at the time.

The heading of the left tunnel reached elevation 3220 early in April 1959 and remained at this elevation until after the river closures. Meanwhile, tights were removed and preparations were underway for lining of the left tunnel beginning at elevation 3224 and working up. A large jumbo was erected for use in scaling and removing tights. Track was laid and the jumbo lowered into the tunnel. Reinforcing steel and drainage pipe were being placed for the tunnel lining at the time of the strike on July 6, 1959. Track was also being laid in the right tunnel for the lowering of men and equipment for resumption of drilling at the time of the strike.

Preparation for resumption of excavation of the right spillway tunnel was started in June 1960. A 20- by 20-foot disposal or haul road tunnel was driven from the intake channel to the canyon wall at elevation 3635 so that the excavated material, after being raised from the heading, could be pushed through by a bulldozer and dropped to the bottom.

Two skips running on tracks were used to bring the muck out of the tunnel and were dumped from a ramp into trucks which hauled the excavated material to the tunnel. It was pushed out of the tunnel by a bulldozer and fell down the canyon wall. Excavation was started the latter part of August and was completed in January 1961, except for the lower elbow section.

200. CONCRETE CONSTRUCTION. Following the strike, work was resumed in the left spillway tunnel on January 4, 1960. Reinforcing steel was placed beginning at elevation 3224, and drainage and grout pipe were installed. Reinforcing steel and grout and drainpipe were placed to elevation 3525, station 21+58, and the first concrete lining was placed the

week of April 18, 1960. An attempt was first made to use the gravity flow system, but mechanical difficulties were encountered and this system was abandoned and all lining was placed by pumpcreting. Placements 1a, 2a, and 3a were made and the special concrete finish applied in the elbow sections. The form was replaced by a straight-section form and placements 1b and 2b made to station 22+96.96, elevation 3328.4. On July 16, 1960, a hoist cable failed while the form was being pulled up for placement 3b. The form was dropped about 50 feet and wedged itself into the concrete elbow lining already placed.

Considerable time was lost in freeing the form, raising it to the level of the 3b placement and repairing it. New ribs and pans had to be ordered and delivery was not made until September. Placement 3b was finally made the week of November 6, 1960. At the end of 1960 the lining had progressed to elevation 3391.14, station 22+53.05. Lining was resumed in January 1961, with placement 3c and continued through placement 13c, station 21+05.04, elevation 3594.18, which was made in August. From this point on, the invert was lined first with placement 1d being made in September and continuing through placement 4d in October to station 20+45, elevation 3638.12. A porous concrete pad was placed under the 8-inch drains in the intake area, and 18 B-holes were grouted in the crest area.

The first walls and pier sections were then placed from rock to elevation 3635 for the spillway intake structure. Placement of the intake structure was suspended because of severe cold weather on December 11, 1961, when the right and left walls were at elevation 3672.5 and the pier at elevation 3657.5. Work was resumed in February when the weather moderated, and lining of the tunnel and placement of the intake structure were completed in June 1962 except for the unexcavated cap in the elbow section just above the left diversion tunnel.

Pedestal bases for the radial gates were then grouted in place and the four pedestals placed. Sillplates were grouted in place and the counterweights placed in the wells. Erection of radial gates 1 and 2, installation of wallplates, and installation of hoist equipment on the bridge deck were then started. Completion was in January 1962, and the radial gates were tested satisfactorily and accepted for operation in March.

A cofferdam was constructed in August 1962 downstream of the outlet channel of the left spillway, and the area was dewatered in preparation for construction of the flip bucket. Air and water lines were run from the spillway outlet to the plug section of the left diversion tunnel in preparation for placing the tunnel plug. Excavation of rock and concrete was started in September in the outlet channel of the left spillway, and the first placement for the deflector bucket was made in October. This work was completed in January 1963. Concrete was hauled in a 20-ton truck in which a chute had been installed. The concrete was dumped into a 4-cubic-yard bucket and moved to the forms by a mobile crane.

During installation of anchor bars for the left deflector bucket, it was found that large stress relief joints or voids existed behind the protective wall in the outlet channel between stations 37+10 and 37+30 and between stations 37+65 and 37+96. Grout or vent holes were drilled horizontally in these two areas at an elevation about 1 foot below the bottom of the anchor bar hole. Prior to grouting in the anchor bars, grout was injected into the new holes, drilled below the row of anchor bars, at a pressure of 10 pounds per square inch, until grout returned from the new grout holes drilled below the next higher row of anchor bars. The combination of anchor bar and grout holes served as vents to preclude any possibility of developing excessive pressures. In November the pressure was increased to 20 pounds per square inch, which did a better job of grouting of the anchor bars. The voids were grouted between elevations 3144 and 3187.

Preparation for construction of the right deflector bucket began in the latter part of July 1963, with the drilling of holes for anchor bars in the concrete protective wall. However, this work was suspended until completion of the lining of the elbow section of the right spillway tunnel. Work on the right deflector bucket was resumed in December 1963, and the final placement was made early in February 1964.

Preparations for lining of the right spillway tunnel were started in February 1961, when the access road to the intake channel was reconstructed to permit passage of cars and trucks. Tights were removed from the floor of the intake channel, and the laying of tracks was started. Part of the wall between the intake channel and the canyon was removed and work continued through June when it was suspended. Preparations for lining were resumed in September when surveys were made at the lower end of the tunnel excavation and assembly of the form for placement 1a was started. Work was again suspended during the week of October 15, 1961, when the form for placement 1a was nearly complete, and the contractor decided to delay start of lining of the right tunnel until all concrete had been placed in the crests and piers of the left spillway intake. The first two lining placements were made in August 1962, from station 24+17.67 to 23+82.04, invert elevation 3196.90 to elevation 3224.33. A special concrete finish was applied to the lining through the elbow section as required by the specifications. A whirley crane was used to handle material and concrete for the lining and the intake structure, and it was placed on the rock ledge above the left side of the intake structure. In order to reach the right wall of the intake structure, it was necessary to extend the tracks. The left wall of the intake was started in October 1962, and was completed in February 1963. The tracks of the whirley crane were extended over the left wall of the intake structure in March 1963, and the right wall of the structure was started in that month. Meanwhile, the lining of the upper spillway tunnel was progressing and this work was completed in October 1963.

The lower elbow section of the right spillway tunnel was excavated in March and April 1963. The first lining placement was made in the invert from station 26+11.72 to station 25+85.50 in September 1963. Two invert sections were lined and then one arch lining placement was made at the lower end of the elbow section. After four invert sections were placed, followed by the four arch lining placements, the remaining four placements were made by placing the entire circumference of the arch at one time. This work was completed in November 1963.

The concrete work for the intake structure of the right spillway was completed in November 1963. The trunnion bases for the radial gates were placed, and assembly of the gates was started on the ledge above the intake.

Hoist equipment was installed on the bridge of the intake structure, counterweights were installed, and the gates erected. Radial gate No. 3 was satisfactorily tested on February 27, 1964, and gate No. 4 was satisfactorily tested on March 10, 1964. Painting of all gates was completed in August 1964.

201. RADIAL GATES. The four 40- by 52.5-foot radial gates were furnished under invitation No. DS-5192 and manufactured by Vereinigte Osterreichische Eisen and Stahlwerke A.G. (VOEST). In the field erection, the workmanship was considered very good. Very minor drifting was required for assembly and no misplaced holes were found. A small omission was noted on the gate arms in that drain holes were not drilled and the holes for guide shoes on top of gates No. 3 and 4 were not drilled. This was a very small percentage of the holes required and the installation contractor made corrections in the field.

Merritt-Chapman and Scott Corp. proposed several changes in the erection procedure shown on figure 268. The prime contractor had well-qualified survey crews and engineering supervision with recent experience on erection of large radial gates. As the erection procedure allowed some latitude in the method of erection, the contractor was allowed to proceed with erection of the first gate, subject to acceptable installation. The contractor's erection methods were sound and all operating checks were satisfactory. The remaining gates were also assembled without unusual difficulties and it was not necessary to use the final adjustment of babbitted joint allowed by design.

The radial gate hoist is a two-drum electric-motor-driven mechanical hoist located on the radial gate operating deck. Two 1-1/2-inch stainless steel wire ropes from each hoist drum are connected to lifting brackets near the bottom of the gate. The four gate hoists installed on the left and right spillways were furnished by Moffett Engineering, Inc., of Berkeley, Calif., for a contract price of $104,000. The four radial gate hoists were installed by the prime contractor in the fall of 1962, and were accepted for operation on March 10, 1964. The hoists were placed on embedded anchor bolts and, by use of double nuts, the drum units and gear cases were alined and leveled. A transit was used for alinement and a precise machinist level was utilized on line shafting joining the drum units and gear cases. The various components and gearing were checked with the hoists running by using dial indicators. No significant difficulties were encountered and good operation of the hoists was obtained.

E. PENSTOCKS AND OUTLET WORKS

202. PENSTOCKS. Water for power generation is delivered to the turbines through eight penstocks (figs. 129 and 130) having a common intake certerline elevation of 3470 feet, which is 245 feet below the crest and 330 feet above the turbine level. They descend through the dam on a 60° incline, bend at the base on a 60-foot radius, and cross over to the powerplant on concrete piers (fig. 269). Centerline length ranges from 406 feet in an outer unit to 449 feet in a center unit; the steel shell thickness ranges from 7/8 inch in the upper sections to 1-13/16 inches in the lower sections. The diameter of the upper, inclined portion is 15 feet, reducing to 14 feet for the lower, horizontal section.

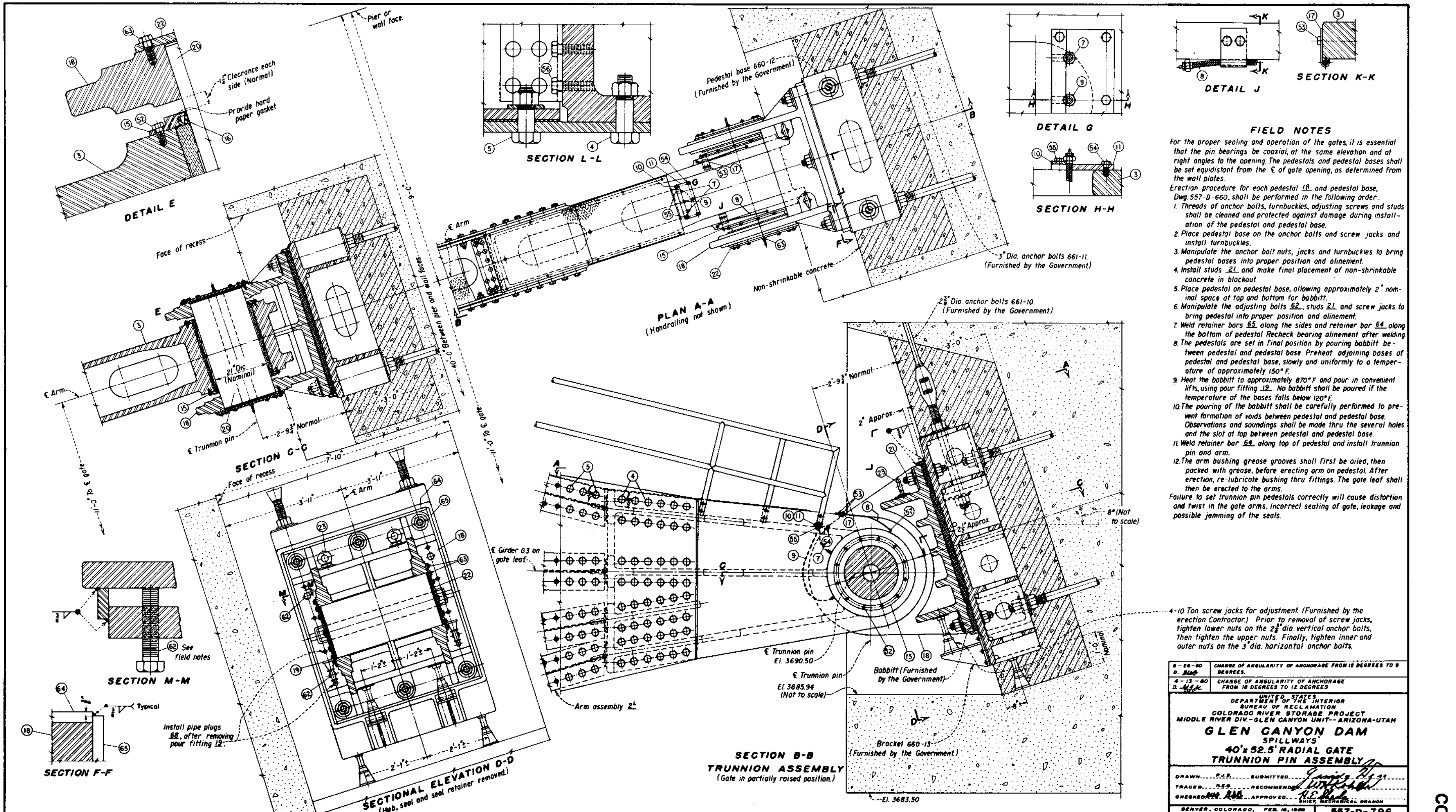


Figure 268.—40 by 52.5-foot radial gate for spillway—Trunnion pin assembly.

The penstocks intakes are protected from ice and debris by massive 200-foot-high trashrack structures (fig. 270). The lower intake section contains six reinforced concrete bays for an overall height of 75 feet. Each bay is a 15-foot inside-diameter, semicircular structure composed of five chords. Each chord constitutes a trashrack opening of about 9 by 12 feet, which is covered and protected by framed metalwork.

Closure of the penstock intakes is accomplished by stoplogs and fixed-wheel gates. The respective sets of stoplogs are spaced several feet in front of the gates and have a total vertical height of 250 feet. Each set is divided into four groups: 9 lower logs, 7 intermediate logs, 7 top logs, and a final upper section of 20 smaller logs. The eight fixed-wheel gates cover only the intake bell of the penstocks. Each gate is 13.96 by 22.45 feet and weighs about 115 tons. Closure control is provided by a large hydraulic hoist. All of the mechanical features along the face of the dam are serviced by a 165-ton gantry crane.

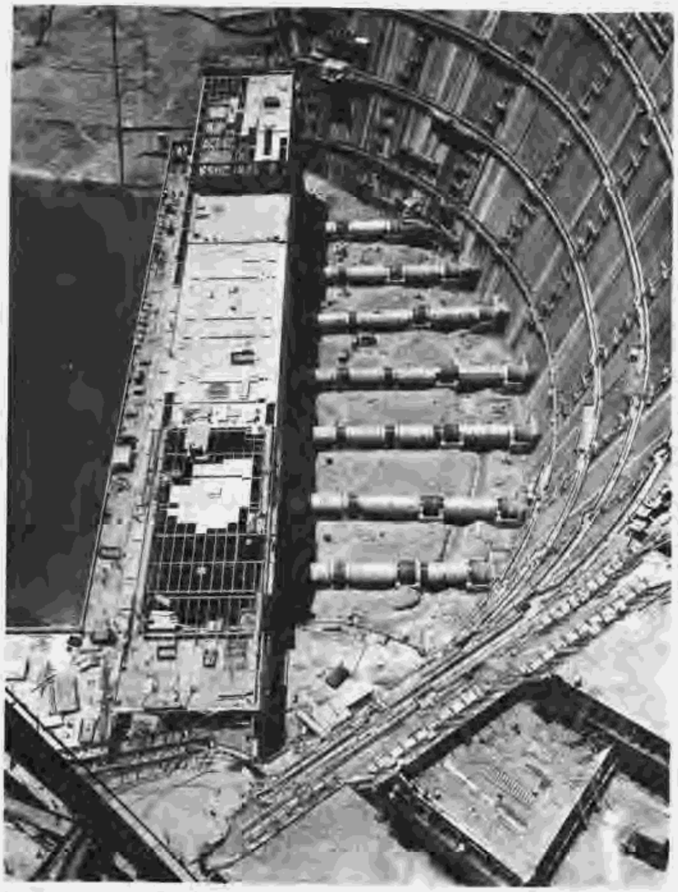


Figure 269.—View looking down on Glen Canyon Dam and Powerplant. Note penstock installation between dam and powerplant. P557-420-7212, July 5, 1962.

Figure 271 shows the penstock transition at the intake of penstock No. 2 at the face of the dam in block 16.

203. PENSTOCK INSTALLATION. The eight penstocks, referred to as No. 1 through 8, are located in dam blocks 18, 16, 15, 13, 12, 10, 9, and 7, respectively. Installation of the penstocks was subcontracted to Chicago Bridge and Iron by the prime contractor, Merritt-Chapman and Scott Corp. The penstocks were furnished by Vinnell Steel of Irwindale, Calif., under invitation No. DS-5052. Initial fabrication was performed at the Irwindale plant. From Irwindale, 10-foot sections, or cans, were shipped by rail to a Flagstaff, Ariz. plant which was established by Vinnell to complete the fabrication (fig. 272). At Flagstaff, the cans were welded into 30-foot sections. Before being shipped to the jobsite by truck, the welds were X-rayed and the sections were placed in an oven for relieving stresses from the welding and were hydrostatically tested. Installation began in December of 1960, with the lower horizontal sections to be embedded in the mass concrete of the dam. The mass concrete was at elevation 3127.5 when the sections were set in place.

During the initial installation of the horizontal and the lower vertical bend sections to be embedded in the máss concrete of the dam, a framework of angle iron was used to support these sections to line and grade until the girth seams could be welded. This supporting framework had been anchored in concrete before any of the penstock sections were brought to the installation site. For extra stability and protection against misalinement by possible concrete bucket collisions with the penstocks, this framework was embedded in the concrete along with the penstock sections. On sections installed on the incline, no supports were used other than the key plates holding two sections together and the actual welded joints as the work progressed. The sections along the incline were for the most part 20 feet in length and installed at a 60° angle from the horizontal plane. In general, no more than one penstock section and a portion of another section would be exposed above the mass concrete. In rare cases when the work was slack for the welders, there would be two complete penstock

Figure 270.—View of upstream face of Glen Canyon Dam showing trashrack construction. Trashrack structure No. 8 is in foreground. P557-420-9794NA, May 27, 1964.

Figure 271.—Forms and reinforcement steel for the penstock transition concrete and trashracks being installed in block 16 on penstock 2. P557-420-7365, July 27, 1962.

sections and a portion of a third section exposed. Figure 273 shows some inclined penstock sections being installed in mass concrete.

All welding performed on the penstock installation was done in accordance with the requirements of the 1951 edition of the "American Petroleum Institute–American Society of Mechanical Engineers (API–ASME) Code for Design, Construction, Inspection and Repair of Unfired Pressure Vessels for Petroleum Liquid and Gasses." All welders doing this work were required to be qualified in accordance with the above code. All the field welded girth joints, except for the closing joints, were radiographed for the entire length of the joint. Defects disclosed by the radiographic tests were repaired in accordance with the code and again inspected by radiograph.

Welding of the penstock closing joints across the A-B contraction joint was not accomplished until all joint movement had subsided. A collar which was 6 inches wide and made from 1/2-inch plate, was welded on the upstream edge of the downstream penstock section. The upper 3 inches of this collar extended beyond the downstream penstock section, forming a sleeve in which the upstream penstock section was merely slipped in and set to line and grade. An asbestos-felt cover was wrapped around the penstock 5 feet on either side of the joint. Grout connections were furnished on the inside of the upstream penstock sections and after the joint was welded from the inside, grout was pumped through the connections to fill any voids. After alinement, the key plates and alinement bars were removed.

The last penstock section to be placed near the intake was fabricated with a dish-shaped bulkhead at the upstream end. This bulkhead was not removed until the fixed-wheel gates were operable and in place, sealing the water passage. Removal of the bulkheads was started by the completion contractor during July 1964. Removal continued as turbine erection

Figure 272.—First of the penstock sections arriving at the damsite. These penstock sections are 15 feet in diameter and 20 feet long. The sections were shipped by rail from California to Flagstaff, Ariz., and trucked to the damsite. P557-420-5052, July 9, 1960.

progressed, with the last bulkhead in penstock No. 8 being removed in April 1965. This upper section contains a 30-inch air vent pipe through which air can be admitted to the penstock during unwatering and expelled during filling. Also, in this section is a 24-inch manhole to provide access to the interior of the penstock immediately downstream from the bellmouth.

The transition forming the bellmouth intake section between the penstock and the intake structure was shaped by a fabricated wooden form which was attached to the upstream end of the penstock section. All of the penstock sections to be welded in place were completed in August 1962, and the forms for all penstock intake transition sections had been placed by the end of September of that year.

Early in June 1962, the contractor started placing the horizontal sections of the penstocks between the dam and the powerplant. These sections were hauled through the powerplant access tunnel to the service bay and lowered from there into the penstock support area by use of the two 50-ton highline cableways. When the penstocks were set on the saddles of the penstock supports, it was found necessary to chip and grind the saddles in order to eliminate pipe distortion and to secure satisfactory connections. This was caused by the allowed tolerances on the radii of the saddles and the outside diameter of the penstock. The saddles were placed at the minimum tolerance and the penstocks fabricated and wrapped at the maximum outside diameter permitted. Each of these sections to be buried in a select backfill material were wrapped.

Installation of the expansion joints and the sleeve couplings in the backfill area between the dam and powerplant created a few problems. These problems were mostly caused by improper storage both on and off the jobsite. The expansion joint components were stored for periods ranging from 8 months to 2 years 8 months in temperatures ranging between 18° F. below zero and 160° F. metal temperature. This probably resulted in the sleeves and shells taking a more or less permanent set, loss of roundness in some sections, and loss of wax lubricant in the 1-1/4-inch flax packing material. Portions of the flax packing had also been exposed to blowing sand just prior to installation. The penstock sections were stored for an average time of 19 months. The ends of the penstock, where an out-of-round condition existed, were reshaped by jacking 6-inch pipe with ratchet bottle jacks at points determined by inside measurements. With the ends under jacking operations, the expansion couplings were installed and bolted. Bolting was accomplished in the following manner. Two impact wrenches were used 100° apart to snug up the nuts, with the distance from the flange of the follower ring to the outer edge of the shell being measured frequently to keep the flange parallel to the end of the shell. The packing was compressed by this method from 3/8 to 1/2 inch. No attempt was made to compress the packing fully. This was left to be done if required when the penstock was filled with water.

The procedure for tightening the sleeve-type couplings was done in the following manner:

First, the torque wrench was set at 150 foot-pounds and all bolts were progressively tightened until most of the slack had been taken up. This would take two or three passes, and it was discovered after a complete pass that the first bolts had very little torque remaining. For simplicity the torque remaining on the bolts will be referred to as "remaining torque" throughout this discussion.

Second, after the remaining torque had increased to 40 or 50 foot-pounds, the wrench setting was reduced to 110 foot-pounds and two or

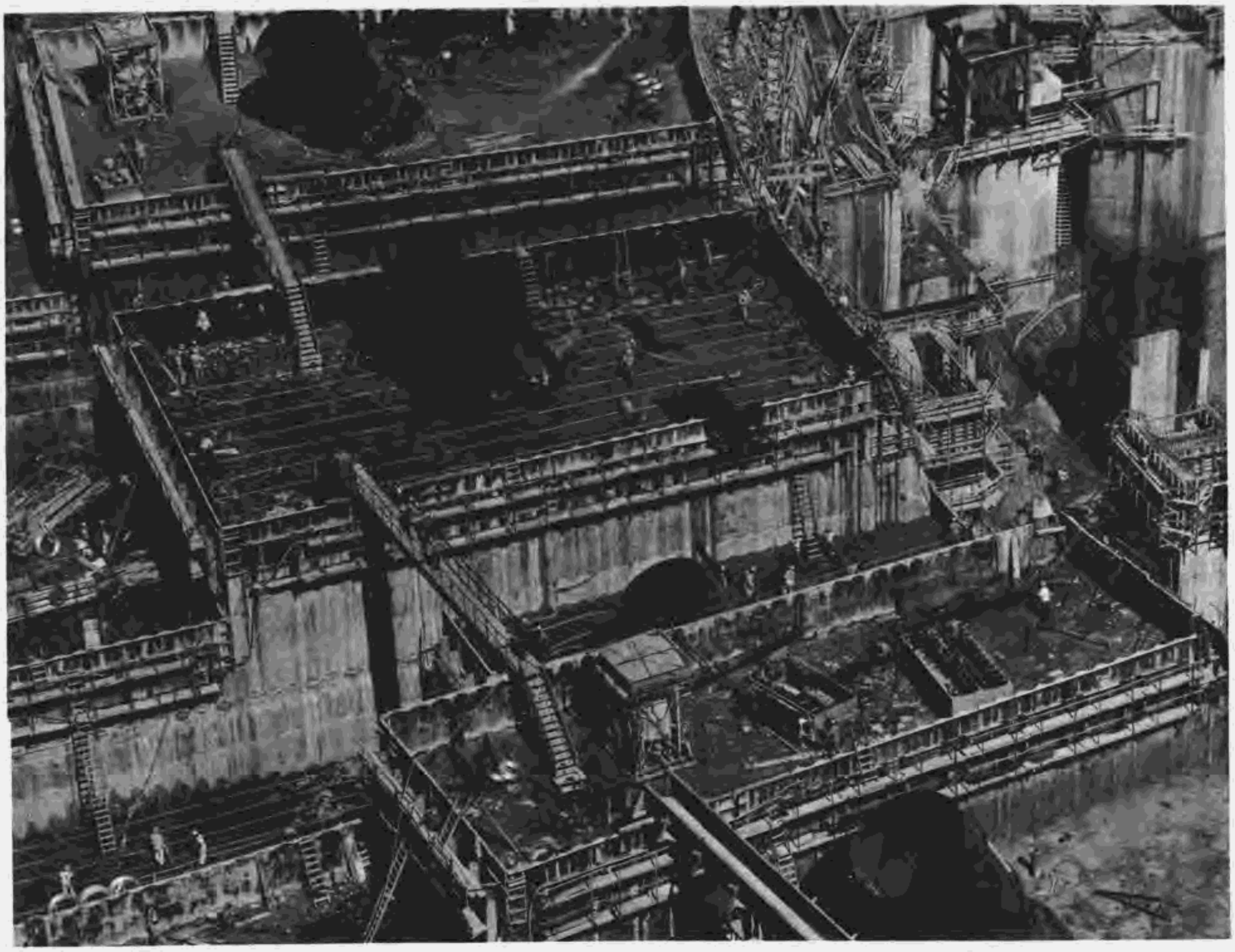


Figure 273.—View looking down from transfer trestle at blocks 14, 13, 12, 11, and 10, penstocks 3, 4, 5, and 6. Numbers run from bottom of photograph in the order listed. Block 10 at top of photograph is being sandblasted; block 12 in center has cooling tubes installed preparatory to concrete placement. P557-420-5931, May 2, 1961.

three more passes made until the remaining torque had increased to 60 or 70 foot-pounds.

Third, the torque wrench setting was reduced to 90 foot-pounds and two or three more passes were made. These passes were different than the passes made previously. After three-fourths of each pass was made, the torque was reduced to 80 foot-pounds and the remaining one-fourth of the pass was torqued at 80 foot-pounds. This was repeated until the remaining torque had increased to 80 foot-pounds.

Fourth, the torque was spot checked at 80 foot-pounds and the few bolts that were deficient in torque were within 10 foot-pounds of the desired value.

Last, after 5 to 7 days the torque was again checked and all bolts were brought to 80 foot-pounds.

In addition, owing to the fact that the penstock sections are not perfectly alined nor perfectly round, the spacing between the coupling ring and gland ranged from 3/8 inch to 1-3/16 inches. Because of this condition, it was felt necessary to put 1/2-inch shims between the ring and the gland when spacing became less than 1/2 inch, to prevent the rubber gasket from being completely forced under the anchor ring. After the required torque had been obtained, the overall width of the couplings measured close to 26 inches, plus or minus 1/4 inch.

204. PENSTOCK FIXED-WHEEL GATES. The 13.96 by 22.45-foot fixed-wheel gates were furnished by Vereinigte Ostereichische Eisen und Stahlwerke Aktiengesell Schaft of Lenz-Donau, Austria, under invitation No. DS-5577. The frames and anchors were fabricated by Rockwell Engineering Co. under invitation No. DS-5469, and the hoists and stems by Pacific Coast Engineering under invitation No. DS-5601. The control cabinets were furnished by Auto-Control Laboratories under invitation No. (D) 190,638-A. The gates and related equipment were installed by the prime contractor under specifications No. DC-4825 (fig. 274).

Figure 274.—Gantry crane lowering 120-ton fixed-wheel gate into the tracks of penstock No. 8. P557-420-9711, April 20, 1964.

The fixed-wheel gates, hoist, and controls are located in the penstock trashrack structure of each of the eight penstocks and are used for emergency closures of the penstock and for maintenance purposes. The eight 13.96- by 22.45-foot fixed-wheel gates are installed on the upstream face of the dam. Each gate is connected through a series of intermediate stems to a hydraulic hoist located at the top of the trashrack structure. The control cabinets are installed adjacent to the hoist. Each fixed-wheel gate is a flat structural steel assembly with brass-clad rubber seals mounted on the downstream skinplate. The seals prevent leakage by contacting seal seats embedded in the concrete on the downstream side of the gate slot. Wheels at each side of the gate bear on tracks, also embedded in the concrete on the downstream side of the gate slot. Guide rollers mounted on each side of the gate bear against guides embedded in the concrete of the gate slot. The nominal gate travel is 26 feet 5 inches.

The hoists are an oil-operated, cylinder type and are installed on the slope of the intake structure. The hoists are supported on steel support beams. Each hoist piston has packing and piston rings, is 30 inches in diameter and has a stroke of 26 feet 5 inches. A series of intermediate stems with an assembled length of 172 feet 9-3/4 inches connects the piston stem to the gate stem.

The fixed-wheel gates were assembled at the paint yard, approximately one-fourth of a mile from the left abutment of the dam. Two concrete pedestals were constructed for each gate and were placed in two lifts with 4- by 4- by 1/2-inch angle stubs embedded in the bottom lift. These stubs were capped with the plate accurately leveled and the second lift was then placed.

The gate body for penstock No. 8 was assembled first on the pedestals nearest the dam, and the remainder of the gates were assembled in reverse numerical order. Spaces between the various gates were sufficient to permit installation of the splice plates as well as the fixed wheels and their covers. Access to the upstream and downstream faces of the gate was obtained by use of easily assembled tubular scaffolds. Planks of sufficient length to reach from one scaffold to the next one provided access to the sides of the gate.

When the four sections comprising each gate were stacked and pinned together, a preliminary check of the seal bases for being "in plane" was made. It was not found necessary to change the position of any of the sections after thorough pinning and bolting. The connecting plates were installed, and after all bolting was accomplished, the gates were plumbed to within one-fourth inch of vertical. Stainless steel piano wire, 0.018 inch in diameter, was drawn taut over accurately machined triangular-shaped blocks by means of small turnbuckles. All measurements were made on the seal bases. These piano wires were also used to set the fixed wheels to the required projection.

Before the fixed wheels were installed and after the roller guide assemblies and seals were in place, the wheel pockets were primed and given the necessary coats of CA-50 coal-tar paint. The paint was applied by brushing and was kept away from areas which would be in contact with the fixed-wheel assemblies. The fixed wheels were then installed and brought out to the proper relation to the piano wire mentioned above.

The prime contractor built a road from the paint yard to the rim of the canyon directly above the left abutment of the dam. A slide, consisting of two parallel runs of 36 140-pound I-beams, was anchored to the almost vertical canyon wall. A car was fabricated to keep the gates from rubbing while being lowered to the dam. The contractor fabricated a short gate stem for use while handling the gates. He also fabricated a short boom for the dragline and a lifting beam for use by both cableways to left the 125-ton gates. The short stem was installed in gate No. 8, and the crane was positioned above the slide and used to lower the gate to the roadway of the dam.

The two cableways moved the gate to the upstream side of the dam, a distance of about 30 feet, where the 165-ton gantry crane could reach it. The short stem was removed and returned to the paint yard for use in gate No. 7. The regular stem for gate No. 8 was installed. The gate was picked up by the 165-ton gantry crane and set in block 7 gate structure above penstock No. 8. Seven intermediate sections of stem and the No. 8 hoist cylinder completed the installation. This procedure was repeated without mishap until all of the eight gate assemblies were completed.

The installation of the gate, the seven intermediate stems, and the hoist require about 10 hours with two to three men and an operator. The welding and other work required 20 working hours for the pipefitters to install the high-pressure oil. The first gate was placed in the gate structure on April 20, 1964, and the last gate was installed and fully tested on June 2, 1964.

The tests were begun in accordance with the appropriate operating drawings and instructions. However, the gate would not stop when the stop button was pressed and the throttle valve was one-fourth open. With this condition, the restoring feature would not operate.

A design engineer from the Denver office determined that valve C of the high-pressure oil piping needed a stronger spring, and new springs were obtained. The new, heavier springs corrected the faulty operation of the gates. However, two of the valve bodies were sensitive to the tightening of all the socket-head body bolts; i.e., one bolt if fully tightened would render the plunger or spool inoperative. An 0.008-inch-thick shim in the joint under the bolt obviated this condition.

One of the checks made during the tests was to have an inspector, the contractor's chief of party, and a rodman enter the bellmouth, allow the gate to be closed, and use a feeler gage 1/32-inch thick to check the seals where daylight was observed. The feeler gage could be inserted between the rubber seals only on one gate. The most daylight appeared at the molded corners and not where the brass-bound rubber seals were placed.

The results obtained in the operational tests are as follows: Any of the eight gates will close in less than 2 minutes and 8 seconds; they will open in less than 20 minutes. While closing, they will stop in a distance of from 3 to 6 inches. The restoring feature cuts in when the gate settles about 11 inches.

In addition to the inadequate springs in valve C of the high-pressure oil piping, the makeup of the hanger stud assembly gave trouble when the hanger collar seized on the top of the piston and acted as a double nut while unscrewing the hanger stud. The lead plug was not positive enough to keep the collar stationary on the hanger stud. It was decided to install a 3/8- by 1-inch cup-point set screw in place of the plug. A 3/8-inch hole, 1/4 inch deep, was drilled into the hanger stud 5-3/8 inches from the end of the stud, the collar positioned, and the new set screw installed. This resulted in 4-7/8 inches of threads below the collar, which is the requirement. All eight hanger studs were modified in the above manner without further difficulties being experienced.

The workmanship on fabrication of the gate assemblies and on the erection and assembly of the gates was excellent.

205. STOPLOG GUIDES. The stoplog guides in the penstock trashrack structure were supplied by Bennett Industries of Peotone, Ill., under schedule 2 of invitation No. DS-5354, and were installed by Merritt-Chapman and Scott Corp., under specifications No. DC-4825.

Discrepancies in the assembled guide sections were discovered. The major error was that the ends of the cast iron guide sections were not in the same plane as the ends of the steel plate seat sections. This was contrary to the specifications and, if the guides were installed on the dam in this condition, tight joints could not have been obtained.

The first attempts to correct this discrepancy were started on September 6, 1962, with a vertical milling machine and an alinement table long enough to hold three complete guide sections fitted together. The first three sections after the ends had been machined were fitted together, checked and failed to meet the specified tolerances in the overall lengths and in the madeup joints. Further work with the equipment on hand failed to correct the fabrication errors.

The manufacturer brought in a heavier duty milling machine on September 24, 1962. Three guide sections machined on this new milling machine were fit together with all tolerances within the specified measurements. Corrective work on each of the 368 guide sections was completed on November 7, 1962. The general procedure used in the corrective work was as follows:

(1) For each guide section where the steel plate seat portion was shorter than the cast iron guide portion, weld was applied to the end for a buildup which equaled or exceeded the cast iron portion.

(2) Each section was placed on the milling machine and the end was machined normal to section. Where heavy cuts were made, a light finishing cut followed the heavier cut.

(3) Three consecutive sections after having been machined on each end, were fitted together on the alinement bench where each joint was measured with a feeler gage. The maximum acceptable gap was 0.004 inch.

(4) Two piano wires were stretched along the guide and the seat portions for a check of plumbness and straightness.

(5) After acceptance, the lower two of the sections were stockpiled. The upper section was then moved to the other end of the alinement table and was assembled to two more consecutive machined sections. This procedure was repeated for the remaining sections.

A crew averaging four men accomplished the corrective work. One man operated the milling machine; one man operated a forklift truck and carried sections to the milling machine, the alinement table, or stockpiles; one man assisted the machinist or assembled the machined sections; and one man applied weld buildup to the uncorrected sections. After the corrective procedures were developed, these men did an excellent job considering the long hours and consecutive days of work.

Because of the sections being shortened in the corrective process, the installation of the sections required that the contractor insert stainless steel shims between some of the sections to make up the required distance. This was necessary since the anchor bolts had been installed in the mass concrete of the dam according to drawings. For accuracy of the placement of the guide sections, the sections were installed in blockouts and on the anchor bolts and then later concreted in place.

During installation, slotting of the splice plates was necessary at various locations due to either a misplacement of the embedded anchor bolts or in some cases due to the shortened sections before or after the addition of shims between the sections. No other unusual problems were experienced by the contractor during the installation of the guide sections.

206. DRAFT TUBE BULKHEAD GATES. Twenty-four 12.42-foot-wide by 15.95-foot-high bulkhead gates are provided for use in unwatering the eight turbine draft tubes. The gates are stored in slots, resting on latches, under the upper guide lug on each side of the gate. A lifting frame is provided for raising and lowering a gate with the 10-ton gantry crane. The bulkhead seats, gates, guides and frame were manufactured by the Ogden Iron Works of Ogden, Utah, under invitation No. DS-5341 and were installed on the downstream face of the powerplant by the prime contractor, Merritt-Chapman and Scott Corp., under specifications No. DC-4825.

An extra work order and an order for changes were executed during the life of the supply contract. Chain guides and pins were furnished with the bulkhead gate lifting frame for use in storage as extra work. Mill scale was removed from the structural surfaces of the steel to which the naval brass seats were secured and a coat of white lead and linseed oil was applied; and additional flathead steel cap screws and hexnuts were furnished for gates 4 and 5. The contractor also requested and was given permission to use naval brass in accordance with Composition 1, Federal Specifications QQ-n35, in lieu of that required in the specifications, and the use of flathead naval brass bolts instead of rivets on seat parts IL, IR and 2.

Timely delivery was made in four separate shipments of six gates each and two separate shipments of guides. Protection of the gates and guides was adequate during shipment and no unusual damage was incurred in transit. Shop painting of the metalwork was not required and all finished surfaces of ferrous metalwork were coated with a rust-preventative compound prior to shipment.

At the jobsite, the rust-preventative compound was removed, the surface prepared, and the gates and guides were painted with two coats of type IV red lead primer and two coats of phenolic-resin aluminum paint. The good paint system obtained was retained, because the installation contractor lowered the gates into place from the painting yard at the top of the canyon to the powerplant gate slots using the traveling cranes, or highlines, for convenience in handling and to avoid damage and paint repairs.

The close tolerances of 1/16 of an inch, plus or minus, top to bottom, required careful installation techniques by the contractor. After installation of the lower seal plate, the gate guides were rough assembled from top to bottom. Special templates were tack-welded to the lower seal plate on each side at the bottom, and steel frames were cantilevered from the concrete for the upper template. Three holes, just larger than No. 16 piano wire, were drilled in the templates for alinement, one each for the outside faces and one for the centerline of the inside of the slots for width and depth. Another set was drilled for the face of the lower steel frame. Piano wires were attached between the two templates using a device tensioned by a ratchet-type reel. Plates were set by precise surveying methods and were checked periodically.

During installation, it was found that the 1/2- by 2-inch naval brass flathead bolts, which secured the brass seal plates to the steel frames, were defective due to fractures, either at the body or head or in the threaded section between the head and the nut. About 15 percent of the bolts were found to be defective prior to installation, and further breakage discovered during erection amounted to about 20 percent of all bolts. As a result, all of the naval brass bolts were replaced with monel metal bolts. However, laboratory tests in Denver failed to confirm either that the bolts were defective or that they failed as a result of factory overtightening. The supplier was therefore not held responsible for the costs involved in replacing the bolts.

Other difficulties were experienced in installation due to insufficient depth in the concrete blockouts and improper location of anchor bolts. Minor difficulties were also experienced as a result of some of the guide sections being distorted or warped which required special jack screws being made to spring the sections into position. Some warpage and twisting of the stationary gate seals and support frames was also noted. As the horizontal frame sections did not match the vertical sections at the top, the contractor jacked the upper frame section into place and welded at the assembly joints. Installation of the gates, seals, and guides was satisfactory and no difficulties were experienced during the testing and initial operation of the gates.

When the gates were placed in service for the first time, it was difficult to raise them due to a collection of sand, silt, and rubbish at the bottom of the gates, probably due to the testing and operation of the hollow-jet valves. The gates were freed with tube-introduced compressed air along the face of the gates. After the gates were raised for the first time, this waste material was flushed out and no further trouble was encountered.

207. RIVER OUTLETS. Nonpower drawdown of the reservoir is provided by four outlet pipes (figs. 109 and 110), each 8 feet in diameter and located 96 feet below the penstock elevation level. Although similar in construction to the penstocks, each major element of the installation differs in some respect. There are two trashrack structures for the four outlet pipes, each structure containing two pipes closely spaced at the same elevation. Upper closure is made by 96-inch ring-follower gates (fig. 275) inset some 50 feet inside the dam and by 10-1/3-foot square bulkhead gates seated in guides at the inlet face of the pipes. No stoplogs are provided for the river outlet intake structures. Embedded their length in concrete, the outlets discharge downstream from the powerplant through 96-inch hollow-jet valves.

Methods utilized for installation of the outlet pipes were essentially the same as those used for the penstocks. Being at the lower elevation of the two sets, horizontal pipe sections of outlets No. 1 and 2 (fig. 276) were first placed in block 6 of the dam and in the mass concrete downstream from the dam in February of 1961. Initial sections of outlets No. 3 and 4, which had intakes in block 5, were placed during the following month, and all sections extending downstream from the dam had been embedded by the end of July 1961. The four 8-foot-diameter pipes were positioned and welded as the blocks rose in elevation. Installation of the outlet pipes was completed in April 1962, when the bellmouth sections for outlets No. 3 and 4 were placed in block 5.

Figure 275.—General view of equipment used in testing of river outlet No. 4 at various flow conditions. Test station is located in the ring-follower gate chamber in dam block 5 at elevation 3387.5. P557-420-11016, May 24, 1965.

Figure 277 shows the outlet pipes partly embedded as of March 20, 1961.

208. RIVER OUTLET BULKHEAD GATES, GATE FRAMES, AND LIFTING FRAME. The bulkhead gate is a flat structural steel assembly with rubber seals mounted on the downstream skinplate. This gate was furnished by Johnson Machine Works, Inc., of Chariton, Iowa, on invitation No. DS-5493. The rubber seals of the gate contact the seal seats embedded in concrete on the upstream face of the dam around the bellmouth of the outlets. The seal seats were supplied by Fulton Shipyard, Antioch, Calif., under invitation No. DS-5370. The gate is closed, opened, and transported by the 165-ton gantry crane at the top of the dam. The crane raises and lowers the gate with a lifting frame which follows the gate guide slots and is coupled to the gate stem by a grapling hook. This lifting frame was supplied by Charles C. Steward Machine Co., of Birmingham, Ala., under

Figure 276.—View of construction operations in the powerplant and dam from the downstream cofferdam. Sections of outlet pipes 3 and 4 are in place in the machine shop bay above pipes 1 and 2 (right foreground). P557-420-5819, March 23, 1961.

purchase order No. (D) 90,637-A. The bulkhead gate was received at Flagstaff, Ariz., on December 22, 1961, the embedded seal seats on February 2, 1961, and the lifting frame on August 14, 1963.

The bulkhead gate, embedded seal seats, and lifting frame were installed, erected, and assembled by the prime contractor. Installation of the seal seats and anchorage was performed in conjunction with the placement of the concrete. The work was substantially completed with the dry testing of the gate operating cycle in October 1963. The assembly of the bulkhead gate consisted of attaching rubber seals and clamp bars with the stainless bolts. Since the assembly work was performed by an experienced crew of millwrights, no difficulties were encountered during the assembly. The seal seats were placed on the anchor bolts in the blockouts and were adjusted to their proper position using templates and fine piano wire stretched tight for measuring control. The positions of the embedded frames and location of wire templates were determined by the contractor's survey crews. The contractor's survey crews were well qualified for this type of construction layout, and many of the alinement problems often associated with this type of installation did not develop.

209. RING-FOLLOWER GATES AND CONTROLS. The four 96-inch ring-follower gates

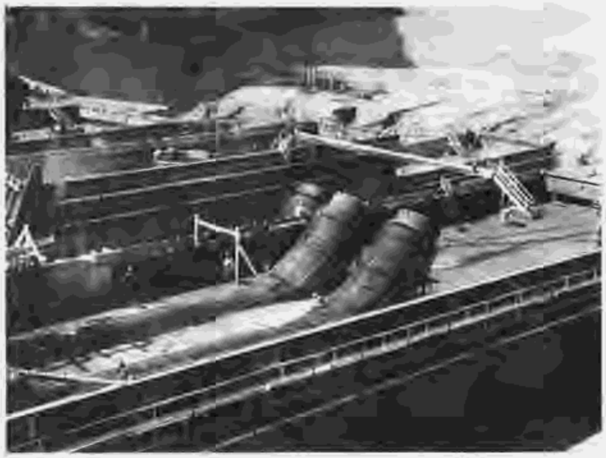


Figure 277.—Installation view of outlet pipes. Partially buried in concrete, two of the outlet pipes poke their way skyward at Glen Canyon Dam. P557-420-5806, March 20, 1961.

were supplied by Goslin-Birmingham Co. under invitation No. DS-5269 and the two control cabinets were furnished by Kendo, Inc., of Denver, Colo. under invitation No. DS-5498.

The ring-follower gate is of cast and welded steel construction and consists of a body, a leaf, and a hydraulic hoist which is an integral part of the gate. The slide-type gate with a leaf includes a follower ring having a circular opening equal to the diameter of the pipe to provide an unobstructed water passage when the leaf is in the open position. The hoist cylinder is 27 inches in diameter and has a travel of 8 feet 8 inches.

The two control cabinets are of steel construction and enclose the control system for two ring-follower gates in each cabinet. The control system was installed in the gate chambers of the river outlets in the dam and was used to operate the four ring-follower gates. The opening and closing of the gates is controlled from a pushbutton station located in each cabinet. The ring-follower gates were received at Flagstaff, Ariz., beginning in May of 1961 and shipment was substantially complete by October 16, 1961. The control cabinets were received at Flagstaff on September 29, 1962.

The ring-follower gate installation was performed by the prime contractor under specifications No. DC-4825. Installation started with placement of main body and lower bonnet in block 6 in February of 1962, and the embedded parts were completed in block 5A in April of 1962. The ring-follower, gate leaf, upper bonnet cover, and hoist were installed in outlets No. 1 and 2 in May 1962, and outlets No. 3 and 4 in June of 1962.

In January of 1963, after the installations had been completed, the contractor was operating gate No. 4 for testing and adjustment. During the final checkout, ring-follower gate No. 4 was taken to the closed position but failed to close completely by approximately 2 inches. This gate was being operated by a remote switch to the control cabinet by the crew inside the river outlet pipe. The safety pressure switches had been calibrated and set in accordance with the appropriate drawing. During the long period of erection of the gates, a large amount of residue from sandblasting of outlet pipes upstream of the ring-follower gates had collected in the bottom bonnet of the gate; also drainage water and cement grout had caused sandblast sand residue to solidify to some extent. When the ring-follower on the gate leaf was brought down toward its closed position, the hydraulic thrust collapsed the ring in the bottom of the gates. Measurements on the diameter found the ring was 1 inch out-of-round and warped approximately 3/8 inch. Several methods were tried in an effort to push the ring to its original shape while the leaf was in position in the gate body. All efforts to restore the ring of the leaf follower to roundness while in the gate body failed. The gate was disassembled and the leaf was taken to the installation contractor's shops for repair, during the week of February 11 to 18, 1965. A failure was noted on the downstream stiffener, 3 feet 11 inches up from the bottom and to the left, and at the fabricator's welded joint near the bottom.

During the next 2-1/2 days, the contractor fabricated a structural steel pressing jig and sandblasted away all the coal-tar enamel coating from the section. The true center of the waterway was located by using the top normal machined surface as a reference plane. Radial distances from the center to the inside of the shell were recorded for each 10°.

This measurement was used to determine the amount any segment had to be moved to obtain the desired roundness and to check on any gains made during the pressing operations.

The initial pressures exerted on the section were 50 tons applied near the right springline and 20 tons applied on the inside bottom surface. Measurements taken on the loaded section indicated the horizontal diameter was shortened by 3/8-inch and the lower vertical radius was lengthened by 0.050 inch. When released, the section assumed the original configuration before the pressure had been applied. During this press, it was noted that the weaker top portion bowed out

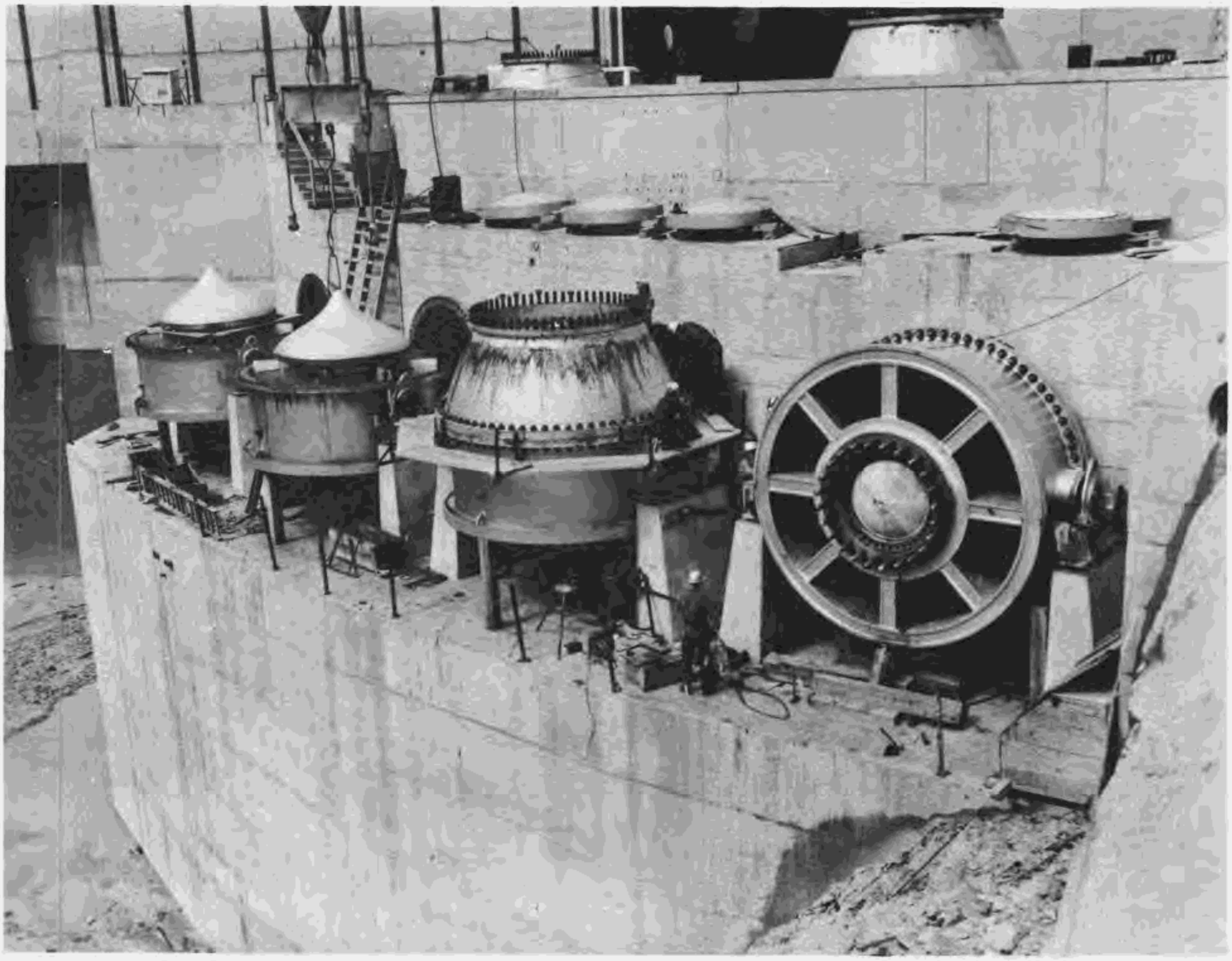


Figure 278.—Hollow-jet valves being installed in outlet works. P557-420-8140, March 18, 1963.

and away from the center point. For added strength to this portion, the contractor added a 1/2-inch-thick plate, 8 inches wide and 9 feet long, across the top of the section on both the upstream and downstream side. For a more evenly distributed load to the top of the section, braces were added between the pressing jig and the two upper quarter points.

In a 3-day period, various pressures were applied to the section to a maximum of 130 tons on the right springline and 32 tons on the lower inside surface. Higher pressure did not seem practical since the top portion had a tendency to bow out. During the last press of this period, the brass seat was removed and the lower half of the section was heated with propane torches. Temperatures of approximately 550° F. were produced along the bottom, 250° to 300° F. at the lower quarter points and decreasing to 113° F. at the springline. These operations accomplished a gain of less than one-fourth inch on the horizontal and vertical diameters.

As the follower could not be pressed back into shape, the contractor cut the stiffeners around the circumference of the ring-follower. This method proved to be successful and the ring was brought back to original shape and rewelded. The gate leaf was again installed and the repair work was completed on March 28, 1965. At a later date, the ring-follower gates were put in service and the repair was satisfactory in that leakage was very small on this gate. Subsequently, the pressure limit switch on the other gates was lowered to a safer value which would just operate the gates during initial installation and testing.

With the exception of incorrect pilot-operated check valves, the installation of the controls proceeded without unusual difficulties. The supplier replaced these incorrect check valves at the site without cost to the prime contractor. The control cabinets for gates No. 1 and 2 were badly damaged on May 23, 1962, when the roof over the gate chamber failed as fresh concrete was being placed in block 6A of the dam. The control cabinet was returned to the supplier's plant, repaired, and returned at the expense of the prime contractor.

The ring-follower gates and controls were first placed in service by passing water through the outlet works in January 1965.

210. HOLLOW-JET VALVES. The four hollow-jet valves and controls (figs. 127, 128, 278, and 279) are used to regulate the flow of water from the river outlets. The hollow-jet valves are of cast and welded steel construction and consist of a circular body and a movable needle which forms an annular passage and seals in the throat. The hollow-jet valve is hydraulically operated by a cylinder within the passage which is concentrically positioned by eight radial splitters. Each control cabinet is of steel construction and encloses the control system for two hollow-jet valves. A cable-driven dial-type position indicator for each hollow-jet valve is located on the cabinet control panel to show the percent of opening. The 96-inch hollow-jet valves were manufactured by the Goslin-Birmingham Manufacturing Co., Inc., of Birmingham, Ala., under invitation No. DS-5363. The controls were supplied under invitation No. DS-5503 by the Rucker Co. of Oakland, Calif.

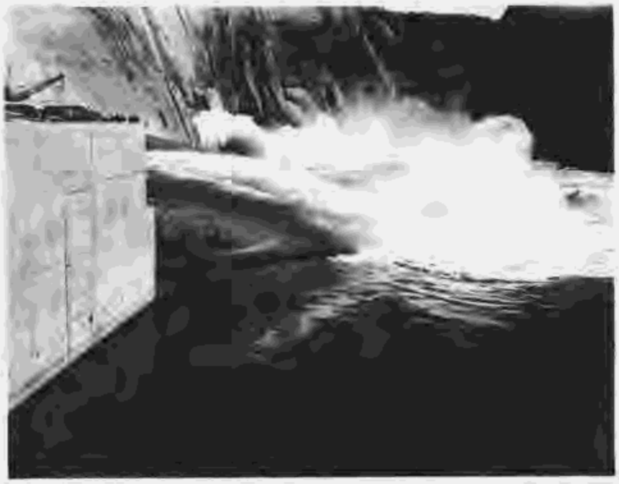


Figure 279.—View looking downstream at No. 2 hollow-jet valve under test, with 10,000 cubic feet per second being discharged from left diversion tunnel in background. P557-420-10685, March 4, 1965.

The four hollow-jet valves were installed at the site by the prime contractor starting on January 15, 1962, and work progressed using conventional erection procedures until completion in April of 1963. The controls were supplied, assembled, and tested by the supplier; only a small amount of field piping work was required to complete the installation. One difficulty noted involved the placement of the flow regulation valves which were inaccessible after assembly of control cabinets. It will eventually be necessary to provide an opening in the cabinets for ready access to adjust the valves. After the first season of operation, the valves were inspected for damage and very little cavitation was noted.

CHAPTER XIV. *Construction*—POWERPLANT

211. GENERAL. Glen Canyon Powerplant was constructed 400 feet downstream of the axis of the dam. This reinforced concrete structure is 649 feet long by 113 feet wide and is 150 feet high above the mass concrete required for a foundation. The structure contains eight 112,500-kilowatt generating units, each driven by a 155,500-horsepower vertical-shaft turbine. The total plant generating capacity is 900,000 kilowatts, or about 81 percent of that authorized for the entire Colorado River Storage Project.

Eight unit bays and a service bay span the width of the canyon with an elevator tower on the west end of the plant and a machine shop downstream of the service bay. The control and office area are housed in a two-story structure placed above the superstructure of units 1 and 2. Placed to a top elevation of 3124.5, the substructure concrete was reinforced around the turbine pits, as well as below the draft tubes, down to elevation 3093.0. Unreinforced mass concrete was placed below this elevation and also upstream of the turbine pits up to elevation 3118.75. The reinforced concrete intermediate structure is from elevation 3124.75 to elevation 3188.50 and contains gallery systems both upstream and downstream of the unit bay crosswalls, and the transformer deck on the downstream side of the powerplant. The superstructure is a structural-steel frame with reinforced concrete walls above elevation 3188.50, and lightweight precast concrete roof slabs for all units except unit 3. Second-stage concrete was used for encasing the turbine spiral cases and for supporting the generators.

First-stage concrete, including the entire closed shell of the powerplant structure, was constructed under the prime contract, specifications No. DC-4825, by Merritt-Chapman and Scott Corp. Second-stage concrete, surfacing concrete, and related work were accomplished by the completion contractor, Ets-Hokin Corp. under specifications No. DC-5750. The generators were furnished and installed under a separate contract, invitation No. DS-5522, with General Electric Co., and the elevators were furnished and installed by Pacific Elevator and Equipment Co., under invitation No. DS-5843.

Air, water and power for the construction work under the prime contract in the powerplant area were furnished from stations on the canyon rim. Air was furnished by supply piping from compressors on the rim. Water was obtained from the wells at Wahweap via the main distribution system. Power was supplied by line from the rim and a temporary distribution system which was installed and moved as the construction work progressed. Construction materials were brought to the work area by high-line cableway or were trucked through the powerplant access tunnel. Reinforcement was bent and bundled for each lift or placement at the steel yard on the east rim. Reinforcing steel was supported by angle iron and on cast concrete blocks of size to provide required cover. Most of the piping was precut and partially assembled when the piping arrived at the site for installation.

212. EXCAVATION. Excavation for the powerplant structure began in January 1959, with the removal of talus on the left side of the canyon in the machine shop and service bay area. A berm for the machine shop roof at elevation 3239 was excavated, intersecting a minor joint at approximately elevation 3240. As the joint nearly paralleled the face of the canyon wall, rock bolts were placed throughout this section. A rock slab was removed on the left side downstream of the m-line between elevations 3290 and 3195. Foundation benches were cut out of the rock by line drilling methods as the excavation progressed. Benches for the river outlet valve structure and training wall area were cut at elevations 3170, 3161.5, 3151.5, 3141.5, 3131.5, 3118, 3112, and 3108. At the time of the strike on July 6, 1959, powerplant excavation had reached elevation 3110 on the left side.

Drilling and shooting for the powerplant structure began on the right side of the canyon in March 1959. The elevation 3118 bench had been exposed at the time of the strike and excavation had reached elevation 3097. Rock bolts, 6 and 8 feet in length, were used at appropriate locations on both sides of the canyon. Loose rock in the nearly vertical walls of rock excavations was pried off by highscalers working from ropes. Periodic inspections were also made for the purpose of removing any additional rock which became loosened after the initial scaling. Performed concurrently with excavation for the dam, conventional methods and equipment were used for the rock and common excavation in the powerplant foundation area. Briefly, the drilling work was performed by wagon drills and a large bench-type work area was blasted for each round. Large, steel-tracked shovels and draglines loaded the muck into large dump trucks for dumping on the upstream and downstream cofferdams. The upstream cofferdam was located so that a work space was available between the foundation of the dam and the upstream cofferdam, until the excavation level reached the inner canyon bed level.

Following an extended strike of 6 months' duration, excavation work for the dam and powerplant foundations resumed in January 1960. Line drilling, shooting, and mucking for the powerplant excavation began on the left side with the elevation 3110 berm.

Some unsafe rock was removed between elevations 3160 and 3184, between the ex- and g-lines, and berms were exposed on the left side to elevation 3060 under unit bay 8.

Powerplant excavation on the right side of the canyon was resumed at elevation 3097 in the middle of March 1960. Except for the haul, excavation of unit bays 1, 2, and 3 was completed at about elevation 3093 early in May 1960. Bedrock in unit bay 4 was at approximately elevation 3035 which was about 58 feet lower than unit bay 3. Bedrock in unit bay 5 was at approximately elevation 3030. Free-draining gravel material from the excavations was stockpiled at the downstream cofferdam for later use in the tailrace area and between the dam and the powerplant.

During May of 1960, steel dowels were placed in unit bay 1 at elevation 3093, in the theoretical bench at elevation 3109, and in the upstream section of unit bay 3. The ground mat installation also began during this period.

213. FOUNDATION. Once diversion had been completed and the dam keyway excavation had progressed to river level, excavation for the foundations of the dam, penstock supports, powerplant, service bay, machine shop, and river outlet area progressed concurrently. The powerplant was constructed across the downstream portion of an inner gorge in the river bottom rock that extended upstream a short distance from the dam foundation. It was necessary to excavate rock to provide a foundation for the substructure of units 1, 2, and 3. For the remaining units, 4 through 8, mass concrete was placed to provide a foundation. Excavation for the mass concrete foundation was held to a minimum in the river bottom rock, providing keying upstream of the longitudinal centerline of the units.

Originally line drilling and benches would have been used for the service bay foundation but, due to bedding of the sandstone, the benches could not be held. In lieu of the benching, anchor bars were grouted into the canyon wall along with some rock bolting.

A stress relief joint and rock slab in the vicinity of the service bay area mass concrete extending into the left abutment keyway of the dam required remedial measures to assure adequacy for the structure foundation. The rock slab was anchored to the canyon wall from near the a-line to just downstream of the intersection of the 2-inch expansion joint between the dam mass concrete and the service bay with 1-3/8-inch anchor bars on 5-foot centers both ways between approximate elevations 3095 and 3182. The anchor bars were embedded a minimum of 10 feet in sound rock past the relief joints. The anchor bars and relief joints were grouted in 5-foot lifts as the mass concrete elevation increased. The additional line of 1-3/8-inch anchor bars, as required on the drawings downstream to the a-line and bx-line, were installed in the same sequence as the anchor bars upstream of the a-line.

Two drainage systems were constructed at the concrete-rock contact from downstream of the 2-inch expansion joint to approximately 3 feet upstream of the d-line with gravel and perforated sewer pipe and open-joint sewer pipe. After the anchor bars and grouting had been completed, 3-inch drain holes were drilled through 4-inch pipe embedded in the concrete, through the sewer pipes into the rock, a minimum of 1 foot past the grouted stress relief joint. The purpose of the drain holes was to insure that water pressures would not build up under the rock slab. This phase of the work was accomplished beginning in September 1960 and was completed with the drilling of the required drain holes in March of 1961. Additional anchor bolts were required for the canyon wall but are associated with the dam and are therefore presented in section 161. A series of 1-1/2-inch bolts and drain holes were required to be installed as a part of the canyon wall bolting. These were located between the canyon wall and the service bay and machine shop superstructure walls upstream of the m-line. The work in this area was performed during the fall and winter of 1961 in advance of the remaining wall bolting, as it was necessary that it be completed prior to constructing the superstructure concrete walls.

During June of 1960, movement amounting to 0.08 foot was detected in the foundation rock in unit bays 1 and 3 and in the area of the penstock supports for penstocks 1, 2, and 3. Check points were established throughout the area and were observed for further movement. Additional movement of about 0.07 foot was noted in August, generally towards the east at the upstream edge of penstock support 3C, at the edge of a bench some 58 feet high.

Several steps were taken to stabilize the foundation material. Construction was suspended in unit bays 1, 2, and 3 until all mass concrete in the powerplant was brought up to the level of the unit bay 3 foundation; construction was also suspended on the supports for penstocks 1, 2, and 3 until the adjacent dam and powerplant concrete was brought up to the level of the foundation of the supports for penstock 3; and the size of the fillets on the dam was increased to fill the void between the toe excavation face and the downstream toe of the dam, extending an enlarged fillet from the near vertical cut for the bases of supports for penstock 4 in block 13 to the right abutment. In addition, it was necessary to support the rock wall between penstocks 3 and 4 with concrete buttresses between the powerplant and dam. Four concrete support buttresses were constructed, one each between stations 5+00 and 5+34, 5+37 and 5+74, 5+88 and 6+93, and 6+39 and 6+60. Completed by January 1961, the buttresses averaged about 30 feet in thickness.

214. CONCRETE. (a) *Mixing and Placing Methods.*—Concrete for the powerplant area was manufactured in the central batching plant, located on the elevation 3540 bench, excavated out of the right canyon wall upstream of the dam and approximately 700 feet upstream of the powerplant. The concrete was delivered to the construction area by high-line cableway using concrete buckets. The first concrete in the powerplant area was placed on May 5, 1960, for penstock support 2B, using one of the main 50-ton high-line cableway with a 12-cubic-yard concrete bucket. The 50-ton cableways were used in the powerplant area to place concrete until late November 1960 when the third high-line cableway, with 25-ton capacity, was placed in service. The remaining concrete placed in the powerplant area was placed using the third high-line cableway with an 8-cubic-yard bucket. The concrete in the downstream part of the tailrace slab and the river outlet area could not be reached using the highline. For these locations, the structural concrete was transferred to a hopper and was placed using a crane and a smaller bucket. Mass concrete for the river outlet was placed using a pumpcrete machine, located within the reach of the highline used to deliver concrete from the concrete batching plant. Concrete mixes varied from 6-inch maximum size aggregate for the mass concrete to 3/4-inch maximum size aggregate, depending upon the complexity of the particular placement and taking into account cover for embedded items, spacing of reinforcing steel, massiveness of the placement, and amount and spacing of the embedded materials. Generally, the typical sizes of aggregate used were as follows:

Location	Maximum size of aggregate, inches
Mass concrete	6
Substructure massive walls	3
Intermediate structure massive walls and columns	3
Structural concrete, superstructure walls, parapets, floors, cover slabs	1-1/2
Stairs, blockout concrete, and miscellaneous small placements	3/4
Bulkhead gates blockouts for guides and seats	Grout

Details of concrete materials, methods, and equipment used in the manufacturing, control, mix designs and compressive strengths are covered in appropriate sections of this publication. Additional information is presented in the following subsections.

(b) *Forms.*—Wooden forms were used for the mass concrete in the powerplant area until the mass concrete was of sufficient height above rock, then steel panel forms were used. Powerplant area structural forms, both exterior and interior finish surfaces, were made of 3/4-inch exterior plywood sheathing on 2- by 6-inch studs spaced on 16-inch centers and with double wales of 3- by 6-inch timbers on 16-inch to 3-foot centers.

Tongue-and-groove sheathing was required to be used for the exterior surfaces of the superstructure concrete on the south elevation of the machine shop, the west elevation of the powerplant, and the remaining surfaces of the cable and elevator tower, for architectural appearances. Forms were tied together with 1/2-inch steel rods and 3/4-inch she-bolts. Form tie spacing ranged from 16-inch centers to a maximum of about 3 feet, taking into account various factors such as size, shape, and depth of the placement. The contractor was quite generous in his use of form ties, resulting from having several form failures early in the concrete construction phase. All wood forms were constructed at the carpenter shop located on the east rim. The quality of the forms constructed at the carpenter shop was outstanding, resulting in excellent concrete surfaces. Exterior concrete forms were usually stripped within 24 hours after placement, with repairs being made as required and she-bolt holes being plugged shortly thereafter.

(c) *Placement Schedule.*—The first mass concrete in the powerplant substructure was placed on May 31, 1960, in the upstream section of unit bay 1 between the 5- and 3-lines to elevation 3105. Placement in unit bays 3 and 5 followed. Concrete placement methods and equipment closely followed the procedures used in the concrete construction for the dam, and the mass concrete in the powerplant structure was completed in December 1960 with a placement in unit bay 8.

Placement of the reinforced substructure concrete began in June 1960 and continued in the logical sequence of construction, followed by initiation of intermediate structure concrete construction in September 1960. By this time, the pattern of concrete placement was established in a routine sequence of unit bays 1, 3, 7, and 5, followed by unit bays 2, 4, 6, and

8. The mass concrete for the outlet works encasement downstream of the dam also became a part of the sequence placements between the dam concrete and the powerplant as the work progressed. During the early part of 1961, concrete placing in the powerplant averaged about 13,000 cubic yards per month. This average dropped to about 1,700 cubic yards per month during the last 6 months of the year, due partly to concentration of effort on placement in the dam and partly to the greater forming and reinforcing steel work requirements in the structural concrete of the powerplant. Concrete construction in the machine shop and service bay followed on an appropriate schedule. Intermediate structure concrete was completed in November 1961 with a placement in unit bay 8.

Placement of the superstructure concrete began in unit bay 1 in October 1961. In November 1961, the contractor was advised that the powerplant cable and elevator tower above elevation 3233.33 and unit bays 1 and 2 above elevation 3231.69 were being redesigned to provide fallout protection. No work was performed above those elevations until the details of the revisions were furnished, and all superstructure concrete was completed early in July 1962. The high bay windows in the powerplant unit bay outside walls were also deleted for additional fallout protection and security purposes.

(d) *Cooling.*—The powerplant unit bay mass concrete was placed in 5-foot lifts and was not cooled. The mass concrete for the service bay, machine shop, and river outlet was required to be cooled to 60° F. One-inch outside-diameter aluminum tubing on nominal 5-foot spacing was embedded in the mass concrete, and refrigerated water was circulated through the cooling coils at a rate of approximately 4 gallons per minute for a period of 12 days. Upon completion of placement of the mass concrete, the concrete temperature was lowered to the required 60° F. Thermocouples and insert thermometers placed in embedded tubing were used to control the progress of the cooling. In March of 1961, following completion of the required cooling, the contraction joint, cooling pipe, and thermometer tubing downstream of the m-line were grouted. The machine shop and service bay cooling pipe was grouted during June of 1961.

The service bay mass concrete cooling was accomplished using the chilled water supply system used for cooling the mass concrete in the dam. A small refrigeration plant with chiller and cooling tower was set up downstream of unit 8 near the m-line to cool the machine shop and river outlet mass concrete. This same plant setup was first used to cool the dam mass concrete prior to placing the main refrigeration plant chilled water system in service.

(e) *Curing.*—Concrete curing was performed as required by the specifications. The exterior surfaces of the mass concrete and powerplant structure concrete were cured using gray membrane curing compound. Horizontal construction joints, floors, and stair treads were cured with water for at least 14 days. Finished floors were covered with plywood for protection during the construction phase; but even so, gouging and spalling of these surfaces required considerable repairs by the prime contractor. Interior surfaces of walls, columns, ceilings, soffits, vertical construction joints, and construction joints were cured by leaving the forms in place for at least 4 days. Mass concrete that contained pozzolan was required to be cured for a minimum of 21 days or until covered with concrete. Dry-packed she-bolt holes were cured by covering with masking tape.

(f) *Tailrace Slab.*—The contractor was instructed to construct a reinforced concrete slab (fig. 280) in the tailrace of the powerplant by order for changes No. 7, in lieu of riprap required by the specifications. The tailrace area was laid out in 17 sections, designated A through Q, with 10 slabs placed in sections A through H, and 9 slabs in sections I through Q. The tailrace slabs were about 30 feet square and were 8 inches in depth for a distance of 150 feet from the powerplant and 12 inches deep for the remaining length of the slab. Following dewatering and backfill compaction, the work was started in September 1961 in section A-1, downstream from unit bay 8, and was completed in April 1962. For placing sections farthest downstream which were beyond cableway reach, it was necessary to transfer the concrete bucket to a crane.

Placing of concrete in the powerplant was suspended on December 11, 1961, due to severe winter weather, and was not resumed until the middle of February 1962, when the weather moderated. Placement of the tailrace slab was resumed late in December, and portable wooden shelters were constructed to protect the slab placement. Electrically driven forced-air heaters were used to assist in the protection against low temperatures. The tailrace slab collapsed 3 years later in April 1965, after the river outlets were operated for an extensive period (see section 57).

215. POWERPLANT SUPERSTRUCTURE. Furnished by the prime contractor, structural steel for the powerplant superstructure had been prefabricated and was ready for erection upon delivery at the site. The structural steel was fabricated and erected by

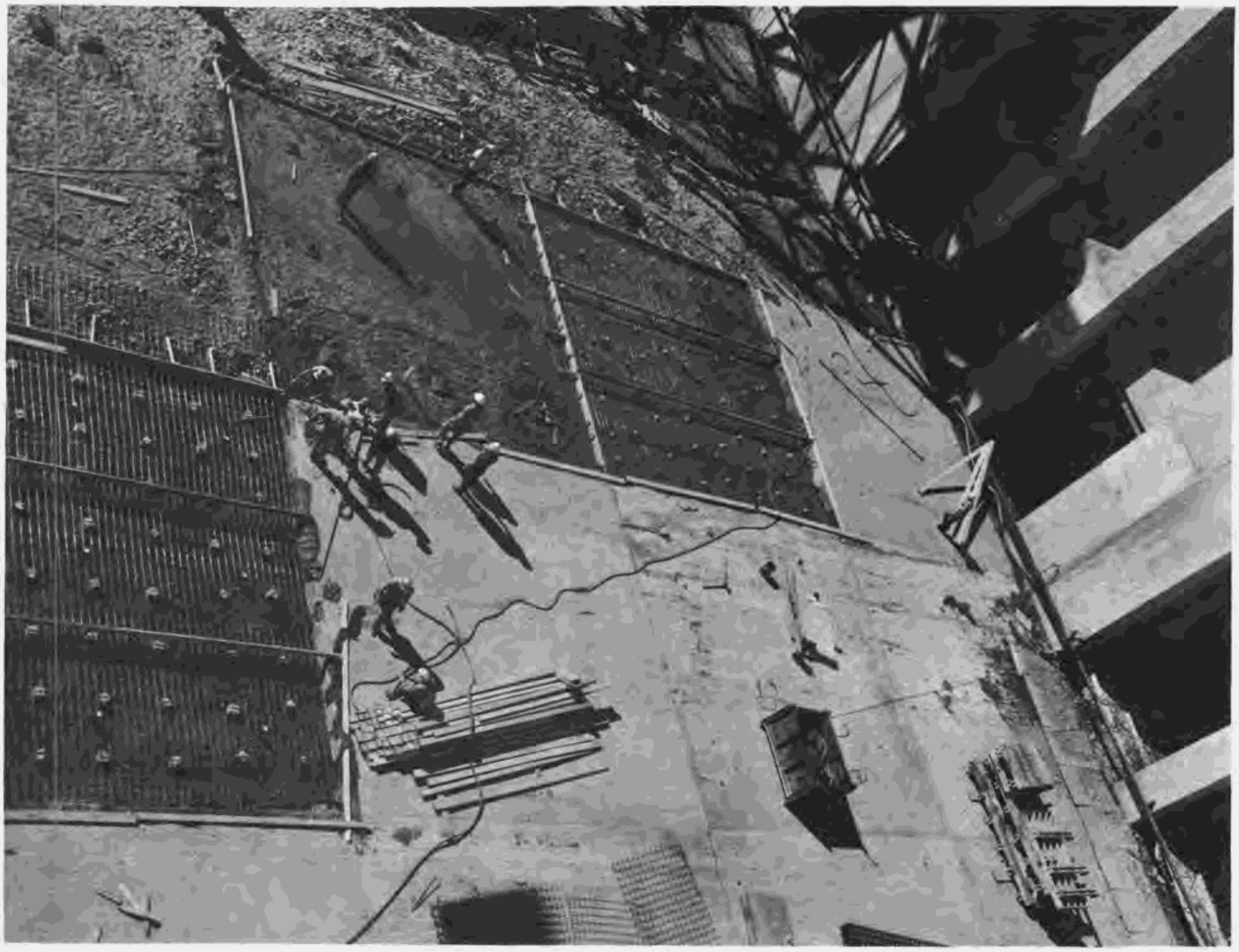


Figure 280.—Concrete being placed in the tailrace in front of unit bays 7 and 8 of the powerplant. P557-420-6457, October 12, 1961.

subcontractor Judson Pacific-Murphy Corp., a subsidiary of Yuba Consolidated Industries, Inc.

The first shipments of steel consisting primarily of welded steel plate arrived at the jobsite in April and May 1959, amounting to some 2 million pounds. Further deliveries were deferred for about 6 months and an additional 2.82 million pounds of steel was shipped over a 3-month period ending in March 1960. Subsequently, a change in the structural steel requirements added 383,000 pounds and increased the structural steel erected into the powerplant to the final total of 5.2 million pounds.

Erection of steel began in unit bay 1 in June 1961 (fig. 281), and was completed in July. This work was then temporarily suspended until September for completion of intermediate concrete construction. A rubber-tired mobile crane was used to handle the steel members, which were field erected with bolted connections. The crane boom was equipped with a boom extension for erection of the control and office area superstructure over unit bays 1 and 2. Work was resumed in September 1961, and the erection of the superstructure steel was completed in January 1962. Items of special interest on the superstructure include exposure of the outside steel column flanges with concrete exterior walls placed between the columns. Given the option to use precast or cast-in-place walls, the contractor elected to construct the walls in place. Also, cast-in-place roof slabs were used for the machine shop and service bay portions of the structure for protection from falling rock as these bays are adjacent to the canyon wall.

The roof of the unit bays, except unit bay 3, consisted of precast concrete decking, waterproofing membrane, and lightweight cover slabs. These precast concrete decking units were covered with a vapor barrier and a five-ply membrane, and the lightweight cover slabs were 3 inches in thickness. Some difficulties were experienced in protecting the membrane from damage during installation of the cover slab and related roof work. Unit bay 3, which would also be a visitors' observation deck, was constructed with a 4-inch regular concrete deck slab, placed on the structural steel, and a five-ply membrane and 3-inch-thick cover slab of regular concrete.

In an amendatory agreement dated May 7, 1962, in order to insure readiness of the powerplant for work under the completion contract, the contractor agreed to complete the walls, floors, structural roof slabs, and all interior installations for unit bays 1, 2, and 3 of the powerplant structure by June 15, 1962; and the same work in unit bays 4 through 8, the service bay and the machine shop bay of the powerplant structure, and the paving of the powerplant parking area by August 15, 1962. As the completion date for all work in the powerplant was established by the specifications as September 23, 1962, meeting the earlier completion date involved additional cost to the contractor for additional materials, labor, equipment and other expenses, and the contractor was reimbursed $128,000 for expediting this work. All work pertaining to unit bays 1, 2, and 3 was accepted on July 15, 1962; the remaining work under the amendatory agreement was substantially complete and accepted on August 15, 1962; and all remaining work involved in the powerplant construction was completed and accepted on September 23, 1962.

216. SECOND-STAGE CONCRETE. Concrete placed under the completion contract included blockout concrete, a control cable entry structure and certain lined sections, and miscellaneous concrete in the powerplant and various other locations. This concrete work also included bonded concrete surfacing, the 4-inch floor surfacing, and 3-1/2-inch terrazzo underbed on the several floor surfaces in the powerplant and other structures. This concrete was batched at a plant in Page, Ariz., transported to the powerplant in transit mixers, and placed by conventional methods. There were several instances where the underbed in the generator floor and on the generator balcony was placed too high, despite properly set screeds, and it was necessary for the contractor to bush these surfaces to allow for an adequate thickness in finish terrazzo. Owing to delays in scheduling the terrazzo work, these rough areas caused inconvenience and additional maintenance.

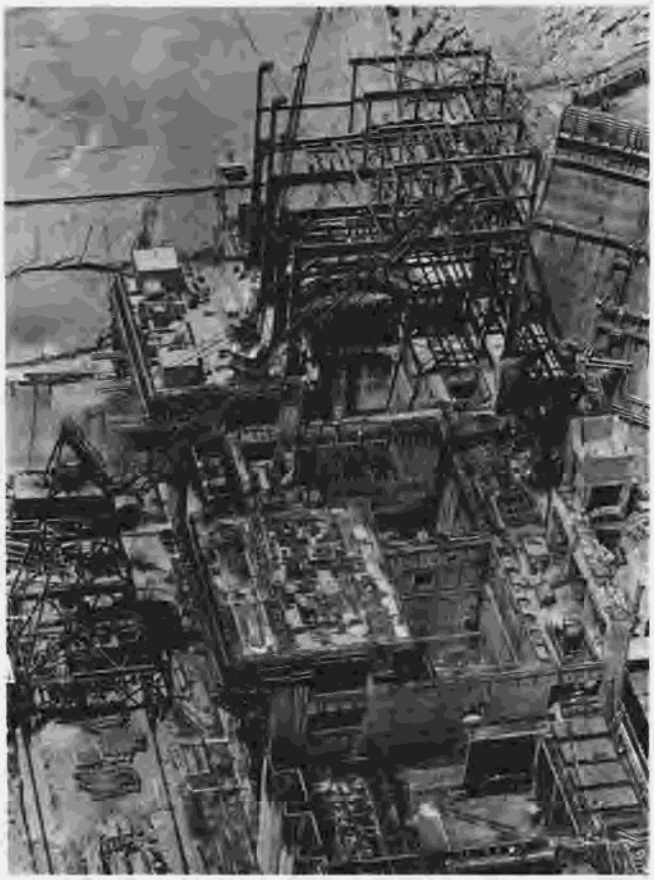


Figure 281.—View looking down on unit bays 1, 2, and 3 of the powerplant. Note steel erection in unit bay 1. P557-420-6105, June 22, 1961.

The completion contractor elected to use prepacked methods for placing second-stage concrete about the embedded parts of turbines (fig. 282), which was allowed under an alternative provision of the specifications. This concrete was placed by a subcontractor, Lee Turzillo and Cleveland Cement Co., Inc., a joint venture.

As this method is not often used, the aggregate placing and grouting methods utilized for this work are described below in greater detail than usual for second-stage concrete.

Prior to placing aggregate, the first-stage concrete was sandblasted and initial cleanup performed. Reinforcing around the draft tube liner was followed by the installation of the grout pipe and grout observation well piping. The grout pipe was installed on 5-foot centers for adequate grout coverage.

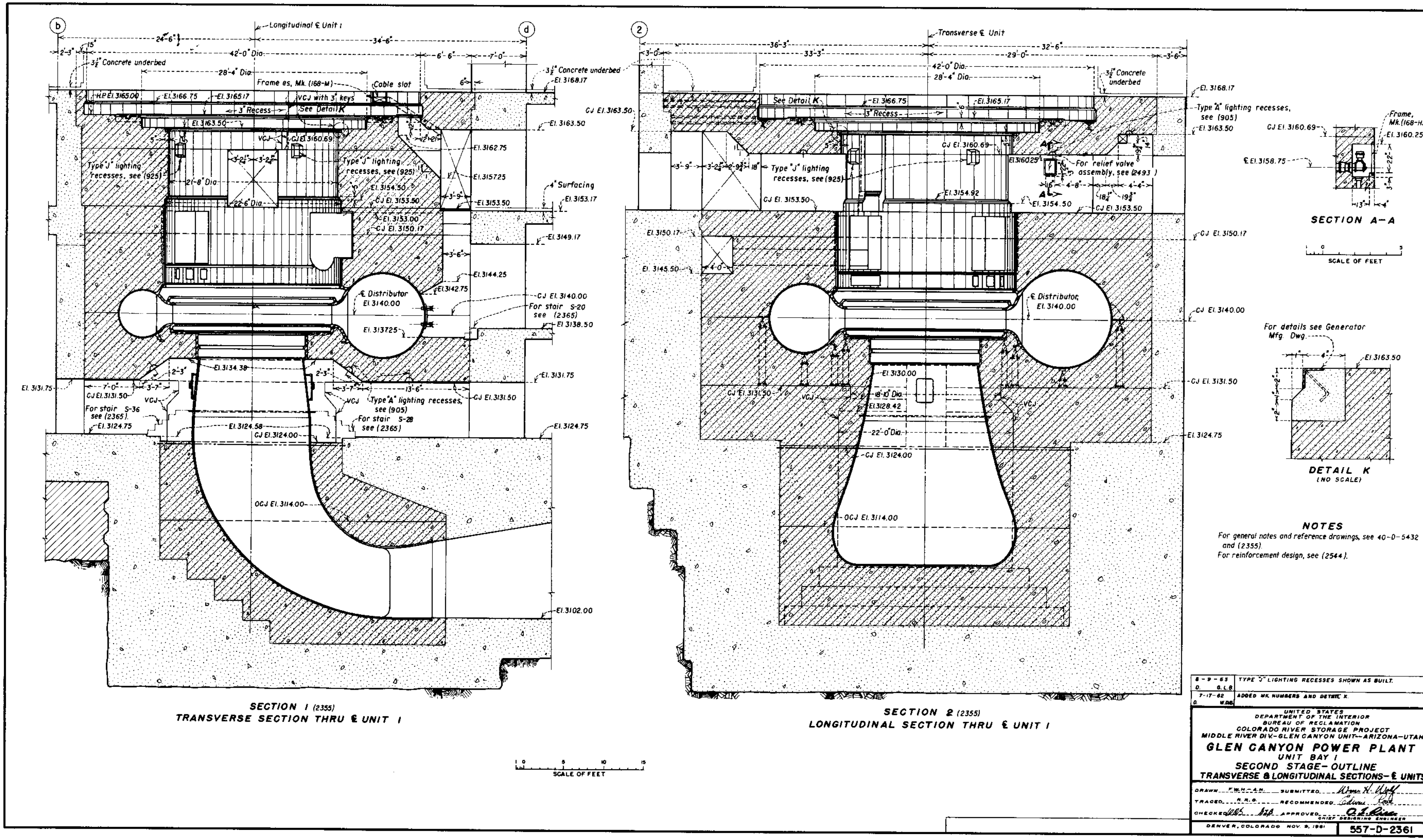


Figure 282.—Unit bay 1—Second-stage concrete outline.

The aggregate was dumped from buckets through a rubber "elephant trunk" into the areas on each side of the liner. The aggregate was then blown into place and into close contact with the liner. Aggregate was also dumped on the inside of the draft tube liner and placed through two 12- by 16-inch holes located approximately 10 feet from the upstream edge of the pier noses and about 15 feet apart in the bottom of the liner. Aggregate was also placed from the inside of the liner into the invert section between the first-stage concrete and the steel liner. The forms in the three outlet sections of the draft tube were left off until this portion was filled. Forms were installed and filling continued. The holes in the steel pier noses were adequate for filling with aggregate. Good results were obtained but placing of the aggregate was very slow.

Placing of the aggregate was usually done during one 8-hour shift and the aggregate was blown into place on the following two shifts. As preplacing progressed to springline and above, there were no difficulties in filling the blockout around the draft tube liner. In placing the aggregate in units 1 through 8, the time varied from 15 days maximum for unit 1 to a minimum of 2 days for unit 5. The average was about 7 days or twenty-one 8-hour shifts. Approximately 750 cubic yards of aggregate were placed around each liner. Very little breakage was noted during the placing of the aggregate.

Placing aggregate around the draft tube liner in unit 1 was started on December 11, 1962, and continued from elevation 3098.50 to elevation 3124.00. Three-inch maximum aggregate was used except for some 25 to 30 cubic yards of 1-1/2-inch maximum size used under the flat section of the draft liner. During the placing of aggregate around the unit 1 draft tube liner, the ambient temperature was low enough that a film of ice formed over the rock after washing. Forced draft air heaters were used to keep ice off the aggregate prior to the start of grouting. The heaters were used on the outside of the liner during the placing of the aggregate and inside the draft tube liner prior to grouting. The heaters inside the liners were not completely satisfactory even though the throat of the liner was covered.

The grouting operation of unit 1 was started on January 10, 1963, when the daily mean temperature had been 31° F. for several days prior to grouting. Temperatures were as low as 14° F. before grouting was completed on January 12, 1963, with a daily mean temperature of 17° F. The surface area at elevation 3124.00 was covered and kept heated in accordance with specifications requirements for winter protection. Temperatures of the draft tube liner were obtained by using a surface thermometer and recorded over a period of 10 days. When temperatures were obtained above 60° F., a fog spray was to be installed for cooling purposes. With the extremely low ambient temperatures it was not necessary to use fog spray for cooling purposes on unit 1 draft tube liner. Temperatures peaked at 60° F. in about 72 hours. Procedures were similar on the other units, except that fog sprays were needed to reduce temperatures in other units. Unit 8 liner was grouted on July 18, 1963, with an ambient temperature of about 92° F.

Two gages were installed in the draft tube liner during the grouting operation. With all holes plugged to hold the head of water there was at no time any pressures built up above 10 pounds per square inch. Generally, 6 to 8 minutes of pumping on each hookup would raise the grout lift approximately 20 inches. There were eight 5-inch holes in the draft tube liner to facilitate aggregate or concrete placing. The holes were plugged with wooden plugs when grouting was started. The 41 tapped holes were also plugged with steel plugs. The forms at each of the three openings between the draft tube liner first-stage concrete could not be made watertight and there was no assurance that there was a head of water ahead of the grout at all times.

Attempts were made to keep grout off the top of the construction joints. The pumping was stopped so that approximately two-thirds of the rock would be embedded. It was noted that excess grout on the surfaces would tend to peel off in wafers even though the surfaces were flooded for curing. Escape of gas through the top of the grouted lift continued for 3 or 4 days from the buildup of hydrogen gas resulting from the aluminum powder in the grouting admixture. After the cores were taken there was some evidence of entrapped air in the core.

Equipment used for pumping were grout pumps and a 2-cubic-yard shop made agitator mixer. Two-inch grout lines were used, with a pumping distance of approximately 500 feet and approximately 50 feet of 1-1/4-inch hose to each hookup. A wye connection with two lines from a 2-inch supply line was used. As many as four hookups were made at a time with two pumps operating. Fair results were obtained by alternately pumping and progressively working over the area. Some difficulties were experienced because of the operating condition of some equipment and use of inexperienced personnel. Several 6-inch cores were obtained from predetermined locations in units 1, 2, 3, and 6. Cores drilled indicated that good results were obtained in filling the voids, based on only short sections of core recovered, and that there was good bond to the aggregate.

Grout was batched into transit mixers on the rim near the south portal of the powerplant access road tunnel and transported through the tunnel to the agitator mixer. Grout temperatures usually ran from 73^{o} to 76^{o} F., but occasionally temperatures of the grout during the summer months ran as high as 94^{o} F. The higher temperatures presented problems of getting grout pumped to locations where needed. Temperatures were kept below 90^{o} F. by the use of crushed ice in the mixing water. The grout lines from the pumps were covered with burlap and kept wet when pumping. Grout slump was checked with a grout flow cone.

When the forms were stripped in the draft tube liner sections between the steel liner and the first-stage concrete, the surfaces resembled a terrazzo finish. The weight of aggregate against the forms appeared to keep the grout from completely covering the aggregate in contact with the forms. External form vibration was exercised but with poor results where much aggregate was overhead. Good results were obtained from external form vibration on the vertical faces. As soon as forms were stripped the imperfections such as surface irregularities were repaired and the she-bolt holes plugged. In order to obtain an F3 (specially formed) finish, the contractor hand-stoned mortar into the terrazzo-looking surfaces. The mortar application was kept on continuous water cure for a minimum of 7 days and good results were obtained with no discoloration.

After the preplaced aggregate was grouted and cooled, the liner was sounded for drummy areas. The 41 one-inch holes in the liner section were water tested at a pressure of 10 pounds per square inch. Only a few holes were interconnected. Only a very small amount of neat grout was used; possibly one-half cubic foot was used to fill the voids in unit 1 draft tube liner. All draft tube liners, units 1 through 8, were water tested and there was no appreciable grout take at 10 pounds per square inch. An occasional hole would be interconnected and show up on water test but would not take grout.

The second placement, from elevation 3124.00 to elevation 3131.75 was conventional concrete with blockout section around the upper draft tube liner. The blockout section from elevation 3124.00 to elevation 3131.75 and to the springline of the spiral case at elevation 3140.00 was filled by the prepacked method.

After joint cleanup, installation of reinforcing steel and embedded items, the grout intrusion piping was installed. This installation included piping for void grouting of the apex between the spiral case and the stay ring. Three-inch maximum aggregate was placed high around the spiral case, leaving enough area adjacent to the case to dump 1-1/2-inch maximum size aggregate. The 1-1/2-inch maximum aggregate was kept to a minimum of approximately 50 cubic yards. Better results were obtained by keeping the 1-1/2-inch aggregate confined to as small an area as possible to facilitate moving the rock into place by blowing. The aggregate was placed in contact with the stay ring and the underneath side of the spiral case by the use of compressed air.

The same procedure as outlined previously for grouting the draft tube liner in unit 1 was used in pumping the grout into the prepacked sections around the spiral cases of units 1 through 8. In one instance on unit 2, a fairly large section did not take grout and it was necessary to repack the aggregate and regrout the voids. The ungrouted area was investigated, additional jacks were placed, and the vents and returns indicated that the grouting was adequate. Grouting of the third placement in unit 1 was started June 14, 1963, and required 26 hours to complete. Unit 2 required 28 hours and unit 3 required 21 hours to grout. The blockout around the upper draft tube liner from elevation 3124.00 to elevation 3131.50 was pumped first and then the grout pipe around the edge of the turbine blockout closing in on the high points of the spiral case was pumped alternately and progressively to get a complete seal. All of the grout pipe in each segment was hooked for grouting if no grout return occurred from other hookups. After all grouting was completed, the 1-1/2-inch line that was installed in the apex between the stay ring and the spiral case was hooked to grout any possible voids. The 1-1/2-inch supply line was left with a hydrostatic head of grout of about 10 feet above elevation 3140.00.

During the grouting operation the spiral case was being prestressed at 225 pounds per square inch. Temperatures were recorded from the time grouting started until the spiral case was embedded and water was circulated in sufficient quantity to maintain a 5^{o} F. minimum temperature rise.

217. ELEVATORS IN POWERPLANT. Two elevator shafts were provided in the right abutment or control area end of the powerplant. One was for a 10,000-pound-capacity passenger elevator and the other was for a 2,000-pound-capacity passenger elevator. The elevators were furnished and installed by Pacific Elevator and Equipment Co., of San Francisco, Calif., under solicitation No. DS-8443. Installation of the elevators was started as soon as the hoistway and machinery rooms were made available on May 15,

1963, and all work under the contract was accepted as substantially complete December 26, 1964. Completion date for the contract was extended 177 days to cover delay to the contractor caused by failure of the Government to make the elevator hoistways and machinery rooms available as specified. The cost of furnishing and installing the two elevators in the powerplant was $130,640. This contract also provided for installation of two elevators in the dam, which is discussed in section 192.

218. TERRAZZO AND TILE. Major areas designated to receive terrazzo floor finishes in the powerplant were the main generator floor, the visitors' walkway balcony floor, and the floors of the rest rooms and first aid rooms. Terrazzo floor finishes could not be placed on the main generator floor or visitors' walkway until the completion contractor was out of the way. However, it was necessary to finish the floors and walls before the completion contractor completed his work in those areas where fixtures and partitions would be mounted on them. Mainly, these areas were the floors and walls of the rest rooms and first aid rooms located throughout the powerplant and office area.

As it was impractical to award a contract for such a small amount of dispersed work, the placing of terrazzo floors and installation of ceramic tile on the walls of the rest rooms and first aid rooms was included with other terrazzo and tile work in the dam under specifications No. DC-6057 which was awarded to Marus Marble and Tile Co., of Greensboro, N.C. Major work under these specifications was previously discussed in section 195.

219. ROOF REPAIRS. The flat powerplant roof for units 1, 2, and 4 through 8 consisted of precast concrete roof slabs supported on the concrete walls and wide-flange roof purlins. These slabs were covered with a vapor barrier and a five-ply membrane waterproofing. This membrane waterproofing was covered with lightweight reinforced concrete cover slabs 21 feet 7 inches by 19 feet 9 inches. The joints between the slabs were filled with corkboard capped with joint compound. Wood cants and monel metal flashings were used at the edge of the roof and at expansion joints at bay lines.

After the powerplant roof was completed, leaks developed from undetermined causes. Repairs were made by Superior Roofing Co., Inc., of Salt Lake City, Utah, under specifications No. 400C-292. Work was started July 6, 1965, and was completed August 31, 1965.

Principal features of work in unit bays 1, 2, and 4 through 8 consisted of removing joint compound and sponge-rubber strips from existing expansion joints and tooled grooves, cleaning grooves and joints to be refilled with corkboard, joint compound and calking compound, and furnishing and placing bonded sheet neoprene cap strips over prepared joints complete with wood cants as required. Roof repairs for unit bay 3, the service bay, and the machine shop bay were less urgent and were deferred until a later season.

Order for changes No. 1 was issued to change the method of preparing and refilling joints to be covered with bonded sheet neoprene cap strips, increase the width of the neoprene cap strips in some locations, and add joints to be covered with neoprene sheets; and (2) to apply sealer, additional primer, and liquid top coat along the edges of the neoprene roof strips to obtain a satisfactory bond of the topcoat. An additional 20 calendar days were allowed for completion of the contract because of these changes and additions.

220. GLASS DOORS. After self-guided tours of the dam and powerplant were started, it was decided that at a number of locations on the tour route full length glass panel doors should be substituted for the existing all-metal doors.

Cedar Glass Co., of Page, Ariz., was awarded the contract under specifications No. 400C-353 for the replacement of existing all-metal doors. Aluminum glass panel entrance doors were installed as follows: Two pair were located in the dam at the downstream adits to the powerplant, four pair at powerplant walkways at elevation 3188.5, two pair at elevator towers at the top of the dam, one pair at the tunnel entrance from the visitor center at the top of the dam, and one pair at the entrance to the unit 3 powerplant observation deck. Two pair of steel doors with glass lights approximately 23 by 34 inches were installed in the wall openings between rooms 1101 and 1102 of the powerplant office area. All work was completed and accepted November 26, 1967.

221. FLOOR REPAIRS. There were several instances where the U-3 (steel-troweled) finish on the concrete floors in the powerplant was damaged by contractors' forces as a result of inadequate precautions to protect these finished surfaces. Some of the floor areas were damaged with pock marks, spauls, and gouges varying in size from less than 1 inch up to 9 inches in circumference and depths up to nearly an inch. Some of these areas were repaired using conventional methods with good results. However, late in 1962, the contractor desired to find a satisfactory method of repair that could be applied more easily and produce a color that would more nearly match the existing concrete finish.

Several brands of epoxy cement mixtures were tried before finding one that proved satisfactory. The epoxy cement was furnished in 1-gallon containers of a part A and a part B, separately. Owing to the rather high material cost, the small containers and small mixes were recommended to eliminate waste. The areas were thoroughly cleaned with wire brushes to remove laitance or any other foreign material in small areas, or by bushing in the larger areas, and the area blown clean. Applied only to spauled areas that were thoroughly dry, equally mixed proportions of epoxy material smoothed with the patching trowel tended to leave the desired smooth finish, but the epoxy "pulled" enough to leave a slight depression in the center of the wet material.

The best results were obtained by leaving the patching material slightly high in the depressions, followed by dry grinding with a power stone of No. 120 grit. The epoxy was ground any time from 48 to 72 hours with good results, and the No. 120 grit stone removed the high spots with no damage to the surrounding U-3 finish. In practice, this method of grinding apparently speeded up the repair work slightly because not so much time was required in smoothing out the newly placed material. The epoxy was applied with an air temperature of 40° to 50° F. in early 1963. The patched area could be opened to normal foot traffic after several hours. In a few days, the patch area had blended with the surrounding areas and was difficult to detect except in the larger repair areas or when the floor was wet. Following an extended period of observation, both the bond and appearance are satisfactory on the repairs made on floors on the interior of the powerplant. Repairs made in areas subject to outside weather conditions are peeling from the concrete surface and will eventually require additional repairs.

222. PENSTOCK SUPPORT AREA. Excavation for penstock support footings was started in April 1960. By order for changes No. 5 the contractor was directed to excavate a minimum of 5 feet into sound rock for these footings. Concrete placement started in May 1960, with placement of 5-foot lifts for supports No. 2-B and 2-C. The first two lifts were 5 feet in height, and after that the H-frame lifts were 10 feet in height. Construction of the penstock supports proceeded along with concrete construction in the dam and powerplant. Beside the A supports which were on the face of the dam, there is one support each for penstocks No. 1 and 8, two supports for penstocks No. 2, 3, 6, and 7, and three supports for penstocks No. 4 and 5. The C and D supports were constructed to elevation 3136.84, and the B footings, over which a vault was later constructed, extended to elevation 3140.0. During construction of the footings, backfill was kept to within 50 feet of the top of all footings at all times. All penstock support footings were completed in November 1961. The concrete vaults in the B supports were completed during 1962 (fig. 283).

Figure 283.—Aggregate backfill placed between dam and powerplant. Trenches are excavated between penstock piers for tile drain pipe. P557-420-6773, February 21, 1962.

The horizontal penstock sections were placed and bolting of the expansion joints and sleeve-type couplings was completed in September 1962. The area around the supports was backfilled (fig. 284) with free-draining material excavated from the riverbed and stockpiled at the downstream cofferdam. Backfill was delivered to the area by conveyor through the powerplant, was spread by a tractor with dozer blade, and was kept within 50 feet of the top of the supports as the supports were constructed.

When backfill in the penstock support area reached elevation 3110 a 2-foot gravel blanket was placed over the area to elevation 3112. Drainage pipe was laid within the gravel blanket and backfill operations were continued. Select backfill material was placed under the horizontal penstock sections. Compaction of the backfill in this area and construction of the drainage

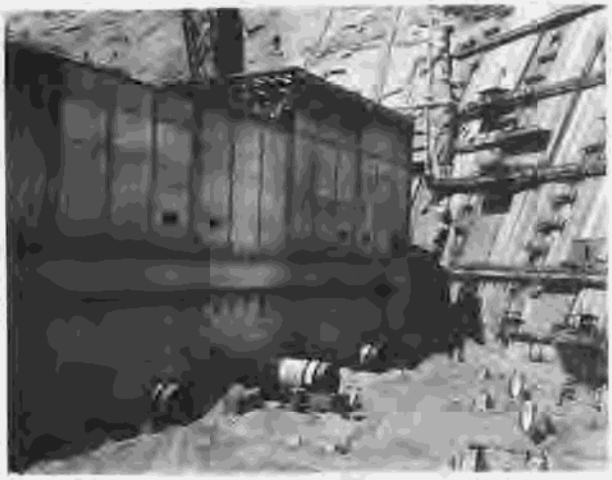


Figure 284.—Construction view of upstream face of powerplant and downstream face of dam. Note penstock installation from powerplant to dam. P557-420-7116, June 7, 1962.

system was done under order for changes No. 6. Compaction to 70 percent of laboratory maximum dry density was accomplished by four passes of a crawler-type tractor weighing approximately 40,000 pounds.

223. POWERPLANT ACCESS TUNNEL. The powerplant service, or access, road tunnel was excavated from both ends beginning in August 1957. Frazier-Davis Construction Co. of St. Louis, Mo., subcontractor for all tunnel work, excavated the lower (upstream) section of the tunnel and subcontracted the upper (downstream) section to Gibson-Roberts, Inc., of Yardley, Wash. By the end of 1957, the tunnel had progressed 907 feet from station 95+00 at the upper end and 1,129 feet from station 1+12 at the lower end.

The original plan for the tunnel called for a 560-foot section of the roadway to be in opencut along the talus from station 47+00 to station 52+60. Because of the jointed nature of the rock at the intermediate portal locations, it was thought better to relocate the opencut roadway into the canyon wall and the opencut section was therefore eliminated. The realined tunnel was increased in length by 667.76 feet and an equation was necessary at station 64+01.23 BK=57+33.47 AHD. The horizontal distance through the tunnel equals 10,056 feet. The tunnel contains 18 adits at approximately 500-foot intervals and they vary in length from 80 to 200 feet. All adits are 12 feet wide by 10 feet high to the A-line and have an arched crown extending 2 feet at the centerline.

The tunnel is 20 feet wide at the A-line and, from a point 12 feet 6 inches above the A-line floor the top section is arched on a 10-foot radius. Excavation was accomplished by blasting methods. Generally, eighty-two 1-1/2-inch-diameter holes, 12 feet deep, were drilled on 2-1/2-foot centers and a burnt cut used as a burden breaker. An average of 350 pounds of 50 percent blasting powder was used in a full face round. Powder holes were drilled from a two-floored, rubber-tired, drilling jumbo. After the dayshift had drilled and blasted, muck was removed by the swingshift with an overhead loader. The muck was loaded into front-dump trucks and wasted into the canyon through the adits. After the muck was removed, the tunnel walls and ceiling were scaled to remove any loose rock. The average depth per round was 11.4 feet.

The two tunnel crews, one working from each end, met at station 54+11.3 on June 24, 1958. Because the rock encountered in the tunnel was much more jointed than was anticipated and had a greater tendency to break up, the number of steel ribs actually required was five times the number originally estimated, and the number of roof bolts was 14 times those estimated.

The drainage system consisted of an 18-inch corrugated metal pipe culvert from either side of the upper portal down the centerline of the tunnel and out the first adit at station 89+98.

Crushed rock base and bituminous surfacing was placed on the tunnel roadway in April 1959. Excavation and placing of crushed rock base for the access from the Bitter Springs Highway to the downstream (upper) portal of the service road tunnel had been completed in 1957.

Concrete lining was placed at the lower (upstream) portal of the tunnel from station 1+10 to station 1+30 during April 1959, and the walls at the upper (downstream) portal were lined from station 95+00 to station 94+72 during that month. The arch and exterior face were completed early in 1960 and concrete was placed above the portal to hold some loose rock in place. The difference in elevation between the two tunnel portals is nearly 750 feet, and the varying grade through the tunnel reaches a maximum of 8 percent.

CHAPTER XV. *Construction*—COMPLETION OF DAM, POWERPLANT, APPURTENANT STRUCTURES, AND INSTALLATION OF GENERATING EQUIPMENT

A. INSTALLATION OF TURBINES AND GENERATORS

224. HYDRAULIC TURBINES AND GOVERNORS. (a) *General Description.*—The hydraulic turbines were furnished by Baldwin-Lima-Hamilton Corp. of Eddystone, Pa., under invitation No. DS-5234. This was the largest single turbine contract in the Bureau of Reclamation history at the time (1960). The eight hydraulic turbines installed in Glen Canyon Powerplant are the vertical-shaft Francis type, with field-welded plate steel spiral cases and elbow-type draft tubes. Each turbine has a capacity of 155,500 horsepower, at full gate opening, when operating at 150 revolutions per minute under a net head of 450 feet. At a head of 510 feet and an output of approximately 150,000 horsepower, the required warranty efficiency was 90 percent.

The governors for regulating the speed of the turbines are of the oil-pressure, cabinet, actuator type with electric-driven speed-responsive elements. They were furnished by Pelton Division of Baldwin-Lima-Hamilton Corp. under invitation No. DS-5562. Each governor is a complete unit, its principal parts consisting of the actuator with its relay valve, indicators, and controls, restoring mechanism, motor-driven pumping units, pressure tank, sump tank, and oil piping used for regulating the speed by controlling the gate opening of the turbine. The governors are rated at 441,700 foot-pounds at an oil pressure of 250 pounds per square inch, corresponding to the capacity of the turbine servomotors.

The turbine installation work was primarily the responsibility of the completion contractor, but the supply contract provided for certain field installation work by the turbine manufacturer in the assembly, alinement, and hydrostatic testing of embedded parts. This division of the work between the supply and completion contractors was sometimes difficult to handle in the field in that there were early problems in differentiation in responsibility for the work between crews. However, as the same subcontractor was employed by both the completion contractor and the turbine manufacturer, the problems were a matter of cost distribution rather than actual work conflict.

The turbines and governors were built by experienced manufacturers of this type of equipment and there were no major technical problems at the factory level. Portions of the turbine parts were manufactured by S. A. Cockerill-Ougree of Seraing, Belgium. These parts consisted of the runner and runner caps; main shaft and sleeve; servomotors, rods, levers, links, pins, operating ring and wicket gates; head cover; distributor ring; stationary and rotating seal rings; guide bearing, bearing support and bearing cover; oil catcher; stuffing box, gland, and lantern rings, walkways, stairs, and handrailings. These are essentially all of the nonembedded parts. Early difficulties were experienced in casting some of the turbine parts but these were resolved by the subcontractor as the work progressed. In addition there were significant delays in delivery of the rotating parts of the turbines which caused field problems in meeting installation schedules. An item of note was that Baldwin-Lima-Hamilton's bid was based on a combination of foreign and domestic subcontracting of the elements of the turbines, which drew protests from foreign and domestic bidders and was submitted to the Comptroller General of the United States for a ruling before award of the contract.

(b) *Installation Procedure.*—Installation of the draft tube liners and placement of second-stage concrete were the responsibility of the completion contractor. The draft tube liners were field welded in sections erected in place in the draft tubes. This method of individual welding of plates was selected by the subcontractor, Chicago Bridge and Iron Co., as the most economical method, both with respect to fabrication and field erection. Basically, the draft tube liners consist of two portions, conveniently referred to as upper and lower sections. The shorter upper section is flanged at the top for bolting to the discharge ring, while the lower end makes a field-riveted joint at the connection to the lower section of the draft tube liner. The lower draft tube liner was set on pipe jack supports as it was erected and welded, up to elevation 3124.51. The upper piece of this lower section, referred to as the makeup section, was not set at this time. Tierods and jack supports were installed and tightened to hold the liner rigidly in place. After final checking for alinement and grade, second-stage concrete was placed around the liner to about elevation 3124.0 by prepacked methods. The next step was installation and positioning of all tierod anchors and jack supports for the casing and stay ring. The makeup section, or upper piece of the lower draft tube liner, was then placed in approximate position followed by the lowering of the upper draft tube liner section into approximate position, but the makeup section was not assembled at this time. A platform and instrument-supporting structure was fabricated at the site from pipe and steel plate in tripod form and was welded in place at the upper end of the lower draft tube liner. Figures 285 and 286 show the distributor ring and spiral case, respectively.

The purpose of this structure was to support a plumbline wire and an engineer's level used during the leveling of the stay ring flange. The discharge ring was in two sections; the upper flange was designed for bolting to the lower flange of the stay ring, while the lower flange mated with the flange on the upper draft tube liner. The upper draft tube liner was placed in position as a unit and the discharge ring was placed in

Figure 285.—Preparations underway for installation of distributor ring in unit bay 1 of powerplant. P557-420-8075, March 7, 1963.

position on jack supports. The discharge ring was closely leveled and centered to the plumbline wire, checked for roundness, cleaned, and then prepared for placing of the stay ring. The stay ring was brought in by sections, placed on its supporting jacks and temporarily bolted to the discharge ring. The centerlines and match marks placed in the shop were used to position the stay ring to lines established by the survey crews. The stay ring halves were made up with large shrink bolts which were heated and stressed to a predetermined amount to make a tight joint between stay ring halves. The spiral case and extension sections were placed in the turbine pit in approximate position on jacks and blocking.

At this point in the installation the work was turned over to the supplier, Baldwin-Lima-Hamilton Corp., for field welding and hydrostatic testing of the spiral cases. Although there was a change in contract responsibility, the field crews remained the same since Chicago Bridge and Iron Co. subcontracted the turbine assembly erection work from both the completion contractor and the turbine manufacturer. Through the turbine supplier, Chicago Bridge and Iron Co. made a proposal to change the spiral case installation to an all-field-welded, no stress relieving, assembly of spiral cases. This proposal involved a very complete welding procedure supervised by Chicago Bridge and Iron Co.'s welding engineers. The use of radiographs on all welded joints provided quality control of production welding. Baldwin-Lima-Hamilton Corp. assumed complete

Figure 286.—Powerplant distributor tube for unit bay 2 being welded. P557-420-8176, April 11, 1963.

responsibility for the adequacy of this method up through the hydrostatic testing of the spiral case.

It was reported that this was the first time the Bureau had allowed this method of installation of spiral cases. From an installation point of view, the method proved to be very satisfactory, primarily due to the high quality of welding and supervision supplied by the subcontractor. When the spiral case had been assembled, the spiral case was alined and leveled by the use of turnbuckles, jacks and supports, as required by the turbine manufacturer. The head cover flange of the stay ring was leveled by the use of micrometer level attachments to a surveyor's instrument. This level and alinement was maintained during hydrostatic testing and placement of concrete.

The next step in the erection was the hydrostatic testing of the spiral case. The opening at the stay vanes was closed by the use of a temporary test ring supplied for this purpose. The tests of the spiral case proved that the fieldwork was completely watertight, and the stressing of the case through the working range of pressure proved the structural quality of the work. Upon satisfactory completion of the hydrostatic test, the pressure in the cases was brought to 225 pounds per square inch and maintained until the case was embedded and the concrete was cured. On completion of the hydrostatic test and checking of the tie-downs, jack supports, alinement, and levels, the spiral case was turned back to the completion contractor for the remainder of the erection and assembly of the turbines.

The pit liner was placed on the spiral case with particular attention given to location of servomotor bases. This was difficult and time consuming during installation of the first three units, as the bases were refitted in the field to the range of desired tolerances; the problem was corrected at the fabrication shop on the last five units. The external piping and grout piping with vents were installed, a final check of all items to be embedded was made, and the unit was released for concrete placement. After the concrete had been placed and cured around the spiral cases, a final set of micrometer levels was taken on the stay ring flange. The adequacy of this method of anchoring and placing of concrete around the spiral case and pit liner was confirmed by these final level readings.

The bottom ring was placed, centered, leveled and bolted down, but not doweled at this point. The lower platform of the boring rig was placed in position on the top flange of the draft tube liner. This platform later proved to be too large to be removed after lower stationary seals had been placed and required extensive repair work in unit 1. The lower and upper stationary seal rings were placed in the unit at this time to be later hung from brackets on the head cover. The head cover with four wicket gates spaced 90° apart was placed in the unit on the stay ring flange. The head cover was carefully positioned to the unit centerline and was alined so that the four index wicket gates were plumb and with the proper clearances. The remaining wicket gate bores were checked with a tribar and were sanded or buffed for proper clearance. The head cover and bottom ring were then drilled and reamed for dowels. The guide bearing housing, guide bearing and packing box were alined to plumb wire centerline and drilled and reamed for dowels. The bottom platform of the boring rig was alined at this time and bolted securely. The head cover, wicket gates, guide bearing, guide bearing housing, and packing box were removed from the unit. The next operation was the placing of the portable boring rig for machining of the stationary seal rings. The basic four-wire plumbing, combined with rotation checking, was adapted to plumb the main shaft.

The stationary seal seats were machined and the seats were bolted in position. The inside diameter of the wearing rings was machined to give proper clearance with the rotating seals of the runner.

The head cover and boring rig were removed from the unit. The main shaft and runner were assembled on the generator deck using the coupling bolts stretched by 0.012 inch to give proper stress in the bolts. The throat ring was installed on top of the upper draft tube liner. The runner and main shaft were then placed in the unit on the ledge provided by the throat ring.

The main shaft was plumbed and the runner was centered in the seal rings using shim stock in the runner clearance space. The wicket gates were placed in the bottom ring in their respective positions and the head cover installed over them. The remaining parts were then assembled in the turbine pit. The servomotors gate shifting ring, gate links, and guide bearing housing were assembled and adjusted. The turbine was then at a point it could be released for generator erection.

Table 4 lists the chronology for the turbine erection.

Table 4.—*Chronology for turbine erection*

Unit No.	Started turbine installation	Completed turbine installation	Unit placed in service (first generation)
1	9-18-62	1-16-64	9-4-64
2	9-20-62	3-9-64	9-22-64
3	9-27-62	4-14-64	12-11-64
4	10-16-62	6-26-64	2-10-65
5	10-24-62	9-18-64	7-19-65
6	10-26-62	11-24-64	10-13-65
7	11-5-62	5-3-65	1-21-66
8	11-2-62	5-11-65	2-28-66

The installation of the governors was under the direction of a factory erection engineer who performed all of the work inside of the cabinets including the operation of the governor during the first start. The completion contractor's work consisted of moving the equipment to the powerplant, alining and leveling it in position, and then placing a grout pad at the base of the cabinets.

(c) *Operational Tests.*—The purpose of the operational tests was to determine that all phases of the installation work had been performed properly, all control devices had been checked and adjusted, and the equipment was ready for operation. As the generators and other equipment were involved in the turbine and governor operational tests, all testing was coordinated through the test coordinator. Preliminary tests were made before admitting water to the turbine. The plumb of the combined main shafts was checked from four plumblines at approximately 90° apart and the runner concentricity was rechecked. The governor and turbine wicket gates were operated several times, the gate squeeze was set, and the rate of movement and operation of all automatic and safety devices were checked and adjusted as required. The slow-closing devices on the turbine servomotors was adjusted to obtain proper rate of closure below the speed no-load positions. The timing of the wicket gates was set and their position recorded for use during other tests. The wicket gate timing could then be set without unwatering the spiral case. The runner clearances and shaft movement were measured under blocked servomotor conditions with full governor pressure applied.

The next set of tests were the bearing runs. After the penstock and spiral case had been filled with water, the unit was started on manual control from the governor cabinet. The unit was started at reduced speed and brought up to synchronous speed in several steps. Each speed was held until all bearing and oil temperatures had leveled off. The next set of tests were the dry-out runs and rotor balancing, all of which were conducted by the generator contractor. The last section of testing was the unit load and load rejections. The unit was loaded in increments of one-tenth gate opening and held for one-half hour up to full gate opening, which was held for 15 hours. When the load reached one-fourth, one-half, three-fourths, and full load conditions the load was rejected and the unit allowed to return to speed no-load position. All the turbine and generator data was recorded for each individual unit.

(d) *Repair Work.*—During the first months of operation of the turbines, it was noted that the stainless steel cladding on the runners was inadequate. Baldwin-Lima-Hamilton was notified of the deficiencies on February 25, 1965. On June 18, 1965, representatives of the Bureau, Baldwin-Lima-Hamilton, and one of the latter's subcontractors met at the Glen Canyon Powerplant. The purpose of the meeting was to inspect the runners and to develop a procedure to accomplish the repair work. It was determined the coating deficiency was the primary responsibility of the subcontractor supplier.

One of the problems at this time was to determine the thickness of the stainless steel coating, in

Table 5.—*Repairs to turbine runners.*

Unit No.	Operational hours prior to repair	Labor man-hours	Dates of repair		Pounds of Welding rod applied
			Start	Complete	
1	9,659	380	November 3, 1965	December 20, 1965	400
2	11,300	600	January 18, 1966	March 30, 1966	560
3	10,488	480	March 30, 1966	June 13, 1966	450
4	11,227	560	June 13, 1966	September 7, 1966	410
5	3,060	223	December 10, 1965	January 17, 1966	215
6	6,941	576	September 7, 1966	January 5, 1967	440
7	8,762	596	May 5, 1967	July 15, 1967	390
8	6,440	588	January 6, 1967	April 26, 1967	380

place, on the runners. The defects were apparent in units that had been in operation for some time; but in units that were not in service, or had only a small number of hours of operation, it was very difficult to determine coating thickness. An inspection determined that the repair work fell into two areas. Some areas were the responsiblity of Baldwin-Lima-Hamilton and others were the Bureau's responsibility. All parties agreed the work was required on all eight runners, but no workable basis was available to determine the amount of repair in advance. A device was developed by a Bureau employee to measure the thickness of the stainless steel cladding in place. The repair work was performed by Chicago Bridge and Iron Co., the installation subcontractor, on a time-and-material basis. Table 5, on the preceeding page, gives the time and material involved in the repair in the Government's area of responsibility.

During early operations, the method of securing the throat ring to the upper draft tube liner was also found to be unsatisfactory. The bolts were sheared off during operation and the throat ring in unit 1 was discovered to be completely free during an inspection. The supplier was required to skip weld the throat ring to the upper draft tube flange—in addition to bolting—on all units and no further difficulties were experienced from this cause.

225. GENERATORS. (a) *General.*—The General Electric Co. of Schenectady, N. Y., furnished and installed the eight generators for the Glen Canyon Powerplant (fig. 287) in accordance with invitation No. DS-5522. The contract provided for furnishing, installing, and testing eight 125,000-kv.-a., 90 percent power factor, 13,800-volt, 150-r.p.m., 60-cycle, vertical-shaft, hydraulic-turbine-drive, alternating-current generators, as well as the associated direct-current excitation system with controls, control of generator cooling water system by flow modulation, and specified spare parts for the generation units. Fabrication of the eight generators was started at the company plant on January 3, 1962, and installation at the site on unit 1 began on December 26, 1963. Assembly and testing, except for final acceptance tests, were completed on February 28, 1966. Construction proceeded concurrently on the other generators. Unit 1 was placed on the line for the first time September 1, 1964; unit 2, September 18, 1964; unit 3, December 10, 1964; unit 4, February 9, 1965; unit 5, July 8, 1965; unit 6, October 6, 1965; unit 7, January 14, 1966; and unit 8, February 25, 1966.

Figure 287.—Interior view of powerplant. The eight generating units have a total installed capacity of 900,000 kilowatts. P557-420-12678A, September 22, 1966.

(b) *Rotor Assembly.*—The first generator shaft was lifted onto the rotor erection pedestal in the erection bay by one of the 300-ton overhead cranes on December 26, 1963. The rotor spider was next placed on the rotor shaft and bolted to the shaft flanges. A suitable scaffolding was erected around the spider for a working area and to carry the bundles of laminations for the rotor stacking. The overhead crane was also used to handle the 500-pound bundles of laminations. As the laminations were stacked, four presses were made to insure a tight mass. This was accomplished by placing pipe sleeves over the rotor rim bolts for the length of the unstacked portion. Rim stud nuts were then tightened on the pipe sleeves. During the final press, the rim studs were tightened with an impact wrench and all the rim stud nuts were lock welded.

Field poles were delivered as required for each unit and were checked for dielectric insulation resistance prior to installation. Those pole pieces that were low in value were internally heated, using electric welders or high-current rectifiers, until the proper dielectric value was obtained. The individual pole pieces were placed on the rotor with the powerplant overhead crane. They were held in place by vertical keys driven tight; the surplus key stock was cut off and stop plates were installed on both ends of the keys. Series field connections between the poles were silver-soldered and final connections were made to the slip rings by a bus system. The rotor was spray-painted with insulating varnishes before being installed in the stator bore. Figure 290 shows a closeup of the rotor for unit 8.

(c) *Stator Assembly.*—Each stator was shipped in three sections on flatcars to Flagstaff, Ariz., then transported to Glen Canyon by special truck. Stator erection began by placing the foundation pedestals to proper elevation. The three sections were placed in approximate position, bolted together, rounded to proper diameter, and then bolted and horizontally doweled to the foundation pedestals.

Each stator section was shipped with the armature coils in place. At each location where the stator sections bolted together, eight makeup coils were installed by armature winders. The 14 coils originally installed at the factory were lifted and replaced after the eight makeup coils were added at each junction. All makeup and lift coils were heated to 85° C. surface temperature for 1 hour, which rendered the insulation flexible. Series coil connection phase jumpers and the bus sections in the splits were silver-soldered. The completed coil assembly with buses and jumper was given two coats of insulating varnish. The completed stator assembly was centered in the pit with reference to the center of the turbine shaft and was set to a grade elevation 0.035 inch high to allow for settlement of the assembled machine.

(d) *Assembly of Major Parts.*—The major assembly work began after the rotor, stator, lower bearing bracket, and upper bearing bracket assemblies were completed. The lower bearing bracket and upper bearing bracket were assembled in the field and all connections were hot-bolted together. The lower bearing bracket, with the lower cover plates in place, was placed on the foundation plates, centered, leveled, and bolted into place. This was used as a working platform for the stator erection. After the stator was rounded out, centered, and winding completed, the brake jacks were placed and leveled to the proper elevation, which prepared the unit for placing of the rotor assembly.

The completed rotor (fig. 288) was connected to the 580-ton-capacity lifting beam, which was supported by the four 150-ton hooks on the two 300-ton overhead cranes. The assembled rotor was lifted off the pedestal and suspended in the erection bay where workmen cleaned the coupling face. The rotor was then moved to the stator bore and carefully lowered into position, resting on the brake jacks. In placing the rotor, masonite guide slats were used to keep the rotor from contacting the stator bore.

Figure 288.—Rotor for generating unit 8 being lowered into stator. P557-420-11335, July 21, 1965.

After the rotor installation was completed, the upper bearing bracket was moved into place. This bracket had been previously centered on the stator and doweled. The thrust bearing base and other bearing parts were assembled on the upper bearing bracket. The thrust block was preheated for 24 hours, then placed on the shaft. The thrust block retaining ring was then bolted into position. The thrust bearing oil tub with exciter support was placed in position.

The brake jack locking cams were released and the rotor assembly was allowed to become suspended on the thrust bearing. The coupling was lined up by adjusting the upper bearing bracket position and was then made up by the contractor. All coupling bolts were stretched 0.010 inch. The alinements were checked by the turbine erector, the generator erector, and the inspector.

After the generator shaft alinement was completed, the shaft was blocked into position and then the upper and lower guide bearings were adjusted for proper clearance. As the major parts assemblies were being installed, the air housing was assembled and welded.

The generator exciter came in two parts, a field assembly and the rotor armature. The rotor armature and exciter ventilating fan were bolted on the generator shaft, and the field assembly was placed in position. The generator contractor installed the amplidyne voltage control unit, the field rheostat assembly, and the controller of the cooling water modulating system. The completion contractor installed the cooling water piping, the generator neutral grounding transformer, and the permanent magnet generator. Figure 289 shows various stages of generator erection.

Figure 289.—Interior of powerplant during generator installation, as seen from the service bay. P557-420-10488, January 27, 1965.

(e) *Contractor's Field Organization.*—The field organization of the contractor normally consisted of a superintendent of erection, an assistant superintendent of erection, an office manager, a foreman millwright, a foreman electrician, a foreman ironworker (rigger), eight millwright journeymen, six electrician journeymen, one ironworker journeyman (rigger), one carpenter journeyman, one pipefitter journeyman, two laborer-truck drivers, and two armature winders. This crew produced 69,640 man-hours of labor in the installation of the eight generating units, or an average of 8,705 man-hours for each unit. The reported gross field cost for the installation labor, including administration and supervision, was about $388,522.

(f) *Contractor's Construction Plant.*—At the time of generator installation, construction of the powerplant had progressed to the stage where the contractor was able to proceed with a relatively small construction plant. The specifications provided that the Bureau would provide overhead crane service, rotor lifting beams, compressed air service, and 480-volt electric power service.

The construction plant consisted of one general purpose truck, one pickup, one motor bus, two fixed air compressors, two engine lathes, two portable electric welding machines, two pedestal grinding stands, two fixed drill presses, two sets oxygen-acetylene gas combination cutting welding units, two high-current electric bracing tong units, one portable boring bar combination unit, one heavy duty alternating-current direct-current rectifier unit, one 35-kilovolt high-voltage testing transformer, twenty-four 50-ton dual-lift screw jacks, eight 50-ton center-hole hydraulic pullers, numerous assorted sizes and lengths of wire rope slings, four heavy-duty pneumatic impact wrenches, six sets of chains hoist and pullers from 3 to 10 tons, one engineers transit, two engineers precision levels, three sets small tools consisting of electric and pneumatic portable drills, grinders, sounders, nibblers and impact wrenches, sockets, wrenches 3 inches down to one-fourth inch, end wrenches, machinist levels, micrometers, gauges, scales, etc., and one power-driven wood saw, one power-driven hacksaw, and a sufficient supply of special tools and instruments as required by the electrical, machinist, and millwright crafts.

The contractor made available, for a fee, one alternating-current high-potential testing transformer with control and power supply for the Government to perform the required alternating-current high-potential proving tests.

(g) *Work Procedures and Practices.*—The completion contract provided under specifications No. DC-5750 that the generator contractor would furnish the lower bearing bracket anchor bolt, lower bearing bracket foundation plates, stator foundation pedestal anchor bolts, stator foundation pedestal, which would be installed and to proper level by the completion contractor. The lower bearing brackets were shipped partially assembled. Two arms were hot-bolted to the center hub in the field. The physical dimensions of an overall width of 22 feet 11 inches made it impractical to preassemble this subassembly at the factory. The lower cover plates were assembled on the bearing bracket before placing it on the lower bearing bracket foundation plates. The bracket assembly was placed on the foundation, brought to elevation by checking from the turbine crown plate, and centered by using a tightwire from a tripod above the generator foundation to the turbine shaft true center. After the alinements were completed, the bearing bracket was doweled to the foundation plates, which were then grouted in place. The brake jacks and brake jack piping were placed into position on the lower bearing bracket, taking advantage of the clear working space.

The stator foundation pedestals were installed under specifications No. DC-5750. Care was taken to give the final grade on these pedestals with the reference to the top of the stator section. By doing this research on the stator section, shims were eliminated between the stator and the foundation pedestals.

The three stator sections were placed in the pit on the foundation pedestals. Premilled vertical shear horizontal keys were placed in the three splices to control the 17-mil penetration of the stator laminations when the sections are fastened together. (It was anticipated that these keys would be inspected for tightness after 3 or 4 years of operation. If not tight, the split through bolts should be retorqued as looseness at this point will develop a soft lamination assembly in the stator, eventually resulting in coil damage from either coil or iron migration.)

The stator section split bolts were tightened until the 17-mil penetration of the lamination was obtained. The stator section was rounded by establishing the true center of the unit using a tripod extending above the stator with the lower bearing bracket as the base. A tightwire was installed from the tripod upper bearing to the center of the lower guide bearing chamber in the bearing bracket. Micrometer readings were taken from this tightwire to the predetermined points on the inner circumference of the stator. Five measuring points were prepared about 6 inches from and on each side of the split. Measuring points were then prepared about every 20° around the inner circumference. At each location one point was about 3 inches from the top of the laminations, one up about 3 inches from the bottom, and a third in the middle of the lamination assembly. At the splits other points were located between the middle point and the upper and lower points. Five dual lift jacks were placed around the outside circumference of each stator section. Near the baseplate line, each jack was brought up to a light compression again at the stator frame. After each set of micrometer readings were taken of the stator inner circumference, the jacks were readjusted to bring the stator into proper conformation at the bottom ring, midsection, and top where it was practical to obtain this condition. Where eccentricity existed between the top and bottom ring, adjustments were made by raising the stator and shimming the pedestal contact to plumb the lamination form. The maximum shim installation was 25 mils on one pedestal.

Before releasing the jack holding the stator in position, the horizontal dowels were drilled and placed in the pedestal-stator junction and all foundation and mounting bolts were torqued to proper tension. A working platform was constructed on the lower bearing bracket to facilitate the installation of the lap coils.

(h) *Handrails.*—As originally constructed, the handrails on the top of the generator air housing enclosed only about one-sixth of the deck, providing a guarded passage from the access stairway landing to the exciter platform stairway and to part of the collector ring brushes. As maintenance workers needed access to the remainder of the deck, during routine maintenance operations, it was necessary to provide additional handrails to encircle the entire deck. Specifications No. DC-6441 provided for furnishing and installing this additional aluminum handrailing and for removing a portion of the existing handrail. Larsen Rigging and Equipment Co. of Salt Lake City, Utah, began work

February 7, 1967, on this modification and all work was completed and accepted February 24, 1967.

(i) *Some Operational Problems.*—Soon after initial operation of unit 1, oil was observed at the lower guide bearing. After various experiments the contractor added another vapor guard. This was done on all units and substantially dimished the oil leakage. An oil slinger was added on unit 8 and extended operation indicates that this will also be added to the other seven units.

A problem occurred with vibration of air deflector baffle plates in the air housing, resulting in cracking of some of the plates. The contractor extensively reinforced the plates in all units.

Figure 290.—Two 300-ton cranes with lifting beam moving unit 8 rotor from assembly pad to final placement location. P557-420-11333, July 21, 1965.

B. INSTALLATION OF MISCELLANEOUS MECHANICAL AND ELECTRICAL EQUIPMENT

1. Miscellaneous Mechanical Equipment

226. GENERAL DESCRIPTION. The turbo-generating units require a substantial amount of mechanical equipment in support of the plant operation. Two 300-ton cranes in the main generator room and a 75-ton crane in the machine shop are required to maintain the units. A complex oil storage and handling system is necessary to lubricate the mechanical equipment, as well as a separate system to service the insulating oil requirements of the electrical equipment. Large water pumps are also needed to dewater the various sumps, others to circulate the generator cooling water, and even larger booster pumps are required for domestic water supply purposes. A compressed air system was required for specific and general service use in operation and maintenance of the plant, and a carbon dioxide fire extinguishing system was necessary to protect the generators and the oil handling rooms.

227. 300-TON POWERPLANT CRANES. Overhead in the main generator room in the powerplant are two 300-ton traveling cranes (fig. 290) with a span of 71.5 feet. Each of the cab-operated cranes has two trolleys with 150-ton-capacity main hoists and 30-ton-capacity auxiliary hoists. The lifting capacity of the two main hoists on each crane may be combined with a lifting beam to produce a total capacity of 600 tons when the trolleys on each crane and the bridges of the two cranes have been locked together. These electric cranes are for use in installing and maintaining the turbines and generators and for handling materials in the service area. Each main hoist has a lift of 80 feet and is equipped with 12 parts of 1-3/4-inch-diameter wire rope and a sister hook drilled for a horizontal lifting pin. The auxiliary hoists are standard single-hook with four parts of 1-inch-diameter wire rope and have a lift of 100 feet. The 300-ton cranes were purchased under invitation No. DS-5260 by Yuba Consolidated Industries, Inc., of Benicia, Calif., and were installed by the prime contractor.

Erection of the 300-ton crane began in December 1961, in the Merritt-Chapman and Scott Corp. fabrication yard at the site. This yard work consisted of the structural steel assembly of the main bridge girders and placement of the trucks. The assembled bridge girders were placed on the powerplant crane rails by high-line cableway before the roof of the powerplant was installed. Most of the installation of equipment could then be performed with the bridges in place on the rails.

On April 11, 1962, the erection engineer representing Yuba Industries reported to perform acceptance testing of the cranes. The cranes were adjusted and run-in for the formal acceptance testing. Tests on the No. 1 crane started April 16. Because of a faulty hoist motor brush position, the test load dropped a sufficient distance to throw out the windings of the main hoist motor. The hoist motor was repaired and returned in July. The testing resumed and an auxiliary motor was damaged. The contactors were then found to be defective on both cranes and were replaced. The test on the No. 2 crane was started July

25 with a 187.5-ton load and this test was completed in August.

By order for changes No. 15, the 300-ton cranes were tested in accordance with paragraph C-4 of invitation No. DS-5260 instead of in accordance with the requirements of subparagraph 182(b) of the construction specifications.

(1) All main and auxiliary hoists were tested at no load, half load, and rated load to determine electrical characteristics and hoisting and lowering speeds.

(2) When conducting the bridge speed test under subparagraph (d)(4) of paragraph C-4, only one main hoist had a test load of 150 tons on the hook. The other hoist was unloaded.

(3) The cranes were tested consecutively so that final adjustment could be made to assure all four hoists operating as nearly alike as possible.

The auxiliary motor on the No. 1 crane was repaired and returned, and tests on both cranes were completed November 1962. Separate technical reports on the cranes were made.

228. 75-TON MACHINE SHOP CRANE. The 75-ton machine shop crane is a cab-operated, indoor traveling-type crane with a span of 67.5 feet. This electric, overhead crane travels on a runway extending the length of the machine shop and is used for handling materials and equipment in the shop. The trolley is equipped with a 75-ton-capacity main hoist and a 15-ton-capacity auxiliary hoist. With rated load the main hoist operates at speeds up to 6.6 feet per minute. The main hoist has a lift of 40 feet and is rigged with eight parts of 1-1/4-inch-diameter wire rope and a sister-type hook bored for a lifting pin. The auxiliary hook is a standard type with the total lift of 55 feet with four parts of 0.785-inch-diameter wire rope. The bridge speeds are variable up to 85 feet per minute.

This crane was supplied by the Legnano Electric Corp. of New York and Torino, Italy, under invitation No. DS-5252 and was installed by the prime contractor. The formal load testing of the 75-ton crane was started on February 21, and was completed on March 2, 1962. The load test pointed up motor defects on the bridge and trolley. The replacement of motors was extended in time until the prime contractor's forces were moving out, and the supplier had the repair work performed by the completion contractor.

The prime contractor had installed the 75-ton crane in the powerplant machine shop during the month of January 1962. As the connections were turned bolts in fitted holes, the assembly was not difficult and very little drifting and no reaming was required. The major mechanical problem encountered involved the operation of the hydraulic bridge brakes. After several unsuccessful attempts to adjust the bridge brakes, a close check revealed that the crane supplier had installed master cylinders at the cab-operated pedal and also at the brake shoe locations on the trucks of the bridge. This situation could not be corrected without extensive revisions of the entire brake system design. Although the brake system would stop the crane under full load at full speed, the mechanical advantage of a large master cylinder to smaller servo-cylinders was not in the system, and a large amount of pressure on the brake pedal was required to stop the crane. However, after further study, it was determined that the brake system would serve the purpose intended and basically met the specifications. The installation was therefore accepted.

229. 10-TON GANTRY CRANE. The 10-ton crane is an outdoor, traveling gantry type and was installed on the transfer deck on a track allowing travel along the length of the powerplant. This crane is for use in operation and maintenance of the turbine draft tube bulkhead gates. The gantry is equipped with a 10-ton base-mounted hoist. The hoist has a lifting speed of 14 feet per minute and a total lift of 80 feet and the gantry travels at a speed of 35 feet per minute. Both hoist and travel motions are controlled from a pushbutton station on the hoist platform of the gantry. The crane was fabricated by Crane Hoist Engineering Co. of Bell, Calif., under invitation No. DS-5398, and was installed by the prime contractor. The 10-ton gantry crane was placed on the rails of the transformer deck of the powerplant in March of 1962 and the counterweight was placed in April of 1962. During a trial run, when the crane was being used to install the draft tube bulkhead gates, the worm gear reducer drive unit was damaged and it was determined that this unit was not of sufficient capacity to meet the specification requirements. The worm gear reducer was replaced by one of greater capacity received on August 21, 1962.

230. OIL STORAGE AND HANDLING SYSTEMS. The lubricating and governor oil storage facilities consists of two vertical 4,000-gallon tanks. These tanks were furnished by American Steel and Iron Works of Denver, Colo., under invitation No. DS-5265 and were installed by Merritt-Chapman and Scott Corp. The tanks were installed, except for final cleanup and painting, during the spring of 1961 in the

machine shop at elevation 3123.67. No unusual difficulties were encountered during this installation phase.

Other components of the oil handling system were installed by the completion contractor, Ets-Hokin Corp., under specifications No. DC-5750, as follows:

(1) A portable oil purifier was furnished by DeLaval Separator Co. of Chicago, Ill., under invitation No. (D) 90,578-A. The purifier is capable of handling 1,200 gallons per hour of transformer oil and 600 gallons per hour of governor and lubrication oil.

(2) One 50-g.p.m. transformer oil transfer pump and one 30-g.p.m. lubricating oil transfer pump were furnished by Colorado Pump and Supply Co. of Denver, Colo., under invitation No. (D) 90,583-A.

(3) One filter paper drying oven was furnished by L. A. Greene-Bowser, Inc., of Cookeville, Tenn., under invitation No. (D) 90,624-A.

(4) The metal piping to the generator housing, turbine pit, and governor cabinet was furnished and installed by a subcontractor, Detweilers, Inc., of Idaho Falls, Idaho, under specifications No. DC-5750.

Installation of the equipment and piping was substantially completed between August and September of 1963, with no difficulties being encountered except for some minor initial troubles in testing the oil purifier. A few of the top disks in the purifying bowl were damaged due to probable mismatching of parts by the manufacturer, but these were corrected in the field.

The most significant difficulty encountered with the oil system occurred after installation by the completion contractor, when clean oil became contaminated in the supply piping between the purifier and the units. Initially, the contractor had cleaned the oil piping according to the requirements of the specifications. This procedure provided that the piping was to be dismantled after erection, cleaned, and then reerected. The steps in the cleaning procedure were to use a rotary wire brush, blow out the piping with compressed air, swab with a cloth impregnated with solvent, and then swab with dry cloths. The cleaning and erection process for this piping extended over a period of approximately 3 months from August to November 1963. During that time the open ends of completed portions of the piping runs were covered with a heavy tape, which appeared to be adequate to keep out foreign matter. The first oil was not introduced in the piping system until April 1964, when the piping was subjected to a pressure test. Several of the flanged joints had minor leaks which were promptly corrected.

In preparation for filling the oil requirements of generating unit 1, oil was circulated from the storage tanks through the purifier, down the main headers, and back to the storage tanks. Samples taken during a period of 5 to 6 weeks, while this filtering operation was in progress, revealed visible metallic particles which had the appearance of oxidized iron flakes. Moisture had apparently entered the piping at the taped openings or through loose flanged joints, during the time between the cleaning procedures and the first time oil was put in the system. In order to fill the unit oil sumps on schedule, a small terminal purifying unit was used just ahead of the unit piping for each unit. This terminal filtration procedure was also used in filling the transformers with insulating oil.

231. UNIT LUBRICATING OIL SYSTEM. The unit lubricating system is a key section of the overall oil handling system and extends from the headers servicing the units to the governors and to the generators and turbines. The generator thrust bearing is an immersed type with inherent oil circulation, and the capacity of the oil reservoir is 2,100 gallons. A self-contained high-pressure oil system is provided for the thrust bearing shoes to force oil between the bearing surfaces prior to starting and during shutdown of the generator in order to maintain a full thickness of oil film between the bearing surfaces. The upper guide bearing and lower guide bearing of the generator, also of the immersed type, have oil reservoirs around the bearings to allow for circulation of oil during rotation of the main shaft. The turbine guide bearing is lubricated by a pressure oil circulating system with two motor-driven oil pumps—one alternating-current pump for normal operation and one direct-current pump for emergency operation. The oil leaving the bearing drains back to an oil reservoir in the turbine head cover. There are provisions made for controlling the low temperature of the oil to assure lubrication on quick starts of the unit. During the first start of the units, the cleanliness of the oil from this system was questioned by the erection engineers of the turbine and generator manufacturers. This problem was resolved by the use of a small portable filter placed at a point where oil enters the bearing.

Oil storage, purifying, and transfer facilities are provided for handling the oil for the main power transformers, each with a capacity of 7,600 gallons. The system was placed so unfiltered oil cannot enter the filtered oil lines. The transformer oil storage room contains two 10,000-gallon transformer oil storage tanks, one for filtered oil and the other for unfiltered oil. Return piping from the main power transformers is not provided. The oil purifier room contains a 60-g.p.m. transformer oil transfer pump, supplied by Colorado Pump and Supply Co. under invitation No. (D) 90,583-A. When a transformer is to be drained or filled with oil, it must be moved on tracks into position near the oil unloading pit and flexible hose connections made between the transformer and the pump. The transformers are provided with filter press connections, so a portable purifier can be used on the transformer deck.

232. AIR COMPRESSOR INSTALLATION. Compressed air is provided for the operation of generator air brakes, charging of the governor pressure tanks, operation of air tools from service outlets located throughout the powerplant, operation of turbine grease pumps, operation of generator cooling water system controls and air conditioning system controls, and depressing the draft tube water during synchronous condenser or standby (spinning reserve) operation. The major items of Government-furnished equipment were installed by the completion contractor.

The high-pressure air supply consists of one stationary 20-c.f.m., 375-p.s.i., two-stage, water-cooled air compressor with aftercooler, and a horizontal air receiver, 18 inches by 6 feet, to supply air for changing the governor pressure tanks and to furnish standby emergency air supply for switchgear equipment on the generator floor. This equipment was supplied by Compressor Service Co., Los Angeles, Calif., and I. G. Downs, Inc., on invitations No. (D) 90-581-A-1 and No. (D) 90-581-A-2.

The service air, generator brake air, and draft tube depressing air systems consist of three stationary 100-p.s.i. air compressors. One is a 100-c.f.m., single-stage horizontal, water-cooled compressor with one air receiver, 48 inches by 12 feet; and two are 500-c.f.m., two-stage, horizontal, water-cooled compressors with two air receivers, 48 inches by 12 feet.

The brake air system has two air receivers, 30 inches by 7 feet, checked off from the main service air system so that proper pressure will be available for the generator air brakes. This equipment was supplied under the invitations previously mentioned.

The draft tube depressing air system has four air receivers, 66 inches by 18 feet, checked off from the main service air system. This depressing air system is provided in order that the units may operate as sychronous condensers or motoring, using a minimum of power. Initial unwatering to depress the water level in the draft tube below the level of the turbine runner is supplied by the stored air in the receivers.

No particular difficulties were encountered during the conventional methods of installation and testing used. The completion contractor's installation work consisted of moving the equipment to the powerplant, placing it on prepared foundations, and providing initial servicing and equipment for operations.

233. AUXILIARY AND SERVICE WATER SYSTEMS PUMP INSTALLATIONS. There are a large number of pumps installed in the equipment operating in the powerplant. The major water handling units are the generator cooling water pumps, the drainage and unwatering pumps, and the domestic water supply booster pumps.

Nine 2,200-g.p.m. generator cooling water supply pumps were furnished on invitation No. (D) 90,601-A by Aurora Pump Division of New York Air Brake Co. of Aurora, Ill. The centrifugal pumps take water from the draft tubes under a positive suction pressure and deliver it to the generator air coolers. Eight of these pumps were installed along with the unit piping in 1963, with one pump to be used as a spare. No significant installation difficulties were encountered.

Drainage and unwatering sump pumps were installed in both the powerplant and dam by the completion contractor. Two 2,900-g.p.m. deep-well turbine pumps were installed in the dam for unwatering the foundation sump. The two pump units are controlled by a float switch with automatic alternator. If one pump fails to keep the sump water level below a set point, an alarm sounds and the second pump will start. Two 500-g.p.m. and two 100-g.p.m. pumps were installed in the powerplant sumps. The pumps were furnished under invitation No. (D) 90,585-A by Fiese and Firstenberger Manufacturers, Inc., of Fresno, Calif. The operation is by automatic floatswitch control.

Four 920-g.p.m. centrifugal multistage booster pump units were installed to pump water to the Page water plant. Furnished by the G. M. Wallace Co. of Denver, Colo., under invitation No. (D) 90,622-A, the pumping units were installed, serviced, and tested by the completion contractor.

234. STATIONARY CARBON DIOXIDE FIRE EXTINGUISHING SYSTEM. The carbon dioxide fire extinguishing system was installed to provide automatic fire protection for the generators and oil handling rooms. These systems were installed by the completion contractor and were furnished by the Cardox Division, Chemetron Corp., under invitation No. (D) 90,606-A. The generator system is made up of an initial and a delayed discharge of gas. The initial discharge of the gas is initiated by thermal switches in the generator air housings. The electrical controls cause operations of the discharge heads in the cylinders at the cylinder bank and open the proper selector valve to direct the gas to the generator where the signal originated. This system can be operated manually from remote control switches, but the routing valves must also be operated manually. The delayed discharge is operated manually at the cylinder bank as required to control the amount of carbon dioxide in the generator. Each bank of initial and delayed discharge cylinders protects two generators through selector valves automatically, or manually, operated to discharge gas to a unit.

A separate group of cylinders was provided for the oil purifier room. Discharge is automatic subsequent to operation of water sprinklers in the room activated by fusible links. The flow of water through a waterflow switch closes a contact of an electric timer switch. After a 5-minute delay, a weight-operated valve is actuated by the electric timer and shuts off the supply of water to the sprinkler system, simultaneously actuating a solenoid valve to control the discharge of carbon dioxide to the oil handling rooms.

235. POWERPLANT MACHINE TOOLS. The following new machine tools were furnished by the Government and were installed in the powerplant by the completion contractor:

15-inch pedestal drill press
11- by 11-inch power hacksaw
300-ton hydraulic shop press with 5-ton arbor press
2-inch pipe and bolt threading machine
Electric heat treating furnace
24-inch vertical drill press

The completion contractor also installed the following used machine tools which were obtained from the Naval Gun Factory, Washington, D.C.:

16-foot boring mill with a 320-kilowatt semiportable motor-generator set
36- by 144-inch-centers lathe
20- by 72-inch-centers lathe
Milling machine
4- by 8-foot surface plate
6-foot radial drill
No. 2 cutter and tool grinder
10- by 1-1/2-inch pedestal grinder
12- by 2-inch pedestal grinder
16- by 30-inch-centers lathe
36-inch heavy-duty shaper
35-ton hydraulic straightening press
6- by 18-inch surface grinder

The installation of the machine tools consisted of leveling, locating floor anchoring if required, and servicing in accordance with manufacturer's instructions. The installation of the new tools was performed without difficulty. Some difficulties were experienced with the used tools, primarily due to lack of detailed manufacturer's instructions and parts lists and lack of repair parts on obsolete equipment.

2. Miscellaneous Electrical Equipment

236. GENERAL DESCRIPTION. The eight generators for Glen Canyon Powerplant were installed in a single row at finish floor elevation 3168.50. Located just downstream from the generators, an electrical gallery at this same floor elevation contains the generator switchgear. The 12 power transformers are located outside the building on the deck 20 feet above the switchgear in the electrical gallery. The isolated-phase bus, mounted on the downstream wall of the powerplant, provides the electrical connection between the power transformers and the generators. Each pair of generating units is connected to a single bank of three single-phase power transformers. Power is generated at 13.8 kilovolts by each of the eight generating units and is transformed to 345 kilovolts at units 1 and 2, 3 and 4, and 5 and 6, and to 230 kilovolts at units 7 and 8. Station-service power is taken off the bus of units 3-4 and 5-6. All major items of electrical equipment were furnished by the Government and were installed by the completion contractor under specifications No. DC-5750.

237. GENERATOR SWITCHGEAR AND BUS STRUCTURES. One station-type switchgear assembly for each generator was furnished by General Electric Co. under invitation No. DS-5828 and the eight

assemblies were installed by the completion contractor. Each assembly contains a 7,000-ampere air-blast circuit breaker, disconnect switch, current transformers, forced air cooling equipment, and provisions for connection to the isolated phase bus and generator protective cubicles. The switchgear for units 1, 3, 5, and 7 also include bus potential transformers used for synchronizing transformer metering and relaying.

Manufacture and shipment of the switchgear were scheduled in pairs to accommodate the installation of associated isolated phase bus, power transformers, and shunt reactors. The switchgear equipment was shipped assembled in two major components: the circuit breaker and disconnect switch in one part, and the bottom frame the other. A third section, the potential cubicle, was also shipped for four units. The first major shipment, which was the complete switchgear for units 1 and 2, arrived at Flagstaff railhead on August 29, 1963. The equipment for units 7 and 8 arrived in September 1963; unit 4 in November 1963, unit 3 in December 1963, and units 5 and 6 in July 1964.

The equipment was trucked to a temporary site storage area by the completion contractor and then moved into the powerplant after the base channels were set and grouted. Because of limited head room at the opening to the electrical equipment gallery at the generator floor level, it was necessary to move the breaker cubicle into position with small pipe rollers. Each breaker was then raised, the bottom frame placed on the base channels, and the breaker lowered to the bottom frame. The contractor fabricated a special lifting beam and A-frame to facilitate this installation work.

The switchgear was assembled in the energization sequence schedule for the switchyard and transformer circuits; namely units 1, 2, 7, 8, 3, 4, 5, and 6. This installation work started on March 10, 1964, and proceeded through August 1964, under the supervision of the manufacturer's erecting engineer. Final functional and timing checks of each breaker were performed just prior to placing the unit in service.

The switchgear and generators are connected to the power transformer by the isolated phase bus furnished by Westinghouse Electric Corp. under invitation No. DS-5828. The generator surge protection equipment, metering and relaying potential transformers, a bus ground switch, and connection terminals were included as part of the isolated phase bus.

The first shipment of bus arrived at the railhead on September 6, 1963, for use on units 1 and 2. The shipment for units 7 and 8 arrived on September 30, and other shipments arrived in November 1963 and June 1964. Several shipments sustained severe shipping damage, which was repaired in the field by welding or replacement of parts. The bus was shipped in sections convenient for handling and loading and the ends were covered and taped. Most of these protective covers were torn open during shipment and extensive cleaning of the interior of the bus housings was necessary.

In order to deliver power at 230 kilovolts from units 5 and 6 pending completion of the Pinnacle Peak 345-kilovolt additions, an additional section of bus for a temporary connection between units 6 and 7 was obtained on an order for changes to invitation No. DS-5828. This bus section was received and installed in May 1965.

Installation of the bus structures was accomplished by the completion contractor under specifications No. DC-5750. Services of the manufacturer's erecting engineer were utilized for approximately 65 percent of the installation work. In addition to repair of shipping damage, the erector corrected fabrication errors and omissions. Owing to the large size of the prefabricated sections, the bus and supporting frame were erected with a rubber-tired mobile crane utilizing conventional structural steel erection methods.

During erection of the bus, considerable difficulty was encountered in obtaining an adequate seal at the gasketed joints in the bus housing. Probably because of temperature cycling, the gasketed joints would not remain airtight. The positive pressure dry air supply system was therefore overloaded and ineffective, resulting in a number of air compressor failures during the first 6 months of operation. After many attempts to correct the leakage, the manufacturer proposed a redesign of the air supply system which included a high-volume low-pressure blower, a dehumidifier, and larger supply headers. A trial installation was made and found to be satisfactory. Modification of all units was completed by February 1967, and this work was performed under the equipment warranty.

All the bus structures were high-potential tested after completion of installation, and successfully withstood the 50-kilovolt tests. The original installation of the bus for units 7 and 8 included taps to shunt reactors which were in service until August 1965. When the reactors were no longer required, the bus sections were removed and permanent end caps were placed on the taps near the power transformers.

238. POWER TRANSFORMERS. Nine 100,000-kv.-a. single-phase power transformers, rated 345 kilovolts on the high side and 13.7 kilovolts on the low side, were furnished by Pennsylvania Transformer Division of McGraw Edison under schedule No. 1 of invitation No. DS-5780. Transformers for the fourth circuit (fig. 291), rated 100,000 kv.-a., 230 kilovolts to 13.8 kilovolts, were furnished by Westinghouse Electric Corp. under schedule No. 2 of invitation No. DS-5780.

The first shipment of the 345-kilovolt transformers from Pennsylvania Transformer arrived at the railhead in June 1963. One 276-kilovolt lightning arrester was damaged and required replacement. During September three more transformers arrived, and again one 276-kilovolt lightning arrester was damaged. The last three transformers required on schedule No. 1 arrived in late November and early December. The three 230-kilovolt transformers furnished by Westinghouse arrived at Flagstaff on October 4, 1963. The only visible shipping damage was a small crack in a weld near the base of the transformer. The tank still held nitrogen pressure and no internal damage was anticipated. The transformers were stored at the railhead until installation was started in June 1964. The 230-kilovolt Westinghouse transformers were hauled to the powerplant on conventional lowboy trailers.

Figure 291.—High-tension bushing being raised into place on transformer K7A. P557-420-9847, June 10, 1964.

Assembly of the 230-kilovolt transformers for units 7 and 8 began on June 6, 1964, under the supervision of a Westinghouse erecting engineer. The internal inspection of the transformers revealed that one set of high-voltage bushing current transformers had broken loose from the support brackets and fallen to the top of the winding. It was necessary to replace the current transformers and make extensive field repairs to insulating barriers and spacers and lead insulation. As was the case under similar conditions in transformer KU5A in the switchyard, the transformer access manhole was not large enough to permit removal and replacement of the damaged current transformer. As a part of the replacement procedure a new manhole was cut in the top of the transformer. New manholes were not installed on the other two transformers, but the necessary materials were furnished so that the manholes can be installed if they are ever required. Completion of assembly of the transformers was delayed nearly 2 months by the damage; however, this did not result in any delay of equipment energization.

The 345-kilovolt transformers furnished by Pennsylvania under schedule No. 1 of invitation No. DS-5780 were also stored at the railhead until installation. The first units were hauled to the powerplant in early July 1964. Because of weight and height restrictions, a special hauling carriage was required for these transformers. The carriage was designed and built by Reliance Trucking Co. These transformers were shipped with a temporary shipping cover welded to the tank. After oil filling, this cover was removed and the upper section of the tank, which provides space for the bushings and current transformers, was welded to the lower part. After the bushings were installed and connected, the oil filling was completed.

The high-voltage bushing draw-leads on three of the nine 345-kilovolt Pennsylvania transformers were too short to reach the bushing terminal. It was therefore necessary to lengthen the leads by cutting them off and splicing on a new piece. Replacement of the leads was not difficult and was considered as a part of the normal assembly of the transformers.

In the final stage of erection of the third transformer on July 30, 1964, a one-half-inch lock washer was accidently dropped into the tank. Many hours of searching did not locate the washer. Since it was necessary to establish liability prior to energizing the transformer, a fourth transformer was immediately assembled and substituted in the K1A bank in order not to delay the on-line schedule of the first generator.

Responsibility was ultimately placed with the construction contractor, and it was proposed that the transformer could be placed in service without untanking to locate the missing lockwasher if the contractor would extend the manufacturer's warranty for 5 years and bear the costs of an extensive combustible gas analysis program. The contractor's decision to accept this proposal was made on January 12, 1965, and the transformer was placed in the K3A bank and energized in conjunction with unit 4 start-up procedure on February 8, 1965. During the first 3 years of operation, no difficulties attributable to the lockwasher appeared.

Assembly of all Pennsylvania transformers was completed December 8, 1964; however, the last unit was not energized until February 1966. All insulating oil for transformers was degasified by vacuum treating as it was pumped into the transformers. With this method, dielectric strengths approaching 40 kilovolts were obtained.

239. SHUNT REACTORS. Twelve 8,000-kv.-a. shunt reactors (fig. 292) were furnished by General Electric Co. under invitation No. DS-5828 for installation on the 13.8-kilovolt bus at transformer K7A. All of the reactors were shipped in the latter part of November 1963, scheduled to arrive at the site in time to meet the installation schedule. However, six reactors sustained major shipping damage in route and were returned to the factory without having reached their destination. The damaged units were beyond repair and were scrapped. The replacements were not shipped from the factory until June 1964.

The services of an erecting engineer, who was already at the site for installation of other equipment at Glen Canyon, were utilized for the installation of all reactors. The first six reactors were installed by the completion contractor on the unit 7 side of transformer K7A and connected to the isolated phase bus. Shortly after the units were energized, excessive temperature rises were noted in the vicinity of the installation. Extensive cracking was observed in the concrete deck supporting the reactors. The reactors were removed from service on September 7, 1964, and the supplier was requested to proceed at once with modifications to limit the temperature rise.

Installation of the six reactors on the unit 8 side of transformer K7A was delayed until an aluminum

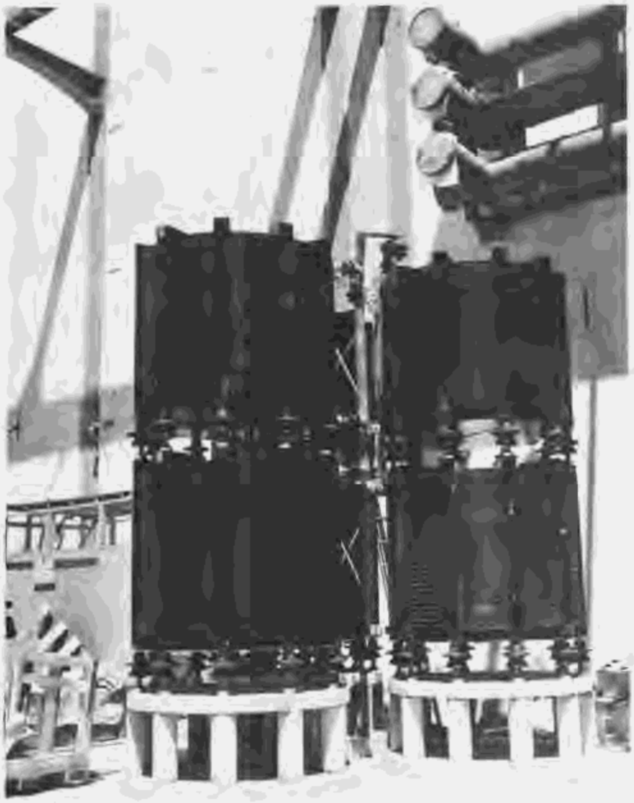


Figure 292.—Closeup view of two shunt reactors. P557-420-9943, July 13, 1964.

shield, 8 feet in diameter and 3/8 inch thick, could be fabricated and placed under the concrete base of the reactor. The assembly was completed and test energized on October 29, 1964. Extensive checks of the temperature rise of the supporting concrete and reinforcing steel with the reactor bank energized, showed that the addition of the circular shields was ineffective. The supplier modified the shielding by filling in the gaps and extending the shield beyond the reactors. Further temperature rise measurements indicated that while the rate of increase had slowed, the total rise at ultimate use would still be beyond specification limits. However, operational requirements were such that the lower half of the reactor banks could be disconnected; and since the temperature rise with this method of operation was not excessive, no further modifications were required.

Extensive tests of the area under the unit 7 side reactor bank were conducted to determine the extent of structural damage. Analysis of test cores by the Bureau's Denver laboratory and the space-frame analysis by the designers indicated no structural defects. Therefore, only replacement of the deck cover slab was required of the supplier. This was necessary in order to restore the membrane waterproofing under the slab, and the work was performed shortly after reactor removal.

The reactor interphase bus links were also found to be overheating during normal operation due to excessive eddy currents. This problem was corrected by redesign of the links so that the short axis of the links was perpendicular to the axis of the reactor.

As provided in the completion contract, the reactors were removed from service in August 1965, and were disassembled and loaded for shipment to the Bureau's Pinnacle Peak Substation.

240. MAIN CONTROL, GRAPHIC, RELAY, AND DISTRIBUTION BOARDS. The main control boards (fig. 200) were fabricated by Westinghouse Electric Corp. under invitation No. DS-5751 and were transported by motor freight. Individual panels were complete with meters, relays, switches, and internal switchboard wiring. Shipped in the latter part of 1963, the boards were installed in the powerplant control room early in 1964 (fig. 293). Interpanel switchboard wiring and some minor modifications were done at the site by Westinghouse.

Figure 293.—Control cables for unit 1 installed under raised floor of control room. P557-420-9949, July 13, 1964.

Interboard control cable was furnished, installed, and terminated by Anderson Electric Co., subcontractor for electrical work under specifications No. DC-5750. Internal board wiring and cable connections were checked and the relays and meters furnished with the boards were calibrated and set. Although no major problems were experienced with the main control boards, the scales and charts of the recorders furnished were initially incorrect. Representatives of Minneapolis-Honeywell corrected this deficiency.

Westinghouse obtained the sequence operations recorder from Rochester Instruments Corp. Installation of this unit was completed under the supervision of a representative of Rochester Instruments, and the unit was placed in operation a short time after initial operation of the powerplant. The annunciator system was supplied by Panellit Division, Information Systems, Inc., and the oscillograph by Hathaway Instruments, Incorporated. Virtually all of the relays were furnished by Westinghouse; exceptions included the unit phase balance relays, device 46, and the overfrequency relays 181.

With the expedited program for early service for units 1 and 2, design representatives from the Denver office were at Glen Canyon to make desired corrections to the control scheme design and wiring details. Temporary connections made at this time included the 230-kilovolt east and west bus overvoltage protection and connection of the reactor bank to the low side of transformer K7A.

Soon after initial operation of units 1 and 2, work began for temporary operation of the Glen Canyon-Pinnacle Peak line No. 1 at 345 kilovolts. This required changes in current transformer connections of transformer K7A, changes in differential and breaker failure relay tripping, and other related changes. The

Pinnacle-Peak line No. 1 controls and relays were also placed in operation and functionally checked at this time.

The need for unit dropping became apparent early in system operation, and a temporary unit dropping scheme was designed. Relays were mounted on a plywood panel which was installed at CCA13 utilizing spare circuit breakers at the rear of the unit control panels. Numerous modifications of the scheme have been made since that time due to changing conditions and the need for improvements.

New boards for the permanent relay system for the Glen Canyon-Flagstaff lines No. 1 and 2 were purchased under invitation No. DS-6227 from Keystone Electrical Manufacturing Co. The spare line relay panel on board CCC was connected to provide relay protection on the Glen Canyon-Flagstaff line No. 1 during the installation of these new boards. Installed by the completion contractor under a change order, substantial additions of control switches, meters, indication, wiring and cable were involved.

Addition of 69-kilovolt lines required additions and modifications of the control boards. The work was performed by the completion contractor under a change order. New indicating meters and control switches were installed on spare panels of the CCB board. Relays in the CCD board, originally provided for future 138-kilovolt lines, were used. The added control equipment was furnished by the Government under minor purchase orders.

241. STATION-SERVICE POWER SYSTEM. The primary 4,160-volt intermediate voltage switchgear, the secondary 440-volt unit substations, and the lighting distribution unit substations required for the station-service power system were furnished by Allis-Chalmers Manufacturing Co. under invitation No. DS-5732. This equipment was shipped to the jobsite during March, April, and May of 1963.

Installation of the equipment was started in October 1963, and continued as surfacing for the floors of the powerplant was completed. Assembly of the switchgear, the transformers, and disconnects by the completion contractor was performed under the technical supervision of the manufacturer's erecting engineer. Final checkout of the operational functions of the equipment was completed by the erecting engineer and Bureau test personnel in June 1964. The only significant difficulty encountered in assembly involved mismatch of rigid bus connections between components, which was corrected by factory refabrication of the bus without delay of installation work.

The primary power supply for the station-service system is provided by three 3,750-kv.-a. power transformers furnished by Westinghouse Electric Corp. under invitation No. DS-5828, and two 500-kv.-a. induction voltage regulators, furnished by General Electric Co. under another schedule on invitation No. DS-5828. The voltage regulators arrived at the railhead at Flagstaff, Ariz., on September 18, 1963. They were hauled to the powerplant by the completion contractor and set in position on the transformer deck during the following week. The three power transformers furnished by Westinghouse arrived on August 7, 1963, and were hauled to the transformer deck by the completion contractor shortly thereafter.

The interconnecting nonsegregated phase bus for the regulators, transformers, and 4,160-volt switchgear was also furnished by Westinghouse under invitation No. DS-5828. This equipment was received at the Flagstaff railhead on January 10, 1964, and installation was accomplished in early February. Other major items of station-service equipment included the unit auxiliary powerboards, air-conditioning powerboards, and miscellaneous power distribution panel boards, which were furnished by Federal Pacific Electric Co. under invitation No. DS-5787. The major portion of this equipment arrived at the jobsite in September 1963.

Installation of the powerboards by the completion contractor was scheduled to coincide with the required in-service dates of the major unit equipment. As installation progressed, equipment deficiencies appeared which required field modification. Some deficiencies were corrected by the completion contractor as required to place equipment in service and other corrections were arranged for in advance by the manufacturer.

The powerplant station-service system was energized from a temporary 12-kilovolt source on July 15, 1964, after successful high-potential testing of the equipment. This temporary source was used for all plant auxiliaries until December 30, 1964, at which time the transfer was made to the permanent connections on the unit 3 and 4 isolated phase bus. By this time the 25-kilovolt cable from the switchyard to the powerplant had been installed by the completion contractor under specifications No. DC-5750 and was available as a standby power source.

242. COMMUNICATIONS. The automatic switchboard and control room operator's turret were

furnished and installed by Stromberg-Carlson under invitation No. DS-5976. The dial telephones, extension bells and the code call relays and signaling devices were also furnished by Stromberg-Carlson and were installed under a change order by the completion contractor.

Installation of the automatic switching equipment was started on February 14, 1964, and was completed on March 6, 1964. As negotiations with the completion contractor for installation of extension equipment and interconnecting wiring were not complete, telephones were initially installed on a temporary basis at critical key locations by Government forces. Service from these lines was discontinued as the permanent installations were made. Interconnection to an outside line was first made by tielines to the attended switchboard at the administration building in Page. When the offices in the powerplant were occupied, a new Bell System switchboard was installed. Interface equipment was also provided to allow direct out-dialing for emergency and power dispatching purposes. An additional trunk adapter was purchased and installed by Government forces to allow for increased direct dial access to the Colorado River storage project microwave system.

A terminal cabinet and a 100-pair cable were installed by Mountain States Telephone Co. in the powerplant telephone equipment room. The cable was routed through the control cable tunnel to telephone company terminal facilities near the switchyard.

For the early stages of operation of the powerplant, interconnections for communications and metering were made to the Arizona Public Service Co. microwave system by way of a Government-furnished 25-pair cable. This cable was installed from the control room area to a terminal pole near the 69-kilovolt area of the switchyard by the completion contractor on a change order. Terminations were made in the powerplant at a telemetering equipment rack furnished by Arizona Public Service.

The permanent terminal cabinet and 100-pair cable from the Colorado River Storage project microwave terminal facilities were furnished and installed by Stromberg-Carlson under specifications No. DC-5853. Digital telemetering transmitters, load control thermoverters, and associated tone transmitters and receivers were furnished by General Electric Co. under invitation No. DS-5921 and were installed and connected by the completion contractor. Transfer trip relaying tone transmitters and receivers, furnished as a part of the microwave system, were also installed and connected by the completion contractor.

243. BATTERY CHARGERS. Two static-rectifier type battery chargers were furnished by Federated Engineering Development Corp. under schedule No. 3 of invitation No. DS-5787. Shipment of the chargers was greatly delayed by priority work in the supplier's plant. The late delivery had no adverse affect since procurement of the station battery was rescheduled to coincide with the delayed delivery. The chargers were received and installed in late February 1964, and the first unit was placed in service in May.

CHAPTER XVI. *Construction*–SWITCHYARD, TRANSFORMER CIRCUITS, AND CONTROL CABLE TUNNEL

244. GENERAL. Glen Canyon Switchyard (figure 216) is located on the right canyon rim starting about 600 feet downstream and 700 feet above the powerplant, within a fenced area approximately 800 feet by 1,200 feet. Seven of the eight transformer circuit towers for the powerplant to switchyard transformer circuits are located inside this fenced area, between the switchyard and the canyon rim. The other transformer circuit tower and a ground wire mast are located on the left canyon rim across from the switchyard. An access road was constructed from U.S. Highway No. 89 to the entrance at the northwest end of the switchyard, and a parking area was constructed at the entrance outside of the fenced area. The high-voltage switching and termination facilities located in the switchyard are described in Chapter IX.

An inclined tunnel was constructed from the powerplant to the switchyard, primarily for use as a control cable duct. This tunnel is about 7 feet square and 1,300 feet in length. A shallow type C cable trench extends from the powerplant control cable tunnel entry structure the entire length of the yard with a branch trench into the 25-kilovolt area. Control and power cables are buried from the cable trench to all equipment.

All electrical equipment for the switchyard was furnished by the Government. All other materials, including steel structures, overhead conductors, hardware, insulators, and control and power cables were contractor furnished. Initial excavation and grading was performed by the prime contractor under specifications No. DC-4825. The construction of the major portion of the switchyard was accomplished under the completion contract, specifications No. DC-5750. The 59-kilovolt facilities and the microwave power supply were constructed under specifications No. DC-6238 and the 5-kilovolt visitor center bay under specifications No. DC-6274. The effect of the expedited completion schedule for specifications No. DC-5750 was quite pronounced, since large areas of the switchyard were behind the original schedule at the start of the expedited program.

245. SWITCHYARD STRUCTURES. Initial grading for the switchyard was accomplished by the prime contractor, Merritt-Chapman and Scott Corp., under specifications No. DC-4825 during April and May of 1958. Initially, the switchyard was constructed to rectangular dimensions of 231 by 956 feet, containing an area of approximately 5.1 acres, and was surfaced with 6 inches of gravel surfacing. Owing to changed requirements of the present and future number of transmission lines and voltages to be provided, the switchyard was enlarged by the completion contractor to an irregular shape with maximum dimensions of 476 by 1,196 feet and the yard was lowered 1 foot at centerline, more than doubling the size to approximately 11.5 acres. Most of the gravel surfacing placed by the prime contractor was salvaged and stockpiled prior to initiating grading for the enlarged switchyard. The switchyard was graded using conventional methods and equipment in the excavated sandstone and blowsand. Much of the excavated sandstone broke down into an unsuitable, uniform-grained sand, which had to be wasted by the completion contractor. Compaction of the embankment was by equipment travel and vibratory-type compactors.

Grading for the switchyard and structure foundation construction was subcontracted to Fago Brothers Construction Co. of Scottsdale, Ariz. Work began on the grading phase early in July 1962 and the general grading had been essentially completed by the end of the year, except for the area adjacent to the control cable tunnel entrance structure. Work in this area was completed after the entry structure concrete was completed in July 1964. The salvaged gravel surfacing was spread, plus additional gravel, to stabilize the switchyard surface for access during the foundation construction phase. Order for changes No. 1 also provided for a 12-inch blanket of pit-run gravel on the switchyard embankments for erosion control. No drainage collection system was provided on the canyon side, and repair of this gravel blanket is necessary after occasional heavy rainstorms. A curb and outfall line system was planned for construction among future completion items. The switchyard access road and parking area embankments and bituminous surfacing were completed in October of 1964. The switchyard gravel surfacing was completed during October of 1966 and application of soil sterilant followed.

Most of the concrete foundations for the switchyard structures were designed for either pad- or rock-type foundations. The type of foundation used for the various structures was determined by the type and quality of material encountered. Concrete for the switchyard foundations was batched at the completion contractor's batching plant, located near the upper portal of the powerplant access tunnel, and was delivered to the switchyard by transit mixer. Conventional practices were followed in constructing the wooden forms and placing reinforcing steel and concrete for the structures and with curing of concrete by membrane curing compound.

The batching plant at the tunnel portal was provided by the subcontractor on the powerplant concrete. At times it was difficult to obtain concrete for the switchyard due to the heavy demands by the powerplant subcontractor. Placement of switchyard concrete foundations began during April 1963, and after changes in subcontractors was completed in November 1963 by the prime contractor, Ets-Hokin Corp. The remaining switchyard concrete, including conduit encasement, pullboxes, cable trench, cable trench crossings, and switch operating platform supports, was completed by the end of July 1964. To provide for temporary 230-kilovolt operation of the 345-kilovolt Flagstaff transmission line No. 1, additional structure foundations were constructed by the completion contractor in November 1964 as provided for by order for changes No. 5.

Structural steel for the switchyard was furnished by Muskogee Iron Works of Muskogee, Okla., and erection was performed by the completion contractor. Switchyard steel was assembled on the ground with loose-bolted connections. The larger structures were assembled in box sections on the ground. After erecting with mobile cranes, the structures were plumbed and all connections torqued. No unusual difficulties were encountered, although some misfabricated members required replacement or reworking. When holes were required to be redrilled or repunched, the galvanizing was repaired using zinc dust-zinc oxide priming paint. Erection of the switchyard structural steel began in the 25-kilovolt yard during August 1963, the 345-kilovolt yard during mid-September, and the 230-kilovolt yard during the last part of October. At the end of the year, all major steel structures in the switchyard had been erected.

246. HIGH-VOLTAGE BUSES. The east and west 345-kilovolt buses are 954,000-circular-mil all-aluminum duplex strain type, and taps from the bus to switches and jumpers between disconnects and breakers are also duplex cable. The connection of unit 5 to the west bus is accomplished with 3-inch iron-pipe-size aluminum rigid pipe welded to form a continuous length. Bus support clamps and connectors are of the bolted type. Corona-free hardware was installed on all 345-kilovolt bus, and corona control rings were used with all compression dead-end assemblies. Particular care was taken to avoid abrasion or contamination of the stranded cable in order to minimize corona.

The 230-kilovolt bus is also a combination of strain and ground mounted rigid 3-inch aluminum pipe bus. Conventional compression fittings and hardware and bolted fittings are utilized. The strain bus and jumpers are single-conductor, 1,272,000-circular-mil all aluminum. In order to dampen vibrations in long lengths of rigid pipe bus, 1,272,000-circular-mil stranded cable was inserted in the full length of the pipe.

The 25-kilovolt strain bus for the tertiary of the 345- to 230-kilovolt autotransformer is 2,500,000-circular-mil all-aluminum with compression dead-end assemblies. Taps to the bus are made with bolted connectors. All other 25-kilovolt strain bus, between the pothead structure and the switch structures, and jumpers from the buses to disconnects and circuit breakers are 1,272,000-circular-mil all aluminum. The main and transfer buses are 2-inch iron-pipe-size aluminum pipe as are connections between line and bypass disconnects. The contractor experienced considerable difficulty in producing the uniform short-radius bends which were necessary in this bus.

The 25- to 69-kilovolt transformer circuit is 954,000-circular-mil all-aluminum cable installed with compression dead-end assemblies. Essentially all other 69-kilovolt bus work is 2-inch iron-pipe-size aluminum pipe supported by post-type insulators.

All bolted taps and connectors to the various buses were made up with torque wrenches in accordance with the recommendations of the hardware manufacturer. Close inspection was maintained to assure that the aluminum conductors were properly cleaned prior to making up connections.

In order to temporarily utilize the 345-kilovolt Flagstaff line at 230 kilovolts as an interconnection to Arizona Public Service Co., a tie circuit between the 230-kilovolt west bus and the 345-kilovolt west bus was installed under a change order on the completion contract.

This temporary tie was utilized from December 1964, through October 2, 1965. A second temporary bus connection tying the 230-kilovolt east bus to the 345-kilovolt west bus was installed by Government forces using live-line bare-hand techniques, in April

1965. The temporary connection to the 230-kilovolt west bus was removed at that time.

247. TRANSFORMER CIRCUITS. Four transformer circuits were constructed from the powerplant takeoff to the switchyard takeoff structures. Circuits No. 1 through 3 are designed for operation at 345-kilovolts and circuit No. 4 at a voltage of 230 kilovolts. In order to provide the required clearances for the four transformer circuits, it was necessary to excavate 3,475 cubic yards of rock at the canyon rim. This work was provided for by order for changes No. 5 and was accomplished by the prime contractor under specifications No. DC-4825. The work started late in November 1960 and continued intermittently until completion in March of 1961.

The completion contractor under specifications No. DC-5750 was required to construct the transformer circuits including furnishing and erecting the takeoff structure at the powerplant and the tower structures, and furnishing and stringing the required conductors, insulators, and hardware. Before work could begin on the west rim structures, it was necessary for the subcontractor to salvage and remove the temporary visitors' parking area and overlook facilities. Work began early in August 1962, and the tower footings were excavated in conjunction with construction work for the switchyard. Foundations for the transformer circuit towers were constructed in rock in accordance with the construction drawings. The foundation outlines were line drilled and the material was loosened by shooting with 60 percent powder. Placement of foundation concrete began during February 1963 and was completed in October of 1963.

The transformer circuit rim towers were fabricated from extra-high-strength, high-strength, and standard-strength steels. The higher strength steel members were furnished painted with zinc dust-zinc oxide paint and the standard steel members were galvanized after fabrication. The remaining towers were fabricated with standard steel galvanized after fabrication. A hydraulic crane with a telescoping boom was used in the assembly of the tower sections on the ground. A 60-ton mobile truck crane with full boom and jib was used to erect the tower structural steel. To provide the reach for erecting the west side rim towers, it was necessary to construct temporary embankment pads adjacent to each tower. These embankments were later removed and the area dressed. The tower sections were assembled with loose bolts and, after erection and plumbing the bolted connections were torqued as required for the various bolt sizes.

The structural steel for the transformer circuit towers was also furnished by Muskogee Iron Works of Muskogee, Okla. Some fabrication errors required repunching and redrilling by the erectors. Resulting injuries to galvanized or paint coatings were repaired using zinc dust-zinc oxide priming paint. Erection of the transformer circuit steel towers began during December 1963, and by the middle of June 1964, was essentially completed.

Stringing and sagging of the transformer circuits was accomplished during late spring and summer of 1964 by the contractor's line crew and equipment being used on the construction of the 345-kilovolt Glen Canyon-Flagstaff transmission line No. 1 under another contract. Circuits No. 1, 2, and 3 were 2,167,000-circular-mil ACSR (aluminum conductor, steel reinforced) conductor, Circuit No. 4 was 954,000-circular-mil ACSR conductor and the overhead ground wires were 1/2-inch galvanized steel cable.

Stringing was done from the transformer deck to the rim towers, and the spans were attached to the towers with double sets of tension insulator strings. Hot-line hardware was used on the switchyard spans, along with corona rings and corona-free fittings in the high-voltage circuits.

248. CONTROL CABLE TUNNEL. The control cable tunnel extends from the powerplant on a 7- by 7-foot section approximately 1,300 feet long excavated in the canyon rock on a slope of 29°32′, terminating with concrete entrance structures. The upper portal is at the north end of the switchyard; the lower portal connects to the powerplant cable tower with a bridge section 53.5 feet long. The control cable tunnel serves as a duct for switchyard control cables, ground wires, communication cables, and a 25-kilovolt auxiliary power supply cable from the switchyard. The tunnel was constructed with galvanized cable trays, steel stairs, and a lighting system.

Excavation for the control cable tunnel, provided for under the prime contract, specifications No. DC-4825, was subcontracted to Frazier-Davis Construction Co., who in turn subcontracted the excavation to Cannon Diamond Drilling Co., of Compton, Calif. Work began with an opencut section in the switchyard area during September 1957, and was holed through into the canyon, approximately 100 feet above the river during March of 1958. A track-mounted, pneumatic overhead mucker was used to load the loosened material into a cable-hoisted muck car for hauling to the upper portal area. The excavated material was dumped into trucks and was wasted. Jack-leg drills were used to drill the powder holes, and about 10 feet of tunnel was excavated each two-shift day.

During January and February of 1963, the lower portal and portal invert concrete was placed by the prime contractor. The control cable duct between the tunnel and powerplant was a structural steel truss bridge, 52 feet 8 inches center-to-center bearing, with insulated metal wall panels and an open steel grating floor. Assembled at one of the rim storage yards, the bridge was set in place on the bearing plates early in April of 1963, using one of the 50-ton high-line cableways. Late in April, the pitched reinforced concrete roof was cast in place on the bridge.

Work performed by the completion contractor on the control cable tunnel included placing concrete for the tunnel walls arch and upper entrance structure, furnishing and installing the steel stairs, cable trays, and completing the required electrical installations.

Concrete for the control cable tunnel was batched at the upper portal area using a skip-loading mixer and a wheelbarrow scale. A specially built rubber-tired car, operated by a cable hoist at the upper portal, was built to transport materials, men, and equipment to the work area in the tunnel. The contractor excavated rock for the wall keys and placed concrete for an enlarged curb to support the wooden wall forms and provide a track for the access car. The enlarged curb concrete was placed beginning in early October 1963 and was completed in early December. Both sides of the wall concrete were placed simultaneously from the bottom between January and March 1964.

Conventional arch concrete was placed during April and May 1964 to the A–joint crossing. The A–joint was left unlined 15 inches on each side except where concrete inserts were required to be installed in the walls. From the joint to the transition to the entrance concrete, the arch was lined with 4 inches of pneumatically applied mortar, which was reinforced with welded wire fabric. Subcontracted to the American Gunite Co. of Salt Lake City, Utah, arch mortar was placed in two 2-inch passes during May of 1964. Concrete for the entry structure was placed during July of 1964.

Installation of the steel stairs and cable trays began in early June from the lower portal of the tunnel and was completed on the downstream side by mid-July. Pulling of control cables under the expedited program schedule followed and was completed by the end of July with installation of the remaining cable trays and cables later.

Techni-Builder, Inc., of Phoenix, Ariz., was awarded a contract under specifications No. DC-6351 (SF) to furnish and install a handrail on one side of the control cable tunnel stairway and to furnish and install chain-link, mesh-covered, safety gates on approximately 25-foot spacings throughout the length of the stairway. Installation was started December 6, 1965, and was completed December 10, 1965.

249. MINOR ADDITIONS. The 69- and 2.4-kilovolt microwave power supply additions to the Glen Canyon Switchyard were constructed under specifications No. DC-6238. Tide-Bay, Inc., of Tacoma, Wash., was awarded a contract for the work under these specifications and began work June 7, 1965. All work was completed December 4, 1965. The switchyard area had been graded, surfaced with 6 inches of gravel surfacing, and fenced by others under specifications No. DC-5750, prior to work under specifications No. DC-6238.

Reinforced concrete foundations were constructed for a 69-kilovolt power transformer, a 25-kilovolt transformer circuit tower, and a 69-kilovolt transformer structure, all in the 69- to 25-kilovolt transformer area of the switchyard. In the 69-kilovolt area, concrete foundations were also constructed for a bus structure, a circuit breaker, a current transformer support, and a potential transformer support, all 69-kilovolt, as well as a microwave supply terminal structure. Four reinforced concrete 69-kilovolt approach tower footings were constructed at P.I. station 1+98.8 of the Glen Canyon-Page 69-kilovolt transmission line.

Both the 69- to 25-kilovolt and the 69-kilovolt areas were located at the north end of the Glen Canyon Switchyard. Structural-steel structures furnished and erected under this addition included two bays of 69-kilovolt bus structure, one transformer structure, one potential transformer support, two current transformer supports, one approach tower, all 69-kilovolt; one 25-kilovolt transformer circuit tower, and one microwave supply terminal structure. Also included were field alterations to an existing 25-kilovolt fuse and bus support, field alterations to a 25-kilovolt bus structure, and furnishing and installing switch-operating platforms. Transformers, circuit breakers, and switches used in this modification and addition were Government-furnished.

Other major items installed or furnished and installed included various sizes of exposed electrical rigid metal conduit; nonmetallic conduit embedded in concrete; lightning arrestors and grounding systems; aluminum strain and jumper buses; rigid aluminum buses; bus supporting insulator assemblies; outdoor bracket-type lighting units; terminal boxes; two 69-kilovolt approach spans of 69-kilovolt transmission line complete with conductors and overhead ground wires; and one span of the microwave two-wire power supply line.

Provision for removing and restringing one span of messenger-supported communication cable in the switchyard was deleted from the specifications requirements and was handled by operation and maintenance personnel to minimize communications interruptions. Other minor modifications and additions were made to the Glen Canyon Switchyard under specifications No. DC-6274, to supply electrical power to the visitor center at Glen Canyon Dam. These involved constructing reinforced concrete foundations for a 5-kilovolt bus structure and a 1,500-kv-a. transformer; installing a 22,900- to 2,400-volt, 1,500-kv.-a., 3-phase transformer; and furnishing and erecting a 5-kilovolt bus structure complete with disconnecting fuses, lightning arresters, rigid aluminum bus, insulator assemblies, and grounding system. The power is transmitted to the visitor center through a concrete-encased buried duct bank. A lateral off the main duct bank was constructed to a dead end near the upstream side of the west abutment of the Glen Canyon Bridge to provide for future requirements.

A. SWITCHYARD ELECTRICAL EQUIPMENT

250. GENERAL. All major switchyard equipment was furnished by the Government and with few exceptions was delivered to the completion contractor at the railhead in Flagstaff and trucked to the site as provided in the specifications. All of the equipment was at the site sufficiently in advance of dates required to allow an orderly equipment installation program.

251. AUTOTRANSFORMERS. Three 100,000-kv.-a., 345-230-25-kilovolt autotransformers were furnished by Westinghouse Electric Corp., under invitation No. DS-5784. The original scheduled shipping date for the units was August 1963. However, all three transformers failed during impulse testing at the factory and were rejected. The transformers were rebuilt, successfully tested, and shipped in March 1964, arriving at the railhead in Flagstaff on April 6.

The transformers were hauled by the completion contractor's hauling subcontractor, Reliance Trucking, to the switchyard and set on their foundations on April 14 and 16, 1964. Assembly of the transformers under the supervision of the manufacturer's erecting engineer began May 12, 1964. Two of the transformers were found to have suffered shipping damage to the bushing current transformers on the high-voltage bushing, caused by failure of the current transformer support arrangement. Assembly and checkout of the undamaged unit was completed on May 25.

Repair and replacement parts were ordered immediately for the damaged units. It was found that the transformer manhole was not large enough to permit removal or reinstallation of the bushing current transformers. Modification and enlargement of the manhole was accomplished as a part of the repair procedure. The necessary parts arrived June 26 and installation was completed July 8. Normal assembly procedure of the two transformers was then continued and was completed July 16. Doble testing of the transformer bushing revealed one bushing which had an open capacitance tap. This bushing was removed and replaced with a bushing intended for transformer K7A. A new bushing was shipped from the factory on July 30 for subsequent installation in transformer K7A.

After the transformer had been assembled, Westinghouse advised that a change in field assembly procedure required that the 230-kilovolt bushing lead be wrapped with paper tape. This was accomplished at the time the faulty bushing was replaced on KU5A3, but the other two units had to be partially drained and refilled specifically for this work.

Other difficulties encountered included a defective oil pump motor on transformer KU5A2, a defective oil pump on KU5A3, and nuisance tripping of all oil pump motor starters. The defective pumps and motors were replaced by Westinghouse and the overload heaters were changed to the ambient-compensated type.

A vacuum degasifier was utilized in handling all oil for the autotransformers in addition to normal filtering equipment. This was not provided for in the specifications but was arranged for under a change order.

252. POWER CIRCUIT BREAKERS. The six power circuit breakers for the 345-kilovolt area of the switchyard were furnished by General Electric Co. under invitation No. DS-5721. These are among the first airblast type 345-kilovolt circuit breakers manufactured by General Electric. Originally shipment

of the breakers was scheduled for May, June, and July 1963. This schedule was delayed approximately 1 month for modifications made at the factory to improve original designs. The breakers were at the site considerably ahead of scheduled installation dates. Two of the breakers received considerable shipping damage, but it was confined primarily to cracked and broken welds which were field repaired during assembly of the breakers.

A number of modification kits were applied to the breakers, primarily connected with the air supply system. Early operating experience indicated that the air compressors for the breakers could be a source of maintenance difficulty. Special test equipment is required for installation and maintenance of these breakers. An electrolytic hygrometer was supplied by General Electric for checking moisture content of air and insulating gas in the breaker during installation work, and a direct-writing oscillograph was borrowed from the Parker-Davis project for making timing tests on the breaker.

An additional bushing current transformer was added to each pole of four of the breakers under specifications No. DC-6238. In order to avoid wasting large quantities of SF_6 (sulfur hexafluoride) insulating gas in the current transformer column when they were opened for adding the current transformer, the gas recovery cart furnished for use with SF_6 breakers on the Parker-Davis project was brought to the switchyard and adapted for conserving and reinstalling the gas.

Four 230-kilovolt oil circuit breakers and four 34.5-kilovolt oil circuit breakers were furnished by Pennsylvania Transformer Division of McGraw Edison Co. under invitation No. DS-5816. The 230-kilovolt breakers were received in May and the 34.5-kilovolt breakers in August of 1963. The breakers were hauled from the railhead to the switchyard in November 1963.

The 34.5-kilovolt circuit breakers sustained considerable shipping damage which was repaired in the field prior to final checkout. During storage, lamps were placed in the tanks to prevent condensation. In two breakers the lamps were in contact with the fiber tank liners and burned them. These liners were replaced by the completion contractor prior to final oil filling.

The 230-kilovolt circuit breakers were assembled by the completion contractor under the supervision of the manufacturer's erecting engineer. One 230-kilovolt bushing was broken by the contractor and was replaced at his expense. Installation, testing, and final checking of the four breakers was completed February 28, 1964, after 23 working days. An additional 4 days was required for oil filling.

These 230-kilovolt circuit breakers did not prove satisfactory for the conditions of operation at Glen Canyon, and after unsuccessful attempts to modify relaying schemes and establish special operating procedures to limit the duty on the breakers, a program of replacement of the breakers was instigated. Circuit breakers originally purchased under invitation No. DS-6287 for Mead Substation were diverted to Glen Canyon and were installed by Government forces. The first breaker was installed in December 1966, and the last in February 1967.

Two 69-kilovolt oil circuit breakers were furnished by Pennsylvania Transformer Division of McGraw Edison under invitation No. (D) 90,700. The breakers were received May 28, 1965, and installed by Tide-Bay, Inc. under specifications No. DC-6238. These breakers were shipped completely assembled and required only routine inspection and insulation checks prior to oil filling.

253. DISCONNECTING SWITCHES. All 345-, 230-, and 25-kilovolt disconnect switches in the switchyard were furnished by Schwager-Wood Co., Inc., under invitation No. DS-5860. The switches arrived at the site in early October 1963. Assembly and installation of the switches by the completion contractor started in December 1963 and continued through July 1964.

Some factory assembly and fabrication errors were encountered which required redrilling or punching mounting holes or replacement of hardware items. In addition to the assembly difficulties, it was found that ground switches would not operate properly. A manufacturer's representative was at the site on two occasions to correct the difficulties. Interphase operating rods, counterweights, and contact shoes were ultimately changed in order to obtain satisfactory operation.

254. VISITOR CENTER COMPLEX. The site selected for the Carl Hayden Visitor Center is on the west rim of the canyon between the right abutment of the dam and the right approach to the highway bridge. This location provides an excellent view of the lower reaches of the reservoir, dam, powerplant, tailrace, and highway bridge. A portion of the rotunda is cantilevered out from the canyon wall to afford a more spectacular view. The selected area was the site of the west cableway tower facilities during construction of the dam.

255. DESCRIPTION. The visitor center building (fig. 294) consists of a steel-framed one-story wing with basement, steel-framed rotunda, and a concrete elevator shaft. The building foundation consists of reinforced concrete columns, piers, pilaster, and foundation walls. The concrete roof of the elevator shaft is covered with sheet neoprene roofing. The building wing and rotunda have wooden roof decking covered with insulation and neoprene roofing, a combination of aluminum walls and entrances, and precast concrete panels with facings of exposed aggregate. Interior finishes consist of terrazzo, vinyl and mosaic tile floor finishes; hardwood paneling; ceramic wall tile; gypsum wallboard, suspended acoustical ceiling tile and ventilating acoustical ceilings; and miscellaneous finishes. The building is complete with air conditioning, electrical power, control and lighting systems, and plumbing system.

Other features of the visitor center are a concrete-lined tunnel and elevator shaft from the top of the dam, including a lighting system; concrete paving, terraces, retaining walls, sidewalks, curbs and gutters, bituminous surfacing, drainage facilities,

Figure 294.—Carl Hayden Visitor Center on the canyon rim overlooking Glen Canyon Dam on the Colorado River near Page, Ariz. P557-420-13030A, July 3, 1967.

roadway and parking area lighting systems; a water supply system, including a storage tank and 4-inch water supply line; and an electrical supply system from the Glen Canyon Switchyard.

256. CONTRACT ADMINISTRATION. The visitor center complex was constructed under specifications No. DC-6274 by Allen M. Campbell Co., General Contractors, Inc., of Tyler, Tex. A significant portion of the speciality work was performed by 12 subcontractors.

Although the contract was awarded on June 30, 1965, and notice to proceed was received by the contractor on July 12, the contractor was not able to start work until August 16 because of the statewide strike of operating engineers which prevented initiation of construction work at the site. A later findings of fact established that this delay was excusable from computation of allowed contract time.

A. CONSTRUCTION

257. FOUNDATION PREPARATION. Overburden was stripped from the general area. Drilling for excavation of the basement of the building was completed and the basement was blasted and excavated. During removal of the blasted basement rock, fill was placed and compacted in the parking areas and drilling and blasting was completed on miscellaneous features, such as sidewalks and the water tank at the base of "Beehive" rock. Excavation of foundations for major walls outside the building and major trenching was then completed using conventional methods and equipment. Concrete from the old high-line tail tower tracks had to be blasted and removed to construct the access road to the upper parking area, and high rock spots in the parking area were also drilled and blasted.

During the excavation of the trench for the 4-inch waterline, a previously covered portion of the old tail tower rail support concrete was found to be considerably thicker than had been anticipated. A rock knoll between U.S. Highway No. 89 and the ramp to visitor center parking area No. 3 also obstructed the view and created a traffic hazard. At another location, two catch basins and some exterior lighting standards were located such that the foundation excavation encountered other sections of the old tail tower support concrete. The excavation in these instances was performed as extra work by the contractor.

258. TUNNEL EXCAVATION. Rock bolts were placed at the portal of the tunnel between the dam and the visitor center, and excavation of the tunnel was then performed, using conventional drilling and blasting methods. Owing to the relatively small size of the tunnel, a small rubber-tired conventional four-wheeled trailer was used as a drilling jumbo, which also served to carry tools and materials. On completion of the excavation in rock for the basement of the building, a pilot hole for the elevator shaft was driven from the top and was holed through satisfactorily. Line drilling and excavation of the elevator shaft to the A-line was then performed. The elevator pit was then excavated and tights were removed in the tunnel, shaft, and basement. Excavated material from the tunnel and elevator shaft was wasted into the reservoir from the top of the dam.

The mining work was performed by a subcontractor, the Mile Hi Drilling Co. As a general rule 35 holes, each 8 feet in depth on 2-foot centers, were drilled to excavate the 7- by 9-foot tunnel, using a 3-hole burn or burden breaker in a triangular pattern. Each round was loaded with 295 sticks of 35 percent powder. Knock-off bits were used on two jackhammers to drill the powder holes. As a general rule, the tunnel was driven on swing shift, sometimes making two rounds per shift and averaging about 7 feet per round. A hydraulic front-end loader was used to bring out the excavated material. To provide clearance for the loader at the points of intersection, the inside corners of the tunnel were removed resulting in considerable overbreak. As excavation progressed, rock bolts were placed in the roof as a safety measure. The drilling crew consisted of a foreman and two miners who loaded, blasted, and installed rock bolts.

The contractor placed considerably more roof bolts than had been normally anticipated from previous experience in Navajo sandstone. This condition was primarily due to the contractor's conservative attitude towards safety in the softer formations at the upper part of the canyon wall. No particular difficulties were experienced in blasting the tunnel, and overbreak was maintained within reasonable limits.

Drilling of the 5-inch drain hole for the elevator pit was initially abandoned at a depth of about 28 feet due to difficulties in drifting. A new 3-inch pilot hole was drilled through to the canyon which was then reamed to a diameter of 5 inches by pulling the bit from the canyon to the pit with a modified wagon drill.

259. FOUNDATION EXCAVATION. No unusual difficulties were experienced in the rock excavation or earthwork construction for the visitor center. Following rock excavation and general grading, trenching for the various utility lines was completed

and the pipes were laid. This included the water supply piping, the corrugated metal pipe surface drainage system, electrical power supply duct bank and manholes from the switchyard, and buried area lighting conduits. The manholes for the surface drainage were precast concrete. In addition, a buried control cable and water supply piping were installed for an automatic sprinkling system for certain areas on the visitor center grounds to be landscaped by others under a future contract.

260. CONCRETE PLACEMENT. Concrete placements began with the placing of sections of the concrete integral retaining wall and sidewalk, parapet wall sequences and manhole invert slabs. On December 30, 1965, a fire gutted the batching plant owned by the concrete supplier. Replacement equipment and materials were ordered promptly, repairs were made, and the plant returned to full operation on January 10, 1966. Concrete was placed by conventional methods, including the use of wooden forms and snap-tie fasteners, generally following a logical sequence of construction. Foundations and basement walls were placed, followed shortly by the basement floors. The first floor was not placed until all of the basement walls had been completed. With placement of the elevator machinery room roof, concrete placements were completed in August 1966. Except for those locations directly accessible to a transa-mix truck, the concrete was placed with a mobile crane using a bucket with a rubber tremie. Figure 295 shows the status of the work as of May 24, 1966.

Subsequent to completion of the concrete construction in the building proper and the completion

Figure 295.—Aerial view looking east over visitor center construction. P557-420-12274, May 24, 1966.

of trenching excavation, embankment was placed about the various major concrete structures and utilities, and the gravel underbed for the paved areas adjacent to the building were completed. Construction of the miscellaneous concrete structures took place over a period of about 1 year, progress being generally slow.

During the colder winter months, the contractor kept the concrete curing temperatures above the minimum specification requirements through the use of plastic sheeting on wooden frames, warmed by space heaters. Construction of the electrical duct bank began during January 1966, and continued intermittently, with placement of concrete being completed during July and the backfill completed during August. Following completion of the tunnel invert concrete in May, no concrete was placed in the shaft or tunnel until September when the tunnel arch lining and elevator lobby arch concrete were placed. The tunnel entrance structure concrete was placed in October and November 1966. The tunnel entrance door installation and tunnel lighting installations were also completed at this time.

261. MISCELLANEOUS. The structural steel for the visitor center was delivered early in the project work and was stored at the site. After the concrete floors were placed and cured, the structural steel was quickly erected without experiencing any unusual difficulties. Terrazzo was placed in November and December 1966, except for the final fine grinding and sealing which was deferred until later. During placing of the terrazzo floors, a rainstorm washed white cement mortar over the nominal protective barriers provided and down over the canyon wall below the rotunda. The resulting white stains on the face of the canyon wall detracted from the appearance of the visitor center and it was necessary for the contractor to remove them. Installation of mechanical and electrical materials roughly followed the sequence of concrete and framing construction. Following discussions with the National Park Service, changes and additions were provided for the audiovisual system for the visitor center and numerous revisions were made in the finish requirements in the exhibit, information, and lounge areas on the main floor of the rotunda.

Numerous delays were experienced by the contractor in obtaining precast concrete panels, primarily due to problems of the supplier. The problems were eventually resolved and the precast panels were delivered to the site. The rate of delivery was somewhat slow and other deficiencies were found in the panels upon delivery to the site. As a result, some of the panels had to be returned for replacement. Delays in installation of the precast concrete panels were also reflected in a slower rate of progress for most of the finish work on the visitor center.

Installation of the buried portions of the 4-inch water supply line, including the fire hydrants, was completed early in 1966. Water supply and sanitary outfall pipes in the tunnel pipe chase were installed during the summer prior to completion of the tunnel arch lining and lobby arch. The water supply tank was completed early in the job so that it could be used for supply of construction water. The exterior surface of the water tank was initially painted an aluminum color in accordance with the specifications but was later repainted to blend with the surrounding sandstone. The VR3 (vinyl resin) paint on the interior of the tank proved to be defective at the welded seams. A reduction in contract price was made due to the expected shortened life of the paint.

On completion of the various excavation and placement of minor concrete foundations, placement of subgrades for the parking areas, roadways, curbs and gutters was performed as space became available. Owing to subsurface conditions, the retaining wall and the sidewalk at the toe of the sloped area between parking areas No. 2 and 3 were combined into a common section, which also affected the two stairways between these parking areas. Portions of the parking areas were used for storage of construction materials at different times. Grading of parking areas and roadway subgrades continued over several months.

262. ELEVATORS. Elevator rails and machinery room beams were delivered to the construction site in June 1966, prior to the arrival of the manufacturer's representatives on July 12. These elevators were furnished under invitation No. DS-6276 by Montgomery Elevator Co. Previous to work on the elevators, the machinery room beams were set to line and grade prior to placement of the machine room floor by the construction contractor. Installation of guide rails began in July and was complete in August. The two car frames were assembled in the elevator pit, electrical conduits for the shaft were installed, and hoist machinery and control panels were set in place. Following installation of the hoist cables, counterweights were installed to raise the car frame to the elevation 3210 landing for assembly of the elevator car. During October 1966, the west elevator car was assembled and placed in manual operation and the east car was assembled early in November. By the end of November, both elevators had been completed and the control wiring and device installation checkout were satisfactorily made. Acceptance tests were satisfactorily completed during April of 1967. In general, the elevators were constructed using conventional installation methods and no unusual problems were experienced during this phase of construction. The hoist for elevator No. 1 did develop vibrations greater than normally expected. Representatives of the contractor returned to the project during June 1967, and corrected an out-of-round brake drum.

263. PRECAST PANELS. Installation of the precast concrete panels for the building continued intermittently. Framing for the building interior and installation of associated conduit and metalwork continued when the panels were not available for installation. About 95 percent of the panels had been set in place by the end of March 1967. The south entrance precast column panels were rejected due to fabrication defects, and replacement panels were delivered and installed during May, which completed the panel installation. Calking of the panel joints began soon after completion of installation of the panels and was rapidly completed.

As soon as the last panels were in place, work began on installation of the building roof insulation and roofing, which was completed within about 1 month. Some difficulty was experienced with handling of adhesive, which was resolved when the critical limits of the temperature installation range were determined and the work schedule adjusted accordingly. Basically, roof surface temperatures of less than 95° F. were necessary at the surface of the black neoprene in order to effect adequate bond with the recommended adhesive. As soon as the building was closed in, work began on installation of the aluminum curtain walls, entrances, wallboard, insulation, and ceiling installations. Interior finish carpentry and electrical installations were followed by the installation of bat and rigid type insulation, gypsum wallboard, walnut paneling and other wall finishes.

264. FINISHING WORK. By the end of June 1967, finish grinding and sealing of terrazzo floor surfaces, installation of plumbing fixtures, and electrical fixtures, and finish painting of the building had been generally completed. Chain link fencing, aluminum handrails, and precast concrete benches were then installed. About a month after the terrazzo had been completed, a small crack appeared in a section of the terrazzo in the rotunda, showing some vertical displacement. Other cracks appeared during the next few weeks and a drummy area of about 1 square yard developed. The area was rebonded by drilling and

grouting with an epoxy sealant, and the cracks were repaired. Although the problem seemed related to differential settlement, no evidence of this could be found on the underside of the rotunda floor and no further cracking occurred during an extended period of observation.

The initial start of the heating and ventilating chiller was made on July 12, 1967. Work began on the checkout and adjustment of the system but was suspended when the chilled water system proved to be deficient in providing the specified volume of water to the air-conditioning units. Two circulating pumps, designated CP-1 and CP-2, were replaced with pumps of greater capacity and the system was satisfactorily tested and completed. All work under the contract was accepted as substantially complete on August 17, 1967.

CHAPTER XVIII. INITIAL POWER GENERATION AND RESERVOIR OPERATION

265. FILLING CRITERIA. The filling criteria for Lake Powell were approved by the Secretary of the Interior on April 2, 1962. Basically, the criteria cover the first filling of Lake Powell up to full reservoir at elevation 3700 feet above mean sea level. Provision was made for sufficient water to be released from Lake Mead and Lake Powell to satisfy water uses below Hoover Dam. The criteria also provided that the United States would make a fair allowance for any deficiency in firm energy generation at Hoover Powerplant.

Until elevation 3490 was first reached, the water stored in Lake Powell was made available to maintain rated head on Hoover Powerplant. A minimum flow of 1,000 cubic feet per second was maintained in the river downstream of Glen Canyon Dam during this initial storage period through the high-pressure gates in the left diversion tunnel. After stored water in Lake Powell reached elevation 3490, the reservoir level was maintained at that level or higher. The storage level in Lake Powell was operated in such a manner that Lake Mead was not drawn below elevation 1123, which is the elevation for rated head on Hoover Powerplant. Over 1,000,000 acre-feet per year was also released while gaining storage to elevation 3490 at Glen Canyon, insofar as inflow and storage permitted.

In summary, the operation of Lake Powell above elevation 3490 and the operation of Lake Mead were coordinated to produce the greatest practical amount of power and energy, but still meet downstream water demands in accordance with the filling criteria. These filling criteria are included in U.S. Senate Document No. 7, 88th Congress, 1st session, along with other related documents.

266. RESERVOIR FILLING. Initial closure of Glen Canyon Dam was accomplished on March 13, 1963, by final closure of the left diversion tunnel gates. The reservoir water surface elevation at the time of closure was 3203.3 feet with a storage of 119,500 acre-feet. Water release through the high-pressure regulating gates in the left diversion tunnel was adjusted to pass a minimum of 1,000 cubic feet per second. This rate of release was maintained for the remainder of the year and the reservoir elevation on December 31, 1963, was 3409.6 feet with a storage of 2,968,000 acre-feet.

Releases were increased by varying amounts to meet downstream requirements from January 30, 1964, through May 11, 1964, when they were again reduced to a minimum of 1,000 cubic feet per second. This flow was maintained through August 17, 1964, when the reservoir attained an elevation of 3490.3 feet and a storage of 6,141,000 acre-feet. Testing of generating unit 1 began on August 20, 1964, unit 2 on September 9, 1964, and testing of the other units followed at approximately 2-month intervals. The downstream water releases were continued as coordinated releases from the available turbines, left diversion tunnel gates, and the outlet works hollow-jet valves until August 30, 1965. By this date units 1 through 5 were on the line and the combined turbine discharges alone were sufficient to meet downstream water requirements.

267. INITIAL OPERATIONAL STAFFING. Key operating personnel were transferred to the project in July 1964, to observe the last phases of construction and equipment installation, assist in final tests, and assume operation of the powerplant upon completion of construction. Operating personnel consisted of a branch chief, an operations foreman, five control room operators, and five assistant operators, providing adequate personnel for efficient operation of Glen Canyon Powerplant on the basis of 24 hours per day and 7 days per week.

268. INITIAL POWER GENERATION AND TRANSMISSION. Initial power transmission was to be made on the Glen Canyon-Shiprock 230-kilovolt transmission line. This line was test energized from Shiprock, N. Mex., to Glen Canyon. Following the initial turbogenerator testing, unit 1 was synchronized to the system on August 31, 1964. Tests were completed and unit 1 was operated 461.5 hours during the month of September 1964. Unit 2 was initially synchronized to the system on September 18, 1964, and was operated 232.0 hours during the remainder of the month. Combined generation of units 1 and 2 produced 46,220,000 kilowatt-hours in September, the first partial month of operation. The official in-service dates for the eight units (dates of first power generation) are as follows:

Unit 1	September 4, 1964
Unit 2	September 22, 1964
Unit 3	December 11, 1964
Unit 4	February 10, 1965
Unit 5	July 19, 1965
Unit 6	October 13, 1965
Unit 7	January 21, 1966
Unit 8	February 28, 1966

Table 6.—*Lake Powell—Reservoir elevation and water storage (Readings at first of month).*

Month	Reservoir elevation in feet above mean sea level	Storage, acre-feet
1964		
October	3491.69	6,211,880
November	3491.91	6,223,320
December	3491.47	6,200,440
1965		
January	3491.94	6,224,880
February	3491.41	6,197,320
March	3491.91	6,223,320
April	3491.88	6,221,760
May	3490.91	6,171,320
June	3491.82	6,218,640
July	3510.85	7,270,300
August	3530.91	8,513,060
September	3531.34	8,541,100
October	3530.12	8,460,920
November	3531.57	8,556,050
December	3532.56	8,625,960
1966		
January	3534.63	8,764,840
February	3535.21	8,804,070
March	3534.38	8,747,840
April	3536.73	8,907,370
May	3538.98	9,061,620
June	3544.57	9,456,040
July	3544.91	9,480,520
August	3540.01	9,133,700
September	3534.00	8,722,000
October	3529.46	8,421,360
November	3527.12	8,269,680
December	3523.57	8,042,910
1967		
January	3521.45	7,909,900

Table 7.—*Glen Canyon Powerplant—Power generated and water used.*

Month	Hours operated								Thousand kilowatt hours produced	Unit releases, acre-feet	Bypass releases, acre-feet
	Unit 1	Unit 2	Unit 3	Unit 4	Unit 5	Unit 6	Unit 7	Unit 8			
1964											
September	461.5	232.0							46,220	152,406	7,934
October	737.0	742.5							85,470	274,121	0
November	710.1	692.6							107,894	346,646	0
December	672.7	744.0	494.4						115,633	371,818	24,668
1964 Total	2,581.3	2,411.1	494.4						355,217	1,144,991	32,602
1965											
January	734.3	744.0	744.0						165,703	530,876	0
February	622.4	510.2	614.2	368.4					138,888	445,108	52,710
March	744.0	742.6	470.1	744.0					128,173	424,256	120,084
April	588.5	716.7	639.5	709.1					117,514	393,276	694,306
May	743.2	743.2	742.7	744.0					170,233	551,516	1,672,428
June	713.2	709.7	720.0	719.9					208,825	668,800	1,552,866
July	740.0	741.5	738.0	741.5	331.2				218,961	707,714	46,162
August	732.4	711.7	730.8	530.5	692.1				258,122	836,904	98,184
September	700.2	711.8	712.5	720.0	694.8				250,393	809,726	708
October	689.2	698.7	715.4	711.4	582.6	231.7			223,871	718,450	4,042
November	0	718.0	702.7	720.0	474.8	702.8			205,006	656,262	0
December	133.2	696.1	680.2	713.7	261.2	712.3			187,166	600,288	376
1965 Total	7,140.6	8,444.2	8,210.1	7,422.5	3,036.7	1,646.8			2,272,855	7,343,176	4,241,866

Table 7.—*Glen Canyon Powerplant—Power generated and water used*.—Continued

Month	Hours operated								Thousand kilowatt hours produced	Unit releases, acre-feet	Bypass releases, acre-feet
	Unit 1	Unit 2	Unit 3	Unit 4	Unit 5	Unit 6	Unit 7	Unit 8			
1966											
January	329.3	405.1	728.8	744.0	240.0	652.9	283.7		166,222	533,384	418
February	663.5	0	514.7	653.6	528.5	651.9	600.8	66.6	172,985	554,600	626
March	370.7	0	516.2	462.4	727.4	647.7	638.4	635.4	226,203	641,838	0
April	630.9	624.6	0	713.2	702.8	697.0	713.8	717.7	299,712	845,376	0
May	720.4	674.1	0	737.7	735.9	726.7	744.0	742.0	358,023	1,003,028	0
June	675.6	694.7	213.1	269.3	696.0	699.3	476.2	713.5	275,683	764,672	0
July	655.2	740.3	686.3	0	727.5	549.1	461.1	468.8	239,646	667,544	0
August	446.4	736.2	667.8	0	736.4	528.6	458.3	601.1	236,062	666,196	0
September	587.6	612.1	654.4	531.8	358.0	141.1	680.1	368.2	216,308	617,846	0
October	612.5	462.1	699.5	614.7	592.5	0	419.0	668.2	197,267	569,188	0
November	588.4	708.6	635.7	705.7	408.3	0	534.7	600.8	208,040	605,172	0
December	581.0	648.0	719.3	715.2	537.0	0	177.1	732.8	189,273	554,010	0
1966 Total	6,861.5	6,305.8	6,035.8	6,147.6	6,990.3	5,294.3	6,187.2	6,315.1	2,785,424	8,022,854	1,044

Powerplant, dam, and switchyard station-service power supply was energized through the permanent powerplant facilities on December 30, 1964. A temporary feed had been installed to the switchyard until the permanent equipment was energized. The 25-kilovolt station-service voltage regulator KV1A was put into initial service on March 19, 1965, and the switchyard station-service was also picked up from autotransformer KU5A at that time.

As mentioned previously, power was initially transmitted on the Glen Canyon-Shiprock 230-kilovolt transmission line. The Glen Canyon-Sigurd 230-kilovolt transmission line was energized on October 19, 1964, and the Pacific-Northwest and Pacific-Southwest regions were interconnected through the switchyard the next day.

The Glen Canyon-Flagstaff section of the Glen Canyon-Pinnacle Peak 345-kilovolt transmission line No. 1 was initially energized at 230-kilovolts on December 22, 1964. The Arizona Public Service Co. Bureau of Reclamation loop was closed for power delivery on December 24, 1964. The Glen Canyon-Pinnacle Peak transmission line No. 1 was opened at Winona on May 24, 1965, and was energized from Glen Canyon to Pinnacle Peak Substation at 230 kilovolts for the first time on May 25, 1965. This line was tested on October 2, 3, and 4, 1965, to permit energizing for service at 345 kilovolts. The Glen Canyon-Pinnacle Peak transmission line No. 2 was first energized on July 1, 1966, being delayed due to the difficulties in obtaining satisfactory acceptance tests for an autotransformer at Pinnacle Peak Substation.

The 69-kilovolt transformer KW1A and the 69-kilovolt switchyard were first energized for testing on January 31, 1966. Power delivery to Arizona Public Service Co. and to Garkane over the Glen Canyon-Page transmission line began on February 4, 1966.

269. TRANSFER TO OPERATION AND MAINTENANCE STATUS. During October 22 through 28, 1966, a preparatory inspection for transfer from construction status to operation and maintenance status was made at Glen Canyon Dam, Powerplant, and Switchyard. Major construction was considered complete except for minor deficiencies which would be corrected, and the transfer was made effective January 1, 1967.

270. ELECTRICAL GENERATION AND RELATED STATISTICS. Tables 6 and 7 show initial reservoir storage and power generation statistics from the fall of 1964 until the end of 1966.

PART IV

APPENDIX--

APPENDIX A

Cost summary for Glen Canyon Dam, Powerplant, and appurtenant structures, showing cost of construction and supply contracts and purchase orders (actual cost to June 30, 1967):[1]

Construction Contracts

Specifications number	Item, contractor, and contract number	Total cost to the Government
DC-4730	Section of access highway from Bitter Springs—Strong Co., contract No. 14-06-D-2036	$ 1,500,501
DC-4756	Section of access highway and Waterholes Canyon Bridge from Bitter Springs—W. W. Clyde and Co., contract No. 14-06-D-2188	1,040,304
DC-4747	Right Diversion Tunnel—Mountain States Construction Co., contract No. 14-06-D-2045	2,278,575
DC-4800	Colorado River Bridge (Glen Canyon Bridge)—Kiewit-Judson Pacific-Murphy Co., contract No. 14-06-D-2240	4,658,498
DC-4825	Prime contract for construction of Glen Canyon Dam and Powerplant—Merritt-Chapman and Scott Corp., contract No. 14-06-D-2403	134,699,717
DC-4865	150,000-gallon elevated storage tank and 3,000,000-gallon storage reservoir—Pittsburgh-Des Moines Steel Co., contract No. 14-06-D-2438	189,935
DC-4886	Foundations for demountable houses—Martin-Rice Construction Co., contract No. 14-06-D-2424	29,484
DC-4887	Surfacing guardrail access highway and Manson Mesa Airstrip—Alexander Construction Co., Inc., contract No. 14-06-D-2437	1,501,621
DC-4896	Access roads, streets, and utilities for Page, Ariz.—W. W. Clyde and Co., contract No. 14-06-D-2622	1,682,361
DC-4912	Sewage treatment plant for Page, Ariz.—W. W. Clyde and Co., contract No. 14-06-D-2629	257,884
DC-4924	Laboratory, municipal building and warehouse—Security Construction Co., contract No. 14-06-D-2647	377,862
DC-4933	Water supply system for Page—Southern Engineering and Construction Co., contract No. 14-06-D-2646	1,108,439
DC-4989	Residences for Page, Ariz.—Mobile Home Corp. et al, contract No. 14-06-D-2698	3,188,873
DC-5066	Lighting installation for Glen Canyon Airport—A and B Lighting, contract No. 14-06-D-2884	27,498
DC-5090	Furnishing and installing control cable from river pumps to filtration plant—A and B lighting, contract No. 14-06-D-2914	23,748
DC-5115	Administration building and garage, fire station and police building for Page, Ariz.—Sierra Construction Corp., contract No. 14-06-D-3110	337,860
DC-5123	50-ton motor truck scale and scale house—W. W. Clyde and Co., contract No. 14-06-D-3130	50,283
DC-5163	Seismograph vault and access road to seismograph station—W. W. Clyde and Co., contract No. 14-06-D-3170	15,600
DC-5206	Parking area and utilities for block 17, Page, Ariz.—A. J. Kelton, contract No. 14-06-D-2388	128,601
DC-5607	Erection of column struts for Glen Canyon Bridge—Yuba Erectors Division, Yuba Consolidated Industries, Inc., contract No. 14-06-D-4040	19,200
DC-5750	Completion of Glen Canyon Powerplant, Switchyard, Dam, and appurtenant works—Ets-Hokin and Galvan, Inc., contract No. 14-06-D-4429	[2] 9,496,591

[1] Summary included in final construction report submitted by regional office July 12, 1968.
[2] This value revised by copy of final voucher received from regional office dated June 3, 1969.

APPENDIX A—Continued

Construction Contracts—Continued

Specifications number	Item, contractor, and contract number	Total cost to the Government
DC-6057	Terrazzo and wall tile installations for Glen Canyon Dam and Powerplant—Marus Marble and Tile Co., Inc., contract No. 14-06-D-5088	$ 71,500
DC-6238	69- and 2.4-kilovolt additions to Glen Canyon Switchyard—Tide Bay, Inc., Tacoma, Wash. contract No. 14-06-D-5493	101,295
DC-6274	Visitor center complex—Allen M. Campbell Co., Tyler, Tex., contract No. 14-06-D-5670	1,275,858
DC-6317	Left diversion tunnel plug and spillway elbow—S. S. Mullen, Inc., Seattle, Wash., contract No. 14-06-D-5670	1,530,013
DC-6351	Furnishing and installing handrailing and safety gates for control cable tunnel—Techni-Builders, Inc., Phoenix, Ariz., contract No. 14-06-D-3773	8,400
DC-6441	Handrailings on generator air housings for Glen Canyon Powerplant—Larsen Rigging and Equipment Co. of Riverton, Utah, contract No. 14-06-D-6020	14,495
—	Hoover deficiencies, reservoir filling expense	2,400,000
DS-5522	Generators—General Electric Co., contract No. 14-06-D-3876	8,476,381
DS-5843	Elevators for powerplant and dam—Pacific Elevator and Equipment Co., contract No. 14-06-D-4628	407,418
DS-6276	Elevators for visitor center—Montgomery Elevator Co., contract No. 14-06-D-5628	97,709
Freight	Reinforcement steel shipped on Government bills of lading for specifications No. DC-4825	454,271
400-424	Single lane access road from damsite (operated equipment)—Union Construction Co., contract No. 14-06-400-520	56,690
400-428	Single lane access road from Utah State line to damsite (operated equipment)—Dee Peterson, contract No. 14-06-414-5; and Ralph Child Construction Co., contract No. 14-06-414-4	6,088
400C-68	Completion of gravel surfacing from Utah State line—Ford-Fielding, Inc., contract No. 14-06-400-544	76,359
400C-83	Earthwork structures and surfacing for Arizona-Utah State line to vista point and parking area—W. W. Clyde and Co., contract No. 14-06-400-624	74,602
400C-98	Earthwork, structures, and surfacing for vista point access road, parking area and shelter—W. W. Clyde and Co., contract No. 14-06-400-685	60,904
400C-140	Soil stabilization and landscaping, Page, Ariz.—L. A. Landscaping Co., contract No. 14-06-400-1007	69,223
400C-189	Water supply modifications for Page, Ariz., Nelson Brothers Construction Co., contract No. 14-06-400-2260	42,091
400C-231(SF)	Sidewalk canopies for Glen Canyon Bridge—Techni-Builders, Inc., Phoenix, Ariz., contract No. 14-06-400-3030	11,189
400C-266q	Aero Avenue waterline extension, Page, Ariz.—E & S Plumbing and Heating Co., contract No. 14-06-400-3718	27,268
400C-291	Log boom for Lake Powell—R. H. Jackson and Assoc., Golden, Colo., contract No. 14-06-400-4021	34,874
400C-292	Roof repairs for Glen Canyon Powerplant—Superior Roofing Co. Inc., Salt Lake City, Utah, contract No. 14-06-400-4025	22,209
—	Miscellaneous construction contracts	91,318
	Total, Construction Contracts	$178,087,355

Supply Contracts

Invitation and purchase order number	Item and contractor	Total cost to the Government
DS-4805	Steel warehouse—Allison Steel Manufacturing Co.	$ 163,529
DS-4810	Mechanical equipment for sewage plant—Process Engineers, Inc.	15,198
DS-4823	Traveling crane—Crane Hoist Engineering Corp.	25,325
DS-4889	Centrifugal pumping units—Allis-Chalmers Manufacturing Co.	32,150
DS-5023	Portland cement—American Cement Corp.	10,649,706
DS-5052	Penstocks and outlet pipes—Vinnell Steel	3,739,784
DS-5053	Pozzolan—J. G. Shotwell	2,510,235
DS-5077	Embedded metalwork for radial gates and pedestal—Dixie Steel and Supply Co.	32,524
DS-5192	Radial gates—United Austrian Iron and Steel Corp.	310,541
DS-5213	Radial gate hoists—Moffett Engineering, Inc.	104,515
DS-5216	Outlet gate valves—Yuba Consolidated Industries, Inc.	509,595
DS-5234	Hydraulic turbines—Baldwin-Lima Hamilton	6,354,975
DS-5252	75-ton traveling crane—The Legnano Corp.	62,598
DS-5260	Traveling cranes and lifting beams—Yuba Consolidated Industries, Inc.	391,102
DS-5265	Oil and water storage tanks—American Steel and Iron Works	11,847
DS-5269	96-inch ring-follower gates—Goslin-Birmingham Manufacturing Co.	278,264
DS-5341	Draft tube bulkhead gates—Ogden Iron Works	126,839
DS-5354 Sch. 1	Stoplogs and lifting frames—United States Steel Corp.	116,893
DS-5354 Sch. 2	Stoplog guides—Bennett Industries, Inc.	180,946
DS-5364	Control for 7- by 10.5-foot outlet gates—Kendo Equipment Co.	5,548
DS-5363	Hollow-jet valves—Goslin-Birmingham Manufacturing Co.	293,242
DS-5370 Sch. 1	Bulkhead gate frames—Omaha Steel Works	46,110
DS-5370 Sch. 2	Anchor bolt for bulkhead gate frames—Fulton Shipyard	1,900
DS-5398	10-ton gantry crane—Crane Hoist Engineering Corp.	10,741
DS-5445	Struts for Glen Canyon Bridge—The Boardman Co.	7,045
DS-5463	Miscellaneous piping and assorted metalwork—The Shaw-Kendall Engineering Co.	89,538
DS-5469	Fixed-wheel gate frames and anchorage for penstock intake—Rockwell Engineering Co.	315,603
DS-5493	Bulkhead gate for river outlets—Johnson Machine Works, Inc.	14,694
DS-5498	Controls for 96-inch ring-follower gates—Kendo, Inc.	17,244
DS-5503	Controls for 96-inch hollow-jet valves—The Rucker Co.	19,013
DS-5512	Trashracks for penstocks and river outlet trashrack stairs—C F and R Steel Fabricating Co.	145,410
DS-5521	165-ton gantry crane--Pacific Coast Engineering Co.	188,733
DS-5562	Governors for turbines—Baldwin-Lima Hamilton Corp.	442,838
DS-5577	Fixed-wheel gates—United Austrian Iron Works	596,322
DS-5591	Spiral stairways—Logan Co.	28,959
DS-5601	Fixed-wheel gate hoists—Pacific Coast Engineering Co.	361,178
DS-5655	Stoplogs and gate slot closures for penstocks and outlet trashracks—Thermo-Dynamics	34,059
DS-5721	345-kilovolt air circuit breakers—General Electric Co.	903,689
DS-5732	Switchgear assembly—Allis-Chalmers Manufacturing Co.	186,797
DS-5751	Main powerplant controls—Westinghouse Electric Corp.	225,826
DS-5579	Steel for service bridge—Missouri Valley Steel, Inc.	36,400
DS-5780 Sch. 1	Power transformers—McGraw Edison Co.	1,135,540
DS-5780 Sch. 2	Power transformers—Westinghouse Electric Corp.	338,399

Supply Contracts—Continued

Invitation and purchase order number	Item and contractor	Total cost to the Government
DS-5784 Sch. 1	345-kilovolt autotransformers—Westinghouse Electric Corp.	$ 350,137
DS-5787 Sch. 1&2	Station-service power boards—Federal Pacific Electric Co.	52,501
DS-5787 Sch. 3	Battery chargers—Federated Engineering Development Corp.	13,894
DS-5787 Sch. 4	Control cabinets—Young Electric and Manufacturing Co.	4,118
DS-5816 Sch. 1&2	Power circuit breakers—Federal Pacific Electric Co.	229,448
DS-5828 Sch. 1	Generator voltage iso-phase bus structures station-service power transformers—Westinghouse Electric Corp.	495,968
DS-5828 Sch. 2, 3, & 4	Voltage regulator, shunt reactors, switchgear assemblies—General Electric Co.	512,366
DS-5830	Spiral stairways—Logan Company	22,573
DS-5860 Sch. 1	23-kilovolt interrupter switch—I-T-E Circuit Breaker Co.	2,001
DS-5860 Sch. 2	Disconnect switches, 23-, 230-, and 345-kilovolt—Schwager-Wood Co., Inc.	145,698
DS-5860 Sch. 3	Disconnect fuses, 345-kilovolt—General Electric Co.	1,250
DS-5875 Sch. 1	25-kilovolt grounding transformer—R. E. Uptegraff Manufacturing Co.	23,636
DS-5875 Sch. 2	Regulation voltage transformer—Ferranti Electric, Inc.	30,758
DS-5976 Sch. 1	Furnish and install automatic telephone switchboard—Stromberg-Carlson	21,737
DS-6164	69-kilovolt power transformer, Glen Canyon Switchyard-McGraw Edison Co.	70,576
DS-6227	Line relay board—Keystone Electrical Manufacturing Co.	20,699
DS-6293	25-kilovolt (1,500 kv.-a.) power transformer—Central Transformer Corp.	9,680
(D)90,502-A	Electrical cabinets for water supply system for Page, Ariz.—Aero Test Equipment Co., Inc.	16,801
(D)90,504-A	VHF radio equipment and antenna tower—Schneider Electric and Equipment Co.	8,700
(D)90,505-A	Anchorage for spillway radial gates embedded metalwork—Gardiner Manufacturing Co.	3,521
(D)90,507-A	Metalwork for spillways—Halley Welding and Manufacturing Co.	1,373
(D)90,507-A1	Spillway hoist bridge bearings—Dixie Steel and Supply Co.	4,992
(D)90,512-A	Terminal board and cover box for 20 meter terminations—Nelson Electric Manufacturing Co.	2,717
(D)90,514-A	Electrical cable—General Electric Co.	16,574
(D)90,515-A	Strainmeters and jointmeters—Roy W. Carlson	26,354
(D)90,517-A	Handrail, spillway intake structure—John E. Casey Co.	6,125
(D)90,519-A	Thermocouple extension wire for concrete cooling control—Brown Instruments Division, Minneapolis-Honeywell Regulator Co.	2,546
(D)90,523-A	Anchor bolts for turbine draft tube bulkhead gate seats and guides—Monarch Steel Products Co., Inc.	3,050
(D)90,524-A	Metalwork for pump chamber shaft and sump—Metalcraft Welding Fabrication Co., Inc.	1,185
(D)90,528-A	Backfill, shields, penstocks—Allison Steel Manufacturing Co.	4,330
(D)90,529-A	Embedded metalwork (Dam)—Allison Steel Manufacturing Co.	2,212
(D)90,530-A	Water supply and drainpiping—Midwest Piping Co.	37,483
(D)90,533-A	Frames and covers for penstocks—Berkeley Steel Construction Co.	1,181
(D)90,534-A	Bellmouth castings for 96-inch river outlet pipes—Chas. C. Steward Machine Co.	15,564

APPENDIX A–Continued

Supply Contracts–Continued

Invitation and purchase order number	Item and contractor	Total cost to the Government
(D)90,538-A	Rubber seals for radial gates–Kirkhill Rubber Co.	$ 2,170
(D)90,539-A	Babbitt for radial gate pedestal bases–Adolph Kiesler dba Peerless Alloy Co.	6,056
(D)90,545-A	Steel pipe, valves, fittings, and accessories–American Steel and Iron Works, Inc.	32,214
(D)90,545-A1	Gate valves, 20-inch–The Chapman Valve Manufacturing Co.	44,012
(D)90,548-A	Handrail, ladders and gratings–Metalcraft Welding and Fabricating Co., Inc.	1,447
(D)90,550-A	Roadway lighting--General Electric Supply Co.	1,131
(D)90,552-A	Drainpiping–The Greene-Wolf Co., Inc.	2,648
(D)90,560-A	Anchor bolts for stoplog–St. Louis Screw and Bolt Co.	3,439
(D)90,562-A	Strainmeters, stressmeters, and jointmeters–Roy W. Carlson	24,161
(D)90,563-A	Electrical cable for connecting strainmeters, jointmeters, etc.–The Okonite Co.	10,674
(D)90,567-A	Ladders and gratings for penstock supports–Berkeley Steel Construction Co., Inc.	1,231
(D)90,568-A	Handling equipment and anchors for ring-follower gates–Walter Weyman Co., Inc.	1,296
(D)90,575-A	Metalwork and handrail–Metalcraft Welding and Fabricating Co., Inc.	3,807
(D)90,576-A	Rubber seals for bulkhead gates–H. K. Porter Co., Inc.	1,944
(D)90,578-A	Portable oil purifiers–The DeLaval Separator Co.	13,108
(D)90,579-A	Transformer deck lighting installation –Gough Industries, Inc.	3,061
(D)90,581-A	Air compressors–Gardner Denver Co.	25,737
(D)90,581-A1	Air compressor, 20-c.f.m.–Compressor Service Co.	2,489
(D)90,581-A2	Air receivers–Richmond Engineering Co., Inc.	8,955
(D)90,581-A3	Aftercoolers–Joy Manufacturing Co.	1,885
(D)90,582-A	Transfer car–Omaha Steel Works	2,655
(D)90,583-A	Transfer pumps for lubricating and insulating oil–Colorado Pump and Supply Co.	1,094
(D)90,584-A	Erection and storage of fixed-wheel and bulkhead gates–The DeLaney Co.	1,328
(D)90,585-A	Pumping units–Fiese and Firstenberger Manufacturing Co., Inc.	29,222
(D)90,586-A	Water level gages–Leupold and Stevens Instruments, Inc.	2,649
(D)90,587-A	Sewage disposal units–Waterworks Equipment Co.	6,913
(D)90,589-A	Inclined stairs–Creamer and Dunlap, Inc.	7,049
(D)90,591-A	Electrical cable–Okonite Co.	1,903
(D)90,595-A	Electric cable for connecting strain, stress and jointmeters–Electrical Distributing Co.	10,948
(D)90,596-A	Strain, stress and jointmeters–Roy W. Carlson	32,463
(D)90,600-A	Terminal boards and cover boxes–Electric Industrial Equipment and Supply Corp.	2,137
(D)90,601-A	Pumping units for cooling water–Aurora Pump Division, The New York Air Brake Co.	14,515
(D)90,604-A	Reinjectable grout valves–Societe D'Equipment Industrial	1,827
(D)90,605-A	Pumping units–Domestic water facilities–Allis-Chalmers Manufacturing Co.	16,094
(D)90,605-A1	Pumping units for domestic water–U.S. Pumps, Inc.	1,195
(D)90,605-A2	Pumping units for domestic water–C. H. Wheeler Manufacturing Co.	1,315

Supply Contracts—Continued

Invitation and purchase order number	Item and contractor	Total cost to the Government
(D)90,606-A	Fire extinguishing system—Cardox Division of Chemetron Corp.	$ 32,390
(D)90,610-A	Ladders and platforms for elevator shafts—Omaha Steel Works	19,986
(D)90,612-A	Thermocouple wire—Minneapolis Honeywell Regulator Co.	2,035
(D)90,616-A	Covers, frames, and anchor bolt—Telluride Iron Works Co.	3,944
(D)90,617-A	Inclined stairways—Grand Junction Steel Fabricating Co.	1,617
(D)90,618-A	Vertical drain covers for stairwells—Bechtold Engineering Services, Inc.	2,323
(D)90,620-A	Strainers for piping system—Staples and Pfeiffer	3,998
(D)90,620-B	Strainers for piping system—Zurn International Division, Zurn Industries, Inc.	37,349
(D)90,621-A	Frame and cover for gate service-shaft river outlets—Bechtold Engineering Services, Inc.	1,009
(D)90,622-A	Water supply pumping units—G. M. Wallace and Co.	39,415
(D)90,622-A1	Booster pump unit—U.S. Pumps, Inc.	2,774
(D)90,623-A	Special valves for powerplant—Central Pipe and Supply Co.	22,611
(D)90,624-A	Portable oil filter press unit—Bowser, Inc.	3,745
(D)90,625-A	Chlorinating equipment for domestic water supply—Fischer and Porter Co.	2,567
(D)90,626-A	Control cubicle and cabinet—Gustaveson, Inc.	17,005
(D)90,627-A	Rails and accessories for crane—Omaha Steel Works	25,485
(D)90,632-A	Gratings—Borden Metal Products Co.	2,252
(D)90,633-A1	Special valves for powerplant—Combination Pump Valve Co.	3,566
(D)90,633-A2	Special valves for powerplant—Cla-Val Co.	3,781
(D)90,635-A	Coupling capacitor potential device—General Electric Co.	40,609
(D)90,636-A	Water supply and drainpiping—Monsey Iron and Metal Co.	4,998
(D)90,637-A	Lifting frame for bulkhead gate and slot closures—Chas. C. Steward Machine Co.	5,078
(D)90,638-A	Penstock gate controls—Auto-Control Lab., Inc.	49,798
(D)90,639-A	25-kilovolt distribution transformer—Esco Manufacturing Co.	3,040
(D)90,641-A	Potential transformers—American Elin Corp.	27,805
(D)90,642-A	Current transformers—Westinghouse Electric Corp.	21,901
(D)90,643-A	Power receptacle box for parapet—Metalcraft Welding and Fabricating Co., Inc.	1,481
(D)90,644-A	Parapet lighting fixtures—General Electric Supply Co.	9,034
(D)90,646-A	Current limiting reactors—General Electric Co.	5,363
(D)90,647-A	Lightning arresters—Ohio Brass Co.	15,987
(D)90,651-A	Power cable for switchyard—Simplex Wire and Cable Co.	21,182
(D)90,652-A	Gate hoist structure removable covers—A and A Iron Works	20,035
(D)90,653-A	Lubricating oil—Texaco, Inc.	10,983
(D)90,654-A	Distribution panelboard—Korr Distributors	2,484
(D)90,655-A	Control and terminal cabinets—Western Switchboard Co.	4,554
(D)90,657-B	Generator neutral grounding equipment—General Electric Co.	19,417
(D)90,658-B	Oil purification equipment—Bowser, Inc.	18,945
(D)90,660-A	Flexible metal hose connections for transformers—Atlantic Metal Hose Co., Inc.	3,507
(D)90,661-A	Control and distribution boards for right spillway gates and lighting—Korr Distributors	3,268
(D)90,663-A1	Pressure reducing valves for powerplant—Keenan Pipe and Supply Co.	8,632

Supply Contracts—Continued

Invitation and purchase order number	Item and contractor	Total cost to the Government
(D)90,665-A	Electrical system distribution panelboards—Federal Pacific Electric Co.	$ 6,682
(D)90,665-A1	Distribution transformers and switches—Korr Distributors	3,963
(D)90,665-A2	Circuit breakers, switches, and relays—Mine and Smelter Supply Co.	1,513
(D)90,668-A	Material for deformation gages—Keuffel and Esser Co.	1,135
(D)90,670-A	Stenographic reporting service on hearing of appeal case, specifications No. DC-4825—Ace-Federal Reporting Co.	2,400
(D)90,672-A	Cabinets and relays—Gustaveson, Inc.	1,800
(D)90,672-B	Relays and terminal blocks—Westinghouse Electric Corp.	1,418
(D)90,673-A	Storage battery and access—Gould National Batteries	12,222
(D)90,675-A	Power supply equipment for vacuum oil purification unit—General Electric Co.	1,783
(D)90,691-A	Unmounted equipment kits for modifications to station-service switchboards—Federal Pacific Electric Co.	4,445
(D)90,700-A	69-kilovolt power circuit breakers—McGraw Edison Co.	22,730
(D)90,708-A	69-kilovolt disconnect switches—H. K. Porter Co., Inc.	9,491
(D)90,715-A	Lightning arresters—General Electric Co.	1,096
(D)90,718-A	Varmeters—Westinghouse Electric Corp.	1,697
(D)90,720-A	Potential and current transformers—American Elin Corp.	10,820
(D)90,721-A	Bushing current transformers—General Electric Co.	17,587
(D)90,725-A	Unmounted control board equipment for 69-kilovolt switchyard—Westinghouse Electric Corp.	2,731
(D)92,064-A	Scanning system equipment—Quindor Electric	2,540
(D)92,039-A	Carrier-current relay equipment—General Electric Co.	2,933
(D)92,050-A	Line trap for powerplant—Westinghouse Electric Corp.	3,500
(D)92,049-A	Field acceptance tests—Westinghouse Electric Corp.	1,950
400-426	Temporary housing units—Transa Homes, Inc.	238,830
—	Miscellaneous materials and supplies for erecting transa homes	54,444
(D)45,1113-A	Plaque—Alton Iron Works	7,000
(D)45,973-A	Out-of-step blocking relay—Westinghouse Electric Corp.	1,501
(D)69,805-A	Comminutor—Chicago Pump Co.	1,904
420-63-6	Furnish and stockpile aggregate—Flagstaff Service and Material Co.	153,745
420-778	Furnish and stockpile aggregate—Flagstaff Service and Material Co.	170,734
—	Miscellaneous materials used in construction of temporary office and other buildings	53,144
purchase orders	Lubricating oil, inhibited type—Texaco, Inc.	22,660
	Miscellaneous small supply contracts and warehouse stock	298,512
	Total, Supply Contracts	$ 35,192,407
	Total, Contractual Costs	$213,279,762

Noncontractual Costs

	Total cost to the Government
Land rights	$ 421,876
Labor by Government forces	394,904
Construction facilities	905,768
Investigations, engineering, and other costs:	
Investigations	1,560,384
Engineering and supervision	8,634,019
Design and specifications	9,669,859
General services	7,797,749
Service facilities	2,348,664
Total, Noncontractual Costs	$ 31,733,223
Total, Contractual and Noncontractual Costs as of June 30, 1967	$245,012,985
Less transfers and credits	142,435
Total cost of Glen Canyon Dam, Powerplant and Appurtenant Structures as of June 30, 1967	$244,870,550

APPENDIX B

Construction costs by pay items—Glen Canyon Dam and Powerplant—Specifications No. DC-4825, contract No. 14-06-D-2403.

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
1.	Diversion and care of the river during construction and unwatering foundations				
	(a) Right diversion gate structure	100%	Lump sum	$ 234,145.00	$ 234,145.00
	(b) Upstream cofferdam	100%	Lump sum	2,100,500.00	2,100,500.00
	(c) Downstream cofferdam	100%	Lump sum	450,000.00	450,000.00
	(d) Foundation dewatering	100%	Lump sum	175,800.00	175,800.00
	(e) Right gate closure and river control	100%	Lump sum	39,500.00	39,500.00
	(f) Removal of downstream cofferdam	100%	Lump sum	674,200.00	674,200.00
2.	Removal of rock slabs from canyon walls	99,443.01	cu. yd.	10.00	994,430.10
3.	Excavation, common, in opencut for dam, powerplant, and appurtenant works, first 500,000 cubic yards	500,000.0	cu. yd.	3.30	1,650,000.00
4.	Excavation, common, in opencut for dam, powerplant, and appurtenant works, over 500,000 cubic yards	464,314.0	cu. yd.	0.60	278,588.40
5.	Excavation, rock, in opencut for dam, powerplant, and appurtenant works, first 800,000 cubic yards	800,000.0	cu. yd.	5.00	4,000,000.00
6.	Excavation, rock, in opencut for dam, powerplant, and appurtenant works, over 800,000 cubic yards	1,531,742.97	cu. yd.	1.50	2,297,614.46
7.	Excavation, all classes, in opencut for diversion tunnels	316,790.0	cu. yd.	3.00	950,370.00
8.	Excavation, all classes, in opencut for spillways	973,705.5	cu. yd.	4.00	3,894,822.00
9.	Deleted.				
10.	Excavation, common, in opencut for switchyard	16,947.5	cu. yd.	5.00	84,737.50
11.	Excavation, rock, in opencut for switchyard	19,040.9	cu. yd.	5.00	95,204.50
12.	Drilling line holes for rock excavation	143,368.45	lin. ft.	1.00	143,368.45
13.	Excavation, all classes, in diversion tunnels	186,596.91	cu. yd.	20.00	3,731,938.20
14.	Excavation, all classes, in spillway tunnels	129,017.73	cu. yd.	20.00	2,580,354.60
15.	Excavation, all classes, in foundation tunnels	5,523.74	cu. yd.	30.00	165,712.20
16.	Excavation, all classes, in control tunnel	3,596.8	cu. yd.	35.00	125,888.00
17.	Furnishing and placing permanent structural-steel tunnel supports	2,184,323.68	lb.	0.30	655,297.10

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	*(a) Materials delivered to site (85 percent of paid invoices)		Lump sum		$ 216,424.35
	(b) Deduction for materials placed	1,731,394.81	lb.	$ 0.125	–216,424.35
18.	Furnishing and installing tunnel roof support bolts	322,384.0	lin. ft.	6.00	1,934,304.00
19.	Furnishing and installing chain-link fabric for tunnel roof support	2,290.27	sq. yd.	9.00	20,612.43
20.	Backfill	293,336.8	cu. yd.	1.00	293,336.80
21.	Constructing embankments for switchyard	17,793.4	cu. yd.	2.00	35,586.80
22.	Rockfill adjacent to powerplant	0	cu. yd.	15.00	0
23.	Riprap	0	cu. yd.	2.50	0
24.	Compacted backfill	7,267.8	cu. yd.	2.00	14,535.60
25.	Core drilling NX holes in stage between depths of 0 and 50 feet	2,127.9	lin. ft.	6.00	12,767.40
26.	Core drilling NX holes in stage between depths of 50 and 100 feet	155.4	lin. ft.	6.00	932.40
27.	Core drilling NX holes in stage between depths of 100 and 200 feet	40.0	lin. ft.	7.00	280.00
28.	Core drilling 5-1/2-in. dia. holes not more than 50 feet deep	0	lin. ft.	9.00	0
29.	Drilling grout holes in stage between depths of 0 and 30 feet	69,228.46	lin. ft.	4.00	276,913.84
30.	Drilling grout holes in stage between depths of 30 and 60 feet	19,809.4	lin. ft.	3.50	69,332.90
31.	Drilling grout holes in stage between depths of 60 and 110 feet	6,828.0	lin. ft.	3.50	23,898.00
32.	Drilling grout holes in stage between depths of 110 and 160 feet	3,470.0	lin. ft.	3.50	12,145.00
33.	Drilling grout holes in stage between depths of 160 and 210 feet	1,631.0	lin. ft.	4.00	6,524.00
34.	Drilling grout holes in stage between depths of 210 and 260 feet	1,108.0	lin. ft.	4.00	4,432.00
35.	Drilling grout holes in stage between depths of 260 and 310 feet	103.0	lin. ft.	4.00	412.00
36.	Drilling grout holes in stage between depths of 310 and 360 feet	0	lin. ft.	4.00	0
37.	Furnishing and placing metal pipe and fittings for foundation grouting and drainage	359,573.15	lb.	0.60	215,743.89

*As provided under Paragraph 17 of the specifications.

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	*(a) Materials delivered to site (85 percent of paid invoices)		Lump sum		$ 42,692.29
	(b) Deduction for materials placed	284,615.28	lb.	$ 0.15	–42,692.29
38.	Hookups to foundation and tunnel grout holes	3,219.0	Hookup	20.00	64,380.00
39.	Pressure grouting foundations and tunnels	24,286.0	Sacks	3.00	72,858.00
40.	Furnishing and installing metal tubing and fittings for grouting contraction joints	723,859.09	lb.	0.75	542,894.32
	*(a) Materials delivered to site (85 percent of paid invoices)		Lump sum		105,435.66
	(b) Deduction for materials placed	351,452.20	lb.	0.30	–105,435.66
41.	Hookups to contraction joint grouting systems	799.0	Hookup	30.00	23,970.00
42.	Pressure grouting contraction joints and cooling systems	59,198.0	Sacks	3.00	177,594.00
43.	Drilling drainage holes in stage between depths of 0 and 25 feet	14,933.86	lin. ft.	5.00	74,669.30
44.	Drilling drainage holes in stage between depths of 25 and 50 feet	13,778.3	lin. ft.	5.00	68,891.50
45.	Drilling drainage holes in stage between depths of 50 and 75 feet	9,878.2	lin. ft.	5.00	49,391.00
46.	Drilling drainage holes in stage between depths of 75 and 100 feet	7,177.8	lin. ft.	4.00	28,711.20
47.	Drilling drainage holes in stage between depths of 100 and 150 feet	5,052.0	lin. ft.	4.00	20,208.00
48.	Drilling drainage holes in stage between depths of 150 and 200 feet	73.0	lin. ft.	4.25	310.25
49.	Drilling drainage holes in stage between depths of 200 and 250 feet	0	lin. ft.	4.25	0
50.	Drilling drainage holes in spillway tunnels to a depth not to exceed 25 feet	13,334.8	lin. ft.	6.00	80,008.80
51.	Constructing 8-inch-diameter sewerpipe drains with open joints	1,353.8	lin. ft.	4.00	5,415.20
52.	Furnishing and laying sewerpipe with calked joints	1,024.15	lin. ft.	5.00	5,120.75
53.	Constructing split sewerpipe drains with cemented joints	1,196.25	lin. ft.	10.00	11,962.50
54.	Porous concrete	39.15	cu. yd.	150.00	5,872.50

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
55.	Drilling holes for anchor bars and grouting bars in place	30,044.95	lin. ft.	$ 2.50	$ 75,112.38
56.	Furnishing and handling cement	94,110.99	bbl.	6.60	621,132.54
57.	Deleted.				
58.	Deleted.				
59.	Deleted.				
60.	Special finishing of concrete surfaces	3,118.83	sq. yd.	20.00	62,376.60
61.	Detailing, furnishing, and placing reinforcement bars	24,959,687.0	lb.	0.15	3,743,953.05
	*(a) Material delivered to site (85 percent of paid invoices and freight, not to exceed $0.0675/lb.)		Lump sum		1,585,622.21
	(b) Deduction for material placed	23,490,699.34	lb.	0.0675	–1,585,622.21
62.	Furnishing and placing reinforcement bars	10,393,647.0	lb.	0.15	1,559,047.05
	*(a) Material delivered to site (85 percent of paid invoices and freight, not to exceed $0.0675/lb.)		Lump sum		542,232.12
	(b) Deduction for material placed	8,033,068.44	lb.	0.0675	–542,232.12
63.	Concrete in dam, first 2,400,000 cubic yards	2,400,000.00	cu. yd.	11.00	26,400,000.00
	*(a) Advance payment for preparatory work for concrete plant 85 percent of expenditures		Lump sum		3,000,000.00
	(b) Deduction for material placed	2,400,000.00	cu. yd.	1.25	–3,000,000.00
64.	Concrete in dam, over 2,400,000 cubic yards	2,462,439.94	cu. yd.	9.50	23,393,179.43
	*(a) Advance payment for preparatory work for concrete plant 85 percent of expenditures		Lump sum		1,500,000.00
	(b) Deduction for material placed	1,200,000.00	cu. yd.	1.25	–1,500,000.00
65.	Concrete in sidewalks and parapets	1,130.98	cu. yd.	120.00	135,717.60
66.	Concrete in elevator towers and vista canopies	1,198.50	cu. yd.	150.00	179,775.00
67.	Concrete in gate hoist and stem storage platform and gate erection platform	106.83	cu. yd.	70.00	7,478.10

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
68.	Concrete in trashracks and gate-hoist structure	17,172.89	cu. yd.	$ 130.00	$ 2,232,475.70
69.	Second-stage concrete in ring-follower gate chamber	618.14	cu. yd.	20.00	12,362.80
70.	Concrete in blockouts	1,791.51	cu. yd.	100.00	179,151.00
71.	Concrete in floors of spillway channels downstream from down-stream tunnel portals	0	cu. yd.	25.00	0
72.	Concrete in spillway hoist bridges	504.50	cu. yd.	90.00	45,405.00
73.	Concrete in crests of spillway intake structures	7,192.30	cu. yd.	35.00	251,730.50
74.	Concrete in walls and piers of spillway intake structures	15,569.92	cu. yd.	30.00	467,097.60
75.	Concrete in lining of right diver-sion tunnel	20,676.27	cu. yd.	36.00	744,345.72
76.	Concrete in lining of left diver-sion tunnel	17,228.93	cu. yd.	37.00	637,470.41
77.	Concrete in lining of spillway tunnels upstream of station 26+11.72	44,654.07	cu. yd.	40.00	1,786,162.80
78.	Concrete in lining of right spill-way tunnel downstream of station 26+11.72	17,696.98	cu. yd.	40.00	707,879.20
79.	Concrete in lining of left spill-way tunnel downstream of station 26+11.72	18,817.71	cu. yd.	40.00	752,708.40
80.	Concrete in tunnel plugs and back-fill concrete	21,911.01	cu. yd.	30.00	657,330.30
81.	Concrete in gutters and drop inlets	161.59	cu. yd.	50.00	8,079.50
82.	Concrete in penstock supports and vaults	14,407.66	cu. yd.	35.00	504,268.10
83.	Concrete in floors of foundation tunnels	1,046.54	cu. yd.	16.00	16,744.64
84.	Mass concrete beneath machine shop	0	cu. yd.	10.00	0
85.	Concrete in river outlets valve structure superstructure	24.97	cu. yd.	90.00	2,247.30
86.	Concrete in gravity-type walls	2,245.56	cu. yd.	17.00	38,174.52
87.	Concrete in line-drilled-type walls	1,241.14	cu. yd.	25.00	31,028.50
88.	Mass concrete around river outlet pipes and in valve structure substructure	0	cu. yd.	10.00	0
89.	Mass concrete for powerplant structure, except machine shop	68,590.04	cu. yd.	12.00	823,080.48
90.	First-stage concrete in powerplant substructure	36,132.75	cu. yd.	14.00	505,858.50
91.	First-stage concrete in powerplant intermediate structure	43,798.29	cu. yd.	50.00	2,189,914.50

APPENDIX B–Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
92.	Concrete in powerplant structure cover slabs	428.03	cu. yd.	$ 55.00	$ 23,541.65
93.	First-stage concrete in powerplant superstructure	4,859.02	cu. yd.	120.00	583,082.40
94.	Cast-in-place or precast concrete powerplant superstructure walls	66,597.54	sq. ft.	6.00	399,585.24
95.	Drilling 10-inch-diameter core specimens in concrete	312.6	lin. ft.	60.00	18,756.00
96.	Furnishing and placing 1-inch outside-diameter metal pipe or tubing and fittings for concrete cooling system	4,139,304.05	lin. ft.	0.27	1,117,612.09
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		155,361.03
	(b) Deduction for material placed	1,726,233.72	lin. ft.	0.09	–155,361.03
97.	Furnishing and placing 1-1/2-inch standard pipe and fittings for concrete cooling systems	245,131.96	lin. ft.	0.60	147,079.18
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		54,596.63
	(b) Deduction for material placed	181,988.76	lin. ft.	0.30	–54,596.63
98.	Constructing type A control joints	1,454.62	lin. ft.	10.00	14,546.20
99.	Furnishing and placing 1/2-inch bituminous joint filler	846.65	sq. ft.	2.00	1,693.30
100.	Furnishing and placing 1/2-inch sponge rubber joint filler	2,891.48	sq. ft.	4.00	11,565.92
101.	Furnishing and placing 2-inch sponge rubber joint filler	6,023.80	sq. ft.	5.00	30,119.00
102.	Furnishing and placing 1-inch cork board joint filler	45,898.48	sq. ft.	1.00	45,898.48
103.	Furnishing and placing 1/2-inch cork board joint filler	1,689.57	sq. ft.	0.80	1,351.66
104.	Furnishing and installing rubber joint strips with metal straps	443.8	lin. ft.	8.00	3,550.40
105.	Furnishing and installing rubber joint strips without metal straps	1,138.7	lin. ft.	5.00	5,693.50
106.	Furnishing and placing type D rubber waterstops	881.83	lin. ft.	4.00	3,527.32
107.	Furnishing and placing type E rubber waterstops	2,612.47	lin. ft.	5.00	13,062.35
108.	Furnishing and placing type F rubber waterstops	7,145.06	lin. ft.	5.00	35,725.30
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		922.08

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	(b) Deduction for material placed		Lump sum		$ -922.08
109.	Furnishing and placing metal seals in dam	124,001.38	lin. ft.	$ 3.00	372,004.14
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		100,523.83
	(b) Deduction for material placed	87,412.03	lin. ft.	1.15	–100,523.83
110.	Furnishing and placing metal seals, type N2, in powerplant	8,770.95	lin. ft.	3.00	26,312.85
111.	Furnishing and placing metal grout groove covers	45,006.3	lin. ft.	3.00	135,018.90
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		4,930.00
	(b) Deduction for material placed	41,083.32	lin. ft.	0.12	–4,930.00
112.	Constructing asphalt seals	2,123.98	lin. ft.	7.00	14,867.86
113.	Furnishing and applying concrete floor hardener	5,565.43	sq. yd.	1.50	8,348.14
114.	Furnishing and placing membrane waterproofing	14,928.37	sq. ft.	1.00	14,928.37
115.	Furnishing and applying 2-coat asphalt-emulsion dampproofing	185.5	sq. yd.	2.00	371.00
116.	Furnishing and installing precast concrete roof slabs	36,840.19	sq. ft.	1.50	55,260.29
117.	Furnishing and placing roof insulation	18,535.81	sq. ft.	0.80	14,828.65
118.	Furnishing and placing 4-ply, asphalt saturated built-up roofing	20,243.24	sq. ft.	0.60	12,145.94
119.	Sandfills for powerplant roofs	0	cu. yd.	7.00	0
120.	Furnishing and installing protective timber covering for powerplant roofs	0	Mb.m.	400.00	0
121.	Furnishing and placing asphalt board and fabric roofing	0	sq. ft.	1.00	0
122.	Furnishing and erecting steel for powerplant superstructure	5,204,006.00	lb.	0.20	1,040,801.20
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		774,090.69
	(b) Deduction for material placed	5,160,604.60	lb.	0.15	–774,090.69
123.	Furnishing and installing aluminum rolling door	1,222.66	sq. ft.	10.00	12,226.60
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		5,525.00

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	(b) Deduction for material placed	1,227.778	sq. ft.	$ 4.50	$ –5,525.00
124.	Furnishing and installing aluminum windows	165.52	sq. ft.	7.00	1,158.64
125.	Furnishing and installing fixed stormproof louvers	268.03	sq. ft.	6.00	1,608.18
126.	Furnishing and installing operable stormproof louvers	30.6	sq. ft.	9.00	275.40
127.	Furnishing and installing automatic louvers	255.36	sq. ft.	12.00	3,064.32
128.	Furnishing and installing cast iron soil pipe and fittings	226,315.40	lb.	0.60	135,789.24
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		14,928.26
	(b) Deduction for material placed	74,641.31	lb.	0.20	–14,928.26
129.	Furnishing and installing cast iron bell traps	4,773.0	lb.	0.60	2,863.80
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		1,324.94
	(b) Deduction for material placed		Lump sum		–1,324.94
130.	Furnishing and installing cast iron bell-and-spigot and flanged pipe, wall pipes, and fittings	115,265.82	lb.	0.35	40,343.04
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		20,135.62
	(b) Deduction for material placed		Lump sum		–20,135.62
131.	Furnishing and installing wrought iron pipe and fittings 2 inches and smaller in nominal diameter	2,341.72	lb.	1.00	2,341.72
132.	Furnishing and installing wrought iron pipe and fittings 2-1/2 inches and larger in nominal diameter	104,593.04	lb.	0.75	78,444.78
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		27,594.03
	(b) Deduction for material placed	86,231.34	lb.	0.32	–27,594.03
133.	Furnishing and installing valves 2 inches and smaller in nominal diameter	2.5	lb.	20.00	50.00
134.	Furnishing and installing valves 2-1/2 inches and larger in nominal diameter, except draft-tube drain valves	24,235.34	lb.	1.50	36,353.01

Pay item	Property and pay item description	Quantity Amount	Quantity Unit	Labor and materials by contractor Unit cost	Labor and materials by contractor Total cost
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		$ 13,012.61
	(b) Deduction for material placed	21,687.68	lb.	$ 0.60	–13,012.61
135.	Furnishing and installing draft-tube drain valves	8.00	Valve	3,000.00	24,000.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		6,005.90
	(b) Deduction for material placed	8.00	Valve	750.74	–6,005.90
136.	Installing metal pipe, fittings, and valves	767,734.56	lb.	0.50	383,867.28
137.	Furnishing and installing gratings for draft-tube piers	16	Grating	600.00	9,600.00
138.	Installing four oil storage tanks	100%	Lump sum	20,000.00	20,000.00
139.	Installing nonembedded metalwork	291,838.72	lb.	0.25	72,959.68
140.	Installing embedded metalwork	118,394.47	lb.	0.30	35,518.34
141.	Installing anchor bolts	39,070.29	lb.	0.50	19,535.15
142.	Installing stairways	325,896.74	lb.	0.30	97,769.02
143.	Installing trashracks and slot closures	1,323,374.0	lb.	0.14	185,272.36
144.	Installing pipe handrails	14,699.28	lb.	0.75	11,024.46
145.	Installing track for 190-ton gantry crane	180,213.08	lb.	0.20	36,042.62
146.	Furnishing and erecting chain-link fences	97.0	lin. ft.	8.00	776.00
147.	Furnishing and installing miscellaneous metalwork in powerplant and machine shop	346,458.65	lb.	0.74	256,379.40
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		51,501.48
	(b) Deduction for material placed	147,147.08	lb.	0.35	–51,501.48
148.	Installing cranes	100%	Lump sum	45,000.00	45,000.00
149.	Testing cranes	100%	Lump sum	20,000.00	20,000.00
150.	Installing penstocks outlet pipes and bellmouths	18,829,646.0	lb.	0.14	2,636,150.44
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		16,688.34
	(b) Deduction for material placed		Lump sum		–16,688.34
151.	Installing bulkhead gate frames and anchor bolts	0	Lump sum	20,000.00	0
152.	Installing fixed-wheel gate frames and anchor bolts	0	Lump sum	85,000.00	0

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
153.	Installing bulkhead gate in outlet works	100%	Lump sum	$ 15,000.00	$ 15,000.00
154.	Installing eight fixed-wheel gates in penstock intakes	100%	Lump sum	240,000.00	240,000.00
155.	Installing high-pressure gates and metal transitions	0	Lump sum	75,000.00	0
156.	Installing control equipment and piping for high-pressure gates	0	Lump sum	17,000.00	0
157.	Furnishing and installing two slide gates, lifts, and anchor bolts	100%	Lump sum	10,000.00	10,000.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		2,544.90
	(b) Deduction for material placed		Lump sum		−2,544.90
158.	Furnishing and installing two flap gates	100%	Lump sum	11,000.00	11,000.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		1,518.10
	(b) Deduction for material placed		Lump sum		−1,518.10
159.	Installing fixed-wheel penstock gate hoists	100%	Lump sum	225,000.00	225,000.00
160.	Installing radial-gate hoists	0	Lump sum	100,000.00	0
161.	Installing radial gates and embedded metalwork	100%	Lump sum	245,000.00	245,000.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		12,176.75
	(b) Deduction for material placed		Lump sum		−12,176.75
162.	Installing four 96-inch hollow-jet valves, adapters, and controls	100%	Lump sum	89,800.00	89,800.00
163.	Installing four 96-inch ring-follower gates, complete with hoists	100%	Lump sum	95,000.00	95,000.00
164.	Installing ring-follower gate and fixed-wheel gate hoist controls	100%	Lump sum	45,000.00	45,000.00
165.	Installing draft-tube bulkhead gates	100%	Lump sum	65,000.00	65,000.00
166.	Installing seats and guides for stoplogs and draft-tube bulk-head gates	1,181,373.19	lb.	0.20	236,274.64
167.	Installing lifting frames	100%	Lump sum	9,000.00	9,000.00
168.	Painting and storing stoplogs	100%	Lump sum	16,000.00	16,000.00
169.	Furnishing and installing grounding system	36,690.26	lb.	3.60	132,084.94

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		$ 16,332.77
	(b) Deduction for material placed		Lump sum		–16,332.77
170.	Finishing recesses for lighting units, lighting panelboards, miscellaneous powerboards, and other equipment	105	Recess	$ 50.00	5,250.00
171.	Furnishing and installing embedded electrical rigid metal conduit 1/2 inch in diameter	17.6	lin. ft.	3.00	52.80
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		62.35
	(b) Deduction for material placed		Lump sum		–62.35
172.	Furnishing and installing embedded electrical rigid metal conduit 3/4 inch in diameter	40,003.9	lin. ft.	2.50	100,009.75
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		7,317.22
	(b) Deduction for material placed		Lump sum		–7,317.22
173.	Furnishing and installing embedded electrical rigid metal conduit 1 inch in diameter	18,564.21	lin. ft.	2.75	51,051.58
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		7,025.25
	(b) Deduction for material placed		Lump sum		–7,025.25
174.	Furnishing and installing embedded electrical rigid metal conduit 1-1/2 inches in diameter	13,676.52	lin. ft.	3.50	47,867.82
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		6,202.70
	(b) Deduction for material placed		Lump sum		–6,202.70
175.	Furnishing and installing embedded electrical rigid metal conduit 2 inches in diameter	10,530.6	lin. ft.	4.00	42,122.40
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		7,308.30
	(b) Deduction for material placed		Lump sum		–7,308.30

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
176.	Furnishing and installing embedded electrical rigid metal conduit 2-1/2 inches in diameter	11,224.54	lin. ft.	$ 4.10	$ 46,020.61
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		4,539.51
	(b) Deduction for material placed	5,674.39	lin. ft.	0.80	–4,539.51
177.	Furnishing and installing embedded electrical rigid metal conduit 3 inches in diameter	4,578.6	lin. ft.	5.25	24,037.65
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		11,875.86
	(b) Deduction for material placed		Lump sum		–11,875.86
178.	Furnishing and installing embedded electrical rigid metal conduit 3-1/2 inches in diameter	6,791.9	lin. ft.	6.50	44,147.35
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		5,145.56
	(b) Deduction for material placed	3,958.12	lin. ft.	1.30	–5,145.56
179.	Furnishing and installing embedded electrical rigid metal conduit 4 inches in diameter	4,106.6	lin. ft.	7.50	30,799.50
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		6,090.76
	(b) Deduction for material placed	3,929.52	lin. ft.	1.55	–6,090.76
180.	Furnishing and installing No. 1 boxes	950	Box	12.00	11,400.00
181.	Furnishing and installing No. 2 boxes	935	Box	13.00	12,155.00
182.	Furnishing and installing type A lighting fixtures	40	Fixture	25.00	1,000.00
183.	Installing type J lighting fixtures	0	Fixture	45.00	0
184.	Installing lighting fixtures in roadway parapet of dam	0	Fixture	66.00	0
185.	Furnishing and installing power contact conductors for two 300-ton powerplant traveling cranes	13,582.45	lb.	1.00	13,582.45
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		1,739.95
	(b) Deduction for material placed	13,384.23	lb.	0.13	–1,739.95

APPENDIX B–Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
186.	Furnishing and installing power contact conductors for 75-ton machine shop traveling crane	1,891.79	lb.	$ 1.00	$ 1,891.79
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		215.05
	(b) Deduction for material placed	1,792.10	lb.	0.12	–215.05
187.	Furnishing and installing 440-volt, 100-ampere, power receptacle outlets complete with enclosures and matching plugs	10	Outlet	50.00	500.00
188.	Furnishing and installing 440-volt, 60-ampere power receptacle outlets complete with enclosures and matching plugs	45	Outlet	76.00	3,420.00
189.	Installing single thermometers, strain meters, stress meters, joint meters, and deformation meters	602.45	Meter	175.00	105,428.75
190.	Installing strainmeter groups	88.0	Group	350.00	30,800.00
191.	Excavation for roadway	86,949.9	cu. yd.	1.25	108,687.38
192.	Watering	1,585.0	M gal.	5.00	7,925.00
193.	Rolling embankments	234.0	Roller-hr.	25.00	5,850.00
194.	Overhaul of excavation for roadway	0	Mile-cu. yd.	0.40	0
195.	Special compaction of roadway embankment at bridge abutments	100%	Lump sum	1,000.00	1,000.00
196.	Excavation for roadway structures	199.1	cu. yd.	100.00	19,910.00
197.	Compacted backfill for roadway structures	120.8	cu. yd.	8.00	966.40
198.	Concrete in bridge sidewalks	15.8	cu. yd.	125.00	1,975.00
199.	Installing metal bridge railing	3,262.23	lb.	1.50	4,893.35
200.	Furnishing and laying 24-inch-diameter No. 16-gage corrugated-metal pipe	244.0	lin. ft.	10.00	2,440.00
201.	Crushed-rock base	1,499.3	Ton	8.00	11,994.40
202.	Cover-coat material	645.4	Ton	20.00	12,908.00
203.	Liquid asphalt MC-2	23.38	Ton	350.00	8,183.00
204.	Liquid asphalt MC-4	22.25	Ton	400.00	8,900.00
205.	Furnishing and erecting beam-type guardrail	1,650.0	lin. ft.	4.00	6,600.00
206.	Excavation for roadway	219,946.81	cu. yd.	3.00	659,840.43
207.	Excavation, opencut, for roadway tunnel approaches	13,852.8	cu. yd.	4.00	55,411.20
208.	Excavation in partial roadway tunnel section	0	cu. yd.	5.00	0
209.	Excavation in roadway tunnel and adits	184,257.85	cu. yd.	20.00	3,685,157.00
210.	Excavation for roadway in talus	0	cu. yd.	7.00	0
211.	Watering	1,451.198	M gal.	6.00	8,707.19

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
212.	Rolling embankments	53.5	Roller-hr.	$ 15.00	$ 802.50
213.	Excavation for roadway structures	1,577.96	cu. yd.	7.00	11,045.72
214.	Compacted backfill for roadway structures	754.89	cu. yd.	3.00	2,264.67
215.	Furnishing and placing permanent structural steel supports for roadway tunnel and adits	1,766,812.05	lb.	0.40	706,724.82
216.	Furnishing and placing permanent timber lagging and struts for roadway tunnel	672.37	Mb.m.	500.00	336,185.00
217.	Furnishing and installing roadway tunnel roof support bolts	89,700.00	lin. ft.	7.00	627,900.00
218.	Furnishing and installing chain-link fabric for roadway tunnel roof support	1,987.25	sq. yd.	6.00	11,923.50
219.	Furnishing and installing 12-inch-diameter No. 16-gage corrugated-metal pipe	0	lin. ft.	7.00	0
220.	Furnishing and laying 15-inch-diameter No. 16-gage corrugated-metal pipe	32.0	lin. ft.	8.00	256.00
221.	Furnishing and laying 18-inch-diameter No. 16-gage corrugated-metal pipe	618.0	lin. ft.	10.00	6,180.00
222.	Furnishing and laying 24-inch-diameter No. 16-gage corrugated-metal pipe	669.5	lin. ft.	13.00	8,703.50
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		1,796.11
	(b) Deduction for material placed		Lump sum		−1,796.11
223.	Furnishing and laying 30-inch-diameter No. 12-gage corrugated-metal pipe	0	lin. ft.	15.00	0
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		231.81
	(b) Deduction for material placed		Lump sum		−231.81
224.	Furnishing and laying 48-inch-diameter No. 12-gage corrugated-metal pipe	0	lin. ft.	28.00	0
225.	Concrete in service bridge abutments	248.10	cu. yd.	100.00	24,810.00
226.	Concrete in service bridge decks	145.67	cu. yd.	90.00	13,110.30
227.	Concrete in roadway tunnel portals	418.87	cu. yd.	75.00	31,415.25
228.	Concrete in drop inlets and manholes for roadway	23.34	cu. yd.	200.00	4,668.00
229.	Drilling holes for anchor bars and grouting bars in place	1,111.0	lin. ft.	5.00	5,555.00

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
230.	Erecting structural steel for service bridges	245,151.0	lb.	$ 0.11	$ 26,966.61
231.	Furnishing and installing miscellaneous metalwork for service bridges	1,618.51	lb.	1.00	1,618.51
232.	Furnishing and erecting beam-type guardrail	1,962.5	lin. ft.	4.00	7,850.00
233.	Crushed-rock base	16,611.94	Ton	5.00	83,059.70
234.	Cover-coat material	2,316.89	Ton	8.00	18,535.12
235.	Liquid asphalt MC-2	92.11	Ton	300.00	27,633.00
236.	Liquid asphalt MC-4	83.41	Ton	140.00	11,677.40
	Total—Original contract				$115,153,393.00
	Order for Changes No. 1				
(a)	Opencut excavation, common, for outlet channel of right diversion tunnel	47,431.0	cu. yd.	$ 1.05	$ 49,802.55
(b)	Opencut excavation, rock, for outlet channel of right diversion tunnel	42,883.0	cu. yd.	3.72	159,524.76
(c)	Drilling line holes for rock excavation for outlet channel of right diversion tunnel	3,431.0	lin. ft.	1.15	3,945.65
(d)	Furnishing and installing 2-inch electrical conduit in bridge abutment	49.25	lin. ft.	4.00	197.00
	Total—Order for Changes No. 1				$ 213,469.96
	Order for Changes No. 2				
(a)	Excavation, all classes, for enlarged gate chamber	719.26	cu. yd.	$ 36.06	$ 25,936.52
(b)	Excavation, all classes, for trashrack structure	1,106.7	cu. yd.	6.58	7,282.09
(c)	Furnishing and installing roof support bolts in gate chamber	4,060.0	lin. ft.	4.539	18,428.34
(d)	Mass concrete in base of left diversion tunnel trashrack structure	551.59	cu. yd.	17.22	9,498.38
(e)	Concrete in left diversion tunnel trashrack structure except mass concrete	1,215.12	cu. yd.	141.27	171,660.00
(f)	Backfill concrete in access tunnel between dam and gate chamber	0	cu. yd.	37.84	0
(g)	Concrete in gate chamber lining and stiffener beams	285.46	cu. yd.	150.38	42,927.47

APPENDIX B–Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
(h)	Furnishing and installing temporary steel struts in gate chamber	14,472.64	lb.	$ 0.31	$ 4,486.52
(i)	Furnishing and installing anchor bolts and plates, for temporary steel struct in gate chamber	646.8	lb.	0.31	200.51
(j)	Hookups for backfill grouting and grouting under steel conduit liners and outlet gates	121	Each	100.00	12,100.00
(k)	Drilling weep holes in gate chamber	133.0	lin. ft.	6.00	798.00
(l)	Installing steel conduit liners, gate, and hoists	100%	Lump sum	396,000.00	396,000.00
(m)	Installing hydraulic control system	100%	Lump sum	14,371.00	14,371.00
(n)	Furnishing and installing hoist anchor for outlet gates	1,140.83	lb.	0.31	353.66
(o)	Furnishing and installing electrical system for operation of outlet gates	100%	Lump sum	20,358.00	20,358.00
(p)	Furnishing and installing steel bulkhead access doorframe in tunnel plug, air piping, and bearing bar grating	100%	Lump sum	6,854.00	6,854.00
(q)	Furnishing and installing diversion tunnel trashrack	234,051.0	lb.	0.305	71,385.56
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		29,750.00
	(b) Deduction for material placed	212,500.0	lb.	0.14	–29,750.00
(r)	Furnishing and placing graded gravel on roof of trashrack structure	96.0	cu. yd.	5.00	480.00
(s)	Drilling weep holes in trashrack structure	113.3	lin. ft.	6.00	679.80
(t)	Furnishing, detailing, and placing reinforcement bars for trashrack structure	158,024.0	lb.	0.175	27,654.20
(u)	Furnishing, detailing, and placing reinforcement bars in gate chamber	46,940.24	lb.	0.1725	8,097.19
(v)	Adjustment for change in left diversion tunnel curve	100%	Lump sum	11,720.00	11,720.00
(w)	Furnishing and installing steel bulkhead and frame between access adit and gate chamber	1,470.0	lb.	0.49	720.30
(x)	Excavation, all classes, for 5- by 7-foot access adit to gate chamber	723.8	cu. yd.	38.00	27,504.40

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity: Amount	Quantity: Unit	Labor and materials by contractor: Unit cost	Labor and materials by contractor: Total cost
(y)	Furnishing and placing 3-inch weep pipe for trashrack structure	200.7	lin. ft.	$ 4.54	$ 911.18
(z)	Mass concrete beneath machine shop and service bay	34,371.25	cu. yd.	10.50	360,898.13
(aa)	Mass concrete around river outlet pipes and valve structure substructure	27,517.06	cu. yd.	13.75	378,359.57
(bb)	Concrete in structural slabs over outlet pipe expansion joint vaults	118.07	cu. yd.	55.00	6,493.85
(cc)	Concrete in cantilever-type walls	266.09	cu. yd.	120.00	31,930.80
(dd)	Lightweight concrete	294.89	cu. yd.	127.31	37,542.45
(ee)	Furnishing and installing steel reinforcing floor forms for lightweight concrete	16,417.57	sq. ft.	2.00	32,835.14
(ff)	Furnishing and installing deck drains and drain adapters	602.0	lb.	1.00	602.00
(gg)	Placing concrete in gravity training wall below elevation 3141.50	4,967.56	cu. yd.	22.95	114,005.50
(hh)	Furnishing and using Pozzolith 8 additive in tunnel concrete	142,783.77	bbl.	0.08	11,422.70
(ii)	Installing thermocouple wire	79,904.50	lin. ft.	0.27	21,574.21
(jj)	Constructing access shafts to counterweight wells in spillway intake structures	100%	Lump sum	1,926.00	1,926.00
(kk)	Pit-run conglomerate surfacing	1,451.9	cu. yd.	1.50	2,177.85
(ll)	Adjustment for enlarging powerplant access road tunnel section	100%	Lump sum	45,257.00	45,257.00
(mm)	Handling Government-furnished cement for spillway tunnel lining	47,520.81	bbl.	0.22	10,454.58
(nn)	Drilling holes from canyon wall and furnishing and installing anchor bars to anchor rock slabs	3,764.0	lin. ft.	18.00	67,752.00
(oo)	Drilling holes from excavated bench and furnishing and installing anchor bars to anchor rock slabs	0	lin. ft.	6.00	0
(pp)	Resloping previously excavated areas in keyways for dam	75,262.00	cu. yd.	7.00	526,834.00
(qq)	Constructing piezometer piping terminal boxes	2	Box	50.00	100.00
(rr)	Adjustment for change in 30-inch slide gate	100%	Lump sum	1,403.00	1,403.00
(ss)	Furnishing and installing piezometer piping, fittings and terminal boxes for outlet pipe No. 3	100%	Lump sum	3,304.60	3,304.60
	Total—Order for Changes No. 2				$ 2,535,280.50

APPENDIX B–Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	Order for Changes No. 3				
	Deduction in accordance with Item 6 of order for changes No. 3	100%	Lump sum	$ –95,800.00	$ –95,800.00
	Total–Order for Changes No. 3				$ –95,800.00
	Order for Changes No. 4				
(a)	Installing backfill shields on penstock supports	100%	Lump sum	$ 4,941.09	$ 4,941.09
(b)	Adjustment for formwork and shoring for portion of left abutment walkway	100%	Lump sum	562.50	562.50
(c)	Installing piezometer piping systems for outlet No. 2 of left diversion tunnel	100%	Lump sum	3,962.50	3,962.50
(d)	Furnishing and installing embedded metalwork for concrete counterweights	8,868.28	lb.	0.74	6,562.53
(e)	Performing additional excavation in left keyway	2,895.34	cu. yd.	8.44	24,436.67
(f)	Installing radial gate hoists	100%	Lump sum	73,714.00	73,714.00
(g)	Furnishing and placing 1-1/2- to 3-inch coarse aggregate	676.32	cu. yd.	7.00	4,734.24
(h)	Constructing 24-inch perforated reinforced concrete pipe drain	100%	Lump sum	2,332.92	2,332.92
(i)	Furnishing and placing 1-1/2- to 3/4-inch drain gravel	24.0	cu. yd.	7.00	168.00
	Total–Order for Changes No. 4				$ 121,414.45
	Order for Changes No. 5				
(a)	Furnishing and installing piezometer piping systems for penstocks	100%	Lump sum	$ 11,551.25	$ 11,551.25
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		
	(b) Deduction for material placed		Lump sum		
(b)	Removal of existing rock bolts downstream of right diversion tunnel outlet portal	262.0	Each	2.60	681.20
(c)	Drilling holes, and furnishing and installing 1-1/2-inch by 20-foot anchor bars	1,120.0	lin. ft.	18.00	20,160.00

APPENDIX B–Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
(d)	Additional excavation in the powerplant machine shop bay area	649.2	cu. yd.	$ 8.44	$ 5,479.25
(e)	Furnishing, installing and removing timber supports in the access adit of the left diversion tunnel plug	100%	Lump sum	398.00	398.00
(f)	Excavation of exploratory drifts in the right abutment	422.0	cu. yd.	38.00	16,036.00
(g)	Backfill concrete for exploratory drifts	408.7	cu. yd.	37.84	15,465.21
(h)	Drilling grout holes in exploratory drifts	65.0	lin. ft.	4.00	260.00
(i)	Furnishing and installing piping for grout systems in exploratory drifts	3,564.68	lb.	0.75	2,673.51
(j)	Furnishing and installing supports for grouting systems in exploratory drifts	228.57	lb.	0.74	169.14
(k)	Hookups to grout supply lines for grouting in exploratory drifts	9	Each	20.00	180.00
(l)	Pressure grouting for exploratory drifts	1,530.0	Sack	3.00	4,590.00
(m)	Constructing drainage system in exploratory drift at elevation 3070	100%	Lump sum	4,346.87	4,346.87
(n)	Furnishing and installing metal tubing and fittings for contact grouting	0	lb.	0.75	0
(o)	Hookups to contact grouting system	0	Each	30.00	0
(p)	Pressure grouting contact grouting system	0	Sack	3.00	0
(q)	Constructing grout outlet foundations	0	Lump sum	350.00	0
(r)	Additional excavation in left keyway between elevations 3200 and 3100	4,558.3	cu. yd.	8.44	38,472.05
(s)	Excavating additional 5 feet for penstock support footings	2,269.0	cu. yd.	6.00	13,614.00
(t)	Furnishing and laying 12-inch 16-gage corrugated-metal pipe	71.55	lin. ft.	7.24	518.02
(u)	Furnishing and laying 24-inch 12-gage corrugated-metal pipe	227.63	lin. ft.	16.50	3,755.90
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		1,688.02
	(b) Deduction for material placed		Lump sum		–1,688.02

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
(v)	Furnishing and laying 24-inch 14-gage corrugated-metal pipe	0	lin. ft.	$ 14.10	$ 0
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		0
	(b) Deduction for material placed	0	lin. ft.		0
(w)	Furnishing and laying 24-inch 16-gage corrugated-metal pipe	96.5	lin. ft.	15.35	1,481.28
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		108.09
	(b) Deduction for material placed		Lump sum		−108.09
(x)	Furnishing and laying 30-inch 12-gage corrugated-metal pipe	41.95	lin. ft.	15.21	638.06
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		126.90
	(b) Deduction for material placed		Lump sum		−126.90
(y)	Furnishing and laying 30-inch 14-gage corrugated-metal pipe	493.45	lin. ft.	16.45	8,117.25
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		104.91
	(b) Deduction for material placed		Lump sum		−104.91
(z)	Excavation for foundation deformation wells in penstock support	10.92	cu. yd.	15.79	172.43
(aa)	Backfill concrete for foundation deformation wells	10.92	cu. yd.	35.00	382.20
(bb)	Furnishing and installing piping for the deformation observation wells	100%	Lump sum	5,342.02	5,342.02
(cc)	Excavation along canyon rim for transformer circuits clearance	3,475.0	cu. yd.	5.00	17,375.00
(dd)	Furnishing, installing, and operating diversion tunnel gate lubrication system	100%	Lump sum	8,989.09	8,989.09
(ee)	Furnishing and installing right diversion tunnel gate hoisting equipment	12,741.0	lb.	0.74	9,428.34
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		2,890.00

APPENDIX B–Continued

Pay item	Property and pay item description	Quantity Amount	Quantity Unit	Labor and materials by contractor Unit cost	Labor and materials by contractor Total cost
	(b) Deduction for material placed		Lump sum		–2,890.00
(ff)	Operation of right diversion tunnel gate hoisting equipment	70.0	Hour	$ 79.85	$ 5,589.50
	Total–Order for Changes No. 5				$ 195,865.37
	Order for Changes No. 6				
(a)	Compacting backfill between dam and powerplant	230,366.40	cu. yd.	$ 0.23	$ 52,984.27
(b)	Furnishing and placing 3/16- to 3/4-inch gravel for backfill drainage	5,527.8	cu. yd.	4.00	22,111.20
(c)	Furnishing and placing 6-inch perforated concrete sewer pipe	1,101.0	lin. ft.	4.00	4,404.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		819.40
	(b) Deduction for material placed	819.4	lin. ft.	1.00	–819.40
(d)	Furnishing and placing 10-inch perforated concrete sewer pipe	451.0	lin. ft.	5.00	2,255.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		706.61
	(b) Deduction for material placed	353.305	lin. ft.	2.00	–706.61
(e)	Furnishing and placing 1-inch sponge rubber joint filler and joint compound	450.1	sq. ft.	4.80	2,160.48
(f)	Constructing additional joints in the elevator shafts of the dam	100%	Lump sum	3,015.00	3,015.00
(g)	Adjustment for change in painting of hollow-jet valves	100%	Lump sum	–4,731.25	–4,731.25
(h)	Furnishing and installing additional angle supports and expansion anchors under the pump sump grating in block 12	100%	Lump sum	302.00	302.00
(i)	Drilling holes and furnishing and installing 1-1/2-inch by 20-foot anchor bolts	1,620.00	lin. ft.	18.00	29,160.00
(j)	Furnishing and installing metal pipe and fittings for grouting of rock slab	1,491.67	lb.	0.95	1,417.09
(k)	Constructing 24-inch perforated sewer pipe drain at elevation 3113.50 in the service bay area	100%	Lump sum	3,100.00	3,100.00

APPENDIX B–Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
(l)	Furnishing and placing 3/4- to 1-1/2-inch drain gravel around 24-inch perforated sewer pipe drain	36.0	cu. yd.	$ 7.00	$ 252.00
(m)	Constructing concrete blocks at intersection of 4-inch metal pipe drains with rock slab drainage systems	6.45	cu. yd.	100.00	645.00
(n)	Adjustment for change in size of oil storage tanks	100%	Lump sum	2,599.50	2,599.50
(o)	Furnishing and installing pressure gage piping for river outlets	100%	Lump sum	250.00	250.00
	Total–Order for Changes No. 6				$ 119,924.29
	Order for Changes No. 7				
(a)	Compacting backfill in tailrace	41,452.23	cu. yd.	$ 0.23	$ 9,534.01
(b)	Furnishing and placing welded wire fabric	153,956.55	sq. ft.	0.20	30,791.31
(c)	Concrete for tailrace slab	4,794.20	cu. yd.	38.00	182,179.60
(d)	Constructing porous weep holes	1,351.00	Each	18.00	24,318.00
(e)	Furnishing and installing 1/4-inch bituminous joint filler	1,659.64	sq. ft.	1.54	2,555.85
	Total–Order for Changes No. 7				$ 249,378.77
	Order for Changes No. 8				
(a)	Adjustment for increase in number of stoplogs	100%	Lump sum	$ 10,125.00	$ 10,125.00
(b)	Constructing concrete buttresses	5,284.5	cu. yd.	26.30	138,982.35
(c)	Excavation for keys for watertight doors	31.1	cu. yd.	38.00	1,181.80
(d)	Concrete in keys for watertight doors	65.0	cu. yd.	130.00	8,450.00
(e)	Furnishing and placing reinforcement bars in concrete keys	4,832.86	lb.	0.20	966.57
(f)	Furnishing and installing two watertight doors with float and latch assemblies	100%	Lump sum	7,000.00	7,000.00
(g)	Furnishing and installing access adit drain and gate chamber drain	100%	Lump sum	1,000.00	1,000.00
(h)	Furnishing, installing, and painting 5- by 1/2-inch stiffener plates on d-line columns	100%	Lump sum	850.00	850.00

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
(i)	Extending cooling pipe headers	100%	Lump sum	$ 17,829.00	$ 17,829.00
(j)	Furnishing and installing polyvinyl chloride piping	100%	Lump sum	56.00	56.00
	Total—Order for Changes No. 8				$ 186,440.72
	Order for Changes No. 9				
(a)	Constructing temporary cofferdam downstream of left spillway outlet channel	100%	Lump sum	$ 12,000.00	$ 12,000.00
(b)	Excavation, rock, in opencut for left spillway erosion control slab	1,069.0	cu. yd.	10.00	10,690.00
(c)	Chipping and removing concrete from existing downstream portal of left spillway tunnel	100%	Lump sum	1,000.00	1,000.00
(d)	Furnishing and placing reinforcing bars for left spillway erosion control slab	18,590.0	lb.	0.20	3,718.00
(e)	Concrete in floor of left spillway erosion control slab	556.75	cu. yd.	25.00	13,918.75
(f)	Concrete in wall of left spillway erosion control slab	788.94	cu. yd.	80.00	63,115.20
(g)	Furnishing and installing 1-1/2-inch-diameter anchor bars	353.5	lin. ft.	18.00	6,363.00
(h)	Constructing two primary theodolite pier platforms with guardrails and furnishing and installing access ladders	100%	Lump sum	7,500.00	7,500.00
(i)	Preparing concrete surface and applying epoxy resin materials	100%	Lump sum	3,791.59	3,791.59
	Total—Order for Changes No. 9				$ 122,096.54
	Order for Changes No. 10				
(a)	Constructing a protective dike and cofferdam for the right spillway outlet channel	100%	Lump sum	$ 25,000.00	$ 25,000.00
(b)	Excavation, all classes, in opencut for right spillway erosion control slab	9,928.0	cu. yd.	4.50	44,676.00
(c)	Exploratory drilling	418.0	lin. ft.	1.00	418.00
(d)	Chipping and removing concrete from existing downstream portal structure of right spillway	100%	Lump sum	1,000.00	1,000.00

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
(e)	Furnishing and placing reinforcement bars for right spillway erosion control slab	18,591.0	lb.	$ 0.20	$ 3,718.20
(f)	Concrete in floor of right spillway erosion control slab	7,250.0	cu. yd.	25.00	181,250.00
(g)	Concrete in wall section of right spillway erosion control slab	2,532.67	cu. yd.	62.35	157,911.97
	Total–Order for Changes No. 10				$ 413,974.17
	Order for Changes No. 11				
(a)	Cleaning, painting, and storing stoplog lifting frame	100%	Lump sum	$ 860.00	$ 860.00
(b)	Furnishing and installing metal pipe and fittings for special rock-joint grouting	7,124.01	lb.	0.95	6,767.81
(c)	Furnishing 16-inch-diameter 8-gage spiral welded steel pipe	120.73	lin. ft.	6.00	724.38
(d)	Furnishing 10-inch-diameter, 10-gage spiral welded steel pipe	32.5	lin. ft.	4.00	130.00
(e)	Extending eave purlins EP-2 in the powerplant	100%	Lump sum	600.00	600.00
(f)	Performing additional excavation for the drainage gallery in block 4	100%	Lump sum	800.00	800.00
(g)	Adjustment for the additional surface preparation and welding for the powerplant structural-steel frame	100%	Lump sum	5,135.00	5,135.00
(h)	Cleaning, painting, and installing two 750-gallon water tanks in the dam	100%	Lump sum	1,268.00	1,268.00
(i)	Adjustment for change in requirements for hospital X-ray unit	100%	Lump sum	10,398.19	10,398.19
	Total–Order for Changes No. 11				$ 26,683.38
	Order for Changes No. 12				
(a)	Furnishing and using water reducing agent in concrete for dam and powerplant	4,046,871.79	lb.	$ 0.09286	$ 375,792.51

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity Amount	Quantity Unit	Labor and materials by contractor Unit cost	Labor and materials by contractor Total cost
(b)	Adjustment for change in painting requirements for outside surfaces of steel columns 2d through 34d	100%	Lump sum	$ 360.00	$ 360.00
	Total—Order for Changes No. 12				$ 376,152.51
	Order for Changes No. 13				
(a)	Drilling 3-1/2- and 4-inch-diameter holes for anchor bars	26,690.0	lin. ft.	$ 12.15	$ 324,283.50
(b)	Furnishing 2-inch-diameter anchor bars and fittings	356,342.3	lb.	0.183	65,210.64
(c)	Installing 2-inch-diameter anchor bars and fittings	26,657.0	lin. ft.	7.42	197,794.94
(d)	Loading 2-inch-diameter anchor bars	514	Each	92.72	47,658.08
	Total—Order for Changes No. 13				$ 634,947.16
	Order for Changes No. 14				
(a)	Performing structural-steel changes to provide for curved control board, dispatcher's room, and floor form supports	100%	Lump sum	$ 5,300.00	$ 5,300.00
(b)	Adjustment for the change in stop-log painting requirements	100%	Lump sum	–800.00	–800.00
(c)	Installing embedded parts of type L lighting fixtures	87	Each	63.50	5,524.50
(d)	Installing embedded parts of type K lighting fixtures	18	Each	42.50	765.00
(e)	Furnishing and erecting structural-steel for right abutment walkway	67,252.40	lb.	0.47	31,608.63
	*(a) Materials delivered to site (85 percent of paid invoices)		Lump sum		14,535.00
	(b) Deduction for materials placed	60,562.49	lb.	0.24	–14,535.00
(f)	Furnishing and installing cable trays with covers	200.2	lin. ft.	21.98	4,400.40
(g)	Concrete in floor deck of right abutment walkway	9.96	cu. yd.	80.00	796.80
(h)	Furnishing and installing stiffener plates for future jib crane	100%	Lump sum	347.00	347.00
(i)	Furnishing and installing additional 6-inch gallery drain-piping	8,036.95	lb.	1.10	8,840.64

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
(j)	Furnishing and attaching 5- by 3/8-inch plates, 2-1/2- by 1/4-inch bars, and 1/2-inch-diameter stud anchors	100%	Lump sum	$ 16,076.00	$ 16,076.00
(k)	Furnishing and installing 5- by 1-1/8-inch fill plates under hatch cover HC-1 and additional B-11 beams	1,785.0	lb.	0.96	1,713.60
(l)	Furnishing and installing 3-inch water supply line in elevator shaft in block 8	100%	Lump sum	3,913.00	3,913.00
(m)	Adjustment for change in concrete roof slabs	100%	Lump sum	4,225.10	4,225.10
(n)	Furnishing and installing salt velocity test piping	100%	Lump sum	2,537.00	2,537.00
	Total—Order for Changes No. 14				$ 85,247.67
	Order for Changes No. 15				
(a)	Installing bulkhead gate frames and anchor bolts	100%	Lump sum	$ 69,235.00	$ 69,235.00
(b)	Removing naval brass bolts and furnishing and installing monel metal bolts in the draft tube bulkhead gate seats	100%	Lump sum	21,022.63	21,022.63
(c)	Installing fixed-wheel gate frames and anchor bolts	100%	Lump sum	109,916.00	109,916.00
(d)	Lowering of concrete surface under transformer track rails	100%	Lump sum	3,405.37	3,405.37
(e)	Constructing concrete cap in drainage sump in block 11 of dam	100%	Lump sum	2,488.00	2,488.00
(f)	Increasing the radius of the pen-stock support saddles	100%	Lump sum	10,501.26	10,501.26
(g)	Furnishing support services for chemical grouting tests	100%	Lump sum	4,052.97	4,052.97
(h)	Adjustment for additional testing of 300-ton powerplant cranes	100%	Lump sum	2,691.94	2,691.94
(i)	Adjustment for change in power-plant louver requirements	100%	Lump sum	709.94	709.94
	Total—Order for Changes No. 15				$ 224,023.11
	Order for Changes No. 16				
(a)	Removing existing concrete from erosion floor slab and walls	287.3	cu. yd.	$ 97.34	$ 27,965.78
(b)	Line drilling and excavating rock	200.6	cu. yd.	20.00	4,012.00

APPENDIX B--Continued

Pay item	Property and pay item description	Quantity Amount	Quantity Unit	Labor and materials by contractor Unit cost	Labor and materials by contractor Total cost
(c)	Drilling and grouting 1-3/8-inch anchor bars	2,130.9	lin. ft.	$ 3.80	$ 8,097.42
(d)	Furnishing and installing reinforcement steel	170,826.0	lb.	0.186	31,773.64
(e)	Placing concrete in left spillway deflector bucket	4,457.4	cu. yd.	63.00	280,816.20
(f)	Furnishing and installing steel grating	100%	Lump sum	69.00	69.00
	Total—Order for Changes No. 16				$ 352,734.04
	Order for Changes No. 17				
(a)	Removing existing concrete from erosion floor slab and walls	174.2	cu. yd.	$ 97.34	$ 16,956.63
(b)	Line drilling and excavating rock	945.8	cu. yd.	20.00	18,916.00
(c)	Drilling and grouting 1-3/8-inch anchor bars	1,602.5	lin. ft.	3.80	6,089.50
(d)	Furnishing and installing reinforcement steel	161,940.0	lb.	0.186	30,120.84
(e)	Placing concrete in right spillway deflector bucket	4,223.89	cu. yd.	63.00	266,105.07
(f)	Furnishing and installing steel grating	100%	Lump sum	69.00	69.00
	Total—Order for Changes No. 17				$ 338,257.04
	Order for Changes No. 18				
(a)	Constructing regular concrete floor slabs at elevation 3257.24 in unit bays 1 and 2	100%	Lump sum	$ 19,304.00	$ 19,304.00
(b)	Excavating service shaft at the end of the service adit for the utility gallery	20.89	cu. yd.	38.00	793.82
(c)	Constructing platforms for secondary theodolite piers L-1, L-4, L-5, R-3, and R-4, and performing additional excavation for pier R-1	100%	Lump sum	32,780.22	32,780.22
(d)	Furnishing preparatory work and services for and access to foundation site tests in left abutment foundation tunnels	100%	Lump sum	13,447.51	13,447.51
(e)	Correcting fabrication deficiencies and performing additional testing on 75-ton machine shop crane	100%	Lump sum	8,064.17	8,064.17

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
(f)	Repairing shipping damage to 75-ton machine shop crane	100%	Lump sum	$ 105.35	$ 105.35
(g)	Adjustment for change in design of trashrack structure bases	100%	Lump sum	12,245.00	12,245.00
	Total—Order for Changes No. 18				$ 86,740.07
	Order for Changes No. 19				
(a)	Adjustment for change in installation requirements for hollow-jet valve controls	100%	Lump sum	$ 6,987.00	$ 6,987.00
(b)	Removing defective worm gear and installing the replacement	100%	Lump sum	409.05	409.05
(c)	Machining outside surface of collars on gutter inlet boxes	100%	Lump sum	494.00	494.00
(d)	Installing power receptacle outlet boxes	13	Each	87.50	1,137.50
(e)	Cleaning and painting interior surfaces of power receptacle outlet boxes	100%	Lump sum	175.00	175.00
(f)	Performing corrective work on fixed-wheel gate frames	100%	Lump sum	4,000.00	4,000.00
(g)	Concrete in top lift of blocks 1 through 26 of the dam	11,259.67	cu. yd.	45.00	506,685.15
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		7,735.00
	(b) Deduction for material placed		Lump sum		−7,735.00
	Total—Order for Changes No. 19				$ 519,887.70
	Order for Changes No. 20				
(a)	Adjustment for constructing extensions of a-, d-, and 35x-line walls of powerplant	100%	Lump sum	$ 82,940.00	$ 82,940.00
(b)	Constructing 9-line walls of powerplant	100%	Lump sum	16,873.00	16,873.00
(c)	Adjustment for changes in metalwork and waterstop due to fallout protection revisions	100%	Lump sum	3,503.00	3,503.00
(d)	Adjustment for additional forming costs for unit bay 1 superstructure walls	100%	Lump sum	5,350.00	5,350.00
(e)	Adjustment for changes in drain and piping materials due to fallout protection revisions	100%	Lump sum	3,828.00	3,828.00

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
(f)	Revising reinforcing floor forms for powerplant control area	100%	Lump sum	$ 2,750.00	$ 2,750.00
(g)	Furnishing and placing 5-ply membrane waterproofing on unit bay roofs	40,868.0	sq. ft.	1.00	40,868.00
(h)	Furnishing and placing a vapor barrier on unit bay roofs	42,477.0	sq. ft.	0.255	10,831.64
(i)	Furnishing and installing monel metal flashing on unit bay roofs	100%	Lump sum	25,545.00	25,545.00
(j)	Constructing lightweight concrete cover slabs on roofs of unit bays 1, 2, and 4 through 8	3,961.24	sq. yd.	16.22	64,251.31
(k)	Constructing regular concrete cover slabs on roof of unit bay 3	100%	Lump sum	12,825.00	12,825.00
(l)	Furnishing and installing reinforcing steel for roof cover slabs	44,383.0	lb.	0.20	8,876.60
(m)	Constructing concrete pedestals and raised floor slab for toilet room	100%	Lump sum	1,302.00	1,302.00
	Total—Order for Changes No. 20				$ 279,743.55
	Order for Changes No. 21				
(a)	Removing and disposing of rock from west canyon wall	100%	Lump sum	$ 18,615.68	$ 18,615.68
(b)	Furnishing and applying two additional body coats to vertical bands of spillway radial gates	100%	Lump sum	196.00	196.00
(c)	For painting of river outlets bulkhead gate	100%	Lump sum	228.00	228.00
(d)	Welding shims to ends of stoplog guide sections and coating joints	100%	Lump sum	576.00	576.00
(e)	Removing, reaming and reattaching splice plates for stoplog guides	110	Each	8.75	962.50
(f)	Constructing reinjectable grout systems	24,601.78	lb.	1.20	29,522.14
(g)	Excavation for left abutment service roads	12,684.4	cu. yd.	6.50	82,448.60
(h)	Concrete in slab adjacent to block 1	244.4	cu. yd.	38.00	9,287.20
(i)	Grouting penstock bulkhead thrust rings	100%	Lump sum	4,137.81	4,137.81
(j)	Furnishing and installing safety baskets on primary theodolite pier ladders	100%	Lump sum	5,806.03	5,806.03

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
(k)	For testing 165-ton gantry crane	100%	Lump sum	$ 682.00	$ 682.00
(l)	For constructing lower portal of control cable tunnel and for constructing cable duct	100%	Lump sum	50,541.36	50,541.36
	Total—Order for Changes No. 21				$ 203,003.32
	Order for Changes No. 22				
(a)	Adjustment for expediting construction of left diversion tunnel outlet works	100%	Lump sum	$ 27,567.00	$ 27,567.00
(b)	Adjustment for expediting construction of penstocks and river outlets trashrack structures	100%	Lump sum	80,783.00	80,783.00
(c)	Adjustment for early installation and testing of the river outlets bulkhead gate	100%	Lump sum	1,347.00	1,347.00
(d)	Adjustment for change in curved section of right spillway deflector bucket	100%	Lump sum	1,235.00	1,235.00
(e)	Furnishing countersunk ribbed bolts and countersinking boltholes	100%	Lump sum	722.00	722.00
(f)	Fabricating 4- by 2-3/4-inch stainless steel nuts and repairing threads on eccentric pins	100%	Lump sum	832.00	832.00
(g)	Additional rock excavation for corrugated-metal-pipe drains, waterline and electrical conduits	73.2	cu. yd.	20.00	1,464.00
(h)	Cleaning and painting interior of 30-inch-diameter floatwell liner	100%	Lump sum	596.00	596.00
(i)	Performing corrective work on spiral stairways for dam	100%	Lump sum	4,731.00	4,731.00
(j)	Furnishing and delivering drill and grout jumbo for left spillway elbow section	100%	Lump sum	10,000.00	10,000.00
(k)	Furnishing, detailing, fabricating, and delivering reinforcement bars for left spillway elbow section	646,196.0	lb.	0.11	71,081.56
(l)	Furnishing and delivering piping for drainage, grouting, and cooling, and metal seals for access adit and left tunnel plug and elbow section	100%	Lump sum	8,962.00	8,962.00
(m)	Furnishing concrete aggregates for access adit and completion of left tunnel plug and elbow section	25,000.00	cu. yd.	2.00	50,000.00

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
(n)	Furnishing concrete form and needle beams for left spillway elbow section	100%	Lump sum	$ 13,000.00	$ 13,000.00
(o)	Processing additional 3- to 6-inch coarse aggregate	100%	Lump sum	164,133.00	164,133.00
(p)	Adjustment for changes in cooling requirements for concrete in dam	100%	Lump sum	27,730.00	27,730.00
(q)	Adjustment for change in painting requirements for gantry crane rails	100%	Lump sum	1,157.00	1,157.00
	Total—Order for Changes No. 22				$ 465,340.56
	Order for Changes No. 23				
(a)	Repairing the right spillway tunnel invert between stations 26+11.718 and 35+22.000	100%	Lump sum	$ 113,397.00	$ 113,397.00
(b)	Waterloading transverse joints 1-2, 2-3, 24-25, and 25-26 in the dam	100%	Lump sum	19,301.00	19,301.00
(c)	Furnishing necessary services and labor for installation of rock deformation gages in the abutment tunnels of the dam	100%	Lump sum	1,921.00	1,921.00
(d)	Drilling holes for expansion anchors in the suspension chamber and reading stations for the plumbline wells in the dam	100%	Lump sum	2,920.26	2,920.26
	Total—Order for Changes No. 23				$ 137,539.26
	Order for Changes No. 24				
(a)	Providing facilities for electric power for elevators in the dam	100%	Lump sum	$ 150.00	$ 150.00
(b)	Furnishing electrical power for installing the elevators in the dam	5	Month	30.00	150.00
(c)	Performing tests on twelve 2-inch anchor bars in canyon walls	100%	Lump sum	1,735.00	1,735.00
(d)	Repairing shipping damage on miscellaneous metalwork for gate hoist structures	100%	Lump sum	759.30	759.30
(e)	Removing and disposing of loose rock on canyon walls above powerplant structure	100%	Lump sum	4,653.61	4,653.61
(f)	Installing drain water collection system in the gate chamber	100%	Lump sum	5,925.00	5,925.00

Pay item	Property and pay item description	Quantity: Amount	Quantity: Unit	Labor and materials by contractor: Unit cost	Labor and materials by contractor: Total cost
(g)	Furnishing and installing 1-1/2-inch anchor bars	1,225.0	lin. ft.	$ 20.00	$ 24,500.00
(h)	Modifying the fixed-wheel gate hoists and controls	100%	Lump sum	2,039.62	2,039.62
(i)	Adjustment for additional forming costs on fixed-wheel gate hoist structures	100%	Lump sum	9,620.00	9,620.00
	Total—Order for Changes No. 24				$ 49,532.53
	Order for Changes No. 25				
(a)	Replacing the existing hot water systems and making other necessary water piping and equipment changes	100%	Lump sum	$ 20,708.00	$ 20,708.00
(b)	Removal and disposal of loose rock from the right canyon wall	100%	Lump sum	1,825.00	1,825.00
(c)	Drilling test hole, replacing packing in expansion joint packing glands in B-vault of penstock No. 1 and performing additional tightening of expansion joint bolts	100%	Lump sum	9,685.31	9,685.31
(d)	As an adjustment for the relocation of footings and extending structural steel for concrete transfer trestle bent No. 4	100%	Lump sum	6,600.00	6,600.00
(e)	Removing lightweight material from sand and coarse aggregate for concrete structures	100%	Lump sum	79,516.00	79,516.00
(f)	As an adjustment for installing Government-furnished piping 4 inches and smaller in diameter	100%	Lump sum	31,453.00	31,453.00
(g)	Replacing the packing in the packing glands on both sides of the expansion joint in the A-vault of penstock No. 3	100%	Lump sum	3,795.00	3,795.00
	Total—Order for Changes No. 25				$ 153,582.31
	Total—Orders for Changes				$ 7,992,759.18
	Total—Original Contract				$123,146,152.08

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	Escalation				
A.	Adjustment for changes in the cost of labor in accordance with paragraph 19 of specifications No. DC-4825, as itemized below				
	Payroll		Period		
	Merritt-Chapman & Scott Corp. Payrolls				
	604 through 885		6/1/59 through 10/25/64		$ 10,111,358.10
	Ironworkers		6/25/62 through 10/25/64		89,251.99
	Painters		11/6/61 through 8/23/64		21,580.45
	Sheet Metal Workers		4/16/62 through 5/13/62		56.80
					$ 10,222,247.34
	MCS Pipe Trades Payrolls (National Mechanical Contractors, Inc.)				
	739 through 876		1/1/62 through 8/23/64		$ 208,349.00
	Plumbers and Pipefitters		1/1/62 through 8/23/64		66,620.87
					$ 274,969.87
	Duffy Electric Payrolls (Morgan Electric Payrolls) (Construction Subcontractor)				
	63 through 47		6/1/59 through 9/20/64		$ 455,132.95
	Electricians		6/1/59 through 9/20/64		16,905.90
					$ 472,038.85
	Yuba Erectors Payrolls (Judson Pacific-Murphy) (Construction Subcontractor)				
			6/3/59 through 6/14/64		$ 194,168.91
	Ironworkers		6/25/62 through 6/14/64		18,774.43
					$ 212,943.34
	Lynch-McClintock Glen Canyon Drillers Payrolls (Construction Subcontractor)				
			6/1/59 through 6/6/64		$ 54,188.10
	Chicago Bridge and Iron Co. Payrolls (Construction Subcontractor)				
			6/1/59 through 2/17/63		$ 33,470.68

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	Otto Buehner and Co. Payrolls (Construction Subcontractor)				
			6/1/59 through 7/10/62		$ 159.08
	Ironworkers		6/12/62 through 7/10/62		27.44
					$ 186.52
	Stiles and Allen Payrolls (Construction Subcontractor)				
			6/1/59 through 9/26/62		$ 1,484.22
	Sheet Metal Workers		8/2/62 through 9/26/62		97.93
					$ 1,582.15
	Wiscombe Painting Co. Payrolls (Construction Subcontractor)				
			6/1/59 through 6/3/62		$ 3,484.16
	Painters		3/19/62 through 6/3/62		1,068.24
					$ 4,552.40
	National Mechanical Contractor's, Inc. Payrolls (Construction Subcontractor)				
	604 through 738		6/1/59 through 12/31/61		$ 199,198.09
	Plumbers and Pipefitters		1/4/60 through 12/31/61		74,984.09
					$ 274,182.18
	Gustafson, Inc. Payrolls (Construction Subcontractor)				
			6/1/59 through 1/31/61		$ 839.16
	Painters		12/5/60 through 1/31/61		199.55
					$ 1,038.71
	Site Six Sand and Rock, Inc. (Construction Subcontractor)		6/28/64 through 7/15/64		$ 689.06
	Union Fence Co., Inc. (Construction Subcontractor)		8/27/64 through 9/20/64		$ 778.65
	Nelson Bros. Construction Co. (Construction Subcontractor)		8/30/64 through 9/12/64		$ 697.68
	Total—Escalation				$ 11,553,565.53

APPENDIX B—Continued

Pay item	Property and pay item description	Quantity: Amount	Quantity: Unit	Labor and materials by contractor: Unit cost	Labor and materials by contractor: Total cost
	Recapitulation				
	Total—Original Contract				$115,153,393.00
	Total—Orders for Changes				7,992,759.18
	Total—Escalation of Wages				11,553,565.53
	Total—Net Earnings				$134,699,717.61

APPENDIX C

Construction costs by pay items—Completion of Glen Canyon Powerplant, Switchyard, Dam, and appurtenant works—Specifications No. DC-5750, contract No. 14-06-D-4429:

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
1.	Removing temporary structures at right side vista point	100%	Lump sum	$ 2,900.00	$ 2,900.00
2.	Excavation, common, for grading	40,212.63	cu. yd.	3.00	120,637.89
3.	Excavation, rock, for grading	60,558.24	cu. yd.	6.50	393,628.56
4.	Excavation, common, for structures	1,955.43	cu. yd.	5.00	9,777.15
5.	Excavation, rock, for structures	2,306.11	cu. yd.	10.00	23,061.10
6.	Excavation, common, for water supply lines and duct banks	448.92	cu. yd.	10.00	4,489.20
7.	Excavation, rock, for water supply lines and duct banks	153.86	cu. yd.	15.00	2,307.90
8.	Excavation from borrow	2,112.30	cu. yd.	2.50	5,280.75
9.	Drilling 20-inch-diameter vertical access hole	303.04	lin. ft.	30.00	9,091.20
10.	Backfill about structures	4,613.66	cu. yd.	1.50	6,920.49
11.	Compacting backfill about structures	3,778.90	cu. yd.	3.80	14,359.82
12.	Compacting embankments	69,280.09	cu. yd.	0.75	51,960.07
13.	Furnishing and placing gravel for gravelfills	197.13	cu. yd.	8.00	1,577.04
14.	Sand backfill and timber protection for buried insulated electrical cables	2,272.7	lin. ft.	2.00	4,545.40
15.	6-inch-thick gravel surfacing	51,506.1	sq. yd.	1.00	51,506.10
16.	Furnishing and applying soil sterilant	51,506.1	sq. yd.	0.15	7,725.92
17.	Drilling holes in rock and concrete for anchor bars and grouting bars in place	19,292.5	lin. ft.	2.00	38,585.00
18.	Drilling holes for ground rods and grouting ground rods in place	3,726.00	lin. ft.	2.00	7,452.00
19.	Removing and reinstalling existing corrugated-metal-pipe culvert	100%	Lump sum	2,450.00	2,450.00
20.	Watering	20.50	M gal.	20.00	410.00
21.	Crushed-rock base	534.99	Ton	12.00	6,419.88
22.	Liquid asphalt MC-2	3.79	Ton	74.00	280.46
23.	Liquid asphalt MC-4	3.70	Ton	74.00	273.80
24.	Cover coat material	112.38	Ton	15.50	1,741.89
	Furnishing and placing reinforcement bars of the following sizes:				
25.	3/8 inch	106,729.0	lb.	0.15	16,009.35
26.	1/2 and 5/8 inch	100,937.0	lb.	0.15	15,140.55
27.	3/4 and 7/8 inch	171,712.0	lb.	0.15	25,756.80
28.	1 inch and larger	837,303.0	lb.	0.14	117,222.42
29.	Reinforcement fabric	16,229.0	lb.	0.20	3,245.80

APPENDIX C—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
30.	Second-stage concrete in power-plant structure	31,124.042	cu. yd.	$ 62.00	$1,929,690.60
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		113,938.77
	(b) Deduction for material placed		Lump sum		−113,938.77
31.	Concrete in blockouts	66.71	cu. yd.	200.00	13,342.00
32.	Miscellaneous concrete in power-plant structure	122.888	cu. yd.	125.00	15,361.00
33.	Concrete in CBR equipment room	52.429	cu. yd.	90.00	4,718.61
34.	Concrete in control cable tunnel	922.30	cu. yd.	83.00	76,550.90
35.	Concrete in miscellaneous structures	1,639.39	cu. yd.	65.00	106,560.35
36.	Grouting waterline and pipes for electrical control cables in 20-inch-diameter vertical access hole	297.5	lin. ft.	2.50	743.75
37.	Pneumatically applied mortar in control cable tunnel	1,141.30	sq. yd.	10.00	11,413.00
38.	Furnishing and installing rubber joint strips	350.0	lin. ft.	2.00	700.00
39.	Furnishing and installing rubber floor strips	418.08	lin. ft.	3.00	1,254.24
40.	Furnishing and applying concrete floor hardener	6,754.46	sq. yd.	0.40	2,701.78
41.	Constructing 8-inch hollow concrete masonry unit partition walls	2,226.33	sq. ft.	1.00	2,226.33
42.	Constructing 12-inch hollow concrete masonry unit partition walls	1,964.25	sq. ft.	1.25	2,455.31
43.	Constructing 8-inch solid concrete masonry unit partition wall	157.44	sq. ft.	2.00	314.88
44.	Furnishing and placing roof insulation	2,012.04	sq. ft.	1.50	3,018.06
45.	Furnishing and placing 4-ply, asphalt saturated felt, builtup roofing	2,625.77	sq. ft.	1.50	3,938.66
46.	4-inch concrete surfacing	4,440.78	sq. yd.	9.00	39,967.02
47.	Bonded concrete finish	100%	Lump sum	3,850.00	3,850.00
48.	3-1/2-inch concrete underbed	5,600.638	sq. yd.	7.50	42,004.78
49.	1-1/2-inch concrete underbed	110.49	sq. yd.	25.00	2,762.25
50.	Furnishing and installing vinyl-plastic tile	15,610.5	sq. ft.	0.80	12,488.40
51.	Furnishing and installing rubber cove base	1,549.0	lin. ft.	0.75	1,161.75
52.	Furnishing and installing rubber floor tile	1,597.55	sq. ft.	1.50	2,396.32

Pay item	Property and pay item description	Quantity Amount	Quantity Unit	Labor and materials by contractor Unit cost	Labor and materials by contractor Total cost
53.	Furnishing and installing suspended acoustical unit ceilings	14,691.38	sq. ft.	$ 1.25	$ 18,364.22
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		1,459.62
	(b) Deduction for material placed		Lump sum		−1,459.62
54.	Furnishing and installing suspended ventilating ceilings	1,164.62	sq. ft.	2.75	3,202.70
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		4,365.60
	(b) Deduction for material placed		Lump sum		−4,365.60
55.	Furnishing and installing insulated metal ceilings	680.94	sq. ft.	2.50	1,702.35
56.	Steel stud and gypsum wallboard partitions and metal deck ceilings	0	Lump sum	7,900.00	0
57.	Furnishing and installing raised metal floor system	2,500.46	sq. ft.	6.00	15,002.76
58.	Painting concrete and concrete masonry walls of rooms 406 and 406A of powerplant structure	0	Lump sum	150.00	0
59.	Painting generators	45,868.40	sq. ft.	0.17	7,797.63
60.	Painting concrete walls of turbine pit No. 1	100%	Lump sum	150.00	150.00
61.	Furnishing and installing steel swinging doors	100%	Lump sum	35,000.00	35,000.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		18,397.40
	(b) Deductions for material placed		Lump sum		−18,397.40
62.	Furnishing and installing metal rolling door	100%	Lump sum	900.00	900.00
63.	Furnishing and installing tin-clad fire doors	100%	Lump sum	1,840.00	1,840.00
64.	Furnishing and installing metal and glazed metal partitions and insulated metal fillers	100%	Lump sum	44,000.00	44,000.00
65.	Furnishing and installing metal toilet stall and urinal partitions	100%	Lump sum	3,150.00	3,150.00
66.	Furnishing and installing generator room balcony partitions, doors, and ceiling	100%	Lump sum	9,000.00	9,000.00
67.	Furnishing and installing insulated metal panel sidewall for control area	100%	Lump sum	3,750.00	3,750.00

APPENDIX C–Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
68.	Furnishing and installing cable trays and cable tray hangers and supports	291,864.49	lb.	$ 1.32	$ 385,261.12
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		40,209.68
	(b) Deduction for material placed		Lump sum		–40,209.68
69.	Furnishing and installing aluminum covers	20,739.0	lb.	1.00	20,739.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		11,105.73
	(b) Deduction for material placed		Lump sum		–11,105.73
70.	Furnishing and installing aluminum handrailings	9,606.81	lb.	6.00	57,640.86
71.	Furnishing and installing miscellaneous metalwork	308,931.36	lb.	0.70	216,251.95
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		60,822.96
	(b) Deduction for material placed		Lump sum		–60,822.96
72.	Installing generator anchor bolts	100%	Lump sum	16,800.00	16,800.00
73.	Furnishing and installing plumbing fixtures and piping for plumbing system	100%	Lump sum	42,617.00	42,617.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		2,657.07
	(b) Deduction for material placed		Lump sum		–2,657.07
73A.	Furnishing and installing plumbing fixtures for CBR protection	0	Lump sum	5,000.00	0
73B.	Furnishing and installing piping for plumbing fixtures for CBR protection	0	lb.	1.25	0
74.	Furnishing and installing frames and covers for conduit pull boxes	1,224.0	lb.	1.10	1,346.40
75.	Furnishing and installing safety barriers and cable recess covers	2,530.0	lb.	1.20	3,036.00
76.	Furnishing and installing heating, ventilating, and air-conditioning systems in the powerplant	100%	Lump sum	263,500.00	263,500.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		39,118.77

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	(b) Deduction for material placed		Lump sum		$ –39,118.77
77.	Furnishing and installing heating and ventilating systems in the dam	100%	Lump sum	$ 9,561.00	9,561.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		5,271.18
	(b) Deduction for material placed		Lump sum		–5,271.18
78.	Furnishing and installing sewage treatment equipment	100%	Lump sum	12,017.00	12,017.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		6,886.06
	(b) Deduction for material placed		Lump sum		–6,886.06
79.	Installing and testing eight 155,500-horsepower hydraulic turbines	100%	Lump sum	1,002,160.00	1,002,160.00
80.	Installing and testing eight 441,700-foot-pound governors for hydraulic turbines	100%	Lump sum	33,102.00	33,102.00
81.	Installing and testing four motor-driven sump pumping units for powerplant	100%	Lump sum	2,907.00	2,907.00
82.	Installing and testing two motor-driven sump pumping units for dam	100%	Lump sum	2,431.00	2,431.00
83.	Installing and testing eight motor-driven generator-cooling water pumping units	100%	Lump sum	2,412.00	2,412.00
84.	Installing and testing four motor-driven booster pumping units for water supply to Page filtration plant	100%	Lump sum	1,336.00	1,336.00
85.	Removing and reinstalling one surge suppressor	100%	Lump sum	220.00	220.00
86.	Installing and testing two motor-driven booster pumping units for water supply to visitor center storage tank	100%	Lump sum	647.00	647.00
87.	Installing and testing two motor-driven oil transfer pumping units	100%	Lump sum	359.00	359.00
88.	Installing and testing three duplex-type sewage disposal units	100%	Lump sum	1,652.00	1,652.00
89.	Installing and testing four motor-driven air compressors	100%	Lump sum	2,286.00	2,286.00
90.	Installing four aftercoolers	100%	Lump sum	474.00	474.00

APPENDIX C–Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
91.	Installing 10 air receivers	100%	Lump sum	$ 1,733.00	$ 1,733.00
92.	Installing and testing one portable oil purifier and one filter paper drying oven	100%	Lump sum	297.00	297.00
93.	Installing two water-level gages	100%	Lump sum	482.00	482.00
94.	Installing and testing chlorinating equipment	100%	Lump sum	403.00	403.00
95.	Installing miscellaneous instruments	218	Each	27.00	5,886.00
96.	Furnishing and installing gage boards for miscellaneous instruments	6,390.0	sq. in.	0.05	319.50
97.	Furnishing and installing nameplates	1,620	Each	2.55	4,131.00
98.	Furnishing and installing one steel workbench	100%	Lump sum	500.00	500.00
99.	Installing and testing fixed carbon dioxide fire extinguishing systems	100%	Lump sum	4,046.00	4,046.00
100.	Installing 100-pound wheeled-portable carbon dioxide fire extinguishers	2	Each	52.00	104.00
101.	Installing 15-pound hand-portable carbon dioxide fire extinguishers	106	Each	15.00	1,590.00
102.	Furnishing and installing surface-mounted firehose cabinets and equipment	29	Each	200.00	5,800.00
103.	Furnishing and installing recess-type firehose cabinets and equipment	3	Each	200.00	600.00
104.	Furnishing and installing two fire-hose carts and equipment	100%	Lump sum	1,272.00	1,272.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		776.90
	(b) Deduction for material placed		Lump sum		–776.90
105.	Furnishing and installing miscellaneous fire-protection equipment	100%	Lump sum	1,500.00	1,500.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		458.86
	(b) Deduction for material placed		Lump sum		–458.86
106.	Furnishing and installing cast iron soil pipe, fittings, bell traps, floor drains, and cleanouts	34,595.75	lb.	0.333	11,520.38

APPENDIX C–Continued

Pay item	Property and pay item description	Quantity Amount	Quantity Unit	Labor and materials by contractor Unit cost	Labor and materials by contractor Total cost
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		$ 2,881.00
	(b) Deduction for material placed		Lump sum		–2,881.00
107.	Furnishing and installing cast iron bell and spigot and flanged pipe and fittings	2,437.58	lb.	$ 0.32	780.03
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		151.56
	(b) Deduction for material placed		Lump sum		–151.56
108.	Furnishing and installing galvanized wrought-iron pipe, fittings, and valves 2 inches and smaller in nominal diameter	18,935.21	lb.	2.27	42,982.93
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		6,545.35
	(b) Deduction for material placed		Lump sum		–6,545.35
109.	Furnishing and installing galvanized wrought-iron pipe fittings, and valves 2-1/2 inches and larger in nominal diameter	481,520.81	lb.	0.88	423,738.31
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		192,494.66
	(b) Deduction for material placed		Lump sum		–192,494.66
110.	Furnishing and installing black wrought-iron pipe, fittings, and valves 10 inches and larger in nominal diameter	94,320.59	lb.	0.92	86,774.94
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		30,294.55
	(b) Deduction for material placed		Lump sum		–30,294.55
111.	Furnishing and installing black steel pipe, fittings, and valves 2 inches and smaller in nominal diameter	17,120.18	lb.	1.90	32,528.34
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		5,415.73
	(b) Deduction for material placed		Lump sum		–5,415.73

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
112.	Furnishing and installing black steel pipe, fittings, and valves 2-1/2 inches and larger in nominal diameter	109,870.14	lb.	$ 0.80	$ 87,896.11
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		21,841.18
	(b) Deduction for material placed		Lump sum		–21,841.18
113.	Furnishing and installing galvanized steel pipe and fittings	2,270.0	lb.	2.64	5,992.80
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		216.23
	(b) Deduction for material placed		Lump sum		–216.23
114.	Furnishing and installing copper tubing, fittings, and valves	1,770.86	lb.	16.00	28,333.76
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		190.09
	(b) Deduction for material placed		Lump sum		–190.09
115.	Furnishing and installing stainless steel pipe, fittings, and valves 3/4 inch in nominal diameter	1,462.02	lb.	4.25	6,213.58
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		2,759.37
	(b) Deduction for material placed		Lump sum		–2,759.37
116.	Installing special valves	29,003.35	lb.	0.23	6,670.77
117.	Furnishing and installing pipe hangers and supports	57,821.68	lb.	1.73	100,031.51
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		9,648.21
	(b) Deduction for material placed		Lump sum		–9,648.21
118.	Furnishing and installing pipe insulation	100%	Lump sum	348.00	348.00
119.	Installing strainers for piping systems	37,278.62	lb.	0.08	2,982.29
120.	Furnishing and installing flexible metal hose connections for air compressors	100%	Lump sum	1,560.00	1,560.00
121.	Furnishing and installing flexible metal and metal-lined oil hoses	100%	Lump sum	465.00	465.00
122.	Installing flexible metal hose connections for transformers	48	Each	12.50	600.00

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
123.	Furnishing and installing one 3-ton electric hoist	100%	Lump sum	$ 2,590.00	$ 2,590.00
124.	Furnishing and installing one 3-ton jib crane	100%	Lump sum	3,131.00	3,131.00
125.	Installing new machine tools	16,058.0	lb.	0.07	1,124.06
126.	Installing used machine tools, except 16-foot boring mill	101,085.0	lb.	0.045	4,548.82
127.	Installing 16-foot boring mill	100%	Lump sum	13,178.00	13,178.00
128.	Furnishing and installing oil storage tanks	100%	Lump sum	8,770.00	8,770.00
129.	Installing penstock makeup pieces	100%	Lump sum	49,716.00	49,716.00
130.	Removing penstock bulkheads and drains	100%	Lump sum	18,930.00	18,930.00
131.	Furnishing and installing 12-inch water supply line and piping	100%	Lump sum	86,268.00	86,268.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		28,260.46
	(b) Deduction for material placed		Lump sum		–28,260.46
132.	Furnishing and erecting takeoff brackets and lightning arrester supports on powerplant	90,336.00	lb.	0.3158	28,528.11
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		13,821.41
	(b) Deduction for material placed		Lump sum		–13,821.41
133.	Furnishing and erecting transformer circuit towers No. C1-T1, C2-T1, and C3-T1	1,209,040.0	lb.	0.3436	415,426.14
134.	Furnishing and erecting steel structures	1,071,814.40	lb.	0.3066	328,618.30
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		182,423.53
	(b) Deduction for material placed		Lump sum		–182,423.53
135.	Furnishing and installing tower leg grounds with ground rod	0	Each	18.00	0
136.	Furnishing and installing tower leg grounds without ground rod	53	Each	9.50	503.50
137.	Airway obstruction painting	100%	Lump sum	5,642.00	5,642.00
138.	Furnishing and installing single-switch-operating platforms	26	Each	156.00	4,056.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		1,989.00
	(b) Deduction for material placed		Lump sum		–1,989.00

APPENDIX C–Continued

Pay item	Property and pay item description	Quantity: Amount	Quantity: Unit	Labor and materials by contractor: Unit cost	Labor and materials by contractor: Total cost
139.	Furnishing and installing double-switch-operating platforms	4	Each	$ 331.00	$ 1,324.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		612.00
	(b) Deduction for material placed		Lump sum		–612.00
140.	Furnishing and installing equipment identification signs	62	Each	28.20	1,748.40
141.	Furnishing and installing phase-identification signs	78	Each	17.25	1,345.50
142.	Furnishing and installing warning and safety signs	6	Each	21.00	126.00
143.	Furnishing and erecting chain-link fence	2,798.50	lin. ft.	2.90	8,115.65
144.	Filtering insulating oil	126,568.0	gal.	0.05	6,328.40
145.	Furnishing and installing bare stranded copper cable for grounding system	41,021.29	lb.	1.04	42,662.14
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		14,519.63
	(b) Deduction for material placed		Lump sum		–14,519.63
	Furnishing and installing the following sizes of embedded and/or exposed electrical rigid metal conduit:				
146.	1/2 inch in diameter	9,387.44	lin. ft.	0.64	6,007.96
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		351.65
	(b) Deduction for material placed		Lump sum		–351.65
147.	3/4 inch in diameter	43,358.97	lin. ft.	0.68	29,484.10
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		5,038.48
	(b) Deduction for material placed		Lump sum		–5,038.48
148.	1 inch in diameter	16,495.54	lin. ft.	0.90	14,845.99
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		4,671.77
	(b) Deduction for material placed		Lump sum		–4,671.77
149.	1-1/2 inches in diameter	8,760.21	lin. ft.	1.27	11,125.47
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		4,327.36

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	Furnishing and installing the following sizes of embedded and/or exposed electrical rigid metal conduit:			$	
	(b) Deduction for material placed		Lump sum		$ −4,327.36
150.	2 inches in diameter	2,095.05	lin. ft.	$ 1.56	3,268.28
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		512.35
	(b) Deduction for material placed		Lump sum		− 512.35
151.	2-1/2 inches in diameter	4,708.27	lin. ft.	2.40	11,299.85
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		1,240.82
	(b) Deduction for material placed		Lump sum		−1,240.82
152.	3 inches in diameter	1,094.60	lin. ft.	2.90	3,174.34
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		527.45
	(b) Deduction for material placed		Lump sum		−527.45
153.	3-1/2 inches in diameter	39.70	lin. ft.	5.05	200.48
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		53.20
	(b) Deduction for material placed		Lump sum		−53.20
154.	4 inches in diameter	189.45	lin. ft.	4.46	844.95
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		226.74
	(b) Deduction for material placed		Lump sum		−226.74
155.	5 inches in diameter	465.36	lin. ft.	7.40	3,443.66
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		419.97
	(b) Deduction for material placed		Lump sum		−419.97
156.	Furnishing and installing nonmetallic conduit 4 inches in diameter	1,778.80	lin. ft.	1.25	2,223.50
157.	Furnishing and installing No. 1 cast-metal outlet boxes	258	Each	21.84	5,634.72
158.	Furnishing and installing No. 2 cast-metal outlet boxes	87	Each	35.33	3,073.71

APPENDIX C–Continued

Pay item	Property and pay item description	Quantity Amount	Quantity Unit	Labor and materials by contractor Unit cost	Labor and materials by contractor Total cost
	Furnishing and installing the following sizes of single-conductor, 600-volt insulated electrical wire and cable:				
	Thermoplastic				
159.	No. 14 AWG	55,148.30	lin. ft.	$ 0.04	$ 2,205.93
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		362.12
	(b) Deduction for material placed		Lump sum		–362.12
160.	No. 12 AWG	261,646.2	lin. ft.	0.05	13,082.31
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		2,543.70
	(b) Deduction for material placed		Lump sum		–2,543.70
161.	No. 10 AWG	152,865.3	lin. ft.	0.06	9,171.92
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		2,393.56
	(b) Deduction for material placed		Lump sum		–2,393.56
162.	No. 8 AWG	37,106.1	lin. ft.	0.08	2,968.49
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		1,512.04
	(b) Deduction for material placed		Lump sum		–1,512.04
163.	No. 6 AWG	11,315.0	lin. ft.	0.13	1,470.95
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		888.42
	(b) Deduction for material placed		Lump sum		–888.42
164.	No. 4 AWG	19,663.0	lin. ft.	0.16	3,146.08
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		651.36
	(b) Deduction for material placed		Lump sum		–651.36
	Synthetic Rubber (RHW)				
165.	No. 2 AWG	18,374.0	lin. ft.	0.38	6,982.12
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		2,525.79
	(b) Deduction for material placed		Lump sum		–2,525.79
166.	No. 1 AWG	12,987.0	lin. ft.	0.48	6,233.76
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		2,305.37

Pay item	Property and pay item description	Quantity: Amount	Quantity: Unit	Labor and materials by contractor: Unit cost	Labor and materials by contractor: Total cost
	Furnishing and installing the following sizes of single-conductor, 600-volt insulated electrical wire and cable:				
	(b) Deduction for material placed		Lump sum		$ –2,305.37
167.	No. 1/0 AWG	12,639.0	lin. ft.	$ 0.57	7,204.23
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		1,366.80
	(b) Deduction for material placed		Lump sum		–1,366.80
168.	No. 2/0 AWG	9,563.0	lin. ft.	0.71	6,789.73
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		1,986.45
	(b) Deduction for material placed		Lump sum		–1,986.45
169.	No. 3/0 AWG	470.0	lin. ft.	0.80	376.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		1,071.46
	(b) Deduction for material placed		Lump sum		–1,071.46
170.	No. 4/0 AWG	31,032.3	lin. ft.	0.95	29,480.68
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		7,250.92
	(b) Deduction for material placed		Lump sum		–7,250.92
171.	250,000-circular mil	97.0	lin. ft.	1.42	137.74
172.	500,000-circular mil	7,191.5	lin. ft.	1.90	13,663.85
	Ozone-Resisting Butyl Rubber				
173.	No. 1/0 AWG	1,879.0	lin. ft.	2.95	5,543.05
	Furnishing and installing the following multiconductor, 600-volt, insulated electrical wire and cable:				
	Thermoplastic				
174.	2-conductor No. 16 AWG	2,703.0	lin. ft.	0.14	378.42
175.	5-conductor No. 16 AWG	4,271.0	lin. ft.	0.48	2,050.08
176.	12-conductor No. 16 AWG	37,512.0	lin. ft.	0.95	35,636.40
177.	2-conductor No. 19/22 AWG	31,851.0	lin. ft.	0.33	10,510.83
178.	3-conductor No. 19/22 AWG	6,581.0	lin. ft.	0.47	3,093.07
179.	4-conductor No. 19/22 AWG	68,246.0	lin. ft.	0.55	37,535.30
180.	5-conductor No. 19/22 AWG	10,207.0	lin. ft.	0.66	6,736.62
181.	7-conductor No. 19/22 AWG	60,569.0	lin. ft.	0.83	50,272.27
182.	9-conductor No. 19/22 AWG	64,806.0	lin. ft.	1.05	68,046.30
183.	12-conductor No. 19/22 AWG	93,789.0	lin. ft.	1.42	133,180.38

Pay item	Property and pay item description	Quantity Amount	Quantity Unit	Labor and materials by contractor Unit cost	Labor and materials by contractor Total cost
	Furnishing and installing the following multiconductor, 600-volt, insulated electrical wire and cable:				
184.	3-conductor No. 10 AWG	529.5	lin. ft.	$ 0.28	$ 148.26
185,	3-conductor No. 8 AWG	0	lin. ft.	0.32	0
186.	2-conductor No. 6 AWG	0	lin. ft.	0.36	0
187.	3-conductor No. 6 AWG	0	lin. ft.	0.47	0
	Synthetic Rubber (RHW)				
188.	3-conductor No. 2 AWG	0	lin. ft.	1.06	0
189.	2-conductor No. 1 AWG	594.0	lin. ft.	1.14	677.16
190.	3-conductor No. 1/0 AWG	1,343.0	lin. ft.	1.33	1,786.19
	Furnishing and installing the following sizes of multiconductor, 600-volt insulated electrical armored cable:				
191.	3-conductor 750,000-circular mil	327.0	lin. ft.	6.00	1,962.00
192.	3-conductor 500,000-circular mil	802.0	lin. ft.	5.22	4,186.44
193.	3-conductor 350,000-circular mil	0	lin. ft.	3.50	0
194.	3-conductor No. 3/0 AWG	120.00	lin. ft.	2.32	278.40
195.	3-conductor No. 2/0 AWG	354.0	lin. ft.	1.90	672.60
196.	3-conductor No. 1/0 AWG	615.0	lin. ft.	1.56	959.40
	Furnishing and installing the following sizes of multiconductor, 5,000-volt insulated electrical armored cable:				
197.	3-conductor No. 1/0 AWG	1,367.0	lin. ft.	2.18	2,980.06
198.	3-conductor No. 1 AWG	4,288.0	lin. ft.	1.80	7,718.40
	Furnishing and installing the following single-conductor, 5,000-volt insulated electrical cable:				
199.	No. 1 AWG	769.5	lin. ft.	1.23	946.48
200.	750,000-circular mil	966.0	lin. ft.	3.20	3,091.20
201.	Furnishing and installing 15,000-volt insulated, No. 1 AWG, single-conductor, shielded, power cable for ungrounded neutral service	310.5	lin. ft.	1.80	558.90
202.	Installing 25-kilovolt insulated No. 1 AWG, 3-conductor, shielded power cable and including one pothead	1,816.0	lin. ft.	0.95	1,725.20
203.	Cable pulling tests	100%	Lump sum	345.00	345.00
204.	Making electrical connections, at equipment installed by others	100%	Lump sum	6,536.00	6,536.00
205.	Furnishing and installing switchboard wire on control and station-service equipment	13,008.75	lin. ft.	1.42	18,472.42

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
206.	Making panel cutouts for mounting equipment on station-service, control, and other equipment	805.25	lin. in.	$ 1.66	$ 1,336.72
207.	Installing control and graphic boards CCA and CCB; relay boards CCC, CCD and CCE; distribution boards BCA and MCA; sequence operations recorder equipment, board CCF; sequence operations recorder printer console	100%	Lump sum	6,694.00	6,694.00
208.	Load and frequency equipment and associated console (if required)	100%	Lump sum	442.00	442.00
209.	Installing one oil purifier room CO_2 control cabinet CMA	100%	Lump sum	93.00	93.00
210.	Installing four main transformer bank terminal cabinets E1A, E3A, E5A, and E7A	100%	Lump sum	427.00	427.00
211.	Installing eight turbine air admission system cabinets E1B, E2A, E3B, E4A, E5B, E6A, E7B, and E8A	100%	Lump sum	615.00	615.00
212.	Installing one 7,160-volt switchgear assembly U4B-U5B	100%	Lump sum	5,651.00	5,651.00
213.	Installing one 4,160-volt motor control center MMH and one control cubicle MMJ for Page water pumps	100%	Lump sum	1,971.00	1,971.00
214.	Installing three 1,500-kv.-a. unit substations No. 1 (K2C-D2B-D3B-K3C), No. 2 (K6C-D6B-D7B-K7C), and No. 3 (KMB-DMA-KMA)	100%	Lump sum	6,229.00	6,229.00
215.	Installing six 150-kv.-a. unit substations KAA-LAA, K1C-L1A, K3D-L3A, K5D-L5A, K7D-L7A, and KMC-LMA	100%	Lump sum	3,529.00	3,529.00
216.	Installing eight unit auxiliary boards D1A-B1A, D2A-B2A, D3A-B3A, D4A-B4A, D5A-B5A, D6A-B6A, D7A-B7A, and D8A-B8A	100%	Lump sum	8,009.00	8,009.00
217.	Installing one pump and compressor room power board MMB	100%	Lump sum	339.00	339.00
218.	Installing one booster pump power board MME	100%	Lump sum	339.00	339.00
219.	Installing four air-conditioning power boards N1A, N3A, N5A, and N7A	100%	Lump sum	2,685.00	2,685.00
220.	Installing six panelboards MMA, M4A, M2A, M3A, M6A, and M7A	100%	Lump sum	3,024.00	3,024.00

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
221.	Installing two direct-current motor-generator sets or static-rectifier chargers B1C and B1D and direct-current boards B1E, B1F, and B1G	100%	Lump sum	$ 1,182.00	$ 1,182.00
222.	Installing 125-volt direct-current metal-enclosed bus duct	148.23	lin. ft.	3.34	495.09
223.	Installing 10 motor starters for various equipment including (a) one timer for chlorination equipment (b) one pushbutton station in turbine pit alcove of each of the eight units	100%	Lump sum	1,170.00	1,170.00
224.	Installing elevator disconnect switch cabinet or cabinets	100%	Lump sum	170.00	170.00
	Installing emergency break-glass control stations as follows:				
225.	Transformer firewater control	8	Station	33.50	268.00
226.	Generator CO_2 control	8	Station	33.50	268.00
227.	Oil purifier room	1	Station	66.00	66.00
228.	Installing unmounted annunciator equipment	100%	Lump sum	2,588.00	2,588.00
229.	Installing station storage battery	100%	Lump sum	525.00	525.00
230.	Installing capacitor potential device adjustment units	13	Each	134.00	1,742.00
231.	Installing carrier-current transmitter-receiver sets	6	Each	660.00	3,960.00
232.	Installing twelve 100,000-kv.-a. single-phase power transformers complete with oil	100%	Lump sum	30,014.00	30,014.00
233.	Installing eight 14.4-kilovolt station-type generator switchgear assemblies	100%	Lump sum	11,397.00	11,397.00
234.	Installing generator voltage bus structures	100%	Lump sum	65,082.48	65,082.48
235.	Installing 3,750/4,687-kv.-a. station-service transformers	3.0	Each	856.00	2,568.00
236.	Installing 500-kv.-a. station-service voltage induction regulators	2	Each	999.00	1,998.00
237.	Installing eight generator neutral grounding transformers and resistors	100%	Lump sum	6,338.00	6,338.00
238.	Installing two 4,160-volt station-service nonsegregated-phase bus structures	100%	Lump sum	20,637.00	20,637.00
239.	Furnishing and installing flush-mounted lighting panelboards in concrete recesses	21	Each	364.00	7,644.00

Pay item	Property and pay item description	Quantity: Amount	Quantity: Unit	Labor and materials by contractor: Unit cost	Labor and materials by contractor: Total cost
240.	Furnishing and installing flush-mounted lighting panelboards in metal wall partitions	2	Each	$ 616.50	$ 1,233.00
241.	Furnishing and installing surface-mounted lighting panelboards	6	Each	334.00	2,004.00
242.	Furnishing and installing lighting system wiring devices	3,175	Each	8.94	28,384.50
243.	Furnishing and installing exterior lighting control time switches	11	Each	54.00	594.00
244.	Furnishing and installing lighting control magnetic contractors, size 1	2	Each	47.40	94.80
245.	Furnishing and installing lighting control magnetic contractors, size 2	2	Each	94.27	188.54
	Furnishing and installing the following types of lighting fixtures:				
246.	Type A	109	Each	20.46	2,230.14
247.	Types B, C, D, and E	601	Each	14.59	8,768.59
248.	Types F and R	53	Each	22.54	1,194.62
249.	Type G	29	Each	33.48	970.92
250.	Type H	23	Each	27.19	625.37
251.	Type J	32	Each	56.09	1,794.88
252.	Type M	140	Each	61.47	8,605.80
253.	Type N	16	Each	16.97	271.52
254.	Type P	14	Each	24.07	336.98
	Furnishing and installing the following types of lighting fixtures:				
255.	Type Q	11	Each	95.45	1,049.95
256.	Type S	32	Each	5.42	173.44
257.	Type T	72	Each	8.29	596.88
258.	Type U	10	Each	15.93	159.30
259.	Type V	8	Each	98.55	788.40
260.	Type W	2	Each	53.62	107.24
261.	Type AA, 4-foot	157	Each	35.21	5,527.97
262.	Type AA, 8-foot	532	Each	45.76	24,344.32
263.	Type BB, 4-foot	22	Each	21.04	462.88
264.	Type BB, 8-foot	69	Each	89.13	6,149.97
265.	Type CC, 4-foot	326	Each	31.67	10,324.42
266.	Type CC, 8-foot	17	Each	49.86	847.62
267.	Type DD, 4-foot, two-lamp	3	Each	38.67	116.01
268.	Type DD, 8-foot, two-lamp tandem	15	Each	60.86	912.90
269.	Type DD, 4-foot, four-lamp	13	Each	56.78	738.14
270.	Type EE	22	Each	72.15	1,587.30

APPENDIX C–Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
271.	Type FF	16	Each	$ 87.32	$ 1,397.12
	Installing nonembedded parts of the following types of lighting fixtures:				
272.	Type K	18	Each	49.47	890.46
273.	Type L	87	Each	12.36	1,075.32
274.	Installing 330, 230-, 25-kilovolt 100,000-kv.-a., single-phase autotransformers, complete with oil and tank-mounted lightning arresters	3.0	Each	2,819.87	8,459.61
275.	Deleted	–	–	–	–
276.	Deleted	–	–	–	–
277.	Installing 25-kilovolt, 16,000-kv.-a. 3-phase voltage regulating transformer, complete with oil	1	Each	1,522.94	1,522.94
278.	Installing 25-kilovolt, 20,000-kv.-a. 3-phase grounding transformer, complete with oil	1	Each	1,522.94	1,522.94
279.	Installing 25,000- to 240-volt, 300-kv.-a., 3-phase distribution transformer	1	Each	848.76	848.76
280.	Installing 14,400-, 120/240-volt, 50-kv.-a., single-phase distribution transformer	1	Each	386.65	386.65
281.	Installing 345-kilovolt, 25,000-mv.-a., power circuit breakers	6	Each	1,513.35	9,080.10
282.	Installing 230-kilovolt, 10,000-mv.-a., power circuit breakers	4.0	Each	1,676.88	6,707.52
283.	Deleted	–	–	–	–
284.	Installing 34.5-kilovolt, 1,000-mv.-a., power circuit breakers	4	Each	678.93	2,715.72
285.	Installing 345-kilovolt, 3-pole, manually gang-operated air switches with ground blades	2	Each	994.17	1,988.34
286.	Installing 345-kilovolt, 3-pole, manually gang-operated air switches	11	Each	911.56	10,027.16
287.	Installing 230-kilovolt, 3-pole, manually gang-operated air switches with ground blades	4	Each	830.30	3,321.20
288.	Installing 230-kilovolt, 3-pole, manually gang-operated air switches	6	Each	832.39	4,994.34
289.	Deleted	–	–	–	–
290.	Deleted	–	–	–	–
291.	Deleted	–	–	–	–

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
292.	Installing 23-kilovolt, 3-pole manually gang-operated air switches	5	Each	$ 500.96	$ 2,504.80
293.	Installing 23-kilovolt, 1-pole, hook-operated air switches	42	Each	84.26	3,538.92
294.	Deleted	–	–	–	–
295.	Installing 23-kilovolt, 3-pole, manually gang-operated interrupter switch	1	Each	668.80	668.80
296.	Installing 230-kilovolt current transformer	3.0	Each	420.37	1,261.11
297.	Deleted	–	–	–	–
298.	Installing 25-kilovolt current transformers	9	Each	108.30	974.70
299.	Installing 230-kilovolt potential transformers	6	Each	139.96	839.76
300.	Deleted	–	–	–	–
301.	Installing 25-kilovolt potential transformers	3	Each	141.55	424.65
302.	Installing 345-kilovolt coupling capacitor potential devices	7	Each	209.95	1,469.65
303.	Installing 230-kilovolt coupling capacitor potential devices	6	Each	208.68	1,252.08
304.	Deleted	–	–	–	–
305.	Installing 1,200-ampere wave traps	1	Each	104.97	104.97
306.	Installing 120/240-volt distribution cabinets	3	Each	461.06	1,383.18
307.	Installing 276-kilovolt station, base-mounted-type lightning arresters	9	Each	208.68	1,878.12
308.	Installing 195-kilovolt station, base-mounted-type lightning arresters	6	Each	205.99	1,235.94
309.	Deleted	–	–	–	–
310.	Installing 25-kilovolt station, base-mounted-type lightning arresters	3	Each	207.41	622.23
311.	Installing 23-kilovolt, 1-pole, hook-operated disconnecting fuses	4	Each	102.60	410.40
312.	Installing 15-kilovolt current transformers	4	Each	84.78	339.12
313.	Deleted	–	–	–	–
314.	Installing metering cabinets	4.0	Each	101.96	407.84
	Furnishing and installing the following sizes of strain buses and transformer circuits:				
315.	954,000-circular-mil all-aluminum	6,402.60	lin. ft.	0.90	5,762.34

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	Furnishing and installing the following sizes of strain buses and transformer circuits:				
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		$ 5,192.57
	(b) Deduction for material		Lump sum		–5,192.57
316.	954,000-circular-mil, ACSR	9,686.50	lin. ft.	$ 1.13	10,954.74
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		7,666.57
	(b) Deduction for material placed		Lump sum		–7,666.57
317.	1,272,000-circular-mil all-aluminum	5,142.50	lin. ft.	1.22	6,273.85
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		2,673.28
	(b) Deduction for material placed		Lump sum		–2,673.28
318.	2,167,000-circular-mil, ACSR	19,167.70	lin. ft.	1.71	32,776.77
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		18,073.07
	(b) Deduction for material placed		Lump sum		–18,073.07
319.	2,500,000-circular-mil all-aluminum	1,956.75	lin. ft.	2.47	4,883.17
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		2,093.13
	(b) Deduction for material placed		Lump sum		–2,093.13
	Furnishing and installing the following sizes of outdoor rigid aluminum buses:				
320.	3-inch iron-pipe-size	1,835.0	lin. ft.	5.36	9,835.60
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		2,018.10
	(b) Deduction for material placed		Lump sum		–2,018.10
321.	2-inch iron-pipe-size	1,142.0	lin. ft.	4.84	5,527.28
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		563.43
	(b) Deduction for material placed		Lump sum		–563.43

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
322.	1-1/2-inch iron-pipe-size	381.60	lin. ft.	$ 6.12	$ 2,335.39
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		323.43
	(b) Deduction for material placed		Lump sum		–323.43
	Furnishing and installing the following bus supporting insulator assemblies:				
323.	TR 131	43	Each	353.00	15,179.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		5,087.25
	(b) Deduction for material placed		Lump sum		–5,087.25
324.	TR 27	52	Each	312.00	16,224.00
325.	Deleted	–	–	–	–
326.	TR 7	98	Each	60.00	5,880.00
327.	TR 44	24	Each	64.00	1,536.00
328.	TR 202	4	Each	91.00	364.00
329.	Furnishing and installing 3/8-inch overhead ground wire	1,091.0	lin. ft.	0.27	294.57
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		104.65
	(b) Deduction for material placed		Lump sum		–104.65
330.	Furnishing and installing 1/2-inch overhead ground wire	8,131.0	lin. ft.	0.32	2,601.92
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		1,189.10
	(b) Deduction for material placed		Lump sum		–1,189.10
331.	Furnishing and installing bracket-type, outdoor lighting units	30	Each	71.00	2,130.00
332.	Furnishing and installing pipe standard type, outdoor lighting units	26	Each	65.00	1,690.00
333.	Furnishing and installing instrument transformer terminal boxes	10	Box	70.00	700.00
334.	Furnishing and installing type G terminal boxes	3	Box	76.00	228.00
335.	Deleted	–	–	–	–
336.	Installing miscellaneous electrical apparatus and equipment in dam and appurtenant structures	9,831.1	lb.	0.80	7,864.88

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
337.	Installing dry-type lighting transformers in dam and appurtenant structures	4,200.5	lb.	$ 0.33	$ 1,386.16
338.	Furnishing and installing fabricated sheet steel boxes and wireways	4,389.2	lb.	1.15	5,047.58
339.	Installing miscellaneous electrical devices in dam and appurtenant structures	4	Each	15.00	60.00
340.	Furnishing and installing electrical equipment for blocks 6 and 19 transformer chambers in dam	100%	Lump sum	12,328.00	12,328.00
341.	Completing electrical equipment installation for block 12 sump pumping units in dam	100%	Lump sum	2,158.00	2,158.00
342.	Furnishing and installing duct bank and armored control cable system for Page, Ariz., water supply system	100%	Lump sum	12,918.00	12,918.00
343.	Furnishing and installing 2-inch electrical metal conduit in duct banks, appurtenant structures	3,878.25	lin. ft.	0.95	3,684.34
344.	Furnishing and installing duct bank pull box drainpipes	142	lin. ft.	1.05	149.10
345.	Furnishing and installing sheet metal enclosure between each generator terminal box and cable duct (eight required)	100%	Lump sum	493.00	493.00
346.	Furnishing and installing materials for Colorado River crossing span for Glen Canyon-Page transmission line	100%	Lump sum	11,885.00	11,885.00
	*(a) Material delivered to site (85 percent of paid invoices)		Lump sum		2,069.30
	(b) Deduction for material placed		Lump sum		–2,069.30
347.	Furnishing and attaching airway obstruction markers to 1/2-inch steel strand overhead ground wire	28	Each	35.00	980.00
348.	Painting exterior surfaces of metal pipe, fittings, valves, and pipe hangers and supports installed under a previous contract	800.00	sq. ft.	0.50	400.00

APPENDIX C—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
349.	Furnishing and installing 2-conductor No. 12 AWG, 600-volt thermoplastic insulated electrical cable	0	lin. ft.	$ 0.19	$ 0
350.	Furnishing and installing type X lighting fixtures	19	Each	32.00	608.00
351.	Furnishing, installing, and removing 5,000-volt insulated electrical armored cable and supports, 3-conductor, No. 4/0	0	lin. ft.	7.00	0
352.	Furnishing, installing, and removing 600-volt insulated electrical armored cable and supports, 3-conductor 750,000-circular mil	0	lin. ft.	17.00	0
353.	Furnishing, installing, and removing 600-volt synthetic rubber insulated (RHW) electrical cable and supports, 2-conductor No. 2 AWG	0	lin. ft.	1.00	0
354.	Installing twelve 8,000-kv.-a. shunt reactors and for disconnecting and removing the reactors and bus connections	100%	Lump sum	6,779.00	6,779.00
355.	Drilling 3-inch-diameter holes through steel-reinforced concrete	40.25	lin. ft.	47.00	1,891.75
	Total—Original Contract				$8,530,008.00
	Order for Changes No. 1				
a.	Constructing a gravel blanket on switchyard embankments	3,876.49	cu. yd.	6.00	$ 23,258.94
b.	Drilling 6-inch-diameter cores in the prepacked second-stage concrete	46.00	lin. ft.	46.00	2,116.00
c.	Furnishing and installing type Y lighting fixtures	2	Each	34.25	68.50
d.	Performing standard dielectric tests of 19,000 volts on 4,160-volt station-service bus structures	100%	Lump sum	1,357.00	1,357.00
e.	Furnishing and installing 300,000-circular-mil, single-conductor, 600-volt, type RHW insulated electrical cable	1,394.0	lin. ft.	2.20	3,066.80
	Total—Order for Changes No. 1				$ 29,867.24

APPENDIX C—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	Order for Changes No. 2				
a.	Adjustment for change in neutral grounding cable terminals for each generator and for drilling and tapping the wrapper plates	100%	Lump sum	$ 328.00	$ 328.00
b.	Adjustment for deletion of sand-blasting for 4-inch concrete surfacing	4,440.78	sq. yd.	–0.70	–3,108.55
c.	Adjustment for deletion of sand-blasting for 3-1/2-inch concrete underbed	5,600.638	sq. yd.	–0.70	–3,920.45
d.	Providing facilities for electric power for installing and testing powerplant elevators	100%	Lump sum	1,640.00	1,640.00
e.	Furnishing electric power for installing and testing power-plant elevators	20.25	Month	30.00	607.50
f.	Furnishing and installing 4-inch aluminum conduit	96.5	lin. ft.	8.00	772.00
g.	Installing 25-kilovolt, single-conductor, 500,000-circular-mil, insulated, shielded power cable	1,890.0	lin. ft.	2.90	5,481.00
h.	Installing potheads for terminating 25-kilovolt, single-conductor cable	6	Each	155.00	930.00
i.	Furnishing and installing TR 46 bus supporting insulator assemblies	25	Each	60.00	1,500.00
j.	Installing 25-kilovolt, 500-ampere series reactors	3	Each	164.00	492.00
k.	Furnishing and installing 4/0 AWG aluminum, bare, outdoor cable buses	104.0	lin. ft.	2.85	296.40
l.	Providing additional influent connection in the sewage receiver tank and making connections thereto	100%	Lump sum	128.00	128.00
m.	Adjustment for change in power distribution panelboards	100%	Lump sum	267.50	267.50
	Total—Order for Changes No. 2				$ 5,413.40
	Order for Changes No. 3				
a.	Furnishing and installing additional steel members and revising existing members for pedestal insulator supports	100%	Lump sum	$ 257.00	$ 257.00

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
b.	Adjustments for changes in heating, ventilating and air-conditioning systems including deletion of unit AC-10	100%	Lump sum	$ –1,370.00	$ –1,370.00
c.	Furnishing and installing additional 3-inch supply and return piping for heating, ventilating and air-conditioning system in the powerplant control room	100%	Lump sum	641.07	641.07
d.	Furnishing and installing plug receptacle with tap circuit	100%	Lump sum	20.70	20.70
e.	Furnishing and installing white metal angle on tubes type C	86.90	lb.	2.66	231.15
f.	Grouting machine beams, hoistway doorframes and sills of the powerplant elevators	100%	Lump sum	7,155.72	7,155.72
g.	Furnishing and installing calking rings and calking for 1-1/2-inch conduits	48	Each	17.25	828.00
h.	Furnishing and installing calking rings and calking for 5-inch conduits	4	Each	20.25	81.00
i.	Furnishing and installing temporary electric facilities for generator erection	100%	Lump sum	4,167.62	4,167.62
j.	Furnishing electrical power for generator erection	12	Month	300.00	3,600.00
k.	Furnishing personnel for operation and maintenance of air compressors for generator erection	12	Month	325.00	3,900.00
l.	Modifying 6-inch airline between air compressor E-22, No. 3 and air receiver	100%	Lump sum	123.52	123.52
m.	Installing terminal boxes E1C and E8C	2	Each	126.50	253.00
n.	Furnishing and installing single-conductor, No. 16 AWG wire with type TW, 600-volt, 3/64 insulation	4,485.0	lin. ft.	0.16	717.60
	Total—Order for Changes No. 3				$ 20,606.38
	Order for Changes No. 4				
a.	Installing type RG-8A/U coaxial cable	4,751.0	lin. ft.	$ 0.36	$ 1,710.36

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
b.	Adjustment for filling three 100,000-kv.-a. single-phase autotransformers in the switch-yard	100%	Lump sum	$ 1,085.00	$ 1,085.00
c.	Filtering lubricating oil	61,390.0	gal.	0.04	2,455.60
d.	Installing tubular bus damper cable	100%	Lump sum	774.86	774.86
e.	Drilling holes in penstock piezometer plugs	32	Each	15.00	480.00
f.	Installing shunt reactor cabinet E8B	100%	Lump sum	266.00	266.00
g.	Installing watt-hour meters in cabinets EV3B, EV4B and EZ7B	100%	Lump sum	107.00	107.00
h.	Performing miscellaneous changes in powerboards D1A, D2A, and MMB	100%	Lump sum	435.29	435.29
i.	Modifying bus tubing connecting to 230-kilovolt and 345-kilovolt disconnect switches	100%	Lump sum	2,587.32	2,587.32
j.	Cleaning lifting beam assembly for powerplant 300-ton cranes	100%	Lump sum	472.00	472.00
k.	Replacing trip coils for circuit breaker JU1A	100%	Lump sum	313.15	313.15
l.	Modifying 230-kilovolt switch supports	100%	Lump sum	576.00	576.00
m.	Adjustments for change in 6-inch expansion joints	100%	Lump sum	1,212.00	1,212.00
n.	Adjustment for expedited delivery of control cable	100%	Lump sum	10,007.15	10,007.15
o.	Breaking the lacing on the cable trunk	100%	Lump sum	71.30	71.30
	Total–Order for Changes No. 4				$ 22,553.03
	Order for Changes No. 5				
a.	Replacing insulator adapters for the 3-inch rigid bus in the 230-kilovolt switchyard	100%	Lump sum	$ 2,388.71	$ 2,388.71
b.	Replacing two circuit breakers in board DVB	100%	Lump sum	241.50	241.50
c.	Replacing tees and furnishing and installing air chamber and vent valve for the cooling water pump suctions	100%	Lump sum	8,396.34	8,396.34
	Installing the following telephone equipment:				
d.	Soft-tone gongs	3	Each	31.00	93.00
e.	Six-inch bells	6	Each	62.00	372.00

Pay item	Property and pay item description	Quantity Amount	Quantity Unit	Labor and materials by contractor Unit cost	Labor and materials by contractor Total cost
	Installing the following telephone equipment:				
f.	Ten-inch bells	10	Each	$ 83.00	$ 830.00
g.	Vibrating horns	6	Each	41.50	249.00
h.	Code-call relays	16	Each	20.75	332.00
i.	Desk telephones	23	Each	20.75	477.25
j.	Wall telephones	71	Each	41.50	2,946.50
k.	Outdoor telephones	12	Each	62.00	744.00
l.	Extension bells	12	Each	83.00	996.00
	Furnishing and installing the following telephone cable supports and cabinets:				
m.	25-pair No. 22 AWG cable	511.0	lin. ft.	1.50	766.50
n.	18-pair No. 22 AWG cable	1,058.0	lin. ft.	1.21	1,280.18
o.	12-pair No. 22 AWG cable	400.0	lin. ft.	1.15	460.00
p.	2-conductor No. 16 AWG telephone cable	40,068.0	lin. ft.	0.23	9,215.64
q.	Epoxy-cemented cable supports	1,116	Each	1.63	1,819.08
r.	25-pair terminal cabinets	1	Each	178.25	178.25
s.	18-pair terminal cabinets	1	Each	177.50	177.50
t.	12-pair terminal cabinets	1	Each	166.75	166.75
u.	Modifying the powerplant sump pumps	100%	Lump sum	647.28	647.28
v.	Constructing a temporary tie for 230-kilovolt operation of the 345-kilovolt switchyard	100%	Lump sum	10,123.79	10,123.79
w.	Excavating, removing and replacing existing timber protection and backfilling for the cable trench	100%	Lump sum	85.11	85.11
x.	Furnishing and installing extensions to the 3-inch embedded pipe sleeves in the turbine pits	100%	Lump sum	590.70	590.70
y.	Connecting spare wire at the unit auxiliary control panels	100%	Lump sum	196.36	196.36
z.	Repairing concrete voids in the unit 6 penstock	100%	Lump sum	1,281.71	1,281.71
aa.	Furnishing and installing 3/4-inch galvanized steel air piping system	100%	Lump sum	1,384.94	1,384.94
bb.	Furnishing, installing and connecting control cables and switchboard wire and installing carrier receiver relays, text switches and auxiliary tripping relays	100%	Lump sum	824.93	824.93
cc.	Adjustment for overtime costs for expediting corrective modifications to the disconnecting switches	100%	Lump sum	413.37	413.37
	Total—Order for Changes No. 5				$ 47,678.39

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	Order for Changes No. 6				
	Furnishing, installing, and connecting the following sizes of single and multiconductor 600-volt insulated wire and cable:				
a.	1-conductor No. 12 AWG	60.0	lin. ft.	$ 0.14	$ 8.40
b.	1-conductor No. 10 AWG	368.0	lin. ft.	0.24	88.32
c.	1-conductor No. 8 AWG	985.0	lin. ft.	0.37	364.45
d.	1-conductor No. 4 AWG	369.0	lin. ft.	0.44	162.36
e.	1-conductor No. 1 AWG	116.0	lin. ft.	0.92	106.72
f.	2-conductor No. 6 AWG	0	lin. ft.	0.92	0
g.	2-conductor 19/22 AWG	2,129.0	lin. ft.	0.46	979.34
h.	4-conductor 19/22 AWG	10,309.0	lin. ft.	0.85	8,762.65
i.	5-conductor No. 16 AWG	0	lin. ft.	0.62	0
j.	7-conductor 19/22 AWG	1,660.0	lin. ft.	1.10	1,826.00
k.	12-conductor No. 16 AWG	234.0	lin. ft.	1.28	299.52
l.	12-conductor 19/22 AWG	7,954.0	lin. ft.	1.84	14,635.36
m.	Rerouting 7-conductor 19/22 AWG control cable	123.0	lin. ft.	0.69	84.87
n.	Making panel cutouts	384.5	lin. in.	2.88	1,107.36
o.	Furnishing and installing swing panel	100%	Lump sum	241.50	241.50
p.	Furnishing and installing brackets	241.85	sq. in.	5.75	1,390.64
q.	Furnishing and installing mimic bus and symbol material	0	lin. in.	1.73	0
r.	Furnishing, installing and connecting switchboard wire	1,693.0	lin. ft.	2.30	3,893.90
s.	Cleaning and priming unit 8 turbine control panelboard	100%	Lump sum	18.98	18.98
t.	Furnishing, installing, and painting (except aluminum ladders) access ladders and platforms in unit 1 through 8 turbine pits	100%	Lump sum	8,835.89	8,835.89
u.	Installing a temporary isolated phase bus connection	100%	Lump sum	11,588.27	11,588.27
v.	Adjustment for overtime costs for unloading lubricating oil	100%	Lump sum	227.32	227.32
w.	Removing a temporary isolated phase bus connection	100%	Lump sum	4,024.47	4,024.47
	Total–Order for Changes No. 6				$ 58,646.32
	Order for Changes No. 7				
a.	Furnishing 31,980 feet of 954,000-circular-mil 48/7 conductor cable	100%	Lump sum	$ 11,821.70	$ 11,821.70
	Total–Order for Changes No. 7				$ 11,821.70

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	Order for Changes No. 8				
a.	Replacing existing air release valve on the unit bearing cooling water supply with a pressure relief valve and rerouting the turbine bearing cooling water discharge line on all eight units	100%	Lump sum	$ 7,079.65	$ 7,079.65
	Total—Order for Changes No. 8				$ 7,079.65
	Order for Changes No. 9				
a.	Installing a 320-kilowatt, 250-volt motor-generator set	100%	Lump sum	$ 637.26	$ 637.26
b.	Removing safety guards and invert cleats and repairing the coal-tar enamel coating for penstocks 3, 5, 6, 7, and 8	100%	Lump sum	1,269.82	1,269.82
c.	Furnishing and installing electrical cables, replacing relay board panels, and installing Government-furnished devices for control and relaying of Glen Canyon-Flagstaff 345-kilovolt transmission lines No. 1 and 2	100%	Lump sum	23,757.47	23,757.47
d.	Replacing contacts in the unit starting circuits	100%	Lump sum	303.07	303.07
e.	Installing breaker position scanning equipment rack CCH and 2-conductor No. 22 shielded cable and furnishing and installing related control cables	100%	Lump sum	682.31	682.31
f.	Removing and delivering cable and relays from shunt reactor cabinet E8B	100%	Lump sum	60.72	60.72
g.	Furnishing and installing a tilting-disk-type check valve on each of the high-pressure vertical discharge risers from the cooling water centrifugal pumps	100%	Lump sum	1,692.44	1,692.44
h.	Painting firehose cabinets and racks exposed to public view to match the wall on which located	100%	Lump sum	85.02	85.02
	Total—Order for Changes No. 9				$ 28,488.11

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	Order for Changes No. 10				
a.	Installing floor plates and embedded angle frames for the 16-foot boring mill and furnishing and installing angle supports on the base of the boring mill	100%	Lump sum	$ 876.63	$ 876.63
b.	Replacing unserviceable conduit and wiring for the 16-foot boring mill	100%	Lump sum	2,799.39	2,799.39
c.	Drilling anchor boltholes for domestic water pumps	100%	Lump sum	1,170.00	1,170.00
d.	Drilling 7/8-inch-diameter anchor boltholes in the bases of four air receivers	100%	Lump sum	200.00	200.00
e.	Installing and removing the shunt reactor enclosures	100%	Lump sum	4,584.70	4,584.70
f.	Repainting the exterior surfaces of eight cubicle-type switchgear assemblies	100%	Lump sum	1,502.65	1,502.65
g.	Installing a 12-inch pipe sleeve in the machine shop floor	100%	Lump sum	319.78	319.78
h.	Painting the galvanized transformer cooling water and fire protection lines and the transformer oil supply lines	100%	Lump sum	2,802.20	2,802.20
i.	Removing and reinstalling working platforms in penstock 3	100%	Lump sum	375.37	375.37
j.	Modifying the 4-inch air line to the turbine shaft in unit bay 8 and replacing the solenoid-operated butterfly valve in unit 1	100%	Lump sum	175.00	175.00
k.	Installing waterflow switches and pressure switches, furnishing and installing 2-inch open vents, and connecting the pressure recorder	100%	Lump sum	13,594.88	13,594.88
l.	Modifying eight pipe trench covers and furnishing, installing, and painting eight sets of guardrails	100%	Lump sum	1,863.32	1,863.32
m.	Furnishing and installing 1/2-inch bleed lines in the transformer cooling water system	100%	Lump sum	931.98	931.98
n.	Removing and reinstalling jumpers from the 230-kilovolt Sigurd and Shiprock transmission lines	100%	Lump sum	152.00	152.00

APPENDIX C—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
o.	Modifying the type EE lighting fixture installation in the powerplant control room	100%	Lump sum	$ 1,136.93	$ 1,136.93
	Total—Order for Changes No. 10				$ 32,484.83
	Order for Changes No. 11				
a.	Reconditioning threads on assembly bolts and replacing a defective jack shaft	100%	Lump sum	$ 304.15	$ 304.15
b.	Removing first-stage concrete, replacing unserviceable and missing parts, and furnishing and installing an extension to the base holddown bracket for the boring mill	100%	Lump sum	1,088.38	1,088.38
c.	Modifying the transformer cooling water supply piping	100%	Lump sum	2,324.78	2,324.78
d.	Modifying metal doorframes to fit existing openings in first-stage concrete	100%	Lump sum	746.19	746.19
e.	Chipping first-stage concrete for switchgear channel supports	100%	Lump sum	1,436.97	1,436.97
f.	Removing bolt heads from temporary support bolts for the turbine pit liners for units 1 and 2, and grinding surface flush	100%	Lump sum	687.40	687.40
g.	Relocating two type M high-bay lighting fixtures	100%	Lump sum	328.18	328.18
h.	Furnishing and installing 1/2-inch check valves and related pipefittings in the generator cooler vent piping	100%	Lump sum	1,425.26	1,425.26
i.	Adjustment for furnishing and installing hot line fittings on deadend insulator assemblies for the powerplant to switchyard transformer circuits	100%	Lump sum	500.00	500.00
j.	Modifying the 5-inch warm water supply piping in unit bay 1	100%	Lump sum	455.54	455.54
k.	Furnishing and installing a 1-1/2-inch gate valve in the turbine bearing water supply line for each of the eight units	100%	Lump sum	337.00	337.00
l.	Applying coal-tar enamel on the interior surfaces of the penstocks downstream from the a—line wall of the powerplant	100%	Lump sum	2,926.78	2,926.78

APPENDIX C--Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
m.	Transferring oil from a switch-yard circuit breaker to a powerplant transformer	100%	Lump sum	$ 1,030.45	$ 1,030.45
n.	Installing and connecting eight adjustable gate position switches	100%	Lump sum	400.00	400.00
o.	Installing out-of-step blocking and time delay relays for the 230-kilovolt Shiprock No. 1 and 230-kilovolt Sigurd transmission lines	100%	Lump sum	300.00	300.00
p.	Installing temporary equipment and furnishing, installing, and connecting control cable for a unit dropping scheme	100%	Lump sum	1,960.80	1,960.80
q.	Installing a carrier-current transmitter-receiver	100%	Lump sum	120.00	120.00
r.	Removing and delivering the aluminum shunt reactor heat shield	100%	Lump sum	102.81	102.81
s.	Transferring insulating oil and replacing oil in the power transformer in bank K5A	100%	Lump sum	339.43	339.43
t.	Replacing unserviceable conduit and internal wiring for the used machine tools	100%	Lump sum	1,824.44	1,824.44
u.	Furnishing and installing thirty-four 1-1/2-inch hose adapter nipples with cap and chain for filter press connections	100%	Lump sum	3,158.77	3,158.77
	Total—Order for Changes No. 11				$ 21,797.33
	Order for Changes No. 12				
a.	Cleaning existing embedded turbine vent piping for units 2 through 8	100%	Lump sum	$ 1,581.15	$ 1,581.15
b.	Modifying the 3-inch service waterline at elevation 3216.21 in unit bay 1	100%	Lump sum	240.99	240.99
c.	Removing and reinstalling a section of the unit 1 isolated phase bus structure	100%	Lump sum	138.00	138.00
d.	Removing a faulty selector switch on the float-operated controller for the sump pumps in the dam and reinstalling the repaired switch	100%	Lump sum	116.74	116.74

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
e.	Furnishing and installing an overflow line from the actuators and a ball check valve on eight units	100%	Lump sum	$ 944.91	$ 944.91
f.	Modifying the 12-inch flange for the sump pumps in the dam and furnishing and installing brackets and supports on the pump column	100%	Lump sum	1,361.13	1,361.13
g.	For the change to furnishing and installing bus support clamps with 8-inch conductor spacing	100%	Lump sum	229.52	229.52
h.	Installing a Government-furnished 12-pair, No. 22 AWG, shielded cable for temporary connection of the telemetering equipment	100%	Lump sum	1,005.00	1,005.00
i.	Installing and dismantling a temporary oil return line	100%	Lump sum	1,174.68	1,174.68
j.	Making additional adjustments in the K1 and K7 transformer cooling water system	100%	Lump sum	1,800.00	1,800.00
k.	Installing eight Government-furnished unmounted circuit breaker kits in station-service powerboards D1A through D8A	100%	Lump sum	1,028.57	1,028.57
l.	Furnishing and installing air release valves, bleed lines and orifice plates in the transformer cooling water system	100%	Lump sum	947.34	947.34
m.	Installing Government-furnished load and frequency rack CCG6, installing two Government-furnished 51-pair No. 22 AWG telephone cables, and making miscellaneous wiring changes to the tone equipment	100%	Lump sum	2,475.00	2,475.00
n.	Alining the motor shaft and pump shaft for the Page water supply booster pumps and welding the 3/4-inch spacer plates to the base plate	100%	Lump sum	962.51	962.51
o.	Cleaning the powerplant unwatering and drainage sumps	100%	Lump sum	375.00	375.00
	Total—Order for Changes No. 12				$ 14,380.54
	<u>Order for Changes No. 13</u>				
a.	Removing grease and oil stains from the first stage concrete in the service bay and machine shop	100%	Lump sum	$ 820.95	$ 820.95

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
b.	Modifying the transformer fire protection piping	100%	Lump sum	$ 2,429.81	$ 2,429.81
c.	Replacing damaged hoist cable on the powerplant No. 2 crane auxiliary hook	100%	Lump sum	542.85	542.85
d.	Furnishing and installing metal plates under electrical equipment	100%	Lump sum	966.00	966.00
e.	Repairing damaged shop-applied paint on the 345-kilovolt switchyard air circuit breakers	100%	Lump sum	1,763.79	1,763.79
f.	Rerouting the turbine bearing cooling water discharge line on each of the eight units	100%	Lump sum	527.84	527.84
g.	Repairing defective shop-applied vinyl-resin paint on the interior surfaces of the turbine oil reservoirs	100%	Lump sum	1,156.99	1,156.99
h.	Removing dust from powerplant structural steel	100%	Lump sum	2,530.00	2,530.00
i.	Removing transfer trip test switch from Shiprock No. 1 line control and graphic board CCB 13	100%	Lump sum	120.00	120.00
j.	For applying and removing coal-tar enamel tape	100%	Lump sum	9,456.42	9,456.42
	Total—Order for Changes No. 13				$ 20,314.65
	Order for Changes No. 14				
a.	Adjustment for the additional costs of constructing the generator cooling water system	100%	Lump sum	11,876.99	11,876.99
b.	Adjustment for installing the motor control center for the Page domestic water pumps in two cubicles	100%	Lump sum	362.00	362.00
c.	Removing the forebay water level gage, shortening the float tape, and reinstalling the gage	100%	Lump sum	333.23	333.23
d.	Changing the forebay water level gage	100%	Lump sum	146.29	146.29
e.	Repairing surge damage to the Page domestic water supply system	100%	Lump sum	2,450.48	2,450.48
f.	Installing pressure gage snubbers on various gages in the power-plant	100%	Lump sum	149.08	149.08
	Total—Order for Changes No. 14				$ 15,318.07

Pay item	Property and pay item description	Quantity Amount	Quantity Unit	Labor and materials by contractor Unit cost	Labor and materials by contractor Total cost
	Order for Changes No. 15				
a.	Furnishing and installing 152 3/4-inch expansion anchors for cable tray supports	100%	Lump sum	$ 2,850.00	$ 2,850.00
b.	Removing shop-applied paint and repainting various turbine parts	100%	Lump sum	3,865.48	3,865.48
c.	Adjustment for installing the generator switchgear in two main assemblies with separate potential transformer sections	100%	Lump sum	16,100.91	16,100.91
d.	Modifying cable tray and lighting fixture supports	100%	Lump sum	351.04	351.04
e.	Adjustment for additional costs of installing the positive air pressure system on the isolated-phase bus	100%	Lump sum	32,103.28	32,103.28
f.	Painting galvanized and inaccessible surfaces of bus structure	100%	Lump sum	2,177.70	2,177.70
g.	Removing, cleaning and reinstalling drainage sump pumps in the powerplant	100%	Lump sum	133.13	133.13
h.	Performing additional flushing of turbine oil lines	100%	Lump sum	3,960.92	3,960.92
i.	Cleaning and painting exposed finished surfaces of turbine operating arms, wicket gate linkages, and head covers	100%	Lump sum	4,619.15	4,619.15
j.	Adjustment for furnishing and installing net head piezometer piping	100%	Lump sum	765.94	765.94
	Total—Order for Changes No. 15				$ 66,927.55
	Order for Changes No. 16				
a.	Furnishing and installing additional cable and wire and for drilling additional 3-inch holes in concrete	100%	Lump sum	$ 11,172.00	$ 11,172.00
	Total—Order for Changes No. 16				$ 11,172.00
	Total—Orders for Changes				$ 414,549.19
	Deductions				
	Less charge for water in accordance with paragraph 27 of the specifications	10,153.05	M gal.	$ 0.20	$ 2,030.61

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	Less charge for bulk cement wasted or for contractor's use in accordance with paragraph 36 of the specifications	2,711.37	bbl.	$ 3.38558	$ 9,179.56
	Less charge for bulk cement unaccounted for by contractor in accordance with paragraph 36 of the specifications	192.57	bbl.	3.38558	651.96
	Less charge for sacked cement wasted or for contractor's use in accordance with paragraph 36 of the specifications		Lump sum		1,482.89
	Less charge for sacked cement unaccounted for by contractor in accordance with paragraph 36 of the specifications	183.69	bbl.	3.79071	696.32
	Less charge for sacked cement used in lieu of bulk cement for contractor's convenience in accordance with paragraph 83 of the specifications		Lump sum		914.89
	Less charge for pozzolan wasted or for contractor's use in accordance with paragraph 36 of the specifications	63.046	Ton	11.40	718.72
	Less charge for pozzolan unaccounted for by contractor in accordance with paragraph 36 of the specifications	5.719	Ton	11.40	65.20
	Less charge for concrete aggregate in accordance with paragraph 36 of the specifications	316.51	cu. yd.	6.74	2,133.28
	Less charge for transportation cost adjustment on cement		Lump sum		10.03
	Deduction for mill price adjustment on cement wasted, used, or unaccounted for by the contractor	–	–	–	51.84
	Deduction for State sales tax on mill price adjustment on cement wasted, used, or unaccounted for by the contractor	–	–	–	0.78
	Less charge for hauling Government-furnished materials to the contractor at the jobsite in accordance with paragraph 36 of the specifications	36,635.27	cwt.	0.39	14,287.76
	Less charge for Government-furnished materials damaged or unaccounted for in accordance with paragraph 36 of the specifications	–	–	–	2,096.74

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	Less charge for Government services, materials, or use of equipment in accordance with paragraph 21 of the specifications				$ 8,644.76
	Less charge of 15 percent for Government overhead for services, materials, or use of equipment in accordance with paragraph 21 of the specifications				3,855.45
	Less liquidated damages deductions in accordance with paragraph 17 of the specifications:				
	Part 1:				
	Unit 1 May 16, 1963 through January 16, 1964	246	Day	$ 600.00	147,600.00
	Unit 2 September 13, 1963 through March 9, 1964	179	Day	600.00	107,400.00
	Unit 3 February 2, 1964 through April 14, 1964	73	Day	600.00	43,800.00
	Part 2: April 7, 1964 through April 25, 1964	19	Day	600.00	11,400.00
	Part 3:				
	Unit 1 March 17, 1964 through April 25, 1964	40	Day	600.00	24,000.00
	Part 4: April 11, 1964 through April 25, 1964	15	Day	600.00	9,000.00
	Part 5: April 11, 1964 through April 25, 1964	15	Day	600.00	9,000.00
	Part 9: July 16, 1966 through July 25, 1966	10	Day	200.00	2,000.00
	Total—Deductions				$ 401,020.79
	Credits				
	Credit of $26,400.00 as a result of an extension of time for part 1, units 1 and 2 authorized by findings of fact by the contracting officer dated January 15, 1964				
	Unit 1	22	Day	$ 600.00	$ 13,200.00
	Unit 2	22	Day	600.00	13,200.00

APPENDIX C—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	Credit of $53,400.00 as a result of an extension of time for parts 2, 4, 5, and deletion of part 3 in accordance with amendatory agreement of June 1, 1964				
	Part 2	19	Day	$ 600.00	$ 11,400.00
	Part 3	40	Day	600.00	24,000.00
	Part 4	15	Day	600.00	9,000.00
	Part 5	15	Day	600.00	9,000.00
	Credit of $272,400.00 for part 1 units 1, 2, and 3 authorized by findings of fact by the contracting officer dated September 9, 1964				
	Unit 1	224	Day	600.00	134,400.00
	Unit 2	157	Day	600.00	94,200.00
	Unit 3	73	Day	600.00	43,800.00
	Credit for operating powerplant cranes for generator installation in accordance with paragraph 47 of the specifications				23,854.51
	Credit in accordance with amendatory agreement of June 1, 1964				
	Item 11b				
	(1)		Lump sum		123,000.00
	(2)		Lump sum		119,000.00
	(3)		Lump sum		123,000.00
	(4)		Lump sum		123,750.00
	(5)		Lump sum		28,750.00
	(6)		Lump sum		28,750.00
	(7)		Lump sum		28,750.00
	Credit of $2,000.00 as a result of an extension of time for part 9 in accordance with order for changes No. 8 dated June 30, 1966	10	Day	200.00	2,000.00
	Total—Credits				$ 953,054.51

APPENDIX C—Continued

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	Recapitulation				
	Total—Original contract				$8,530,008.00
	Total—Orders for Changes				414,549.19
	Total—Deductions				–401,020.79
	Total—Credits				953,054.51
	Total—Net Earnings				$9,496,590.91

APPENDIX D

Chronology of important events related to the construction of the Glen Canyon unit:

Date	Event
	1956
May 19, 1956	Start of alinement survey of access road from Arizona-Utah State line to damsite by four-man party under General Investigation Funds.
July 1	Start of project. Assignment of project construction engineer and one construction engineer.
July 3	Bids opened at Salt Lake City, Utah, for rental of operated equipment for use in surfacing of access road, Arizona-Utah State line to damsite and the Wahweap Creek road under invitation No. 400-424, and contract No. 14-06-D-520 was awarded to Union Construction Co. on that date.
July 20	Bids opened at Salt Lake City, Utah, under invitation No. 400-426 for 50 portable houses.
July 23	Contract and Grant of Easement signed July 23, 1956, by Chairman, Navajo Tribal Council, for right-of-way for the Bitter Springs access road. This document subject to departmental approval by the Bureau of Indian Affairs, Washington, D.C.
July 27	Bids opened at Salt Lake City, Utah, for rental of operated equipment for use in surfacing access road, Arizona-Utah State line to the damsite and the Wahweap Creek road, under invitation No. 400-428.
August 1	Contract No. 14-06-414-6 awarded to Transa Homes Corp. under invitation No. 400-426 for furnishing and erecting 50 portable houses.
August 14	Access road from Kanab, Utah, to damsite opened to conventional travel.
August 14	Bids opened at Salt Lake City, Utah, under specifications No. 400C-63 for exploratory drilling and water testing at Glen Canyon damsite. Contract No. 14-06-414-7, awarded same date to Cannon Diamond Drilling Co., Compton, Calif.
August 15	Bids opened at Kanab, Utah, under specifications No. DC-4730 for earthwork and culverts, access highway station 158+67.70 to station 400+00, Glen Canyon Dam.
August 22	Access road to Wahweap area opened to conventional travel.
August 29	Notice to proceed received by Cannon Diamond Drilling Co., contract No. 14-06-414-7, specifications No. 400C-63 for exploratory drilling and water testing at Glen Canyon damsite.
September 1	First occupancy of Government trailer camp No. 1.
September 1	Start of construction, contract No. 14-06-414-7, specifications No. 400C-63.
September 6	Bids opened at Kanab, Utah, under specifications No. DC-4746 for Waterholes Canyon Bridge access highway, Glen Canyon Dam.
September 11	All bids received under specifications No. DC-4746 rejected by supplemental notice No. 2, the work to be readvertised as part B of specifications No. DC-4756.

Date	Event
	1956—Continued
September 11	Bids opened at Kanab, Utah, under specifications No. DC-4747 for Glen Canyon Dam, right diversion tunnel.
September 21	Award of contract No. 14-06-D-2036 to Strong Co. under specifications No. DC-4730.
September 24	Notice to proceed received by the Strong Co. under contract No. 14-06-D-2036, specifications No. DC-4730.
September 29	Start of construction, contract No. 14-06-D-2036, specifications No. DC-4730.
October 1	Award of contract No. 14-06-D-2045 to Mountain States Construction Co. under specifications No. DC-4747.
October 2	Notice to proceed received by Mountain States Construction Co. under contract No. 14-06-D-2045, specifications No. DC-4747.
October 8	Start of construction by Mountain States Construction Co. under contract No. 14-06-D-2045, specifications No. DC-4747.
October 9	Bids opened at Kanab, Utah, under specifications No. 400C-68 for completion of gravel surfacing, Arizona-Utah State line to Glen Canyon damsite road and Wahweap Creek road and construction of airstrip.
October 9	Bids opened at Kanab, Utah, under specification No. DC-4756 for earthwork and structures, access highway, station 400+00 to station 1497+50, Glen Canyon Dam.
October 10	Contract No. 14-06-420-1 awarded to Squires Drilling Co. for rental of portable rotary drill rig and crawler tractor.
October 15	Setting off initial blast at the downstream portal of the right diversion tunnel at a signal transmitted by President Eisenhower in connection with the Glen Canyon "Ground Breaking" ceremonies.
October 30	Award of contract No. 14-06-D-2188 to W. W. Clyde and Co. under specifications No. DC-4756.
November 5	Notice to proceed received by W. W. Clyde and Co. under contract No. 14-06-D-2188, specifications No. DC-4756.
November 7	Start of construction work under contract No. 14-06-D-2188, specifications No. DC-4756.
November 8	Award of contract No. 14-06-400-544 to Ford-Fielding, Inc., under specifications No. 400C-68.
November 9	Notice to proceed received by Ford-Fielding, Inc., under contract No. 14-06-400-544, specifications No. 400C-68.
November 14	Start of construction by Ford-Fielding, Inc., under contract No. 14-06-400-544, specifications No. 400C-68.

Date	Event
	1956—Continued
December 7	First occupancy of Government trailer camp No. 2.
December 18	Bids opened at Kanab, Utah, under specifications No. DC-4800, for Colorado River (Glen Canyon) Bridge.
	1957
January 1, 1957	Award of contract No. 14-06-D-2240 to Kiewit-Judson Pacific Murphy under specifications No. DC-4800.
January 28	Notice to proceed received by Kiewit-Judson Pacific Murphy under contract No. 14-06-D-2240, specifications No. DC-4800.
January 29	All work completed and accepted under contract No. 14-06-400-544, specifications No. 400C-68 by Ford-Fielding, Inc.
February 14	Start of construction by Kiewit-Judson Pacific Murphy under contract No. 14-06-D-2240, specifications No. DC-4800.
March 22	Announcement by Commissioner Dexheimer that townsite would be on the east side of the river and would be called Page, Ariz. in honor of John C. Page, former Commissioner of the Bureau of Reclamation.
April 11	Bids opened at Kanab, Utah, on specifications No. DC-4825, Glen Canyon Dam and Powerplant.
April 16	Bids opened at Kanab, Utah, on specifications No. DC-4865, 150,000-gallon elevated water tank and 3,000,000-gallon storage reservoir for Glen Canyon community facilities.
April 17	Contract No. 14-06-420-19 awarded to Ralph Child Construction Co. for operated-equipment rental for construction of a jeep trail for access to the Colorado River.
April 23	Bids opened at Kanab, Utah, on specifications No. DC-4886, grading area and constructing foundations for demountable houses at Glen Canyon community.
April 29	Award of contract No. 14-06-D-2403 to Merritt-Chapman and Scott Corp. for construction of Glen Canyon Dam and Powerplant under specifications No. DC-4825.
May 2	Notice to proceed received by Merritt-Chapman and Scott Corp. under contract No. 14-06-D-2403, specifications No. DC-4825.
May 7	Bids opened at Kanab, Utah, on specifications No. DC-4887, surfacing and guardrail access highway Glen Canyon Dam and Manson Mesa Airstrip.
May 13	Award of contract No. 14-06-D-2438 to Pittsburgh-Des Moines Steel Co. for furnishing and erecting a 150,000-gallon elevated water tank and a 3,000,000-gallon storage reservoir for Glen Canyon community facilities under specifications No. DC-4865.

Date	Event
	1957—Continued
May 15	Award of contract No. 14-06-D-2424 to Martin-Rice Construction Co. for grading area and constructing foundations for demountable houses at Glen Canyon community under specifications No. DC-4886.
May 18	Notice to proceed received by Martin-Rice Construction Co. under contract No. 14-06-D-2424, specifications No. DC-4886.
May 28	Bids opened at Kanab, Utah, on specifications No. DC-4896, access roads, streets and utilities for Page, Ariz.
May 28	Award of contract No. 14-06-D-2437 to Alexander Construction Co., Inc., for surfacing and guardrail access highway Glen Canyon Dam under specifications No. DC-4887.
May 31	Notice to proceed received by Alexander Construction Co., Inc., under contract No. 2437, specifications No. DC-4887.
June 4	Bids opened at Kanab, Utah, on specifications No. 400C-83, earthwork, structures and surfacing for Arizona-Utah State line to Vista Point road and parking area.
June 7	Notice to proceed received by Pittsburgh-Des Moines Steel Co. under contract No. 14-06-D-2438, specifications No. DC-4865.
June 7	Start of construction by W. W. Clyde and Co. under specifications No. DC-4896 before award of contract.
June 12	Start of construction by Merritt-Chapman and Scott Corp. under contract No. 14-06-D-2403, specifications No. DC-4825.
June 13	Award of contract No. 14-06-400-624 to W. W. Clyde and Co. for earthwork, structures and surfacing for Arizona-Utah State line to Vista Point road and parking area under specifications No. 400C-83.
June 14	Notice to proceed received by W. W. Clyde and Co. under contract No. 14-06-400-624, specifications No. 400C-83.
June 18	Start of construction by Martin-Rice Construction Co. under contract No. 14-06-D-2424, specifications No. DC-4886.
June 19	Bids opened at Kanab, Utah, on specifications No. DC-4912, sewage treatment plant for Page, Ariz.
June 20	All work completed and accepted under contract No. 14-06-414-7, specifications No. 400C-63.
June 27	Start of construction by Alexander Construction Co. under contract No. 14-06-D-2437, specifications No. DC-4887.
July 9	Bids opened at Kanab, Utah, on specifications No. DC-4924, laboratory, municipal building and warehouse for Page, Ariz.

Date	Event
	1957—Continued
July 9	Start of construction by W. W. Clyde and Co. under contract No. 14-06-400-624, specifications No. 400C-83.
July 12	All work completed and accepted under contract No. 14-06-D-2188, specifications No. DC-4756.
July 25	Award of contract No. 14-06-D-2622 to W. W. Clyde and Co. for access roads, streets and utilities for Page, Ariz., under specifications No. DC-4896.
August 8	Award of contract No. 14-06-D-2629 to W. W. Clyde and Co. for sewage treatment plant for Page, Ariz., under specifications No. DC-4912.
August 13	Bids opened at Kanab, Utah, on specifications No. DC-4933, water supply system for Page, Ariz.
August 16	All work completed and accepted under contract No. 14-06-D-2424, specifications No. DC-4886.
August 22	Award of contract No. 14-06-D-2647 to Security Construction Co. for laboratory, municipal building and warehouse for Page, Ariz., under specifications No. DC-4924.
August 22	Start of construction by W. W. Clyde and Co. under contract No. 14-06-D-2629, specifications No. DC-4912.
September 6	Notice to proceed received by W. W. Clyde and Co. under contract No. 14-06-D-2622, specifications No. DC-4896.
September 6	Notice to proceed received by W. W. Clyde and Co. under contract No. 14-06-D-2629, specifications No. DC-4912.
September 10	Award of contract to Southern Engineering and Construction Co. under contract No. 14-06-D-2646, specifications No. DC-4933.
September 12	Bids opened at Kanab, Utah, on specifications No. DC-4945, residences for Page, Ariz.
September 12	All work completed and accepted under contract No. 14-06-400-624, specifications No. 400C-83.
September 13	Notice to proceed received by Security Construction Co. under contract No. 14-06-D-2647, specifications No. DC-4924.
September 13	Notice to proceed received by Southern Engineering and Construction Co., Inc., under contract No. 14-06-D-2646, specifications No. DC-4933.
September 27	Start of construction by Security Construction Co. under contract No. 14-06-D-2647, specifications No. DC-4924.
October 1	Start of construction by Southern Engineering and Construction Co., Inc., under contract No. 14-06-D-2646, specifications No. DC-4933.

Date	Event
	1957—Continued
October 1	All work completed and accepted under contract No. 14-06-D-2036, specifications No. DC-4730.
October 3	Specifications No. DC-4945, supplemental notice No. 5 rejected all bids received.
October 12	All work completed and accepted under contract No. 14-06-400-624, specifications No. 400C-83.
October 28	Weather station installation completed.
October 29	Weather station at Page, Ariz., completed and in operation.
October 31	Bids opened at Kanab, Utah, on specifications No. 400C-98, earthwork, structures and surfacing for Vista Point access road, parking area and shelter.
November 7	Bids opened at Kanab, Utah, on specifications No. DC-4989, residences for Page, Ariz.
November 15	Award of contract No. 14-06-400-685 to W. W. Clyde and Co. for earthwork, structures and surfacing for Vista Point road, parking area and shelter, Glen Canyon Damsite under specifications No. 400C-98.
November 16	Notice to proceed was received under specifications No. 400C-98.
December 10	Award of contract No. 14-06-D-2698 to Mobilhome Corp., for residences at Page, Ariz., under specifications No. DC-4989.
December 16	Start of construction by W. W. Clyde and Co. under contract No. 14-06-400-685, specifications No. 400C-98.
December 31	Prime contractor given a 30-day notice of funds exhaustion.
	1958
January 4, 1958	Bitter Springs access highway completed and accepted.
January 9	Notice to proceed received by Mobilhome Corp., et al, under schedule No. 1, contract No. 14-06-D-2698, specifications No. DC-4989.
January 29	Start of erection of the water tank and storage reservoir by Pittsburgh-Des Moines Steel Co. under contract No. 14-06-D-2438, specifications No. DC-4865.
February 18	Start of construction by Mobilhome Corp., et al, under contract No. 14-06-D-2698, specifications No. DC-4989.
March 3	All work by Alexander Construction Co. under contract No. 14-06-D-2437, specifications No. DC-4887, completed and accepted.
March 13	All work by Mountain States Construction Co. under contract No. 14-06-D-2045, specifications No. DC-4747, completed and accepted.

Date	Event
	1958—Continued
April 15	All work by W. W. Clyde and Co. under contract No. 14-06-400-685, specifications No. 400C-98, completed and accepted.
May 2	All work by W. W. Clyde and Co. under contract No. 14-06-D-2629, specifications No. DC-4912, completed and accepted.
June 16	Notice to proceed received by Mobilhome Corp., et al, under schedule No. 2, contract No. 14-06-D-2698, specifications No. DC-4989.
July 2	All work by Security Construction Co. under contract No. 14-06-D-2647, specifications No. DC-4924, completed and accepted.
July 10	Bids opened at Kanab, Utah, on specifications No. DC-5066, lighting installation for Glen Canyon Airport at Page, Ariz.
July 23	Award of contract No. 14-06-D-2884 to A and B Lighting for lighting installation for Glen Canyon Airport at Page, Ariz., under specifications No. DC-5066.
July 28	Notice to proceed received by A and B Lighting under contract No. 14-06-D-2884, specifications No. DC-5066.
August 25	Start of construction by A and B Lighting under contract No. 14-06-D-2884, specifications No. DC-5066.
August 25	Work completed by Pittsburgh-Des Moines Steel Co., under contract No. 14-06-D-3438, specifications No. DC-4865.
September 4	Bids opened at Kanab, Utah, on specifications No. DC-5090, furnishing and installing control cable from river pumps to filtration plant at Page, Ariz.
September 24	Award of contract No. 14-06-D-2914 to A and B Lighting for furnishing and installing control cable from river pumps to filtration plant at Page, Ariz., under specifications No. DC-5090.
October 2	Notice to proceed received by A and B Lighting under contract No. 14-06-D-2914, specifications No. DC-5090.
November 1	All work on the filtration plant and sedimentation tank to supply completely treated water to the distribution system under specifications No. DC-4933 was completed and accepted.
November 8	Start of construction by A and B Lighting under contract No. 14-06-D-2914, specifications No. DC-5090.
November 13	Bids opened at Kanab, Utah, on specifications No. DC-5115, administration building and garage, and fire station and police building for Page, Ariz.
November 25	Headquarters of the Glen Canyon unit moved from Kanab, Utah, to Page, Ariz.
November 25	All work by A and B Lighting under contract No. 14-06-D-2884, specifications No. DC-5066, completed and accepted.

APPENDIX D—Continued

Date	Event
	1958—Continued
December 4	Award of contract No. 14-06-D-3110 awarded to Sierra Construction Corp. for administration building and garage, fire station and police building for Page, Ariz. under specifications No. DC-5115.
December 10	Notice to proceed received by Sierra Construction Co. under contract No. 14-06-D-3110, specifications No. DC-5115.
December 19	Start of construction by Sierra Construction Corp. under contract No. 14-06-D-3110, specifications No. DC-5115.
December 19	All work completed by W. W. Clyde and Co. under contract No. 14-06-D-2622, specifications No. DC-4896.
	1959
January 20, 1959	Bids opened at Page, Ariz., on specifications No. DC-5123, 50-ton motor truck scale and scale house for Glen Canyon Dam.
January 22	Unauthorized walkout of five basic crafts of prime contractor's forces, specifications No. DC-4825.
February 4	Five basic crafts returned to work on prime contract after unauthorized walkout.
February 6	All work by A and B Lighting under contract No. 14-06-D-2914, specifications No. DC-5090, completed and accepted.
February 11	Initial diversion of river through the right diversion tunnel at 7:30 a.m.
February 16	Award of contract No. 14-06-D-3130 to W. W. Clyde and Co. for 50-ton motor truck scale and scale house for Glen Canyon Dam.
February 17	All work by Kiewit-Judson Pacific Murphy under contract No. 14-06-D-2240, specifications No. DC-4800, except for painting below the deck, completed and accepted.
February 20	Glen Canyon Bridge and Page Hospital dedication ceremonies.
March 3	Notice to proceed received by W. W. Clyde and Co. under contract No. 14-06-D-3130, specifications No. DC-5123.
March 16	Hydrostatic test of waterline successfully completed by Southern Engineering and Construction Co., specifications No. DC-4933.
March 20	Start of construction by W. W. Clyde and Co. under contract No. 14-06-D-3130, specifications No. DC-5123.
March 25	All work by Kiewit-Judson Pacific Murphy under contract No. 14-06-D-2240, specifications No. DC-4800, completed and accepted.

Date	Event
	1959—Continued
April 20	All work by Mobilhome Corp., et al, under contract No. 14-06-D-2698, specifications No. DC-4989, completed and accepted.
April 29	Bids opened at Page, Ariz., on specifications No. DC-5163, seismograph vault and access road for Glen Canyon seismograph station.
May 15	Work by Sierra Construction Corp., on garage and fire station under contract No. 14-06-D-3110, specifications No. DC-5115, completed and accepted.
May 19	First diversion through left diversion tunnel at 6:00 p.m.
May 20	Bids opened at Page, Ariz., on specifications No. 400C-135, soil stabilization and landscaping, Page, Ariz.
May 26	Award of contract No. 14-06-D-3170 to W. W. Clyde and Co. for seismograph vault and access road for Glen Canyon seismograph station.
May 28	Notice to proceed received by W. W. Clyde and Co. under contract No. 14-06-D-3170, specifications No. DC-5163.
June 2	Start of construction by W. W. Clyde and Co. under contract No. 14-06-D-3170, specifications No. DC-5163.
July 3	All work by W. W. Clyde and Co. under contract No. 14-06-D-3130, specifications No. DC-5123, completed and accepted.
July 6	Start of strike against the prime contractor by the five basic crafts; construction on the dam and powerplant halted.
August 5	Bids opened at Page, Ariz., on specifications No. 400C-140, soil stabilization and landscaping, Page, Ariz.
August 12	All work by W. W. Clyde and Co. under contract No. 14-06-D-3170, specifications No. DC-5163, completed and accepted.
August 20	All work by Sierra Construction Corp. under contract No. 14-06-D-3110, specifications No. DC-5115, completed and accepted.
August 28	Award of contract No. 14-06-400-1007 to L. A. Landscaping Co. for soil stabilization and landscaping, Page, Ariz., under specifications No. 400C-140.
August 29	Notice to proceed received by L. A. Landscaping Co., under contract No. 14-06-400-1007, specifications No. 400C-140.
September 17	Start of construction by L. A. Landscaping Co. under contract No. 14-06-400-1007, specifications No. 400C-140.
December 22	Agreement signed between prime contractor and the five basic crafts for settlement of strike.

APPENDIX D—Continued

Date	Event
	1960
January 4, 1960	Return to work of prime contractor's forces after the strike which began July 6, 1959.
January 21	Bids opened on specifications No. DC-5206, parking area and utilities for block 17, Page, Ariz.
February 15	Award of contract No. 14-06-D-2388 to A. J. Kelton for parking area and utilities for block 17, Page, Ariz.
February 20	Notice to proceed received by A. J. Kelton under contract No. 14-06-D-2388, specifications No. DC-5206.
February 24	Start of construction by A. J. Kelton under contract No. 14-06-D-2388, specifications No. DC-5206.
May 31	First placement of mass concrete in unit bay 1 of the powerplant.
June 10	All work by A. J. Kelton under contract No. 14-06-D-2388, specifications No. DC-5206, completed and accepted.
June 17	First placement of mass concrete in the dam block 10-B.
June 24	All work by L. A. Landscaping Co. under contract No. 14-06-400-1007, specifications No. 400C-140, completed and accepted.
	1961
January 24, 1961	Bids opened on specifications No. 400C-161, temporary tourist center at Glen Canyon Dam.
February 10	Award of contract No. 14-06-400-1389 to Dearden Construction Co. for temporary tourist center at Glen Canyon Dam.
February 13	Notice to proceed received by Dearden Construction Co. under contract No. 14-06-400-1389, specifications No. 400C-161.
February 28	Start of construction by Dearden Construction Co. under contract No. 14-06-400-1389, specifications No. 400C-161.
March 21	Bids opened on specifications No. DC-5494, constructing foundations and furnishing and erecting steel towers for Glen Canyon-Four Corners 230-kilovolt transmission line.
March 24	All bids received under specifications No. DC-5494 rejected under supplemental notice No. 6.
May 8	One-millionth cubic yard of concrete placed in dam.
June 19	All work by Dearden Construction Co. under specifications No. 400C-161 completed and accepted.

APPENDIX D—Continued

Date	Event
	1961—Continued
June 20	Bids opened on specifications No. DC-5607, erection of column struts for Glen Canyon Bridge.
June 28	Award of contract No. 14-06-D-4040 to Yuba Consolidated Industries, Inc., for erection of column struts for Glen Canyon Bridge.
July 18	Bids opened on specifications No. DC-5610, Glen Canyon-Shiprock 230-kilovolt transmission line.
July 18	Start of construction by Yuba Consolidated Industries, Inc., under contract No. 14-06-D-4040, specifications No. DC-5607.
July 19	One-millionth barrel of cement delivered to project.
July 25	Notice to proceed received by Yuba Consolidated Industries, Inc., under contract No. 14-06-D-4040, specifications No. DC-5607.
August 4	All work by Yuba Consolidated Industries, Inc., under specifications No. DC-5607, completed and accepted.
September 6	Award of contract No. 14-06-D-4075 to Electrical Constructors for Glen Canyon-Shiprock 230-kilovolt transmission line.
October 5	Notice to proceed received by Electrical Constructors under contract No. 14-06-D-4075, specifications No. DC-5610.
October 17	Two-millionth cubic yard of concrete placed in dam.
December 28	Bids opened on specifications No. DC-5710, constructing foundations and furnishing and erecting steel towers for 42 miles of Shiprock-Cortez section of Shiprock-Curecanti 230-kilovolt transmission line.
	1962
January 2, 1962	Award of contract No. 14-06-D-4259 to Irby Construction Co. for constructing foundations and furnishing and erecting steel tower for 42 miles of Shiprock-Cortez section of Shiprock-Curecanti 230-kilovolt transmission line.
February 8	Start of construction by Electrical Constructors under contract No. 14-06-D-4075, specifications No. DC-5610.
February 20	Bids opened on specifications No. 400C-189, water supply modifications for Page, Ariz.
February 28	Award of contract No. 14-06-400-2260 to Nelson Brothers Construction Co. for water supply modifications for Page, Ariz.
April 5	Notice to proceed received by Nelson Brothers Construction Co. under contract No. 14-06-400-2260, specifications No. 400C-189.

Date	Event
	1962—Continued
April 5	Bids opened on specifications No. DC-5753, Cortez-Curecanti 230-kilovolt transmission line and stringing conductors and overhead ground wires for Shiprock-Cortez 230-kilovolt transmission line.
April 23	Award of contract No. 14-06-D-4415 to Electrical Contractors for Cortez-Curecanti 230-kilovolt transmission line.
April 23	Award of contract No. 14-06-D-4645 to Malcolm W. Larson Contracting Co. for stringing conductors and overhead ground wires for Shiprock-Cortez 230-kilovolt transmission line.
May 10	Start of construction by Nelson Brothers Construction Co. under contract No. 14-06-400-2260, specifications No. 400C-189.
May 14	Notice to proceed received by Irby Construction Co. under contract No. 14-06-D-4259, specifications No. DC-5710.
May 15	Three-millionth cubic yard of concrete placed in dam.
May 24	Bids opened on specifications No. 400C-199, repeater station antenna towers, huts, and diesel engine generator sets for VHF radio communication system for Glen Canyon-Shiprock 230-kilovolt transmission line.
May 29	Bids opened on specifications No. DC-5766, Glen Canyon-Flagstaff-Pinnacle Peak 345-kilovolt transmission line.
June 4	Award of contract No. 14-06-400-2441 to Bixby Builders, Inc., for repeater station antenna towers, huts, diesel engine generator sets for VHF radio communication system for Glen Canyon-Shiprock 230-kilovolt transmission line.
June 5	Bids opened on specifications No. DC-5750, completion of Glen Canyon Powerplant, Switchyard, Dam, and appurtenant works.
June 13	Start of construction by Electrical Constructors under contract No. 14-06-D-4415, specifications No. DC-5753, schedule No. 1.
June 22	Award of contract No. 14-06-D-4428 to Ets-Hokin and Galvan, Inc., for Glen Canyon-Flagstaff-Pinnacle Peak 345-kilovolt transmission line.
June 25	Award of contract No. 14-06-D-4420 for completion of Glen Canyon Powerplant, Switchyard, Dam, and appurtenant works to Ets-Hokin and Galvan, Inc.
July 5	Start of construction by Irby Construction Co. under contract No. 14-06-D-4259, specifications No. DC-5710.
July 16	Notice to proceed received by Ets-Hokin and Galvan, Inc., under contract No. 14-06-D-4428, specifications No. DC-5766.
July 19	All work by Nelson Brothers Construction Co. under specifications No. 400C-189, completed and accepted.

Date	Event
	1962–Continued
July 19	Notice to proceed received by Ets-Hokin and Galvan, Inc., under contract No. 14-06-D-4429, specifications No. DC-5750.
July 30	Start of construction by Ets-Hokin and Galvan, Inc., under contract No. 14-06-D-4429, specifications No. DC-5750.
August 1	Notice to proceed received by Electrical Constructors, under contract No. 14-06-D-4415, specifications No. DC-5753, schedule No. 1.
August 2	Notice to proceed received by Bixby Builders, Inc., under contract No. 14-06-400-2441, specifications No. 400C-199.
August 16	Start of construction by Bixby Builders, Inc., under contract No. 14-06-400-2441, specifications No. 400C-199.
October 2	Start of construction by Ets-Hokin and Galvan, Inc., under contract No. 14-06-D-4428, specifications No. DC-5766, schedules No. 1 and 2.
October 31	All work by Bixby Builders, Inc., under specifications No. 400C-199 completed and accepted.
November 19	Four-millionth cubic yard of concrete placed in the dam.
November 27	Bids opened for specifications No. DC-5850, 230-kilovolt transmission lines, vicinity of Shiprock Substation.
December 19	Award of contract No. 14-06-D-4642 to Reynolds Electrical and Engineering Co., Inc., for 230-kilovolt transmission lines, vicinity of Shiprock Substation.
	1963
March 1, 1963	Notice to proceed received by Reynolds Electrical and Engineering Co., Inc., under contract No. 14-06-D-2642, specifications No. DC-5850.
March 13	Closure of Colorado River and start of storage of water in Lake Powell at 2:00 p.m.
March 26	Bids opened on specifications No. DC-5907, Shiprock Substation (stage 01).
April 9	Start of construction by Reynolds Electrical and Engineering Co. under contract No. 14-06-D-4642, specifications No. DC-5850.
April 17	Award of contract No. 14-06-D-4694 to Reynolds Electrical and Engineering Co. for Shiprock Substation (stage 01).
May 8	Award of contract No. 14-06-400-3030 to Techni-Builders, Inc., for sidewalk canopies for Glen Canyon Bridge under specifications No. 400C-231 (SF).
May 25	Notice to proceed received by Techni-Builders, Inc., under contract No. 14-06-400-3030, specifications No. 400C-231 (SF).

Date	Event
	1963—Continued
June 18	Bids opened on specifications No. DC-5953, Experimental section of the Glen Canyon-Shiprock 230-kilovolt transmission line.
June 21	Notice to proceed received by Reynolds Electrical and Engineering Co., under contract No. 14-06-D-4694, specifications No. DC-5907.
June 22	Notice to proceed received by Malcolm W. Larson Contracting Co., under contract No. 14-06-D-4645, specifications No. DC-5753, schedule No. 2.
July 9	Start of construction by Reynolds Electrical and Engineering Co., under contract No. 14-06-D-4694, specifications No. DC-5907.
July 10	Award of contract No. 14-06-D-4897 to Construction Helicopters, Inc., for experimental section of Glen Canyon-Shiprock 230-kilovolt transmission line.
July 11	Start of construction by Malcolm W. Larson Contracting Co., under contract No. 14-06-D-4645, specifications No. DC-5753, schedule No. 2.
July 22	Notice to proceed received by Construction Helicopters, Inc., under contract No. 14-06-D-4897.
August 9	Start of construction by Construction Helicopters, Inc., under contract No. 14-06-D-4897, specifications No. DC-5953.
August 12	Start of construction by Techni-Builders, Inc., under contract No. 14-06-400-3030, specifications No. 400C-231 (SF).
August 22	All work by Techni-Builders, Inc., under contract No. 14-06-400-3030, specifications No. 400C-231 (SF), completed and accepted.
September 5	All work completed and accepted under contract No. 14-06-D-4645, specifications No. DC-5753, schedule No. 2, by Malcolm W. Larson Contracting Co.
September 5	All work completed and accepted under contract No. 14-06-D-4259, specifications No. DC-5710, by Irby Construction Co.
September 13	Last bucket of mass concrete placed in dam.
October 21	Walkout by all crafts, except electrical workers, under the completion contract only. All work stopped on the completion contract except erection of structural steel structures in the switchyard and wiring, etc., in the powerplant.
November 12	Return to work of all crafts which were on strike against the completion contractor.
November 26	Bids opened on specifications No. DC-6030, Glen Canyon-Page transmission line.
December 31	Award of contract No. 14-06-D-5043 to Construction Helicopters, Inc., for Glen Canyon-Page transmission line.

APPENDIX D—Continued

Date	Event
	1964
January 3, 1964	Notice to proceed received by Construction Helicopters, Inc., under contract No. 14-06-D-5043, specifications No. DC-6030.
January 23	Start of construction by Construction Helicopters, Inc., under contract No. 14-06-D-5043, specifications No. DC-6030, Glen Canyon-Page transmission line.
January 28	All work by Electrical Constructors under specifications No. DC-5610 completed and accepted.
February 28	All work by Electrical Constructors under specifications No. DC-5753, schedule No. 1 completed and accepted.
March 5	Bids opened on specifications No. DC-6057, terrazzo and wall tile installations for Glen Canyon Dam and Powerplant.
March 10	All bids on specifications No. DC-6057 rejected by supplemental notice No. 3.
March 25	All work by Construction Helicopters, Inc., under specifications No. DC-5953 completed and accepted.
March 26	Bids again opened on specifications No. DC-6057, terrazzo and wall tile installations for Glen Canyon Dam and Powerplant.
April 20	Award of contract No. 14-06-D-5088 to Marus Marble and Tile Co., Inc., for terrazzo and wall tile installations for Glen Canyon Dam and Powerplant and notice to proceed received by the contractor April 21, 1964.
April 23	Bids opened on specifications No. 400C-255, water supply system for Shiprock Substation.
April 30	Award of contract No. 14-06-400-3491 to Ed. H. Foster's Son for water supply system for Shiprock Substation.
May 25	Notice to proceed received by Ed. H. Foster's Son under contract No. 14-06-400-3491, specifications No. 400C-255.
June 5	All work by Reynolds Electrical and Engineering Co., Inc., under specifications No. DC-5907 completed and accepted.
June 11	Bids opened on specifications No. DC-6100, Glen Canyon-Flagstaff transmission line No. 2.
June 13	All work by Reynolds Electrical and Engineering Co., Inc., under specifications No. DC-5850 completed and accepted.
July 7	Bids opened on specifications No. 400C-262, bituminous seal coat for streets, parking area and service areas, Page, Ariz.
July 9	Award of contract No. 14-06-D-5237 to Ets-Hokin Corp. for Glen Canyon-Flagstaff transmission line No. 2, specifications No. DC-6100.

Date	Event
	1964—Continued
July 14	Notice to proceed received by Ets-Hokin Corp. under contract No. 14-06-D-5237, specifications No. DC-6100.
July 17	Award of contract No. 14-06-400-3703 to Nelson Brothers Construction Co. for bituminous seal coat for streets, parking area and service areas, Page, Ariz.
July 23	Bids opened on specifications No. 400C-266, Aero Avenue waterline extension, Page, Ariz.
July 24	Notice to proceed received by Nelson Brothers Construction Co. under contract No. 14-06-400-3703, specifications No. 400C-262.
July 24	All work by Ed. H. Foster's Son under specifications No. 400C-255 completed and accepted.
August 3	Start of construction by Ets-Hokin Corp. under specifications No. DC-6100.
August 7	Start of construction by Nelson Brothers Construction Co. under specifications No. 400C-262.
August 7	Award of contract No. 14-06-400-3718 to E&S Plumbing and Heating Co. for Aero Avenue waterline extension, Page, Ariz.
August 7	All work by Construction Helicopters, Inc., under specifications No. DC-6030 completed and accepted.
August 20	Bids opened on specifications No. 400C-268, exterior painting—200 houses at Page, Ariz.
August 27	Notice to proceed received by E&S Plumbing and Heating Co. under contract No. 14-06-400-3718, specifications No. 400C-266.
September 4	Start of commercial power generation by generating unit 1 at 11:30 p.m.
September 10	Start of construction by E&S Plumbing and Heating Co. under specifications No. 400C-266.
September 22	Start of commercial power generation by generating unit 2 at 3:00 a.m.
September 23	Award of contract No. 14-06-400-3775 to H. Underwood Co., exterior painting—200 houses at Page, Ariz.
September 28	Start of work by Marus Marble and Tile Co., Inc., under specifications No. DC-6057.
October 2	All work by Merritt-Chapman and Scott Corp. under specifications No. DC-4825 accepted as substantially complete.
October 12	Notice to proceed received by H. Underwood Co. under contract No. 14-06-400-3775, specifications No. 400C-268.

APPENDIX D—Continued

Date	Event
	1964—Continued
October 16	All work by Nelson Brothers Construction Co. under specifications No. 400C-262 completed and accepted.
October 19	Start of work by H. Underwood Co. under specifications No. 400C-268.
November 10	All work by E&S Plumbing and Heating Co. under specifications No. 400C-266 completed and accepted.
November 12	All work by Ets-Hokin Corp. under specifications No. DC-5766 completed and accepted.
December 11	Start of commercial power generation by generating unit 3 at 6:22 p.m.
December 11	All work by H. Underwood Co. under specifications No. 400C-268 completed and accepted.
December 30	Station service power supply for the powerplant, dam, and switchyard picked up through the powerplant generator facilities at 4:30 p.m.
December 31	Prime contractor, specifications No. DC-4825, relieved of operation of the hospital and of furnishing electrical power to the Government and completion of cleanup of the contractor's camp.
	1965
February 10, 1965	Generating unit 4 on line for scheduled generation at 3:45 p.m.
February 16	Opening of bids under specifications No. DC-6220, Flagstaff Substation (stage 01).
March 4	Award of contract No. 14-06-D-5461, specifications No. DC-6220 to Howard P. Foley Co., Tucson, Ariz.
March 15	Notice to proceed received by the Howard P. Foley Co., Tucson Airz., for Flagstaff Substation (stage 01).
March 18	Opening of bids under specifications No. DC-6238, 69-kilovolt and microwave power supply additions to Glen Canyon Switchyard.
March 30	Award of contract No. 14-06-D-5493, specifications No. DC-6238 to Tide-Bay, Inc., Tacoma, Wash.
April 9	Notice to proceed received by Tide-Bay, Inc., Tacoma, Wash., for 69-kilovolt and microwave power supply additions to Glen Canyon Switchyard.
April 23	Start of construction by Howard P. Foley Co. under specifications No. DC-6220.
April 23	All work by Marus Marble and Tile Co. specifications No. DC-6057, accepted as substantially completed.
May 20	Opening of bids under specifications No. 400C-291, log boom for Lake Powell.

APPENDIX D—Continued

Date	Event
	1965—Continued
May 27	Opening of bids under specifications No. 400C-292, roof repairs for Glen Canyon Powerplant.
May 27	Opening of bids under invitation No. DS-6276, passenger elevators for Glen Canyon Visitor Center.
June 4	Award of contract No. 14-06-400-4021, specifications No. 400C-291 to R. H. Jackson and Associates, Golden, Colo.
June 4	Award of contract No. 14-06-400-4025, specifications No. 400C-292 to Superior Roofing Co., Inc., Salt Lake City, Utah.
June 7	Start of construction by Tide-Bay, Inc., under specifications No. DC-6238.
June 8	Opening of bids under specifications No. DC-6274, Visitor Center complex.
June 14	Notice to proceed received by R. H. Jackson and Associates for log boom for Lake Powell, specifications No. 400C-291.
June 17	Notice to proceed received by Superior Roofing Co., Inc., for roof repairs for Glen Canyon Powerplant, specifications No. 400C-292.
June 23	Start of construction by R. H. Jackson and Associates under specifications No. 400C-291.
June 30	Award of contract No. 14-06-D-5634, specifications No. DC-6274, to Allen M. Campbell Co., Tyler, Texas.
July 7	Start of construction by Superior Roofing Co., Inc., under specifications No. 400C-292.
July 12	Notice to proceed received by Allen M. Campbell Co. for construction of Visitor Center complex, specifications No. DC-6274.
July 19	Generating unit 5 on line for scheduled generation at 1:05 p.m.
July 29	Opening of bids under specifications No. DC-6317, left diversion tunnel plug and spillway elbow.
August 6	Award of contract No. 14-06-D-5670, specifications No. DC-6317, to S. S. Mullen, Inc., Seattle, Wash.
August 16	Start of construction by Allen M. Campbell Co. under specifications No. DC-6274.
August 23	Notice to proceed received by S. S. Mullen, Inc., for left diversion tunnel plug and spillway elbow, specifications No. DC-6317.
August 31	All work by Superior Roofing Co., Inc., under specifications No. 400C-292, completed and accepted.
September 13	Start of construction by S. S. Mullen, Inc., under specifications No. DC-6317.

Date	Event
	1965—Continued
October 4	Glen Canyon-Pinnacle Peak 345-kilovolt transmission line No. 1 in service at 5:30 p.m.
October 7	Opening of bids under specifications No. DC-6351 (SF), furnishing and installing handrailing and safety gates for control cable tunnel.
October 13	Generating unit 6 on line for scheduled generation at 1:53 p.m.
October 25	All work by R. H. Jackson and Associates accepted as substantially complete, specifications No. 400C-291.
October 28	Award of contract No. 14-06-D-5773, specifications No. DC-6351 (SF), to Techni-Builders, Inc., Phoenix, Ariz. Notice to proceed received by Techni-Builders, Inc., for furnishing and installing handrailing and safety gates for control cable tunnel, specifications No. DC-6351 (SF).
December 4	All work by Tide-Bay, Inc., under specifications No. DC-6238, accepted as substantially complete.
December 6	Start of construction by Techni-Builders, Inc., under specifications No. DC-6351 (SF).
December 10	All work by Techni-Builders, Inc., under specifications No. DC-6351 (SF), completed and accepted.
	1966
January 21, 1966	Generating unit 7 on line for scheduled generation.
February 18	Initial start and bearing run for generating unit 8.
February 28	Generating unit 7 on line for scheduled generation.
April 8	Interior Board of Appeals decision IBCA-365 in favor of prime contractor, Merritt-Chapman and Scott Corp., specifications No. DC-4825.
April 24	All work by Ets-Hokin Corp. under specifications No. DC-6100 accepted as substantially complete.
July 12	Start of construction by Montgomery Elevator Co. under invitation No. DS-6276.
August 2	First bid opening under specifications No. DC-6441, handrailings on generator air housings for Glen Canyon Powerplant.
August 25	Opening of bids under re-advertised specifications No. DC-6441, handrailings on generator air housings for Glen Canyon Powerplant.
September 7	Specifications No. DC-5750—Changeover from river pumping plant to permanent water supply system for Page, Ariz.

APPENDIX D—Continued

Date	Event
	1966—Continued
September 14	Award of contract No. 14-06-D-6020, specifications No. DC-6441, to Larsen Rigging and Equipment Co. of Riverton, Utah.
September 15	Notice to proceed received by Larsen Rigging and Equipment Co. for generator air housing handrails, specifications No. DC-6441.
September 22	Dam and powerplant dedicated by the first lady, Mrs. Lyndon B. Johnson.
October 25	All work by Ets-Hokin Corp., under specifications No. DC-5750, accepted as substantially complete.
December 9	All work by the Howard P. Foley Co. under specifications No. DC-6220 accepted as substantially complete.
December 31	Project transferred to O&M status.

APPENDIX E

Glen Canyon Bridge—Technical information:[1]

GLEN CANYON BRIDGE
Colorado River Storage Project
Arizona-Colorado-New Mexico-Utah-Wyoming

General Information

Purpose:

Glen Canyon Unit: To create a reservoir to provide conservation storage, control sediment, abate floods, facilitate recreational development, aid in fish and wildlife conservation, and produce electrical energy.

Glen Canyon Bridge: To serve as a vital link in the new highway to the remote damsite, extending between Flagstaff, Ariz., and Kanab, Utah, a distance of about 200 miles. The bridge is also essential to the transportation of construction materials and equipment by truck to the damsite as there are no rail facilities near the dam.

Location: On the Colorado River in Arizona near the Utah border (approximately 17 miles upstream from Lees Ferry and 12 river miles downstream from the Arizona-Utah State line).

Name Change: In the authorization, appropriations, specifications, original drawings, etc., this bridge was identified as the Colorado River Bridge. However, in 1959 the bridge was officially named the Glen Canyon Bridge as an aid in obtaining a more precise location identification.

Technical Information

Bridge type: Steel-arch type with a single span
Height above river: Approximately 700 feet
Length of bridge: 1,271 feet including abutments with an arch span of 1,028 feet
Location: 865 feet downstream from axis of Glen Canyon Dam
Elevation of bridge deck: 3,828 feet
Bridge roadway: The concrete roadway is 30 feet wide with 4-foot sidewalks on each side
Approximate construction quantities:
Structural steel: 7,837,000 pounds
Reinforcing steel: 371,000 pounds
Handrailings: 110,000 pounds
Concrete for bridge abutments, skewbacks and deck: 2,550 cubic yards

[1] Reprinted from a previously published technical record on this feature.

Frontispiece—Part of the crowd which attended the Glen Canyon Bridge dedication ceremony, February 20, 1959. P-557-420-03356.

1. GENERAL DESCRIPTION AND LOCATION.–The Glen Canyon Bridge across the Colorado River is a steel-arch type with a single span some 700 feet high and is the highest of its type in the United States. Its length is 1,271 feet including abutments, with an arch span of 1,028 feet, making it the second longest of its type in the United States. The bridge is located 865 feet downstream from the axis of Glen Canyon Dam. The elevation of the bridge deck at 3,828 feet is 113 feet higher than the crest of the dam. The concrete roadway is 30 feet wide with 4-foot sidewalks on each side and with 4-foot-high vertical bar type handrails. For general views of the bridge, see figures 1 and 2.

The bridge is a link in the highway between Flagstaff, Ariz., located on U.S. Highway No. 89 and Kanab, Utah (fig. 3). The 24-mile access highway from Bitter Springs, Ariz., to the Glen Canyon Bridge is a paved highway built to Arizona State Highway standards by the Bureau of Reclamation under three construction contracts. From the west abutment to the Arizona-Utah State line, a distance of about 8 miles, there is a temporary paved road built by the Bureau of Reclamation; and from the State line to a point 8 miles east of Kanab, the State of Utah has constructed a paved highway which joins Utah State Highway No. 136 at that point. The nearest railroad point is Flagstaff, Ariz., 135 miles away, and the nearest town is Kanab, Utah, 80 miles west of the site.

Visualized after completion of the dam is a recreational area with its tourist attraction similar to the National Park recreational area at Hoover Dam. The records show that during the last few years the latter park had more visitors than any other National Park in the United States.

2. REQUIREMENTS AND PRELIMINARY STUDIES.–At first, consideration was given to routing the highway over the dam. The crest of the dam is over a hundred feet below the rim of the canyon. To build highway approaches to fit the curved crest of the arch dam and meeting present specifications for curvatures, sight distances, and grades would have required a 90-foot-deep rock cut on one side and a tunnel on the other. Further, the crest of the dam would have to be widened and limited parking facilities, paralleling and bordering the traveled highway, would create a traffic hazard. Another and most serious concern was that until the dam was completed the two new highways, not being connected, would be of limited use to the public.

Realizing that a vehicular crossing over the canyon for use during construction would be of tremendous advantage, cost estimates were made for a suspension bridge serving construction purposes only. Estimates were made for a 24-foot roadway and 2-foot safety curbs, timber floor, timber runway planks and timber railway, for an H 20 loading. Stiffening trusses, towers and cables were designed for an H 15 loading. Further, the stiffening trusses were designed using 25 percent higher than normal unit stresses for live load, and 40 percent higher stresses for a combination of live and windloads. The cables were proportioned using a design stress of 70,000 pounds per square inch. Such a bridge was estimated to cost $1,800,000.

Feasibility-type estimates were also made for a permanent bridge located approximately 1,000 feet downstream from the dam, with site data taken from available topographic sheets. The canyon rims are approximately 1,100 feet apart and 700 feet above the river. The canyon walls are red Navajo sandstone. Estimates were made for a steel arch and a suspension span and were based on a 24-foot roadway, two 4-foot sidewalks and H 20-S 16 loading. The arch had a clear span of 1,040 feet while the towers of the suspension span were 1,200 feet apart. Having made allowance in the unit prices for the difficulty of erecting the arch, the arch bridge was estimated to cost $2,920,000 and the suspension bridge $3,020,000. The cost of the two types of bridge being nearly the same, the choice fell to the arch as it offered greater flexibility in planning road approaches and vista areas, and further, was much more pleasing in appearance.

3. JOINT FINANCING AGREEMENT.–When the State of Arizona and the Bureau of Public Roads expressed their willingness to participate in the cost of a permanent crossing, all the factors previously mentioned were reconsidered. It was concluded that a permanent bridge below the dam, constructed at the earliest possible date, would be the best solution. Negotiations with the parties concerned led to an agreement whereby the cost of the bridge would be borne as follows:

Bureau of Public Roads	$600,000
State of Arizona	$800,000 plus 25 percent of the cost above $3,200,000
Bureau of Reclamation	$1,800,000 plus 75 percent of the cost above $3,200,000

In view of the benefits derived from early completion of the bridge, it was decided to issue specifications for the bridge ahead of specifications for the dam and limit the construction period to 2 years.

(a) *Cost.*—Construction costs by pay items for the bridge, which was constructed under specifications No. DC-4800, are presented later in this appendix.

4. AUTHORIZATION.—Construction of the Glen Canyon Bridge was authorized by an act of April 11, 1956 (Public Law 485, 84th Congress, 70 Stat. 105) and is appurtenant to the Glen Canyon unit.

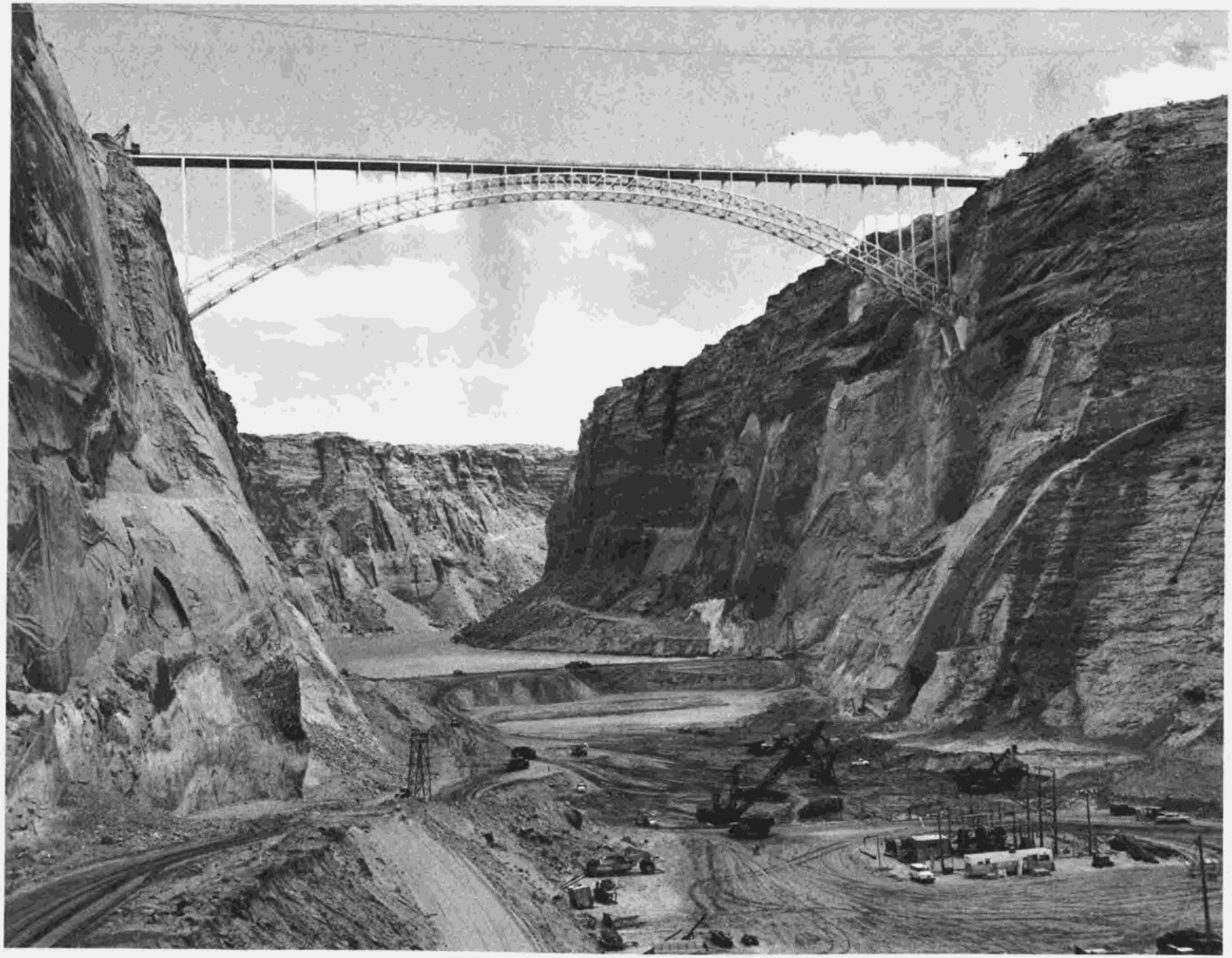


Figure 1.—General view of Glen Canyon Bridge across the Colorado River just downstream from Glen Canyon Dam in Arizona. P-557-420-03765, May 18, 1959.

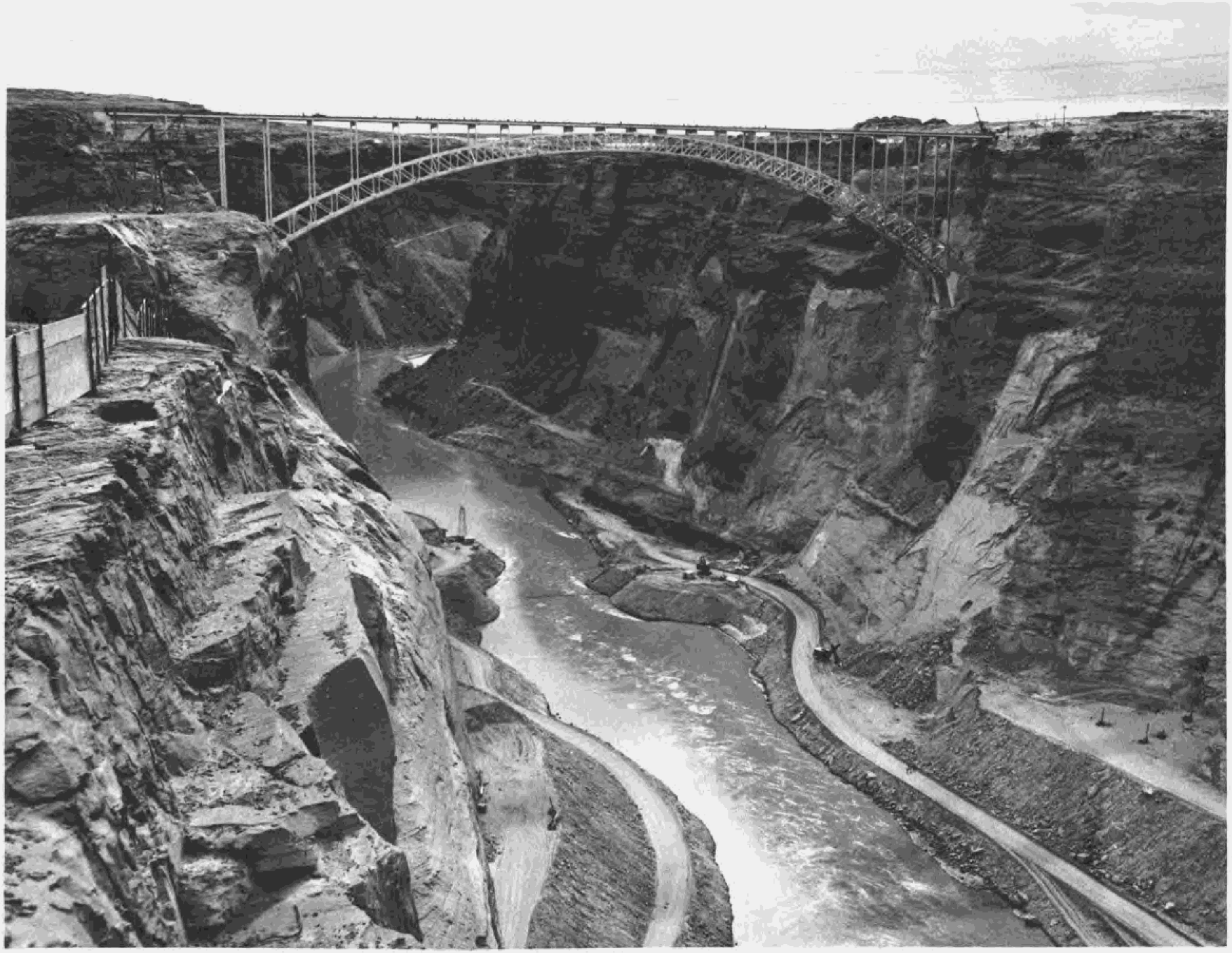


Figure 2.—View downstream showing the nearly completed Glen Canyon Bridge, the downstream cofferdam, and the right keyway. P557-400-236, January 30, 1959.

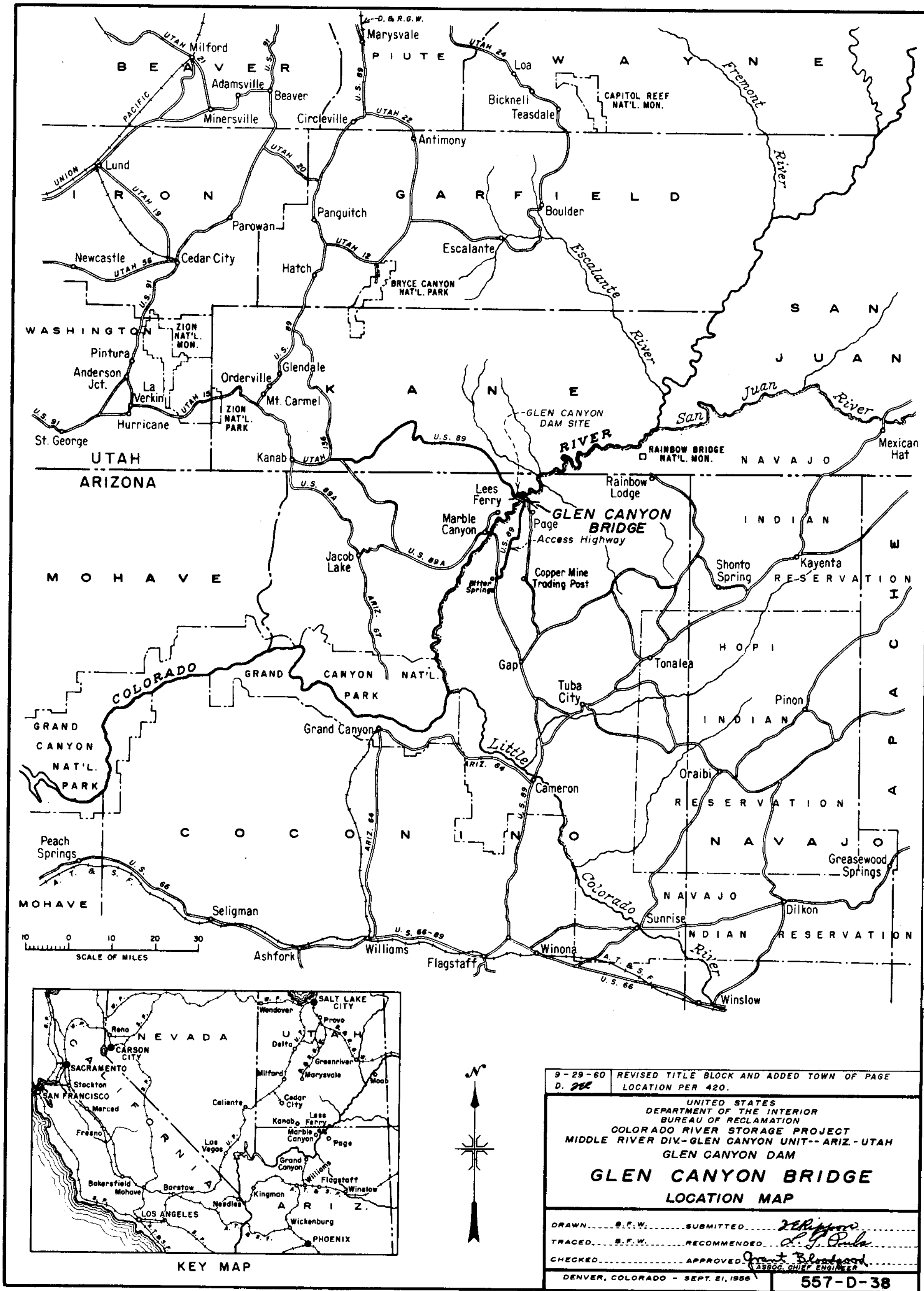


Figure 3.—Glen Canyon Bridge location map.

APPENDIX E—Continued

Construction costs by pay items—Glen Canyon Bridge—Specifications No. DC-4800, Contract No. 14-06-D-2240:

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
	Original Contract				
1	Excavation for abutments and pedestals	882.58	cu. yd.	$ 60.30	$ 53,219.57
2	Excavation for skewbacks	1,017.11	cu. yd.	98.65	100,337.90
3	Drilling holes for dowel bars and grouting bars in place	3,750.0	lin. ft.	6.68	25,050.00
4	Concrete in abutments and pedestals	376.21	cu. yd.	98.51	37,060.45
5	Concrete in skewbacks	245.68	cu. yd.	52.32	12,853.98
6	Concrete in bridge deck	1,048.3	cu. yd.	145.00	152,003.50
7	Furnishing and handling cement	4,284.28	bbl.	14.26	61,093.83
8	Furnishing and placing reinforcement bars in abutments, pedestals and skewbacks	128,178.99	lbs.	0.18	23,072.22
9	Furnishing and placing reinforcement bars in bridge deck	243,021.8	lbs.	0.17	41,313.71
10	Furnishing and placing rubber waterstops	86.34	lin. ft.	3.26	281.47
11	Furnishing and placing 1/2-inch elastic filler material	145.34	sq. ft.	1.81	263.07
12	Furnishing and erecting structural carbon steel for skewback anchor rod frames	47,953.0	lbs.	0.53	25,415.09
13	Furnishing and erecting structural carbon steel exclusive of arch	2,776,346.67	lbs.	0.44	1,221,592.53
14	Furnishing and erecting structural carbon steel in arch	1,770,117.56	lbs.	0.50	885,058.78
15	Furnishing and erecting structural alloy steel in arch	3,175,363.83	lbs.	0.51	1,619,435.55
16	Furnishing and erecting cast steel	31,261.53	lbs.	2.46	76,903.36

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
17	Furnishing and installing miscellaneous metal-work	36,837.43	lbs.	$ 1.23	$ 45,310.04
18	Furnishing and erecting metal railing on bridge	106,688.79	lbs.	0.80	85,351.03
19	Furnishing metal railing for abutments	3,262.23	lbs.	0.62	2,022.58
20	Furnishing and installing roadway lighting system	100%	Lump sum	5,098.00	5,098.00
	Total—Original Contract				$4,472,736.66
	Order for Changes No. 1				
(a)	Excavation and disposing of over-hanging rock above skewback No. 2	9,223.38	cu. yd.	$ 10.00	$ 92,233.80
	Order for Changes No. 2				
(a)	Placing concrete in skewback No. 2	886.1	cu. yd.	$ 76.00	$ 67,343.60
	Order for Changes No. 3				
(a)	Adjustment for the change in painting requirements for the structural steel	3,949.9	Ton	$ –0.30	$ –1,184.97
(b)	Adjustment for the change in requirements for elastic filler material	100%	Lump sum	153.00	153.00
(c)	Drilling holes and furnishing, installing, pre-stressing, and grout-ing 1-1/2-inch steel bars	600.0	lin. ft.	15.50	9,300.00

Pay item	Property and pay item description	Quantity		Labor and materials by contractor	
		Amount	Unit	Unit cost	Total cost
(d)	Furnishing and placing concrete and wire mesh	100%	Lump sum	$ 1,250.00	$ 1,250.00
	Total—Order for Changes No. 3				$ 9,518.03
	Order for Changes No. 4				
(a)	Furnishing and installing damping cables and burning holes	100%	Lump sum	16,665.48	16,665.48
	Total—Order for Changes				185,760.91
	Total—Original Contract				4,472,736.66
	Total—Gross Earnings				$4,658,497.57

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APPENDIX F

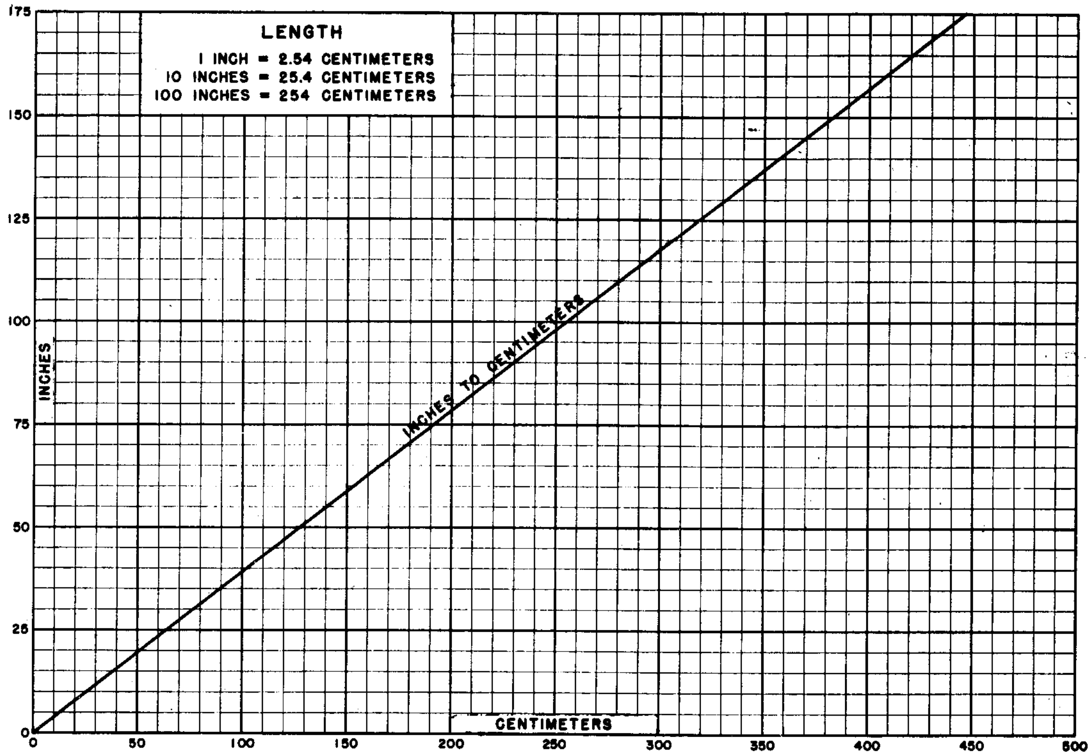
Conversion factors--English to metric system of measurement:

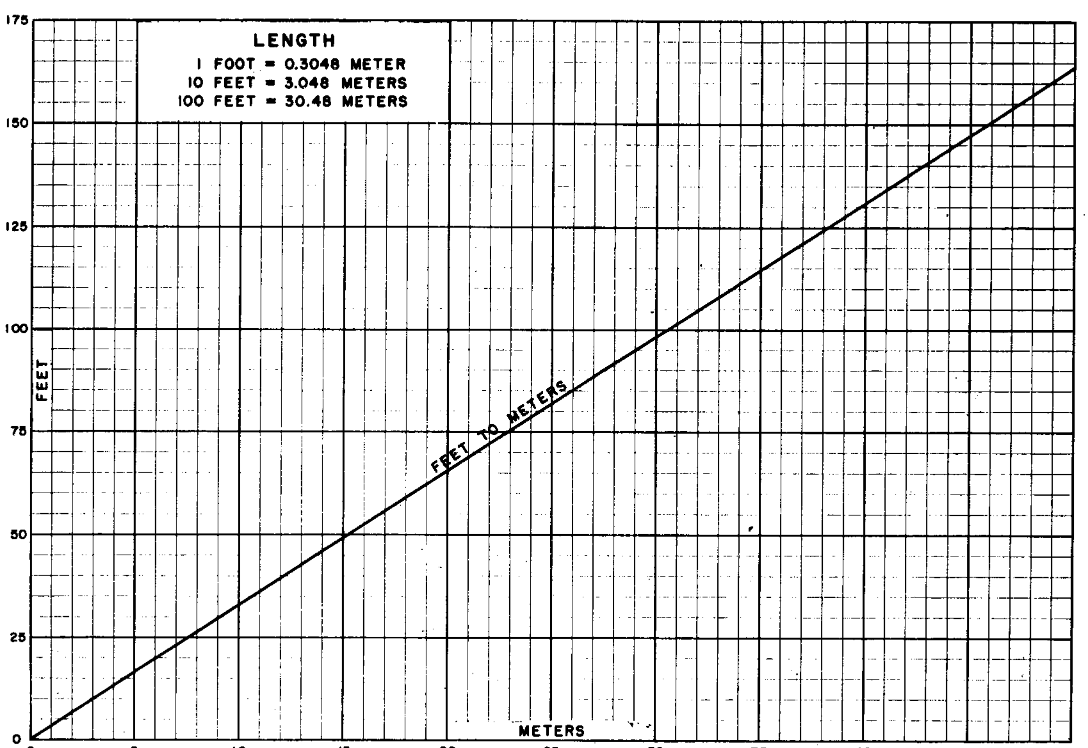
Quantity	English unit	Multiply by	To get metric equivalent
Length	inches	2.54*	centimeters
	feet	30.48*	centimeters
		0.3048*	meters
		0.0003048	kilometers
	yards	0.9144*	meters
	miles	1,609.3	meters
		1.6093	kilometers
Area	square inches	6.4516*	square centimeters
	square feet	929.03	square centimeters
	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.	liters
	cubic inches	16.387	cubic centimeters
	cubic feet	0.028317	cubic meters
	cubic yards	0.76455	cubic meters
		764.55	liters
Velocity	feet/second	0.3048*	meters/second
	miles/hour	1.6093	kilometers/hour
Acceleration	feet/second x second	0.3048*	meters/second x second
Discharge	cubic feet/second or second-feet	0.028317	cubic meters/second
Weight	pounds	0.45359	kilograms
	tons (2,000 pounds)	0.90718	tons (metric)

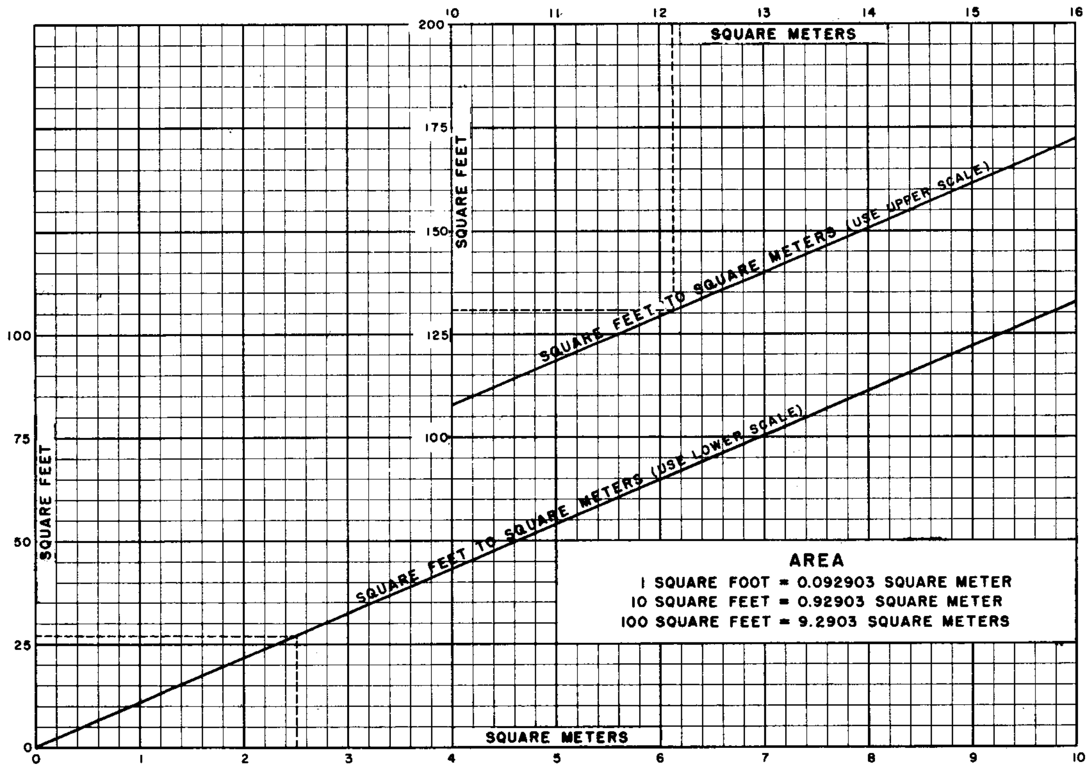
*Exact value.

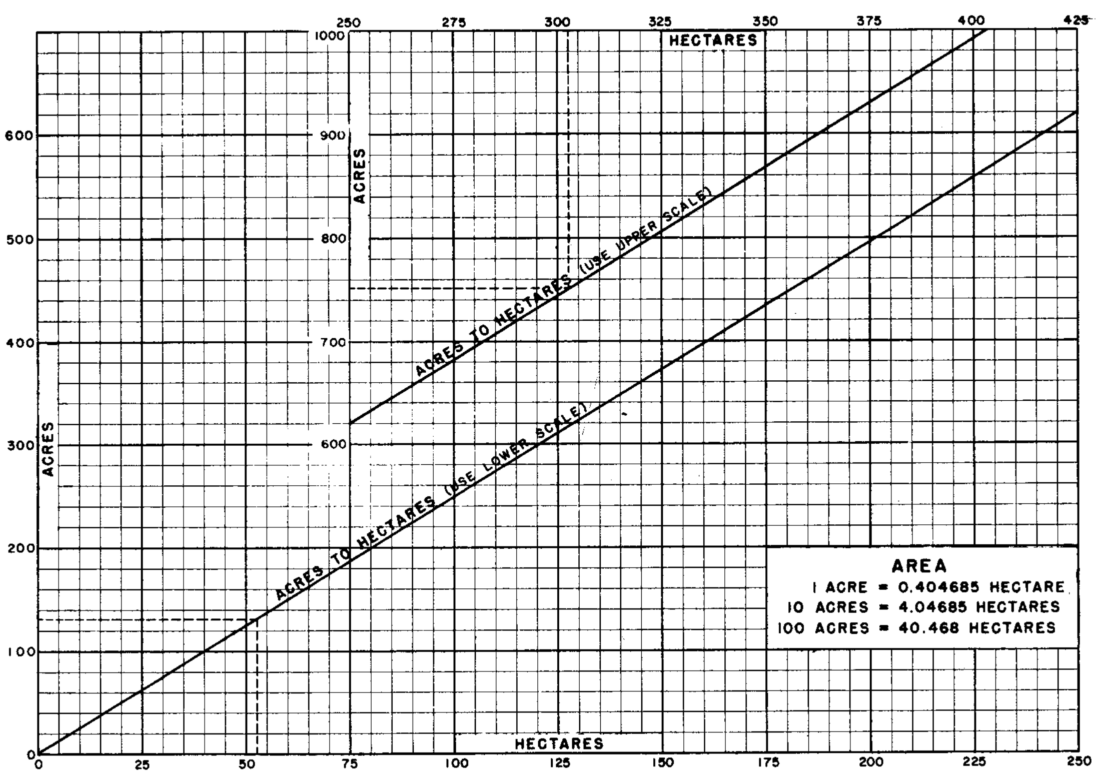
APPENDIX G

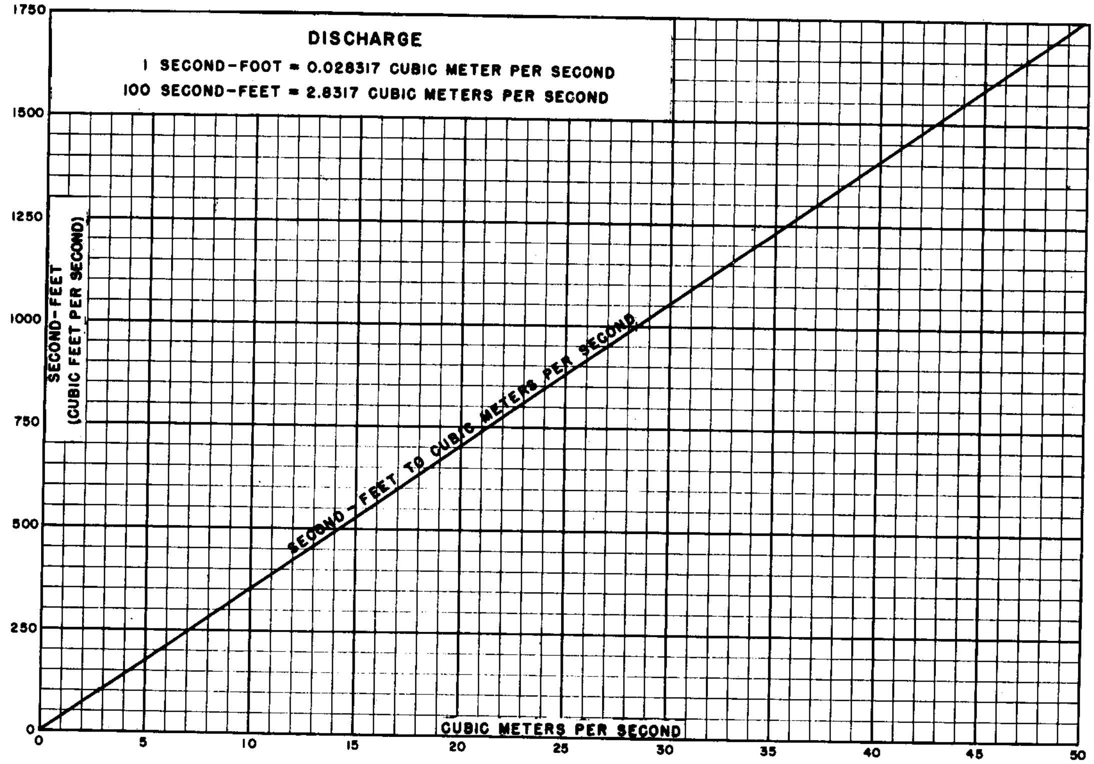
Charts for conversion of English to metric system of measurement:

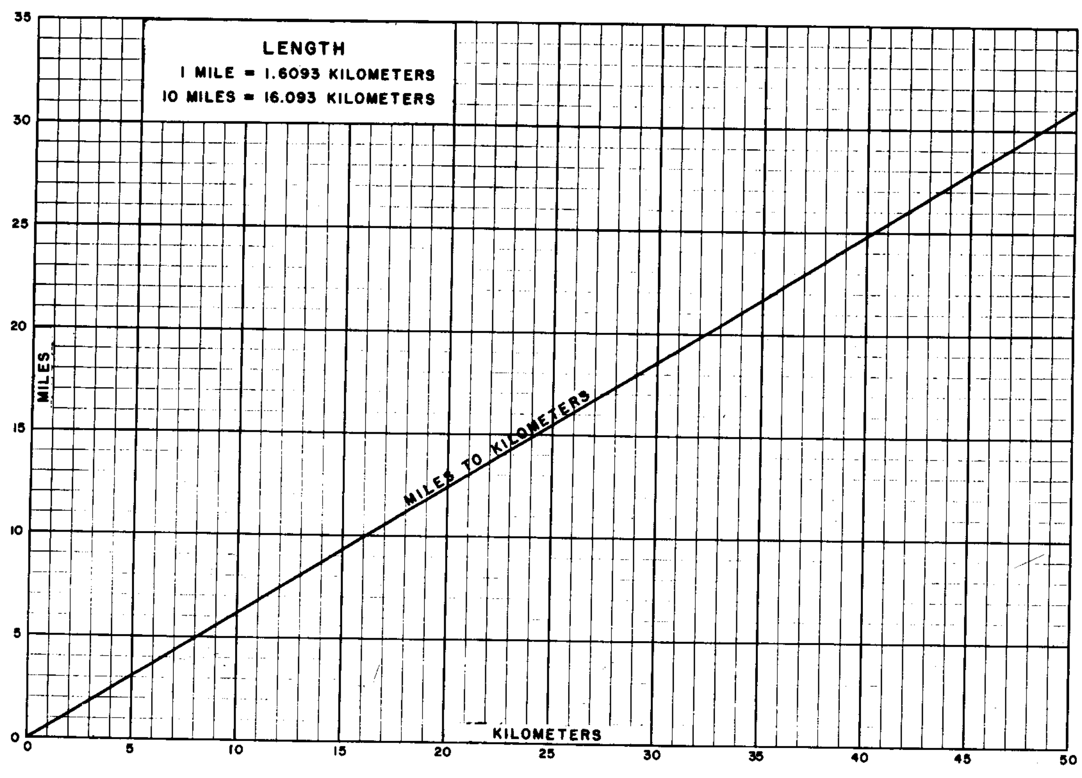





SQUARE METERS
SQUARE FEET
SQUARE FEET TO SQUARE METERS (USE UPPER SCALE)
SQUARE FEET TO SQUARE METERS (USE LOWER SCALE)
AREA
1 SQUARE FOOT = 0.092903 SQUARE METER
10 SQUARE FEET = 0.92903 SQUARE METER
100 SQUARE FEET = 9.2903 SQUARE METERS


HECTARES
ACRES
ACRES TO HECTARES (USE UPPER SCALE)
ACRES TO HECTARES (USE LOWER SCALE)
AREA
1 ACRE = 0.404685 HECTARE
10 ACRES = 4.04685 HECTARES
100 ACRES = 40.468 HECTARES


DISCHARGE
1 SECOND-FOOT = 0.028317 CUBIC METER PER SECOND
100 SECOND-FEET = 2.8317 CUBIC METERS PER SECOND
SECOND-FEET (CUBIC FEET PER SECOND)
SECOND-FEET TO CUBIC METERS PER SECOND
CUBIC METERS PER SECOND
0
250
500
750
1000
1250
1500
1750
5
10
15
20
25
30
35
40
45
50


LENGTH
1 MILE = 1.6093 KILOMETERS
10 MILES = 16.093 KILOMETERS
MILES
MILES TO KILOMETERS
KILOMETERS
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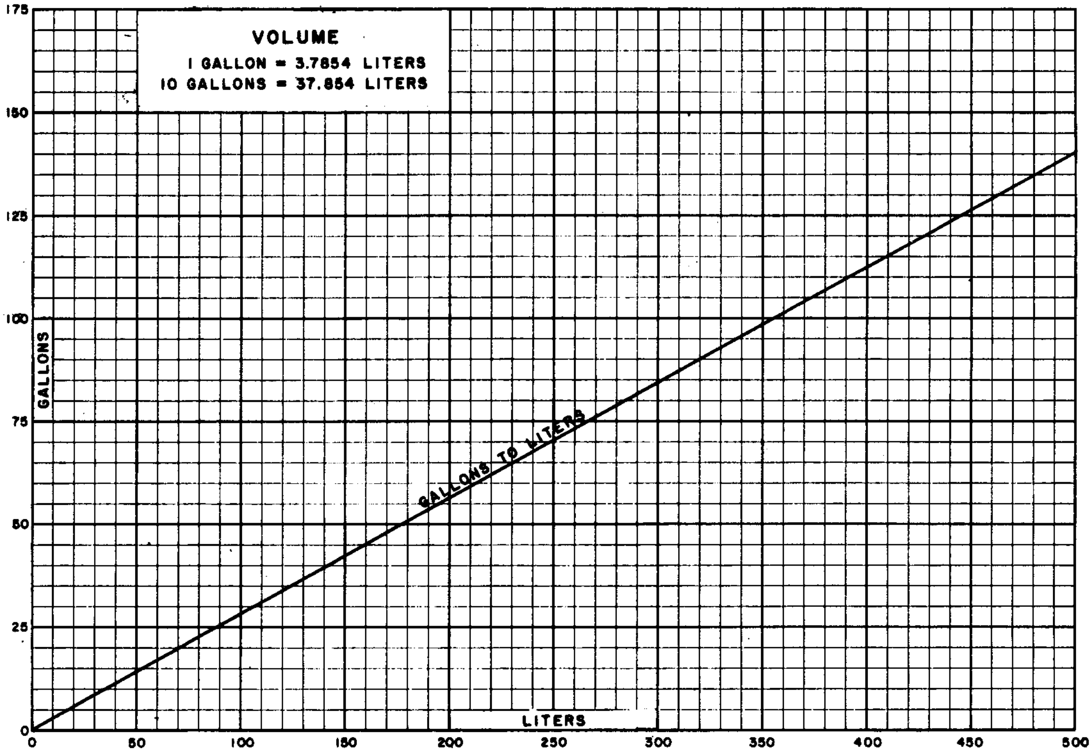
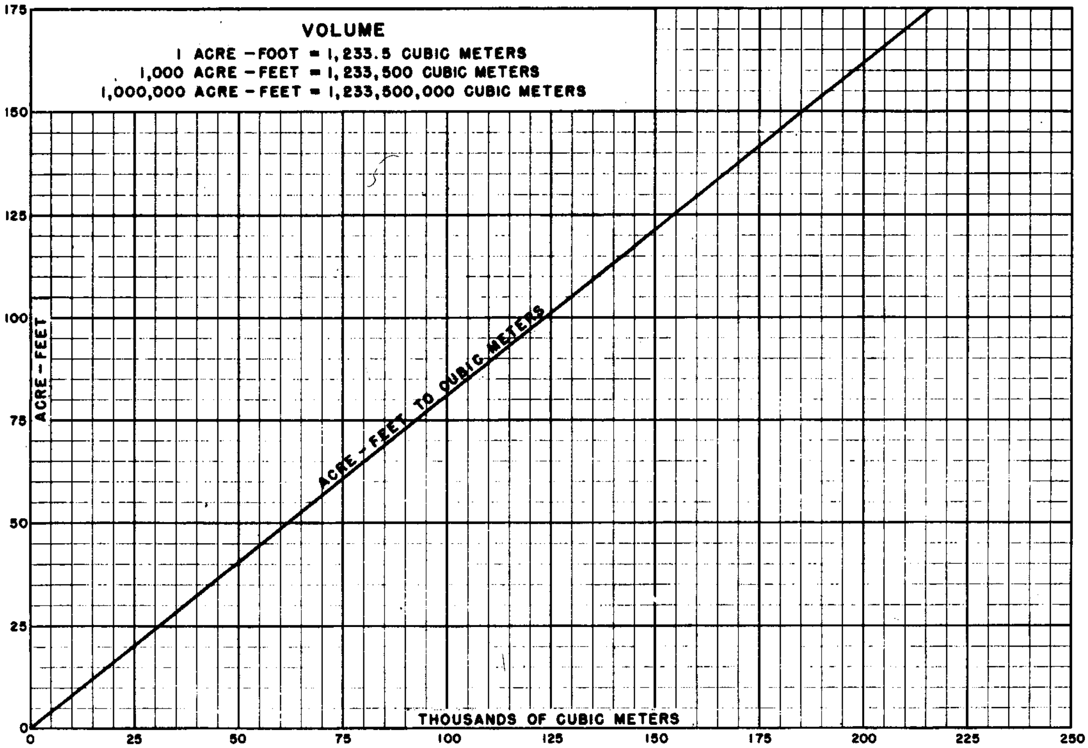
5
10
15
20
25
30
35
40
45
50


VOLUME
1 GALLON = 3.7854 LITERS
10 GALLONS = 37.854 LITERS
GALLONS
LITERS
GALLONS TO LITERS
0
25
50
75
100
125
150
175
0
50
100
150
200
250
300
350
400
450
500


VOLUME
1 ACRE-FOOT = 1,233.5 CUBIC METERS
1,000 ACRE-FEET = 1,233,500 CUBIC METERS
1,000,000 ACRE-FEET = 1,233,500,000 CUBIC METERS
ACRE-FEET
THOUSANDS OF CUBIC METERS
ACRE-FEET TO CUBIC METERS
0
25
50
75
100
125
150
175
0
25
50
75
100
125
150
175
200
225
250

APPENDIX H

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