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 1,560 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 alignment 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 alignment 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) Division Tunnels.—Division 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.

Figure 25.—Left spillway—Plan and sections.
Figure 26.—Damsite diversion requirements hydrographs—Snowmelt season. From drawing No. 557-D-3220.
(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 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. 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 at 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.

<table>
<thead>
<tr>
<th>Material</th>
<th>Type</th>
<th>Percent of yield point</th>
<th>Percent of ultimate tensile strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>Rolled or forged</td>
<td>40</td>
<td>25</td>
</tr>
<tr>
<td>Bolt steel</td>
<td>Rolled or forged</td>
<td>25</td>
<td>16.5</td>
</tr>
<tr>
<td>Cast steel</td>
<td>Cast</td>
<td>33</td>
<td>20</td>
</tr>
<tr>
<td>Brass or bronze</td>
<td>Rolled or cast</td>
<td>33</td>
<td>16.5</td>
</tr>
</tbody>
</table>

(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

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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
calculate 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
dam site 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 damsites 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 powerhouse. 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 powerhouse 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 powerhouse.

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. The base of the maximum section was assumed to be at elevation 3040. Estimated costs of a dam and powerhouse 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.
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 cantilever stresses.
ARCH AND CANTILEVER STRESSES

<table>
<thead>
<tr>
<th>ELEVATION</th>
<th>ARCH STRESSES</th>
<th>CROWN STRESSES</th>
<th>CANT. STRESSES</th>
</tr>
</thead>
<tbody>
<tr>
<td>3715 U.S.</td>
<td>+413</td>
<td>+631</td>
<td>0</td>
</tr>
<tr>
<td>D.S.</td>
<td>+692</td>
<td>+405</td>
<td>0</td>
</tr>
<tr>
<td>3665 U.S.</td>
<td>+273</td>
<td>+770</td>
<td>+83</td>
</tr>
<tr>
<td>D.S.</td>
<td>+664</td>
<td>+410</td>
<td>+44</td>
</tr>
<tr>
<td>3550 U.S.</td>
<td>+160</td>
<td>+698</td>
<td>+221</td>
</tr>
<tr>
<td>D.S.</td>
<td>+506</td>
<td>+211</td>
<td>+33</td>
</tr>
<tr>
<td>3450 U.S.</td>
<td>+229</td>
<td>+997</td>
<td>+281</td>
</tr>
<tr>
<td>D.S.</td>
<td>+704</td>
<td>+254</td>
<td>+111</td>
</tr>
<tr>
<td>3350 U.S.</td>
<td>+223</td>
<td>+057</td>
<td>+352</td>
</tr>
<tr>
<td>D.S.</td>
<td>+774</td>
<td>+244</td>
<td>+180</td>
</tr>
<tr>
<td>3250 U.S.</td>
<td>+173</td>
<td>+827</td>
<td>+416</td>
</tr>
<tr>
<td>D.S.</td>
<td>+549</td>
<td>-38</td>
<td>+251</td>
</tr>
<tr>
<td>3150 U.S.</td>
<td>+161</td>
<td>+727</td>
<td>+430</td>
</tr>
<tr>
<td>D.S.</td>
<td>+508</td>
<td>+59</td>
<td>+378</td>
</tr>
<tr>
<td>3010 U.S.</td>
<td>+108</td>
<td></td>
<td>+394</td>
</tr>
<tr>
<td>D.S.</td>
<td></td>
<td></td>
<td>+632</td>
</tr>
</tbody>
</table>

CONSTRUCTION PROGRAM:
(l) 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. Construction 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. 3150, and earthquake effects included. All stresses are parallel to faces and are in pounds per square inch.
For other assumptions, see drawing No. 557-DG-94.

Figure 36.—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
NOTE: STRESSES ARE BASED ON CROWN ADJUSTMENT

<table>
<thead>
<tr>
<th>Elevation (ft)</th>
<th>Arch Stresses (kips)</th>
<th>Crown Cant (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>1100</td>
<td>900</td>
</tr>
<tr>
<td>200</td>
<td>1200</td>
<td>1000</td>
</tr>
<tr>
<td>300</td>
<td>1300</td>
<td>1100</td>
</tr>
<tr>
<td>400</td>
<td>1400</td>
<td>1200</td>
</tr>
<tr>
<td>500</td>
<td>1500</td>
<td>1300</td>
</tr>
</tbody>
</table>

Figure 37.—Dam design A-B (specifications design)—Plan and maximum section.
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
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 raised to elevation 3700, and joint ungrouted.
2.Concrete placed to elevation 3480, reservoir water surface raised to elevation 3700, and joint grouted.
3. Concrete placed to elevation 3480, reservoir water surface raised to elevation 3700, and joint grouted.

Radial adjustment only.

Crown of arch
Intermediate arch points

All arches analyzed as symmetrical with symmetrical loads.
All 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.

ARCH STRESSES  CANTILEVER STRESSES
PROFILE ON AXIS LOOKING DOWNSTREAM (DEVELOPED)

SYMBOLS

(+) indicates downstream shear
(-) indicates upstream shear

(+) indicates downstream shear
(-) indicates upstream shear

(+) indicates downstream shear
(-) indicates upstream shear

(+) indicates downstream shear
(-) indicates upstream shear

January 1958

Figure 40.—Dam design A-19 (study 1), complete trial-load analysis—Arch and cantilever stresses.
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     | b     | c     | 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 46.—Dam study A-22c (final), complete trial-load analysis—Arch stresses.
ARCH STRESSES

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 STAYDIA A-22d (MODIFIED GROUTING PROGRAM)

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.

M indicates downstream shear.

All stresses are in pounds per square inch.

(1) Compression, (2) Tension

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.
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.
Figure 58.—Dam study A-22d (final), complete trial-load analysis—Principal stresses at upstream face.
PRINCIPAL STRESSES - DOWNSTREAM FACE
PROFILE ON AXIS LOOKING DOWNSTREAM (DEVELOPED)

NOTES

\( \theta \) - Angle first principal stress (\( \sigma_1 \)) makes with the vertical,
one positive angle measured in a clockwise direction on the
left side of the dam, and in a counterclockwise direction
on the right side of dam.
\( \sigma_1 \) - First principal stress.
\( \sigma_2 \) - Second principal stress.
\( \tau_{max} \) - Maximum horizontal shear stress at rock abutment planes.
[ ] Indicates downstream shear.

Compression ---- Tension

For constants, assumptions and loading conditions see
Drawing 537-DG-161.

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 $50^\circ$ 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 $35^\circ$ F. This would be possible only if the maximum placing temperature was held to $50^\circ$ F. or less. Without this restriction, placing temperatures could be as high as $80^\circ$ F. in the summer with a resulting temperature drop of about $70^\circ$ 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 indicated that the temperature rise could be limited to $25^\circ$ F. for blocks adjacent to rock. This meant that the maximum temperature drop would be limited to $35^\circ$ F. for those areas where restraint was high.

The specifications provided 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 $40^\circ$ F. in the lower part of the dam, ranging from $40^\circ$ F. at elevation 3450 to $50^\circ$ 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 $40^\circ$ F. up to elevation 3300, $45^\circ$ F. between elevations 3300 and 3360, $50^\circ$ F. between elevations 3360 and 3600, and $55^\circ$ 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 $50^\circ$ 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 $60^\circ$ 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

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3Concrete 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.)
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 powerhouse 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 powerhouse with the adits in blocks 7 and 18. It provides access to the sump pump
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 3687.50 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. 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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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

STRENGTH CLASSIFICATION, LB. PER SQ. INCH AT 28 DAYS: $f'_c$, 2000, 2500, 3000, 3750

FLEXURE: $f_c$, $f_t$, lb. per sq. inch
- Extreme fiber stress in compression: $0.45 f'_c + f_t$, 900, 1250, 1500, 1688
- 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: $0.03 f'_c (\text{max. 90}) = V_c$, 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): $0.12 f'_c (\text{max. 360}) = V_c$, 240, 300, 360, 360
- Footings: $0.03 f'_c (\text{max. 75}) = V_c$, 60, 75, 75, 75

BOND: $u$, lb. per sq. inch of surface area of bar
- Top bars: $0.07 f'_c (\text{max. 245}) + u$, 140, 175, 210, 245
- In two-way footings (except top bars): $0.08 f'_c (\text{max. 260}) + u$, 160, 200, 240, 280
- All others: $0.10 f'_c (\text{max. 350}) + u$, 200, 250, 300, 350

BEARING: $f_b$, lb. per sq. inch
- Full area loaded: $0.25 f'_c + f_b$, 500, 625, 750, 938
- Load on partial area, maximum: $0.375 f'_c + f_b$, 750, 938, 1125, 1405

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

NOMINAL STRESSES FOR 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

NOTE
(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:

<table>
<thead>
<tr>
<th>Floor</th>
<th>Elevation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elevator machinery</td>
<td>3739.65</td>
</tr>
<tr>
<td>Men's restrooms</td>
<td>3728.40</td>
</tr>
<tr>
<td>Floatwell room (block 8 only)</td>
<td>3728.40</td>
</tr>
<tr>
<td>Main lobby</td>
<td>3715.90</td>
</tr>
<tr>
<td>Women's restrooms</td>
<td>3703.40</td>
</tr>
<tr>
<td>Ventilating equipment</td>
<td>3692.15</td>
</tr>
<tr>
<td>Electrical equipment</td>
<td>3680.90</td>
</tr>
</tbody>
</table>

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 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 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 85.—Layout of structural behavior instruments in dam—Blocks 2, 3, 4, 6, and 7.
Figure 96.—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.
Figure 89.—Dam instrument terminations.
Figure 90.—Layout of structural behavior instruments in dam—Reading station and conduit details.
NOTES
See drawing 40-D-5095 for installation details of vertical stressmeters. Horizontal stressmeters to be placed with diaphragms in vertical planes. No aggregate greater than $\frac{1}{4}$" to be in contact with diaphragms.

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 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
Figura 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 x 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.
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 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. 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, PDO, 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 pipe 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 side 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 side 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.