DESIGN FEATURES OF GLEN CANYON DAM

By

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SYNOPSIS

The designs for the various features of Glen Canyon Dam are described in this paper. The various topics discussed include: geology of the foundation, stress studies of the dam, structural behavior instrumentation, foundation treatment, temperature control of the mass concrete, design of the penstocks, river outlets, and spillway and diversion and care of the river.

INTRODUCTION

Glen Canyon Dam is the principal structure of the Bureau of Reclamation's Colorado River Storage Project in the Upper Colorado River Basin. The dam is under construction on the Colorado River in northcentral Arizona, about 15 miles upstream from Lees Ferry and 12 river miles downstream from the Arizona-Utah state line. The dam is to be a concrete-arch structure, 710 feet high above foundation and will have a volume of 4,865,000 cubic yards. When it is completed early in 1964, it will be the second highest dam in the

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Western Hemisphere, exceeded in height only by the 726-foot-high Hoover Dam. The location of Glen Canyon Dam and the Glen Canyon Powerplant, under construction with the dam, is shown in Figure 1.

The reservoir to be impounded by the dam, named Lake Powell in honor of Major John Wesley Powell, renowned explorer of the Colorado River and its tributaries, will have a total storage capacity of 28,040,000 acre-feet and will extend 186 miles up the Colorado River and 71 miles upstream on the San Juan River. The reservoir will be a major storage feature to provide the longtime regulatory storage needed to permit the states of the Upper Colorado River Basin to utilize their apportioned water and still meet their flow obligations at Lee Ferry, Arizona (the dividing point between the Upper and Lower Basins), under the terms of the 1922 Compact of the Colorado River. The 900,000-kilowatt Glen Canyon Powerplant will generate the principal portion of the electrical energy needed in the Upper Basin. Surplus revenue from sale of this energy will assist irrigators in the Upper Basin to repay costs of constructing the participating projects which were authorized by the Congress in 1956 to be developed with the Colorado River Storage Project.

Construction of Glen Canyon Dam and Powerplant began in April 1957 under a $107,955,522 contract awarded to the Merritt-Chapman and Scott Corporation of New York. After 4 years of construction, the river has been diverted around the damsite, excavation for the dam and powerplant has been completed, and more than 700,000 cubic yards of concrete have been placed in the dam and powerplant. First power from
LOCATION MAP
GLEN CANYON DAM AND POWER PLANT

FIGURE 1
the powerplant is scheduled to go on the line by June 1964. The layout of the dam and powerplant and a view of present construction at the damsite are shown in Figures 2 and 3.

GEOLoGY OF THE FOUNDATION

The foundation for the damsite lies wholly in the Jurassic Navajo sandstone formation. This formation is more than 1,300 feet thick with its base lying several hundred feet below river level. The deepest hole drilled in the floor of the canyon reached a depth of 434 feet and was still in sandstone at its bottom elevation. The formation lies in a nearly horizontal position with a slight dip in a direction upstream and toward the left abutment. Wide-spaced near-vertical joints run generally parallel and normal to the river axis.

The Navajo sandstone is buff to reddish, medium-to-fine grained and moderately hard to soft. It is massive with pronounced cross-bedding. It is composed essentially of quartz grains with a minor amount of feldspar, and is poorly to moderately well cemented principally by secondary quartz, chalcedony, and to a much lesser extent by calcite and hematite.

Foundation investigations for Glen Canyon Dam were initiated in October 1947, by drilling of holes, the excavation of horizontal drifts for bearing tests, and field grouting experiments. Subsequent field investigations conducted in 1949 and in 1956 consisted of three principal phases: Geologic mapping of the immediate dam area, driving another drift and drilling additional core holes ranging from vertical holes 500 feet deep to angle holes in each abutment drilled approximately
GLEN CANYON DAM AND POWER PLANT
AREA PLAN
FIGURE 2
parallel to the direction of the maximum principal stresses. Geophysical seismic measurements in the bore holes and on the rock surface to secure an approximation of the in-situ elastic properties of abutment rock was also made.

Concurrently with the field work, very extensive laboratory studies were performed on cores from the foundation to determine the physical properties of the Navajo sandstone for use in the designs. From the laboratory tests, it was found that the compressive strength of the sandstone was highest in a direction normal to the stratification of the beds. The tests also disclosed that the strength of dry sandstone was considerably greater than when the specimen was saturated.

The cores were divided into four main groups according to strength. The average compressive strength and relative amount of each strength group for saturated cores are:

<table>
<thead>
<tr>
<th>Group</th>
<th>Compressive strength (psi)</th>
<th>Amount percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highest strength</td>
<td>6,250</td>
<td>4</td>
</tr>
<tr>
<td>Medium strength</td>
<td>3,870</td>
<td>71</td>
</tr>
<tr>
<td>Medium low strength</td>
<td>1,770</td>
<td>17</td>
</tr>
<tr>
<td>Lowest strength</td>
<td>770</td>
<td>8</td>
</tr>
<tr>
<td>Weighted strength</td>
<td>3,360</td>
<td>100</td>
</tr>
</tbody>
</table>

The distribution of strength groups in the abutment and foundation permitted the damsite to be divided into three zones: the upper zone, the middle zone, and the lower zone. The locations of these zones and
VIEW OF CONSTRUCTION PROGRESS

FIGURE 3
the percent distribution of strength groups by zones are shown in Figure 4. The distribution is such that the average strength of the foundation increases with depth below the canyon rims. The following tabulation gives the average unconfined (cores) compressive strength weighted according to percentage of each strength group in the zone:

- Upper zone: \(2,870 \text{ psi}\)
- Middle zone: \(3,640 \text{ psi}\)
- Lower zone: \(3,830 \text{ psi}\)

Triaxial shear tests indicate that the strength of the rock in place at the site will have higher values than those resulting from unconfined laboratory tests. It was found that a lateral restraint of only 5 percent would increase the axial strength about 40 percent. This assumption indicates that the rock in situ in the medium strength zone would have a strength of \(5,300 \text{ psi}\).

Unlike the compressive strength, the secant modulus of elasticity of the rock decreases with depth. The average moduli weighted according to the percentage of each strength group in the zone and tabulated for various stresses and load cycles are:

<table>
<thead>
<tr>
<th>Stress range</th>
<th>0-200 psi</th>
<th>0-400 psi</th>
<th>0-600 psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>load cycle</td>
<td>First</td>
<td>Second</td>
<td>First</td>
</tr>
<tr>
<td>Upper zone</td>
<td>0.49</td>
<td>0.54</td>
<td>0.58</td>
</tr>
<tr>
<td>Middle zone</td>
<td>0.43</td>
<td>0.49</td>
<td>0.50</td>
</tr>
<tr>
<td>Lower zone</td>
<td>0.36</td>
<td>0.41</td>
<td>0.40</td>
</tr>
</tbody>
</table>
FOUNDATION ROCK INVESTIGATION
PERCENT DISTRIBUTION OF STRENGTH GROUPS BY ZONES

CIRCLE SHOW PERCENTAGE OF STRENGTH GROUPS IN EACH ZONE

**AVG. COMP. STR.**
- High Strength: 6250 P.S.I.
- Medium Strength: 3870 P.S.I.
- Medium Low Strength: 1770 P.S.I.
- Low Strength: 770 P.S.I.

**FIGURE 4**
The specific gravity of the rock increased with strength, ranging from 1.92 to 2.12. Absorption and porosity decreased with strength. The absorption varied from 14.6 to 9.5 percent by weight. The percent porosity by volume varied from 28.1 to 20.1.

After the first load cycle, the permanent set of the rock, that is the ratio of unrecovered strain to applied strain, was about 12 percent for medium strength rock. Thereafter, in successive load cycles, the rock exhibited more normal elastic behavior.

Completion of the rock excavation for the dam and powerplant has revealed that the massive sandstone abutments and foundation are generally of highly satisfactory quality and freer from weak zones and joints than is commonly expected in the average dam foundation. A few localized zones of sandstone less well cemented than the higher quality rock which surrounds them have been exposed. Additional excavation, principally in the downstream portions of the keyways of the dam, has been accomplished to reshape portions of the keyways where less competent sandstone existed and in some cases to secure an adequate rock shoulder. The excavated rock face of the keyways against which the dam concrete will rest are remarkably free of joints and present an almost monolithic appearance.

Vertical to steeply dipping joints roughly parallel to the canyon wall are present and exist principally in the rock adjacent to the surface of the canyon walls. These joints are chiefly of the stress-relief type and are most noticeable in the excavation faces of the outlet channels for the diversion tunnels, the ends of the powerplant and
downstream from the right abutment keyway. In the case of joints parallel to the canyon face, rock bolting, using principally 6- and 8-foot, and some 20-foot lengths, have been employed extensively and with a high degree of success.

The adjustment of excavation to conform to local variations in physical conditions, as exposed by the excavation, is a normal procedure in dam construction. The adjustments at Glen Canyon are by no means extraordinary in scope, and are in fact less than normally expected.

LAYOUT AND DESIGN STUDIES

An arch dam was selected for the Glen Canyon site because:

(1) the canyon configuration provides suitable proportions to transfer loads effectively by arch action to the abutments; (2) an arch dam requires less materials than a gravity dam; (3) a greater factor of safety may be obtained with more economy than is possible with a gravity dam.

The layout of the dam which was adopted for construction is the result of numerous design studies and extensive investigation. Twenty different design layouts were made and more than fifty analytical studies completed before the final design was approved. The large number of studies, which included crown cantilever adjustments, radial adjustments, and complete adjustments, could not have been accomplished without the use of programs developed by the Bureau, to obtain data for the trial-load studies by means of an IBM 650 digital electronic computer. This computer is located in the office of the Assistant Commissioner and

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Chief Engineer of the Bureau of Reclamation in Denver. In addition, a number of photoelastic studies was performed on arch and cantilever sections of the dam to study stress distribution in the abutments where it was necessary to step the excavation because of local foundation conditions.

Two structural models were constructed of the adopted design primarily to provide a check on the computed deformations of the foundation rock. One had a scale of 1:720 and the other a scale of 1:240. The stresses determined from these models do not represent the actual conditions in the dam, since the loading is equivalent only to hydrostatic pressure and does not include temperature changes, weight of the dam, earthquake, or effects of the construction and grouting program.

A simplified form was selected for the layout of Glen Canyon Dam to minimize construction costs. The radius at the axis of the dam is 900 feet; the upstream face, having a radius of 955 feet, is vertical below elevation 3300 and is concentric with 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 1555.68 feet. All horizontal arcs 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. The layout of the dam is shown in Figure 5.

The properties of the foundation rock require that the load be transferred to the abutments in direct compression by arch action and, consequently, substantially reducing the transfer of load by shear friction. The elastic properties of the rock are such that it deflects five times as much as concrete under the same load. To keep abutment stresses within (or near) the allowable limits, it was necessary to increase the contact area of the dam with the abutment rock as much as economically feasible. The wide contact area results in an increased stiffness against rotation, thereby reducing somewhat the downstream deformation of the structure.

The dam is designed for a computed minimum factor of safety of about 5. This means that, based upon the crushing strength of rock or concrete, the actual working stresses, which will be imposed by the maximum loading on the dam, will be about one-fifth of the stresses which could cause failure.

The design study of the adopted layout for Glen Canyon Dam is based on the following loading conditions and assumptions:

1. Top of dam, elevation 3715 (base elevation 3005), or a structural height of 710 feet.

2. Normal reservoir water surface, elevation 3700 (minimum tail water surface, elevation 3142), or a hydraulic head of 558 feet.

3. Temperatures used in the analysis are changes between average arch temperature at the time of joint closure and minimum operating temperatures. Operating temperatures are assumed to vary linearly from upstream to downstream faces.
4. Earthquake was assumed to move the dam upstream and downstream horizontally in a direction parallel to the plane of centers with an acceleration of one-tenth gravity and a period of vibration of 1 second. The increased water pressure was assumed to act equally on all cantilevers. Effects of vertical acceleration were not included.

5. Modulus of elasticity of concrete, 3,000,000 pounds per square inch.

6. Modulus of elasticity of foundation rock, 600,000 pounds per square inch.

7. Effects of silt and uplift were neglected.

8. Poisson's ratio of concrete, 0.20; of foundation rock 0.08.

9. Unit weight of concrete, 150 pounds per cubic foot.

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

11. The effects of the construction and grouting program were included as follows:

   a. Concrete placed to elevation 3540; reservoir water surface, elevation 3240; joints ungrouted; no arch action.

   b. Concrete cooled to 40°F from the base of the dam to elevation 3480; varying from 40°F at that elevation to 43°F at elevation 3540; dam grouted to elevation 3540; concrete placed to elevation 3715; reservoir water surface raised to elevation 3490. Arch action was assumed below elevation 3540 in this analysis and loads above this elevation are carried by cantilever actions only.
Reference plane for dam. All fillet radii = 350'.

Intrados center below El. 3165. Loci of fillet center.

Contraction joints.

Assumed excavation contours.

Top of dam - El. 3715.

DAM LAYOUT

SCALE OF FEET

100 0 100 200

FIGURE 5
c. Concrete above elevation 3540 cooled to temperatures varying from 43° F at elevation 3540 to 50° F at elevation 3715; reservoir water surface raised to elevation 3700; effects of earthquake, and saturated backfill at the downstream face included. In this analysis, arch action is assumed throughout the dam.

Total stresses were computed by superpositions of forces from these three stages.

Allowable maximum compressive stresses are as follows:

1. Maximum computed stresses at arch abutments, 500 pounds per square inch for water load alone and 600 pounds per square inch for a combination of all loads, including earthquake, with reservoir at normal water surface. Increases above these limits may be tolerated in limited areas.

2. Maximum computed compressive stresses in concrete at points other than arch abutments, 1,000 pounds per square inch for a combination of all loads, including earthquake, with reservoir at normal water surface.

In accordance with the design criteria stated, stresses were computed for the dam. The maximum compressive stress computed at the arch abutments is 740 psi at the intrados, elevation 3715. Provisions will be made for special joint treatment in this area to redistribute the load and thus reduce the stress at the top of the dam. All other arch stresses at the abutment were found to be less than 600 psi except at the intrados at elevations 3400 and 3325, where the stresses were 611 psi and 657 psi, respectively.
The maximum cantilever stress computed at the base of the cantilever elements is 510 psi compression parallel to the downstream face of the crown cantilever. Stresses at the bases of all other cantilever elements are less than 500 psi. The maximum tensile computed in the cantilever elements is 91 psi on the downstream face at elevation 3625 near the abutment.

The maximum compressive principal stress computed at the upstream face occurs at elevation 3715 and has a value of 551 psi. The maximum compressive principal stress found on the downstream face at the abutment is 740 psi at elevation 3715. The compressive principal stresses at the abutments on the downstream face are 615 psi, 661 psi, and 601 psi at elevations 3400, 3325, and 3250, respectively. All other computed compressive principal stresses at the abutments are less than 600 psi. The maximum tensile principal stress computed for the abutments is 124 psi at the downstream face, elevation 3625.

STRUCTURAL BEHAVIOR INSTALLATION

The dam is extensively instrumented to determine from the several systems of measurement: (1) the conditions of stress and temperature in the structure; (2) the foundation deformation at the base and abutments; and, (3) the opening of the contraction joints. The deflection that occurs in arch and cantilever elements are charted by precise surveying methods. A developed elevation and a maximum section of the dam with locations of instruments are shown in Figure 6.
MAXIMUM SECTION
(8 BLOCK 12)

UPSTREAM ELEVATION
(DEVELOPED)
Showing general major instrument locations
FIGURE 6

KEY
- Stressmeter
- Group of 12 strainmeters
- Group of 3 strainmeters
- Pair of no-stress strainmeters
- Jointmeter
- Deformation meter

SCALE OF FEET
All instruments are of the elastic-wire type, except thermometers which are of the resistance-wire type. All instruments are embedded in the mass concrete of the dam and are connected by electrical cables to terminal boards located in the galleries of the dam where readings are made systematically using portable Wheatstone bridge test sets.

Strain meters, installed in clusters of 12 instruments each at several points on radial lines that define sections of arches and cantilevers in the dam, form the major system of instruments from which the 3-dimensional conditions of stress are determined. The lines of instruments are selected to generally furnish stress at locations of maximum stress indicated by the trial-load design analysis of the dam, as well as to furnish stress at a number of arch-cantilever intersections that are common to both methods of investigation. The radial lines of strain meter clusters are located near the base of the maximum dam section and in three arches uniformly spaced between the base of the dam and about the two-thirds elevation of the dam. In each arch the lines of instrument clusters are in the maximum section, near each abutment and, where applicable, at intermediate locations between the abutments and the maximum section. The lowest arch contains three lines of instruments; the second arch contains five lines of instruments; and the third arch contains seven lines of instruments. Two arches between the two-thirds elevation of the dam and the top of the dam are instrumented using series of stress meters installed on seven radial lines in each arch. These instruments determine stress in the direction of arch thrust only. Pairs of strain meters
installed under free-surface metal canister-like covers, one pair of meters with each elevation of instruments, detect possible autogeneous growth or shrinkage of the mass concrete.

At three widely separated locations near the top of the dam, trios of 3-dimensionally arranged strain meters are installed near the upstream and downstream faces of the dam to determine surface stress in the structure.

A layout of resistance thermometers is installed in the maximum dam section in a vertical grid pattern, having an approximate 40-foot by 50-foot spacing, forms the basic means of determining temperature in the section. The temperatures from the thermometers are augmented by temperatures from the strain meters and stress meters in the same section. Special-purpose installations of series of thermometers are made near one penstock at two elevations, and near the downstream face of the dam at three widely separated locations, each to determine localized temperature conditions.

In conjunction with the installations of strain meters and stress meters through the dam, jointmeters are placed on the radial contraction joints at the same elevations as the meter clusters. Where the dam's longitudinal joint crosses blocks containing strain meters or stress meters, jointmeters are installed on the longitudinal joint near its intersection with the radial joints. Additional jointmeters are installed on the longitudinal joint at intermediate elevations between the arches containing the strain meters, and stress meters.
Patterns of jointmeters are installed on each of the two radial joints nearest each dam abutment and in the upper 200 feet of the dam elevation. These instruments will be used to determine stability in the joints at the time of grouting when final joint closures are made.

The foundation of the dam is a deformable sandstone that when loaded exhibits an initial unrecoverable compressibility before assuming properties of elastic behavior. Accordingly, deformation meters, which in reality are long-range jointmeters having anchorages that are made approximately 20 feet beyond the foundation surface, are installed vertically on several radial lines at the base portion of the dam, and in horizontal and sloping patterns at several elevations on each abutment. These instruments will detect initial foundation deformation as load is applied to the sandstone, indicate when the phenomenon ceases, and then indicate elastic deformation of the foundation.

In numbers, the several systems of embedded instruments comprise 1,142 strain meters, 60 stress meters, 74 resistance thermometers, 264 jointmeters, and 118 deformation meters.

In addition to the embedded instrument installations, two systems of measurement employing refined methods of surveying are provided for determining the manner in which the dam deflects during periods of reservoir filling and reservoir operation.

One system comprises five plumb lines, each in a formed well extending from the top of the dam to a point near the foundation. The wells are located in the maximum section and at points approximately one-third and two-thirds the distances between the maximum section and the
Deflection measurements are made at the bottom end and at several elevations on each line from reading stations in the dam galleries using a micrometer and microscope apparatus.

The second system for determining deflection of the dam consists of a grid system of 68 targets placed on the downstream face of the dam, and 17 targets on the foundation along the abutments. Locations of the targets are charted periodically from primary theodolite stations on each abutment downstream of the dam and from secondary stations on the canyon rim, using precise triangulation surveying methods.

Hydrostatic uplift at the base of the dam will be measured at 41 locations by pipes connecting to wells at the concrete-rock contact plane and terminating in the dam galleries. The pipes are arranged in 7 lines, each line being made up of from 5 to 7 pipes.

A seismograph station located approximately 11 miles northwest of the damsite records earthquake shocks. Records from the station, in addition to indicating the magnitude of possible earthquake shocks in the vicinity of the structure, will serve to determine any possible change in local seismic activity that may occur in the area due to the increased weight of the reservoir on the earth's surface.

A program for investigation is being developed that, when in operation, will allow the determination of the actual conditions of stress, joint opening, foundation deformation and temperatures, at the various instrument locations to be possible within approximately 30 days after field readings are made. The solution of the problems will be possible through the expedient of automatic data processing using punched cards and high-speed digital computers.
OPENINGS IN THE DAM

To service the dam, a series of galleries and shafts have been formed in the dam. These openings are kept to a minimum. The systems for grouting contraction joints and cooling the concrete by embedded pipe coils are serviced from the downstream face of the dam to reduce the number of openings required. All the service galleries in the dam are 5 feet wide by 7 feet 6 inches high. A cross section of the system of galleries in the dam is shown in Figure 7. All galleries are located a minimum distance in feet equal to 10 percent of the hydraulic head at the gallery floor from the upstream face of the dam.

The foundation gallery follows the excavated foundation very closely by using inclined and spiral stairways. Minimum distance between the foundation rock and the floor of the gallery is 5 feet. The high-pressure grout curtain for the dam foundation as well as the drainage curtain is drilled from this gallery. A drainage gutter 12 inches wide with a minimum depth of 12 inches is provided in the floor to handle drainage from the foundation drainage and from the vertical-formed drains located near the upstream face of the dam. Another gallery for additional foundation drainage is provided about 130 feet downstream, and extends from elevation 3187.5 on the left abutment to the same elevation on the right abutment.

At elevation 3187.5 a pump chamber gallery is constructed longitudinally through the dam. The gallery provides access to the pump chamber, located in the center of the dam, where two pumps, each having a capacity
of 3,000 gallons per minute, are located. The pump chamber is connected to the pump sump located at the foundation gallery level by a vertical shaft. All drainage below the elevation of the pump chamber gallery is collected in the sump and pumped to the pump chamber level where it is discharged through a pipe by gravity to the downstream face of the dam. Drainage from this elevation to the filling line gallery at elevation 3480 is collected in the gallery floor gutters and flows by gravity into the discharge pipe for the sump pump.

A longitudinal adit is located in the left abutment extending from Blocks 3 through 8 to service the four ring-follower gates for the river outlets. A 5-foot by 11-foot vertical shaft located just downstream from the gate chamber in Block 6 provides a means for removing gate parts from the dam. Access manholes to the outlets are provided from the gate chambers.

The next longitudinal gallery in the dam is located at elevation 3480 and provides access to the filling line chambers. One filling line chamber with necessary valves is provided for each penstock. Access to each penstock is also made through access shafts from the filling line gallery. Drainage in the dam above this elevation is collected in the gallery floor gutters in this gallery and fed by gravity to pump sump discharge pipe.

A utility gallery which runs through the entire length of the dam is located close to the top of the dam at elevation 3697.5.

Two elevator shafts, 19 feet 6 inches by 9 feet 6 inches, are located in the dam; one in Block 8 and one in Block 17. These elevators
operate from the pump chamber gallery level to the top of the dam. Each shaft will have an elevator with a capacity for 50 persons. The elevators will be used to escort tourists to and from the powerplant from the roadway on the dam and to provide easy access to the various gallery levels when servicing the dam. A 7-foot 8-inch by 8-foot 3-inch adit to the downstream face of the dam is provided at elevation 3187.50 from each elevator shaft. Access to the powerplant from these adits is by covered walkways. Transformer chambers, and other utility adits are provided in the dam.

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 is based on photoelastic studies of standard size openings. Special photoelastic studies were made for the larger openings and of openings located near contraction joints where grout pressure in a joint would affect the opening.

FOUNDATION TREATMENT

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

The essential features of the foundation treatment for the dam will include: (1) Shallow holes drilled and grouted in the upstream part of the foundation area; (2) deep holes drilled and grouted from the
foundation gallery and abutment tunnels to establish a cutoff beneath the dam, and (3) drainage holes drilled from the foundation gallery, the downstream drainage gallery, and the abutment tunnels. A layout of the foundation grouting and drainage is shown in Figure 8.

The shallow grout holes, designated as "B" holes will be grouted at low pressures and will be drilled to depths of about 25 feet on 20-foot centers. They will be grouted to seal any near-surface cracks. This grouting has been done in the bottom of the foundation prior to placement of concrete. Because of the steepness of the canyon walls, "B" hole grouting above elevation 3110 plus or minus has been done through embedded pipes placed in the first lifts of concrete on the foundation.

The extent of this grouting will depend on conditions encountered as the work progresses. To date, four rows of holes have been drilled and grouted along the upstream side of the foundation area. "B" hole grouting has been completed to elevation 3110 and the average take per linear foot of hole has been 0.3 cubic foot.

Two water-bearing seams were discovered in the foundation on the right abutment at elevation 3115 and elevation 3070 and one on the left abutment at elevation 3080. The one on the left was treated by extending the "B" hole grouting downstream in this area an additional 60 feet. The two on the right were treated by excavating drifts into the abutment at the upstream side of the foundation area and backfilling with concrete to increase the path of percolation.
FOUNDATION GROUTING AND DRAINAGE PROGRAM

FIGURE 8
The lower drift was extended 160 feet and the upper drift 60 feet into the abutment. The extent of the drifts was determined by field inspection of the leakage in the water-bearing seams.

The main cutoff or grout curtain beneath the dam will be formed by drilling and grouting deep holes known as "A" holes from the foundation gallery and from the foundation tunnels excavated in both abutments at elevation 3630, 3480, and 3247.5. Grout pressures up to 500 psi may be used. These holes will be drilled through pipe set in the foundation gallery and tunnels at 10-foot centers. The maximum depth of hole is expected to be about 250 feet. A pattern of depths will be used such as 100 feet, 150 feet, 100 feet and 250 feet, after which the pattern will be repeated except when geologic conditions require some other arrangements. The pattern depths will decrease along the higher elevations on the abutments. Water tests of some of the exploratory holes drilled in the foundation area beneath the highest part of the dam have indicated that the sandstone formation is fairly tight. This may make it necessary to modify the above plan for the cutoff grouting as the work progresses.

The main drainage curtain for the dam will be drilled from the foundation gallery just downstream from the main grout curtain. These holes will be drilled on 10-foot centers to a maximum depth of 85 feet. Another line of holes will be drilled to the same depths and on the same spacing from the drainage gallery located 133 feet downstream. Drainage holes will also be drilled outside the main drainage curtain from the roof and the floor of the abutment tunnels at 30-foot centers to intercept any seepage which may tend to bypass the dam through the abutments. The
design of the drainage system for the dam foundation was based partially on the results of electric-tray analogy studies. In addition to the drainage curtains drilled in the foundation of the dam, 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. These drains connect to the gutters of the penstock filling line gallery and the foundation gallery.

CONTRACTION JOINTS

Glen Canyon Dam is being built in blocks formed by a system of transverse and longitudinal joints. These contraction joints are formed in the mass concrete of the dam to prevent random and irregular cracking of the concrete. By confining shrinkage to predetermined planes throughout the mass concrete, systems of grout pipe and outlets can be embedded in the concrete and the contraction joints filled with a cement grout after the shrinkage has taken place to form a continuous monolithic structure. This type of construction was used successfully at Hoover, Grand Coulee, Shasta, Hungry Horse, and other Bureau dams. The transverse joints will be keyed vertically to resist horizontal shear and the longitudinal joints will have horizontal keys to resist vertical shear.

The spacing of the transverse contraction joints was guided by the location of the penstocks and outlets in the dam. Final block spacing varied from 40 to 70 feet in width at the axis. The planes of all transverse joints are radial through the dam. The longitudinal joints are staggered between adjacent rows so as not to form a continuous joint.
parallel to the axis of the dam. In the central part of the dam, these longitudinal joints are located 150 feet and 161.5 feet downstream from the vertical portion of the upstream face. Toward the abutments these distances were increased to 151 feet and 179.5 feet so as to make the upstream and downstream blocks more nearly equal in length. The longitudinal joint will not be carried through the downstream face of the dam, but will be terminated below this surface. With this joint arrangement, the maximum size of block is approximately 60 feet wide by 210 feet long. Special cooling of the blocks in the area of joint termination and reinforcement in the lift above the joint will be provided to insure that the joints do not continue in mass concrete above. A layout of the contraction joints in the dam is shown in Figure 9.

All blocks are being placed in 7-1/2-foot lifts. The contractor has decided to place the downstream blocks ahead of the upstream blocks. The specifications state that the maximum differential between adjacent blocks shall be 30 feet and that the highest block in the dam shall not be more than 52.5 feet above the lowest. For construction reasons, permission was granted to increase these height differentials to 37.5 and 60 feet. The rate of placement in any one block is limited to not more than one lift in 72 hours. Every effort is being made to maintain regular periodic placements of successive lifts throughout the dam.

The transverse and longitudinal contraction joints will be grouted as soon as sufficient concrete has been placed above each grout lift and as soon as the blocks to be grouted have been lowered to the required temperature by secondary cooling. Below elevation 3450, the dam
will be cooled to 40° F and above this elevation to temperatures varying from 40° to 50°. The joints will be grouted through systems of embedded grout pipe in lifts 60 feet in height. Except for the two end joints at each end of the dam, a maximum pressure of 50 psi measured at the top of each grouting lift will be used. The two end joints are to be filled with water under pressures up to 200 psi to load the abutment rock and impose unrecoverable strain on the rock so that the foundation will act elastically under loading conditions. Subsequent to this water loading, these joints will be pressure grouted. Double, stainless steel seals will be installed around each grout lift in these end joints to assure retention of the high pressure in the joints. At other joints in the dam, double stainless steel seals are at the upstream face and a single seal at the top, bottom, and downstream side of each grout lift. The grouting of all joints in the dam will assure a continuous monolithic structure.

TEMPERATURE CONTROL

The system for controlling temperature in the concrete at Glen Canyon Dam is designed to avoid cracking of the blocks as the concrete reaches its final stable temperature. Mass concrete increases in temperature after placement due to the heat of hydration of the cementing materials. After reaching its peak temperature, the concrete will then start cooling toward a final stable temperature which is dependent upon exposure conditions. Cracking of the concrete very often occurs during this period immediately after the concrete has set, due either to the amount and rate of temperature drop or to extreme temperature gradients near the surface.
NOTES
Odd numbered blocks shall be the low blocks. Dotted lines in Blocks 3, 5, 7, 21, 23, and 25 indicate shape and maximum size of block above termination of longitudinal joint.
Temperature control measures for Glen Canyon Dam were deemed necessary for several reasons. First, the structure is an arch dam which must be artificially cooled in a relatively short time so that the contraction joints can be opened and grouted during construction before the reservoir load comes against the dam. Secondly, grouting the contraction joints at temperatures of 40° to 50° F results in a temperature benefit in the arches where it reduces the tensions caused by external loads. And, thirdly, temperature control measures reduce the cracking tendencies present in the concrete. The size of the blocks and the relatively high placing temperatures which would normally exist at the site require that the temperature drop from the maximum concrete temperature to the final stable temperature be reduced to the practicable minimum. For the size of the blocks at Glen Canyon Dam, a maximum temperature drop of 35° was used for designing the cooling system.

To meet the above design requirements, embedded cooling coils in which refrigerated water is circulated are placed on the top of each 7-1/2-foot lift of concrete in the dam, and it is required that the concrete be placed in the forms at a temperature not higher than 50° F. Further measures include the use of a Type II low-alkali cement and the addition of pozzolan as a part of the cementing material.

Because the majority of structural cracking occurs at or near the foundation where the foundation restraint is the highest, the most rigid temperature control measures are employed in those lifts of concrete within 30 feet of the foundation. At the foundation, the cooling pipes will be spaced 2-1/2 feet apart. This close spacing, combined with 3-foot
spacing on the top of each lift, will hold the temperature rise to 20°F ±. Elsewhere through the dam, spacing of cooling pipe will vary from 4 feet to 6 feet apart and temperature rises will be held to 25°F, which will result in a maximum temperature drop, to the grouting temperature of 35°F.

Since the actual stresses tending to crack the blocks occur primarily during a drop in temperature, every effort will be made to maintain a uniform placement schedule.

Cooling of all blocks prior to contraction joint grouting will take place in a relatively slow manner so as to create the lowest possible tensile stresses. The concrete will be cooled to 40°F in that part of the dam below elevation 3450 and to temperatures varying 40°F at elevation 3450 to 50°F at the top of the dam, except that all concrete within 30 feet of the foundation will be cooled to a minimum of 45°F. Thermocouple wires and insert resistance-type thermometers are used throughout the dam to check temperatures in the blocks at various elevations.

The contractor has provided a refrigeration plant which has a rated refrigeration capacity of 4,000 tons. This refrigeration effort is directed into two categories. The first category is the precooling of the concrete ingredients to hold the maximum placing temperature of the concrete to 50°F. To accomplish, about 2,400 tons of refrigeration are used to cool the mixing water, provide chipped ice for addition to the mix, sprinkle the coarse aggregate en route to the batchers, and cool the coarse aggregate by means of refrigerated airblasts in the batching bins.

The secondary category is the postcooling of the concrete by circulating refrigerated water through pipes embedded in the concrete.
Approximately 1,600 tons of refrigeration will be used for postcooling. Postcooling is accomplished by an initial cooling period of 12 days immediately following the placement of the concrete which reduces the maximum temperature attained by the concrete, and by a secondary cooling period of about 60 days which lowers the concrete temperature to the temperature desired for contraction joint grouting.

Although the secondary cooling period normally does not start until required for the grouting program, a limited amount of secondary cooling is accomplished in the concrete where the longitudinal joints are to be terminated. To prevent this formed crack from progressing upwards into the concrete blocks above the termination of the longitudinal joint, the specifications require that the concrete be cooled to 50° F before the overlying concrete is placed.

**PENSTOCKS**

Glen Canyon Powerplant is under construction downstream from the dam. The building will be 649 feet long and 128 feet wide and will have a reinforced concrete substructure and a superstructure of structural-steel frames with reinforced concrete enclosure walls. The powerplant will house eight 112,500-kw generating units, each driven by a 155,500-horsepower turbine.

Eight 15-foot-diameter steel penstocks for delivering water to the turbines in the powerplant are embedded in the dam. The centerline of the penstock intake is at elevation 3470 which is about 45 feet above estimated silt elevation after 150 years of reservoir operation. Minimum
water surface for power operation is elevation 3490. A 13.96- by 22.45-
foot fixed-wheel hydraulically-operated closure gate, for each penstock,
which can operate under unbalanced head, is provided at the upstream face
of the dam. An unlined transition section from rectangular to round is
provided at the entrances of the penstocks to reduce the width required
for each closure gate. A reinforced concrete trashrack structure with
structural steel bars protects the entrances. To reduce head losses, the
ribs and columns of the trashrack structures are streamlined. Slots for
stoplogs just upstream from the closure gate slots provide a means for
inspecting the gates and the gate guides if required. Special joints to
take movement in three directions have been designed for the penstocks,
where they leave the dam, to provide for movements of the dam when it is
fully loaded. These joints are set in special vaults at the toe of the
dam. The penstocks are carried between the dam and the powerplant on
reinforced concrete supports which, in turn, are carried to bedrock. The
backfill in the area between the dam and powerplant is to be placed above
the top of the penstocks. A section through the dam on the centerline of
a penstock is shown in Figure 10.

The layout of the penstocks in the dam was dependent upon the
following criteria:

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

2. The minimum distance from the centerlines of the penstocks and
the radial contraction joints in the dam had to be one and one-half pipe
diameters.
3. There were to be no bends in the penstock between the dam and the powerplant.

4. The minimum permissible bend radius was to be \( \frac{1}{4} \) pipe diameters.

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

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

The steel penstocks were designed for full bursting pressure and water-hammer effects. The concrete surrounding the penstocks in the dam is reinforced for tensile forces due to dam stresses and temperature effects, except in the areas where the penstocks leave the dam and in the areas of the unlined transitions where additional reinforcement is provided for internal bursting pressure. The reinforced concrete columns supporting the penstocks, between the dam and powerplant, are H-shaped columns designed for dead weight, penstock reactions, and earthquake effects. The reinforced concrete trashracks are designed for a differential water load of 20 feet.
RIVER OUTLETS

River outlets having a capacity of 15,000 cfs with the reservoir water surface at elevation 3490, minimum water surface for power operation, are installed in the dam near the left abutment. The river outlets provide for releases for downstream commitments when the powerplant is not in operation and during the period of final closure of the diversion tunnels. The outlets will also be used to maximum capacity during maximum flood releases.

The outlet works consists of four 96-inch-diameter steel pipe with cast-iron bellmouth intake, hollow-jet valves for regulation, and ring follower gates for emergency closure. A bulkhead gate, which operates under balanced head, is provided at the upstream face of the dam to provide access for servicing the ring-follower gates. A reinforced concrete trashrack structure with structural steel bars protects the entrance. The maximum permissible velocity in the outlets is 75 feet per second. The best arrangement for the outlets in the dam was found to be two parallel outlets in each of two 60-foot-wide blocks. The centerline distance between the outlets in the dam, 15 feet 7 inches, was dictated by the required clearance for the bulkhead gate frame metalwork around the bellmouth intake. The minimum distance between the centerlines of the outlets and the radial contraction joints in the dam was set at 1-1/2 outlet diameters. Radii of bends are 4 diameters except where lack of space required using a bend of 3 diameters. The centerline of the intake is at elevation 3374 which is about 30 feet above the estimated 100-year silt level in the reservoir. The ring-follower gates are located in a chamber in the dam 60 feet
downstream from the face of the dam. To facilitate installation of the
gates, blockouts are provided in the gate chamber floor. A vertical shaft
from the chamber to the roadway at the top of the dam is provided for
removing gate parts for servicing. Special joints to take movement in
three directions have been designed for the outlet pipes where they leave
the dam, to provide for movements of the dam when the dam is fully loaded.
A section through the dam on the centerline of one of the outlets is
shown in Figure 11.

As the powerplant structure occupies the entire bottom of the
canyon from abutment to abutment, it was necessary to locate the outlets
in the mass concrete beneath the service bay and machine shop as they
leave the dam. In this area, they are set two above each other at mini-
mum spacing. Beyond the machine shop the outlets are encased in concrete
and are located below the powerplant parking area. The hollow-jet regu-
larizing valves are located about 700 feet downstream from the axis of the
dam. In this area, the outlet pipes are spread apart and are all brought
to elevation 3175. The hollow-jet valves are located at this point so that
their operation would not interfere with the powerplant tailrace. The
location of the valves and their operation were studied on a large hydraulic
model. Piezometers have been provided in the outlet pipes to check the dis-
charges from the outlets and to study the losses in the pipes and pipe
bends.

The steel outlet pipes are designed for full bursting pressure.
The outlet pipes in the dam are reinforced for tensile forces due to dam
stresses and temperature effects, except in the area of the bellmouth
entrance and the area where the pipes leave the dam. In these areas additional reinforcement is provided for internal bursting pressure. The reinforced concrete trashrack structures are designed for a differential water load of 20 feet. The second-stage concrete in the blockouts around the ring-follower gates is reinforced for internal bursting pressure within the gate frames and bonnets. The mass concrete around the pipes in the area from the downstream face of the dam through the service bay and machine shop is reinforced for tensile forces due to internal bursting pressure and for unbalanced forces in the bends. From the downstream end of the machine shop to the hollow-jet valves the mass concrete around the pipes is reinforced for tensile forces due to internal bursting, unbalanced forces in the bends, and for truck and trailer loads in the powerplant parking area.

SPILLWAY STRUCTURE

The spillway structures for Glen Canyon Dam are designed to pass the maximum probable snowmelt flood which occurs in the spring of the year and is predicted to have a peak discharge of 380,000 cfs and a volume of 29,060,000 acre-feet over a 4-month period--April, May, June, and July. The peak discharge is about 1.7 times as large as the maximum recorded discharge of 220,000 cfs. Also determined was a maximum probable rain flood, which occurs in the fall of the year and which has a predicted peak discharge of 417,000 cfs and a 6-day volume of 2,063,600 acre-feet. Flood routing studies showed the maximum probable snowmelt flood to be the more critical. All studies assumed that the reservoir behind
Top of dam - E1.3715

Utility gallery

-6" Dia. air vent

Gate service shaft

Contraction joint

Gate chamber

E1.3374

96" Ring - follower gate

96" I.D. outlet pipe

Foundation gallery

Axial of dam

Drainage gallery

Service bay

Machine shop

E1.3168.50

E1.3179

E1.3175

96" Hollow - jet valve

Scale of feet

SECTION THRU RIVER OUTLETS

FIGURE II
Glen Canyon Dam was at normal water surface, top of conservation storage, elevation 3700, at the start of routing. By storing approximately 1,850,000 acre-feet of the flood water and allowing the reservoir to rise to elevation 3711, the maximum discharge through the spillways would be 276,000 cfs. In bypassing the maximum flood, an additional 15,000 cfs would be discharged through the river outlets, and 9,000 cfs through four units of the powerplant for a total flood release of 300,000 cfs.

Two circular tunnel-type spillways will be used to pass the flood waters, one located in each abutment. The entrances for the spillways are located about 600 feet upstream from the dam and each consists of an unlined approach channel and a reinforced concrete crest structure with piers. Each crest is set at elevation 3648, and discharge through the tunnel is controlled by two 40-foot by 52.5-foot structural-steel radial gates with counterweights. The crest shape is designed for a gate opening of 10 feet. The coefficient of discharge for maximum flow as determined by model studies is 3.46. A reinforced concrete hoist bridge, spanning the piers, supports the hoists for operating the gates and to provide access. A section through the spillway is shown in Figure 12.

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

FIGURE 12
rock surrounding the tunnel upstream from the downstream end of the elbow and at the downstream portal is to be grouted from radial holes varying from 20 to 40 feet in depth, spaced at up to 20-foot centers measured along the centerline of the tunnels. Drainage holes 3 inches in diameter, drilled radially 25 feet deep at 20-foot centers, are also provided in the tunnels from the entrances to a point 400 feet downstream from the elbows. Requirements for drainage were determined by electrical tray analogy studies. For discharge at the downstream portals, a reinforced concrete deflector bucket was designed to lift the jet of water a safe distance into the center of the river channel and also to deflect the jet away from the canyon wall. The original layout for the spillway tunnels was based on data obtained from other tunnel spillways built by the Bureau, routing of the flood through the tunnels, and adapting the lower ends of the tunnels to the diversion scheme.

Extensive hydraulic model studies of both spillway tunnels were made on a 1:63.5 scale model. These studies aided in determining the final dimensions of the approach channel, the transition section of the tunnel, and the location and final shape of the deflector buckets. Model studies indicated that the maximum velocity in the spillway tunnel will be 162 feet per second.

The thickness of the tunnel lining for the circular sections was made 0.8 inch per foot of the tunnel diameter in inches. The lining was reinforced where required for tension caused by dam and reservoir water loads. The stresses in the vicinity of the tunnels were determined by
Newmark's method of computation of stresses in elastic foundations.

Stresses in the lining caused by the stresses in the rock were determined by both analytical and model studies.

Moments, thrusts, and shears in the noncircular transition section of the tunnels were determined by model studies of various sections for hydrostatic load, dead load, grout loading and rock load based on the assumption that the height of the rock, acting on the structures, is equal to 0.35 of the excavated width. The results of these studies were used to determine final concrete thickness and reinforcement.

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

The reinforced concrete deflector buckets were designed for dead weight, static and dynamic forces from water at maximum discharge, and uplift pressure. In general, the bucket was designed as a monolithic structure setting on an elastic foundation. Part of the design was determined by photoelastic model studies. The dynamic forces from the deflecting jet were determined from the hydraulic models.
Diversion and Care of the River

Diversion of the Colorado River during construction of Glen Canyon Dam is being accomplished through two 41-foot-diameter concrete-lined tunnels, one located in each abutment, and each an upstream extension of the horizontal portions of the two spillway tunnels. The size of the tunnels was determined from routing studies made using the 25-year frequency flood which has a peak flow of 196,000 cfs and a 15-day volume of 3,550,000 acre-feet. Routing of this flood through the tunnels resulted in a maximum water surface elevation of 3,277 and a combined discharge of 143,000 cfs. The upstream cofferdam designed by the contractor has a top elevation of 3300 and the downstream cofferdam a top elevation of 3165. The tunnels were laid out so that there would be ample space between the upstream face of the dam and the upstream cofferdam to dispose of all excavation material from foundation excavation. The contractor used all excavated foundation material for his cofferdams except for a bentonite cutoff in the upstream cofferdam.

The tunnel through the right abutment is 2,749 feet long and has an invert elevation at its entrance of 3137.37, essentially at river level. The tunnel is concrete lined throughout, with the downstream portion, which will also be used as a spillway tunnel, having a minimum lining thickness of 2 feet 9 inches and the upstream portion a minimum lining thickness of 1 foot 3 inches. Final closure of this tunnel will be made by installing a plug section 150 feet in length constructed in three 50-foot sections. Provisions to grout the joint between each
section, the joint between the plug section and the tunnel lining, and
the rock area around the plug, is provided. The plug section will also
be cooled by an embedded pipe system. To facilitate construction of the
plug section, the entrance of the tunnel is 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 tunnel through the left abutment is 3,011 feet long; the
entrance invert elevation is at 3170.67, or 33.30 feet higher than the
right tunnel. The features of this tunnel are similar to those for the
right abutment except that outlet gates are to be installed in the plug
section prior to closing of the right tunnel. The tunnel entrance was
set at a different elevation so that the outlet gates can be installed
in the left tunnel during a low flow season without the use of an entrance
closure structure. Riverflows less than 15,000 cfs will be confined to
the right diversion tunnel. A section through the left diversion tunnel
is shown in Figure 13.

The outlet works in the left diversion tunnel are to be
installed to meet downstream requirements after the right diversion tunnel
is closed and until these requirements can be met through the power facil-
ities and the permanent river outlets. It was determined from routing
studies that an outlet works with a capacity of approximately 33,000 cfs
with a head of 410 feet, in conjunction with the river outlets, would
satisfy all requirements.
The outlet works are installed in the two upstream sections of the tunnel plug, and consist of three steel-lined conduits through the plug section with two 7-foot by 10.5-foot high-pressure slide gates, in tandem, in each conduit and an operating chamber. The conduit liners upstream from the high-pressure gates are 7-foot by 10.5-foot welded steel with rectangular bellmouth entrances. Downstream from the gates the conduits are 7 feet by 14.5 feet with 1.5-foot fillets in the roof and are provided with welded steel liners in the floor and in the sides up to the fillets. The additional height of the conduits permits free flow conditions, and the space between the water surface and the top of the conduits provides an air passage to the gates. Due to the space required for the large gates, it was necessary to excavate above the roof of the tunnel to provide a gate chamber. The gates are designed for a normal operating head of 350 feet using normal design stresses (40 percent of yield point) or a maximum nonregulating head of 560 feet using increased design stresses (75 percent of yield point).

Access to the gate operating chamber is from an adit to the gallery system in the dam. An adit from the gate chamber to the downstream end of the plug section for use during construction and to serve as an auxiliary air supply during operation of the outlets is also provided. The outlet works is protected by a semicircular reinforced concrete trashrack structure, with structural-steel trashbars designed for a 40-foot head. A 1:24 scale hydraulic model of the outlet works which included the downstream section of the tunnel was tested in the Bureau of Reclamation’s Hydraulic Laboratory in Denver.
ELEVATION ALONG 41' DIA. LEFT DIVERSION TUNNEL

FIGURE 13
The concrete in the tunnel plug sections containing the outlet gates and conduits was reinforced for the following loads: (1) Internal bursting pressure in the conduits upstream from the gates; (2) differential pressure between adjacent conduits upstream from the gates, caused by one gate being open and an adjacent gate closed; (3) internal bursting pressure in the gate frames and bonnets; and (4) external grouting loading caused by grouting between the plug sections and the tunnel lining. The magnitude and location of the stresses caused by the grout loading were determined by photoelastic model studies.

Diversion of the river was begun in February 1959. Storing of water in the reservoir can begin after the outlet gates are installed in the left tunnel and final closure is made of the right tunnel. Closing of the right tunnel is scheduled for December 1962, according to the contractor's latest schedule. However, before this can be accomplished, according to the specifications, the concrete in the lowest block will have to be at elevation 3540 or higher, or there must be assurance that the lowest block will reach that elevation by January 1, 1963. In addition, the installation of the high-pressure gates in the left tunnel must have been completed and in operating condition and there must be assurance that the 96-inch-diameter outlets through the dam will be in operating condition by the beginning of the flood season following closure. The construction heights for the dam are specified, so that sufficient storage is available in the reservoir to pass anticipated flood flows without overtopping the dam. To avoid complete blockage of the river during closure, the contractor will be required to pass a minimum of 1,000 cfs
through the right tunnel intake closure structure until sufficient head is available to pass a minimum of 1,000 cfs through the left tunnel gates. The contractor may then complete closure of the right tunnel, and downstream releases will be made through the gates in the left tunnel.

Whenever, under the operating criteria for filling the reservoir as finally determined, sufficient permanent storage has been attained to permit substitution of hydraulic capacity of the powerplant turbines for the then required discharge capacity of the left tunnel gates, final closure of these gates and installation of the final plug section will be accomplished. Regulation of the river at Glen Canyon will thereafter be accomplished by releases through the turbines, through the river outlets, and through the spillways.

SUMMARY

The second highest dam in the Western Hemisphere, Glen Canyon Dam has advanced to the stage of concrete placement with over 700,000 cubic yards of 4,865,000 cubic yards in place.

Designs for all the major features of the dam which was begun in 1956 are virtually completed. Extensive field and laboratory investigations were conducted to determine the properties of the foundation rock. A large number of trial-load stress studies were made of the dam using a digital electronic computer. A very thorough plan of foundation treatment and instrumentation has been included. The hydraulic designs for spillways, diversion tunnels and river outlets were checked by hydraulic
model studies. The problems of temperature control of mass concrete in the dam and contraction joint grouting were also extensively studied.

At the present rate of construction, first power from the powerplant should be available by June 1964.

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Floyd E. Dominy at Washington, D.C., is Commissioner of the Bureau of Reclamation. Grant Bloodgood is Assistant Commissioner and Chief Engineer at Denver, Colorado; L. G. Puls is Chief Designing Engineer of the Bureau. Glen Canyon Dam is in the Bureau's Region 4; F. M. Clinton at Salt Lake City, Utah, is Regional Director. L. F. Wylie is Project Construction Engineer for the Glen Canyon Unit at Page, Arizona.