Denver, Colorado
February 7, 1949

Memorandum

To:              R. F. Blanks
From:            R. E. Glover

Subject: Estimates of bank losses and recoverable bank storage for Glen Canyon Reservoir.

1. Introduction

The reservoir to be created by the proposed Glen Canyon Dam will rest against walls of the Navajo sandstone for many miles. This sandstone is somewhat permeable and there are other beds present in the basin which are also capable of carrying seepage waters. Several questions have presented themselves because of the presence of these beds, among which are: a. How much water will be lost by seepage into the sandstones? b. What amount of bank storage may be recoverable? c. What seepage losses around the dam may be expected? It is the purpose of this memorandum to present some outside estimates from which the possible magnitude of these factors may be judged.

2. Conclusions

The present estimates are based on available data. It is anticipated that a much better estimate will be possible as soon as the geological data and permeability test data can be studied and correlated. The geology of the reservoir basin is very complex.

The following principal conclusions may be drawn:

a. The loss due to seepage into the permeable sandstones of the reservoir basin could amount to about 1,800,000 acre-feet the first year. In 10 years the accumulated loss would be about 6,000,000 acre-feet and in 100 years, 18,000,000 acre-feet. This may be compared with the evaporation loss, which with a full reservoir would be about 850,000 acre-feet per year. Thus the bank loss would be about twice the evaporation loss for the first year and the two losses would be about equal for the second year. The bank loss diminishes with time and at the end of the 10th year the loss to the banks would be only about one-third of the evaporation loss. In 100 years the accumulated loss to the banks would be about one-half of enough to fill the reservoir.

b. A yearly fluctuation of the reservoir level of 250 feet due to river regulation and power demands would cause water to be stored in the banks at high levels which would be recovered when the reservoir level is low. The bank storage would be about enough to increase the effective storage capacity of the reservoir from 30,000,000 acre-feet to 30,500,000 acre-feet.
c. The seepage water entering the permeable beds upstream from the dam, passing around the ends of the dam and returning to the river below the dam, would amount to about 14 cubic feet per second.

The methods of making these estimates are given in detail in the following paragraphs.

3. Loss by Seepage Into the Sandstone

To obtain an outside estimate of the amount of water which might be lost by seepage into the sandstone let it be supposed that the beds are indefinitely extended in directions at right angles to the stream so that seepage water might be stored in them forever. It will be assumed, for the sake of simplicity, that flow takes place in directions at right angles to the stream, whose canyon will be supposed straight. A typical cross-section at some time after filling the reservoir is illustrated in Figure 1.

![Diagram of seepage into sandstone](image)

The requirement that a difference of flow through two sections dx apart must appear as a rise or fall of the phreatic line between the sections gives rise to the relation:

$$\frac{\partial y}{\partial t} = \frac{h^2}{\partial x^2} \frac{\partial^2 y}{\partial x^2} \ldots \ldots (1)$$
where:

- \( K \) represents the permeability
- \( D \) the depth of permeable beds
- \( v \) the voids ratio
- \( t \) time

and

\[ h^2 = \frac{KD}{v} \]

In developing the above expression, the gradient of the phreatic line \( dy/dx \) is assumed to apply from there to the bottom of the permeable bed. This is a common assumption and appears to be valid where gradients are moderate. The rise of the water level in the bed; \( y \) has been neglected as compared to \( D \), the original depth.

A solution of the differential equation (1) which meets the condition that:

- \( y = y_o \) when \( x = 0 \) for \( t > 0 \).
- \( y = 0 \) for \( x > 0 \) when \( t = 0 \).

is:

\[ y = y_o \left[ 1 - P \left( \frac{x}{\sqrt{4h^2t}} \right) \right] \quad \ldots \ldots \ldots \ldots \quad (2) \]

where \( P \left( \frac{x}{\sqrt{4h^2t}} \right) \) represents the probability integral for the argument \( \frac{x}{\sqrt{4h^2t}} \). The probability integral is a tabulated function.

The flow to each bank is:

\[ Q = -KLD \left( \frac{dy}{dx} \right)_o = \frac{KLDy_o}{\sqrt{\text{\Pi} h^2t}} \quad \ldots \ldots \ldots \quad (3) \]

where \( L \) represents the length of the beds exposed in the reservoir basin. This quantity is measured in the direction of stream flow.

\( \left( \frac{dy}{dx} \right)_o \) represents the gradient where \( x = 0 \).

\[ \left( \frac{dy}{dx} \right)_o = \frac{-y_o}{\sqrt{\text{\Pi} h^2t}} \]

The total volume lost to each bank is

\[ V = \int_0^t Q \, dt \]
then

\[ v = \frac{2KLdy_0}{\sqrt{\pi} h^2 t} \]

or

\[ v = 2Qt \ldots \ldots \ldots \ldots \ldots (4) \]

In the present case the distribution of permeable beds is very complex but after consultation with Messrs. Ahrens, Heaton, and Thompson, it seems reasonable to take the following figures:

- \( L = 800,000 \text{ feet (150 miles)} \)
- \( D = 500 \text{ feet} \)
- \( y_0 = 300 \text{ feet} \)
- \( K = 200 \text{ ft./yr.} \)
- \( v = 0.20 \)

then:

\[ h^2 = \frac{KD}{v} = 500,000. \text{ ft.}^2/\text{yr.} \]

\[ h^2 = 1253.3 \]

The following table has been prepared on the basis of equations (3) and (4).

**Table 1**

<table>
<thead>
<tr>
<th>Time years</th>
<th>Losses second-feet</th>
<th>Losses acre-feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>--</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>1,220.</td>
<td>1,760,000.</td>
</tr>
<tr>
<td>2</td>
<td>860.</td>
<td>2,490,000.</td>
</tr>
<tr>
<td>3</td>
<td>700.</td>
<td>3,040,000.</td>
</tr>
<tr>
<td>4</td>
<td>610.</td>
<td>3,530,000.</td>
</tr>
<tr>
<td>5</td>
<td>540.</td>
<td>3,920,000.</td>
</tr>
<tr>
<td>10</td>
<td>385.</td>
<td>5,560,000.</td>
</tr>
<tr>
<td>100</td>
<td>122.</td>
<td>17,600,000.</td>
</tr>
</tbody>
</table>
The following approximate reservoir data have been supplied by Mr. Riter.

<table>
<thead>
<tr>
<th>Data</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reservoir capacity</td>
<td>30,000,000 acre-feet</td>
</tr>
<tr>
<td>Surface area</td>
<td>170,000 acres</td>
</tr>
<tr>
<td>Evaporation loss</td>
<td>5 feet per year</td>
</tr>
<tr>
<td>Fluctuation of reservoir level during the year due to river control and power demands</td>
<td>250 feet</td>
</tr>
</tbody>
</table>

With these data it is possible to make some comparisons. The evaporation loss would be about \((5)(170,000) = 850,000\) acre-feet per year. This is equivalent to a flow of about 1,200 second-feet. That is, it would take about this much inflow to supply the evaporation losses. During the first year the loss to the banks would be about double the evaporation losses. The second year they would be a little less than the evaporation losses and after ten years they would drop to less than one-third the evaporation losses.

In 100 years the accumulated loss to the banks would be approximately half the quantity required to fill the reservoir. The flow of the Colorado River averages about 23,000 cubic feet per second and the runoff averages about 16,000,000 acre-feet per year. Then the losses to the banks at the end of the first year would be about 4 percent of the inflow and in 100 years the accumulated loss would equal 1 year's runoff. These losses are not recoverable unless the reservoir is emptied. If the reservoir were kept empty for some time after having been filled, a part of the water previously stored in the banks would return.

4. Recoverable Bank Storage

Due to the yearly fluctuation of the reservoir level some water will be stored in the banks when the reservoir is high which will return when the reservoir is drawn down. The following formula is developed in the informal memorandum of July 6, 1948, by Messrs. Zangar, Kahm, and Bruggeman on "Experimental and Theoretical Investigation of Seepage and Ground Water Storage Possibilities at Mullen Damsite—Missouri Basin Investigation, Nebraska." It is, for one bank:

\[
V_1 = KDLZ_m \sqrt{\dfrac{2T}{\pi h^2}}
\]  

\[
(5)
\]

where

- \(V_1\) is the volume stored and recovered
- \(Z_m\) the amplitude of the reservoir fluctuation (assumed sinusoidal)
- \(T\) the period of the fluctuation
with \( Z_m = 250/2 = 125 \) feet, \( T = 1 \), \( K \), \( D \) and \( h^2 \) as before. The recoverable storage from both banks is

\[ 2V_1 = 518,000 \text{ acre-feet}. \]

Then the effect of the water stored in and recovered from the banks due to the yearly fluctuation of 250 feet is to increase the available storage from 30,000,000 acre-feet to 30,518,000 acre-feet.

5. **Seepage Loss Around the Dam**

To estimate the amount of seepage around the dam it may be assumed, as an idealization, that the dam is placed in a straight canyon and the seepage water flows in circular paths around each end of the dam. A developed profile of one of the flow paths is shown in Figure 2.

![Reservoir Diagram](image)

**Fig 2.**

Let:

- \( q \) represent the seepage flow per unit of length measured in the direction of \( r \),
- \( r \), the radius measured into the abutment from the center of the end of the dam,
- \( \theta \), a radian angle specifying the position of a radius in the horizontal plane,
- \( y \), the depth of ground water in the permeable bed,
- \( Q_1 \), the total flow between two radii \( r_1 \) and \( r_2 \).
Then if the gradients in the direction of \( r \) are nil, so that all flow is normal to the radii

\[ q = -Ky \frac{dy}{r \, d\phi} \]

or

\[ y \frac{dy}{d\phi} = -\frac{qr}{K} \ldots \ldots \ldots (6) \]

This is a differential whose solution, subject to the conditions

\[ y = y_0 \text{ when } \phi = \phi_0 \]
\[ y = y_1 \text{ when } \phi = \phi_1 \]

is:

\[ q = \frac{K(y_0^2 - y_1^2)}{2r(\phi_1 - \phi_0)} \ldots \ldots (7) \]

The total flow for one side is:

\[ Q_1 = \int_{r_1}^{r_2} q \, dr \]

or

\[ Q_1 = \frac{K(y_0^2 - y_1^2)}{2(\phi_1 - \phi_0)} \log_e \frac{r_2}{r_1} \ldots \ldots (8) \]

In the present case we may suppose that the dam will average about 500 feet thick so that \( r_1 = 500/2 = 250 \) feet, and that the sandstones extend upstream to about the mouth of the San Juan, a distance of about 63 miles or 333,000 feet. Then \( r_2 = 333,000 \) feet. A depth of sandstone below river level of 500 feet may be assumed as before. The water level is assumed to be raised 600 feet by the dam.

Then the assumptions may be summarized as follows:

\[ y_0 = 1,100 \text{ feet} \]
\[ y_1 = 500 \text{ feet} \]
\[ (\phi_1 - \phi_0) = \pi \]
\[ K = 200 \text{ feet per year} \]
\[ r_1 = 250 \text{ feet} \]
\[ r_2 = 333,000 \text{ feet} \]
then
\[ \frac{r_2}{r_1} = 1,330. \quad \log_e \frac{r_2}{r_1} = 7.19292 \]

And the seepage around one side is:

\[ Q_1 = \frac{200 (1,100^2 - 500^2)}{2 \pi} (7.19292) = 219,800,000 \text{ cubic feet per year.} \]

The seepage around both ends of the dam is:

\[ 2Q_1 = 440,000,000 \text{ cubic feet per year.} \]

This is equivalent to about 10,000 acre-feet per year or 14 cubic feet per second.

Robert E. Glover

Job 4-557-18-00-9001
Glen Canyon - "Preliminary design and estimates coordinating with design." Feb 1.
Memorandum

To: Chief Research Engineer
Through: Chief, Concrete Laboratory Branch
From: R. E. Glover
Subject: Correlation of abutment modulus data as obtained from tests on cores, in-situ tests and seismic tests

Purposes

Laboratory tests on core samples, field tests of the rock in place, and tests by seismic methods often yield very discordant values for the elastic modulus of the foundation rock which is to carry the thrust of an arch dam. It is important that the elastic modulus of the foundation rock be properly evaluated in order to permit the preparation of a design for a dam which will not only be adapted to the site but will also be both economical and safe. The studies described herein have been made for the purpose of correlating the data from different sources which have a bearing on the effective modulus of the foundation.

Tests on Cores

The difficulties heretofore experienced may be traced to the characteristics of abutment rocks. If these rocks showed an ideal elastic behavior, there should be no discrepancies among the laboratory, in-situ, and seismic test results. A few rocks do approach this ideal but others depart from it materially.

Stress-strain curves for a number of rocks are shown in the Concrete Laboratory Report No. SP-39 on "Physical Properties of Some Typical Foundation Rocks." About 38 rocks of varying types are described in this report. The stress-strain curve is shown for the first cycle. In most cases, the curve plotted on the chart, having stress as ordinate and strain as abscissa, is concave upward indicating an increase of elastic modulus with an increase of stress.

These rocks behave as though the relaxations of restraints which accompanied the removal of the core permitted some of the grain contact boundaries to separate. A reapplication of stress closes these gaps progressively so that the specimen shows an initial stiffening with increase
of stress. If this is the true explanation of their behavior, then it should be expected that these peculiarities should be present in the rocks in dam abutments and tunnels especially where the rocks have been subjected to blasting. Field observations seem to show that these exceptions are realized. In Reference 4 it is related, for example, that a few scattered experiments have disclosed that blasting in a tunnel can cause the walls to be displaced inward more than a centimeter. In Reference 5, a record of strains in the foundation rocks due to the loads imposed by placing of concrete in the dam are shown. In one case, the granite rock of the foundation deformed under the increasing weight of the dam as though it had an elastic modulus of only 50,000 kg/cm². This is equivalent to 710,000 pounds per square inch. Test specimens of this rock showed a modulus of 117,000 to 180,000 kg/cm² or 1,660,000 to 2,560,000 pounds per square inch after the specimen had been placed under 60 kg/cm² or 854 pounds per square inch stress.

The deformation was measured by a strain gage placed vertically in the foundation near the surface. It measured the compression of the surface rock.

**Triaxial Tests**

Tests reported in Reference 11 indicate that hydrostatic compressions improve the elastic behavior of the rock. In these tests, a fluid pressure of 1,000 pounds per square inch was first imposed on samples of Navajo sandstone and an additional mechanical load was then applied. These tests were repeated with fluid pressures of 500, 250, and 0 pounds per square inch. With no fluid pressure, the elastic modulus of the dry rock was 782,000 pounds per square inch. This is a secant modulus under 800 pounds per square inch axial stress. When the fluid pressure was 1,000 pounds per square inch, the elastic modulus, determined from the strains produced by the axial loads, was 1,820,000 pounds per square inch. When fluid pressure was applied, the stress-strain curves, due to application of an increased axial load, were found to be linear. When fluid pressures are absent, the stress-strain curve is nonlinear. The specimen stiffens with increase of load. These tests were performed on Navajo sandstone specimens from the Glen Canyon area.

**In-situ Tests**

Tests of the rock in place are generally made by excavating a tunnel into the abutment, smoothing an area on the wall or roof, loading a portion of the smoothed area, and measuring the deformation of the rock under load. At the Glen Canyon site, circular metal discs 13.56 inches in diameter were placed against opposite sides of a tunnel and forced apart by an hydraulic jack placed between them (Reference 2). The separation of the two discs was measured as a means of finding the displacement produced by the load.
The Portuguese Engineers (Reference 5) have made use of cushions as a means of applying the load. These are hollow disc-shaped devices made of thin sheet metal. They are placed against a prepared surface of the tunnel wall or roof and supported from below. The load is applied by pumping a fluid under pressure into the cushion. An essentially uniform pressure is maintained over the loaded area by this device, whereas, if a heavy metal bearing plate is used, the stress distribution applied to the wall is unknown. It will be seen later, however, that the mean deflection of the surface is almost independent of the stress distribution.

The formulas for the mean deflection of a loaded area on the face of a semi-infinite elastic solid may be used to interpret the results of these in-situ tests. If \( w \) represents the mean deflection of a circular surface of radius \( a \) under the action of a total load \( P \). The elastic modulus \( E \) is given for a uniformly distributed pressure by the expression:

\[
E = \frac{0.5h}{aw} P(1-\mu^2)
\]

The symbol \( \mu \) represents Poisson's ratio.

For a load applied by a rigid die, the elastic modulus is given by:

\[
E = \frac{0.50}{aw} P(1-\mu^2)
\]

There is only an 8 percent difference between these two values. This indicates that the load distribution is not a factor of main importance insofar as the determination of a modulus is concerned.

**Creep Tests**

These tests are customarily made by maintaining a load on a cylindrical specimen for a long period of time and observing the shortening under load. It is found that many materials continue to deform for a long time under the maintained loads but will ultimately reach stability. The ultimate total strain may be about twice the elastic strain produced when the specimen was first put under load.

**Tests on Restrained Specimens**

Some tests have been made in the Bureau of Reclamation laboratories on rock specimens which were embedded in plaster and loaded over a circular area on an exposed flat face. In this case, the dimensions of the flat face were large compared to the diameter of the loaded area. Such tests can be interpreted in terms of the strength of an abutment which receives the thrust of an arch dam.
Seismic Tests

These tests are made in the field by observing the speed of propagation of a seismic wave through the rock. The wave is generated by some means such as a hammer blow or the detonation of a small charge of explosive. An average value for the Young’s modulus can be obtained for an entire abutment by this means.

Correlations

Having described tests of several types, it is of interest to inquire how well they agree in regard to a determination of the elastic modulus of the rock. It may be noted at the outset that if the test of a cylinder of rock yields a straight line, stress-strain plot and the cylinder shows no creep under sustained load, then all of these types of tests should yield substantially identical values for Young's modulus. If, however, the deformations of a cylinder specimen are plotted in rectangular coordinates with strain as abscissa and stress as ordinate and the curve so obtained is not a straight line, then agreement of the results of the several types of tests is not to be expected. The results of a cylinder test then permits several interpretations. A series of secant moduli can be obtained by selecting a series of points on the stress-strain curve and using the corresponding stress and strain to define a modulus. A series of tangent moduli may be obtained by finding the slope of the tangent at various points on the curve. The test of a granite, as shown on Figure 8 of Reference 3, may be used as an example. The secant modulus at 400 pounds per square inch is about 1,600,000 pounds per square inch, whereas the tangent modulus at 2,000 pounds per square inch is about 3,330,000 pounds per square inch. A belief is expressed in Reference 5 that if the rock has been subjected to blasting these differences may be accentuated.

It will be worthwhile to consider what modulus should be used to correlate the results of an in-situ test with data from tests of an unrestrained cylinder. In the first instance, it may be noted that, since the strains in the rock under the test block grow continuously as load is added, a modulus of the secant type is appropriate.

A modulus of the tangent type is not appropriate since a tangent modulus relates, essentially, to the strain accompanying a small increment of load.

Having disposed of the tangent modulus, there still remains much uncertainty as to which, if any, of the secant moduli obtainable from the stress-strain curve within the contemplated stress range is appropriate. While trying to make this selection, it must be borne in mind that initial lateral pressure tends to raise the modulus and that while lateral pressures will generally be present in the rocks at the site, they were absent in the test on the cylinder. Some additional lateral pressures will be imposed by the loadings at the in-situ test, whereas no increase of
lateral pressure is present in the cylinder test. A reasonable compromise may be made by using the secant modulus for the cylinder for the maximum expected loading, thereby balancing the stiffening effect of lateral pressure in the foundation against the reduced moduli corresponding to the reduced pressures met at increasing distances from the loaded area. This choice leaves out of consideration the factors of moisture content, creep, previous history of the cylindrical specimen, and the possibility of joints in the rock at the site.

The immediate modulus obtained from in-situ tests at Glen Canyon (Reference 2) was, for example, 700,000 pounds per square inch for the E-W test under 625 pounds per square inch load. After the load was sustained for 46 hours, this modulus had decreased to 600,000 pounds per square inch. Tests on laterally unrestrained cylinders of Glen Canyon sandstone reported in Reference 14 gave an immediate modulus of 682,000 pounds per square inch and a sustained modulus of 648,000 pounds per square inch dry. The corresponding moduli for saturated rock are 649,000 and 598,000 pounds per square inch. These are secant moduli at 625 pounds per square inch axial stress. These sustained moduli were measured at 1,000 days. The correspondence is reasonably good.

**Comparison for Navajo Sandstone**

Navajo sandstone is the abutment rock at the Glen Canyon Dam site. A tabular comparison of all types of test data is possible for this rock. This comparison follows.
### Table 1

**YOUNG'S MODULUS DETERMINATIONS FOR NAVAJO SANDSTONE FROM GLEN CANYON DAMSITE**

<table>
<thead>
<tr>
<th>Type of test</th>
<th>Source of data</th>
<th>Young's modulus (lb/in²)</th>
<th>Poisson's ratio</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 NX cores in compression</td>
<td>Reference 11</td>
<td>757,000*</td>
<td>0.218*</td>
<td>Secant modulus at 625 lb/in²; dry</td>
</tr>
<tr>
<td>4 NX cores in compression</td>
<td>Reference 11</td>
<td>568,000*</td>
<td>-</td>
<td>Secant modulus at 625 lb/in²; 75 percent saturated</td>
</tr>
<tr>
<td>4 NX cores in compression</td>
<td>Reference 11</td>
<td>1,110,000*</td>
<td>-</td>
<td>Tangent modulus at 1,000 lb/in²; dry</td>
</tr>
<tr>
<td>4 NX cores in compression</td>
<td>Reference 11</td>
<td>1,250,000*</td>
<td>-</td>
<td>Tangent modulus at 1,000 lb/in²; 75 percent saturated</td>
</tr>
<tr>
<td>In-situ test on 13.56-inch circular area</td>
<td>Reference 2, 6</td>
<td>600,000</td>
<td>-</td>
<td>Effective modulus at 625 lb/in²; sustained 46 hours; horizontal east-west test</td>
</tr>
<tr>
<td>Seismic tests</td>
<td>Reference 9</td>
<td>1,760,000</td>
<td>-</td>
<td>Effective modulus; 75 percent saturated; average for the drill holes</td>
</tr>
<tr>
<td>Dynamic tests</td>
<td>Reference 8</td>
<td>1,095,000</td>
<td>-</td>
<td>Effective modulus; average of 38 specimens tested in flexure; dry</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1,162,000</td>
<td>-</td>
<td>Effective modulus; average of 37 specimens tested by longitudinal vibration; dry</td>
</tr>
<tr>
<td>Creep tests</td>
<td>Reference 14</td>
<td>654,000</td>
<td>-</td>
<td>Secant modulus; average of 3 cores held under 625 lb/in² for one year; dry</td>
</tr>
<tr>
<td>Creep tests</td>
<td>Reference 14</td>
<td>600,000</td>
<td>-</td>
<td>Secant modulus; average of 3 cores held under 625 lb/in² for 1 year; saturated</td>
</tr>
<tr>
<td>Tests with lateral stress</td>
<td>Reference 11 4 cores from Drill Hole 56Lm in bottom of canyon</td>
<td>782,000</td>
<td>0.209</td>
<td>Secant modulus at 800 lb/in²; no lateral stress; dry</td>
</tr>
<tr>
<td></td>
<td></td>
<td>628,000</td>
<td>-</td>
<td>Secant modulus at 800 lb/in²; no lateral stress; 75 percent saturated</td>
</tr>
</tbody>
</table>

*Computed by this writer from given data.*
Table 1 -- Continued

<table>
<thead>
<tr>
<th>Type of test</th>
<th>Source of data</th>
<th>Young's modulus lb/in²</th>
<th>Poisson's ratio</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tests with lateral</td>
<td>Reference 11</td>
<td>1,170,000</td>
<td>0.197</td>
<td>Secant modulus at 800 lb/in² axial stress with 250 lb/in² lateral pressure; dry</td>
</tr>
<tr>
<td>stress</td>
<td>4 cores from Drill Hole 56Lₙ in bottom of canyon</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tests with lateral</td>
<td>Reference 11</td>
<td>1,040,000</td>
<td>-</td>
<td>Secant modulus at 800 lb/in² axial stress with 250 lb/in² lateral pressure; 75% percent saturated</td>
</tr>
<tr>
<td>stress</td>
<td>4 cores from Drill Hole 56Lₙ in bottom of canyon</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tests with lateral</td>
<td>Reference 11</td>
<td>1,450,000</td>
<td>0.194</td>
<td>Secant modulus at 800 lb/in² axial stress with 500 lb/in² lateral pressure; dry</td>
</tr>
<tr>
<td>stress</td>
<td>4 cores from Drill Hole 56Lₙ in bottom of canyon</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tests with lateral</td>
<td>Reference 11</td>
<td>1,360,000</td>
<td>-</td>
<td>Secant modulus at 800 lb/in² axial stress with 500 lb/in² lateral pressure; 75% percent saturated</td>
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<td>stress</td>
<td>4 cores from Drill Hole 56Lₙ in bottom of canyon</td>
<td></td>
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<tr>
<td>Tests with lateral</td>
<td>Reference 11</td>
<td>1,820,000</td>
<td>0.169</td>
<td>Secant modulus at 800 lb/in² axial stress with 1,000 lb/in² lateral pressure; dry</td>
</tr>
<tr>
<td>stress</td>
<td>4 cores from Drill Hole 56Lₙ in bottom of canyon</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1,680,000</td>
<td>-</td>
<td>Secant modulus at 800 lb/in² axial stress with 1,000 lb/in² lateral pressure; 75% percent saturated</td>
</tr>
</tbody>
</table>

For purposes of estimating the Glen Canyon abutment deformations under a first time loading, the tangent moduli should be rejected for reasons given previously. It will be noted that they are definitely higher than were obtained from the in-situ test. The seismic and dynamic moduli are also comparatively high. This seems to be due to the absence of creep during the very brief loading periods associated with these tests. Mr. Harboe states that as his compression tests times grow shorter his elastic modulus values grow higher. This lends support to the belief that creep is the factor responsible for such differences.

Lateral stress leads to an increased Young's modulus and stress-strain relationships which are very nearly linear. There will be some lateral stress along the lines of arch thrust entering the abutment because of stresses imposed on horizontal planes by the weight of the abutment rock. This factor is present in the in-situ tests.
For reasons previously given it would be reasonable to expect correlation among the cylinder test data and the in-situ test data if the cylinder data are interpreted in terms of the secant modulus at the maximum expected stress. This comparison is shown in Table 2 below:

**Table 2**

**COMPARISON OF SELECTED MODULI FOR NAVAJO SANDSTONE**

<table>
<thead>
<tr>
<th>Type of test</th>
<th>Secant modulus lb/in^2</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>NX cores in compression</td>
<td>757,000*</td>
<td>At 625 lb/in^2 stress; dry</td>
</tr>
<tr>
<td>NX cores in compression</td>
<td>568,000*</td>
<td>At 625 lb/in^2 stress; 75% percent saturated</td>
</tr>
<tr>
<td>In-situ test</td>
<td>600,000</td>
<td>Effective modulus at 625 lb/in^2 load</td>
</tr>
<tr>
<td>Creep test</td>
<td>654,000</td>
<td>Average of three 6-inch-diameter cores</td>
</tr>
<tr>
<td>Creep test</td>
<td>600,000</td>
<td>Average of three 6-inch-diameter cores</td>
</tr>
</tbody>
</table>

*Computed by this writer from given data.

A substantial agreement will be noted among these values.

**Comparisons for a Granite**

A similar comparison may be obtained from the paper on "Deformability of Foundation Rocks" by Manuel Rocha, J. Laghina Serafim, and Antonio Ferreira Da Silveira, presented at the Fifth Congress on Large Dams in Paris 1955 (Reference 5). Experiences at a number of sites are reported in this paper. The comparisons will be made for the Canicada site where the foundation rock is described as a granite. The results of cylinder tests, in-situ tests and tests of the performance of the actual foundation under the dead load of the dam are available for comparison. The comparison is shown in Table 3 below.
<table>
<thead>
<tr>
<th>Type of test</th>
<th>Youngs modulus kg/cm²</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test on a 10-by 10- by 20-cm prism Prism 1</td>
<td>114,000*</td>
<td>Loading cycle; load held for 60 minutes; secant modulus at 20 kg/cm² stress</td>
</tr>
<tr>
<td>Test on a 10-by 10- by 20-cm prism Prism 2</td>
<td>40,000*</td>
<td>Loading cycle; load held for 60 minutes; secant modulus at 20 kg/cm² stress</td>
</tr>
<tr>
<td>Jack test in situ</td>
<td>10,000</td>
<td>Jack test on gallery in very altered rock; test made before grouting</td>
</tr>
<tr>
<td>Jack test in situ</td>
<td>19,000</td>
<td>As above except test made after grouting</td>
</tr>
<tr>
<td>Jack test in situ</td>
<td>30,000</td>
<td>Jack test in gallery on very altered rock; test made before grouting</td>
</tr>
<tr>
<td>Jack test in situ</td>
<td>50,000</td>
<td>As above except test made after grouting</td>
</tr>
<tr>
<td>Data from strain gages embedded in rock foundation</td>
<td>50,000</td>
<td>Strains observed were due to the weight of concrete placed in the dam</td>
</tr>
</tbody>
</table>

*Computed by this writer from stresses and strains read from the chart of Figure 2 of Reference 5.

The jack tests reported in this paper are made on circular areas. Load was applied through a "metallic cushion" of 1-square-meter area. These cushions are hollow discs of thin metal. They are filled with oil under pressure. This arrangement assures a uniform pressure distribution. The effective modulus was computed from

\[ E = \frac{2\pi r (1-\nu^2)}{w_0} \]

where \( p \) represents the pressure applied
\( r \) the radius of the loaded circular area
\( w_0 \) the displacement at the center of the loaded circle
\( \mu \) the Poisson's ratio

Some observations of the behavior of rock under the dam were reported.

Deformations of the actual foundation were due to the increasing height of concrete during the construction. Three strain gages were used to observe the performance of the actual foundation. These were placed near the
The surface of the rock. These strain gages indicated a modulus of 50,000 kg/cm² as noted above. The tangent modulus obtained from Prisms 1 and 2 at 60 kg/cm² were, respectively, 180,000 and 117,000 kg/cm².

The average modulus obtained from the two prism tests and the two jack tests made on ungrouted rock is 48,500 kg/cm². It is not known to this writer how much grouting may have been done on the abutment rock before the strain gage data were obtained. The authors note that the hard and unaltered rocks are apt to show inconsistent behavior under an in-situ test because of the presence of fissures. Grouting increased the apparent modulus whether fissures were present or not.

It seems obvious that the rock at this site was nonuniform in character. This is indicated by the wide variation of the results obtained from the tests on the prisms. The data provide an instructive example to show the wide range of properties which may sometimes be found at a single site.

Strength of Rock in Place

When loads are applied over a limited area on the surface of a rock abutment, the rock has a greater strength than will be shown by a cylindrical test specimen taken from the abutment. This is due to the lateral restraint provided by the rock surrounding the area of load application.

Two tests were made on Navajo sandstone from the Glen Canyon Dam area. In the first of these tests (Reference 7), segments of core 5.9 inches in diameter and about 3.76 inches long were loaded at the center through a steel disc 2.1 inches in diameter. The core was placed in a jacket made from iron pipe to simulate the effect of lateral restraint but the jacket was not fully effective for this purpose because the cement used to fill the space between the specimen and the jacket did not harden satisfactorily. The steel bearing block was probably too rigid to represent well the effect of concrete bearing against this rock.

The average compressive strength of the rock as obtained from 3 cylindrical companion test specimens was 5,930 lb/in². These results have been corrected to represent an L/D ratio of 2 since the actual specimens were approximately 2.1 inches in diameter and 3.7 inches long. When the load was applied to the flat surface of the rock through the steel disc, the rock showed strengths of 15,610 and 17,690 lb/in². This is about 2.8 times the corrected strength of the companion specimens.

In a later test, a core segment of Navajo sandstone about 6 inches in diameter and 6 inches long was embedded in reinforced plaster in a sheet iron mould and loaded through a cylinder of high strength concrete 2.12 inches in diameter and 3.44 inches long (Reference 10). In this test, the concrete cylinder failed at a stress of 8,450 lb/in². The two 2-inch-diameter by 4-inch long companion test specimens of Navajo sandstone failed at 3,720 and 3,710 lb/in². The sandstone under the cylinder, therefore, sustained, without failure, a pressure equal to 2.27 times the crushing strength of the sandstone cylinders. A steel disc was then substituted for the concrete cylinder and the test continued. The sandstone
finally failed at 14,000 lb/in² or 3.77 times the unconfined compressive strength of the sandstone.

It is evident from these tests, that the strength of rock in place as loaded by arch thrusts applied to an abutment, is much higher than the crushing strength of cylindrical test specimens having a length to diameter ratio of two. These tests would indicate a ratio of strengths of about 3.8. In Reference 5, the Portuguese authors state that "The rupture of the foundation will take place only when the applied stresses are at least equal to five times, the compression strength." This is a somewhat higher ratio than was found from the Bureau tests. More tests of this kind would lead to a better understanding of the strength of rock in place but it may be noted that in the Bureau test a partially confined soft sandstone was able to crush an exceptionally strong concrete cylinder. This indicates that an abutment of this material could sustain any stresses that a concrete dam could bring to it.

Conclusions

1. The following conclusion may be drawn from the tests and comparisons described. If a rock tested in the customary way in the form of a cylinder shows a linear stress-strain relationship and no creep under sustained loadings, it can be expected that cylinder tests, in-situ tests, seismic tests, sonic tests* and triaxial tests will all yield substantially identical values for its Young's modulus.

2. Many rocks on first loading show a nonlinear stress-strain relationship indicating a tendency to stiffen as the load is applied. This is explainable on the basis that some of the grain contacts separate when the specimen is relieved of stress. These rocks will yield different moduli when tested by different methods. Such rocks generally show a compaction as a result of loading.

3. For estimating an effective foundation modulus from tests of cylindrical specimens, it is preferable to use a secant modulus derived by dividing the stress by the total strain produced by that stress when applied the first time rather than a tangent modulus derived by dividing an increment of stress by the corresponding increment of strain after the cylinder has been compressed sufficiently to stabilize its behavior.

4. A secant modulus obtained from cylinder tests, on first loading to a stress corresponding to the maximum load to be imposed by the dam on the abutment, may be expected to correlate reasonably well with an effective modulus obtained from a well planned in-situ test at the same intensity of loading.

*There are some technical difficulties in sonic testing methods. It is assumed here that these have been successfully overcome. For an account of these difficulties see References 15 and 16.
5. Lateral pressures applied to a rock, which stiffens with an increase of loading when laterally unrestrained, will raise its elastic modulus and induce a more consistent elastic behavior.

6. Pressure grouting may be expected to raise the effective modulus of a rock abutment.

7. In-situ tests at pressures equal to or above the maximum pressure expected to be imposed on the abutment by the thrusts from an arch dam should yield the most reliable values for the effective modulus of the foundation.

8. The best results from in-situ tests should be obtained from those in which the forces are applied in the direction of the thrusts to be imposed by the dam because in these tests the modifying effects of lateral pressure will be best accounted for.

9. A circular metal or concrete bearing plate is suitable for applying the loads to the rock in in-situ tests because the distribution of pressure under the plate influences the mean displacement of the loaded surface in only a minor way.

10. The diameter of the bearing plate used in in-situ tests should be large enough to span the cracks and joints which may cause an inconsistent elastic behavior in the rock. A bearing plate covering an area of one square foot of fine grained uniform rock has been used successfully. On granites, a loaded area of 1 square meter has given good results.

11. Seismic tests provide a means for testing an entire abutment.

12. Due to the very brief period of pressure application in seismic testing the modulus obtained is apt to be higher* than that obtained from cylinder or in-situ tests because creep is not present in the seismic tests but will ordinarily be present in the other types of tests.

13. Dynamic testing is apt to produce a modulus higher than that obtained for cylinder or in-situ tests because of the very brief period of loading imposed during dynamic testing.

14. The effective modulus of the foundation should include the effect of creep because the dam will apply a long continued loading to the abutments.

15. Where loads are applied over a limited area of the surface of an extended rock mass, the resistance to crushing is generally more than three times what it is for this same rock material in the form of a standard cylindrical test specimen.

*In some cases these relationships are not observed. The cause remains obscure at the present time.
16. Widely varying rock properties may be found at various locations within a single damsite.

17. Although correlation among cylinder tests, in-situ tests, triaxial tests, and creep tests appears to be obtained when the stress-strain characteristics of the rock on first loading are taken into account, more in-situ tests and some measurements of abutment deformations under the thrusts imposed by arch dams are needed to confirm the validity of such correlations. Core tests, creep tests, triaxial tests, in-situ tests, seismic tests, and sonic tests should be made on the rock of future important damsites. These investigations should be continued until the correlation among the various types of tests is clarified. The strength of these rocks to sustain pressures over a limited area of a flat face should also be tested. There is now a lack of data on the behavior of abutments under the loads imposed by arch dams. Some measurements of abutment movement at actual dams should therefore be made.

Acknowledgements

This memorandum has profited from data supplied by Messrs. O. J. Olsen, F. E. Rippon, and L. J. Mitchell.

Robert E. Glover
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(3) "Physical Properties of Some Typical Foundation Rocks," Concrete Laboratory Report No. SP-39, Engineering Laboratories Branch, Department of the Interior, Bureau of Reclamation, August 13, 1953


(5) "Deformability of Foundation Rocks" by Manuel Rocha, J. Laghina Serafim, and A. Ferreira Da Silveira, Publication No. 82, Ministerio das Obras Publicas, Laboratorio Nacional de Engenharia Civil, Lisbon, Portugal, 1956


(7) "Effect of Lateral Restraint on the Compressive Strength of Navajo Sandstone--Glen Canyon Dam--Upper Colorado River Storage Project," Memorandum to the Chief Designing Engineer from Chief, Division of Engineering Laboratories Branch, Denver, Colorado, March 1, 1957

(8) "Dynamic Elasticity Tests on Glen Canyon Foundation Rock--Colorado River Storage Project"-Memorandum to O. J. Olsen from L. J. Mitchell, Bureau of Reclamation, Denver, Colorado, April 16, 1957

(9) "Review of Status of Seismic Method of Measuring the Elastic Modulus of Foundation Rock in Place," Memorandum to W. H. Irwin from Dart Wantland, Bureau of Reclamation, Denver, Colorado, October 30, 1957

(10) "Effect of Lateral Restraint on the Compressive Strength of Navajo Sandstone--Glen Canyon Dam--Upper Colorado Storage Project," Memorandum to the Chief Designing Engineer from Chief, Division of Engineering Laboratories, U. S. Department of the Interior, Bureau of Reclamation, April 5, 1957


(15) "A Study of Rheological and Damping Properties of Concrete" by Mageed Girgrah and Clyde E. Kesler, T&AM Report No. 173, Department of Theoretical and Applied Mechanics, University of Illinois, August 1960
Memorandum

To: L. G. Puls

From: R. E. Glover

Subject: Earthquake hazards in the Glen Canyon area

References

Reference is made to the following:

(1) Location of Earthquake Epicenters in Utah-Arizona, 1852-1945, Bureau of Reclamation Drawing X-D-3938

(2) Strength and Elastic Properties of Navajo Sandstone Core from Glen Canyon Dam Site, Mile 15, Colorado River Storage Project, Arizona Structural Laboratory Report No. SP-30, Part 1, Bureau of Reclamation, Engineering Laboratories Branch, October 1, 1951

(3) Preliminary Layout Drawing for Glen Canyon Dam Design A-8, Denver, Colorado, July 9, 1956

(4) Earthquake Stresses in Frame Structures by Robert E. Glover, Journal of the American Concrete Institute, April 1942

(5) Spectrum Analyses of Strong-Motion Earthquakes by J. L. Alford, G. W. Housner, and R. R. Martel, California Institute of Technology, August 1951

(6) intensity of Ground Motion During Strong Earthquakes by G. W. Housner, California Institute of Technology, August 1952


(8) Analysis of the Taft Accelerogram of the Earthquake of July 21, 1952, by G. W. Housner, California Institute of Technology, September 1953

(10) Theory of the Action of a Beam or Frame Subjected to an Earthquake by R. E. Glover, Bureau of Reclamation, Technical Memorandum No. 603, April 26, 1940

(11) Spectrum Curve Determined by Earthquake Analyzer for N. S. Trace, El Centro Accelerograph record, Imperial Valley Earthquake, Informal Bureau of Reclamation Memorandum to R. E. Glover from George C. Rouse, June 17, 1942 (Includes Drawing 214-REG-550 showing comparison of Bureau of Reclamation and Biot analyses for the E-W Helena, October 31, 1935, trace)

(12) Inelastic Structural Action in Earthquake Resistant Design by E. D. Rose, Bureau of Reclamation Informal Memorandum, December 10, 1943

(13) Correlation between Various Methods of Indicating Earthquake Intensities by F. E. Cornwell, August 2, 1946, Bureau of Reclamation Informal Memorandum


(15) Water Pressure on Dams During Earthquakes by H. M. Westergaard, Bureau of Reclamation Technical Memorandum No. 123, February 19, 1930


(17) Hydrodynamic Pressures on Dams due to Horizontal Earthquake Effects by C. N. Zangar, Engineering Monograph No. 11, Bureau of Reclamation, May 1952

Earthquake record
The Glen Canyon Dam site is located a few miles south of the Utah-Arizona state line on the Colorado River.(2) A reference to the
Bureau of Reclamation epicenter maps will show that the following earthquakes have occurred in this area.

**Table 1**

EARTHQUAKE OCCURRENCES SINCE 1852 IN THE GLEN CANYON DAM VICINITY

<table>
<thead>
<tr>
<th>State</th>
<th>County</th>
<th>Intensity</th>
<th>Distance from dam site, miles</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arizona</td>
<td>Coconino</td>
<td>VIII</td>
<td>115</td>
</tr>
<tr>
<td></td>
<td></td>
<td>IX</td>
<td>65</td>
</tr>
<tr>
<td></td>
<td></td>
<td>VIII</td>
<td>30</td>
</tr>
<tr>
<td>Utah</td>
<td>Washington</td>
<td>VIII</td>
<td>120</td>
</tr>
<tr>
<td></td>
<td>Sevier</td>
<td>IX</td>
<td>125</td>
</tr>
<tr>
<td></td>
<td></td>
<td>IX</td>
<td>135</td>
</tr>
</tbody>
</table>

Numerous smaller quakes are recorded. There are 17 of these in the Grand Canyon National Park area, 5 near Kanab, Utah, and 2 in the Zion National Park area. These places are, respectively, about 70, 60, and 80 miles from the dam site. The evidence therefore shows that the dam site is in an active earthquake area in which earthquakes of destructive intensity have occurred in the past.

**Correlation between Intensity Rating and Ground Acceleration**

The chart shown in the memorandum on Correlation between Various Methods of Indicating Earthquake Intensities (13) indicates the following relationships as a reasonable expectation.

**Table 2**

CORRELATIONS REASONABLY TO BE EXPECTED BETWEEN INTENSITY AND MAXIMUM GROUND ACCELERATION IN AN EARTHQUAKE AREA

<table>
<thead>
<tr>
<th>Maximum modified Mercalli intensity rating</th>
<th>Maximum ground acceleration as a fraction of gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>VII</td>
<td>0.08</td>
</tr>
<tr>
<td>VIII</td>
<td>0.14</td>
</tr>
<tr>
<td>IX</td>
<td>0.22</td>
</tr>
<tr>
<td>X</td>
<td>0.37</td>
</tr>
</tbody>
</table>

3
Since data of this kind are not capable of precise correlation, it is of interest to compare these values with some cases where accelerations have been measured. Some cases of this kind are shown in the table below.

Table 3

INTENSITIES AND ACCELERATIONS IN ACTUAL EARTHQUAKES
(DATA FROM REFERENCES 6 AND 8)

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Maximum assigned intensity</th>
<th>Measured acceleration fraction of gravity</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>El Centro</td>
<td>1940 X</td>
<td>0.33</td>
<td></td>
</tr>
<tr>
<td>El Centro</td>
<td>1934 IX</td>
<td>0.26</td>
<td></td>
</tr>
<tr>
<td>Olympia</td>
<td>1949 VIII</td>
<td>0.31</td>
<td></td>
</tr>
<tr>
<td>Vernon</td>
<td>1933 IX</td>
<td>0.19</td>
<td></td>
</tr>
<tr>
<td>Santa Barbara</td>
<td>1941 VII</td>
<td>0.24</td>
<td>Taft record obtained for a location about 30 miles from the epicenter, intensity VII at Taft</td>
</tr>
<tr>
<td>Tehachapi</td>
<td>1952 XI</td>
<td>*0.17</td>
<td></td>
</tr>
</tbody>
</table>

*Read from graph.

It is concluded that ground accelerations up to about one quarter gravity are a reasonable expectation at the Glen Canyon Dam site.

Effects of Resonance and Damping

Earthquake motions appear to be of a random sort which have no period of their own but which are, nevertheless, capable of producing resonance effects of limited amounts when applied to structures which do have a natural period. These possibilities have been evaluated by several investigators (11) (5) (14) who have obtained results in substantial agreement. The most extensive investigations are those of Alford, Housner, and Martel. They used an electronic analog computer in their work which permitted the introduction of enough driving force to overcome friction. Their
results for the case of no damping are therefore somewhat higher than those obtained at the Bureau of Reclamation (11) and by Biot (14) who used torsion pendulums. The effect of damping is quite marked. If critical damping is defined as that amount of damping which will just suffice to render a system nonoscillatory, it will provide a convenient basis for specifying the amounts of damping present. On this basis, a torsion pendulum with a piano-wire suspension will have a natural damping due, probably, to elastic hysteresis and air friction which is about 1/100 critical. A comparison of the resonance charts obtained from torsion pendulums with those of Reference (5) indicates that for periods around 0.2 seconds even this amount of damping is able to cut the resonance factor in two. For periods greater than about 0.5 second, this amount of damping has little effect. As used here, a resonant factor may be considered as the ratio of the maximum momentary forces applied to a structure when resonance is included, as compared to the force which the maximum ground acceleration would produce on the structure if applied continuously.

Real structures have finite amounts of damping. In the report on Spectrum Analyses of Strong-Motion Earthquakes by Alford, Mousner, and Martel (5), a few determinations for concrete structures are quoted. These are summarized in the following table.

Table 4

OBSERVED DAMPING RATES IN CONCRETE STRUCTURES
(DATA FROM REFERENCE 5)

<table>
<thead>
<tr>
<th>Structure</th>
<th>Damping rate expressed as fraction of critical</th>
<th>Authority</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monolithic reinforced concrete storage building</td>
<td>0.07</td>
<td>White—Bulletin Seism. Soc. Amer., 1941, Volume 31 pages 93–99</td>
</tr>
<tr>
<td>Storage building with reinforced concrete frame and floor and hollow tile walls</td>
<td>0.14</td>
<td>USGS—Earthquake Investigations in California, 1934-1935, Special Publication 201, page 125</td>
</tr>
<tr>
<td>Four-story monolithic concrete building</td>
<td>0.07 to 0.08</td>
<td>Earthquake Engineering Research Institute</td>
</tr>
</tbody>
</table>
Based upon these few observations, it would appear to be reasonable and conservative to assume that the damping rate for the dam is 0.05 critical.

The spectra given in Reference (5) indicate that the amount of resonance depends upon the natural period of the structure subjected to the earthquake. An estimate of the natural period of the Glen Canyon Dam by W. T. Moody of the Photoelastic Laboratory based upon the relation between forces and deflections obtained from a trial load analysis gave an undamped natural period of 0.86 second. A check computation made by the writer based upon a single arch ring at elevation 3400, and accounting for the yielding of the concrete and abutments with elastic moduli of 3,000,000 and 700,000 lb/in², respectively, gave a value of 0.91 second for the period. It seems reasonable, therefore, to assume an undamped natural period of 0.86 second as a basis for estimating the effects of resonance. The following factors are interpolated from the graphs shown in References (5) and (8).

Table 5

MAXIMUM EQUIVALENT ACCELERATION FOR A STRUCTURE WITH 0.05 CRITICAL DAMPING AND AN UNDAMPED NATURAL PERIOD OF 0.86 SECOND

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Trace</th>
<th>Maximum acceleration of gravity</th>
<th>Maximum ground acceleration during quake as fraction of gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vernon, California</td>
<td>N 08 E</td>
<td>0.50</td>
<td>0.13</td>
</tr>
<tr>
<td>March 10, 1933</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vernon, California</td>
<td>S 82 E</td>
<td>0.50</td>
<td>0.19</td>
</tr>
<tr>
<td>March 10, 1933</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vernon, California</td>
<td>N 08 E</td>
<td>0.10</td>
<td>0.09</td>
</tr>
<tr>
<td>October 2, 1933</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vernon, California</td>
<td>S 82 E</td>
<td>0.10</td>
<td>0.12</td>
</tr>
<tr>
<td>October 2, 1933</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Location</td>
<td>Direction</td>
<td>Date</td>
<td>Value 1</td>
</tr>
<tr>
<td>----------------------------------</td>
<td>-----------</td>
<td>---------------</td>
<td>---------</td>
</tr>
<tr>
<td>Los Angeles Subway Terminal</td>
<td>N 39 E</td>
<td>March 10, 1933</td>
<td>0.13</td>
</tr>
<tr>
<td>Los Angeles Subway Terminal</td>
<td>N 51 W</td>
<td>March 10, 1933</td>
<td>0.16</td>
</tr>
<tr>
<td>Los Angeles Subway Terminal</td>
<td>N 39 E</td>
<td>October 2, 1933</td>
<td>0.07</td>
</tr>
<tr>
<td>Los Angeles Subway Terminal</td>
<td>N 51 W</td>
<td>October 2, 1933</td>
<td>0.17</td>
</tr>
<tr>
<td>El Centro, California</td>
<td>N-S</td>
<td>December 30, 1934</td>
<td>0.60</td>
</tr>
<tr>
<td>El Centro, California</td>
<td>E-W</td>
<td>December 30, 1934</td>
<td>0.20</td>
</tr>
<tr>
<td>El Centro, California</td>
<td>N-S</td>
<td>May 18, 1940</td>
<td></td>
</tr>
<tr>
<td>El Centro, California</td>
<td>E-W</td>
<td>May 18, 1940</td>
<td></td>
</tr>
<tr>
<td>Helena, Montana</td>
<td>N-S</td>
<td>October 31, 1935</td>
<td>0.1</td>
</tr>
<tr>
<td>Helena, Montana</td>
<td>E-W</td>
<td>October 31, 1935</td>
<td>0.20</td>
</tr>
<tr>
<td>Ferndale, California</td>
<td>N 45 E</td>
<td>September 11, 1938</td>
<td>0.12</td>
</tr>
<tr>
<td>Ferndale, California</td>
<td>S 45 E</td>
<td>September 11, 1938</td>
<td>0.22</td>
</tr>
<tr>
<td>Ferndale, California</td>
<td>N 45 E</td>
<td>February 9, 1941</td>
<td>0.16</td>
</tr>
<tr>
<td>Ferndale, California</td>
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<tr>
<td>Seattle, Washington</td>
<td>S 02 W</td>
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<tr>
<td>Taft, California</td>
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<td></td>
<td></td>
<td></td>
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<tr>
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<td>July 21, 1952</td>
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This last record at Taft was obtained at a point about 30 miles distant from the epicenter of the Arvin-Tehachapi earthquake. Intensity at Taft rated Modified Mercalli VII.

If the El Centro quake of May 18, 1940, and the Arvin-Tehachapi shock of July 21, 1952, are excluded as being of an intensity
greater than would be expected in the Glen Canyon area, then it would appear that horizontal forces reaching 60 percent of gravity could be applied momentarily to the dam by earthquakes of the intensity which could be expected to occur in this area.

Factor of Safety under Earthquake Conditions

An acceleration of 60 percent of gravity would apply to the Glen Canyon Dam a force about 1.5 times that due to normal water load. This figure includes the effect of the equivalent mass of the water as estimated from Westergaard's formula (15)

\[ b' = 0.38 \sqrt{hy} \]

where \( b' \) represents the equivalent thickness of concrete at a distance \( y \) below the top of a dam of height \( h \). This thickness accounts for the inertia effect of the reservoir water.

Since the water pressure would remain, the total force applied to the dam under these conditions would be 2.5 times that due to normal water load. The computed factor of safety for this dam, as estimated by comparing the maximum stress computed by trial load methods, with the crushing strength of the concrete should be about 6. Under earthquake conditions, as described, the computed safety factor would then be reduced to about 2.4. Models of arch dams tested in Europe under normal and ultimate load conditions show that such dams will sustain loads considerably greater than the computed safety factors would indicate. This is because an arch dam is a redundant structure capable of shifting loads from highly stressed areas to less heavily loaded areas as stresses rise toward crushing strength levels. In some cases, the real factor of safety attains values as high as about twice the safety factor computed by stress and strength comparisons. It would not be unreasonable to expect that the real factor of safety of the Glen Canyon Dam under earthquake conditions, as described, would reach 4.

Conclusion

If the arch loads are distributed to the abutment rock in such a way that the full strength of the concrete in the dam can be developed, then it is concluded that the Glen Canyon Dam, constructed approximately as shown on Design A-8, will be capable of withstanding any earthquake that can reasonably be expected to occur in the area in which it is to be built.
Checks

The quantities and computations presented herein have been checked by Mr. Q. L. Florey, with the exception of the estimate of the period of the dam. The figure for the period, as used herein, was computed by Mr. W. T. Moody and checked by the writer.
Memorandum

Mr. Louis C. Puls

Mr. R. F. Glover

Details of Final Trial Load Study for Section 19-A for Glen Canyon Dam—Colorado River Storage Project

Introduction

In a conference on May 27, 1957, among Messrs. O. L. Rice, E. R. Dexter, C. L. Townsend, E. R. Schultz, Merlin D. Copen and the writer, consideration was given to some details of the Final Trial Load Study to be made for Section 19-A for the Glen Canyon Dam. My concern was particularly with the effect of the exceptionally large subcooling allowance, as planned for this dam, on the stress distribution. Having been deeply involved in the development and application of the embedded pipe cooling method, it is my belief that excessive amounts of subcooling can lead to difficulties similar to those resulting from too little cooling. In both cases the dam tends to change its shape after being made a structural unit by grouting. Stress patterns are drastically altered by the resulting readjustment.

The horizontal expansion accompanying a restoration of temperatures would be of benefit in this case as a compensation for the abutment yielding to be anticipated because of the low elastic modulus of the abutment rock. It is this improvement which the subcooling allowance was intended to secure. The effects of the vertical expansion may not be so desirable, however. Such an expansion would be resisted at the abutment and could be expected to change the stress distribution in the dam materially. An up and downstream expansion would also occur which would cause local stress disturbance near the abutment. Estimates of the stress changes due to the vertical expansion may be obtained by a modification of the tangential adjustment of the Final Trial Load Study and a procedure on making this modification is described in a subsequent paragraph. The local effect of abutment restraint of the upstream-downstream expansion can be investigated by photoelastic means if desired. Stress changes due to this cause may be locally important.

Suggested Procedure for Accounting for Vertical Displacements in Trial Load Studies

It has been realized for a long time that vertical displacements occur when an arch dam is loaded and that these displacements
should be accounted for in the tangential adjustment. The model studies made as a part of the Hoover Dam design studies indicated, however, that, for the types of arch dams tested, a satisfactory stress analyses could be obtained if the vertical displacements were assumed to be zero. This simplification has been used to the present time.

Subcooling amounts of up to 20° F under consideration for the Glen Canyon Dam would, if restrained, produce stresses of the order of 300 pounds per square inch and if unrestrained would cause the dam to grow in height on the order of an inch. This factor was not included in the model test conditions and the use of the above simplification in the case of the Glen Canyon Dam would, therefore, seem to be unwarranted.

An estimate of the vertical displacements will be needed to initiate the treatment of the effect of the vertical expansion in the tangential adjustment. For this purpose a reasonable assumption would be that the shear stress varies linearly from the crown to the abutment. This will lead to a parabolic distribution of the vertical displacements. An allowance for foundation deformation at the base of the dam should be made and there will also be a vertical component of displacement at the abutments due to the upward forces created by the vertical expansion.

It is suggested to make the tangential adjustment in the same way as before except that in computing the tangential displacement, an accounting be made of the tiltings produced by the variations of the vertical displacement along the arch center lines. If \( W \) represents the vertical displacement and \( x \) a distance measured from the crown along the arch center line, then these tiltings would have the form

\[
\frac{\partial W}{\partial x}
\]

An integral of these quantities with respect to height from the base of cantilever upward would yield the contribution of these tiltings to the tangential deflection of the cantilever element. In computing the cantilever vertical displacement, vertical loadings derived from the gradients of the shear forces must be employed. The thrust originating from shear forces in the tangential adjustment are now computed for the arch elements in this manner.

**Stress Changes Due to Variations From the Assumed Conditions**

It is important to know what magnitudes of stress changes would be caused if the conditions assumed for making the Final Trial Load Study are not realized, for some reason, or if the amount of
subcooling must be reduced. For this purpose it is suggested to compute the stresses in a selected arch, using the loadings obtained from a trial load study, on the basis that the arch is neither cooled nor grouted. Cracking will probably be found in this case and it is recommended to use the Westergaard formulas for estimating the effect of these cracks. Mr. Copen has a copy of a discussion in which I have described the use of these formulas. These formulas were worked out by Dr. Westergaard as a part of his work in the Bureau of Reclamation.

REGlover:mmh-s
Memorandum

To: Mr. L. G. Puls

From: Mr. R. E. Glover

Subject: Relief of stress concentrations at the abutment at the top of the Glen Canyon Dam

Stress Concentration

The trial load studies for the Glen Canyon Dam indicate a concentration of stresses at the top of the dam at the abutment. This concentration of stresses raises the pressure applied to the abutment at these points to about twice the highest value that occurs elsewhere. The computed figure for the A-19 section is 969, lb/in².

This finding is in accord with results obtained by the methods of the Theory of elasticity and by photoelastic experiments. There is no question that a stress concentration should be found here. It could be expected that a higher computed value would be found if a more closely spaced arch and cantilever grid were used.

Effect of the Concentration on the Safety of the Dam

A stress concentration should be expected to exist at the top of the dam at the abutment of any arch dam whose top arch ring is loaded. Trial load studies show such loading to be present in most cases. The situation is considered to be a normal one. For the present case the condition is exaggerated, however, by the softness of the abutment rock and the steepness of the abutment.

The threat which this concentration makes to the safety of the dam or to its proper functioning is not considered to be great. It is believed that the dam would behave well even though nothing were done about this concentration. It is likely that a certain amount of crushing would occur locally the first time the reservoir was filled and that this local crushing would provide relief from the high stresses which could otherwise occur if the abutment continued to behave elastically. A permanent set could result which might cause a crack to appear between the dam and the abutment near the top of the dam if the reservoir level was again drawn down. Such a crack would not cause leakage because it would close again if the reservoir level rose. If some diligent caretaker grouted it up, however, high stresses would appear when the reservoir was filled again.
The writer has seen two arch dams where there was some evidence of distress at the abutment near the top of the dam. It is my recollection that the abutment rock was a red granite in both cases. The appearance in one case seemed to be explained on the basis that the reaction had produced a tension crack as shown in Figure 1 and that a thin shell of material had spalled off. A similar crack can be seen in a glass marble which has been hit hard enough by another marble. Neither the strength of the dam nor the abutment should be impaired by the development of such tension cracks since the rock below the cracks would be sound. At both of the dams where this behavior of the rock was observed there was evidence of alkali-aggregate expansion and it could be expected that the thrusts applied to their abutments at the top of the dam were unusually high.

Laboratory tests have been made on Navajo sandstone loaded locally, as is the case with a dam abutment.

In the first test, the loads were applied to the flat surface of the sandstone through a high strength concrete cylinder about 2 inches in diameter. The 6-inch long by 6-inch-diameter sandstone specimen was embedded in plaster of paris in an 18-inch-diameter by 12-inch long galvanized iron mold for this test. The plaster of paris was reinforced by circular bars near the outside. In the second test, the loading was applied through a 2-inch-diameter by 1/2-inch thick steel disc. In the first test, the concrete cylinder failed at a stress of 8,450 pounds per square inch. The sandstone was indented about 1/32 of an inch. This is 2.27 times the strength of the sandstone as tested in the usual cylindrical form. With the steel block used in the second test, the sandstone was failed at 14,000 pounds per square inch. This is 3.77 times the unconfined compressive strength of the sandstone. The sandstone was deeply indented in this case. These tests indicate that the abutment will stand up without failure under loads much higher than the computed 969 lb/in² concentration.

Reduction of the Stress Concentration

If some way can be found to equalize the abutment stresses, an improved structural behavior should result. It should be pointed
out, however, that these provisions should be made with the utmost care lest the situation be made worse instead of better. Confirmation of this is afforded by a study recently completed in the Bureau Photoelastic Laboratory. In this case, the effect of a narrow vertical slot adjacent to the abutment was studied. A slot of this sort could be expected to relieve the stress concentration at the top of the dam, and it does, but a stress concentration of 4,600 pounds per square inch appears at the bottom of the crack.

To relieve 969 lb/in² at a cost of 4,600 lb/in² would be no bargain.

If the computed stress concentration is to be relieved without creating a stress concentration somewhere else, a certain principle should be carefully followed. This principle may be stated as follows: No stress concentration will be caused by a slot if (1) the base of the slot has zero width when the structure is under no stress; (2) if its width is continuous and increases continuously with distance from the base; and (3) if it closes progressively as the load is applied. The trouble with the slot of the photoelastic study was that it had a finite width at the base. An example of the application of this principle is afforded by the stresses at the contact of a car wheel and a rail. As load is applied to the wheel, the area of contact between it and the rail widens but the stress at the boundary of the area of contact is always zero. A slightly different case has a drastically different outcome. If a rigid flat die is pressed against the flat face of an elastic solid the stresses around the edge of the area of contact tend toward infinite values. The idea of a slot to relieve the stress concentration is basically sound and may be made effective if it is carried out in a way which does not provide an opportunity for stress concentrations to develop at the crack. A possible way of doing this is suggested below.

Slot Arrangement

The forming of a relief slot may possibly be accomplished in the following way:

1. By the method of Vogt (1) compute the abutment deformation near the top of the dam for the design conditions using the thrusts obtained from the trial load study.

2. Choose a suitable load distribution near the top of the dam and recompute the abutment deformations due to the
assumed load condition. Since the last grout lift will extend from elevation 3660 to the top of the dam at 3715, it is suggested that the new load condition be chosen in this interval. At elevation 3660, the new and old distributions should merge.

3. The difference in tangential displacement as computed in (1) and (2) represents the shape the relief slot should have to realize the assumed load condition. Some computations made by this writer indicate that this slot should taper from zero width at elevation 3660 to about 0.5-inch width at elevation 3715.

4. During the construction of the dam, leave two slots about 4 feet wide with an approximately 4-foot wide cantilever element between, as shown in Figure 2. The radial section should be near the abutment but at a sufficient distance therefrom to occupy the full section of the dam and to leave a sufficient area between the section and the abutment to support the concrete between the section and the abutment with acceptable stress intensities. At the upstream face the section should be sufficiently away from the abutment to permit the installation of water stops and to avoid knife-edge configurations. The cantilever should be reinforced and prestressed for reasons to be explained later.

5. With jacks arranged as shown, bend the cantilever to the right to the shape of one-half the slot width as computed in (3) and fill the right hand slot with concrete. This concrete should be attached to the concrete to the right by dowels but the face which abuts the cantilever should be greased so that it will separate from the cantilever.

6. Install jacks to bend the cantilever to the left and place concrete in the left hand slot as in 5, and remove the top jacks. The lower jacks may be concreted in.

7. The faces at a, b, c, and d should be keyed. Water stops should be installed along the upstream face, the top and the downstream face of Joints b and c to completely enclose them and exclude dirt. Joints b and c are to remain permanently open unless closed by arch thrust. They are not to be grouted.
When the cantilever returns to its unstressed position after removal of the jacks, two tapered joints should remain. The sum of the widths of these joints should be approximately that of the widths computed in (3). The work described in Items (4), (5), and (6) should be done after the dam has been grouted to elevation 3715 and while the water level in the reservoir is low.

Operation of the Joint

As the reservoir level rises, the top arch becomes loaded. As the thrust increases between elevation 3660 and 3715, the joints at b and c of Figure 2 close progressively. They become completely closed before the reservoir level reaches the design level. At the design level, the abutment pressures between elevation 3660 and 3715 should be somewhat below those chosen in Item (2), the difference being due to absence of earthquake forces. With earthquake forces included the abutment loads should reach the values chosen in Item (2).

Photoelastic Check

Because of the technical difficulties inherent in this procedure, the arrangement should be checked carefully by photoelastic means to determine the stress changes which will occur as the reservoir level rises and falls. It should be borne in mind that this is a new and untried device whose ability to accomplish the desired results should be completely confirmed before it is put to use.

Reasons for Prestressing the Cantilever

If the cantilever is constructed as an ordinary concrete beam, the application of the jack loads may be expected to crack the concrete on the tension side. This is customary behavior in concrete and no structural impairment would result. If these cracks are present, however, and the reservoir level rises above them, seepage through them may be expected to produce an unsightly discoloration on the downstream face. If the concrete is prestressed, no crack will be produced and the discoloration will be avoided. The prestressing therefore has no structural significance, its purpose is to avoid unsightly laitance marks on the face of the dam.
References

1. About the calculation of foundation deformations, by Dr. Frederick Vogt, 1925, Bureau of Reclamation, Technical Memorandum No. 77

2. A Study of Patterns of Abutment Movements Corresponding to Some Simple Patterns of Abutment Thrust, by N. M. Newmark, 1931, Bureau of Reclamation, Technical Memorandum No. 224

3. Theory of Elasticity, by S. Timoshenko, Bureau of Reclamation, Library No. 65/34

4. Beams on an Elastic Foundation, by Hetenyi, Bureau of Reclamation, Library No. 63/46.2 C-1


There are also some foundation deformation data in the Boulder Project Final Reports.
For a concentrated load \( f \) (14):

\[
W = \frac{f}{\pi L r}
\]

Suppose \( r = \sqrt{x^2 + y^2} \)

\[
p = \frac{f}{\pi r^2}
\]

Then

\[
W = \int_0^1 \int_0^1 \frac{p y}{\pi r^2} \frac{\xi (1-\xi^2)}{\sqrt{1 + \xi^2 + y^2}} dx dy
\]

\[
W = \frac{f (\pi R^2)}{\pi L r} \int_0^1 \int_0^1 \frac{\xi p y}{\sqrt{1 + \xi^2 + y^2}} dx dy
\]

\[
W = \frac{f (\pi R^2)}{\pi L r} \int_0^1 \int_0^1 \frac{\xi p y}{\sinh^{-1} \frac{y}{\xi}} dx dy
\]

\[
W = \frac{f (\pi R^2)}{\pi L r} \int_0^1 \int_0^1 \frac{\xi p y}{\cosh \frac{y}{\xi} x} dx dy
\]

\[
W = \frac{f (\pi R^2)}{\pi L r} \left[ y \cosh^{-1} \frac{y}{\xi} x + \frac{y \sinh \frac{y}{\xi}}{\xi} \right]
\]

Check

\[
\frac{dW}{dy} = \frac{f (\pi R^2)}{\pi L r} \left[ y - \frac{x}{\sqrt{1 + \xi^2 + y^2}} + \cosh^{-1} \frac{y}{\xi} + \frac{1}{\sqrt{1+\xi^2+y^2}} \right]
\]

\[
= \frac{f (\pi R^2)}{\pi L r} \cosh^{-1} \frac{y}{\xi} = \frac{f (\pi R^2)}{\pi L r} \sinh^{-1} \frac{y}{\xi}
\]
\[
\frac{d^2w}{dx^2} = \frac{E}{\pi E} \frac{1}{x^2 + y^2}
\]

\[
w = \frac{E}{\pi E} \left( \frac{1}{x^2 + y^2} \right)
\]

This is the deflection at the corner of a corner rectangle of dimensions \(X\) and \(Y\). Check dimensions.

\[
w = \frac{E}{\pi E} \frac{X^2}{Y^2} \quad \text{OK.}
\]

For a load \(S\) per unit of length at a distance \(y\) from the origin, the deflection is

\[
w = \frac{S(1-y^2)}{\pi E} \int_0^\infty \frac{dx}{x^2 + y^2}
\]

\[
w_1 = \frac{S(1-y^2)}{\pi E} \sinh \frac{X}{Y}
\]

For a loaded line extending from \(-X\) to \(+X\)

\[
w_2 = \frac{2S(1-x^2)}{\pi E} \sinh \frac{1}{Y}
\]
Memorandum

To: Chief Designing Engineer

From: R. E. Glover

Subject: Effect of vertical displacement--Glen Canyon Dam--Colorado River Storage Project

Introduction

The foundation rock at the Glen Canyon Dam site has a low elastic modulus. For this reason, abutment deformations are of more than ordinary importance at this site. To compensate for the expected yielding of these abutments, it is proposed to cool the dam below the final stable temperature configuration so that the horizontal expansion of the dam, as it returns toward the temperature of its environment, will offset the abutment yieldings. This will undoubtedly be of much benefit but the fact that the dam may also expand vertically is of importance because the upward movement will be restrained by the abutments. This restraint will cause stresses which will, to some extent, modify the stress system caused by the loads. To account for this factor, it is proposed to modify the tangential adjustment of the trial load studies. In the usual cases, when only a modest subcooling or none is used, the vertical displacements appear to be of minor importance and it is usual to ignore them. Comparisons with model test data confirm, for ordinary cases, this evaluation of their significance. With around 1/4°F of subcooling, these comparisons may no longer be valid.

Because the vertical adjustment has not been made before, we are now faced with the task of developing a new analytical technique at a time when the trial loads group is hard pressed to keep up with their work load. The techniques by which the vertical adjustment may be made seem to be fairly clear; but the task of finding, by trial and error, the stress patterns needed to accomplish the adjustment could easily consume more time than it would be convenient to spare.
Purposes of This Investigation

If we could find, by some means, a stress distribution conforming approximately to that due to the restraint of vertical expansion, it would greatly facilitate the work of making the vertical adjustment by reducing the number of trials needed for its accomplishment. The purpose of this investigation is to find such a stress system.

Procedure

For the purpose of obtaining an approximate knowledge of the stresses associated with the restraint of the vertical expansions, we assimilate the developed arch to a flat plate of uniform average thickness and subject it to the following treatment:

a. Compress the plate vertically by an amount sufficient to suppress the upward expansion due to temperature.

b. While in the compressed condition, attach the sides of the plate to an unyielding abutment.

c. Release the compressive forces at the top.

It now remains to construct an Airy's Function conforming to this idealization. The nature of this stress function is explained in Paragraph 13 of Timoshenko's Theory of Elasticity. It will be sufficient here to note that if the Airy's Function is denoted by $\phi$, the horizontal and vertical coordinates are $x$ and $y$ and the normal stresses acting in the coordinate directions $x$ and $y$ are $\sigma_x$ and $\sigma_y$, respectively, and the shear stress is $\tau_{xy}$, the conditions of equilibrium and compatibility are both satisfied if $\phi$ satisfies the relation:

$$\frac{\partial^4 \phi}{\partial x^4} + 2 \frac{\partial^4 \phi}{\partial x^2 \partial y^2} + \frac{\partial^4 \phi}{\partial y^4} = 0 \quad \cdots (1)$$

The stresses are related to the Airy's Function in the form:

$$\sigma_x = \frac{\partial^2 \phi}{\partial y^2} \quad \sigma_y = \frac{\partial^2 \phi}{\partial x^2} \quad \tau_{xy} = -\frac{\partial^2 \phi}{\partial x \partial y} \quad \cdots (2)$$
An Airy's Function suited to our purposes is

$$\phi = -E\alpha t \frac{1^2}{\pi^2} \sum_{n=1,3,5}^{n=\infty} \frac{1}{n^3} \sin \frac{n\pi x}{L} (e^{-\beta y} + \beta ye^{-\beta y})$$

Then

$$\frac{E\alpha t x^2}{2} \quad \ldots \quad (3)$$

$$\sigma_x = -E\alpha t \frac{4}{\pi} \sum_{n=1,3,5}^{n=\infty} \frac{1}{n} \sin \frac{n\pi x}{L} (\beta ye^{-\beta y} - e^{-\beta y}) \quad \ldots \quad (4)$$

$$\sigma_y = +E\alpha t \frac{4}{\pi} \sum_{n=1,3,5}^{n=\infty} \frac{1}{n} \sin \frac{n\pi x}{L} (e^{-\beta y} + \beta ye^{-\beta y}) - E\alpha t \quad \ldots \quad (5)$$

$$\tau_{xy} = -E\alpha t \frac{4}{\pi} \sum_{n=1,3,5}^{n=\infty} \frac{1}{n} \cos \frac{n\pi x}{L} (\beta ye^{-\beta y}) \quad \ldots \quad (6)$$

Where

$$\beta = \frac{n\pi}{L}$$

and L represents the width of the dam.
The origin is at the top of the dam at one end, \( x \) is measured horizontally toward the opposite end of the dam, and \( y \) is measured downward from the crest.

The remaining symbols have the following significance:

- \( E_c \): Young's modulus for the concrete
- \( \alpha \): The coefficient of thermal expansion of the concrete
- \( \beta = \frac{PA}{L} \)
- \( L \): The width of the dam
- \( t \): The temperature change
- \( n \): An integer

The stress notation used here follows Timoshenko:

![Stress Diagram](image)

- \( \sigma_x \) is the horizontal direct stress
- \( \sigma_y \) is the vertical direct stress
- \( \tau_{xy} \) is the shear stress

**Stresses**

On the basis that

- \( E_c = 3,000,000 \) \( \text{lb/in}^2 \)
- \( \alpha = 0.0000056 \) \( 1/\circ \text{F} \)
- \( t = 16 \circ \text{F} \)
- \( \mu = 0.2 \)

\( \mu_r = 0.06 \)
The complete restraint stress is \( E_0 \times t = 268.8 \text{ lb/in}^2 \).

With a completely rigid abutment, computations by these formulas for a dam with a crest length of 1,550 feet and a height of 700 feet yield a compressive stress of 71 lb/in\(^2\) at the base of the crown cantilever and a shear stress of 150 lb/in\(^2\) at the base of Cantilever B. (See data supplied to Consultants April 17, 1959.)

If the complete restraint stress acted at the base of the dam and was resisted by shear stresses at the abutments, computations by Vogt's formulas (T.M. 77) would indicate that the base would sink 0.142 foot and the abutments rise 0.154 foot. The total movement would then be 0.296 foot. These movements are due to elastic yielding of the foundation rock. If the crown cantilever were without restraint, a rise of 16\(^\circ\) F in temperature would cause its top to rise 0.0627 feet.

A comparison of abutment yieldings of 0.296 foot under the effect of the complete restraint stresses with the rise of the cantilever due to thermal expansion, if unrestrained, indicates that abutment yielding will absorb nearly all of the stresses which might arise due to restraint of the vertical expansion.

The magnitude of the actual stresses may be estimated by applying to the computed stresses a factor which is the ratio of rise with no restraint to the movement which could be produced by the complete restraint stresses. This ratio is

\[
\frac{0.0627}{0.296} = 0.212
\]

Then an estimate of the stresses produced by the vertical displacements would be:

Compression at the base of the crown
Cantilever (71) (0.14) = 15 lb/in\(^2\)
Shear stress at the base of
Cantilever B at elevation 3400
\((160) (0.14) = \frac{34}{\text{lb/in}^2}\)
Comments

Stresses computed for the highly idealized case used here could not be expected to check closely those found by a vertical adjustment made by trial load procedures. It is probable, however, that they are of the right order of magnitude. A good deal of time could probably be saved by using stresses computed from the Airy's Function as a first trial when making the vertical adjustment. These estimates indicate that the vertical expansion will be largely accommodated by abutment yieldings and that this factor will contribute in only a minor way to the stresses present in the dam.

Checks

This development has been checked by D. E. Morrisett.
Duplicate pages not scanned

See originals in folder
Memorandum

To: Chief, Dams Branch
Attention: E. R. Schultz

Through: Chief, Technical Engineering Analysis Branch
Chief Designing Engineer

From: H. Boyd Phillips and Ira E. Allen

Subject: Photoelastic stress analysis of two arch elements in the lower part, Glen Canyon Dam--Colorado River Storage Project

Introduction
This memorandum presents the results of a photoelastic stress analysis of two different arch elements in the lower part of Glen Canyon Dam, Colorado River Storage Project.

Based on the dam layout designated as Design A-20 by Concrete Dams Section, a two-dimensional study has been made of the arches at elevations 3250 and 3100. A uniform radial load has been considered to be acting over the upstream face of each arch.

To evaluate the effect of abutments with a low modulus of elasticity relative to the arch, three ratios of modulus of elasticity of arch to abutment, $E_c/E_a$, have been studied.

Results
The results of this stress analysis are given on Figures 1 and 2 for elevations 3250 and 3100 respectively. Stresses along the upstream and downstream face of the arch and along the center line and the abutment are shown. In addition the deflections at the center line and the abutment are given.

Stress curves and deflections are shown for $E_c/E_a = 1$, 2, and 5.

Total forces acting at the center line and abutment of the two arches have also been determined and are given in the tables below. Values are for an arch slice 1 inch thick.
<table>
<thead>
<tr>
<th>$E_c/E_a$</th>
<th>Normal Force on $G$ (lb)</th>
<th>Normal Force on Abutment (lb)</th>
<th>Shearing Force on Abutment (lb)</th>
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</thead>
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<tr>
<td>1</td>
<td>382,500</td>
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<td>556,800</td>
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<tr>
<td>2</td>
<td>397,100</td>
<td>450,900</td>
<td>562,500</td>
</tr>
<tr>
<td>5</td>
<td>412,600</td>
<td>434,200</td>
<td>548,800</td>
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</table>

**TABLE 2**

<table>
<thead>
<tr>
<th>$E_c/E_a$</th>
<th>Normal Force on $G$ (lb)</th>
<th>Normal Force on Abutment (lb)</th>
<th>Shearing Force on Abutment (lb)</th>
</tr>
</thead>
<tbody>
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<td>182,900</td>
<td>248,400</td>
</tr>
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<td>2</td>
<td>210,400</td>
<td>177,000</td>
<td>243,200</td>
</tr>
<tr>
<td>5</td>
<td>172,000</td>
<td>158,400</td>
<td>220,100</td>
</tr>
</tbody>
</table>

**Note:** Red figures are for trial load analysis of some arches.

**Conclusions**

It can be seen from a study of the results that a considerable readjustment of stress distribution occurs when the modulus of elasticity of the abutment becomes less than that of the arch. The deflection of the arch also increases with the decrease of the modulus of elasticity of the abutment.

It will also be noted that the stress distribution at the center line and abutment deviates somewhat from a linear variation.

The stresses at the upstream and downstream ends of the abutment are indeterminate because of the abrupt change of boundary conditions.

**Basic Data**

Dimensions based on Design A-20

Arches studied

<table>
<thead>
<tr>
<th>Elevation</th>
<th>Arch</th>
</tr>
</thead>
<tbody>
<tr>
<td>3250</td>
<td></td>
</tr>
<tr>
<td>3100</td>
<td></td>
</tr>
</tbody>
</table>

Uniform radial pressure acting 100 psi

Ratios of modulus of elasticity of arch to abutment studied ($E_c/E_a$) 1, 2, and 5
Technical Details

This stress analysis has been made experimentally. The photoelastic interferometer was used to determine the stresses and micrometer microscopes were used to evaluate the deflections.

The model material was Columbia Resin, CR-39, of 3/8-inch nominal thickness. Model scales were 1 to 2400 for the arch at elevation 3250 and 1 to 1722 for elevation 3100.

The reduced modulus of elasticity of the abutments was obtained by drilling closely spaced holes throughout the abutment area. The size and spacing of holes was determined from a series of tests in which the spacing was held constant and the hole size was progressively increased with $E$ being determined for each size of hole. From a plot of hole diameter vs $E$, the size of hole to give the desired ratio of $\frac{E_c}{E_a}$ was easily determined. Figure 3 is a photograph of the calibration specimen for determining the relationship of hole diameter to modulus of elasticity.

Figure 4 is a photograph of the model of the arch at elevation 3250. Shown is the model for $\frac{E_c}{E_a} = 1$ and for $\frac{E_c}{E_a} = 5$.

The desired uniform radial loading was obtained by means of a fluid loading shoe. A plastic sac was cast to the radius of the extrados of the arch and secured in a clamping block so that only the surface in contact with the arch was free to expand. The sac was filled with water and an adjustable reservoir provided a means to regulate the pressure in the loading device. A concentrated load was applied to the shoe which distributed it uniformly through the water to the upstream face of the arch. Details can be seen in Figure 4.

Personnel

This study was made under the general supervision of W. T. Moody. H. E. Willmann assisted in the experimental work and also prepared the figures.

W. Boyd Phillips

Ira E. Allen
Colorado River Storage Project
Glen Canyon Dam
Photoelastic Stress Analysis
Calibration of
Modulus of Elasticity
MEASURED DEFORMATION BEHAVIOR OF GLEN CANYON DAM

By Joe T. Richardson, M. ASCE

INTRODUCTION

The Bureau of Reclamation included in the plans and design of Glen Canyon Dam, as in those of its other major concrete dams, provisions for measuring the behavior of the structure. These measurements include determining by precise surveying methods (4) the deformation of the structure, and simultaneously detecting (by embedded instruments) the changing strains, temperatures, and other variations which occur within its mass.

The measurements of primary interest to engineers involved with surveying problems are those made on the dam by surveying methods. The use of surveying-type measurements, such as triangulation, offers a convenient and reasonably accurate means of determining the manner in which a large concrete arch dam deforms. Accordingly, representative results of that measurement program, as well as results from plumb line measurements, each of which program has been conducted for the past 4-1/2 yr to obtain the deformation history of the dam, will be presented herein.

The measurements from the embedded instruments are in a separate category. These measurements, when combined with laboratory-determined data that govern temperature, creep of concrete under sustained load, autogeneic growth, Poisson's ratio, and other effects, furnish the stress history of the structure.

The results of the precise surveying measurements are correlated with the

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Note.—Discussion open until February 1, 1969. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Surveying and Mapping Division, Proceedings of the American Society of Civil Engineers, Vol. 94, No. SU2, September, 1968. Manuscript was submitted for review for possible publication on November 8, 1967.

1Presented at the October 16-20, 1967, ASCE Annual Meeting and National Meeting on Water Resources Engineering, at New York, N. Y., where it was available as Preprint 525.


2Numerals in parentheses refer to corresponding items in the Appendix.—References.
results of the embedded instrument measurements. These data together furnish the complete behavior history of the dam.

GLEN CANYON DAM

Glen Canyon Dam, principal feature of the Bureau of Reclamation’s Colorado River Storage Project, is located on the Colorado River in northeastern Arizona (Fig. 1). It is a concrete arch dam, 710 ft high, having a crest length of 1,560 ft and containing 4,901,000 cu yd of concrete. It has two channel-approached spillways of 276,000-cfs capacity emptying into 41-ft-diam tunnels approximately 2,000 ft long. Glen Canyon Power plant, with a 900,000 kw capacity, is immediately downstream from the dam (6). The dam serves the dual purpose of impounding a 27,000,000 acre-ft storage reservoir to regulate the river flow for release to dams downstream, and creating head for power generation. For the construction of Glen Canyon Dam, two general classes of surveys, control and construction, were laid out (3).

PURPOSE OF MEASUREMENTS

The measurements by precise surveying methods were made periodically to determine with time the manner in which the dam would deform during the annual cycle of temperature to which it is subjected, to the applied load of reservoir water, and to determine whether there would be detectable effects of deformation in the foundation rock. Comparisons of these results with analytical results are needed to determine the validity of assumptions used for the design of concrete dams.

MEASUREMENT SYSTEMS

Systems for measuring deformation installed at Glen Canyon Dam include: (1) A control net of theodolite piers located downstream from the dam and on each rim of the canyon along with a base line located near one side of the canyon; (2) a system of targets in a grid pattern on the downstream face of the dam to which angular measurements from off-dam stations of the control net were made with a theodolite; (3) plumb lines installed in vertically formed wells at five locations, each extending between the top of the dam and locations near the foundation rock; and (4) wells extending into the foundation rock at three locations near the downstream toe of the dam. Measurements were made with the view of using first order methods and procedures insofar as applicable and practicable to the systems of triangulation.

TRIANGULATION

The behavior-control net downstream from the dam and the pattern of targets on the dam were arranged as shown in Fig. 2. Fig. 3 shows the upper and lower portions of the behavior-control net with lengths of lines indicated only to feet. This behavior-control net was tied into the major control net which
had been laid out for construction of the dam and other features of the project (Fig. 4).

FIG. 3.—LINE LENGTHS IN CONTROL NET

FIG. 4.—TRIANGULATION CONTROL NET AND CONTROL NET FOR DEFORMATION SURVEYS

In the behavior-control net, two piers, (Lola and Roma), which are located on the nearly vertical canyon walls on platforms, are the primary piers from which a majority of angles to the targets on the dam are turned. Angles to targets in the central part of the dam are also turned from piers on each side of the canyon, as Lila and Rita, Leta and Rena, or from other combinations of pairs of the six piers. Piers which are located on the dam (Lucy and Ruth) in combination with other piers were installed to aid with measurements on the dam near the abutments, and on rock in the keyways of the dam.

The grid system of targets on the dam is laid out to form horizontal and vertical rows which represent arch and cantilever elements of the dam. There are seven horizontal rows of targets and nine vertical rows of targets. The vertical rows of targets at five locations are in the same blocks as plumb lines. Fig. 5 shows the maximum section of the dam and the elevations of the horizontal rows of targets.

FIG. 5.—SECTION ON REFERENCE PLANE, BLOCK 12

Surveys.—Surveys have been conducted approximately quarterly each year, for the past 4-1/2 yr and to date, 17 surveys have been made. The survey from which all subsequent data are based was made from January, 1964 to March, 1964. That survey has been selected as the base datum survey for comparative purposes as it was the first survey which included all parts of the several systems. Also, during that period the installation of the five plumb lines was completed, and initial records from the system began. Three surveys which were made during the year prior to the selected base datum survey have been considered as preliminary and for orientation, as each survey covered only parts of the complete systems.
The surveys made subsequently to the base datum survey took place during January, April, July, and October. The October surveys occurred during the annual high-air-temperature periods, and the January, or January-February, surveys occurred during the annual low-temperature periods. During the high- and low-temperature periods, the dam's deformation rate of change from results of targets on the dam and from plumb lines had been noted to be a minimum, remaining approximately steady for periods of about two weeks. These conditions favored personnel who were making the measurements and furnished uniform results, as trial observations showed that by the time the last of the targets on the dam were observed, repeat measurements on the first targets observed showed only minor variations. The measurements were made during April and July when the dam was deforming at a uniform rate, and deformation data derived therefrom did not show the over-all uniformity obtained during January and October. Data from all surveys are not included herein due to their voluminous nature. Representative results are shown which illustrate the nature of the data derived, and furnish an indication of the accuracy obtained with the type of measurements.

The representative results from the targets show the deformation of the dam with respect to off-dam stations. The plumb line results, which are obtained weekly since installation, show the deformation of the dam with respect to the lowest station on a plumb line. Compatibility is shown between the two systems which are independent of each other and representative results from the measurements at the deformation wells show only slight random movement of the targets.

**Field Procedures.**—Procedures of interest are those used in conjunction with the measurements relating to base line, control net of theodolite piers, targets on the dam, and deformation wells. Those procedures were developed by field personnel. Procedures used with plumb line measurements are variations of procedures which were developed at other Bureau of Reclamation dams, and have resolved themselves to a standard routine. The measurements program was under the supervision of an engineer with background experience of a number of years in first order surveying practices.

**Base Line.**—The base line used for determining all line lengths in the control net was established late in 1962. It has a reduced length of 938+ ft and has been rechained at intermittent intervals of time. It forms with another line, also measured by chaining, two sides of a triangle. The third side of the triangle becomes a computed base line which is a side of a quadrilateral of the behavior-control net. All angles of the base line triangle are turned. The measured base line was established on fairly level ground and the ends of the base line were monumented with concrete piers.

Low-coefficient base line tapes which were compared with standardized tapes were used for all measurements. During the early period of measurements the tape was supported and tensioned in the conventional manner. Later the scheme was refined by field personnel, wherein tensioning posts of pipe and concrete tape-support piers were constructed at 100-ft intervals on the base line. At the half points between those piers, concrete tape-support piers were also constructed. At the tensioning posts, a portable tensioning device which had been fabricated by the field personnel was used. This device, shown in Fig. 6, consists of a wing nut and threaded rod with yolk which attaches to the tension scales and base line tape and to a bracket attached to the tension post. Adjustment for tension was easily made. Temperature measurements were obtained during chaining. Forward and backward chaining measurements were made and the mean obtained. The length of the measured base line for several surveys is shown in Table 1.

**TABLE 1.—BASELINE MEASUREMENTS**

<table>
<thead>
<tr>
<th>Date of Study</th>
<th>Date of Traverse</th>
<th>Baseline Length—Ft</th>
<th>Remarks</th>
</tr>
</thead>
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<tr>
<td>(1)</td>
<td>(2)</td>
<td>(3)</td>
<td>(4)</td>
</tr>
<tr>
<td>February, 1964</td>
<td>January, 1963</td>
<td>938.0170</td>
<td>Initial</td>
</tr>
<tr>
<td></td>
<td>March, 1963</td>
<td>938.0174</td>
<td>Run</td>
</tr>
<tr>
<td>October, 1965</td>
<td>August, 1964</td>
<td>938.0137</td>
<td>Mean 2 Traverses</td>
</tr>
<tr>
<td></td>
<td>January, 1965</td>
<td>938.0214</td>
<td></td>
</tr>
<tr>
<td>January, 1967</td>
<td>April, 1966</td>
<td>938.0165</td>
<td>After Restd Tape</td>
</tr>
<tr>
<td></td>
<td>January, 1967</td>
<td>938.0122</td>
<td></td>
</tr>
</tbody>
</table>

**Triangulation, Pier System.**—Initial angles were turned at each pier in the behavior-control net in December, 1962, and January, 1963 using a single first-order theodolite. Fig. 7 shows theodolite Station Rta. All piers for the theodolite were constructed as 4-ft high by 1-ft square concrete prisms dowelled with reinforcing steel into rock. A bronze plate with anchors was cast into the pier's concrete and mounted level on the top of each pier. On the pier's sur-
face, three V-shaped grooves spaced at 120° had been machined for mounting the theodolite. By removing the tri-branch from the theodolite, the leveling screws positioned directly in the grooves. The theodolite was made captive to a pier by two short lengths of chain with a slight amount of slack, which latch into eyes cast into a pier. Fig. 8 shows the theodolite, pier, pier plate, and a safety chain. Each pier was supplied with a sighting target which positioned in a tapered hole at the center of the pier plate (Fig. 9). During daytime obser-

vations, an umbrella or other shading was provided for the instrument. Initial angular measurements at piers on the control net were made taking the usual 16 positions at each pier. After several runs over the net had been made, the procedure was modified and eight positions were taken.

Project personnel determined by trial over a period of several months the most satisfactory time for measurements. After making early morning daylight basis during the winter and spring months, and nighttime observations from midnight to shortly after dawn were made during the summer and fall months. Early morning hours were found best suited for daytime observations, because heat waves rising from the dam and from the bottom of the canyon, as a result of absorption of solar radiation and updraft air movements, caused observational difficulties. The personnel began work before daylight, with base line measurements being made before sufficient light was available for sightings. When sufficient light was available, triangulation was begun.

**Triangulation Targets on Dam.**—Initial measurements to targets on the dam were made at lower elevations in February, 1963 and March, 1963, and to targets at lower elevations on the abutment rock in the keyways in September, 1963 to November, 1963. The first complete run made over all targets on the
GLEN CANYON DAM

The dam was from January, 1964 to March, 1964. In May, 1964 and June, 1964, the first run over the piers of the control net and the targets on the dam was made using two first order theodolites; a crew operated simultaneously from each side of the canyon.

### TABLE 2.—ANGULAR VARIATIONS IN SECONDS BETWEEN THEODOLITE STATIONS

<table>
<thead>
<tr>
<th></th>
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<th></th>
<th></th>
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<tbody>
<tr>
<td>Ford south base to</td>
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<td></td>
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</tr>
<tr>
<td>Rae</td>
<td>00.00</td>
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<td>00.00</td>
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</tr>
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<td>27.22</td>
<td>32.68</td>
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<tr>
<td>Rae</td>
<td>52.35</td>
<td>47.58</td>
<td>52.70</td>
</tr>
<tr>
<td>Rita to</td>
<td></td>
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</tr>
<tr>
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<tr>
<td>Mid</td>
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<td>42.30</td>
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<td>Roma to</td>
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<tr>
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<tr>
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<tr>
<td>Roma</td>
<td>35.68</td>
<td>35.75</td>
<td>35.25</td>
</tr>
</tbody>
</table>

### Leveling for Vertical Angles

Striding level corrections were made to observed angles on steeply inclined lines during observations on piers and on targets on the dam. In lieu of a striding level for the particular instrument...
used, the plate level which had a sensitivity of 7.0 sec per 2 mm graduation on the level vial was used to obtain the readings for corrections. The correction is the angle of inclination in seconds as shown by the plate level bubble when at a right angle to the line of sight, times the tangent of the angle, above or below the horizon of the point sighted upon. The effect of vertical angle on horizontal measurements and the method for correction is explained elsewhere (3).

**Time Requirements.**—During the initial 1-1/2 yr of the deformation surveys program, the time required to obtain necessary information was found to be more than had been anticipated. Accordingly, efforts to shorten the time required for a run over the system resulted in the use of two theodolites, and time saving devices and techniques. A run included measurements over the control net, measurements over the targets on the dam and at the abutments, and measurements to the deformation wells. During some of the runs, base line measurements were made also. By October-November, 1964, the program was in smooth operation with a complete run being made in about two weeks. By that time and at the periods thereafter, techniques and operations had been developed by field personnel so that by using two theodolites and two-man crews working on each side of the canyon, the pier triangulation could be made in six days and the measurements to the targets on the dam and on the abutment rock made in six days. The remainder of the time was required for base line measurements (when made), and for deformation well measurements. Tests were made to detect differences or deviations between the two theodolites. Also, personnel handling instruments were interleaved periodically. Communications between crews on opposite sides of the canyon were made by walkie-talkie radio. During these periods of measurement, angles at piers were turned taking eight positions in contrast to the earlier 16 positions. For intersections at targets on the dam, five positions were taken. During the earliest runs, a 3.0-sec rejection limit was attempted for turned angles, but experience showed that a 4.0-sec rejection limit had to be adopted. El. 3500 was used as the horizontal base datum for adjustments.

In the fall of 1965, transfer of the measurement program from project personnel to an operations office began, with the October run being made by intermixed personnel. Presently, the measurements are made by the operations office personnel with high quality of the measurements being maintained. During the initial three yr of measurements, techniques and aids which were developed included the use of a tension device for base line chaining and two first order theodolites which reduced observation time in half, thus compressing the time element wherein temperature change which occurred lessened possible movement of the measurement points.

**Tabulations.**—During the early stages of the surveys, angles and base line data were recorded in field books and extracts of these data made on standard Government tabulation sheets. During the later stages of the surveys, abstracts and summaries were made on forms which were devised to fit the system and to simplify computations. Abstracts of pier triangulation and triangulation for target intersections were tabulated and these data sent to Denver for computations.

During the initial triangulation runs, abstracts of angles and comparison with earlier runs were made as performed. In later triangulation runs, angles were turned during the morning and abstracts and comparisons made in the afternoon. If repeat measurements were required, they were made the next day. This procedure was found to save time and generally speed up the whole operation and seemed to increase efficiency. Closures of triangles were usually to 1.5 sec. Variations in seconds of angles with time between runs for several theodolite stations are shown in the Table 2 illustrating the consistency between measurements of the runs. Fig. 11 shows the variation in distance between piers on opposite sides of the canyon for those same three runs.

**PLUMB LINES**

The plumb lines are located in Blocks 4, 7, 12, 18, and 21. Each plumb line is installed in a formed vertical well which extends between the gallery, El. 3700.5, 7-1/2 ft below the top of the dam and gallery locations near the foundation. At the reading stations the wells are 12 in. in diam but due to construction alignment problems in forming, an increase to 14 in. in diam was allowed between reading stations. Reading stations are located on each plumb line at several intermediate elevations between the top and bottom elevations of a well. The reading stations are oriented in the plan so that measurements of deformation are in planes which are radial and tangential to the dam's axis. By this expedient the measurements require no trigonometric resolution to obtain deformation in the desired directions as was required with data obtained from plumb lines at earlier dams. The plumb lines were installed during February, 1964 and March, 1964.

During construction of the dam, the installation was aided by extending the wells to the top of the dam. On the top of the dam at each well a radial line which passed through the center of the well was established using a transit. On these lines, two temporary plumb lines were suspended which established a plane extending from the top of the dam to the lowest reading station in each.
well. At each reading station on a well, the apparatus was then oriented, aligned, and anchored with respect to the plane. The reference plumb lines were removed and single wires of 0.030-in. diam stainless steel then suspended for the permanent installation. As the well extension between the top of the dam and the suspensions of the plumb lines served no further purpose, they were sealed using watertight covers at the top of the dam.

Problems encountered after the apparatus was installed in the reading stations were minor. Readings of dam deflection with respect to the plumb lines are made using an optical type reading device, which consists of a microscope with crosshairs mounted on a screw-actuated sliding base. Data are tabulated on forms which are laid out in terms of computer requirements so that card punching can be made directly from the data sheets without transposition or trigonometric resolutions. Computations are then made by computer with results being obtained in terms of radial and tangential dam deflection at each reading station. Plotted records of deflection with time are then prepared.

FOUNDATION DEFORMATION WELLS

Vertical wells of 8-in. diam pipe were installed with their base elevations five ft below the foundation rock surface at three locations near the downstream toe of the dam. Monuments were located at the bottom of each well. The monuments were triangulated and tied into the coordinate grid system for the dam at the time of their installation. At progressive intervals after the installation, the monumented locations have been projected vertically using an optical plummet, and appropriate illumination to accessible locations which are retriangulated to the control net for determining whether detectable horizontal movement at the foundation is evident.

The target of the deformation well at the toe of the dam in the maximum section, Block 13, projected to the surface was included in the triangulation of the lower control net. The targets of the other two deformation wells, Block 7 and Block 18, projected to the surface were triangulated from the Block 13 well, and the two piers on the dam, Ruth and Lucy. The respective distances to these projected well target locations were measured by a base line tape from D-18.

COMPUTATIONS

Computations required for obtaining lengths of lines in the control net have been made thus far by conventional methods using eight-place trigonometric tables and a desk calculator. Triangles were adjusted so that compatible geometric figures would result and a computer program has been devised for these computations. Computer programs have been devised and used for determining results from targets on the dam data and for deformations from plumb lines (1).

RESULTS

Representative results of measurements from targets on the central part of the downstream face of the dam are shown on Figs. 12 and 13. Fig. 12 shows deformation plotted on arches and cantilevers for October, 1965 with respect to the base data measurements of February, 1964. Fig. 13 shows similar deformation for January, 1967. The deformation from the plumb lines for the same dates are shown as superimposed. No adjustments to compensate for the differences between initial data dates for the two systems of measurement have been made. During the time period of these results the reservoir has remained above el. 3500, increasing in two annual steps to a 50-ft greater head and then gradually decreasing to approximately el. 3500.

Differences which are noted between deformation results from targets and from plumb lines as shown on Figs. 12 and 13 are due to the time difference between initial data for each system. Also, the change in thermal conditions
of concrete will have a greater effect on target deformations at lower elevations on the dam due to the radial thickness of the dam at those target locations, than on plumb line deformations. A careful evaluation of temperature effect on each system and appropriate adjustment would bring the deformations from each system into close agreement. Close agreement is shown between the results from two systems of measurement in a dam of radial thickness less than that of Glen Canyon Dam (5).

The time plots of deformation at several elevations from the five plumb lines are shown on Figs 14 and 15. Time plots of the reservoir elevation and the concrete temperature measured five ft from the downstream face of the dam have been included on Fig. 14. These figures show the range of deformation during three annual cycles of temperature. Each year the top of the dam will be noted to move through a range of about 1/2 in. The total deformation at the top of the dam from the time period of the initial datum to the maximum temperature period in 1966 is approximately 1-1/2 in. upstream in Block 12, and less in Blocks 4, 7, 18, and 21. The slope indicated by the periods of cyclic deformation variation is due to the gradual warming of the mass concrete of the dam deforming it upstream. The upstream deformation, which is opposite in direction to that measured at other dams, is caused by the gradual increase in temperature of the dam's concrete in the absence of reservoir water load on the dam between the reservoir level shown, el. 3490 to el. 3535, and a full reservoir level, el. 3700. Table 3 shows the recorded temperature conditions which existed on radial gage lines of embedded instruments at the time of the representative results. Temperature values, left to right, are in the downstream direction.

Deformation results from targets on portions of the dam between the central portion as shown, and the abutments, indicate erratic deformation which is apparently due to the acute intersection angles at targets in those areas and in some cases to the steep inclinations of sight lines. Accordingly, those results are rejected as unacceptable and consequently are not shown.

Representative results from measurements at deformation well D-13 shows...
slight deformation upstream or downstream, 0.10 in. or less, with no apparent correlation to temperature cycle. At that location there is approximately 150 ft of sandfill at the downstream face of the dam between the dam and the powerplant—temperature is accordingly uniform. The low order of magnitude

TABLE 3.—CONCRETE TEMPERATURES, IN DEGREES FAHRENHEITA

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a For three values per block, temperature points are: 15 ft from upstream face; at center of section, and 10 ft from downstream face.

For six values per block, temperature points are: 15 ft from upstream face; at center of sections, and at 5 ft to each side of longitudinal joints; and 10 ft from downstream face.

of deformation may be an indication that measurable deformation is beyond the limitations of the specific measurements.

EVALUATION OF RESULTS

The layout, installation, operation, and results of the deformation measurements system have deviated in parts from initial planning as presented by the earlier paper (4). Those deviations were necessitated by conditions such as terrain of the area, lines of sight, strength of figures, intersections at targets, temporary construction obstacles, and variations in observation schedules, all of which had been recognized in the planning stage and over which control could not be exercised due to one cause or another.

The execution of these measurements illustrates the various surveying methods used to obtain results, and further indicates the planning required for items of equipment which can be devised to minimize or eliminate error and to save time. The ingenuity, cooperation, and care exercised by field personnel was recognized.

In general, the measurements have furnished the desired results from the several systems as anticipated initially. Those results show the manner in which the structure deforms due to annual temperature cycle and to the longer time-warming period of the concrete mass.

At some future date, when the reservoir has filled to normal operating level, the measurements which will be continued periodically, will further afford the means for determining the part of deformation caused by the added water load on the dam. Theoretical investigations presently in progress will furnish results for comparison with those from the measurements.

CONCLUSIONS

Results of the deformation measurements which have been made to date on Glen Canyon Dam lead to the conclusion that the surveying methods of measurements offer a convenient and reasonably accurate means of determining the deformation of a large mass concrete arch dam. Further conclusions are: (1) More than one base line is desirable in a behavior triangulation net, and (2) the time-consuming procedure of turning angles for precise triangulation probably can be economically supplemented in cost and in time by using presently available electronic distance measuring devices. With these devices the result would be that all measured lengths in the control net would be comparable to computed base lines.

Examinations of sets of data indicate that measurements of any dam should be initiated either during the high temperature or low temperature period. Experience gained from these measurements at Glen Canyon Dam can be used to advantage at other dams to be constructed in the future.

ACKNOWLEDGMENTS

The information obtained, the equipment developed, and methods used for the measurements represent the combined efforts of many individuals. Keith Jones computed the results from the behavior triangulation net. L. H. Roehm devised the computer programs for use with target measurements and with plumb line measurements, and supervised the computations of those results. These engineers and the writer are staff members of the Bureau of Reclamation's Chief Engineer, B. P. Bellport, at Denver, Colorado.

Richard J. Hannon, Maynard J. Willis, J. J. Johnson, Charles Murphy, and others contributed to the layouts, made the installations at the dam site, and obtained measurements from the behavior systems.
APPENDIX.—REFERENCES


ABSTRACT: Representative results obtained during 3 years of measurements from each of two systems installed to determine the deformation behavior of the Bureau of Reclamation's Glen Canyon Dam, a large concrete arch structure, are presented. The deformation is caused by water loading and by changing temperature of the mass concrete. Measurements are by precise triangulation from piers downstream from the dam to targets on its face, using a theodolite, and by observations on five plumb lines installed in formed vertical wells in the dam. Also, monuments in three wells which extend into the foundation near the toe of the dam are triangulated. The results show the deformation of the dam is comparable from the measurements by both systems which are independent of each other. The annual range of deformation from the plumb lines during a 60° temperature range is approximately 1/2 in. at the top of the dam in its maximum section and the deformation of the dam is upstream approximately 1-1/2 in. for the period of record. The measurement system layout and photographs of apparatus are shown. The programs of periodic observations, field procedures, techniques, timesaving and error minimizing equipment devised by field personnel, computing methods, and results are described.

The
Colorado River Storage Project

and Participating Projects

DEPARTMENT OF THE INTERIOR
Fred A. Seaton, Secretary

BUREAU OF RECLAMATION
Floyd E. Dominy, Commissioner
The Frontier...

The Upper Colorado River Basin is, in many ways, appropriately described as frontier. The Upper Basin is a scenic and rugged plateau land. Snowcapped mountains mark much of the Basin's rim. Rushing, white water streams flow from the mountains to become muddy, silt-laden rivers in the deep canyons that lace the central part of the Upper Basin.

SPECTACULAR, BUT DIFFICULT, AREA

The very features that make the Upper Basin spectacular and scenic to today's tourists made the Basin formidable to the pioneers and have made it slow to grow and develop.

The high, "tough-to-cross" mountain rims, the wide and dry plateaus cut by sweeping lines of towering cliffs, and the ever-present canyons all combined to defy settlement and use on any large scale. The Oregon Trail crossed the northern part of the Basin where the mountain fringes are lower. The Santa Fe Trail stayed to the south away from the canyons of the Colorado River system. The modern highway equivalents of these historic trails—U.S. 30 and 66—still are the most used east-west highways.

COLORADO NOT CONQUERED UNTIL 1869

In 1869, the year that the first transcontinental railroad was completed, the Green and Colorado Rivers were explored and mapped for the first time by Major John Wesley Powell. In fact, the Powell party was working on their newly launched boats at Green River, Wyo., when the first Union Pacific train from San Francisco crossed the trestle above them in May 1869.

Even by 1940—70 years after Powell's first exploration—development of the Upper Colorado River Basin was still meager. Grand Junction, Colo., was the largest city with a population of 12,500. Only four Upper Basin cities had populations of more than 5,000 in an area equal in size to Illinois and Wisconsin combined.

The Upper Colorado River Basin may have been late in exploration, slow in settlement, and limited in development, but the Upper Basin boldly faces a new future which will see its many resources utilized on an ever-widening scale.

The Basin...

DRAINAGE AREA........... 110,000 square miles
LENGTH: Upper Colorado River........ 640 miles
Green River................ 730 miles

COLORADO RIVER:
Average yearly runoff (virgin flow):
From Upper Basin........... 15,638,000 acre-feet
To Gulf of California...... 16,973,000 acre-feet

The Future...

The future of the Upper Colorado River Basin lies in its resources. The most important resource is water—water which is corralled and put to work rather than allowed to plunge wildly toward the sea, wasting its energy in the rapids of the colorful canyons.

THE RESOURCES ARE MANY

The Upper Colorado River Basin has the water—it has land to be irrigated—it has canyons with dam sites where much water can be stored and where hydroelectric power can be produced—it has petroleum, coal, and natural gas—it has oil shales and rare hydrocarbons—it has mineral resources of uranium and other atomic ores, of many strategic metals, of phosphate and other needed nonmetallic ores.

But, these many resources are largely dormant—sleeping giants yet to be awakened. The future will see the use of Upper Basin resources on an ever-widening scale under a development program which will bring together the resources of water, power, land, and minerals.

BASINWIDE DEVELOPMENT THE ANSWER

The key that will unlock the doors leading to full development and use of Upper Basin resources is the basinwide program of the Department of the Interior, because this program centers on water—the basic resource.

The program will result in control of the flows of the Upper Colorado River in large reservoirs, will produce sizable blocks of marketable hydroelectric power, will bring about irrigation of lands from Upper Basin tributary streams, and will supply water for industrial, municipal, and other beneficial uses.

This basinwide program—Colorado River storage project and participating projects—was authorized by the Congress in 1956. Construction by the Bureau of Reclamation began in 1956 on Glen Canyon Dam, and in 1958 on Flaming Gorge and Navajo Dams. In 1959, work started on the first two participating projects—Paonia project in western Colorado and the Vernal unit of the Central Utah project in northeastern Utah.

The future begins to unfold for the Upper Colorado River Basin.
The Upper Basin States are entitled to the beneficial consumptive use of Colorado River water pursuant to the provisions of the Colorado River Compact of 1922. The 1922 compact apportions the use of Colorado River water between the Upper and Lower Basins and requires the Upper Basin to pass to the Lower Basin for its use substantial quantities of the full flow of the river in each successive 10-year period. Thus, the availability of the Upper Basin States share depends on regulation of stream flows by the several storage units of the Colorado River storage project. Increased beneficial consumptive use of Upper Basin water will be accomplished on the participating projects.

Use of the Upper Basin States apportionment of the Colorado water will be in accordance with the Upper Colorado River Basin Compact of 1948. This compact divides the consumptive use of water as follows: Colorado, 51\(\frac{1}{4}\) percent; New Mexico, 11\(\frac{1}{4}\) percent; Utah, 23 percent; Wyoming, 14 percent; and Arizona, an amount not to exceed 50,000 acre-feet per year.

### THE STORAGE PROJECT

#### GLEN CANYON STORAGE UNIT

**DAM:**
- Type: Concrete arch
- Height: 700 feet
- Crest length: 1,500 feet
- Volume: 4,943,000 cubic yards

**RESERVOIR:**
- Capacity: 28,040,000 acre-feet
- Length: 186 miles
- Area: 254 square miles
- Normal water surface elevation: 3,700 feet

**POWERPLANT:**
- Total installed capacity: 900,000 kilowatts
- Number of generating units: 8
- Capacity of each unit: 112,500 kilowatts

#### FLAMING GORGE STORAGE UNIT

**DAM:**
- Type: Concrete arch
- Height: 490 feet
- Crest length: 1,180 feet
- Volume: 922,000 cubic yards

**RESERVOIR:**
- Capacity: 3,739,000 acre-feet
- Length: 91 miles
- Area: 66 square miles
- Normal water surface elevation: 6,040 feet

**POWERPLANT:**
- Total installed capacity: 108,000 kilowatts
- Number of generating units: 3
- Capacity of each unit: 36,000 kilowatts

#### NAVAJO STORAGE UNIT

**DAM:**
- Type: Earth and rockfill
- Height: 400 feet
- Crest length: 3,700 feet
- Volume: 26,300,000 cubic yards
- Reservoir capacity: 1,709,000 acre-feet

#### CURECANTI STORAGE UNIT

**DAMS:** A series of 2 or 3
**RESERVOIRS:** Total capacity, about 1,000,000 acre-feet

**POWERPLANTS:**
- Total installed capacity, about 160,000 kilowatts

### PARTICIPATING PROJECTS

- **NUMBER OF PROJECTS AUTHORIZED:** 11
- **NEW LAND TO BE IRRIGATED:** 115,000 acres
- **OLD LANDS TO RECEIVE MORE WATER:** 231,000 acres
- **WATER FOR CITIES AND INDUSTRIES:** 48,800 acre-feet
- **POWERPLANTS:** Total installed capacity, 61,000 kilowatts

[1959]
KEYSTONES: THE UPPER COLORADO DAMS

GLEN CANYON DAM

Like keystones of arches, the great dams in the Upper Colorado River Basin will rise between canyon walls and put to productive use the waters now coursing uncontrolled from the snow fields of the Rocky Mountains.

Without the large dams and reservoirs, increased Upper Basin use of water would be limited. Annual flows which may vary from 4 million to 22 million acre-feet will be evened out. Lower Basin water rights can then be met on a firm basis. At the same time, increased diversion and use of water can be undertaken throughout the Upper Basin to make possible widespread and sound growth.

Without the power production made possible by the large dams and reservoirs, basinwide development and use of water resources could not be self-liquidating. Wealth-creating projects to provide irrigation water supplies could not be built in the past because the farmers could not guarantee to repay the costs of such projects. However, the revenues from the sale of power produced at the large dams will provide funds to assist in the repayment of all costs. Thus, the long-run direct benefits from productive use of water and the myriad of indirect benefits to the Nation can be realized.

Glen Canyon, Flaming Gorge, Navajo, and the Curecanti Dams make possible basinwide development and use of Upper Basin natural resources.

Glen Canyon will be the colossus of the Upper Colorado River. It will be the third highest dam in the world, will create the third largest reservoir in the world, and will have the seventh largest hydroelectric powerplant in the United States.

The dam is located on the Colorado River within 15 miles of the lower margin of the Upper Basin. Except for one small stream (Pari River), all tributaries in the Upper Basin enter the Colorado River above Glen Canyon Dam. It is as if the dam were located in the stem of a gigantic funnel which collects water from the entire Upper Basin.

The dam will rise about 700 feet above bedrock between the vertical walls of massive red sandstone. Its 1,500-foot curved crest will span the 1,200-foot straight line distance between the canyon walls. It will be 300 feet thick at the base and 35 feet at the crest. With approximately 4,943,000 cubic yards of concrete in the dam proper and a total of 5,400,000 cubic yards in the dam and related works, Glen Canyon will be one of the largest concrete dams in the world.

Construction began at the dam site with a blast from the canyon wall on October 15, 1956, just six months after authorization by the Congress. Excavation of the right diversion tunnel was the first construction undertaken.

On April 11, 1957, exactly one year from the date of authorization, bids were opened for the prime contract for construction of the dam and powerhouse. The contract was awarded on April 29 to the Merritt-Chapman and Scott Corp. on a bid of $107,955,122—more than twice as large as any previous Bureau of Reclamation construction contract. Construction under the prime contract is scheduled for completion early in 1964. Generating units will be installed and the powerplant completed under a separate contract. The powerplant will have eight generators, of 112,500 kilowatts capacity each, for a total of 900,000 kilowatts.

Drilling and lining of the two diversion tunnels and construction of the coffer dams to carry the river around the dam site were completed in 1959. Then, following excavation of the foundation, the placing of concrete in the dam proper will center in the years 1960, 1961, and 1962. According to present plans, the first production of power at the dam could occur in 1964.

Nearly one million tons of manufactured or processed materials will be used in the dam and powerplant. Industrial plants in the Midwest and East will supply a large part of such materials. All supplies must be trucked to the dam site from the nearest railhead, which is 135 miles away.

First blast from the canyon wall, October 15, 1956—Glen Canyon Dam site.
FLAMING GORGE DAM

Flaming Gorge Dam will regulate the flows of the upper Green River, which drains an area of 15,000 square miles. The dam will be located on the north flank of the Uinta Mountains where the Green River has cut a gorge which is more than 1,500 feet deep in places.

The dam will be located about 2 miles below Ashley Falls, which was visited in 1825 by General William Henry Ashley, early fur trader in the Green River Basin. The lower 25 miles of the Flaming Gorge Reservoir will be confined to a scenic canyon, bordered by pine forests with cold mountain streams and abundant wildlife.

The Utah-Wyoming State line is just 6 miles north of the dam site. The nearest towns are Green River and Rock Springs in Wyoming, and Vernal in Utah. The railhead for Flaming Gorge Dam is at Green River, 60 miles from the dam site.

Flaming Gorge Dam will be a concrete arch dam rising 490 feet above bedrock and 450 feet above the river level. The graceful curved crest of the dam will be 1,180 feet long. The maximum thickness of the dam at its base will be 150 feet; it will taper to a minimum thickness of 20 feet near the crest.

The powerplant, which will be situated at the foot of the dam, will contain three generating units of 36,000 kilowatts capacity each. When all generating units are installed, the total rated capacity of the Flaming Gorge Powerplant will be 108,000 kilowatts.

First construction was undertaken early in 1957 with the building of the access road from Linwood, Utah, to the dam site area.

The prime contract for Flaming Gorge Dam was awarded on June 18, 1958, to the Achen Dam Contractors. About five years will be required to build the dam and powerplant. A single diversion tunnel 1,100 feet long and 25 feet in diameter will carry the Green River flows around the dam site during construction. Almost one million cubic yards of concrete will be placed in building Flaming Gorge Dam.

The 91-mile-long reservoir to be created by Flaming Gorge Dam will offer excellent recreational opportunities. This high-level lake (6,040 feet in elevation) will be reached easily from cross-continental highways U.S. 30 and 40.

NAVajo DAM

Navajo Dam will make possible the direct diversion of water for the Navajo Indian irrigation project. This 110,000-acre irrigation development on the Navajo Indian Reservation is an urgently needed agricultural expansion to help support the increasing Navajo Indian population.

By controlling the flows of the San Juan River, Navajo Dam will make possible the upstream diversion of San Juan water for use in New Mexico along the Rio Grande River under the proposed San Juan-Chama project.

Navajo Dam will be a large earth and rockfill dam 400 feet in height, 3,700 feet long at the crest, and about 26,300,000 cubic yards in volume. It will be the second largest earth dam constructed by the Bureau of Reclamation.

The prime contract for Navajo Dam was awarded on June 25, 1958, to a joint venture group composed of Morrison-Knudsen, Henry J. Kaiser, and the F and S Contracting Co. About 1½ years will be required to complete the dam.

CURECANTI DAMS

Engineering and economic studies required by the April 11, 1956, act of Congress authorizing the Curecanti Dams will lead to a plan for construction of a series of dams in a 40-mile reach of the Gunnison River. The Curecanti Dams would provide storage capacity for controlling the flows of the Gunnison River and for the production of hydroelectric power.

A series of earthfill and/or concrete dams will be built to utilize about 940 feet of drop in elevation in the deep, 40-mile long canyon of the Gunnison River. A total installed capacity of about 160,000 kilowatts would be possible at the series of dams.

Storage of Gunnison River water would be provided by the uppermost dam in the series, which would create the Blue Mesa Reservoir. The additional dams in the series would be built downstream to operate powerplants utilizing water released from the Blue Mesa Reservoir.
THE COLORADO RIVER STORAGE PROJECT

AND PARTICIPATING PROJECTS: A basinwide program for the development of the water and power resources of the Upper Colorado River.

THE PARTICIPATING PROJECTS

The participating projects are the projects by which we achieve our goals of development and water supplies. These direct use consumer water and create wealth.

Water will be supplied in several ways. For those projects which will provide for dry lands to create new farms and new homes for farm families. For every farm family making a living on a new irrigated farm, economic support is created in a nearby town or at least one additional farm is continually active.

Supplemental water supplies will be made available to irrigated lands now subject to adverse times in the irrigation season. Such shortages reduce agricultural production and thus lower the income and standard of living of the farm families dependent upon such lands.

Water will be supplied to growing cities and to new industries, using natural resources which are now largely underdeveloped.

Without adequate and dependable water, wasteful developments result in new opportunities for our growing population are limited in the arid plains of the Upper Colorado River Basin.

LA BARGE PROJECT
- 4,500 acres in new farms
- 8,000 acres for old farms
- 2,000 acres for town sites

SEEDS RIVER PROJECT
- 5,000 acres in new farms
- 1,500 acres for town sites

LYMAN PROJECT
- 5,000 acres in new farms
- 1,000 acres for town sites

CENTRAL UTAH PROJECT
- 20,000 acres in new farms
- 10,000 acres for town sites

SALT PROJECT
- 2,000 acres in new farms
- 5,000 acres for town sites

EMERY COUNTY PROJECT
- 1,000 acres in new farms
- 5,000 acres for town sites

PADOAN PROJECT
- 2,000 acres in new farms
- 5,000 acres for town sites

SMITH FORK PROJECT
- 1,000 acres in new farms
- 2,000 acres for town sites

FLORIDA PROJECT
- 2,000 acres in new farms
- 1,000 acres for town sites

PINE RIVER PROJECT EXTENSION
- 1,000 acres in new farms

HARRIMAN PROJECT
- 5,000 acres in new farms

BENEFITS

IRRIGATION WATER

Direct irrigation benefits will result from supplies of water. For years and industrial use on the Central Utah project. Water will be used in the growing industrial complex lying at the western end of the Wasatch Mountains in Utah.

MUNICIPAL AND INDUSTRIAL WATER

The presently authorized project calls for the supply of water for towns and industries only on the Central Utah project. Water will be used in the growing industrial complex lying at the western end of the Wasatch Mountains in Utah.

The growth of population and the increased industrialization expected in the Upper Basin States, the need for additional water for industrial and commercial development will be met by the completion of the Colorado River Storage Project. With the completion of this project the irrigation of the land in the Upper Basin will permit increased consumptive use for municipal and industrial purposes.

HYDROELECTRIC POWER CAPACITY

The powerhouse capacity totaling about 12,000,000 kilowatts at the authorized Glenn Canyon, Flaming Gorge, and Cearce storage units will meet only a small portion of the future power needs of the Upper Basin States.

RECREATION AREAS

Unperfected opportunities for recreational development and land use improvement on the west mountain areas to be created by the dam in the Upper Colorado River Basin.

Glen Canyon Reservoir, in particular, will have areas on the first of the unclassified scenic canyon areas being used and enjoyed by only a few people.

Studied to develop are areas suitable for recreational development and land use, including the National Park Service and other appropriate agencies. Recreational areas of substantial interest will be developed in the Upper Colorado River Basin.
GLEN CANYON DAM
The Glen Canyon Dam, Powerplant, and Reservoir, which will be known as Lake Powell in honor of the western explorer and geologist, John Wesley Powell, are the principal storage and power features of the Upper Colorado River Storage Project. The dam will be on the Colorado River in Arizona, 13 river miles below the Utah border. Lake Powell will store about 28,000,000 acre-feet of water—next in size to Lake Mead, downstream. America's largest man-made lake—to help solve a water resource development problem in the Colorado River Basin.

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Glen Canyon Storage Unit will be the keystone in this whole structure. The dam spans the river near its exit from the Upper Basin, as if in the spout of a great funnel where it can control all of the water in the funnel's cone—the Colorado's own flow and all that its tributaries feed into it upstream from the dam. The powerplant will generate about 75 percent of the project's total power and the reservoir will contribute about 75 percent of the water storage that the Congress authorized in 1956 as initial development for the Upper Basin. This reservoir or lake, extending 186 miles behind the dam, will be flanked by remarkably beautiful scenery. The Nation's gain in new public and private wealth will be tremendous.

The Federal Government will finance the project, but the people who use the water and power will repay about 99 percent of the cost—about two-thirds with interest.
CONSTRUCTION

Glen Canyon Dam, like all large Reclamation dams, is being built by private construction companies that are awarded contracts by competitive bidding. The prime contract, totaling $107,955,122, was awarded to the Merritt-Chapman and Scott Corporation of New York City, April 29, 1957. It provides for construction of the dam and powerhouse and is the largest single contract the Bureau has ever awarded and probably the largest for any type of construction project.

By June 1960, the contractor had completed the diversion and spillway tunnels, lined them with concrete, built the coffer dams (temporary earth structures diverting the river around the damsite during construction), and excavated the foundation of the dam. First placement of concrete in the foundation of the dam and powerhouse was observed by public ceremonies at the damsite on June 17, 1960. Initial storage of water behind the dam is scheduled for early 1962.

A $6,392,000 contract for the manufacture of eight 155,500-horsepower, 150-r.p.m., vertical-shaft hydraulic turbines for the powerplant has been awarded to the Baldwin-Lima Hamilton Corp. Additional contracts for generators and other adjuncts will be awarded later to equip the dam and powerplant. Glen Canyon’s first hydroelectric generating unit is scheduled to go on line in 1964.

BRIDGE AT THE DAMSITE

Bridges were among the first essentials at Glen Canyon damsite. The vertical walls of the Canyon rise about 700 feet above the river. The distance from rim to rim is only 1,200 feet in a straight line, but it is about 190 miles by road.

The Glen Canyon Bridge, a spectacular rim-to-rim highway, spans the canyon immediately downstream from the dam. It is the highest and second-longest steel arch bridge in the United States; its 1,028 foot arch stands 700 feet above the river. The deck is 1,271 feet long. The roadway is 30 feet wide and is paralleled by 4-foot sidewalks. The bridge was dedicated and opened to public use on February 20, 1959.

Materials and equipment are transported to the canyon floor by highlines—heavy cableways stretched between towers, two on each rim. Loads of 50 tons are lowered from them on pendant hooks.
The Glen Canyon of the Colorado River is an unusually placid, 162-mile reach from Hite, Utah, to Lees Ferry, Arizona. Major John Wesley Powell, who headed the first expedition down the river in 1869, named it Glen Canyon because of the occasional oak glens along its banks and at its junctions with tributaries.

The 186-mile-long Glen Canyon Reservoir (Lake Powell) will extend upstream into Cataract Canyon. The lake and adjoining lands have been established as the Glen Canyon National Recreation Area under the National Park Service of the Department of the Interior. This is the status of Lake Mead and its environs behind Hoover Dam.

The Park Service will soon undertake construction of recreational facilities for public use as Lake Powell begins to fill in 1962. The Glen Canyon Recreation Area promises to become one of the Nation's outstanding tourist attractions.
Lake Powell, behind Glen Canyon Dam, will be flanked by varied and beautiful scenery.

HIGHWAYS TO GLEN CANYON DAM.—Excellent, new, paved highways have been built to the Glen Canyon damsite. A 76-mile highway through the highly scenic area has been built from Kanab, Utah, to the damsite. A new 25-mile highway extends northward from Bitter Springs to the damsite. Both of these highway links connect with the Glen Canyon Bridge to form a new link in U.S. Highway 89. The Glen Canyon Bridge was completed in February 1959.

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* Two 16 mm. color, sound films, CANYON CONQUEST and KEY TO THE FUTURE, are available upon request for showing to school and civic groups, clubs, and other public gatherings. Both films show men and machines at work on this challenging Reclamation project. Send your requests to: U.S. Department of the Interior, Bureau of Reclamation, P.O. Box 360, Salt Lake City 10, Utah. Eastern area residents may write to the U.S. Department of the Interior, Bureau of Reclamation, Washington 25, D.C.

[1961—REVISED]

U.S. GOVERNMENT PRINTING OFFICE: 1960 OF—578577
GLEN CANYON DAM
THE PROJECT AND THE DAM

The Glen Canyon Dam, Powerplant, and Reservoir, which will be known as Lake Powell in honor of the western explorer and geologist, John Wesley Powell, are the principal storage and power features of the Upper Colorado River Storage project. The dam will be on the Colorado River in Arizona 13 river miles below the Utah border. Lake Powell will store about 28,000,000 acre-feet of water—next in size to Lake Mead, downstream, America’s largest man-made lake—to help solve a water resource development problem in the Colorado River Basin.

The crux of the problem is the division of the river’s water between the Upper and Lower Basins of the Colorado River, as provided by an interstate compact. The volume of water flowing down the Colorado fluctuates sharply from year to year. Consequently, there must be long-term holdover storage capacity in order to meet downstream needs and compact requirements—including requirements for Mexico under an international treaty—and still permit the Upper Basin States to deplete the river for upstream use.

This problem will be solved by construction of a system of storage dams and reservoirs in the Upper Basin, of which Glen Canyon, the largest, is one of four initial units authorized. Only 15 miles above the dividing line between the Upper and Lower Basins, it will store no water for use upstream or in the immediate vicinity of the dam, but is the principal unit storing water to regulate the river and thereby fulfill compact commitments to the Lower Basin. The sale of hydroelectric energy generated at the multipurpose dams will return practically all of the cost of the project and a large part of the cost of 11 participating irrigation projects authorized for initial Upper Basin development. That, in general, is how Glen Canyon Dam on the Colorado and three other initial dams on its tributaries will aid in developing the area. The participating projects just referred to, and scattered throughout the Upper Basin (11 of them authorized for construction), will irrigate about 130,000 acres in new farms and improve irrigation on about 230,000 acres in old ones. Some 25 other projects are under various phases of study. Farming, in consequence, will greatly increase.

Water from the 4 big storage reservoirs will, as planned, turn generators of about 1,200,000 kilowatt capacity, and industry will use the power. Mineral deposits of inestimable value, uranium among them, will be mined. Flood control and navigation on the Colorado will be improved, and the nation’s playgrounds will be greatly enlarged, for some of the world’s finest recreation places will lie along the shores of the reservoirs or lakes that will form behind the dams.

Glen Canyon Storage Unit will be the keystone in this whole structure. The dam spans the river near its exit from the Upper Basin, as if in the spout of a great funnel where it can control all of the water in the funnel’s cone—the Colorado’s own flow and all that its tributaries feed into it upstream from the dam. The powerplant will generate about 75 percent of the project’s total power and the reservoir will contribute about 75 percent of the water storage that the Congress authorized in 1956 as initial development for the Upper Basin. This reservoir or lake, extending 186 miles behind the dam, will be flanked by remarkably beautiful scenery. The whole project’s payoff in new public and private wealth and benefits will be tremendous.

The Federal Government will finance the project, but the people who use the water and power will repay about 99 percent of the cost—about two-thirds with interest.
Construction

Glen Canyon Dam, like all large Reclamation dams, is being built by private construction companies that are awarded contracts by competitive bidding. The prime contract, totaling $107,955,122, was awarded to the Merritt-Chapman and Scott Corporation of New York City, April 29, 1957. It provides for construction of the dam and the powerhouse, and is the largest single contract the Bureau has ever awarded.

By spring 1960, the contractor had completed the diversion and spillway tunnels, lined them with concrete, built the coffers dams (temporary earth structures diverting the river around the damsite during construction), excavated the foundation for the dam, and was about ready to place the concrete in the foundation of the dam and powerhouse.

Additional contracts for generators, turbines, and other adjuncts will be awarded later to equip the dam and the powerplant. When both are complete, probably in 1964, power production will begin.

Bridge at the DamSite

Bridges were among the first essentials at Glen Canyon damsite. The vertical walls of the Canyon rise about 700 feet above the river. The distance from rim to rim is only 1,200 feet in a straight line, but it is about 190 miles by road.

The Glen Canyon Bridge, a spectacular rim-to-rim highway, spans the canyon immediately downstream from the dam. It is the highest and second-longest steel arch bridge in the United States; its 1,028 foot arch stands 700 feet above the river. The deck is 1,271 feet long. The roadway is 30 feet wide and is paralleled by 4-foot sidewalks. The bridge was dedicated and opened to public use February 20, 1959.

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The 186-mile-long Glen Canyon Reservoir (Lake Powell) will extend upstream into Cataract Canyon. The lake and adjoining lands will be established as a national recreation area under the National Park Service of the Department of the Interior. This is the status of Lake Mead and its environs behind Hoover Dam.

The Park Service is studying the area and will prepare a recreation plan. Facilities will be provided at suitable points for boating, fishing, and other recreation.
Lake Powell, behind Glen Canyon Dam, will be flanked by varied and beautiful scenery.

HIGHWAYS TO GLEN CANYON DAM.—Excellent, new, paved highways have been built to the Glen Canyon damsite. A 76-mile highway through the highly scenic area has been built from Kanab, Utah, to the damsite. A new 25-mile highway extends northward from Bitter Springs to the damsite. Both of these highway links connect with the Glen Canyon Bridge to form a new link in U.S. Highway 89. The Glen Canyon Bridge was completed in February 1959.

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- Phoenix, Arizona: 300 miles
- Salt Lake City, Utah: 394 miles
- National Parks:
  - Zion: 100 miles
  - Bryce: 138 miles
  - Grand Canyon, North Rim: 124 miles
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* If you want to show your club a color film of men and machines beginning work on this dam, ask the Bureau to lend you CANYON CONQUEST, specifying the date on which you wish to show it. Address your request as follows: United States Department of the Interior, Bureau of Reclamation, P.O. Box 360, Salt Lake City 10, Utah.

[1960]

U.S. GOVERNMENT PRINTING OFFICE: 1960 O-522772
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Construction at the damsite moved ahead rapidly, and in February 1959 the Colorado River was diverted through the two large tunnels which carry the river around the damsite during construction. Excavation then proceeded to the lowest foundation rock, 137 feet below the former river level.

On June 17, 1960, the first bucket of concrete was placed in the dam. In May 1961 the 1,000,000th yard was placed, and by November 1961 the 2,000,000th yard was in place. By the end of 1961, the dam stood nearly 350 feet above lowest bedrock where the first bucket of concrete was placed.

The initial storage of water behind the dam is scheduled for early in 1963. Glen Canyon's first generating unit is scheduled to go on the line in 1964.

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The first placement of concrete—June 17, 1960.

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[1962]
Memorandum

To: Chief, Dams Branch
   Attention: E. R. Schultz

Through: Head, Technical Engineering Analysis Section
         Chief, General Engineering Branch

From: H. Boyd Phillips and Ira E. Allen

Subject: Photoelastic stress analysis of the right abutment at elevation 3130 and at Station 14+10--Glen Canyon Dam--Colorado River Storage Project

Introduction

Additional excavation of the right abutment of Glen Canyon Dam in the vicinity of elevation 3130 will be necessary because of weak rock seams encountered at the original line of excavation.

As a result, the shape of the contact surface between the dam and the abutment must be altered from that originally proposed.

This study has been made to investigate the stress distribution in the abutment near the contact surface to determine if any critical stress concentrations develop in this region due to the change in shape caused by the additional excavation.

The arch element at elevation 3130 and the cantilever element at Station 14+10 have been studied in this analysis. These two elements intersect the abutment in the area of the weak rock seams.

Results

The six accompanying figures present the results of this analysis in the form of stress curves. These are given along a line 4 feet into the abutment from the contact surface.

For comparison the stress distributions for full radial abutments have also been studied, and the results are included on the figures.

From the stress curves, it can be seen that the maximum stresses are lower in all cases for the proposed excavation than for the full radial abutment.

Conclusions

No critical stress concentrations develop along the line studied due to the proposed change in excavation. In fact, the stresses are actually lowered by the change in excavation since the length of the contact surface is increased over that assumed for the full radial abutment.
Basic Data

The dimensions and loadings used in this study were furnished by personnel of the Concrete Dams Section and were designated as Study A-20a.

Elevation of arch 3130
Width of arch, full radial abutment 322 feet
Assumed stress resultants at center of arch at abutment:
M = 149,373,000 ft-lb/ft
T = 8,447,700 lb/ft
S = 2,849,500 lb/ft

Station of cantilever 14+10
Width of cantilever, full radial abutment 322 feet
Assumed stress resultants at center of cantilever at abutment:
M = 176,320,000 ft-lb/ft
T = 13,678,600 lb/ft
S = 3,598,700 lb/ft

Technical Details

This study was made experimentally, utilizing the photoelastic interferometer.

Model material for both the abutment and the dam was Columbia Resin, CR-39, 3/8 inch thick. A similar previous study\(^1\) indicated that variation of modulus of elasticity ratios between abutment and dam of the order of 1 to 6 had no apparent effect on the stress distribution.

Scale of model to prototype was 1 to 1200.

In each case, the dam element of the model was grouted to the abutment element by a thin layer of plaster of paris.

Reference


Personnel

This study was made under the general supervision of W. T. Moody. The drawings were prepared by H. E. Willmann.

H. Boyd Phillips

Ira E. Allen
NOTES

+ is compression.
- is tension.

Assumed stress resultants at center of arch at abutment:

\[ M = 149,373,000 \text{ Ft-lb/ft.} \]
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+ is compression for normal stress.
- is tension for normal stress.
Assumed stress resultants at center of arch at abutment:
\[ M = 149,373,000 \text{ Ft}-\text{lb/ft} \]
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\[ S = 2,845,500 \text{ Lb/ft} \]

Assumed average shear load at full radial abutment
(Trial load analysis)

\[ \sigma_x \]

\[ \tau_{xy} \]

COLORADO RIVER STORAGE PROJECT
GLEN CANYON DAM
PHOTOELASTIC STRESS ANALYSIS
RIGHT ABUTMENT - EL. 3130
ARCH ELEMENT
\[ \sigma_x \] AND \[ \tau_{xy} \]

AUG. 31, 1959

557-PEL-92
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COLORADO RIVER STORAGE PROJECT
GLEN CANYON DAM
PHOTOELASTIC STRESS ANALYSIS
RIGHT ABUTMENT - EL. 3130
ARCH ELEMENT
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NOTES

+ is compression;
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COLORADO RIVER STORAGE PROJECT
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PHOTOELASTIC STRESS ANALYSIS
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PHOTOELASTIC STRESS ANALYSIS
RIGHT ABUTMENT - STA. 14 + 10
CANTILEVER ELEMENT
PRINCIPAL STRESSES

NOTES
Assumed stress resultants at center of cantilever at abutment:
\[ V = 176,320,000 \text{ ft-lb/ft} \]
\[ T = 3,078,600 \text{ Lb/ft} \]
\[ S = 3,939,700 \text{ Lb/ft} \]
Memorandum

To: Mr. L. G. Puls

From: Mr. R. E. Glover

Subject: Relief of stress concentrations at the abutment at the top of the Glen Canyon Dam

Stress Concentration

The trial load studies for the Glen Canyon Dam indicate a concentration of stresses at the top of the dam at the abutment. This concentration of stresses raises the pressure applied to the abutment at these points to about twice the highest value that occurs elsewhere. The computed figure for the A-19 section is 969. lb/in².

This finding is in accord with results obtained by the methods of the Theory of elasticity and by photoelastic experiments. There is no question that a stress concentration should be found here. It could be expected that a higher computed value would be found if a more closely spaced arch and cantilever grid were used.

Effect of the Concentration on the Safety of the Dam

A stress concentration should be expected to exist at the top of the dam at the abutment of any arch dam whose top arch ring is loaded. Trial load studies show such loading to be present in most cases. The situation is considered to be a normal one. For the present case the condition is exaggerated, however, by the softness of the abutment rock and the steepness of the abutment.

The threat which this concentration makes to the safety of the dam or to its proper functioning is not considered to be great. It is believed that the dam would behave well even though nothing were done about this concentration. It is likely that a certain amount of crushing would occur locally the first time the reservoir was filled and that this local crushing would provide relief from the high stresses which could otherwise occur if the abutment continued to behave elastically. A permanent set could result which might cause a crack to appear between the dam and the abutment near the top of the dam if the reservoir level was again drawn down. Such a crack would not cause leakage because it would close again if the reservoir level rose. If some diligent caretaker grouted it up, however, high stresses would appear when the reservoir was filled again.
The writer has seen two arch dams where there was some evidence of distress at the abutment near the top of the dam. It is my recollection that the abutment rock was a red granite in both cases. The appearance in one case seemed to be explained on the basis that the reaction had produced a tension crack as shown in Figure 1 and that a thin shell of material had spalled off. A similar crack can be seen in a glass marble which has been hit hard enough by another marble. Neither the strength of the dam nor the abutment should be impaired by the development of such tension cracks since the rock below the cracks would be sound. At both of the dams where this behavior of the rock was observed there was evidence of alkali-aggregate expansion and it could be expected that the thrusts applied to their abutments at the top of the dam were unusually high.

Laboratory tests have been made on Navajo sandstone loaded locally, as is the case with a dam abutment.

In the first test, the loads were applied to the flat surface of the sandstone through a high strength concrete cylinder about 2 inches in diameter. The 6-inch long by 6-inch-diameter sandstone specimen was embedded in plaster of paris in an 18-inch-diameter by 12-inch long galvanized iron mold for this test. The plaster of paris was reinforced by circular bars near the outside. In the second test, the loading was applied through a 2-inch-diameter by 1/2-inch thick steel disc. In the first test, the concrete cylinder failed at a stress of 8,450 pounds per square inch. The sandstone was indented about 1/32 of an inch. This is 2.27 times the strength of the sandstone as tested in the usual cylindrical form. With the steel block used in the second test, the sandstone was failed at 14,000 pounds per square inch. This is 3.77 times the unconfined compressive strength of the sandstone. The sandstone was deeply indented in this case. These tests indicate that the abutment will stand up without failure under loads much higher than the computed 969 lb/in² concentration.

Reduction of the Stress Concentration

If some way can be found to equalize the abutment stresses, an improved structural behavior should result. It should be pointed
out, however, that these provisions should be made with the utmost care lest the situation be made worse instead of better. Confirmation of this is afforded by a study recently completed in the Bureau Photoelastic Laboratory. In this case, the effect of a narrow vertical slot adjacent to the abutment was studied. A slot of this sort could be expected to relieve the stress concentration at the top of the dam, and it does, but a stress concentration of 4,600 pounds per square inch appears at the bottom of the crack.

To relieve 969. 1b/in$^2$ at a cost of 4,600. 1b/in$^2$ would be no bargain.

If the computed stress concentration is to be relieved without creating a stress concentration somewhere else, a certain principle should be carefully followed. This principle may be stated as follows: No stress concentration will be caused by a slot if (1) the base of the slot has zero width when the structure is under no stress; (2) if its width is continuous and increases continuously with distance from the base; and (3) if it closes progressively as the load is applied. The trouble with the slot of the photoelastic study was that it had a finite width at the base. An example of the application of this principle is afforded by the stresses at the contact of a car wheel and a rail. As load is applied to the wheel, the area of contact between it and the rail widens but the stress at the boundary of the area of contact is always zero. A slightly different case has a drastically different outcome. If a rigid flat die is pressed against the flat face of an elastic solid the stresses around the edge of the area of contact tend toward infinite values. The idea of a slot to relieve the stress concentration is basically sound and may be made effective if it is carried out in a way which does not provide an opportunity for stress concentrations to develop at the crack. A possible way of doing this is suggested below.

**Slot Arrangement**

The forming of a relief slot may possibly be accomplished in the following way:

1. By the method of Vogt (1) compute the abutment deformation near the top of the dam for the design conditions using the thrusts obtained from the trial load study.

2. Choose a suitable load distribution near the top of the dam and recompute the abutment deformations due to the
assumed load condition. Since the last grout lift will extend from elevation 3660 to the top of the dam at 3715, it is suggested that the new load condition be chosen in this interval. At elevation 3660, the new and old distributions should merge.

3. The difference in tangential displacement as computed in (1) and (2) represents the shape the relief slot should have to realize the assumed load condition. Some computations made by this writer indicate that this slot should taper from zero width at elevation 3660 to about 0.5-inch width at elevation 3715.

4. During the construction of the dam, leave two slots about 4 feet wide with an approximately 4-foot wide cantilever element between, as shown in Figure 2. The radial section should be near the abutment but at a sufficient distance therefrom to occupy the full section of the dam and to leave a sufficient area between the section and the abutment to support the concrete between the section and the abutment with acceptable stress intensities. At the upstream face the section should be sufficiently away from the abutment to permit the installation of water stops and to avoid knife-edge configurations. The cantilever should be reinforced and prestressed for reasons to be explained later.

5. With jacks arranged as shown, bend the cantilever to the right to the shape of one-half the slot width as computed in (3) and fill the right hand slot with concrete. This concrete should be attached to the concrete to the right by dowels but the face which abuts the cantilever should be greased so that it will separate from the cantilever.

6. Install jacks to bend the cantilever to the left and place concrete in the left hand slot as in 5, and remove the top jacks. The lower jacks may be concreted in.

7. The faces at a, b, c, and d should be keyed. Water stops should be installed along the upstream face, the top and the downstream face of Joints b and c to completely enclose them and exclude dirt. Joints b and c are to remain permanently open unless closed by arch thrust. They are not to be grouted.
When the cantilever returns to its unstressed position after removal of the jacks, two tapered joints should remain. The sum of the widths of these joints should be approximately that of the widths computed in (3). The work described in Items (4), (5), and (6) should be done after the dam has been grouted to elevation 3715 and while the water level in the reservoir is low.

Operation of the Joint

As the reservoir level rises, the top arch becomes loaded. As the thrust increases between elevation 3660 and 3715, the joints at b and c of Figure 2 close progressively. They become completely closed before the reservoir level reaches the design level. At the design level, the abutment pressures between elevation 3660 and 3715 should be somewhat below those chosen in Item (2), the difference being due to absence of earthquake forces. With earthquake forces included the abutment loads should reach the values chosen in Item (2).

Photoelastic Check

Because of the technical difficulties inherent in this procedure, the arrangement should be checked carefully by photoelastic means to determine the stress changes which will occur as the reservoir level rises and falls. It should be borne in mind that this is a new and untried device whose ability to accomplish the desired results should be completely confirmed before it is put to use.

Reasons for Prestressing the Cantilever

If the cantilever is constructed as an ordinary concrete beam, the application of the jack loads may be expected to crack the concrete on the tension side. This is customary behavior in concrete and no structural impairment would result. If these cracks are present, however, and the reservoir level rises above them, seepage through them may be expected to produce an unsightly discoloration on the downstream face. If the concrete is prestressed, no crack will be produced and the discoloration will be avoided. The prestressing therefore has no structural significance, its purpose is to avoid unsightly laitance marks on the face of the dam.

[Signature]
References

1. About the calculation of foundation deformations, by Dr. Frederick Vogt, 1925, Bureau of Reclamation, Technical Memorandum No. 77

2. A Study of Patterns of Abutment Movements Corresponding to Some Simple Patterns of Abutment Thrust, by M. M. Newmark, 1931, Bureau of Reclamation, Technical Memorandum No. 224

3. Theory of Elasticity, by S. Timoshenko, Bureau of Reclamation, Library No. 65/34

4. Beams on an Elastic Foundation, by Hetenyi, Bureau of Reclamation, Library No. 63/46.2 C-1


There are also some foundation deformation data in the Boulder Project Final Reports.
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

COLORADO RIVER STORAGE PROJECT

REPORT ON FOUNDATION ADEQUACY AND DESIGN CONSIDERATIONS

of

GLEN CANYON DAM

by

BOARD OF CONSULTANTS

Julian Hinds, Chairman
John J. Hammond
Raymond E. Davis
Edward B. Burwell, Jr.
John W. Vanderwilt

Denver, Colorado
May 7, 1957
Denver, Colorado

May 7, 1957

Mr. L. N. McClellan
Assistant Commissioner and Chief Engineer
Bureau of Reclamation
Denver Federal Center
Denver, Colorado

Dear Mr. McClellan:

In accordance with the instructions contained in your letter of April 26, 1957, copy attached for reference, the undersigned members of the Consulting Board for Glen Canyon Dam assembled at Kanab, Utah, at 8 a.m., May 1, 1957, in the office of the Project Construction Engineer. A book of technical data was furnished to each member of the Board, and descriptions of the design of the dam and the geology of the dam site were given by Acting Chief Designing Engineer L. G. Puls and Chief Geologist W. H. Irwin.

The Board was then flown to the dam site and viewed the left abutment from the observation platform on the right abutment. In the company of Geologists Irwin, Murdock, and Lassen, and Engineer Puls, the Board members were lowered to the bottom of the canyon, from which both abutments were observed. Walking down the canyon to the upstream portal of the right diversion tunnel, examination of the portal excavation and tunnel was made. About 800 feet of the tunnel had been excavated.

Later the Board was taken across the river by boat and taken by the high-line skip to the exploratory adit in the left abutment at about elevation 3400. The rock in both this adit and the one near the base of the left abutment was examined. This concluded the first day's inspection tour, after which the Board returned to Kanab.

The Board returned to the dam site Thursday morning and was flown directly to the left abutment. From a vantage point near the axis of the dam the right abutment was viewed. After returning to the right abutment the Board inspected the drill cores of recent explorations stored at the field office at the dam site. These cores indicated the uniformity of the rock and the absence of deleterious foreign material.
The Board was then taken to the Wahweap aggregate deposits where the bridge contractor's processing plant and the processed aggregate were inspected. A tour of the A, B, and C aggregate deposit areas gave a good general picture of the approximate quantities and character of material available.

After returning to Kanab, an examination was made of the drill cores taken from the dam site during the 1947 and 1948 explorations. These cores, which were taken from the rock of the canyon floor, were examined in their entirety. During the evening a conference was held to discuss the results of the inspection of the dam site and to review the information contained in the volume of technical data. Friday was spent in the conference room at Kanab with Messrs. Wiley, Puls, Irvin, Murdock, and Lassen discussing the various data on designs of the dam and the geology of the dam site. Particular attention was given to the matters referred to in your letter of April 26.

The conference adjourned at 4:30 p.m.

On May 4, the Board traveled to Denver, where it reconvened in your office in Denver at 9 a.m. May 6.

The morning of May 6 was spent in the Bureau laboratories. There, foundation cores A to L which had been previously shipped to Denver for testing purposes were examined. The operations of mixing and placing of a trial batch of mass concrete such as it is expected will be employed in the dam were observed. The Petrographic Laboratory was visited and specimens of the Navajo sandstone from the dam site were examined by microscope. The triaxial laboratory was visited where methods of testing were explained and the results obtained to date were discussed. The hydraulic model of the Glen Canyon spillways was observed in operation.

The balance of May 6 and all of May 7 were devoted to further review of the data and the preparation of the report which follows, discussing in order the six items enumerated in your letter of April 26.

1. ADEQUACY OF FOUNDATION

The Navajo sandstone, which will form the foundation and both abutments of the dam, is a remarkably uniform, massive, fine- to medium-grained silica sandstone, containing no known structural weaknesses that would seriously impair its mass suitability as a foundation material for the high arch dam. The formation is made up of thick,
obscurely defined horizontal strata with marked cross-bedding. Jointing is inconspicuous, widely spaced and generally tight. Although this sandstone is relatively soft and locally friable and has a high porosity, its compressive strength, as disclosed by the field and laboratory tests and by its behavior under the existing loads that are superimposed on it in the lower part of the canyon walls, is regarded as more than adequate to meet the load conditions which will be imposed on it under Design A-18 as set forth in the volume of technical data furnished the Board.

The Board is of the opinion that both the mass modulus and the mass compressive strength of the Navajo sandstone are considerably greater than the values obtained in the laboratory tests on unconfined samples. This opinion is supported by the geophysical survey, by the in situ tests conducted in the left abutment adit, and by the triaxial compression tests. Further testing and study of the rock, as planned by the Bureau, are desirable. This sandstone has a tendency to exfoliate or develop sheeted joints parallel and close to the surfaces when subjected to blasting operations. This, an effect of stress relief, will need to be considered in the preparation of foundation surfaces and in the low-pressure or area grouting of the abutments.

2. DESIGN CRITERIA FOR THE DAM

Careful attention was given to the design criteria as set forth in the last section of the volume of technical data supplied at Kanab, with particular attention to the criteria for "Design A-18" as set forth on pages 2 and 3. Special attention was also given to the general layout shown on Drawing No. 557-D-330, which has been developed as a means of meeting the criteria. The plan shown on this drawing differs from that of the specifications in some details but not to an extent which will interfere with the execution of the contract.

As stated elsewhere in this report, the rock at the Glen Canyon site is particularly competent as to uniformity, consistency, and freedom from important defects. Compared to many rock foundations, its compressive strength is low. Nevertheless, its unstrained compressive strength is at least twice the foundation pressures commonly permitted for concrete dams. If constructed with "customary" stress values, in the order of 800 to 1,000 psi, this dam would have a factor of safety at the abutments of certainly more than 2. However, because of the great importance of this monumental structure, its cost, and the destruction that would be wrought by its
failure, the Bureau's designers have considered it advisable to go to much higher factors of safety. In Design A-13 a safety factor of 5 has been assumed when the structure is subjected to full waterload plus earthquake forces. In the field of dam design, such a factor of safety is unusually high, and far exceeds any factor of safety that can be attained with a reasonably designed concrete gravity, earth, or rockfill structure. However, by adopting unique methods of design, this outstandingly conservative degree of safety has been achieved without extravagant use of construction materials.

It is not considered necessary to list or discuss individually all of the criteria enumerated on pages 1 and 2 of the last chapter of the volume of technical data, as most of them obviously follow from the dimensions of the dam. However, the more important ones are discussed as follows:

**Earthquake loading.** Allowance for an earthquake of an intensity of 0.1 g applied both to the mass of the dam and the water in the reservoir is liberal for this particular location, perhaps a little higher than absolutely necessary but not extravagantly so. The Board concurs in the use of this value.

**Arch stresses at the abutments.** Foundation strengths assumed in designs are based entirely on the results of compression tests of unrestrained, saturated rock cores. In only a few cases have the observed compressive strengths been less than 2,000 psi. There is every reason to believe that the true strength of the rock in situ, where lateral expansion is restrained, is much higher than indicated by the tests. Triaxial tests now in progress support this expectation. Nevertheless, in the interest of absolute safety, the criteria provide that the arch abutment pressures shall not exceed 400 psi. This is believed to be a conservative value.

The Board is of the opinion that an eminently safe structure would result if maximum stress in the arch be limited to 400 psi at the abutments for full waterload and temperature, without earthquake, and to 500 psi for the brief, infrequent, and unlikely combination of both a full reservoir and a full earthquake stress.
Stresses at base of maximum section. Compression tests of the rock across the bottom of the river channel indicate that it is appreciably stronger than the rock at higher elevations. For this reason, the criteria allow a higher stress at the base of the cantilever in the deepest portion of the foundation. The Board concurs in this proposal and recommends that the design proceed on the basis of 600 psi in this region for waterload and temperature and 750 psi for the combination of maximum waterload and maximum earthquake effects.

Principal stresses in concrete. In accordance with modern practice, the criterion that the principal stresses in concrete at points removed from contact with the foundation and abutment rock be permitted to go as high as 1,000 psi, assuming a concrete for which the ultimate compressive strength is 4,000 psi, is proposed. The Board concurs in the use of such a stress for concrete not in contact with foundation rock.

Modulus of elasticity of concrete. The criteria provide for the use of 2,000,000 psi as a modulus of elasticity of concrete. For the class of concrete proposed, this value is thought to be low. It is probable that the actual value will be nearer 3,000,000 psi. The conservatively low value for the 2,000,000 psi is on the side of safety.

Modulus of elasticity of rock. Results of tests on unconfined, saturated cores indicate a relatively low modulus of elasticity for the foundation rock, the average being greater than 500,000 psi, which is assumed in design. As previously stated, the Board is of the opinion that modulus of the rock in situ is considerably greater than that obtained from tests on cores. The effect of modulus of elasticity of the rock on the stresses in the dam is extremely complex, but careful investigation shows that the use of a relatively low modulus is on the side of safety.

Poisson's ratio—Rock. A value of 0.06 of Poisson's ratio of rock, as determined from preliminary tests, seems to be low. However, it conforms to the data now available. Because the assumption of a low ratio is on the side of safety, the Board concurs in the assumed value of 0.06.

Poisson's ratio—Concrete. The value of 0.20 for Poisson's ratio in concrete as provided in the criteria is customary and its use is concurred in by the Board.
3. THERMAL CONTROL OF CONCRETE

From a review of the results of analyses made by Bureau engineers, if serious cracking that might adversely affect the structural integrity of Glen Canyon Dam is to be avoided, the maximum temperature of the concrete should at no time exceed about 75° F. To achieve this objective it will be necessary to pre-cool the concrete materials as well as to post-cool the concrete after placement. For the concrete mix which it is contemplated will be employed in the mass of the dam, if post-cooling is started immediately after placement, it has been calculated that the temperature rise need not exceed about 25° F.

The specifications require that the temperature of the concrete at the time of placement shall not exceed 50° F. This will necessitate refrigeration of the coarse aggregate and will require the use of slush ice in the mixing water. Precedents for pre-cooling are a number of dams completed by the Corps of Engineers in recent years where this method has been employed with success.

For post-cooling, the specifications require that water be circulated through cooling pipes in the bottom of each lift of concrete, and the height of lift may be 7-1/2 feet. An initial cooling period of not less than 12 days is required, and the temperature of the water for initial cooling can be no warmer than the concrete being placed about the cooling coils. The flow of cooling water is to be started through the cooling pipes immediately before the concrete is placed about the pipes, and it is required that the rate of flow be not less than 2 feet per second, or approximately 4 gallons per minute. It is expected that the Colorado River water will be at a sufficiently low temperature for primary cooling except during late summer, when some refrigeration will be required.

At some later time, prior to grouting of contraction joints, secondary cooling by pumping refrigerated water through the cooling coils will be employed to reduce the temperature of the concrete below elevation 3450 to 40° F, and to reduce the temperature of the concrete above elevation 3450, in varying amount, from 40 to 50° F, depending upon the elevation.

It is expected that this secondary cooling will produce joint openings which will be on the order of 0.1 inch. As indicated by the specifications, subsequent to grouting, the temperature of the mass will be permitted to reach its natural equilibrium, which for the mass as a whole may be in the order of 60° F.
Your Board believes that the provisions which have been made for concrete temperature control are most excellent and approves the requirements of the specifications, except in two matters. First, because of the importance of keeping the maximum concrete temperature below 75° F, it would seem desirable that Paragraph 140 should also require that at no time during the period of primary cooling should the temperature of the concrete exceed about 75° F; also until a lift has been covered, it is believed that during periods of warm weather, the top of the lifts should be continuously sprinkled with cold water to the end that the concrete exposed to sunshine during hot days should be no warmer than about 75° F.

Second, because the dam would be required to carry its workload long before equilibrium temperatures would be reached in the natural way, after post-cooling to 40° F and grouting the contraction joints, it would seem essential artificially to bring the mass to near equilibrium temperatures by circulating warm water through the coils.

4. REVIEW OF CONTRACTION JOINT LAYOUT

The Board has reviewed the joint arrangement proposed by the Bureau, and believes that it is both necessary and adequate without being extravagant. The details shown on the specifications are based on the specification drawings which are now to be changed to conform to Design A-10. This will undoubtedly require some alteration in the details of the arrangements of the contraction joints but should not alter the general plan.
5. GROUTING OF CONTRACTION JOINTS FOR CONTROL OF STRESSES

The successful operation of this structure in accordance with the trial load method of analysis being applied depends largely upon the proper manipulation of concrete temperatures at the time of final closing of the contraction joints by grouting. This factor is being given the most careful consideration by the designers. The program proposed appears to the Board members to be reasonable and adequate.

6. FOUNDATION GROUTING AND DRAINAGE

The proposed program of foundation grouting and drainage is considered satisfactory with the following exceptions:

The area grouting proposed to be done through a series of shallow B holes prior to placement of concrete can, in the Board’s opinion, be accomplished more satisfactorily and with less danger of damaging the foundations in the steep abutment walls by connecting the area grout holes to a contact grouting system and grouting them after concrete has been placed and after the contraction joint grouting has been accomplished. Mr. Burwell will supply a drawing showing a method of connecting a contact grouting system with the area grout holes which has proven satisfactory.

Because of the nearly vertical attitude of the rock joints and their wide spacing, it is recommended that all grout holes be given an inclination that will effectively intersect the joint systems.
The conclusion and finding have been stated throughout the report and the Board feels that it is unnecessary to summarize them here.

The Board has enjoyed the assignment and appreciates the opportunity in serving your organization.

Respectfully submitted

Julian Hinds, Chairman

John J. Hammond

Raymond E. Davis

Edward B. Burwell, Jr.

John W. Vanderwilt

Enclosure 1

APPROVED: May 9, 1957

L. N. McClellan
Assistant Commissioner
and Chief Engineer
April 26, 1957

Board of Consultants
Bureau of Reclamation
Kanab, Utah

Gentlemen:

The following subjects are suggested for consideration during the meetings of the consulting board May 1 to 3, 1957, at Kanab, Utah, and May 6 and 7, 1957, at Denver, Colorado, relating to the design and construction of Glen Canyon Dam:

1. Inspect dam site, review results of the geological investigations and appraise the adequacy of the foundation.
2. Review design criteria for the dam.
4. Review contraction joint layout.
5. Review program for construction procedure with respect to grouting of contraction joints for control of stresses.
6. Review program for foundation grouting and drainage.

Very truly yours

(Signed)
L. N. McClellan
Assistant Commissioner
and Chief Engineer
Report to  
U. S. Department of the Interior  
Bureau of Reclamation

Colorado River Storage Project  
GLEN CANYON DAM  
DESIGN AND CONSTRUCTION PROBLEMS

by  
Board of Consultants

Raymond E. Davis  
John W. Vanderwilt  
John J. Hammond  
Edward B. Burwell, Jr.

Julian Hinds, Chairman

Page, Arizona  
October 12, 1961
October 12, 1961

Mr. Grant Bloodgood
Assistant Commissioner & Chief Engineer
Bureau of Reclamation
Denver Federal Center
Denver 25, Colorado

Dear Mr. Bloodgood:

In accordance with your letter dated August 25, 1961, the members of the Board of Consultants for Glen Canyon Dam, with the exception of Dr. John W. Vanderwilt, assembled in Page, Arizona, Sunday evening, October 7, 1961, and spent the following three days inspecting the job and reviewing data furnished by your office, with particular attention to the points enumerated in your letter. Unfortunately Dr. Vanderwilt was grounded in Phoenix, Arizona, by weather, and was unable to reach Page until late afternoon on Monday, October 8.

The Board was assisted as required by the following members of the Bureau staff:

O. L. Rice Acting Chief Designing Engineer, Denver, Colorado
C. S. Rippon Assistant Regional Director, Salt Lake City, Utah
E. R. Schultz Head, Concrete Dams Section, Denver, Colorado
L. F. Wylie Project Construction Engineer, Page, Arizona
B. B. David Project Field Engineer, Page, Arizona
G. B. Lasson Project Geologist, Page, Arizona
C. V. Gezelius Chief, Concrete Control Branch, Page, Arizona
On Monday morning, October 8, the Board (less Dr. Vanderwilt) and most of the Bureau personnel listed above, met in the offices of the Bureau of Reclamation at Page for briefing and for preliminary review of data furnished by your office to Board members in advance, and new computations and records subsequently accumulated, which were furnished by the Acting Chief Designing Engineer.

The afternoon of October 8 was spent in a general field inspection of present conditions, and a review of progress made since the Board's previous meeting on May 1 to 5, 1960.

On Tuesday morning, October 9, with all members present, a much more detailed field inspection of the site was made. The applicability of design and construction procedures, as given in Bureau data books, and as discussed on the previous day, were carefully considered on the site. Particular attention was given to the feature generally referred to as the "A Joint," at the lower downstream corner of right abutment, and to rock conditions immediately downstream of the lower arches on the left abutment.

The most valuable feature of this inspection was made from a cableway skip. The skip was lowered slowly down the left abutment, and taken slowly up the right abutment. Stops were made as required for special
observation and discussion. In this way the group effectively could be brought into close proximity to all points of special interest.

Later, the concrete placement operations in the dam were carefully observed, with special attention given to the adequacy of compaction procedures mentioned in your letter of instructions.

Also lining operations in the left spillway intake were observed.

On the afternoon of October 9, careful inspections were made of the Wahweap aggregate plant, and the concrete plant at the damsite, both of which were in operation. The purpose of these inspections was the gathering of data to assist in judging the quality of the concrete, as per your request.

Wednesday, October 10, was spent analysing the data accumulated during the previous two days, in further study of data furnished by Bureau personnel, and in preparation of this report.

The four items upon which the Board’s views were requested in your letter dated August 25, 1961, are as follows:

1. Review the foundation treatment program for the dam and appraise its adequacy.
2. Review the results of the latent trial-load analysis of the dam based on final excavations.

3. Review concrete test results of interior mass concrete to date.

4. Comment on compaction practices for mass concrete.
Item 1. The program of consolidation grouting for the right abutment of the dam outlined in the book of technical data, involving an initial phase of low-pressure grouting in 7-1/2-foot lifts from the top of each lift of the abutment block and a later phase of higher-pressure grouting from transverse galleries, should provide an adequate consolidation of the abutment rock. The modification of the "B" zone grouting program for the left abutment to include the entire keyway area and the provision of transverse galleries for a second phase of higher pressure consolidation are considered desirable improvements and meet with the approval of the Board.

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

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

In its report of May 5, 1960 the Board recommended that suitable instrumentation be provided to determine possible future movements along the "A" joint. Accordingly, Carlson strain meters and a joint meter were installed in February, 1961. The small vertical and horizontal movements which have been subsequently recorded appear to be the result of temperature changes. The Board recommends that the measurements be continued.

The primary provisions which have been made for draining the foundations and abutment rocks consist of drain holes which are to be drilled from
the grouting and drainage gallery in the dam and from grouting and drainage galleries at three elevations in both abutments, the latter extending a maximum distance of 450 feet inside the abutments.

As a safeguard against the possible development of hydrostatic pressure in the rock joints of the abutments downstream from the dam, the Board recommends that additional drainage be provided in the canyon walls downstream of the arch keyways by drilling a series of horizontal holes from the surface extending, say 100 feet inside the walls. It would be prudent to extend these drainage measures somewhat downstream from the anchorage treatment.
Item 2. The complete trial-load analysis, with earthquake forces included, furnished in the data book is complete and well presented.

The principal stresses at the junction of the concrete and the abutment rock are shown on Drawings Nos. 557-D-2417 and 557-D-2418. Values of minor and major principal stresses are shown, respectively in columns headed $\sigma P_1$ and $\sigma P_2$ of these drawings.

It will be noted, that except for a calculated tensile stress of 2. psi at the bottom of the dam at the left abutment, all of the upstream principal stress values are compressive and well within the target limit of 600 psi.

On the downstream face (Drawing No. 557-D-2418) the unit stress of 600 psi, previously approved by the Board, is exceeded at only a few points as follows:

<table>
<thead>
<tr>
<th>Location</th>
<th>Elevation</th>
<th>Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Left Abutment</td>
<td>3180</td>
<td>620 psi</td>
</tr>
<tr>
<td>Left Abutment</td>
<td>3120</td>
<td>691 psi</td>
</tr>
<tr>
<td>Right Abutment</td>
<td>3300</td>
<td>650 psi</td>
</tr>
<tr>
<td>Right Abutment</td>
<td>3360</td>
<td>617 psi</td>
</tr>
<tr>
<td>Right Abutment</td>
<td>3120</td>
<td>716 psi</td>
</tr>
</tbody>
</table>

Three of these stresses exceed the limit of 600 psi by as much as 20%, which is undesirably high. However these excesses are at relatively low elevations and the construction has progressed to a point that renders their reduction impossible. Modified copies of drawings 557-D-2419 and 557-D-2420 are attached to this report showing the
relative importance of the maximum principal stresses (disregarding
direction) compared to arch, cantilever and target stresses. The
minor principal stresses 6 P2, Drawing No. 557-D-2418 are negative
at the downstream face for all elevations below 3300 on the left abut-
ment and below 3180 on the right abutment. Except for values of
205 psi at elevation 3180 and 224 psi at elevation 3060, both on the
left abutment, these tensile values are relatively moderate.

The high compressive stresses at the extreme top of the dam, shown in
previous analyses were eliminated by the omission of grouting of the
end joints above elevation 3660. The Board approves of this procedure.

The arch and cantilever stresses of the concrete as shown on Drawings
Nos. 557-D-2415 and 557-D-2416 are generally moderate. One relatively
high value of 975 psi, shown for the arch intrados at elevation 3480,
at the one-half point on the right side of the dam is not beyond a
safe value for the concrete being produced at Glen Canyon. It should
be noted that the stresses shown on Drawings No. 557-D-2415 and No.
557-D-2416 are arch and cantilever stresses and should not be confused
with principal stresses.

It is the opinion of the Board that the design, as indicated by stresses
determined by the complete trial-load analysis, is satisfactory.

There was considerable discussion of the possibility of attempting to
eliminate observed minor cracks of some of the blocks already placed,
by increasing the minimum temperature to which the concrete is cooled, prior to joint grouting. The original purpose of specifying concrete cooling to 40°F. was to keep the stresses within the target value. It has not been demonstrated that the concrete cracking is solely caused by the degree of concrete cooling. The rate of cooling may be of greater importance. Before adopting any change in the specified cooling requirements new stress analyses should be made based on any proposed higher terminal cooling limit to determine whether the affects would be negligible. The cracks described, which were not available for inspection by the Board, are undesirable but are not believed to endanger the safety of the structure. Their elimination at the expense of appreciably increasing principal abutment stresses above the present specified limit is not justified.

The results of the final trial computations without earthquake were not available for the meetings at Page, but it is understood they will be ready soon. There is no reason to expect that the results of such analysis will be adverse. However this analysis will be reviewed when received and if necessary will be covered by an addendum letter.
Item 3. The statements which follow are based upon the consideration of
(1) The Section entitled "Field Trials on Use of Pozzolith 8 in Mass
Concrete" of The Book of Technical Data, for use of the Board of Consultants,
dated September 15, 1961; (2) a review of the test records obtained from
the U.S.B.R. Glen Canyon laboratory; (3) statements of U.S.B.R. staff
members at Glen Canyon; (4) inspection of aggregate processing plant, and
batching and mixing plant, and (5) observation of placing, mixing, and
compacting interior mass concrete for the dam.

There is presently being employed a mass concrete mix in which Pozzolith
8 (same as MBL) in the amount of 0.35% of the weight of cement, plus
pozzolan, is used. The mass mix contains 188 pounds of Portland cement
and 80 pounds of pozzolan per cubic yard. In a memorandum dated August 24,
1956, prepared by a member of the present Board, the opinion was expressed
that the interior mass concrete should contain not less than two (2) sacks
of cement and one (1) of pozzolan per cubic yard. In the May 5, 1960 report
of the Board appears the following statement:

"From the data which are presently available, it appears that
adequately high strengths may be obtained with a somewhat leaner
mix. For this reason and consideration of the advantages that
would be obtained by reduced heat of hydration, the Board would
favor a modest lowering of these minimum cement and pozzolan
requirements, if the results of the tests now in progress and
later field experiments make such reduction seem desirable."

The results of tests on mass mixes made at Glen Canyon by representatives of the Denver laboratory during the period May 23 through June, 1961, indicate that when Pozzolith 8 was employed in the designated amount, the design compressive strength of 3,000 psi at the age of 6 months for the interior mass mix of the dam could be obtained with reduction in cement content of at least 10% below that of the control mix without pozzolith. Such a reduction resulted in a portland cement content of 186 pounds per cubic yard and a pozzolan content of 72 pounds per cubic yard.

Because of what appears to have been some misunderstanding of the recommendations of the Board there has actually been employed an interior mass mix containing two (2) sacks of cement (188 pounds) and one (1) sack of pozzolan (80 pounds). Based upon the results of tests on 6 x 12-inch job cylinders taken from 142 batches of mass concrete, made during the period from June 9 to September 25, 1961, the average compressive strength at the age of 28 days is 3035 psi. This strength is substantially the same as the average obtained with the 10% reduction in cement and pozzolan with pozzolith 8, obtained in the trial test program conducted by members of the staff of the Denver laboratory.
The interior mass mix containing two (2) sacks of cement and one (1) sack of pozzolan per cubic yard, which is presently being employed using Pozzolith 8, was observed to be most excellent so far as placeability is concerned, and the mix retains its mobility for several hours. It appears certain that a mix containing a somewhat lower content of cementing materials (cement plus pozzolan) could be satisfactorily placed and compacted without any increase in labor costs.

In some of the interior mass concrete along galleries where secondary cooling to a temperature of not greater than 40°F. is required, some cracking has been observed to take place when the temperature, as determined by electrical resistance thermometers, had been lowered to about 45°F. No secondary cooling has been completed since the present mix containing pozzolith has been employed. It is therefore too early to say whether the reduction of 6.9% in cementing materials will lead to more favorable conditions as regards cracking tendency. However, any such reduction is a step in the right direction and, so far as cracking is concerned, the lower the content of cementing materials the better.

For this reason and also because further reduction would lead to a lowering of costs to the Bureau, the Board believes it would be desirable to employ a mix for which the reduction in cementing materials as compared with the control is 10.0%, rather than the 6.9% reduction as represented by the mix presently being employed. Based upon the relationship between
the 28-day compressive strength of 6" x 12" job cylinders and the compressive strength of 18" x 36" mass-cured cylinders, it would appear that the mix which the Board recommends would with assurance provide the 180-day compressive strength of 3,000 psi assumed in design of Glen Canyon Dam.

In this connection a study of the records indicates that the coefficient of variation in job cylinders at times has been on the order of 15%. Even for this high value of the coefficient of variation, calculations indicate that the mix just recommended would be adequate. However, where quality control has been properly exercised on similar work, the coefficient of variation has generally not exceeded about 12.0%, and there seems to be no good reason why a similarly low value should not be obtained at Glen Canyon. As a result of inspection of the aggregate processing, batching and mixing operations and a study of the records, it appears that the major reason for the large variability in quality of interior mass concrete is likely to be the rather large variations in moisture content of the sand. Specifications require that abrupt change in the moisture content of the sand shall not be more than 0.5 percentage points and that the gradual change in moisture content shall not exceed 1.0 percentage point in 4 hours. The concrete control report for the month of September, 1961, states that the conditions regarding uniformity of moisture content of sand have improved. Nevertheless, the records show that 14.5% of all samples of natural sand failed to meet the abrupt change requirement and 20.2% of the heavy media sand failed to meet the abrupt change requirement. The greatest change in
moisture content from one batch to the next was 2.7% for the natural sand, and 3.7% for the heavy media sand. Also, for the 18.1% of the tests the natural sand failed to meet the gradual change requirement and 25% of the heavy media sand failed to meet this requirement.

The Board is of the opinion that steps should be taken to eliminate such large variations to the end that the sand, as delivered to the batch plant, will substantially meet the moisture content requirements of the specification at all times.

Final screening of all coarse aggregate sizes is being employed. This is desirable because of the substantial breakage that occurs between the time of delivery of aggregate to the stockpiles at the processing plant and its delivery to the batching plant. As the final screening operation is presently being carried out, a blend of two (2) sizes (3/4" and 3", or 1 1/2" and 6") is final screened. Experience on other jobs has indicated that this practice results in layers of undersize, predominantly composed of flat and elongated particles, produced by breakage. These flat and elongated particle layers are subsequently discharged into the batcher as slugs, which produces concrete mixes of poor workability and higher requirement than those containing corresponding natural gravel particles of rounded shape. The better practice, leading to greater degree of uniformity, is to finish-screen in one operation a blend of all four coarse aggregate
sizes. The Board suggests that consideration be given to the possibility of such finish-screening for the coarse aggregates entering the construction of Glen Canyon Dam. It is believed that the greater degree of uniformity that would be achieved through such finished screening would result in a greater degree of uniformity in concrete strengths.

At the batching plant it was learned that variations in moisture content in the No. 4 to 3/4-inch gravel had been at times undesirably large. Such variations would also contribute to lack of uniformity in the quality of concrete. The Board would like to suggest that thought be given to aerating the No. 4 to 3/4-inch gravel in the batching bins, using, say, 35°F. air. The same sort of operation might also be expected to lead to a greater degree of uniformity in the moisture content of sand.

The coarse aggregate specifications also require that there be retained on the 3/8-inch sieve not less than 50% of the 3/4-inch size; on the 1/4-inch sieve, not less than 25% of the 1/2-inch size; and on the 2 1/2-inch sieve, not less than 20% of the 3-inch size. For the month of September the records show that while the aggregates furnished in these three sizes on the average met these specification requirements, they failed to meet these requirements in a considerable number of instances. For the 3/4-inch size the percentage retained on the 3/8-inch sieve ranged from 35 to 82; for the 1 1/2-inch size the percentage retained on the 1 1/2-inch sieve ranged
from 12 to 84, and for the 3-inch size the percentage retained on the
2\frac{1}{2}-inch sieve ranged from 3 to 54. This lack of uniformity may be ex-
pected likewise to contribute to significant variations in the concrete
strengths.
Item 4. During the time of the Board's visit to the work it was observed that mass concrete was being placed with a crew of only six men, three of whom were each using one-man Malin vibrators, with a very workable mix being employed for which the reported slump was not in excess of two inches. Each batch of 12 cubic yards, in the opinion of the Board, was fully compacted in a remarkably short period of time. Workmen and engineers alike were loud in their praise of the use of pozzolith in contributing so greatly to this most excellent workability. It was reported that one vibrator man-hour was required to compact 80 to 100 cubic yards of concrete. While this rate of placement per vibrator man-hour is much higher than is customarily found necessary on most similar work, in the opinion of the Board, the compaction was complete and thorough and the one-man vibrators were entirely adequate. The specifications require that two-man vibrators be used. There seems to be no good reason why this requirement should be enforced.

Representatives of the Bureau have expressed interest in the economic possibility of using dry mass mixes to be spread with bulldozer and compacted by gang vibrator mounted on a small Caterpillar tractor. This practice of employing "minus zero slump" concrete, spread and compacted by the equipment just mentioned, has been and is being employed in Europe and more lately in Japan. In the opinion of the
members of the Board who have witnessed this operation, the degree of compaction obtained from such dry mixes with such equipment, as judged by the appearance of formed surfaces, is less than that obtained by the practices followed in Bureau work using manually-operated vibrators to compact air-entrained mass concrete. Also, in foreign work the cement content of mass concrete has been substantially greater than for mass concrete of recent Bureau works. The Board is of the opinion that the present practices of the Bureau result in more complete compaction and lower overall cost than would be the case if super dry concretes, bulldozer spreading and gang-vibrated compacting had been employed.

Respectfully submitted,

Raymond E. Davis
John J. Hammond
John W. Vanderwilt
Edward B. Burwell, Jr.

Julian Hinds, Chairman

APPROVED: OCT 20, 1961
Grant Bloodgood
Assistant Commissioner
and Chief Engineer
Addendum

to Consulting Board Report,

Glen Canyon Dam

October 12, 1961

Drawings comparing principal stresses at abutment faces with the allowable stress of 600 psi, prepared by Consulting Board at Page, Arizona, October 11, 1961.

Note: Only one copy available. Denver office will furnish additional copies.
GLEN CANYON DAM

An Artist's Conception

Colorado River Storage Project

BUREAU OF RECLAMATION
REGION 4
GLEN CANYON DAM

Glen Canyon Dam, Reservoir, and Powerplant comprise the Glen Canyon Storage Unit.

The dam will be a concrete gravity-arch type, utilizing both its weight and arch design for strength. It will rise 700 feet above bedrock and 573 feet above the normal level of the Colorado River. It will be 1500 feet long at its crest. Thickness will vary from 300 feet at the base to 35 feet at the crest.

The cover photograph shows an artist’s interpretation of the completed project.

Glen Canyon Dam will contain more than 5 million cubic yards of concrete. By comparison, Hoover Dam has 3½ million and Grand Coulee has 10½ million cubic yards.

The reservoir behind Glen Canyon Dam will store more than 28 million acre-feet of water. (1 acre-foot equals 326,000 gallons, or enough water to cover 1 acre of land to a depth of 1 foot.) This reservoir capacity will be sufficient to contain approximately two years average flow of the Colorado River.

The lake formed will be long, winding, and narrow. It will extend 186 miles up the Colorado River and 71 miles up a principal tributary, the San Juan River.

The river elevation at Glen Canyon is 3142 feet and the normal high water elevation of the reservoir will be 3700 feet.

A single powerplant will be built, having eight generators with a total capacity of 900,000 kilowatts.

POWERPLANT ACCESS TUNNEL. Permanent vehicular tunnel from rim to river, 20' wide, 22' high, and 1 1/4 miles long.

BRIDGES AT GLEN CANYON DAMSITE

Bridges at the Glen Canyon damsite were one of the first and essential needs. The vertical walls of Glen Canyon rise about 700 feet above the river. Although it is only 1200 feet in a straight line from rim-to-rim, it is about 190 miles by road from one rim to the other.

A spectacular highway bridge is being constructed immediately downstream from the axis of Glen Canyon Dam. It will be the highest and second longest steel arch bridge in the United States. The 1028-foot arch of the Glen Canyon Bridge will stand 700 feet above the Colorado River. (See bridge as it will look when completed on the cover photo.)

The deck of the Bridge will be 1271 feet long and provide a 30-foot roadway with two 4-foot sidewalks.

A suspension-type footbridge was built by Merritt-Chapman and Scott Corp., the prime contractor for Glen Canyon Dam. Located upstream from the axis of the dam and completed in November 1957, the footbridge has been heavily used. It is not open for use by the public.

Also, several catwalks (temporary suspension-type walks) have been built at river level for the use of workmen at the damsite.

Materials and equipment are transported from rim-to-rim and to the canyon bottom by highlines. These are heavy cableways stretched between towers on each rim, with hooks and cables which permit moving and lowering loads up to 50 tons.
DEVELOPMENT OF UPPER COLORADO RIVER BASIN

The Glen Canyon Storage Unit is the key to the basinwide program for development and use of water resources in the Upper Colorado River Basin. Glen Canyon Dam is located on the Colorado River near the point where the river leaves the Upper Basin — as if in the stem of a great funnel so that runoff from the Upper Basin can be controlled.

Glen Canyon Reservoir will provide 80% of the storage capacity authorized by the Congress in 1956 as initial development for the Upper Basin. The Glen Canyon Powerplant will provide about 75% of the hydroelectric power installations.

Storage of water in Glen Canyon Reservoir will make possible diversion and beneficial uses of water at upstream points throughout the Upper Colorado River Basin. All irrigation, municipal, and industrial uses of water will be made on the participating projects — of which 11 are authorized and some 25 additional are in various stages of study.

Even though water will not be taken directly from Glen Canyon Reservoir for irrigation, this reservoir, along with the 3 other authorized storage unit reservoirs, will permit use of Colorado River water elsewhere.

The storage of river flows in high years will permit (1) regulated releases to meet downstream rights under the Colorado River Compact of 1922 and thus, (2) increased diversions for consumptive use on participating projects. Revenues from the sale of power produced at the large dams will assist in the repayment on a basinwide basis of all reimbursable costs, including interest on the investment in power facilities. Thus, the Upper Colorado River Basin development is self-liquidating with all costs repaid as required by law.
PHYSICAL DATA — GLEN CANYON STORAGE UNIT

HIGHWAYS TO GLEN CANYON DAM

Excellent, new, paved highways have been built to the Glen Canyon damsite. A 76-mile highway through highly scenic areas has been built from Kanab, Utah, on U. S. Highway 89 to the damsite. A 25-mile highway leaves U. S. 89 at Bitter Springs and extends northward to the damsite. Both of these highway links will connect with the Glen Canyon Bridge to form an alternate route for U. S. Highway 89. The Glen Canyon Bridge is scheduled for completion in February 1959.

DISTANCES TO GLEN CANYON FROM:

- Kanab, Utah: 76 miles
- Flagstaff, Arizona: 135 miles
- Cedar City, Utah: 161 miles
- Phoenix, Arizona: 500 miles
- Salt Lake City, Utah: 384 miles
- National Parks:
  - Zion: 100 miles
  - Bryce: 138 miles
  - Grand Canyon, North Rim: 134 miles
  - Grand Canyon, South Rim: 142 miles

LOCATION MAP — GLEN CANYON DAM

Glen Canyon Dam is being constructed in northern Arizona at a point about 13 river miles downstream from the Arizona-Utah State line, and about 15 miles upstream from Lee’s Ferry in Arizona.

Kanab, Utah and Flagstaff, Arizona, are the nearest existing towns to the damsite.

LOCATION MAP — GLEN CANYON DAM

Flagstaff, 135 miles from the damsite, and Marysvale, Utah, 190 miles from the damsite are the nearest railheads.

RECREATION PLANS FOR GLEN CANYON RESERVOIR

The Glen Canyon of the Colorado River is an unusually placid, 162-mile long reach from Hite, Utah, to Lee’s Ferry, Arizona. It was named “Glen Canyon” by Major John Wesley Powell, who headed the first expedition down the river in 1869, because of the “narrow gorges” along the banks and where tributary canyons joined the river.

The 166-mile long Glen Canyon Reservoir will extend upstream beyond Glen Canyon into Cataract Canyon.

The reservoir area will be established as a National Recreation Area under the National Park Service. (Lake Mead behind Hoover Dam is a National Recreation Area.)

Studies of the reservoir area are now being made by the Park Service, and a master recreation plan will be prepared. Later, facilities will be provided at suitable points in the Glen Canyon Reservoir area for boating, fishing, and other recreational activities.

CONSTRUCTION OF GLEN CANYON DAM

Glen Canyon Dam, like all large Reclamation dams, is being built by private construction companies under competitive contracts. The prime contract for Glen Canyon Dam was awarded to the Merrill-Chapman and Scott Corp. of New York City on April 19, 1956. This contract totalled $107,905,122 — the largest single competitively bid construction contract in the history of Reclamation.

Through the prime contract, the Federal Government buys one large package — the Glen Canyon Dam and powerhouse. The M-C & S Corp. marshals its resources and technical skills and does the job in accordance with designs and specifications prepared by the Bureau of Reclamation.

In general, the prime contractor will drill and concrete the diversion and spillway tunnels, build either dams (temporary earth structures) to divert the river around the dam site during construction, excavate the “foundation for the dam, place the concrete of the dam from bedrock to the crest, and build the powerhouse.

Certain materials (for example cement and prestress steel pipes) are supplied by the Bureau of Reclamation, but these are also purchased through competitive bids.

Additional contracts for generators, turbines, and other equipment will be awarded later to complete the dam and powerhouse.

It is anticipated that the dam and powerhouse will be completed and the first power produced in 1964.