Denver, Colorado  
June 4, 1959

Memorandum

To:     Chief Designing Engineer

From:   R. E. Glover

Subject: Features of recent European dam designs

Introduction

European and American design practices show some striking differences. Engineers on both continents seem to be in substantial agreement as to why this is so. The basic reasons are to be found in differences in topography and in a different economic balance. It should be profitable, however, to visit some of their structures and to discuss the details of design methods with their engineers to assess the possibilities of adapting some of their developments to American conditions. This is especially true since the remaining dam sites in the United States present difficulties of one kind or another.

The trend in Italy, Portugal, France, and Norway seems to be strongly toward the arch type of dam both for reasons of economy and because of the far greater strength of this type of dam. In countries where labor is cheap and plentiful, materials are dear, the construction season is short and transportation is difficult, refinement of design is imperative. Some or all of these factors are present at many of the European sites. The new developments which this situation has fostered, are the following:

(a) The dome or double curvature type of arch dam  
(b) The "pulvino" or cushion  
(c) The use of the crest of the dam as a spillway  
(d) The use of models as a means of structural design  
(e) The use of models for spillway design  
(f) Measurements of field performance of dams  
(g) The adaptation of the Trial-Load procedure for machine computation

The following structures and laboratories should provide an opportunity to study these developments.
Italy

The (S.A.D.E.) Societa Adriatica del Elettricità with offices at Venice. Dr. Carlo Senenza, Chief Engineer, Hydraulic Construction Department.

Vajont Dam. This is now under construction. It is of the dome type and has a "pulvino." When completed it will be the world's highest dam. The "pulvino" is a footing of concrete between the dam and its foundation. It generally extends completely around the perimeter of the dam. It has the following purposes:

(a) It provides a smooth, profile for which the dam may be designed without fear of changes due to foundation difficulties disclosed by the foundation excavation.

(b) It provides a means of spreading the arch loads on the abutments, controlling the pressures applied to the abutment, of bridging foundation weaknesses and of eliminating tension stresses at the abutment extrados.

The Europeans do not ordinarily cool their dams as we do and they, therefore, go through a period of "settlement" as the heat of setting is lost. The famed "perimetral joint" which is the joint between the dam, and the "pulvino" provides the flexibility to permit this. This joint is provided with waterstops but is generally left ungrouted, at least until the settlement period is over. The Vajont Dam will then permit the following factors to the observed:

(a) The topography of the site
(b) The dome type of dam
(c) The use of the "pulvino"
(d) Construction methods
(e) Transportation conditions
(f) Arrangement of spillways and powerplant with reference to the dam

Lamiei Dam

This is also a high dam of the dome type. A visit to it would permit a study of factors.

(a), (b), (c), and (f) as listed under the Vajont Dam.
(This dam is also called Nain di Sauris.)

Val Cellina Dam

This is a dam of the dome type as adapted to a somewhat wider site than is found at Vajont Dam and Lamiei. A visit to this
dam will permit a study of factors (a), (b), (c), and (f) as adapted to a moderately wide site.

**Pieve di Caddore**

This is a dam of the arch-gravity type. It is in an extremely wide site for an arch dam. A visit to this dam will show how the Italian engineers have adapted the arch type of dam to a very wide site. It has a massive "pulvino."

**Other Dams**

There are many other dams in the area including the Ponte-Alto arch dam. This is a very old dam which has been in service since 1611. It is near Trento.

The S.A.C.E. organization maintains an office of studies. This group makes measurements of the structural and thermal performance of actual dams and assists with design studies. It is this office, under Dr. D. Tomini, which has adapted the Trial-Load method of arch dam design to machine computation. A number of studies have been made by this adaptation of the Trial-Load method. It is an approved method of design in this organization.

**Societa Edison, Milan, Italy**

Claudio Marcello, Construction Engineer and Technical Director

**Santa Giustina Dam.** This is a remarkable structure. It is about 500 feet high and is constructed in a very narrow valley with almost vertical side walls. The design worked out for this case has only horizontal curvature. This dam is, therefore, not of the dome type but is what they call a single curvature arch dam. It has been extensively instrumented and its performance in service has been evaluated. The factors of topography and transportation should be quite evident here. The abutment treatment is of interest.

**Acciaiere e Ferriere Lombardie Fatck**

(Name of director not available)

**Osiglietta Dam.** This dam was constructed in the period 1937-1939. It is of the double curvature type. It has a "pulvino" and a perimetral joint. It is in a fairly wide site and has a height of 76.8 meters. The OSiglietta Dam has historical significance since it is the first of the dome type dams incorporating a "pulvino" and a perimetral joint. It is in the Apennines of Liguria and may be difficult to reach.
Earthquakes

Some of the thin arch dams of Italy have been subjected to severe earthquakes. Their experiences with this factor should make a valuable addition to our knowledge of arch dam behavior.

The I.S.M.E.S. Laboratory at Bergamo

Dr. Guido Oberti, Director

The initials are an abbreviation of the name Instituto Sperimentale Modelli e Strutture (Model and Structure Testing Institute). This fine new laboratory was built through the cooperative efforts of a group of companies engaged in the production of power, of manufacturers, and contractors to perform the functions indicated in the name of the laboratory. Elaborate arrangements have been installed to facilitate the testing of models of dams. The Italians depend heavily on such model tests when perfecting their dam designs. This laboratory has developed special materials for simulating, to scale, the strength and elasticity of prototype concrete and abutment rock. They make normal load tests and ultimate load tests on these models.

In addition to learning about the use of models as a tool for designing dams, a visit to this laboratory would permit a more fruitful study of the relation between computed and real factors of safety than could probably be obtained anywhere else. This is because they have many comparisons of safety factors derived from their normal load and ultimate load tests. In arch dams the ultimate strength is consistently above that indicated by the normal load tests. Computations are generally based upon the assumption of elastic behavior and for this reason correspond, more nearly, to the normal load test. Except for the most highly developed computation procedures the usual design methods ignore certain elements of strength in an arch dam. For these reasons ultimate factors of safety for arch dams commonly exceed the computed safety factors by a comfortable to a wide margin. This is not so with some other types of dams.

The Instituto di Idraulica e Construzioni Idrauliche at Milan

This is a facility of the Politecnico di Milano (University of Milan). De Marchi, Director - P. L. Romita, Assistant.

This laboratory makes designs for important hydraulic structures by the method of model testing. A knowledge of the advantages of this method of design could be obtained by a visit to this laboratory.
Portugal

The Laboratoris Nacional de Engenharia Civil, Lisbon, Portugal, M. Rocha, Acting Director and the Hydro-Electrica do Zezere, A. C. Xeres, Technical Director.

The national laboratory makes design studies by the use of structural models, Trial-Loads, and by Hydraulic models. The excellent results of these methods can be seen in the dams, Castelo do Bode, Bouca, and Cabril, owned by the Hydro-Electrica do Zezere. All of these dams have interesting features. Bouca has an overflow spillway of unusual design which has been tested by flood waters in service.

France

The E.D.F. (Electricite de France), Paris, France.

This organization has an arch dam of most unusual design. It is a thin arch designed for a stress of about 1,700 pounds per square inch. It is probably the most daring of all arch dam designs. This is the Le Gage Dam. It is located in the Department of Haute Loire and is probably most easily accessible by traveling down the Rhone Valley from Lyon. The larger dam, La Palisse, is close by. Because of its daring design, Le Gage has been extensively instrumented. A most instructive comparison between computed and measured stresses has been obtained from such measurements and computations. It is said that, the foundation deformations have exerted important influences. This dam is stated to have only 18 percent of the volume of a gravity dam designed for the same site. The effect of cracking has been observed here also.
MAY 8 1959

Mr. T. J. Lyons
Personnel Office
U. S. Geological Survey
Building 25
Denver Federal Center
Denver 25, Colorado

Dear Mr. Lyons:

In a telephone conversation with a member of this office on May 5, 1959, you advised that you had received approval from the Director, U. S. Geological Survey, for the additional 100-hour detail assignment of Mr. Robert E. Glover. If satisfactory with you, we will get in touch with him whenever his services are needed. We will ask that Mr. Glover negotiate with appropriate officials of your office to obtain whatever administrative concurrence is necessary before performing any further work under the old or newly authorized detail assignments. This is in lieu of arranging an advance schedule of assignments for Mr. Glover, which is almost impossible because of various factors which enter into the needs for his services, both in the U. S. Geological Survey and in the Bureau of Reclamation.

For your information, our records show that to date Mr. Glover worked 29 hours under the detail assignment about which we wrote you on December 4, 1958. If you will prepare your bill for reimbursement covering these services, we shall be glad to process it promptly. We are asking Mr. Glover to report further time he works for us to the customary segment of your office, so you may have information upon which to base future bills for reimbursement.

Funds have been prevalidated in the amount shown on the affixed stamp for services to be performed by Mr. Glover under the newly authorized 100-hour reimbursable detail assignment. Please refer to the encumbrance or obligation number indicated on this stamp when you prepare your bill for reimbursement, or correspondence regarding this transaction.
We wish to thank you very much for your cooperation in arranging Mr. Glover's detail to this office.

Sincerely yours,

C. H. Kadie, Jr.
Chief, Division of Administrative Services

Blind to: Mr. Robert E. Glover
1936 South Lincoln Street
Denver, Colorado

Note for Mr. Glover: As indicated in this letter, whenever we have need for your services, we will get in touch with you, informing you of our requirements and the approximate number of hours' work which is involved. You then should obtain approval from appropriate administrative officials of the U. S. Geological Survey to perform work for us. You should report all time worked by you in any segment of this organization to Mrs. Box in Mr. Garstka's office, to assure that the 100-hour limitation on your detail assignment is not exceeded. In addition, you should report all time you work for us to the office of the U. S. Geological Survey which customarily keeps your time record.
Stresses due to recovery of subcooling
Glen Canyon Dam

\[ E = 3,000,000 \text{ lb/lin}^2 \quad E_r = 600,000 \text{ lb/lin}^2 \]
\[ \alpha = 0.0000056 \text{ }^\circ F \quad \alpha_r = 0.08 \]
\[ c = 16^\circ F \]
\[ u = 0.2 \]
\[ \tau_{ct} = 268.8 \text{ lb/lin}^2 \]

For a height of dam of 700 feet and an average thickness of about 233 feet, there would be a ratio 2.3. From Voot’s "About the Calculation of Foundation Deformation (T.M. 77)" page 18, for \( m = 10 \).

\[ b = 233 \text{ ft} \]
\[ a = 700 \text{ ft} \]
\[ ab = 163,100 \text{ ft}^2 \]

The tabular value for \( b/a \) is 1.3 and \( m = 10 \) is 1.071

From formula 22:
\[ \Delta N_e = \frac{m^2 - 1}{m^2} \frac{b}{a} \left[ \frac{6 (b + a^2 + b^2)}{a} + \frac{m}{m - 1} \frac{6 (b + 1a^2 + b^2)}{b} \right] \]
\[ m = \frac{1}{0.08} = 12.5 \]
\[ \frac{m^2 - 1}{m^2} = \frac{155.25}{156.25} = 0.993600 \]
\[ m^2 = 156.25 \]
\[ \frac{m}{m - 1} = \frac{12.5}{11.5} = 1.086956 \]
\[ b + a^2 + b^2 = \sqrt{700^2 + 233^2} = 787.75 \]
\[ \frac{b + a^2 + b^2}{a} = 1.38678 \]
\[ \log 1.387 = 0.3275 \quad (0.3271) \]
\[ \log 1.8197 = 0.3045 \quad 1.8197 \quad \frac{1.8197}{3} = 0.6066 \]
\[ a + \sqrt{a^2 + b^2} = 6.17060 \]
\[ \log 6.170 = 0.7875 \quad (0.7877) \]
\[ \log 1.8197 = 0.3045 \]

Dwight
Cheek.
Thickener  
3715  25  
3360  185  
3010  295  

Mean Thickness

\[
\frac{25 + 4(185) + 295}{6} = \frac{1060}{6} = 177.16 \text{ ft (use 175.)}
\]

With \( a = 700 \) ft, \( b = 177.16 \) ft, \( 9/6 = 4. \) (nearly)

From Vogt's *Über die Berechnung*  
for normal load, \( b = 4a \)

\[
\Delta z = \frac{m^2 - 1}{m^2} \frac{E}{\rho} \alpha (1.664) \\
= \frac{0.9936}{0.75} \rho (175)(1.664) \\
= 3.35 \times 10^{-6} \rho \\
86,400,000. 
\]

\[
p = \frac{f \cdot x^2}{l} \frac{(550)}{(2)(700)} = \frac{(432)(60)(5.6)(16)(1535)}{1400} = 42800 \text{ lbf}
\]

Then \( \Delta z = (3.35)(10^{-6})(42800) = 0.1433 \text{ feet} \)

From the table 7 page 18, with \( h/\rho = 4 \) m = 10

The depth is 1.87. Then the shear deformation waves be \( (0.1433)(1.87) = 0.154 \text{ feet} \)

\[
\frac{f \cdot x^2}{l} = \frac{(432)(60)(5.6)(16)(1535)}{1400} = 38700 \text{ lbf}
\]
Then
\[ \Delta x'_c = \frac{Bd}{E} \left( \frac{0.9736}{3.1416} \right) \left[ 0.9862 \right] = \frac{Bd}{E} \left( 0.3119 \right) \]

With \( E = 86400000 \text{ in}/\text{f}^2 \quad a = 700 \text{ in} \)
\[ p = 1000 \text{ in}^2/\text{f}^2 \]
\[ \Delta x'_c = 0.002526 \text{ feet} \] (This is at a corner.)

At the center of the area \( \Delta x = 0.005052 \text{ feet} \) \( b = 1000 \text{ in}^2/\text{f}^2 \)
The unrestrained upward expansion due to 16° of uplifting would be \( \frac{1}{16} (700)(16)(6000.056) = 0.06272 \text{ feet} \)

Compute the vertical stress \( \sigma_y \) at the base by
The drawn circumference
\[ \sin \theta = 0.5 \quad \theta = \frac{\pi}{6} \]
\[ \sigma_y = \frac{E \Delta x}{L^2} \sum_{n=1}^{\infty} \frac{n \sin \frac{n \pi}{L} x}{n} \left( e^{-n^2 \beta_y} - e^{-n^2 \beta_y} \right) - \frac{E \Delta x}{L^2} \]

\n| \( n \) | \( \sin \frac{n \pi}{L} \) | \( \beta_y \) | \( 1 + \beta_y \) | \( e^{\beta y} \) | Term |
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\[ L = 1550 \text{ in} \]
\[ \frac{L}{2} = 002.02683 \]
\[ \frac{L}{4} = 1.27324 \]
\[ \frac{L}{4} \left( 2.067 \right) = 2.6318 \]

\[ 0.2067 \]
\[ \frac{L}{2} = 1.41878 \]

\[ 0.2067 \]
\[ \frac{L}{4} = 1.41878 \]
For a base thickness of 295 feet and an assumed mean thickness of 7.936 inches, the stress in the base due to complete restraint would be

\[
\frac{175}{295} \cdot 38700 = 23000 \frac{\text{lb}}{\text{ft}^2}
\]

On the basis that the deflection to the base would be

\[
(23.0)(0.0620) = 0.142 \text{ feet}
\]

The total displacement would be

Abutments: 0.1124 feet
Base: 0.142 feet
Total: 0.296 feet

The stress ratio would be

\[
\frac{0.06272}{0.296} = 0.212
\]

At the base the corrected stress would be

\[
(0.212)(71) = 15.4 \frac{\text{lb}}{\text{in}^2}
\]

The shear stress at dead 3400 would be about

\[
(0.212)(160) = 34.4 \frac{\text{lb}}{\text{in}^2}
\]

Check from Timoshenko's Elasticity, p. 335:

For a normal load with \(v = 0.5\), \(E = 155000\), \(f = 0.82\)

\[
\omega = \frac{P(1-\mu^2)}{E \frac{1}{A}} = \frac{(10.54)(0.5^2)(0.36)(0.82)}{(864)(70)(677)} = 0.147 \text{ lb}
\]

\[
A = (1550)(295) = 458000 \text{ ft}^2 \quad \frac{1}{A} = 677 \text{ ft}^2
\]

\[
P = (23000)(458000) = 10.54(10)^9
\]
Suppose the base of the dam covers an area
295 ft. by 1550 feet. \( b/d = 5 \)
\( b = 295 \) feet, \( d = 650 \text{ feet} \)

Hence, give the maximum vertical

displacement due to a load \( P \).

\[
\Delta z = \frac{m^2 - 1}{m^2} \frac{P_0}{E} (1.824)
\]

\[
= 0.9936 \frac{P_0}{E} 1.824 = \frac{P_0}{E} (1.816)
\]

\( P = 1000 \text{ lb/ft}^2, \ E = 86,400,000 \text{ psi} \)

\[
\Delta z = \frac{1000}{86,400,000} (295) (1.816) = 0.0062 \text{ ft}
\]

With complete vertical face, the load causing
wedge will be about 28.8 \( \text{lb/ft}^2 \) or
\( 38700 \text{ lb/ft}^2 \) on the base.

Thus, cause of deflection to be base of
\( (38700)(0.0062) = 0.24 \text{ ft} \).

(The deflection will be

With a completely rigid dam, the abutment stress
waver be in shear.

\[
\frac{(38700)(458,000)}{2(163,000)} = \frac{54400}{15} \text{ too high}
\]

The deflection of the abutment upward would be
by from page 2.

\[
(0.005052)(544) = 2.75 \text{ ft}
\]

Max value.

The total displacement of base and abutment
would be

\[
0.24 + 0.275 = 0.515 \text{ ft}
\]

The whole expansion would be about 0.6272 \( \text{ft}^2 \)

Then the stressed area would be

\[
0.14 = \frac{0.6272}{52.44} = 0.12 \text{ of base computed.}
\]

At the base of the concrete abutment the stress
would be about

\[
(198)(0.14) = 27.71 \text{ psi (1/4)}
\]

\[
(198)(0.14) = 27.71 \text{ psi (1/4)}
\]
Estimate the shear stress at the base of embankment B at elevation 3400, \( y = 315 \text{ feet} \)

\[
\sigma_y = \frac{Ey}{h} \sum \frac{1}{n} \cos \frac{n\pi}{13.5} y \left( \beta y e^{-\beta y} \right)
\]

\( n \) | \( \cos \frac{n\pi}{13.5} y \) | \( \beta y e^{-\beta y} \) | term.
--- | --- | --- | ---
1 | 0.638 | 0.5283 | -0.3370
3 | 1.914 | 0.1475 | -0.0941
5 | 3.192 | 0.0412 | -0.0263
7 | 4.469 | 0.0114 | -0.0073
9 | 5.746 | 0.0032 | -0.0020
11 | 7.022 | 0.00089 | -0.0006
13 | 8.299 | 0.00025 | -0.0002
15 | 9.576 | 0.00007 | -0.0000

Total.

\[
\frac{\Pi y}{h} = \frac{\Pi 315}{13.5} = 638.4
\]

Then the shear stress is 15 with no embankment yielding

\[
\sigma_y = (268.8)(1.27324)(0.4675) = 160 \frac{lb}{in^2}
\]

With embankment yielding accounted for, this would be reduced to about \((160)(12) = 19 \frac{lb}{in^2}\)

\((160)(14) = 22\).
\[ z = e^{-\rho y} \]

\[ \frac{\partial^2 z}{\partial y \partial y} = -\beta e^{-\rho y} \]

\[ \frac{\partial^2 z}{\partial y^2} = \beta^2 e^{-\rho y} \]

\[ z_1 = \beta y e^{-\rho y} \]

\[ \frac{\partial z_1}{\partial y} = \beta^2 y e^{-\rho y} + \beta e^{-\rho y} \]

\[ \frac{\partial^2 z_1}{\partial y^2} = \beta^3 y e^{-\rho y} - \beta^2 e^{-\rho y} - \beta e^{-\rho y} \]

\[ = \beta^3 y e^{-\rho y} - 2\beta^2 e^{-\rho y} - \beta e^{-\rho y} \]

\[ \frac{\partial^4 z_1}{\partial y^4} = \beta^5 y e^{-\rho y} - 2\beta^4 e^{-\rho y} - 2\beta^3 e^{-\rho y} - 2\beta^2 e^{-\rho y} - \beta e^{-\rho y} \]

\[ = \beta^4 \left( \beta y e^{-\rho y} - 4e^{-\rho y} \right) \]
\[ z_2 = \sin \frac{n\pi x}{L} e^{-\beta y} \]
\[ \frac{\partial^2 z_2}{\partial x^2} = -\frac{n^2 \pi^2}{L^2} \sin \frac{n\pi x}{L} e^{-\beta y} \]
\[ \frac{\partial^2 z_2}{\partial y^2} = \beta^2 \sin \frac{n\pi x}{L} e^{-\beta y} \]

Then, \( \left( \frac{\partial^2 z_2}{\partial x^2} + \frac{\partial^2 z_2}{\partial y^2} \right) = 0 \) if \( \beta = \frac{n\pi}{L} \)

\[ z_3 = \sin \frac{n\pi x}{L} \beta ye^{-\beta y} \]
\[ \frac{\partial^2 z_3}{\partial x^2} = -\frac{n^2 \pi^2}{L^2} \sin \frac{n\pi x}{L} \beta ye^{-\beta y} \]
\[ \frac{\partial^2 z_3}{\partial y^2} = \beta^2 \sin \frac{n\pi x}{L} \left( \beta ye^{-\beta y} - 2e^{-\beta y} \right) \]
\[ \frac{\partial^4 z_3}{\partial x^4} = +\frac{n^4 \pi^4}{L^4} \sin \frac{n\pi x}{L} \beta ye^{-\beta y} \]
\[ 2 \frac{\partial^4 z_3}{\partial x^2 \partial y^2} = -\beta^2 \frac{n^2 \pi^2}{L^2} \sin \frac{n\pi x}{L} \left( \beta ye^{-\beta y} - 2e^{-\beta y} \right) \]
\[ \frac{\partial^4 z_3}{\partial y^4} = +\beta^4 \sin \frac{n\pi x}{L} \left( \beta ye^{-\beta y} - 4e^{-\beta y} \right) \]

Then, \( \frac{\partial^4 z_3}{\partial x^4} + 2 \frac{\partial^4 z_3}{\partial x^2 \partial y^2} + \frac{\partial^4 z_3}{\partial y^4} = 0 \) if \( \beta = \frac{n\pi}{L} \)
The type term is

\[ z = \sin \frac{n \pi}{L} x (e^{-\beta y} + \beta y e^{-\beta y}) \]

\[ \frac{\partial^2 z}{\partial x^2} = -\frac{n^2 \pi^2}{L^2} \sin \frac{n \pi}{L} x (e^{-\beta y} + \beta y e^{-\beta y}) \]

\[ \frac{\partial^2 z}{\partial x \partial y} = +\frac{n \pi}{L} \beta \cos \frac{n \pi}{L} x (\beta y e^{-\beta y}) \]

\[ \frac{\partial^2 z}{\partial y^2} = \beta^2 \sin \frac{n \pi}{L} x (-\beta y e^{-\beta y} - \beta y e^{-\beta y}) \]
Memorandum

Chief Designing Engineer

Denver, Colorado
June 15, 1959

R. E. Glover

Control of stresses at abutments at Glen Canyon Dam

Introduction

At the present time the situation with regard to the Glen Canyon Dam appears to be as follows:

a. The abutment excavation is now in progress.

b. The contractor may be in a position to begin placing of concrete sometime this fall.

c. A meeting of the Consulting Board will probably be held in August or early September.

d. The trial load studies group will be occupied on present urgent studies until about July 1.

e. The photoelastic group has work under way which will be completed about July 1.

f. The vertical adjustment has not yet been incorporated into the trial load studies.

It is possible that the abutment excavations may reveal some poor rock which would need to be removed and in this way modify the shape of the profile in an undesirable way or require the local filling of an excavated portion of the abutment with a concrete plug. A study of Items (c), (b), and (a) above will show that by the time the Board of Consultants had reviewed the abutment shaping and made their recommendations, data on the proper shaping of any required plug or replacement would be urgently needed. It is the purpose of this memorandum to outline studies which would contribute to the early solution of such problems, if they should arise. Consideration has also been given to the possibility that a "cushion," a commonly used device in Europe, can be used to reduce the stresses which the dam applies to the abutment.

*The Italians call this a "pulvino." For a description, see the paper by Carlo Senese of Reference (g). It is a footing interposed between an arch dam and its abutments.
References

Reference is made to the following:

a. Details of final trial load study for Section 19-A for Glen Canyon Dam--Colorado River Storage Project; memorandum to L. G. Fuls from R. E. Glover, June 4, 1957

b. Evaluation of the effect of stepped abutments--Glen Canyon Dam--Colorado River Storage Project; memorandum to L. G. Fuls from R. E. Glover, February 27, 1959

c. Review of "Dams for Hydroelectric Power in Italy." Memorandum to L. G. Fuls from R. E. Glover, March 16, 1959

d. Preliminary report--Photoelastic Analysis of Arch and Abutment Stresses at Elevation 3500--Glen Canyon Dam--Colorado River Storage Project; memorandum to Chief, Dams Branch from H. Boyd Phillips and Ira E. Allen, April 29, 1959

e. Photoelastic Study of Abutment Stresses--Glen Canyon Dam--Colorado River Storage Project, by H. Boyd Phillips and Ira E. Allen, May 21, 1957

f. Stress at a crack, size of a crack, and the bending of reinforced concrete by H. M. Westergaard, in the Journal of the American Concrete Institute for November-December 1933, page 93

g. Arch Dams--Development in Italy, by Carlo Semenza--ASCE Symposium on Arch Dams, Paper 1017, August 1957. Published by Colorado State University, Fort Collins, Colorado.

Studies in Progress

Since the problem of designing a plug is similar to that of designing a cushion, the two cases may be considered together. In both cases, the problem consists of transferring a thrust into the abutment while holding the abutment stresses to the lowest practicable value. The thrust line at the ends of the arches cuts the abutment line downstream of the abutment center and is also inclined
to it. An equitable stress distribution along the foundation line would be obtained if the plug or cushion, heretofore referred to as a footing, could be shaped to make the thrust line pass through the center of its base. Due to the position and inclination of the thrust line at the abutment of the arch, the footing must have an extension downstream.

Another consideration enters here since, if the downstream extension is made too abrupt, tension stresses may be present in the overhanging portion of the footing. Some trial footing configurations are shown in Figure 1.

In the Case (a) of Figure 1, the conditions are as described for the arch ring at elevation 3400 in Reference (d). Here, the resultant cuts the arch abutment at a point 0.356L from the upstream face with an inclination whose tangent is about 0.12. In Case (b), the resultant is assumed to cut the arch abutment at a point 0.557L from the upstream face at an inclination whose tangent is 0.125. In Case (b) the resultant is at the downstream edge of the mid-third of the arch abutment. In both Cases (a) and (b) the resultant, extended, cuts the base of the footing at its center.

These configurations may serve as a starting point for more comprehensive studies. When incorporated into an actual construction, the footing would have a load on its upstream face and would also influence the position of the arch resultant through its effect upon the elastic properties of the arch ring. To evaluate these factors, a layout incorporating such footings has been prepared by Mr. Copan and the writer. One of Mr. Copan's assistants will make comparative studies for a selected arch ring to determine the effect of the footing on the position of the resultant and the stresses in the arch and on the foundation rock.

To determine whether footings of this kind will be free of tension, it is requested that the photoelastic laboratory be asked to study the configurations shown on Figure 1 to evaluate the stresses in the rock below the footings, the stresses everywhere in the footings, and the stresses in the ends of the arches over a length of 0.5L adjacent to the contact with the footings. It is of particular interest to find the principal stresses in the footings to learn whether tension occurs anywhere in them.

**Comments**

In European practice, the "purlina" or cushion commonly extends around the entire foundation line of the dam from the top on
one side, around the base to the top on the other side. All of the loads sustained by the arch dam must then be transmitted through the "pulvino" to the foundation rock. The famed "perimetal joint" extends around the entire perimeter of the dam at its junction with the "pulvino." (See Reference (2).)

Cooling of dams to bring them to a stable temperature state before the final grouting is done is not common in European practice. Their dams, therefore, go through a period of "settlement" as their excess temperatures, due to setting heat, are lost and the corresponding shrinkage occurs. The perimetal joint is not grouted but is, instead, provided with a waterstop so that it can open as necessary to provide the rotation needed to accommodate the settlement. (The reason why this does not produce excessive stresses may be found in Reference (7).)

The factor which determines whether cooling is required or not is thickness. For a given amount of natural cooling, the time required is proportional to the square of the thickness. For dams as high and as wide as Glen Canyon, the thicknesses will be enough to require cooling if temperatures are to be brought under control in any permissible time.

When cooling is to be done anyway, it becomes possible to carry the temperatures below the ultimate stable configuration so that some expansion of the concrete will occur after the grouting is done. The horizontal component of expansion produces bending moments in the arch rings which are opposed to those due to the water loads and such subcooling is, therefore, an effective means of equalizing the stresses in the arch rings. Provision has been made to subcool the Glen Canyon Dam to secure these advantages.

It is important to understand the difference this factor makes in the operation of a "pulvino." When subcooling is used, there is no "settlement" and no tendency for a perimetal joint to open. The pulvino becomes, then, an integral part of the arch and behaves quite differently than it would if settlement should occur. It should not be surprising, therefore, if the studies show that for a subcooled dam like Glen Canyon, the pulvino offers no important advantages and the arch rings may just as well be carried directly to the foundation. The repair of a foundation weakness would, however, make the use of some sort of a footing necessary and the photoelastic studies will provide design data which would be urgently needed if such a repair should be required.
FIG 1. TRIAL FOOTING CONFIGURATIONS
Compute stresses in Arch due to

\[ M = 103,760,000 \text{ lb} \text{ ft} \]
\[ T = 10,215,000 \text{ lb ft} \]
\[ S = 1,964,000 \text{ lb ft} \]

T is from Phillips & Allen

Eccentricity \[ \frac{103,760,000}{10,215,000} = 10.14 \text{ ft} \]

Arch Thickness \[ \frac{215}{2} = 107.5 \text{ ft} \]

\[ 107.5 + 10.14 = 117.64 \]

\[ (2)(117.64) = 235.28 \]

\[ \frac{1964,000}{10,215,000} = 0.384 \]

To make the line of Junction go through the center of the base of the pad, the center edge of the pad, or footing, should be on line 9.
Suppose we make the overhang half the thickness of the pad, $h$. Then

$$20.28 + 0.384h = \frac{h}{2}$$

$$20.28 = (0.500 - 0.384)h$$

$$h = \frac{20.28}{0.116} = 175 \text{ ft}$$

The overhang is $\frac{175}{2} = 87.5 \text{ ft}$

The total width of the pad is $215 + 87.5 = 302.5$

The bearing stress on the rock is

$$\frac{10 \times 215000}{302.5} = 33800 \frac{lb}{ft^2} \approx 235 \frac{kN}{m^2}$$

This is a lower value than necessary. The average value on the arch ring is

$$\frac{10 \times 215000}{215} = 47600 \frac{lb}{ft^2} \approx 331 \frac{kN}{m^2}$$

$$\frac{10 \times (103,760,000)(107.5)}{828,000} = 13560 \frac{lb}{ft^2}$$

or $94 \frac{kN}{m^2}$

Max. stress $331 + 94 = 425 \, \frac{lb}{ft^2}$
Conventional reinforced concrete practice permits 0.03 $f' /$ as the allowable shear without web reinforcement. If the 4000 $f' /$ strength (after 1 year) is used, the allowable shear is $$120 \text{ lb/in}^2.$$ From Table 1 for $f_c = 4000 \text{ lb/in}^2$,
$$f_c = 0.45 \quad f'_c = 1800 \text{ lb/in}^2;$$
$$N = 8, \quad f_s = 20000 \text{ lb}.$$

With $M = \frac{(235)(875)(12)^2}{2} = 129,700,000 \text{ lb}$.

The area of steel required for each of beam is

$$A_s = \frac{129,700,000}{(0.857)(175)(12)(20000)} = \frac{129,700,000}{36,000 \text{ sq in}} = 3.6\text{ sq in per each of weight of beam.}$$

For a span of lift the area of web be

$$(5)(12)(3.6) = 216. \text{ sq in}.$$  

The shear stress is

$$V = \frac{(235)(875)(12)}{(1)(0.857)(175)(12)} = \frac{235}{1.714} = 137.1\text{ lb/in}^2.$$
Check
The pull on the bar is, ft on each 12 ft length

\[
\frac{M}{10^6} = \frac{(235) (87.5) x^2}{(0.857)(175)(12)} = 72000 \text{ ft-lb}
\]

Then \[ A_1 = \frac{72000}{20000} = 3.6 \text{ in}^2 \]

The No. 11 bars have 1.56 square inches of reinforcement with more it would require.

\[
\frac{216}{1.56} = 139 \text{ bars} \text{ at each 5 foot lift}
\]

At 3 bars per foot this would require

\[
\frac{139}{3} = 46.3 \text{ feet}
\]

By turning up half of shear reinforcement they could be placed in 2 layers over about 24 feet.

By putting in a layer of straight bars a layer of turned up bars and a layer of straight bars on top of each other. They are laid in a band about 15 feet wide.
The worst case for the arch is when there is no tension at the extrados at the abutment as shown below.

\[ \frac{P}{1.333L} \]

\[ \frac{\phi}{1.333L} \cdot \frac{L}{2}\phi = \frac{1}{2.666} = 0.375 \approx \frac{3}{8} \]
At 3400 ft. (See memo of April 29, 1957)

Arch Thickness 286 Feet.

\[ M = 278,910,000 \, \text{lb} \]
\[ T = 17,437,000 \, \text{lb ft} \]
\[ S = 2,012,800 \, \text{lb ft} \]

\[ T = \frac{17,437,000}{286} = 61,200 \, \text{lb ft} = 424.1 \, \text{lb in}^2 \]

\[ T = \frac{286}{12} = 19,500,000 \, \text{lb in}^2 \]

\[ \frac{M \, \text{lbf}}{I} = \frac{(278,910,000)(43)}{19,500,000} = 20,450 \, \text{lb in}^2 \]

\[ = 150 \, \text{lb in}^2 \]

Then the extreme center are 274 1/2 in. and 574 1/2 in.

Check

\[ \frac{274 + 574}{2} = 424 \, \text{lb in}^2 \text{ OK} \]

Please check with plot in the above memo R42
The eccentricity is

\[
\frac{278,910,000}{17,437,000} = 16.0 \text{ feet}
\]

The tangent of the angle which the thrust makes with the base will be

\[
\frac{2,012,800}{17,437,000} = 0.1153
\]

\[
\frac{286}{16.0} = 143 + 16 = 159 \text{ ft}
\]

\[
\frac{159}{286} = 0.556
\]

Then the case will be as shown on the next page
Note: Max stress 540 \( \frac{\text{N}}{\text{m}^2} \) with resultant at downstream edge of the mid plane of the arch ring, with resultant at center of base 349 \( \frac{\text{N}}{\text{m}^2} \).

\[ (2)(0.556) = 1.112 \]

To make the overhang \( \frac{h}{2} \) set,

\[ 0.112L + 0.24h = \frac{h}{2} \]

\[ B = 1.112L + 0.24h = 1.715L \]

\[ h = \frac{0.112L}{0.24} = 0.431L \]
Average stress on the base of the pad.

\[
\frac{T}{1.215L} = 0.823 \frac{T}{L}
\]

\[
\frac{T}{L} = \frac{17437000}{286} = 61200 \text{ lb/in}^2 \text{ or } 424.15 \text{ lb/in}^2
\]

\[
(0.823) \frac{T}{L} = (0.823)(424) = 349. \text{ lb/in}^2
\]
Suppose the resultant acts the downstream edge of the mid third, at the end of the arch at an angle whose tangent is 0.125.

The path would be as shown on the following page.
\[(2)(0.665)L = 1.333L\]. To make the overhang \(\frac{L}{2}\),
set \(0.333L + 0.25h = \frac{h}{2}\), \(h = 1.333L\)

\(B = 1.333L + 0.25h = (1.333)(1.25)L = 1.667L\)
JUN - 9 1959

D-209-D

To: Commissioner
Attention: 200

From: Assistant Commissioner and Chief Engineer

Subject: Review of “Dams for Hydroelectric Power in Italy”
(your letter of January 8, 1959)

Enclosed is a review of Volumes 2 to 7 of the subject publication. The review, dated March 16, 1959, has been made by Robert E. Glover on detail assignment as consultant to the Bureau. Also enclosed is a memorandum dated May 18, 1959, from Chief Designing Engineer L. G. Puls commenting on the review and comparing foreign and Bureau design practices for arch dams.

Mr. Glover was selected to make this initial review on the basis of his recognized eminence in the dam design field, particularly in the area of mathematical analysis and experimentation techniques. Prior to his retirement from the Bureau in 1954 he was closely associated with our dam design practices for 30 years, and he was a major contributor to the development of design theory during much of that period. In 1956, he was chairman of the American Society of Civil Engineers Symposium on Arch Dams at Knoxville, Tennessee. The Symposium was beneficial in the exchange of information on dam design practices in Italy as well as other countries, and you will note Mr. Glover’s use of this background information in his review of the Anidel publication.

As suggested by Mr. Puls in his memorandum, we wish to defer comment on the proposal to send a team of our engineers overseas to study foreign structures. We believe that the plan to give Dams Branch Chief G. L. Rice the opportunity to inspect several foreign dams enroute home from his current assignment to Turkey may be helpful in reaching a decision on further such activities. After Mr. Rice makes his report we will be in a better position to evaluate the benefits to the Bureau from sending our engineers to study European dams in detail.

Enclosures

Blind to: 209
210
\- R. E. Glover

E. G. NEILSEN
ACTING ASSISTANT COMMISSIONER
AND CHIEF ENGINEER
Denver, Colorado  
May 18, 1959

Memorandum

To: Assistant Commissioner and Chief Engineer
From: Chief Designing Engineer
Subject: Review of "Dams for Hydroelectric Power in Italy"

Attached is a preliminary review by Robert E. Glover of Volumes 2 to 7 of the subject publication. Volume 1 is not available. This review was made in partial compliance with the Commissioner's letter of January 8, 1959. A more thorough review will be beneficial but will require extended period of time for research and study.

We subscribe to the selection of an arch dam at sites where this type can be used to advantage. Reducing volume of dams and increasing labor generally are not conducive to obtaining minimum costs in our sphere of economy. Unlike conditions that exist in many foreign countries, our labor costs are comparatively high and materials supply generally adequate. In many foreign countries, the reverse is true; materials are scarce while labor is cheap and abundant.

Economic considerations are constantly changing; and an important phase of our dam design work is to resolve the suitable balance of these considerations without compromise to acceptable safety criteria.

Although our interest is significant and foreign design practices contain features and methods of interest to Reclamation, it is suggested that we defer comment on the Commissioner's proposal to send a team of experts to examine foreign structures, permitting additional time for consideration of future international meetings.

/s/ L. G. Puls

Enclosure
Memorandum

To: Mr. L. G. Pula, Chief Designing Engineer

From: R. E. Glover

Subject: Review of "Dams for Hydroelectric Power in Italy"

Purposes

The memorandum of January 8, 1959, to the Assistant Commissioner and Chief Engineer from the Commissioner, on the above subject, notes that the seven-volume publication entitled "Dams for Hydroelectric Power in Italy" has been acquired and states that:

"We believe that it would be desirable for your office to assign some of your design engineers to review the drawings of the features incorporated in these dams. We would like to have you list the outstanding structures and to analyze their principal differences in design practice as revealed by the drawings. Also list the pros and cons for the use of such features and whether they would have potential application in Reclamation."

These notes are being prepared at your request to assist in making this review.

Material Available

Of the seven volumes of the complete set on "Dams for Hydroelectric Power in Italy" only the Volumes 2 to 7 are now available since Volume 1 has not yet been delivered. The first volume is to contain the technical data concerned with design and construction of dams while the other volumes contain the descriptions of over 130 dams. For the technical data relating to Italian practice, the writer has therefore relied on the papers presented at the Symposium on Arch Dams which was held at Knoxville, Tennessee, in June 1956 (Reference 2). (Of these papers four were contributed by Claudio Marcello who is also the Chairman of the A.N.I.D.E.L. (Association of Italian Electric Utility Companies) committee for the study of problems concerning dams who have produced the seven volumes under review.) Italian practices were well-represented in the Symposium.

Denver, Colorado
March 16, 1959
Increasing Use of Arch Dams in Italy

It is of interest to note the growth of popularity of the arch dam in Italy as revealed in the tabular summary, in the seven volumes, of the dams built before 1950, and those built or under construction after that date. Of the 237 dams built before 1950, there were 132 solid or cellular gravity dams and 34 arch or arch gravity dams. Of the 27 dams in construction since that date, 12 are solid or cellular gravity dams, while 11 are arch or arch gravity dams. One reason for this may have been expressed by Semenza in the paper on "Arch Dams: Development in Italy" (Reference 2, Paper 1017, Page 2) that "From a technical point of view, it is now considered, in short, that the overall safety of an arch structure is far higher than that of a gravity structure, as has indeed been proved in model tests to failure."

Italian Trends

The reasons for the arch dam development trends in Italy are also well-summarized by Dr. Semenza and his statements also give a clue to the reasons why American practice does not follow the same course. He says (Reference 2, Paper 1017, Page 2) "The rather remarkable development of arch dam construction in Italy is due to several factors which are partly interconnected. 
(a) The geological characteristics of the country. In the Alps, which constitute the principal hydraulic power area, and still more, in part, the Appenines, there is a prevalence of relatively recent formations, where fairly narrow gorges are quite common, for which the arch dam is the most natural solution.
(b) The cost of skilled and specialized labor in the construction industry is still relatively low and hence its factor in total cost is small. Materials, on the contrary, particularly cement and steel, are an important factor. The volume of concrete therefore constitutes the major element of cost.
(c) It is recognized in Italy as in any other country, that given the same degree of safety, the choice of the type of dams should depend only on the economic factor. Italian engineers for this reason have been driven to prefer increasingly refined structures in order to reduce the volume to a minimum.
(d) The peculiar characteristics of Italian mentality, which is fairly individualistic, and which therefore tends to examine every problem on its own merits, and free from any preconceived set of ideas. As a result of this attitude of mind, the principle, valid for any country, that each dam constitutes
a problem in itself to be solved according to criteria free from any preconceived idea, has found in Italy an ideal atmosphere for its full development, sometimes indeed beyond common limits. Hence, the widespread and elastic application of the most varied structural forms.

(e) This general tendency, which I will call mental, has been reinforced to some extent, both by tradition and artistic environment, since the arch, from the time of ancient Rome, has been a common architectural element. Thus, it was both logical and natural that modern designers of hydraulic structures should use it.

(f) The realization of the exceptional resistance of the arch has grown through centuries of experience in the Italian building workers whose craft has ancient traditions and deep intuition. Even for modest structures in house building, small and slender brick arches have been used for centuries, as, for example, in Romagna and Tuscany ***.

The principles which he here expresses so well, when applied to American conditions, with generally wider sites and a different economic balance, would naturally lead to thicker arch dams and with sections chosen to minimize the cost of form work. The double curvature dam, for example, seems much better adapted to Italian conditions than to American conditions.

Methods of Arch Dam Design

A perusal of the Symposium papers will indicate that while the Norwegians (Reference 2, Groen, April 1957) use Trial Loads, and the Portuguese (Reference 2, Rocha et al., Paper 997) use a combination of Trial Loads and models, the Italians rely principally on models. To facilitate testing of models of dams, the Italians have built a new and well-equipped laboratory at Bergamo. (Reference 2, Oberti, Paper 1351.) The Instituto Sperimentale Modelli e Strutture "I.S.M.E.S." (Model and Structural Testing Institute) operates this laboratory to solve specific structural problems which cannot be handled readily elsewhere. A study of the records of arch dams (Reference 2, Glover, Paper 1217) will indicate that in the rest of the world Trial Loads, Separate Arches, the thin cylinder formula, as well as the methods of Tolke and Smith, have also been used for design purposes. A method of plunging arches is used by the French (Reference 2, Coyne, Paper 959) who also make use of models. These methods of design are described in detail in a subsequent paragraph.
All of these methods have been used with conspicuous success since there is no actual record of the failure of an arch dam*. (Reference 2, Coyne, Paper 959) (Reference 2, Glover, Paper 1217.) With this experience one may well ask what is to be gained by a comparison of design methods. The answer is that the closer we can predict the performance of our structures, the better we will be able to use the resources at our disposal to achieve the results we desire in the most economical ways. A rather extreme example is given by Coyne (Reference 2, Coyne, Paper 959, Page 7) who estimates that the thin LeSage arch dam required a volume of concrete only 18/100 of what would have been needed for a gravity dam at the same site.

**Analytical Requirements**

An understanding of the requirements which must be fulfilled to arrive at a correct knowledge of stress levels and distributions in a dam is important because of its bearing on this review and seems best approached through the theorem of Kirchhoff** (Reference 3, Paragraph 118, Page 170). This theorem is concerned with the possibility that there might be more than one stress system which would sustain a load placed upon an elastic structure. He proves that there can be but one. Such a theorem is known as a uniqueness theorem. It is important to have this proved*** but the theorem has an additional importance in our case because it tells us, by implication, what requirements must be met if we are to obtain from our computations that one unique stress distribution which must prevail under the imposed conditions. These requirements are very easy to understand. If we imagine the volume of the dam divided into small elementary, approximately cubical elements, by passing through it and series of planes or surfaces, these requirements are:

---

*Two alleged failures can be found in the literature. These were the Lake Lanier and Moyie River dams. What actually happened was that both lost an abutment down to the base but both dams remained intact. The Lake Lanier Dam was repaired. The Moyie River Dam was not repaired (Reference 2, Glover, Paper 1217).

**Gustav Robert Kirchhoff (1824-1887). The theorem was published in 1859.

***Some similar engineering cases are not unique and this leads to difficulties whose source is often obscure. Soil pressures against retaining walls provide an example.
1. That each element must be in equilibrium under the forces and stresses which act upon it.

2. That as the dam passes from the unstrained to the strained state, each of these elements must deform in such a way as to continue to fit with its neighbors on all sides, and

3. The stresses or displacements must conform to those imposed at the boundaries.

These three conditions are usually referred to as the equilibrium, continuity, and boundary conditions, respectively, and we will use this terminology in what follows.

Models and Computations

We are now in a position to consider the question as to whether a model will give us a different stress distribution than we will get by computation. The answer to this question is that if the conditions represented by the model and accounted for in the computations are the same and the Kirchhoff requirements are met by the computations there can be no difference. This was demonstrated by Bureau of Reclamation model tests and computations made as a part of the design studies for Hoover Dam. The results of model tests* made by the group organized by I. E. Hooke were repeatedly compared with trial load analyses and it was found that as the computations were brought into progressively closer agreement with Kirchhoff's requirements, they approached also a complete agreement with the results of the model tests. Many comparisons of this sort are contained in the reports of the Arch Dam Investigations (Reference 7, Part II, Model of Gibson Dam, Figure 76), and The Boulder Canyon Project Final Reports. (Reference 6, Pages 336 to 399, inclusive.) It makes no difference how we go about the analytical work, if we finally succeed in satisfying the Kirchhoff requirements we will get the one unique solution which must prevail. Additional evidence on this score is furnished by the recent work of A. J. S. Pippard and Associates (Reference 8). They applied Relaxation procedures (Reference 11) to the computation of stresses in an arch dam. This method is quite distinct from Trial Load procedures but it will be noted (Reference 8, Page 227) that they also obtained a very satisfactory agreement with the results of model tests.

*These tests had the benefit of the technical skill of Dr. Fredrik Vogt.
It can be concluded that there will be no difference between the results of model tests and computations when both are correctly done.

**Difficulties and Advantages of Structural Model Testing**

The philosophy of model testing is very easily grasped but in practice some very real difficulties are present. The materials of a dam and its abutments may, and generally do, have different elastic properties. To properly model such cases it is necessary to develop a material whose properties can be controlled. The Bureau of Reclamation in their model work developed the plaster-celite material (Reference 6, Paragraph 8, Page 24). For such a material, workability, freedom from excessive shrinkage, availability, good bonding characteristics, a proper Poisson's ratio, and a proper relation between compressive and tensile strength, are required. If, in addition to tests under normal load conditions, where elastic behavior should be expected, tests to destruction are to be made to find the ultimate strength of the dam, then the strength characteristics of the dam concrete and the abutment rock must also be properly represented.

The task of finding such a material could fairly merit the description "impossible." And, so it is, but by diligent search, research, and testing, model materials can be developed which approach these ideals close enough to be useful.

Aside from the problem of applying the proper loadings, including the important vertical loads, a model has a certain inflexibility. It is known from the results of strain meter tests in actual dams (Reference 9), for example, that the method of construction has an important influence on the ultimate stress distribution in the dam. It is difficult to include this factor in model tests. The factors of temperature changes and earthquake effects also present difficulties to the model tester.

On the other hand, for the investigation of the ultimate strength of an arch dam, models have no competitors and information of the greatest importance has been obtained from them. It is found, for example, that an arch dam will ordinarily go well beyond the factor of safety estimated on the basis of elastic action. Sometimes the ratio of the observed to the computed factor of safety is as high as 2 to 1. They have also shown that failures can occur by the dam sliding up the abutments.

Model testing calls for adequate facilities, highly developed technical skills, special materials, and time.

In spite of the very real difficulties which model testing presents, it is possible, where the facilities and skills are available, to do a very satisfactory job of structural design with them. It is the impression of this writer that the Italian model work is excellent.
It should be mentioned here that the advantages of hydraulic models for such things as spillway design seem to be universally accepted.

**Difficulties and Advantages of Computation Methods**

Computation methods are limited to elastic conditions but inasmuch as the dam will be designed to work well within this range, this is not a serious drawback. The real difficulty with analytical methods is the almost heroic computation task they present to the designer. This writer has noted with some dismay that complete trial load analyses are seldom attempted outside of the Bureau organization*, although studies based on radial adjustments only are fairly common. If the computation difficulties could be overcome, this method would become a very effective design tool since many more studies could be made than are now possible because of the expense and time involved.

At the present time the development of digital computers affords a new possibility for reducing the time and cost of trial load studies. The Bureau of Reclamation Trial Loads group have already succeeded in programming one of these computers to do the arch computations. I saw the cards for the Fleming Gorge arch computations sent down to be run through the machine. These computations were completed in about 4-1/2 hours. In the earlier days, these arch computations represented half of the work of a trial load study and required about 2 months of time. Present indications, based upon an extrapolation of the results already obtained, indicate that if the trial load procedure can be completely programmed for machine computation, a complete study can be made in a week and at a cost of about 10 percent of what it would cost by the old methods. There is now good reason to believe that this can be done.

**Some Analytical Procedures Used for the Design of Arch Dams**

The design procedures used for arch dams may be described in the following way. An understanding of them is important in this review because of their bearing on questions of differences in design practice.

Thin cylinder formula: The dam is imagined to be divided into a series of horizontal arch rings by passing through

---

*The Portuguese engineers have recently made studies with twist and tangential adjustments included. Some Italian work has also been done with complete trial load analyses, based upon the use of digital computers. (Reference 18.)
it a series of horizontal cutting planes. Each of these arch rings is assumed to behave as a portion of a complete cylinder which sustains the reservoir pressure by the development of a uniform compression through the arch ring. It is realized that bending will be present and, to allow for this, the ring stress used in the computations is reduced to a half or less of the stress which is considered allowable.

Individual Arches

The entire volume of the dam is imagined to be divided into a series of horizontal arch rings, as described above. Each of these rings is assumed to be free to slide with respect to the arch rings above and below it. Stresses are computed by the elastic arch theory. The factors of reservoir load, temperature changes, abutment restraints, and bending can now be introduced.

Crown Adjustment

An elementary trial load procedure which employs arch computations of the type described for the individual arches method but which also makes an accounting of the ability of the dam to resist bending stresses which act vertically by introducing the effect of bending in vertical beam elements. Such a beam element is called a cantilever element. They are thought of as being formed by passing through the dam a system of coaxial vertical planes. The vertical line common to all of these vertical planes is located at or near the center of curvature of the arch elements. The totality of such cantilever elements make up the entire volume of the dam in the same manner as the totality of the arch elements make up the whole volume of the dam. When a crown adjustment is made, one of these cantilever elements near the crown or midpoint of the arches is selected as representative and the water load is divided between the arch and cantilever systems by trial to make their downstream deflections approximately equal to all elevations. The representative cantilever element so selected generally has its base at the lowest part of the canyon profile and it is therefore the tallest cantilever element of any in the dam. As the abutments are approached, the arch deflections will be less and the cantilever elements will generally be shorter so that their deflections will be less also. While these factors tend to preserve the deflection agreement reached at the crown, a check generally shows that the agreement away from the crown is imperfect.

Complete Radial Adjustment

In this version of the Trial Load Method the water load is divided among the arch and cantilever elements so that the downstream deflections of the two systems agree everywhere. It is to
be understood, of course, that since this is a cut and try process the agreement is never perfect.

**Complete Trial Load Study**

The volume of the dam is divided into arch and cantilever elements as before but it is now recognized that there are six components of displacement which must be brought to agreement. These comprise the linear displacements in the downstream, tangential, and vertical directions and the rotations about the arch radius, the arch tangent, and the vertical. It turns out that only three adjustments are needed. Forces are now introduced between the arch and cantilever elements to eliminate the deflection disagreements which may be thought of as having originated from an initial application of the entire water load to the arch elements only. These forces can be identified as the resistances which various elements of strength develop to assist the dam to carry the imposed loads. These elements of strength include the resistance to shear deformations caused by the tendency of the arch elements to deflect toward the abutment by different amounts at different elevations, the resistance to twist, and, the resistance to bending of the vertical elements as described under the complete radial adjustment heading.

This identification provides an explanation of the observation that arch dams designed by a wide variety of methods have invariably performed well. The explanation is that the simplified design methods ignore certain elements of strength. The dams designed in this way are thicker than necessary but this does not prevent them from standing up well.

**Relaxation Procedures**

A method of analysis based upon the concept of systematic relaxation of constraints. This method was developed by the English elastician R. V. Southwell. (Reference 11.) It was recently applied to arch dam stress analysis by Pippard and associates. (Reference 8.)

**The Method of Smith**

A method of stress analysis for arch dams developed by the Australian B. A. Smith. (Reference 12.) He considers the dam as a tapered cylindrical shell and develops a solution in terms of Michell functions. When the boundary conditions are imposed the deflection of the structure is determined. The Michell functions are related to Bessel functions.

**Method of Tolke**

A method of stress analysis of arch dams developed by Tolke for triangular, trapezoidal and hyperbolic profiles. The
solution for the triangular profile is obtained in terms of Bessel and Hankel functions of Order 1. When the boundary conditions are imposed the deflections are determined as in Smith's method. (Reference 13.)

Plunging Arches

The thrust trajectories for an arch dam, subjected to water pressures, are nearly horizontal at the crown but acquire a downward inclination as the abutments are approached. In the plunging arch method, arch rings following these thrust trajectories are substituted for the horizontal arches of the Individual Arches Method.

Tests of Arch Dam Design Methods Against the Kirchhoff Requirements

<table>
<thead>
<tr>
<th>Method</th>
<th>Equilibrium requirement</th>
<th>Satisfies continuity requirement</th>
<th>Satisfies boundary conditions</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thin cylinder</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>Effect of temperature changes cannot be included</td>
</tr>
<tr>
<td>Individual Arches</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Continuity approached for downstream deflections</td>
</tr>
<tr>
<td>Crown Adjustment</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Continuity restored for downstream deflections</td>
</tr>
<tr>
<td>Complete Radial Adjust.</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>All six displacements brought to agreement Kirchhoff's requirements substantially met</td>
</tr>
<tr>
<td>Complete Trial Load Study</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>If properly done substantially meets the Kirchhoff requirements</td>
</tr>
<tr>
<td>Relaxation</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>Plunging Arches</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>Smith</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td>Tolke</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td></td>
</tr>
</tbody>
</table>
Definition of Terms

The following definitions may be useful:

Gravity, Arch Gravity, and Arch Dam

As used by European engineers these terms seem to bear a closer relation to the mental state of the designer than to the physical realities.

A gravity dam is one designed to carry the reservoir loads by its resistance to overturning and sliding. An arch-gravity dam is designed to carry the loads by a combination of arch and cantilever action, and an arch dam is designed to carry its loads by arch action alone.

The physical realities may be considerably out of accord with these concepts. A scrutiny of Volumes 2 to 7 of the series under review will show that the Italians often curve their gravity dams. An analysis would probably show that arch action is an important element of strength in such structures. Nearly all arch dams develop some gravity action although it may not contribute much to the strength of the very thin dams.

The "Pulvino" or Cushion

This is a footing interposed between an arch dam and its abutments. It may serve several purposes, among which may be listed the following:

a. It distributes the thrusts imposed by the arches to the abutments.

b. It may be applied to bridge foundation weaknesses or to accommodate a transition from hard to softer rocks when the rock quality is not uniform over the full height of the abutment.

c. It permits the dam to be designed for a preselected profile which is often made symmetrical. This profile is constructed in the Pulvino.

The Perimetral Joint

The use of embedded pipe cooling to control temperatures in arch dams before closure is not common in Europe. As a consequence, their dams go through a period of "settlement" after construction. When the Pulvino is used, the joint between the dam and the Pulvino is deliberately used to provide freedom for rotation. A water stop is provided to prevent leakage but it is intended that the joint
will function to prevent the development of tensions at the upstream face by opening to accommodate the movements of the dam. This joint between the dam and the Pulvino or cushion is the perimetral joint.

**Comparison of American and European Dams**

As examples of American dams we may choose Hoover, Hungry Horse, Glen Canyon and Gibson, all by the Bureau of Reclamation, and the Ross Dam of the City of Seattle, and for purposes of comparison we may select the Vajont, Lussi, Santa-Giustina, Caglietta, Pieve di Cadore, all of Italian design. The Gibson Dam of the Bureau of Reclamation is an early design, having been completed in 1929. It is included here for comparison with the Pieve di Cadore design which also occupies a very wide site.

Some comparative dimensions are shown in the following two tables:

**Table 1**

<table>
<thead>
<tr>
<th>Dam</th>
<th>Height (feet)</th>
<th>Crest length (feet)</th>
<th>Maximum thickness (feet)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hoover</td>
<td>726</td>
<td>1,244</td>
<td>660</td>
<td>Completed 1934</td>
</tr>
<tr>
<td>Glen Canyon</td>
<td>700</td>
<td>1,550</td>
<td>340</td>
<td>Under construction</td>
</tr>
<tr>
<td>Hungry Horse</td>
<td>564</td>
<td>2,060</td>
<td>321</td>
<td>Completed 1952</td>
</tr>
<tr>
<td>Ross</td>
<td>540</td>
<td>1,300</td>
<td>208</td>
<td>Present stage</td>
</tr>
<tr>
<td>Gibson</td>
<td>195.5</td>
<td>960</td>
<td>87</td>
<td>Completed 1929</td>
</tr>
</tbody>
</table>
Table 2

Dimensions of Italian Arch Dams

<table>
<thead>
<tr>
<th>Dam</th>
<th>Height (meters)</th>
<th>Height (feet)</th>
<th>Crest Length (meters)</th>
<th>Crest Length (feet)</th>
<th>Maximum Thickness (meters)</th>
<th>Maximum Thickness (feet)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vajont</td>
<td>265.5m</td>
<td>871</td>
<td>190.5m</td>
<td>625</td>
<td>22.7m</td>
<td>74</td>
<td>Under construction</td>
</tr>
<tr>
<td>Main di Sauris</td>
<td>136.15m</td>
<td>447</td>
<td>138.36m</td>
<td>454</td>
<td>15.87m</td>
<td>52</td>
<td>Completed 1947</td>
</tr>
<tr>
<td>(Lumei)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Reference 1 (h)</td>
</tr>
<tr>
<td>Santa Giustina</td>
<td>152.5m</td>
<td>500</td>
<td>124.2m</td>
<td>406</td>
<td>16.50m</td>
<td>54</td>
<td>Completed 1950</td>
</tr>
<tr>
<td>Osiglietta</td>
<td>76.80m</td>
<td>252</td>
<td>224.0m</td>
<td>735</td>
<td>10.74m</td>
<td>35</td>
<td>Completed 1939</td>
</tr>
<tr>
<td>Fieve di Cadore</td>
<td>112 m</td>
<td>357</td>
<td>410 m</td>
<td>1,345</td>
<td>35.61m</td>
<td>117</td>
<td>Completed 1949</td>
</tr>
</tbody>
</table>

Note: Conversion to English units made by this writer.

A comparison of the Hoover and Vajont designs shows that while they are roughly of the same height (Vajont is the higher) the maximum thickness of the Vajont Dam is only about one-ninth that of Hoover. At Hoover, however, the top width approaches twice the height while at Vajont the top width is less than the height. Admittedly, Hoover could have been made thinner but it went beyond all precedents for height for its time, and was designed to very conservative standards. If designed to the same standards as Vajont it would probably be much the thicker dam because the site is less favorable. The Maini di Sauris and Santa Giustina dams may be compared to the Ross Dam. Again the ratio of top width to height is much in favor of the Italian designs. It is the belief of this writer that Italian type designs in the Ross site would not reduce materially the volume of concrete required below that of the present design.

The Osiglietta Dam illustrates the characteristics of the dome-type of design. There is no comparable American dam in the list.

The Gibson and Fieve di Cadore dams are both in wide sites. Except for a plug of concrete filling a deep gorge at
the Fierce di Cadore site, this dam rests on a horizontal base. The height from the Pulvino to the crest is 55 meters or 180.4 feet. On this basis they are not far from the same height. The 26 meters thickness at the base of the arch is equivalent to 85.1 feet. This is almost identical with the Gibson base thickness of 87 feet. The ratio of top width to height in the two cases is: Gibson 4.9 to 1 and Fierce di Cadore 7.4 to 1 based on height above the Pulvino. These two dams compare well.

As compared with Vajont the Glen Canyon site is very wide. Even though a somewhat soft abutment rock has required thickening the dam to spread the loads over a greater abutment area than would be required with a hard rock, this design is much thinner than Hoover which is of comparable height. Even with a hard abutment rock this dam should not be made as thin as Vajont.

**Comparison of Canyons**

A comparison of canyons for two high dams may be obtained from Figures 5 and 6. The very narrow canyon available at the site of the Vajont Dam may be noted. It is also V-shaped which is an aid to keeping the vertical components of the stresses within bounds.

**Differences of U. S. and Italian Practices**

The Italians often use the device called a "Pulvino" or cushion. This device is clearly shown in Figure 5. It is a footing interposed between the dam and the abutments. It can be used to distribute the arch thrusts to the abutment, to bridge foundation weaknesses, to smooth transitions between abutments of different elastic moduli and to provide a predetermined and properly shaped abutment for the dam.

The joint between the dam and the Pulvino is the famed "perimetral joint." A water stop is provided at this joint but no restraint against rotation at this joint is permitted. This is done to prevent the development of tensions at the upstream face at the abutments as the dam deflects under load. This device seems to have been used very successfully. There are good reasons to believe that the stresses developed at the joint will not be excessive (References 16 and 17).

Where the site is suitable they also use the dome-type dam (See Figure 7). Their reasoning is that, with curvatures in both the horizontal and vertical directions, this dam will act somewhat like a sail to distribute the loads in both directions.
and thereby make a better use of the concrete. Their claim probably has merit since a spherical shell will sustain an external pressure with only one-half the stress developed in a cylindrical shell of the same radius and thickness sustaining the same pressure. This dome shape leads to overhang and doubly curved surfaces which have been avoided in U. S. practice because of formwork complications.

The Portuguese engineers make another effective point favoring this type of dam. They note that when arch dam models are tested to destruction they often fail by sliding up the abutments. In the dome-type of dam the water forces are applied in such a way as will prevent this type of failure.

The use of the cushion, the perimetral joint, and the dome-type of dam are regarded as being technically sound. The choice between a single curvature and a double curvature dam could well rest solely on economic considerations.

A scrutiny of Volumes 2 to 7 of the "Dams for Hydroelectric Power in Italy" will show that even where the Italians design a gravity dam they often give it a curvature in plan. This is regarded as a practice which should be given the most serious consideration. It is difficult to design a gravity dam with a factor of safety in excess of two and even this can only be realized under normal conditions. An arch dam will generally have a factor of safety of from 5 to 10. Adding a curvature to such a dam can convert it from a dam with a poor factor of safety to a dam of tremendous strength. The argument that gravity dams should be made straight to facilitate the design of overflow spillway crests is considered to be without foundation. The curved plan should permit a longer crest and a better approach to the channel downstream.

The Italians often make use of the crest of a thin arch dam as an overflow spillway. Except for the reservation that this should never be done without a thorough study of its behavior by hydraulic model tests, there seems to be no reason why this type of spillway should not be used. There are certainly many examples of successful use of overflow spillways in Europe and America. The use of such spillways is facilitated in the Italian dams by their location of their powerhouses. These are generally away from the dam. A study of the volumes under review will show many cases where their arrangement comprises a dam, a tunnel leading away from the dam, a surge tank to control the flow in the tunnel, and penstocks conveying the water under pressure to a powerplant located on the river. This arrangement makes effective use of the fall of the river between the dam and the powerplant, and it also facilitates the use of an overflow spillway.
Allowable Stresses

The stresses considered allowable in Italy are believed to be comparable to those used in the U. S. Sesenza (Reference 2, Paper 1017) mentions stresses of the order of 60 to 70 kg/sq cm. This is equivalent to about 853 to 995 pounds per square inch. Bureau of Reclamation Standards permit up to 1,000 pounds per square inch. There is always a comparison with the ultimate strength of the concrete to be used. He mentions the factors one-fifth of the 28-day strength and one-seventh of the 90-day strength. (Reference 2, Page 1017.)

The LeGage Dam has stresses up to about 1,700 lb/in² (Reference 2, Paper 959, Page 4) but this is believed to be a special case, which should not be considered as representing normal practices.

Summary

There is no longer any mystery concerning what must be done to arrive at a proper evaluation of the stresses in an arch dam subjected to a specified system of loads. Satisfactory evaluations can be obtained by skillful model testing or by calculation. To be satisfactory, in this sense, the computations must fulfill the Kirchhoff requirements. Comparison of model test results and comparable computed results where Kirchhoff’s conditions are met invariably show agreement.

Under present conditions an acceptable evaluation by either structural model testing or Trial Load procedures requires a great deal of time and money. The alternatives may be presented in the following way:

a. Model tests: Require special skills, proper equipment, time, and money

b. Complete Trial Load studies: Require special skills, time, and money

c. Simplified analytical procedures: They buy relief from the cost of more elaborate studies by the use of excessive amounts of concrete in the dam

In large dams this cost may well be too high. In small dams the extra volume may not be objectionable since if they were designed rigidly to a stress allowance they could well be too thin to resist weathering.
It is the opinion of this writer that the differences between American and Italian Arch Dam designs is explainable principally by differences in topography and the economic balance in the two countries and that the differences in design methods have very little to do with it.

**Recommendations**

The cost in time and money for skillfully made model tests or acceptable stress analyses is at present high. If the cost and time required to make these studies could be reduced it would open the way to a more effective evaluation of possibilities than is now available. With improved methods of study, economy could be obtained by better and more effective use of materials and labor. To reach these desired objectives it is recommended to:

1. Explore diligently the possibilities of programming the high speed digital computers to perform the computations required for an acceptable stress analysis
2. To explore with contractors the probable costs of building arch dams with cushions, doubly curved surfaces, and overhang
3. To avoid the use of gravity dams where arch dams may be used both for reasons of economy and of strength
4. To use models to explore the ultimate strength of comparable arch and gravity designs.

/s/ Robert E. Glover
References

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17. Stresses at a Crack, Size of the Crack, and the Bending of
Reinforced Concrete, by H. M. Westergaard--Proceedings
of the American Concrete Institute, Vol. XXX, 1934,
Page 93

18. Algebraic Load Method of Analyzing Arch Dams, by
Dino Tonini--Sixth Congress of Large Dams, New York, 1958
Figure 1. Hoover Dam

Figure 2. Hungry Horse Dam
Figure 3. Gibson Dam

Figure 4. Ross Dam
Figure 5. Glen Canyon

Figure 6. Gorge at Site of Vajont Dam
Figure 7. Osiglietta
Figure 8. Maina di Sauris (Lusei)
Figure 9. Pieve di Cadore
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: Preliminary report—Photoelastic analysis of arch and
abutments stresses at elevation 3400—Glen Canyon Dam—
Colorado River Storage Project

Introduction

A photoelastic investigation is currently being made to
determine the stress distribution in the abutment of a dam from
the applied arch loads. Also being studied is the variation in
abutment stress distributions for a full radial, half radial, and
stepped half radial abutment. Another factor being investigated
is the effect of different ratios of moduli of elasticity of arch
and abutment.

This memorandum presents the preliminary results, to
date, of this investigation.

Results

The following aspects of the problem have been investigated:

1. Full radial abutment. See Figure 1:

   Change in normal stress, \( \sigma_y \), 5 feet into the abutment
   for ratios of moduli of elasticity of arch and
   abutment of 1 to 1, 3 to 1, 6 to 1, and 29 to 1

2. Full radial abutment. \( E_{dam}/E_{abut.} = 1 \). See Figures 2 and 3:

   \( \sigma_x, \sigma_y, \) and \( \tau_{xy} \) stresses at various depths into the abut-
   ment and in the arch 5 feet from the abutment

3. Half radial abutment. \( E_{dam}/E_{abut.} = 1 \). See Figures 4 and 5:
\(\sigma_x, \sigma_y, \tau_{xy}\) stresses at various depths into the abutment and in the arch 5 feet from the abutment

4. Stepped half radial abutment. \(E_{dam}/E_{abut.} = 1\). See Figures 6, 7, and 8:

\(\sigma_x, \sigma_y, \text{ and } \tau_{xy}\) stresses at various depths into the abutment and in the arch 5 feet from the abutment

Results of the study are presented in the form of stress curves on the figures referred to above.

Conclusions

From a study of Figure 1, it can be seen that, for a given arch reaction at the abutment, variation of ratios of modulii of elasticity of arch and abutment has no appreciable effect on the normal stress at the plane of contact for the full radial abutment. The sharpness of the arch corners seems to have more effect on the maximum concentrations than the hardness of the arch. It was possible to obtain sharper arch corners with CR-39 and aluminum, and it will be noted that these two gave the highest stress concentrations. Further studies are being conducted to verify this same conclusion for the half radial abutments.

From the curves of \(\sigma_y\) stress for the three conditions studied, it can be seen that high maximum stress concentrations tend to develop in the abutment at the corners of the arch. Comparing the curves, it appears the maximums are somewhat lower for the half radial and stepped half radial than for the full radial abutment. This may be due, in part, to the fact that the half radial and stepped half radial arches were grouted to the abutment with plaster of paris while the arch for the full radial abutment study rested directly on the abutment. A study will be made of the effect of grouting on the full radial abutment. From the data available at present, it appears that the half radial and the stepped half radial abutments actually provide a better distribution of normal stress in the abutment in that the high stress concentration at the upstream corner of the arch is almost completely eliminated.

Basic Data

<table>
<thead>
<tr>
<th>Elevation of arch studied</th>
<th>3400</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width of arch, full radial abutment</td>
<td>236 feet</td>
</tr>
</tbody>
</table>
Forces on arch at abutment per foot of thickness:

\[ M = 273,910,000 \text{ ft-lb} \]
\[ T = 17,437,000 \text{ lb} \]
\[ S = 2,012,800 \text{ lb} \]

Technical Details

This study was made experimentally, utilizing the photoelastic interferometer.

Model material for the abutment was Columbia Resin, CR-39, 1/4 inch thick. For the ratios of modulii of elasticity of arch to abutment of 1, 3, 6, and 29, the arch was made of CR-39, masonite, birch, and aluminum, respectively.

Scale of model to prototype was 1 to 960.

Personnel

The study was made under the general supervision of W. T. Moody.

H. E. Willmann assisted in the experimental work and prepared the drawings.

\[ \text{Ira E. Allen} \]

\[ \text{Ira E. Allen} \]
Abutment Material: CR-39 with all arches.
- 15 compression
COLORADO RIVER STORAGE PROJECT
GLEN CANYON DAM
PHOTOLEASTIC STRESS ANALYSIS
RIGHT ABUTMENT-ELEV 3400
FULL RADIAL ABUTMENT $\sigma_x$ AND $\tau_{xy}$

APR. 30, 1959
COLORADO RIVER STORAGE PROJECT
GLEN CANYON DAM
PHOTONELASTIC STRESS ANALYSIS
RIGHT ABUTMENT - ELEV 3400
HALF RADIAL ABUTMENT

NOTES
+
- tension
compression

APR. 30, 1959
Assumed full radial abutment (Trial load analysis)

U.S. DISTANCE FROM E FEET D.S.

NOTES
- tension and  — compression and

To 1000 psi

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

COLORADO RIVER STORAGE PROJECT
GLEN CANYON DAM
PHOTOELASTIC STRESS ANALYSIS
RIGHT ABUTMENT - ELEV. 3400'
HALF RADIAL ABUTMENT \(\sigma_x\) AND \(\tau_{xy}\)

APR. 30, 1959
Figure 6

Assumed full radial abutment (Trial load analysis)

U.S. DISTANCE FROM E FEET D.S.

200 160 120 80 40 0 40 60 120 160 200

Stress: psi

- Tension
- Compression

Assumed load distribution along full radial abutment (Trial load analysis)

NOTES

COLORADO RIVER STORAGE PROJECT
GLEN CANYON DAM
PHOTOELASTIC STRESS ANALYSIS
RIGHT ABUTMENT-EL. 3400
STEPPED HALF RADIAL ABUTMENT

APR. 30, 1969
COLORADO RIVER STORAGE PROJECT
GLEN CANYON DAM
PHOTOELASTIC STRESS ANALYSIS
RIGHT ABUTMENT-EL. 3400
STEPPED HALF RADIAL ABUTMENT

APR. 30, 1959
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

TECHNICAL DATA
FOR THE USE OF THE
BOARD OF CONSULTANTS
GLEN CANYON DAM
COLORADO RIVER STORAGE PROJECT

PREPARED BY THE OFFICE OF THE
ASSISTANT COMMISSIONER AND CHIEF ENGINEER

DENVER, COLORADO
APRIL 17, 1959

FOR ADMINISTRATIVE USE ONLY
FOREWORD

The data presented herein have been prepared by the Divisions of Design and Engineering Laboratories from studies and investigations conducted during the past year in the Denver office. The status of construction of the dam and powerplant was prepared by the Division of Construction.
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Status of Construction as of April 3, 1959--Glen Canyon Dam and Powerplant

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Thermal Control of Concrete in Foundation Lifts

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Concrete Mix Investigations to Determine Effects of Water-reducing, Cement-dispersing, Retarding Agents
STATUS OF CONSTRUCTION AS OF APRIL 3, 1959
GLEN CANYON DAM AND POWERPLANT

Right Abutment Excavation for Dam

Excavated to elevation 3152.

Excavation of the three 5-foot by 7-foot foundation tunnels completed.

Left Abutment Excavation for Dam

Excavated to elevation 3134.

Excavation of the three 5-foot by 7-foot foundation tunnels completed.

Dam Foundation

Excavation of overburden in river in progress.

Left Diversion Tunnel

Excavation and concrete lining completed.

Backfill and pressure grouting in progress.

Excavation of the access adit to the gate chamber had progressed 160 feet, leaving 240 feet to excavate.

Concrete at the entrance portal was placed to elevation 3228, about 16 feet above the roof of the tunnel.

Concrete lining for the temporary outlet works gate chamber was placed.

Right Diversion Tunnel

Excavation and concrete lining completed.

Backfill grouting completed.

Pressure grouting mostly completed before river diversion.

Drilling of drainage holes deferred until completion of diversion.
Left Spillway

Excavation in open cut in intake channel completed except for haul ramps and fine grading.

Excavation for spillway tunnel progressed from top to elevation 3255, approximately 140 feet, slope distance, remained.

Right Spillway

Excavation in open cut in intake channel completed except for haul ramps and fine grading.

Excavation for spillway tunnel started at top and advanced to elevation 3476 where work was suspended due to operations in diversion tunnel. Spillway tunnel excavation remaining after April 3, approximately 400 feet slope distance.

Powerplant Service Road Tunnel

Excavation of tunnel and adits completed.

Placing of crushed rock base on the roadway completed.

Control Cable Tunnel from Powerplant to Switchyard

Excavation completed.

Diversion and Care of River

The upstream cofferdam was constructed to average elevation 3225, a distance of 90 feet above the invert of the right diversion tunnel. The downstream cofferdam was constructed to average elevation 3155.

Powerplant

Excavation was in progress on both sides of the river between elevations 3285 and 3160.

Contractor's Plant and Equipment

The high-towered mobile cableway had been completed and tested.
The low-towered mobile cableway was complete except for installation of electrical control equipment.

Erection of cement and pozzolan silos had been completed.

Installation of the refrigeration plant was in progress.

Erection of the cooling tower was complete.

The concrete foundation for the main concrete batching and mixing plant had been placed and equipment was being received at the job site.

New rock ladders had been installed in the permanent storage area for aggregates.

Structural steel was being erected for the main aggregate plant in the Wahweap area and equipment was being received at the job site.
GEOL O GICAL CONDITIONS

April 1, 1959

From time to time during the past several months, attention has been given to individual joints or groups of joints exposed by excavations in the sandstone in localized areas such as the following: Open cut channel at the outlet of the left diversion and spillway tunnel including the downstream portion of that tunnel; left end of powerhouse (machine shop area) and adjacent vehicular "turn-around" area at the portal of the powerhouse access road tunnel; right end of the powerhouse; upstream face of the left keyway excavation; downstream face of right keyway excavation. In each case, consideration was given to the origin, position and possible widening of the joints and, as appropriate, to the need for remedial action such as special rock bolting, redesign of excavations, and measurements to record movement or widening of the joints. Up-to-date data on the localized joint areas consisting of photographs, geologic sections and measurements are being assembled by the Project Construction Engineer and will be supplied by him to the consultants at the time of the field examinations.
DESIGN STUDIES FOR GLEN CANYON DAM

April 9, 1959

1. Introduction

As the excavation progressed at the Glen Canyon Dam site, it became apparent that more rock cover at the downstream toe of the dam and a more nearly normal approach to the canyon walls were desirable. In order to achieve this condition, the lengths and directions of the tangents to the fillets on the downstream face of the dam were changed by moving their points of tangency toward the plane of centers. The design layout resulting from these changes is shown on revised Drawing No. 557-D-435, and is referred to as Design A-20.

Laboratory tests of rock samples from the dam abutment area have consistently indicated a lower modulus of elasticity in the bottom part of the canyon than in the upper portion. Trial-load analysis A-20a was made to determine the effect of this varying modulus in addition to the changes made in the layout.

2. Design A-20

A plan and maximum section of Glen Canyon Dam, Design A-20, are shown on revised Drawing No. 557-D-435. This design is the same as A-19 except for the changes in the tangent area near the abutments which were described in the introduction.

3. Design Criteria

Studies of Design A-20 were based on the following loading conditions and assumptions:

a. Top of dam, elevation 3715.

b. Normal reservoir water surface, elevation 3700.

c. Top of fill on downstream face, elevation 3158.

d. Minimum tail water surface, elevation 3142.

e. Temperatures used in analyses are changes between average arch temperatures at time of joint closure and minimum operating temperatures. Operating temperatures are assumed to vary linearly from upstream to downstream faces.
f. The effects of a construction and grouting program are included as follows:

(1) Concrete placed to elevation 3480; reservoir water surface, elevation 3240; joints ungrouted; no arch action.

(2) Concrete cooled to 40° F and contraction joints grouted to elevation 3480; concrete placed to elevation 3715 and water surface raised to elevation 3490. In the analysis, arch action is assumed below elevation 3480 but cantilevers only carry the loads above this elevation.

(3) Contraction joints grouted from elevation 3480 to 3715 after concrete has been cooled to temperatures varying from 40° F at elevation 3480 to 50° F at elevation 3715; reservoir water surface raised to elevation 3700 and the effects of earthquake, earth embankment, and tail water included. In the analysis, arch action is assumed throughout the dam.

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

g. Earthquake was assumed to move the dam upstream and downstream horizontally in the direction of the plane of centers with an acceleration of 0.1 gravity and a period of vibration of 1 second. The increased water pressure was assumed to act equally on all cantilevers. Effects of vertical acceleration were not included.

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

i. Modulus of elasticity of foundation rock is assumed to have a value of 500,000 pounds per square inch below elevation 3325, varying to 550,000 pounds per square inch at elevation 3400 then to 600,000 pounds per square inch above elevation 3475.

j. Poisson's ratio of concrete, 0.20.

k. Poisson's ratio of foundation rock, 0.08.

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

m. Coefficient of thermal expansion of concrete, 0.000,005,6 per degree Fahrenheit.
4. Arch, Cantilever, and Principal Stresses

Arch and cantilever stresses parallel to the faces of the
dam are shown on Drawing No. 557-D-1658. The maximum compressive
stress computed at the arch abutments is 821 psi at the intrados,
elevation 3715. Stresses of 612 psi and 620 psi were found at the
arch abutment intrados, elevations 3400 and 3325, respectively. All
other abutment arch stresses are near or below 500 psi. On Drawing
No. 557-D-1659 are plotted the stresses at the extrados and intrados
of the arch abutments and their variation from a stress of 500 psi.
Drawing No. 557-D-1660 shows the arch elements analyzed and the stresses
normal to their crowns and abutments. The dashed lines across the
stress diagrams at the abutments indicate the average stress at these
locations.

The maximum cantilever stress found at the bases of canti-
lever elements is 507 psi parallel to the downstream face of the crown
cantilever. All other stresses computed at the bases of cantilevers
were less than 500 psi. The maximum tensile stress found in the canti-
lever elements is 70 psi at the downstream face of Cantilever A, eleva-
tion 3625.

Principal stresses at the upstream and downstream faces of
the abutments are shown on Drawing No. 557-D-1661. The maximum com-
pressive principal stress at the upstream face occurs at elevation
3715 and has a value of 520 psi. All other principal stresses on the
upstream face are less than 500 psi. The maximum principal compressive
stress computed on the downstream face of the abutment is 821 psi at
elevation 3715. From elevation 3475 down to the base of the dam,
principal stresses exceeding 500 psi were found at every elevation at
the downstream face of the abutment. Only at two elevations studied
did these stresses exceed 600 psi, however; 628 at elevation 3400,
and 631 at elevation 3325. The maximum tensile principal stress at
the abutments is 115 psi at the downstream face, elevation 3625.

Load distributions and movements at arch and cantilever ele-
ments for Study A-20a are illustrated on Drawings No. 557-D-1662,
557-D-1663, 557-D-1664, and 557-D-1665.

5. Summary

The abutment stresses resulting from this analysis of
Design A-20 of Glen Canyon Dam are generally higher than those deter-
mined for Design A-19. The maximum compression of 821 psi is, however,
only 8 percent greater than the comparable stress found in Design A-19.
The increase in stress can be attributed primarily to thinning the
abutments in an effort to better satisfy the foundation conditions of
the site. The higher elastic modulus of the rock in the upper portion
of the site also contributes to higher stresses in this area. A dis-
cussion of the stress concentrations near the top of the dam and possible
means of relieving them are contained in a following paragraph.
6. Stress Concentration Near Top of the Dam

Drawing No. 557-D-1659 graphically illustrates the stresses acting at the ends of the arch element. Extrados stresses are plotted at the left and intrados stresses at the right of the canyon profile. The stresses at both faces have a marked tendency to peak at the top elevation of the dam. They amounted to 520 psi at extrados and 820 at intrados as computed for the subject Study A-20a, which includes reservoir full plus earthquake loading.

The principal cause of this stress concentration at the top of the dam is the very low rock modulus of the sandstone formation on which the dam is to be constructed.

Some of the methods which may be employed to reduce the peak stress are listed below:

(a) Two transverse contraction joints near each end of the dam have been provided with double seals to permit high pressures to be used in loading the abutments. Portions of these joints can be filled with water under high pressure (up to 200 psi) and later grouted with high pressure to permit more of the abutment load to be carried in the relatively low stress zone between elevations 3480 and 3660.

(b) The top grout lifts from elevation 3600 to elevation 3715 may be left ungrouted or grouted with reduced pressure to relieve some stress near the top of the dam.

As soon as the criteria for filling the reservoir is known, it will be possible to prepare a contraction joint grouting program which would provide for better stress distribution in the abutments.
THERMAL CONTROL OF CONCRETE  
IN FOUNDATION LIFTS 

April 6, 1959

Because the majority of structural cracking occurs at or near the foundation where the foundation restraint is the highest, the most rigid temperature control measures will be employed in those lifts of concrete which are within 30 feet of the foundation. Elsewhere through the dam, temperature rises will be held to 25°, which will result in a maximum temperature drop, to the grouting temperature, of 35°.

At the foundation of each block, certain measures will be taken to reduce cracking tendencies. The first measure will be to provide substantially plane surfaces for the beginning of each block by filling the depressions in the rock foundation with concrete before placing overlying concrete. Such dental work will be placed in the same manner as concrete in the dam, that is, with horizontal lifts not exceeding 7-1/2 feet in depth at any point and with a minimum of 3 days between successive lifts. Such a program of placement at the foundation should control those cracks which quite often develop due to unequal settlement around foundation irregularities.

The second step will be to control the maximum temperature of the concrete. As with all concrete in the dam, this concrete will be placed in the forms at a temperature of 50° F or less. On the foundation, the cooling pipes will be spaced 2-1/2 feet apart. This close spacing, combined with 3-foot spacing on the top of each lift, will hold the temperature rise to 20° to 22° in those lifts within 30 feet of the foundation rock.

Since the actual stresses tending to crack these blocks occur primarily during a drop in temperature, every effort will be made to maintain a uniform placement schedule near the foundation of each block. A uniform placement program will create minimum temperature (and stress) differentials because each lift will be placed when the concrete in the lift below will be at a temperature fairly close to its maximum temperature.

Cooling of all blocks prior to contraction joint grouting will take place in a relatively slow manner so as to create the lowest possible tensile stresses. The concrete will be cooled to 40° F in that part of the dam below elevation 3450 and to temperatures varying from 40° F at elevation 3450 to 50° F at the top of the dam, except that all concrete within 30 feet of the foundation will be cooled to a minimum of 45° F. Maximum temperature drops of less than 30° will therefore be obtained for all concrete within 30 feet of foundation rock. This would give the equivalent results to that obtained by placing this concrete in 2-1/2-foot lifts.
Several of the above temperature control measures will be improved in those blocks in the base of the dam because the contractor's program calls for the first concrete placement in the dam to be about November 1. As such, placing temperatures in the lowest part of the dam may be as low as 45°F for the first 2 or 3 months of placement. Another definite advantage will be the relatively low exposure temperatures to which these long blocks will be exposed during the first 3 or 4 months after placement.

The above measures, combined with the natural advantage of an extremely low modulus of elasticity of foundation rock, are believed capable of obtaining the desired monolithic structure.
Creep of Glen Canyon Dam Foundation Rock Cores

Under Sustained Load

April 2, 1959

Introduction

Creep measurements under sustained load have been made on twenty 6- by 16-inch cores from 2 vertical holes drilled in the foundation at Glen Canyon Dam site. These cores were placed under loads varying from 200 to 800 psi, and the results up to 2 years under sustained load are contained in this report.

Modulus of elasticity was measured by loading in a compression testing machine prior to placing under sustained load.

Data obtained on a previous test program started in 1950, is included at the end of this report.

Conclusions

The cores show little creep after the first few hours under sustained load despite their low modulus of elasticity (Figure 3).

These cores have low modulus of elasticity and are quite variable. Modulus ranges from 485,000 to 858,000 psi on initial loading (Table 1). The 1950 test program showed a range of modulus from 435,000 to 628,000 (Table 3).

Figure 3 is apparent creep or total deformation under sustained load and indicates the saturated cores as having a lower modulus, but less creep than the dry cores. However, the saturated control specimens which are not under load indicate an expansion which, if applied as a correction to the creep specimens, would result in the saturated cores having a higher creep than the dry cores.

The creep of the dry cores loaded to 200 psi is on the same order of magnitude as an average mass concrete loaded between the ages of 1 and 5 years. At higher loads, increase in creep of concrete is proportional to the increase in load, but these rock cores do not show a corresponding proportional increase in creep with increase in load.

The increase in modulus of elasticity from the first loading to the second loading was 24 percent for the dry cores and 14 percent for the saturated cores on an overall average. No change occurred between the second and third loadings.

Saturated cores showed 88 percent strain recovery and dry cores 83 percent when unloaded after initial loading for elasticity.
measurements (Figure 2). The 1950 series showed strain recovery of 74 to 85 percent on dry cores when unloaded after extended loading periods (Figure 5).

TEST PROCEDURE

Description of Cores

Twenty-three 6-inch-diameter cores were used in this test program. These were taken from 2 drill holes drilled vertically in the foundation. The identifying nomenclature given to the cores consists of 3 parts. The first number designates the drill hole number, the second number designates the core number, and the third number indicates the depth from which the core was taken. Core No. 56 Ka-23-40.1 indicates the core was taken from Drill Hole No. 56 Ka, that it was Piece No. 23 taken from this hole, and it was taken from a depth of 40.1 feet.

Half the cores were placed under test in an oven-dry condition, and the other half in a 100 percent water-saturated condition. All specimens were sealed in soldered copper jackets to prevent moisture change during test.

Initial Loading and Modulus of Elasticity

Modulus of elasticity of the cores was determined by loading the cores in compression in a testing machine. Two tests were made on each core with a time interval of 5 minutes between unloading from the first test to start of loading on the second test. Twelve days elapsed between the second loading and the third loading, which was the sustained load for measuring creep.

Sustained Loading

Sustained loads applied ranged from 200 to 800 psi, as shown in Table 1. Three cores were left unloaded as control specimens to measure any autogenous volume change. Two of these control specimens were saturated and one oven-dry.

Test Results

Modulus of elasticity on first loading varied from 486,000 to 858,000 psi (Table 1). Cores in the dry condition averaged 12 percent higher modulus than the saturated cores. Figure 1 is a typical stress-strain diagram obtained on one core showing the higher secant modulus values obtained at higher loads.

The dry cores showed 17 percent nonrecoverable deformation after the first unloading and the saturated cores 12 percent.
Practically no additional nonrecoverable deformation occurred after the second loading. The cores showed no recovery of this set during the 12-day interval at constant temperature and zero load between the second and third loadings (Figure 2).

Creep of these cores under sustained load up to 2 years' age is shown in Figure 3. Curves for 200, 400, and 625-psi loads in both the dry and saturated condition are each an average of 3 cores. The curve for the 800-psi load is from a single core. One core loaded to 800 psi in a saturated condition had high modulus of elasticity and the creep curve for this single specimen is about the same as for the saturated cores loaded to 625 psi. This curve is not shown. The cores showed considerable creep the first few hours under load. The sustained modulus of elasticity values given in Table 2 are calculated from total strain from the original condition before the first loading to 1 year and 2 years after the sustained load was applied.

Of the 3 control specimens which were not under load, the specimen in the dry condition has shown no length change after 2 years. The saturated specimen from Drill Hole 56 Ka shows 50-millionths expansion, and the saturated core from Drill Hole 56 La shows 90-millionths inch per inch expansion. However, most of this results from an expansion in a single gage line which leaves some question as to the exact cause of this expansion.

The modulus of elasticity determined from these tests may be higher than the overall average of the foundation rock as determined on smaller cores since only the longer pieces could be used in this program, and the weaker rock had a greater tendency to break into smaller pieces on removal from the drill holes.

1950 Test Program

Elasticity and creep results obtained on a preliminary test program conducted in 1950 gave values similar to those in the present program, although the modulus of elasticity was a little lower on an average. Modulus of elasticity and sustained modulus values are given in Table 3. Creep results are shown on Figure 4 and strain recovery in Figure 5.

Tests were made on 6-inch-diameter cores from holes drilled vertically and horizontally, and tested in saturated and oven-dry conditions. Deformation readings were taken from gage points cemented to each core with iron cement. Specimens tested in a saturated condition had a chemical reaction take place between the iron cement and the sandstone, which resulted in a movement of the inserts and a deposit of iron sulfate being formed which broke the moisture seal on all these specimens. Readings on these specimens, except for the initial readings, are not considered reliable and are not included in the reported data.
<table>
<thead>
<tr>
<th>Core No.</th>
<th>Condition</th>
<th>Stress, psi</th>
<th>Modulus of elasticity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1st loading</td>
<td>2nd loading</td>
</tr>
<tr>
<td>56 Ka-5-15</td>
<td>Dry</td>
<td>200</td>
<td>650,000</td>
</tr>
<tr>
<td>56 Ka-10-21.2</td>
<td>Dry</td>
<td>400</td>
<td>696,000</td>
</tr>
<tr>
<td>56 Ka-13A-25</td>
<td>Dry</td>
<td>625</td>
<td>645,000</td>
</tr>
<tr>
<td>56 Ka-13B-26.4</td>
<td>Saturated</td>
<td>200</td>
<td>545,000</td>
</tr>
<tr>
<td>56 Ka-14-28.4</td>
<td>Saturated</td>
<td>400</td>
<td>623,000</td>
</tr>
<tr>
<td>56 Ka-15-30</td>
<td>Saturated</td>
<td>625</td>
<td>632,000</td>
</tr>
<tr>
<td>56 La-1-40.1</td>
<td>Dry</td>
<td>200</td>
<td>858,000</td>
</tr>
<tr>
<td>56 La-2A-42.4</td>
<td>Dry</td>
<td>400</td>
<td>628,000</td>
</tr>
<tr>
<td>56 La-2B-43.7</td>
<td>Dry</td>
<td>625</td>
<td>792,000</td>
</tr>
<tr>
<td>56 La-3-46.3</td>
<td>Saturated</td>
<td>200</td>
<td>631,000</td>
</tr>
<tr>
<td>56 La-4-49.7</td>
<td>Saturated</td>
<td>400</td>
<td>615,000</td>
</tr>
<tr>
<td>56 La-5-52.7</td>
<td>Saturated</td>
<td>625</td>
<td>486,000</td>
</tr>
<tr>
<td>56 La-8-56.4</td>
<td>Dry</td>
<td>200</td>
<td>600,000</td>
</tr>
<tr>
<td>56 La-11-60</td>
<td>Dry</td>
<td>400</td>
<td>571,000</td>
</tr>
<tr>
<td>56 La-15-64.8</td>
<td>Dry</td>
<td>625</td>
<td>610,000</td>
</tr>
<tr>
<td>56 La-18-68</td>
<td>Dry</td>
<td>800</td>
<td>701,000</td>
</tr>
<tr>
<td>56 La-32-80.8</td>
<td>Saturated</td>
<td>200</td>
<td>600,000</td>
</tr>
<tr>
<td>56 La-36-85.7</td>
<td>Saturated</td>
<td>400</td>
<td>593,000</td>
</tr>
<tr>
<td>56 La-43A-95</td>
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<tr>
<td>56 La-43B-96</td>
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<td>56 Ka-23-40.1</td>
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<td>Control</td>
<td></td>
</tr>
<tr>
<td>56 La-41A-91.4</td>
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<td>Control</td>
<td></td>
</tr>
<tr>
<td>56 La-41B-92.5</td>
<td>Saturated</td>
<td>Control</td>
<td></td>
</tr>
</tbody>
</table>

**NOTE:** Five minutes elapsed between first and second loadings. Twelve days elapsed between second and third loadings.
<table>
<thead>
<tr>
<th>Condition</th>
<th>Load, psi</th>
<th>1st loading</th>
<th>2nd loading</th>
<th>3rd loading</th>
<th>Sustained modulus</th>
<th>Sustained modulus percent of initial modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry</td>
<td>200</td>
<td>703,000</td>
<td>821,000</td>
<td>849,000</td>
<td>81.8</td>
<td>82.5</td>
</tr>
<tr>
<td>Dry</td>
<td>400</td>
<td>698,000</td>
<td>853,000</td>
<td>859,000</td>
<td>87.7</td>
<td>86.7</td>
</tr>
<tr>
<td>Dry</td>
<td>665</td>
<td>692,000</td>
<td>889,000</td>
<td>954,000</td>
<td>95.3</td>
<td>95.9</td>
</tr>
<tr>
<td>Satuated</td>
<td>200</td>
<td>659,000</td>
<td>696,000</td>
<td>700,000</td>
<td>93.1</td>
<td>93.1</td>
</tr>
<tr>
<td>Satuated</td>
<td>400</td>
<td>610,000</td>
<td>690,000</td>
<td>733,000</td>
<td>89.3</td>
<td>89.3</td>
</tr>
<tr>
<td>Satuated</td>
<td>665</td>
<td>649,000</td>
<td>738,000</td>
<td>733,000</td>
<td>92.4</td>
<td>92.4</td>
</tr>
</tbody>
</table>

NOTE: Each result is an average of 3 cores.
<table>
<thead>
<tr>
<th>Condition</th>
<th>Load, psi</th>
<th>Direction of drill hole</th>
<th>Modulus of elasticity</th>
<th>Sustained modulus at 1,000 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry</td>
<td>300</td>
<td>Horizontal</td>
<td>576,000</td>
<td>496,000</td>
</tr>
<tr>
<td>Dry</td>
<td>600</td>
<td>Horizontal</td>
<td>628,000</td>
<td>577,000</td>
</tr>
<tr>
<td>Saturated</td>
<td>300</td>
<td>Horizontal</td>
<td>480,000</td>
<td></td>
</tr>
<tr>
<td>Saturated</td>
<td>600</td>
<td>Horizontal</td>
<td>514,000</td>
<td></td>
</tr>
<tr>
<td>Saturated</td>
<td>600</td>
<td>Vertical</td>
<td>435,000</td>
<td></td>
</tr>
</tbody>
</table>
FIGURE I. SUSTAINED LOAD TESTS
GLEN CANYON DAM FOUNDATION CORES
STRESS-STRAIN CURVES ON INITIAL LOADINGS
Figure 2. Sustained Load Tests - Glen Canyon Dam Foundation Cores Initial Loadings and Recovery

- Core No. 56 KA - 15-30
- Core Saturated

- Sustained load 625 psi
- Initial point on figure
- Loaded to 600 psi
- Specimen unloaded
- Non-recovered deformation
- Time in days

Strain - Millionth inches per inch

1 2 3 4 5 6 7 8 9 10 11 12 13 14 15
FIGURE 3. SUSTAINED LOAD TESTS—GLLEN CANYON DAM FOUNDATION CORES
CREEP DEFORMATION

NOTE: 800 PSI - 1 Core
All others - average of 3 cores

- - - Saturated
- - - Dry
**Figure 4 — Creep of Navajo Sandstone Cores from Glen Canyon Damsite**
FIGURE 5—CREEP AND CREEP RECOVERY OF CORES FROM GLEN CANYON DAMSITE
AGGREGATE AND DURABILITY DATA

April 1, 1959

No new information regarding aggregate testing or aggregate quality has come to light since the board meeting in 1958. Information regarding aggregate tests, including tests of physical and chemical properties of the aggregate, heavy media separation, and preliminary data on freezing and thawing of concrete, is summarized in the attached Report No. C-875, "Laboratory Investigation of Concrete Aggregate, Glen Canyon Dam," published June 2, 1958.
STATUS OF CEMENT AND POZZOLAN
April 2, 1959

CEMENT

Summary

The 4,000-pound sample of Riverside Cement from Oro Grande Mill (Laboratory No. M-3329) meets the specifications requirements. The 8 percent C3A determined is the maximum permitted under the specifications.

A series of 7-day heat of hydration tests has shown a characteristic difference in the values determined in the Oro Grande and Denver laboratories. Consistently, Oro Grande values have been 60 Cal./g. while Denver values have been 66 Cal./g.

After blending in our cement blending machine, this sample tested false set free. After aerating 20 hours, false set developed. (See Table 1).

The 3-day heat of hydration values determined in the Denver laboratory are:

Heat of Solution Method 54.4 Cal./g.
Conduction Calorimeter Method 56.4 Cal./g.

Discussion

Paragraph C-1 of Invitation No. DS-5023, Portland Cement for Glen Canyon Dam and Powerplant, specifies that "All cement shall be type II, low-alkali, in accordance with Federal Specifications No. BS-C-192-b, and shall meet the heat of hydration at 7 days and false-set limitations specified therein, except that the false-set limitation (difference between initial and final penetration) for cement to be supplied under Item 2 of the schedule, shall be not more than 12 millimeters **.""

Until their Clarkdale, Arizona Mill is in production, the American Cement Corporation proposes to furnish cement under Invitation No. DS-5023 from the Oro Grande, California Mill of Riverside Cement Company.

Four thousand pounds of cement (Laboratory No. M-3329), meeting specifications requirements, was obtained from the Oro Grande Mill for concrete mix investigations. Results of physical
and chemical tests of this sample are shown in the attached Table 1, "Cement Investigations."

Heat of hydration studies made in the USBR Denver Laboratory and the Riverside Cement Company Laboratory at Oro Grande, California, are compared in the attached Table 2, "Summary of Heat of Hydration Test Data (Laboratory Sample No. M-3329)." These data point out the following characteristic differences in results determined in the two laboratories:

<table>
<thead>
<tr>
<th>Zinc Oxide Calibration</th>
<th>Oro Grande</th>
<th>Denver</th>
</tr>
</thead>
<tbody>
<tr>
<td>C--Heat capacity of calorimeter as determined by ZnO, Calories/degree C.</td>
<td>394.519</td>
<td>398.646</td>
</tr>
<tr>
<td>R--Corrected temperature rise, degrees C.</td>
<td>4.557</td>
<td>4.509</td>
</tr>
</tbody>
</table>

Heat of Solution of Dry Cement

| H<sub>1</sub>--Heat of solution, Cal./g. | 604.008 | 605.332 |
| Ignition loss, percent | 1.61 | 1.33 |
| R--Corrected temperature rise, degrees C. | 4.521 | 4.490 |

Heat of Solution of 7-day Hydrate

| H<sub>2</sub>--Heat of solution, Cal./g. | 543.521 | 539.042 |
| Ignition loss, percent | 26.36 | 27.79 |
| R--Corrected temperature rise, degrees C. | 4.246 | 4.060 |

Heat of Hydration of 7-day Hydrate

\[ H_n = (H_1 - H_2) \]

| H<sub>n</sub> | 60.487 | 66.290 |

Heat of Hydration of 3-day Hydrate

| Heat of solution method | 54.4 |
| Conduction calorimeter method | 56.4 |

1/ These characteristic differences occurred even after exchanging samples of ZnO.

2/ 7-day hydrates prepared and cured the first 3 days in one laboratory gave the characteristic results of the laboratory in which they were prepared when tested in either laboratory.

3/ A sample prepared in Oro Grande Laboratory had 60 Calories/gram heat of hydration when tested in the Denver Laboratory. A sample prepared in Denver Laboratory had 66 Calories/gram heat of hydration when tested in the Oro Grande Laboratory.
Comparison of the Denver laboratory heat of hydration test results with the average results of 23 laboratories participating in the recent NBS Bi-monthly program, will be of interest here since the differences in the 7-day heat of hydration results are about the same as those existing between the Denver and Oro Grande Laboratories.

NBS Bi-monthly Test Program

<table>
<thead>
<tr>
<th>Sample Number</th>
<th>Sample</th>
<th>Avg. of 23 Labs.</th>
<th>USBR Minus Avg.</th>
</tr>
</thead>
<tbody>
<tr>
<td>7-day heat of hydration, Cal./g.</td>
<td>9</td>
<td>74.68</td>
<td>68.89</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>69.91</td>
<td>63.61</td>
</tr>
<tr>
<td>7-day ignition loss, percent</td>
<td>9</td>
<td>27.32</td>
<td>26.79</td>
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<td></td>
<td>10</td>
<td>26.85</td>
<td>26.35</td>
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<tr>
<td>28-day heat of hydration, Cal/g.</td>
<td>9</td>
<td>81.56</td>
<td>82.04</td>
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<tr>
<td></td>
<td>10</td>
<td>76.58</td>
<td>76.89</td>
</tr>
<tr>
<td>28-day ignition loss, percent</td>
<td>9</td>
<td>27.88</td>
<td>26.93</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>28.12</td>
<td>26.95</td>
</tr>
</tbody>
</table>

These data are being studied to try to determine the reasons for the differences shown.

A portion of the sample tested in the Oro Grande and Denver Laboratories will be sent to the Seattle, Washington Laboratory of National Bureau of Standards since that laboratory will make the acceptance tests.

Heat of hydration of Glen Canyon cement (M-3329) has been determined for the first 4 days in the conduction calorimeter and the results are tabulated below. For comparative purposes, laboratory blend Type II cement (M-3100) is also tabulated.

<table>
<thead>
<tr>
<th>Cement</th>
<th>Heat of hydration (cal/gm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 day</td>
</tr>
<tr>
<td>M-3329 (Glen Canyon)</td>
<td>42</td>
</tr>
<tr>
<td>M-3100 (Laboratory blend)</td>
<td>42</td>
</tr>
</tbody>
</table>

POZZOLAN

Summary

The supplier's (J. G. Shotwell's) anticipated source of pozzolan is the R. B. Bonner claims on U.S. Forest Service land, located approximately 25 miles north of Flagstaff, Arizona, adjacent to U.S. Highway 89, near Mile Post 440.5. Four thousand pounds of processed pozzolan was shipped from Flagstaff on March 30, 1959.
The attached Table III shows test results of all pozzolan samples in which Mr. Shotwell has been interested.

He submitted 3 Samples No. M-3287, M-3288, and M-3289, from the R. B. Bonner claims, July 22, 1958. M-3289 passed all of the specifications requirements.

It is expected that specification tests on the 4,000-pound pozzolan sample will be started about April 8, 1959, and completed about May 15, 1959.
CEMENT INVESTIGATIONS

Concrete Laboratory
Report No. C-
Date: January 15, 1959
Denver, Colorado
Compiled By: H. McFarland
Checked By: P. R. Tremutt
Reviewed By: R. J. Elfert

Project: Glen Canyon Dam, Colorado River Storage Project
Sample No. M-3329
Specifications No.: Invitation No. DS-5023
Date Received: September 9, 1958
Amount Received: 4,000 pounds
Type: Type II low alkali
Mill: Oro Grande, California
Letter of Transmittal: From: None--See memorandum dated August 27, 1958, from Mr. E. A. Curley, Chief Chemist to Mr. J. M. Sauer

<table>
<thead>
<tr>
<th>PHYSICAL PROPERTIES</th>
<th>CHEMICAL ANALYSIS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity:</td>
<td>SiO₂</td>
</tr>
<tr>
<td>3.13</td>
<td>23.25 Percent</td>
</tr>
<tr>
<td>Autoclave Expansion</td>
<td>Al₂O₃</td>
</tr>
<tr>
<td>0.026</td>
<td>4.65 Percent</td>
</tr>
<tr>
<td>Normal Consistency:</td>
<td>Fe₂O₃</td>
</tr>
<tr>
<td>25.2</td>
<td>2.34 Percent</td>
</tr>
<tr>
<td>Initial Set: Hours 3 Minutes 30</td>
<td>CaO</td>
</tr>
<tr>
<td>Final Set: Hours 6 Minutes 00</td>
<td>MgO</td>
</tr>
<tr>
<td>Compressive Strength of 2- by 2-inch Cubes</td>
<td>Loss on Ignition 1.40 Percent</td>
</tr>
<tr>
<td>3 Days 2,540 psi</td>
<td>Insoluble Residual 0.07 Percent</td>
</tr>
<tr>
<td>7 Days 3,773 psi</td>
<td>Na₂O</td>
</tr>
<tr>
<td>28 Days 6,217 psi</td>
<td>K₂O</td>
</tr>
<tr>
<td>Percent Air in Mortar</td>
<td>Na₂O + 0.658 K₂O</td>
</tr>
<tr>
<td>False Set ¼</td>
<td>SO₃ 1.91 Percent</td>
</tr>
<tr>
<td>Initial ½-min pen, mm 36</td>
<td>C₃S 42 Percent</td>
</tr>
<tr>
<td>5-min pen, mm 29</td>
<td>C₂S 35 Percent</td>
</tr>
<tr>
<td>Remix ½-min pen, mm</td>
<td>C₃A 8 Percent</td>
</tr>
<tr>
<td>Remix 5-min pen, mm</td>
<td>CaSO₄ 3.2 Percent</td>
</tr>
<tr>
<td>Wagner Surface, cm²/g</td>
<td>C₃AF 7 Percent</td>
</tr>
<tr>
<td>Percent Passing No 325 Sieve</td>
<td>C₃A 8 Percent</td>
</tr>
<tr>
<td>Blaine Surface, cm²/g 3,407</td>
<td>CaSO₄ 3.2 Percent</td>
</tr>
</tbody>
</table>

Remarks: Heat of hydration at 7 days: 66 calories/gram
Note: This sample meets the specification requirements of Invitation No. DS-5023. However, it should be noted that the 8 percent C₃A is the maximum allowed under these specifications.

Additional False Set Tests

<table>
<thead>
<tr>
<th>Date</th>
<th>Time of aerated test hours ½</th>
<th>Water: cc</th>
<th>Initial penetration: mm</th>
<th>5-min penetration: mm</th>
<th>Initial penetration: mm</th>
<th>5-min penetration: mm</th>
<th>Remix</th>
</tr>
</thead>
<tbody>
<tr>
<td>9-15-58</td>
<td>--</td>
<td>132</td>
<td>28</td>
<td>25</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>9-15-58</td>
<td>--</td>
<td>135</td>
<td>36</td>
<td>29</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>2-3-59</td>
<td>--</td>
<td>135</td>
<td>31</td>
<td>20</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>2-4-59</td>
<td>20</td>
<td>138</td>
<td>25</td>
<td>0</td>
<td>28</td>
<td>30</td>
<td>--</td>
</tr>
<tr>
<td>2-4-59</td>
<td>20</td>
<td>146</td>
<td>39</td>
<td>1</td>
<td>40</td>
<td>39</td>
<td>--</td>
</tr>
</tbody>
</table>

1/ Aerated sample placed in pan to a depth of approximately 1 inch and exposed to laboratory air for time shown.
<table>
<thead>
<tr>
<th>Group: tests</th>
<th>Riverside Cement Co. Lab, Oro Grande</th>
<th>USBR Laboratory, Denver</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of tests</td>
<td>1: IGN</td>
<td>2:</td>
</tr>
<tr>
<td>Group: tests</td>
<td>t, tloss, 0*40-620, C, R, H</td>
<td></td>
</tr>
<tr>
<td>No. averaged</td>
<td>°C</td>
<td>%</td>
</tr>
<tr>
<td>1 2</td>
<td>24.124</td>
<td>0.050</td>
</tr>
<tr>
<td>2 8</td>
<td>23.898</td>
<td>0.069</td>
</tr>
<tr>
<td>3 4</td>
<td>24.012</td>
<td>1.61</td>
</tr>
<tr>
<td>4 5</td>
<td>23.505</td>
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<tr>
<td>5 5</td>
<td>23.714</td>
<td>26.36</td>
</tr>
</tbody>
</table>

Typical 7-day heat of hydration, Cal/g, (Group 3 minus Group 5) 60.487
Typical 7-day heat of hydration, Cal/g, (Group 8 minus Group 10) 66.290

Heat of hydration--3-day hydrate

<table>
<thead>
<tr>
<th>Method</th>
<th>Heat of solution method</th>
<th>Conduction calorimeter method</th>
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<tbody>
<tr>
<td>Value</td>
<td>54.4</td>
<td>56.4</td>
</tr>
</tbody>
</table>

Note: 1/Final calorimeter temperature at end of 20-minute solution period.
2/Correction for heat lost or gained during solution period.
3/Heat of solution.
Group 1: Sample of zinc oxide from Denver Laboratory tested at Oro Grande Laboratory.
Group 4: Sample prepared in Denver Laboratory, cured 3 days, then shipped via airmail to Oro Grande Laboratory for test.
Group 6: Sample of zinc oxide from Oro Grande Laboratory tested at Denver Laboratory.
Group 9: Sample prepared in Oro Grande Laboratory, cured 3 days, then shipped via airmail to Denver Laboratory for test.
<table>
<thead>
<tr>
<th>Number</th>
<th>Date of</th>
<th>Site of</th>
<th>Date of</th>
<th>Petrographic description</th>
<th>Fractionation for test</th>
<th>Physical properties</th>
<th>Dedication of</th>
<th>Chemical composition</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
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<td>exposure</td>
<td></td>
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<td>30</td>
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<td>8-25/50</td>
<td>9-10</td>
<td>100</td>
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<td>8-309612</td>
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<td>8-25/50</td>
<td>9-10</td>
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<tr>
<td>8-3090</td>
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<td>8-25/50</td>
<td>9-10</td>
<td>100</td>
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<tr>
<td>8-1500</td>
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<td>9-10</td>
<td>100</td>
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<tr>
<td>8-1599</td>
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<td>9-10</td>
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<td>8-25/50</td>
<td>9-10</td>
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<td>8-2081</td>
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<td>8-25/50</td>
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<td>8-2081</td>
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<td>Flagstaff, Arizona</td>
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<tr>
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<td>80</td>
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<td>8-25/50</td>
<td>9-10</td>
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</tr>
</tbody>
</table>

**Table 3**

**SUMMARY OF SPECIFICATION TESTS OF POZZOLAN SAMPLES**

<table>
<thead>
<tr>
<th>Number</th>
<th>Date of</th>
<th>Site of</th>
<th>Date of</th>
<th>Petrographic description</th>
<th>Fractionation for test</th>
<th>Physical properties</th>
<th>Dedication of</th>
<th>Chemical composition</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
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<td>sample</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Chemical composition**

- *Mo* = % by weight of moisture content
- *Loss on ignition* = % by weight of dry sample
- *Ash* = % by weight of ash residue

**Notes**

- *Percent shrinkage of pozolane bar* is the percent shrinkage of control bar.
TEST RESULTS ON VOLUME CHANGE TESTS FOR GLEN CANYON POZZOLANS

April 1, 1959

TESTS WERE CONDUCTED ON 4" BY 4" BY 30-INCH BEAMS

Conclusions

All pozzolans except fly ash increase the drying shrinkage.

All pozzolans tested in this program increased the water content in order to yield constant slump.

Autogenous shrinkage is increased when 30 percent of the cement is replaced by pozzolans.

Pozzolans increase expansion of continuously fog-cured concrete.

Mixes GCD-31 through GCD-72 were made with pozzolans from deposits near Glen Canyon Dam (see Table 5, Concrete Laboratory Report C-526A).

Test results are shown on the attached Table 1, Figure 1.
### Table 1

**GLEN CANYON DAM POZZOLAN INVESTIGATION**  
Volume Change in Millionths  
4- by 4- by 30-inch Bars  
Average of Two Specimens

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>Pozzolan No. and type</th>
<th>Drying shrinkage</th>
<th>Autogenous shrinkage</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>50% r.h.--73.4°F</td>
<td>73.4°F</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1 year</td>
<td>1 year</td>
</tr>
<tr>
<td>G.C.D.-31</td>
<td>Control</td>
<td>407</td>
<td>26</td>
</tr>
<tr>
<td>G.C.D.-32</td>
<td>2833 clay</td>
<td>472</td>
<td>31</td>
</tr>
<tr>
<td>G.C.D.-33</td>
<td>2834 clay</td>
<td>482</td>
<td>31</td>
</tr>
<tr>
<td>G.C.D.-34</td>
<td>2835 volcanic ash</td>
<td>568</td>
<td>43</td>
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<tr>
<td>G.C.D.-37</td>
<td>Control</td>
<td>400</td>
<td>25</td>
</tr>
<tr>
<td>G.C.D.-38</td>
<td>2858 volcanic ash</td>
<td>443</td>
<td>57</td>
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<td>G.C.D.-41</td>
<td>Control</td>
<td>400</td>
<td>-7*</td>
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<td>2883 pumice</td>
<td>507</td>
<td>35</td>
</tr>
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<td>7</td>
</tr>
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<td>Control</td>
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<td>21</td>
</tr>
<tr>
<td>G.C.D.-58</td>
<td>2907 shale</td>
<td>506</td>
<td>35</td>
</tr>
<tr>
<td>G.C.D.-53</td>
<td>Control</td>
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<td>34</td>
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<tr>
<td>G.C.D.-64</td>
<td>2942A pumice</td>
<td>560</td>
<td>71</td>
</tr>
<tr>
<td>G.C.D.-69</td>
<td>Control</td>
<td>380</td>
<td>14</td>
</tr>
<tr>
<td>G.C.D.-70</td>
<td>2834 volcanic ash</td>
<td>492</td>
<td>19</td>
</tr>
<tr>
<td>G.C.D.-71</td>
<td>2858 volcanic ash</td>
<td>492</td>
<td>14</td>
</tr>
<tr>
<td>G.C.D.-72</td>
<td>2907 shale</td>
<td>492</td>
<td>23</td>
</tr>
</tbody>
</table>

*Minus sign indicates expansion*
VOLUME CHANGE TESTS

Drying Shrinkage in Millions Inches Per Inch

Glen Canyon Dam Pozzolan Investigation Drying Shrinkage

PRELIMINARY
Introduction

The effect of "Pozzolith 8" retarding agent on the heat generation of cement has been investigated in both an adiabatic temperature rise test of concrete to 28 days, and in neat cements in the conduction calorimeter to age 4 days. Temperature rise test concretes contained Glen Canyon Dam aggregate and three sacks of laboratory blend Type II cement per cubic yard. One test contained no retarder, and the second test contained 0.37 percent of "Pozzolith 8" by weight of cement.

Conduction calorimeter tests were made using the same cement and retarder in amounts of 0.25 and 0.50 percent by weight of cement.

In addition to "Pozzolith 8," tests on four other retarders have been made in the conduction calorimeter and the results summarized in Table 3.

Conclusions

1. The adiabatic temperature rise of concrete with "Pozzolith 8" retarding agent was less at early ages, the same at age 4 days, and slightly higher at later ages than the concrete without retarder having the same cement content (Figure 1).

2. The slightly higher apparent temperature rise of concrete with retarder is due in part to the delay in the initial heat of wetting, which is delayed until after the initial temperature is taken and results in higher measured temperature rise (Figure 2).

3. The length of time of initial retardation as estimated from the rate of heat generation curves (Figure 4) was:

<table>
<thead>
<tr>
<th>Test Type</th>
<th>Retarder %</th>
<th>Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conduction calorimeter test</td>
<td>0.25%</td>
<td>6 hours</td>
</tr>
<tr>
<td>Conduction calorimeter test</td>
<td>0.50%</td>
<td>14 hours</td>
</tr>
<tr>
<td>Temperature rise test</td>
<td>0.37%</td>
<td>7 hours</td>
</tr>
</tbody>
</table>

4. The retardation effect on heat generation is similar for all retarders tested (Table 3), although the amount of retarder required to give the same retardation may vary.
5. Use of retarder in the amount of 0.50 percent by weight of cement results in significant reduction in heat generation through 4 days' age in all cases, even though smaller percentages of retarder frequently result in increased heat generation (Table 3).

6. The differences in the rate of heat generation in the two test methods as shown in Figure 4 are due to the differences in the cement temperature during the test as shown on Figure 5.

Discussion of Tests

The retarder identified by number as Retarder No. 5 on Figures 1 through 5 is "Pozzolith 8." The cement used in all tests was a laboratory blend Type II (M-3100). Similar results were obtained on the previous laboratory blend Type II cement (M-2400).

The adiabatic temperature rise tests were conducted on concrete containing Glen Canyon aggregate, and both the mixes with and without retarder contained three sacks of cement per cubic yard. The specific heat of this concrete is expressed by the equation:

\[
\text{Specific heat} = 0.197 + 0.000277t
\]

This gives a specific heat of 0.216 Btu/lb/°F at 70° F.

Diffusivity of this concrete is very high, being 0.063 ft²/hr at 70° F.

Initial temperatures of the concretes in the temperature rise tests were about 49° F in both cases.
Table 1

<table>
<thead>
<tr>
<th>Days cured:</th>
<th>:</th>
<th>:</th>
<th>:</th>
<th>:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>3</td>
<td>7</td>
<td>14</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Without retarder</th>
<th>:</th>
<th>:</th>
<th>:</th>
<th>:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>19</td>
<td>33</td>
<td>47</td>
<td>52</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>With 0.37% Pozzolith 8</th>
<th>:</th>
<th>:</th>
<th>:</th>
<th>:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>16</td>
<td>33</td>
<td>48</td>
<td>53</td>
</tr>
</tbody>
</table>

Table 2

<table>
<thead>
<tr>
<th>Days cured:</th>
<th>:</th>
<th>:</th>
<th>:</th>
<th>:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>3</td>
<td>7</td>
<td>14</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Without retarder</th>
<th>:</th>
<th>:</th>
<th>:</th>
<th>:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>32</td>
<td>56</td>
<td>80</td>
<td>90</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>With 0.37% Pozzolith 8</th>
<th>:</th>
<th>:</th>
<th>:</th>
<th>:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>27</td>
<td>56</td>
<td>83</td>
<td>93</td>
</tr>
<tr>
<td>Table 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td><strong>HEAT GENERATION OF CEMENT CONDUCTION CALORIMETER</strong></td>
<td>Heat generation</td>
<td>calories per gram</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Age in days</strong></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>No retarder</td>
<td>42</td>
<td>51</td>
<td>56</td>
<td>60</td>
</tr>
<tr>
<td>0.25% retarder</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pozzolith 8</td>
<td>37</td>
<td>49</td>
<td>55</td>
<td>60</td>
</tr>
<tr>
<td>Pozzolith 3-R</td>
<td>40</td>
<td>51</td>
<td>58</td>
<td>64</td>
</tr>
<tr>
<td>WRDA</td>
<td>40</td>
<td>51</td>
<td>59</td>
<td>65</td>
</tr>
<tr>
<td>PDA</td>
<td>41</td>
<td>52</td>
<td>59</td>
<td>66</td>
</tr>
<tr>
<td>Plastiment</td>
<td>35</td>
<td>50</td>
<td>56</td>
<td>61</td>
</tr>
<tr>
<td>0.50% retarder</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pozzolith 8</td>
<td>21</td>
<td>38</td>
<td>45</td>
<td>50</td>
</tr>
<tr>
<td>Pozzolith 3-R</td>
<td>25</td>
<td>43</td>
<td>49</td>
<td>54</td>
</tr>
<tr>
<td>WRDA</td>
<td>25</td>
<td>43</td>
<td>51</td>
<td>57</td>
</tr>
<tr>
<td>PDA</td>
<td>25</td>
<td>43</td>
<td>49</td>
<td>54</td>
</tr>
<tr>
<td>Plastiment</td>
<td>15</td>
<td>41</td>
<td>47</td>
<td>52</td>
</tr>
</tbody>
</table>
FIGURE 2 - EFFECT OF RETARDERS ON THE ADIABATIC TEMPERATURE RISE OF CONCRETE AT EARLY AGES

CEMENT (TYPE II) LB/CU. YD.
RETARDER NO. 5 - % BY WT. OF CEMENT
INITIAL TEMPERATURE OF CONCRETE

1. 282 282
   NONE 0.37
   49.4°F 48.6°F

TEMPERATURE RISE - °F

AGE IN DAYS

0 1 2 3 4 5 6 7 8
CONCRETE MIXES FOR
GLEN CANYON DAM

April 3, 1959

Results of concrete mix investigations are presented in the attached Concrete Laboratory Report No. C-526A, published July 28, 1958. Additional compressive strengths for mixes obtained since that time are tabulated on Attachment A which is inserted in the Laboratory Report.
CONCRETE MIX INVESTIGATIONS TO DETERMINE EFFECTS OF WATER-REDUCING, CEMENT-DISPERSING, RETARDING AGENTS

April 8, 1959

An investigation consisting of seven mass concrete mix series, Table A, has been recently conducted for this project, the primary interest being to evaluate the effect of a lignin-type, water-reducing, cement-dispersing retarder when incorporated in Glen Canyon concrete.

Mass concrete contained varying amounts of cement and pozzolan, ranging from 2-1/2 to 3-1/2 sacks of total cementing materials, with pozzolan replacements varying from 0 to 43 percent. All cement was laboratory blend, Type II. Pozzolans used were a pumice and pumice; these materials were expected to be similar to the pozzolan to be used in the job, and came from deposits in the vicinity of the pozzolan deposit currently being exploited by the contractor for this material.

Early in this investigation, differing quantities of admixture were added to the concrete in an effort to determine an "optimum" amount (i.e., that amount giving maximum water reduction in the concrete without excessive retardation). The optimum established was 0.35 pound of lignin per sack of cementing materials.

Conclusions are necessarily tentative; however, certain key information is highlighted by repetition of test results throughout the entire concrete investigation:

1. Rate of hardening tests showed that concrete containing the "optimum" amount of admixture took from 8 to 33 percent longer to reach "final set" than did similar concrete (control) made without the admixture.

2. Addition of the lignin admixture resulted in from 8 to 12 percent reduction in water requirement, when compared with control mixes made without admixture.

3. Concrete containing the "optimum" amount of admixture produced higher compressive strengths than control at each test age from 2 days to 180 days (some test ages, including specimens for test at 1 and 2 years of age, have yet to be determined).

Available compressive strengths of 18- by 36-inch, sealed-cured cylinders containing the full mass mix provide an interesting comparison. Test results indicate that strength increases from 14 to 46 percent (compared to control cylinders) have been obtained from concrete specimens which contained the admixture and which were
tested at 28, 90, and 180 days' age. The 90-day strengths of four specimens (made from different mixes in the investigation) all exceed design criterion (3,450 psi at 180 days' age). Furthermore, a 28-day strength of 4,420 psi was obtained on one specimen made from concrete containing 2 sacks of cement and 1 sack of pozzolan per cubic yard, plus 0.35 pound of lignin per sack of cementing materials.
<table>
<thead>
<tr>
<th>MIX NO.</th>
<th>TYPE</th>
<th>AVERAGE WATER REQUIREMENT (Lb/YD(^3))</th>
<th>AVERAGE SLUMP (IN.)</th>
<th>AVERAGE AIR CONTENTS, PRESSURE METER (L%)</th>
<th>RETARDATION TEST TIME TO REACH 4000 PSI, HR-MIN.</th>
<th>AT 4000 PSI</th>
<th>2 DAYS</th>
<th>7 DAYS</th>
<th>28 DAYS</th>
<th>90 DAYS</th>
<th>180 DAYS</th>
<th>28 DAYS</th>
<th>90 DAYS</th>
<th>180 DAYS</th>
</tr>
</thead>
<tbody>
<tr>
<td>39</td>
<td>(\frac{1}{2}) Sacks cement, Control</td>
<td>157</td>
<td>1.7</td>
<td>4.9</td>
<td>7 hr., 35 min.</td>
<td>45</td>
<td>730</td>
<td>1840</td>
<td>3010</td>
<td>3625</td>
<td>3970</td>
<td>2610</td>
<td></td>
<td></td>
</tr>
<tr>
<td>93</td>
<td>(\frac{1}{2}) Sacks cement</td>
<td>140</td>
<td>1.7</td>
<td>4.9</td>
<td>6 hr., 35 min.</td>
<td>53</td>
<td>950</td>
<td>2270</td>
<td>3410</td>
<td>4310</td>
<td>4440</td>
<td>3020</td>
<td></td>
<td></td>
</tr>
<tr>
<td>94</td>
<td>(\frac{1}{2}) pound of lignin per sack</td>
<td>132</td>
<td>1.5</td>
<td>4.9</td>
<td>12 hr., 0 min.</td>
<td>31</td>
<td>970</td>
<td>2450</td>
<td>3880</td>
<td>4780</td>
<td>5020</td>
<td>3070</td>
<td></td>
<td></td>
</tr>
<tr>
<td>95</td>
<td>2 Sacks cement plus (\frac{1}{2}) sack M-2942 (pumice) Control</td>
<td>154</td>
<td>1.6</td>
<td>4.6</td>
<td>6 hr., 25 min.</td>
<td>55</td>
<td>570</td>
<td>1410</td>
<td>3010</td>
<td>4080</td>
<td>4410</td>
<td>2360</td>
<td></td>
<td></td>
</tr>
<tr>
<td>96</td>
<td>2 Sacks cement plus (\frac{1}{2}) sack M-2942 (pumice) (\frac{1}{4}) pound of lignin per sack</td>
<td>145</td>
<td>2.0</td>
<td>5.1</td>
<td>10 hr., 20 min.</td>
<td>47</td>
<td>585</td>
<td>1560</td>
<td>3260</td>
<td>4150</td>
<td>4910</td>
<td>2950</td>
<td></td>
<td></td>
</tr>
<tr>
<td>97</td>
<td>2 Sacks cement plus (\frac{1}{2}) sack M-2942 (pumice) (\frac{1}{4}) pound of lignin per sack</td>
<td>144</td>
<td>2.4</td>
<td>4.6</td>
<td>13 hr., 40 min.</td>
<td>41</td>
<td>570</td>
<td>1730</td>
<td>3640</td>
<td>4680</td>
<td>5310</td>
<td>3140</td>
<td></td>
<td></td>
</tr>
<tr>
<td>98</td>
<td>2 Sacks cement plus (\frac{1}{2}) sack M-2942 (pumice) Control</td>
<td>159</td>
<td>2.1</td>
<td>4.7</td>
<td>9 hr., 45 min.</td>
<td>56</td>
<td></td>
<td></td>
<td>2680</td>
<td>3980</td>
<td>4240</td>
<td>2850</td>
<td></td>
<td></td>
</tr>
<tr>
<td>99</td>
<td>2 Sacks cement plus (\frac{1}{2}) sack M-2942 (pumice) (\frac{1}{4}) pound of lignin per sack</td>
<td>145</td>
<td>2.2</td>
<td>4.7</td>
<td>12 hr., 0 min.</td>
<td>50</td>
<td></td>
<td></td>
<td>2840</td>
<td>4090</td>
<td>4410</td>
<td>3240</td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>2 Sacks cement, Control</td>
<td>154</td>
<td>1.8</td>
<td>5.1</td>
<td>9 hr., 45 min.</td>
<td>62</td>
<td>710</td>
<td>1430</td>
<td>2690</td>
<td>3600</td>
<td>3850</td>
<td>2750</td>
<td></td>
<td></td>
</tr>
<tr>
<td>101</td>
<td>2 Sacks cement, Control</td>
<td>142</td>
<td>1.7</td>
<td>4.8</td>
<td>10 hr., 45 min.</td>
<td>57</td>
<td>930</td>
<td>2130</td>
<td>3550</td>
<td>4370</td>
<td>4740</td>
<td>3260</td>
<td></td>
<td></td>
</tr>
<tr>
<td>102</td>
<td>2 Sacks cement plus (\frac{1}{2}) sack M-3337 (pumice) Control</td>
<td>159</td>
<td>2.1</td>
<td>4.7</td>
<td>14 hr., 10 min.</td>
<td>55</td>
<td>570</td>
<td>1520</td>
<td>3560</td>
<td>4890</td>
<td>3030</td>
<td>3970</td>
<td></td>
<td></td>
</tr>
<tr>
<td>103</td>
<td>2 Sacks cement plus (\frac{1}{2}) sack M-3337 (pumice) (\frac{1}{4}) pound of lignin per sack</td>
<td>140</td>
<td>2.0</td>
<td>4.7</td>
<td>15 hr., 15 min.</td>
<td>53</td>
<td>930</td>
<td>2090</td>
<td>4710</td>
<td>6370</td>
<td>4410</td>
<td>5440</td>
<td></td>
<td></td>
</tr>
<tr>
<td>104</td>
<td>2 Sacks cement plus (\frac{1}{2}) sacks M-3337 (pumice) Control</td>
<td>169</td>
<td>2.0</td>
<td>4.4</td>
<td>9 hr., 0 min.</td>
<td>55</td>
<td>630</td>
<td>1510</td>
<td>3780</td>
<td>5440</td>
<td>5540</td>
<td>2950</td>
<td></td>
<td></td>
</tr>
<tr>
<td>105</td>
<td>2 Sacks cement plus (\frac{1}{2}) sacks M-3337 (pumice) (\frac{1}{4}) pound of lignin per sack</td>
<td>157</td>
<td>1.6</td>
<td>4.0</td>
<td>12 hr., 0 min.</td>
<td>56</td>
<td>745</td>
<td>2090</td>
<td>4760</td>
<td>6680</td>
<td>3620</td>
<td>5320</td>
<td></td>
<td></td>
</tr>
<tr>
<td>106</td>
<td>3 Sacks cement (Temperature Rise Mix) Control</td>
<td>145</td>
<td>1.5</td>
<td>4.3</td>
<td>12 hr., 0 min.</td>
<td>56</td>
<td>745</td>
<td>2090</td>
<td>4760</td>
<td>6680</td>
<td>3620</td>
<td>5320</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1/ Air contents determined on concrete handpicked to \(\frac{1}{2}\)-inch maximum aggregate.  
2/ Elapsed time for wet-screened mortar to reach 4000 psi as determined by standard Proctor needle.  
3/ Compressive strength of cylinders containing \(\frac{1}{2}\)-inch wet-screened concrete when mortar reached 4000 psi.  
* Flagstaff Pumice  ** Siegel Deposit  
Aggregate: Glen Canyon, M-2692  
Cement: Laboratory blend Type II, M-3100  
Air-Entraining Agent: Vinsol resin
Memorandum

Mr. L. G. Puls, Chief Designing Engineer

R. E. Glover

Review of "Dams for Hydroelectric Power in Italy"

Purposes

The memorandum of January 8, 1959, to the Assistant Commissioner and Chief Engineer from the Commissioner, on the above subject, notes that the seven-volume publication entitled "Dams for Hydroelectric Power in Italy" has been acquired and states that:

"We believe that it would be desirable for your office to assign some of your design engineers to review the drawings of the features incorporated in these dams. We would like to have you list the outstanding structures and to analyze their principal differences in design practice as revealed by the drawings. Also list the pros and cons for the use of such features and whether they would have potential application in Reclamation."

These notes are being prepared at your request to assist in making this review.

Material Available

Of the seven volumes of the complete set on "Dams for Hydroelectric Power in Italy" only the Volumes 2 to 7 are now available since Volume 1 has not yet been delivered. The first volume is to contain the technical data concerned with design and construction of dams while the other volumes contain the descriptions of over 180 dams. For the technical data relating to Italian practice, the writer has therefore relied on the papers presented at the Symposium on Arch Dams which was held at Knoxville, Tennessee, in June 1956 (Reference 2). (Of these papers four were contributed by Claudio Marcello who is also the Chairman of the A.N.I.D.E.L. (Association of Italian Electric Utility Companies) committee for the study of problems concerning dams who have produced the seven volumes under review.) Italian practices were well-represented in the Symposium.
Increasing Use of Arch Dams in Italy

It is of interest to note the growth of popularity of the arch dam in Italy as revealed in the tabular summary, in the seven volumes, of the dams built before 1950, and those built or under construction after that date. Of the 237 dams built before 1950, there were 132 solid or cellular gravity dams and 34 arch or arch gravity dams. Of the 27 dams in construction since that date, 12 are solid or cellular gravity dams, while 11 are arch or arch gravity dams. One reason for this may have been expressed by Semenza in the paper on "Arch Dams: Development in Italy" (Reference 2, Paper 1017, Page 2) that "From a technical point of view, it is now considered, in short, that the overall safety of an arch structure is far higher than that of a gravity structure, as has indeed been proved in model tests to failure."

Italian Trends

The reasons for the arch dam development trends in Italy are also well-summarized by Dr. Semenza and his statements also give a clue to the reasons why American practice does not follow the same course. He says (Reference 2, Paper 1017, Page 2) "The rather remarkable development of arch dam construction in Italy is due to several factors which are partly interconnected. (a) The geological characteristics of the country. In the Alps, which constitute the principal hydraulic power area, and still more, in part, the Appenines, there is a prevalence of relatively recent formations, where fairly narrow gorges are quite common, for which the arch dam is the most natural solution.

(b) The cost of skilled and specialized labor in the construction industry is still relatively low and hence its factor in total cost is small. Materials, on the contrary, particularly cement and steel, are an important factor. The volume of concrete therefore constitutes the major element of cost.

(c) It is recognized in Italy as in any other country, that given the same degree of safety, the choice of the type of dams should depend only on the economic factor. Italian engineers for this reason have been driven to prefer increasingly refined structures in order to reduce the volume to a minimum.

(d) The peculiar characteristics of Italian mentality, which is fairly individualistic, and which therefore tends to examine every problem on its own merits, and free from any preconceived set of ideas. As a result of this attitude of mind, the principle, valid for any country, that each dam constitutes
a problem in itself to be solved according to criteria free from any preconceived idea, has found in Italy an ideal atmosphere for its full development, sometimes indeed beyond common limits. Hence, the widespread and elastic application of the most varied structural forms.

(c) This general tendency, which I will call mental, has been reinforced to some extent, both by tradition and artistic environment, since the arch, from the time of ancient Rome, has been a common architectural element. Thus, it was both logical and natural that modern designers of hydraulic structures should use it.

(i) The realization of the exceptional resistance of the arch has grown through centuries of experience in the Italian building workers whose craft has ancient traditions and deep intuition. Even for modest structures in house building, small and slender brick arches have been used for centuries, as, for example, in Romagna and Tuscany.

The principles which he here expresses so well, when applied to American conditions, with generally wider sites and a different economic balance, would naturally lead to thicker arch dams and with sections chosen to minimize the cost of form work. The double curvature dam, for example, seems much better adapted to Italian conditions than to American conditions.

Methods of Arch Dam Design

A perusal of the Symposium papers will indicate that while the Norwegians (Reference 2, Groner, April 1957) use Trial Loads, and the Portuguese (Reference 2, Rocha et al., Paper 997) use a combination of Trial Loads and models, the Italians rely principally on models. To facilitate testing of models of dams, the Italians have built a new and well-equipped laboratory at Bergamo. (Reference 2, Oberti, Paper 1351.) The Instituto Sperimentale Modelli e Strutture "I.S.M.E.S." (Model and Structural Testing Institute) operates this laboratory to solve specific structural problems which cannot be handled readily elsewhere. A study of the records of arch dams (Reference 2, Glover, Paper 1217) will indicate that in the rest of the world Trial Loads, Separate Arches, the thin cylinder formula, as well as the methods of Tolke and Smith, have also been used for design purposes. A method of plunging arches is used by the French (Reference 2, Coyne, Paper 959) who also make use of models. These methods of design are described in detail in a subsequent paragraph.
All of these methods have been used with conspicuous success since there is no actual record of the failure of an arch dam*. (Reference 2, Coyne, Paper 959) (Reference 2, Glover, Paper 1217.) With this experience one may well ask what is to be gained by a comparison of design methods. The answer is that the closer we can predict the performance of our structures, the better we will be able to use the resources at our disposal to achieve the results we desire in the most economical ways. A rather extreme example is given by Coyne (Reference 2, Coyne, Paper 959, Page 7) who estimates that the thin LeCage arch dam required a volume of concrete only 18/100 of what would have been needed for a gravity dam at the same site.

Analytical Requirements

An understanding of the requirements which must be fulfilled to arrive at a correct knowledge of stress levels and distributions in a dam is important because of its bearing on this review and seems best approached through the theorem ofKirchhoff**(Reference 3, Paragraph 118, Page 170). This theorem is concerned with the possibility that there might be more than one stress system which would sustain a load placed upon an elastic structure. He proves that there can be but one. Such a theorem is known as a uniqueness theorem. It is important to have this proved*** but the theorem has an additional importance in our case because it tells us, by implication, what requirements must be met if we are to obtain from our computations that one unique stress distribution which must prevail under the imposed conditions. These requirements are very easy to understand. If we imagine the volume of the dam divided into small elementary, approximately cubical elements, by passing through it and series of planes or surfaces, these requirements are:

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*Two alleged failures can be found in the literature. These were the Lake Lanier and Moyie River dams. What actually happened was that both lost an abutment down to the base but both dams remained intact. The Lake Lanier Dam was repaired. The Moyie River Dam was not repaired (Reference 2, Glover, Paper 1217).

**Gustav Robert Kirchhoff (1824-1887). The theorem was published in 1859.

***Some similar engineering cases are not unique and this leads to difficulties whose source is often obscure. Soil pressures against retaining walls provide an example.
1. That each element must be in equilibrium under the forces and stresses which act upon it

2. That as the dam passes from the unstrained to the strained state, each of these elements must deform in such a way as to continue to fit with its neighbors on all sides, and

3. The stresses or displacements must conform to those imposed at the boundaries

These three conditions are usually referred to as the equilibrium, continuity, and boundary conditions, respectively, and we will use this terminology in what follows.

Models and Computations

We are now in a position to consider the question as to whether a model will give us a different stress distribution than we will get by computation. The answer to this question is that if the conditions represented by the model and accounted for in the computations are the same and the Kirchhoff requirements are met by the computations there can be no difference. This was demonstrated by Bureau of Reclamation model tests and computations made as a part of the design studies for Hoover Dam. The results of model tests* made by the group organized by I. E. Houk were repeatedly compared with trial load analyses and it was found that as the computations were brought into progressively closer agreement with Kirchhoff's requirements, they approached also a complete agreement with the results of the model tests. Many comparisons of this sort are contained in the reports of the Arch Dam Investigations (Reference 7, Part II, Model of Gibson Dam, Figure 76), and The Boulder Canyon Project Final Reports. (Reference 6, Pages 396 to 399, inclusive.) It makes no difference how we go about the analytical work, if we finally succeed in satisfying the Kirchhoff requirements we will get the one unique solution which must prevail. Additional evidence on this score is furnished by the recent work of A.J.S. Pippard and Associates (Reference 8). They applied Relaxation procedures (Reference 11) to the computation of stresses in an arch dam. This method is quite distinct from Trial Load procedures but it will be noted (Reference 8, Page 227) that they also obtained a very satisfactory agreement with the results of model tests.

*These tests had the benefit of the technical skill of Dr. Fredrik Vogt.
It can be concluded that there will be no difference between the results of model tests and computations when both are correctly done.

**Difficulties and Advantages of Structural Model Testing**

The philosophy of model testing is very easily grasped but in practice some very real difficulties are present. The materials of a dam and its abutments may, and generally do, have different elastic properties. To properly model such cases it is necessary to develop a material whose properties can be controlled. The Bureau of Reclamation in their model work developed the plaster-celite material (Reference 6, Paragraph 3, Page 24). For such a material, workability, freedom from excessive shrinkage, availability, good bonding characteristics, a proper Poisson's ratio, and a proper relation between compressive and tensile strength, are required. If, in addition to tests under normal load conditions, where elastic behavior should be expected, tests to destruction are to be made to find the ultimate strength of the dam, then the strength characteristics of the dam concrete and the abutment rock must also be properly represented.

The task of finding such a material could fairly merit the description "impossible." And, so it is, but by diligent search, research, and testing, model materials can be developed which approach these ideals close enough to be useful.

Aside from the problem of applying the proper loadings, including the important vertical loads, a model has a certain inflexibility. It is known from the results of strain meter tests in actual dams (Reference 9), for example, that the method of construction has an important influence on the ultimate stress distribution in the dam. It is difficult to include this factor in model tests. The factors of temperature changes and earthquake effects also present difficulties to the model tester.

On the other hand, for the investigation of the ultimate strength of an arch dam, models have no competitors and information of the greatest importance has been obtained from them. It is found, for example, that an arch dam will ordinarily go well beyond the factor of safety estimated on the basis of elastic action. Sometimes the ratio of the observed to the computed factor of safety is as high as 2 to 1. They have also shown that failures can occur by the dam sliding up the abutments.

Model testing calls for adequate facilities, highly developed technical skills, special materials, and time.

In spite of the very real difficulties which model testing presents, it is possible, where the facilities and skills are available, to do a very satisfactory job of structural design with them. It is the impression of this writer that the Italian model work is excellent.
It should be mentioned here that the advantages of hydraulic models for such things as spillway design seem to be universally accepted.

Difficulties and Advantages of Computation Methods

Computation methods are limited to elastic conditions but inasmuch as the dam will be designed to work well within this range, this is not a serious drawback. The real difficulty with analytical methods is the almost heroic computation task they present to the designer. This writer has noted with some dismay that complete trial load analyses are seldom attempted outside of the Bureau organization*, although studies based on radial adjustments only are fairly common. If the computation difficulties could be overcome, this method would become a very effective design tool since many more studies could be made than are now possible because of the expense and time involved.

At the present time the development of digital computers affords a new possibility for reducing the time and cost of trial load studies. The Bureau of Reclamation Trial Loads group have already succeeded in programming one of these computers to do the arch computations. I saw the cards for the Flaming Gorge arch computations sent down to be run through the machine. These computations were completed in about 4-1/2 hours. In the earlier days, these arch computations represented half of the work of a trial load study and required about 2 months of time. Present indications, based upon an extrapolation of the results already obtained, indicate that if the trial load procedure can be completely programmed for machine computation, a complete study can be made in a week and at a cost of about 10 percent of what it would cost by the old methods. There is now good reason to believe that this can be done.

Some Analytical Procedures Used for the Design of Arch Dams

The design procedures used for arch dams may be described in the following way. An understanding of them is important in this review because of their bearing on questions of differences in design practice.

Thin cylinder formula: The dam is imagined to be divided into a series of horizontal arch rings by passing through

*The Portuguese engineers have recently made studies with twist and tangential adjustments included. Some Italian work has also been done with complete trial load analyses, based upon the use of digital computers. (Reference 18.)
it a series of horizontal cutting planes. Each of these arch rings is assumed to behave as a portion of a complete cylinder which sustains the reservoir pressure by the development of a uniform compression through the arch ring. It is realized that bending will be present and, to allow for this, the ring stress used in the computations is reduced to a half or less of the stress which is considered allowable.

**Individual Arches**

The entire volume of the dam is imagined to be divided into a series of horizontal arch rings, as described above. Each of these rings is assumed to be free to slide with respect to the arch rings above and below it. Stresses are computed by the elastic arch theory. The factors of reservoir load, temperature changes, abutment restraints, and bending can now be introduced.

**Crown Adjustment**

An elementary trial load procedure which employs arch computations of the type described for the individual arches method but which also makes an accounting of the ability of the dam to resist bending stresses which act vertically by introducing the effect of bending in vertical beam elements. Such a beam element is called a cantilever element. They are thought of as being formed by passing through the dam a system of coaxial vertical planes. The vertical line common to all of these vertical planes is located at or near the center of curvature of the arch elements. The totality of such cantilever elements make up the entire volume of the dam in the same manner as the totality of the arch elements make up the whole volume of the dam. When a crown adjustment is made, one of these cantilever elements near the crown or midpoint of the arches is selected as representative and the water load is divided between the arch and cantilever systems by trial to make their downstream deflections approximately equal at all elevations. The representative cantilever element so selected generally has its base at the lowest part of the canyon profile and it is therefore the tallest cantilever element of any in the dam. As the abutments are approached, the arch deflections will be less and the cantilever elements will generally be shorter so that their deflections will be less also. While these factors tend to preserve the deflection agreement reached at the crown, a check generally shows that the agreement away from the crown is imperfect.

**Complete Radial Adjustment**

In this version of the Trial Load Method the water load is divided among the arch and cantilever elements so that the downstream deflections of the two systems agree everywhere. It is to
be understood, of course, that since this is a cut and try process the agreement is never perfect.

**Complete Trial Load Study**

The volume of the dam is divided into arch and cantilever elements as before but it is now recognized that there are six components of displacement which must be brought to agreement. These comprise the linear displacements in the downstream, tangential, and vertical directions and the rotations about the arch radius, the arch tangent, and the vertical. It turns out that only three adjustments are needed. Forces are now introduced between the arch and cantilever elements to eliminate the deflection disagreements which may be thought of as having originated from an initial application of the entire water load to the arch elements only. These forces can be identified as the resistances which various elements of strength develop to assist the dam to carry the imposed loads. These elements of strength include the resistance to shear deformations caused by the tendency of the arch elements to deflect toward the abutment by different amounts at different elevations, the resistance to twist, and, the resistance to bending of the vertical elements as described under the complete radial adjustment heading.

This identification provides an explanation of the observation that arch dams designed by a wide variety of methods have invariably performed well. The explanation is that the simplified design methods ignore certain elements of strength. The dams designed in this way are thicker than necessary but this does not prevent them from standing up well.

**Relaxation Procedures**

A method of analysis based upon the concept of systematic relaxation of constraints. This method was developed by the English elastician R. V. Southwell. (Reference 11.) It was recently applied to arch dam stress analysis by Pippard and associates. (Reference 8.)

**The Method of Smith**

A method of stress analysis for arch dams developed by the Australian B. A. Smith. (Reference 12.) He considers the dam as a tapered cylindrical shell and develops a solution in terms of Michell functions. When the boundary conditions are imposed the deflection of the structure is determined. The Michell functions are related to Bessel functions.

**Method of Tolke**

A method of stress analysis of arch dams developed by Tolke for triangular, trapezoidal and hyperbolic profiles. The
solution for the triangular profile is obtained in terms of Bessel and Hankel functions of Order 1. When the boundary conditions are imposed the deflections are determined as in Smith's method. (Reference 13.)

**Plunging Arches**

The thrust trajectories for an arch dam, subjected to water pressures, are nearly horizontal at the crown but acquire a downward inclination as the abutments are approached. In the plunging arch method, arch rings following these thrust trajectories are substituted for the horizontal arches of the Individual Arches Method.

**Tests of Arch Dam Design Methods Against the Kirchhoff Requirements**

<table>
<thead>
<tr>
<th>Method</th>
<th>Satisfies equilibrium requirement</th>
<th>Satisfies continuity requirement</th>
<th>Satisfies boundary conditions</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thin cylinder</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>Effect of temperature changes cannot be included</td>
</tr>
<tr>
<td>Individual Arches</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Continuity approached for downstream deflections</td>
</tr>
<tr>
<td>Crown Adjustment</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Continuity restored for downstream deflections</td>
</tr>
<tr>
<td>Complete Radial Adjustment</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>All six displacements brought to agreement Kirchhoff's requirements substantially met</td>
</tr>
<tr>
<td>Complete Trial Load Study</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>If properly done substantially meets the Kirchhoff requirements</td>
</tr>
<tr>
<td>Relaxation</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>Plunging Arches</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td></td>
</tr>
<tr>
<td>Smith</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td>Tolke</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td></td>
</tr>
</tbody>
</table>
Definition of Terms

The following definitions may be useful:

Gravity, Arch Gravity, and Arch Dams

As used by European engineers these terms seem to bear a closer relation to the mental state of the designer than to the physical realities.

A gravity dam is one designed to carry the reservoir loads by its resistance to overturning and sliding. An arch-gravity dam is designed to carry the loads by a combination of arch and cantilever action, and an arch dam is designed to carry its loads by arch action alone.

The physical realities may be considerably out of accord with these concepts. A scrutiny of Volumes 2 to 7 of the series under review will show that the Italians often curve their gravity dams. An analysis would probably show that arch action is an important element of strength in such structures. Nearly all arch dams develop some gravity action although it may not contribute much to the strength of the very thin dams.

The "Pulvino" or Cushion

This is a footing interposed between an arch dam and its abutments. It may serve several purposes, among which may be listed the following:

a. It distributes the thrusts imposed by the arches to the abutments.

b. It may be applied to bridge foundation weaknesses or to accommodate a transition from hard to softer rocks when the rock quality is not uniform over the full height of the abutment.

c. It permits the dam to be designed for a preselected profile which is often made symmetrical. This profile is constructed in the Pulvino.

The Perimetral Joint

The use of embedded pipe cooling to control temperatures in arch dams before closure is not common in Europe. As a consequence, their dams go through a period of "settlement" after construction. When the Pulvino is used, the joint between the dam and the Pulvino is deliberately used to provide freedom for rotation. A water stop is provided to prevent leakage but it is intended that the joint
will function to prevent the development of tensions at the upstream face by opening to accommodate the movements of the dam. This joint between the dam and the Pulvino or cushion is the perimetral joint.

Comparison of American and European Dams

As examples of American dams we may choose Hoover, Hungry Horse, Glen Canyon and Gibson, all by the Bureau of Reclamation, and the Ross Dam of the City of Seattle, and for purposes of comparison we may select the Vajont, Lumei, Santa-Giustina, Osiglietta, Pieve di Cadore, all of Italian design. The Gibson Dam of the Bureau of Reclamation is an early design, having been completed in 1929. It is included here for comparison with the Pieve di Cadore design which also occupies a very wide site.

Some comparative dimensions are shown in the following two tables:

Table 1

<table>
<thead>
<tr>
<th>Dam</th>
<th>Height (feet)</th>
<th>Crest Length (feet)</th>
<th>Maximum Thickness (feet)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hoover</td>
<td>726</td>
<td>1,244</td>
<td>660</td>
<td>Completed 1934</td>
</tr>
<tr>
<td>Glen Canyon</td>
<td>700</td>
<td>1,550</td>
<td>340</td>
<td>Under construction</td>
</tr>
<tr>
<td>Hungry Horse</td>
<td>564</td>
<td>2,060</td>
<td>321</td>
<td>Completed 1952</td>
</tr>
<tr>
<td>Ross</td>
<td>540</td>
<td>1,300</td>
<td>208</td>
<td>Present stage</td>
</tr>
<tr>
<td>Gibson</td>
<td>195.5</td>
<td>960</td>
<td>87</td>
<td>Completed 1929</td>
</tr>
</tbody>
</table>
Table 2

<table>
<thead>
<tr>
<th>Dem</th>
<th>Height meters</th>
<th>Crest length meters</th>
<th>Maximum thickness meters</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vajont</td>
<td>265.5m</td>
<td>190.5m</td>
<td>22.71m</td>
<td>Under construction</td>
</tr>
<tr>
<td></td>
<td>871</td>
<td>625</td>
<td>74</td>
<td>Reference 14</td>
</tr>
<tr>
<td>Maini di Sauris</td>
<td>136.13m</td>
<td>138.36m</td>
<td>15.87m</td>
<td>Completed 1947</td>
</tr>
<tr>
<td>(Lumei)</td>
<td>447</td>
<td>454</td>
<td>52</td>
<td>Reference 1 (4)</td>
</tr>
<tr>
<td>Santa Giustina</td>
<td>152.5m</td>
<td>124.2m</td>
<td>16.50m</td>
<td>Completed 1950</td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>408</td>
<td>54</td>
<td>Reference 1 (2)</td>
</tr>
<tr>
<td>Osiglietta</td>
<td>76.80m</td>
<td>224.0m</td>
<td>10.74m</td>
<td>Completed 1939</td>
</tr>
<tr>
<td></td>
<td>252</td>
<td>735</td>
<td>35</td>
<td>Reference 1 (6)</td>
</tr>
<tr>
<td>Pieve di Cadore</td>
<td>112 m</td>
<td>410 m</td>
<td>35.81m</td>
<td>Completed 1949</td>
</tr>
<tr>
<td></td>
<td>367</td>
<td>1,345</td>
<td>117</td>
<td>Reference 1 (4)</td>
</tr>
</tbody>
</table>

Note: Conversion to English units made by this writer.

A comparison of the Hoover and Vajont designs shows that while they are roughly of the same height (Vajont is the higher) the maximum thickness of the Vajont Dam is only about one-ninth that of Hoover. At Hoover, however, the top width approaches twice the height while at Vajont the top width is less than the height. Admittedly, Hoover could have been made thinner but it went beyond all precedents for height for its time, and was designed to very conservative standards. If designed to the same standards as Vajont it would probably be much the thicker dam because the site is less favorable. The Maini di Sauris and Santa Giustina dams may be compared to the Ross Dam. Again the ratio of top width to height is much in favor of the Italian designs. It is the belief of this writer that Italian type designs in the Ross site would not reduce materially the volume of concrete required below that of the present design.

The Osiglietta Dam illustrates the characteristics of the dome-type of design. There is no comparable American dam in the list.

The Gibson and Pieve di Cadore dams are both in wide sites. Except for a plug of concrete filling a deep gorge at
the Pieve di Cadore site, this dam rests on a horizontal base. The height from the Pulvino to the crest is 55 meters or 180.4 feet. On this basis they are not far from the same height. The 26 meters thickness at the base of the arch is equivalent to 85.1 feet. This is almost identical with the Gibson base thickness of 87 feet. The ratio of top width to height in the two cases is: Gibson 4.9 to 1 and Pieve di Cadore 7.4 to 1 based on height above the Pulvino. These two dams compare well.

As compared with Vajont the Glen Canyon site is very wide. Even though a somewhat soft abutment rock has required thickening the dam to spread the loads over a greater abutment area than would be required with a hard rock, this design is much thinner than Hoover which is of comparable height. Even with a hard abutment rock this dam should not be made as thin as Vajont.

Comparison of Canyons

A comparison of canyons for two high dams may be obtained from Figures 5 and 6. The very narrow canyon available at the site of the Vajont Dam may be noted. It is also V-shaped which is an aid to keeping the vertical components of the stresses within bounds.

Differences of U. S. and Italian Practices

The Italians often use the device called a "Pulvino" or cushion. This device is clearly shown in Figure 3. It is a footing interposed between the dam and the abutments. It can be used to distribute the arch thrusts to the abutment, to bridge foundation weaknesses, to smooth transitions between abutments of different elastic modulus and to provide a predetermined and properly shaped abutment for the dam.

The joint between the dam and the Pulvino is the famed "perimetral joint." A water stop is provided at this joint but no restraint against rotation at this joint is permitted. This is done to prevent the development of tensions at the upstream face at the abutments as the dam deflects under load. This device seems to have been used very successfully. There are good reasons to believe that the stresses developed at the joint will not be excessive (References 16 and 17).

Where the site is suitable they also use the dome-type dam (See Figure 7). Their reasoning is that, with curvatures in both the horizontal and vertical directions, this dam will act somewhat like a sail to distribute the loads in both directions

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and thereby make a better use of the concrete. Their claim probably has merit since a spherical shell will sustain an external pressure with only one-half the stress developed in a cylindrical shell of the same radius and thickness sustaining the same pressure. This dome shape leads to overhang and doubly curved surfaces which have been avoided in U. S. practice because of formwork complications.

The Portuguese engineers make another effective point favoring this type of dam. They note that when arch dam models are tested to destruction they often fail by sliding up the abutments. In the dome-type of dam the water forces are applied in such a way as will prevent this type of failure.

The use of the cushion, the perimetal joint, and the dome-type of dam are regarded as being technically sound. The choice between a single curvature and a double curvature dam could well rest solely on economic considerations.

A scrutiny of Volumes 2 to 7 of the "Dams for Hydroelectric Power in Italy" will show that even where the Italians design a gravity dam they often give it a curvature in plan. This is regarded as a practice which should be given the most serious consideration. It is difficult to design a gravity dam with a factor of safety in excess of two and even this can only be realized under normal conditions. An arch dam will generally have a factor of safety of from 5 to 10. Adding a curvature to such a dam can convert it from a dam with a poor factor of safety to a dam of tremendous strength. The argument that gravity dams should be made straight to facilitate the design of overflow spillway crests is considered to be without foundation. The curved plan should permit a longer crest and a better approach to the channel downstream.

The Italians often make use of the crest of a thin arch dam as an overflow spillway. Except for the reservation that this should never be done without a thorough study of its behavior by hydraulic model tests, there seems to be no reason why this type of spillway should not be used. There are certainly many examples of successful use of overflow spillways in Europe and America. The use of such spillways is facilitated in the Italian dams by their location of their powerhouses. These are generally away from the dam. A study of the volumes under review will show many cases where their arrangement comprises a dam, a tunnel leading away from the dam, a surge tank to control the flow in the tunnel, and penstocks conveying the water under pressure to a powerplant located on the river. This arrangement makes effective use of the fall of the river between the dam and the powerplant, and it also facilitates the use of an overflow spillway.
Allowable Stresses

The stresses considered allowable in Italy are believed to be comparable to those used in the U. S. Semenza (Reference 2, Paper 1017) mentions stresses of the order of 60 to 70 kg/sq cm. This is equivalent to about 853 to 995 pounds per square inch. Bureau of Reclamation standards permit up to 1,000 pounds per square inch. There is always a comparison with the ultimate strength of the concrete to be used. He mentions the factors one-fifth of the 28-day strength and one-seventh of the 90-day strength. (Reference 2, Page 1017.)

The LeGage Dam has stresses up to about 1,700 lb/in² (Reference 2, Paper 959, Page 4) but this is believed to be a special case, which should not be considered as representing normal practices.

Summary

There is no longer any mystery concerning what must be done to arrive at a proper evaluation of the stresses in an arch dam subjected to a specified system of loads. Satisfactory evaluations can be obtained by skillful model testing or by calculation. To be satisfactory, in this sense, the computations must fulfill the Kirchhoff requirements. Comparison of model test results and comparable computed results where Kirchhoff's conditions are met invariably show agreement.

Under present conditions an acceptable evaluation by either structural model testing or Trial Load procedures requires a great deal of time and money. The alternatives may be presented in the following way:

a. Model tests: Require special skills, proper equipment, time, and money

b. Complete Trial Load studies: Require special skills, time, and money

c. Simplified analytical procedures: They buy relief from the cost of more elaborate studies by the use of excessive amounts of concrete in the dam

In large dams this cost may well be too high. In small dams the extra volume may not be objectionable since if they were designed rigidly to a stress allowance they could well be too thin to resist weathering.

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It is the opinion of this writer that the differences between American and Italian Arch Dam designs is explainable principally by differences in topography and the economic balance in the two countries and that the differences in design methods have very little to do with it.

Recommendations

The cost in time and money for skillfully made model tests or acceptable stress analyses is at present high. If the cost and time required to make these studies could be reduced it would open the way to a more effective evaluation of possibilities than is now available. With improved methods of study, economy could be obtained by better and more effective use of materials and labor. To reach these desired objectives it is recommended to:

1. Explore diligently the possibilities of programming the high speed digital computers to perform the computations required for an acceptable stress analysis

2. To explore with contractors the probable costs of building arch dams with cushions, doubly curved surfaces, and overhang

3. To avoid the use of gravity dams where arch dams may be used both for reasons of economy and of strength

4. To use models to explore the ultimate strength of comparable arch and gravity designs.
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of the American Concrete Institute, Vol XXX, 1934,  
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18. Algebraic Load Method of Analyzing Arch Dams, by  
Dino Tonini--Sixth Congress of Large Dams, New York, 1958
Figure 5. Glen Canyon

Figure 6. Gorge at Site of Vajont Dam
Figure 7. Osiglietta
Figure 8. Mina di Sauris (Lumei)
Figure 9. Pieve di Cadore
Figure 10. Santa Giustina Dam
Memorandum

L. C. Puls

R. E. Glover

Evaluation of the effect of stepped abutments—Glen Canyon Dam—Colorado River Storage Project

Abutment excavation

Due to varying qualities of the abutment rock disclosed by the abutment excavation the shape of the abutment line has been modified somewhat from the shape originally intended. The modified lines are shown as contours on the two attached photostats. In places where an original half-radial abutment line was intended there is now a stepped abutment line. The effect of these changes on the stress distribution can best be evaluated by photoelastic means and the following program has been prepared after conferring with Messrs. Copen, Scrivener, Moody, and Schultz in regard to the abutment shapes, the stress distribution in the dam, as disclosed by the recent trial load studies and the availability of photoelastic materials with suitable elastic modulus values to represent the abutment rock and the concrete.

On the basis that the worst stress conditions may be found in the abutment where the downstream face of the dam meets the abutment near the surface of the canyon wall and the worst stress concentrations on the arch may be expected where the downstream face of the dam meets the abutment at some distance back from the canyon wall, two comparative studies are recommended, one for shallow embedment and the other for deep embedment, as described. In each case it is recommended to compare a half-radial abutment line as originally intended, with a stepped abutment line as provided by the present abutment surface so that the stress changes caused by these modifications may be evaluated. These studies will, in addition show how the arch thrusts spread out into the abutments. Mr. Moody's present workload would probably permit him to begin these studies by about Monday, March 9, 1959. The shallow embedment case with the stepped footing should be started first. Some results for this case should be available about 2 weeks after this work begins. This would be about March 23, if the work were begun on March 9.

It is estimated that to complete the studies would require 60-man-days and 6 weeks of time. The total cost would be $3,000. The recommended program is shown in the following table:
Table 1. Photoelastic tests to evaluate the effect of stepped abutment lines.

<table>
<thead>
<tr>
<th>Abutment</th>
<th>(E_d/E_a)*</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shallow embedment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Radial</td>
<td>5</td>
<td>Read abutment stresses</td>
</tr>
<tr>
<td>Radial</td>
<td>2 to 3</td>
<td>Read abutment and arch stresses</td>
</tr>
<tr>
<td>Stepped</td>
<td>5</td>
<td>Read abutment stresses</td>
</tr>
<tr>
<td>Stepped</td>
<td>2 to 3</td>
<td>Read abutment and arch stresses</td>
</tr>
<tr>
<td>Deep embedment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Make identical studies for a deep embedment.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*E_d and E_a—Young’s modulus values for dam and abutment.

Vertical adjustment

When the Trial Load Method was being developed it was realized that the vertical displacements should be accounted for in the tangential adjustment. Comparison of computed results with results obtained from model testing showed, however, that good correlations could be obtained by accounting for the tangential displacements only and this practice has been followed since that time. There is a new factor present in the case of the Glen Canyon Dam, however, that warrants a reconsideration of this practice. This factor is the unusual amount of subcooling to be used. The horizontal expansion of the arch rings, which will be obtained as the concrete temperatures return to normal, will compensate effectively for the unusually large abutment displacements permitted by the soft abutment rock at this site. This horizontal expansion will be accompanied, however, by a vertical expansion also. This tendency of the dam to increase in height will be restrained by the abutments. The forces developed in this way will affect the stress distribution in the dam by some amount. Some idea of the amount of such modification can be obtained from a calculation of the stresses which would be necessary to completely restrain these expansions. These complete restraint stresses could reach about 270 pounds per square inch. Actually the stress changes should be much less than this because of abutment yielding but it seems, nevertheless, important that this factor be evaluated so that its effect on the stress distribution in the dam may be known.

The time of two men for 4 weeks is estimated to be needed to make these computations. It is recommended that this study be made as soon as the present workload will permit.
To: Assistant Commissioner and Chief Engineer

From: Commissioner

Subject: Review of "Dams for Hydroelectric Power in Italy"

Recently, the Department of the Interior Library acquired the 7-volume publication entitled "Dams for Hydroelectric Power in Italy," published by Aniedel. It is our understanding that your office has also acquired this set of volumes which depicts the major dams and design practices in Italy. A cursory review of this publication reveals that many large and unusual structures have been constructed in Italy, some of which incorporate practices which differ from those used in this country.

We believe it would be desirable for your office to assign some of your design engineers to review the drawings of the features incorporated in these dams. We would like to have you list the outstanding structures and to analyze their principal differences in design practices as revealed by the drawings. Also list the pros and cons for the use of such features and whether they would have potential application in Reclamation. After such an analysis is made, it might be desirable to prepare a suitable technical discussion of these design differences for presentation at an American Society of Civil Engineers convention, perhaps the one that will be held in Washington in October of 1959.

It may be possible that if the design practices contain features of interest to Reclamation, you may be able to justify sending a team of experts to examine these structures. This perhaps could be done in connection with some travel to an International Congress, for example the International Irrigation and Drainage Congress which will be held in Spain in 1960. If our interest were significant, it might be possible to send our engineers to visit the engineering and design offices in Italy and perhaps other countries as a part of a training program which could be authorized under the Federal Training Act recently enacted.

We recognize that a review of the seven volumes will entail some time, but we believe it would be beneficial to distribute this task among several engineers to acquaint them with the practices revealed by this publication.

[Signature]

J. A. Decker
REPORT TO

U. S. DEPARTMENT OF INTERIOR
BUREAU OF RECLAMATION
Denver, Colorado

COLORADO RIVER STORAGE PROJECT
Foundation Adequacy and Design Considerations
of
Glen Canyon Dam

by
Board of Consultants

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

Julian Hinds, Chairman

Kanab, Utah    May 28, 1958
Mr. Grant Bloodgood  
Assistant Commissioner and Chief Engineer  
Bureau of Reclamation  
Denver Federal Center  
Denver, Colorado

Dear Mr. Bloodgood:

In accordance with instructions contained in a letter from Mr. E. G. Nielsen, Associate Chief Engineer, dated May 9, 1958, the undersigned members of the Consulting Board for Glen Canyon Dam assembled in Kanab, Utah, at 9:00 a.m. May 26, 1958.

Following a briefing by Construction Engineer L. F. Wylie, Chief Designing Engineer L. O. Puls, and Chief Geologist W. H. Irwin, the Board was flown to the dam site where inspections were made of the rock conditions in the excavations for the spillway approach channels, the abutment keys and the left bank diversion tunnel.

In the morning of May 27 the Board met in Kanab with Messrs. Wylie, Puls, and Irwin and Mr. Neil Murdock, Regional Geologist and representative of the Regional Director, and discussed the information contained in the book of technical data prepared for the Board dated May 23, 1958, by the Denver office, from studies and investigations relating to foundation materials and design analysis conducted within the past year.

In the afternoon of May 27, the Board returned to the dam site for additional examinations of the abutment rocks and for an inspection of the pilot aggregate processing plant on Wahweap Creek. During the day, at the request of the Board, the Kanab office prepared certain graphs showing the relationships between depth and the strength and modulus of elasticity and compressive strength of the rocks of the abutments.

May 28 was devoted to further review and discussion of the data with Messrs. Puls and Irwin and in preparing the report that follows.

The four items upon which the Board's views were requested in Mr. Nielsen's letter of May 9th are as follows:

"1. Examination of the present status of the excavated foundation of the dam, the excavation in the spillway channels and diversion tunnels, to confirm suitability for design loadings."
2. Reappraisal of the modulus of elasticity for use in analysis of the dam, and of the abutment rock, based on tests and observations of character of rock revealed by the excavations.

3. Review of results of the complete trial load analysis of the dam.

4. Review of layout of contraction joints and thermal problems resulting in the largest blocks of concrete.

In addition, comment is made concerning the heavy media process for elimination of low specific gravity aggregates.

These items are discussed in order, as follows:

I. FOUNDATION STRESSES

On the basis of the bedrock conditions disclosed by the excavations made thus far, and the results of the investigation made by the Bureau during the past year, it is the Board's opinion that the design foundation stresses suggested in its report of May 7, 1957, entitled "Report on Foundation Adequacy and Design Considerations of Glen Canyon Dam," namely, 500 p.s.i. for the arches and 750 p.s.i. for the cantilevers, for conditions of normal water level plus earthquake, are satisfactory.

The Board was favorably impressed with the character and condition of the rock in the excavated faces.

II. MODULUS OF ELASTICITY OF FOUNDATION ROCK

In its report of May 7, 1957, the Board recommended a value of 500,000 p.s.i. for the modulus of elasticity of the foundation rock. Additional data accumulated during the past year indicate that this value should be retained below elevation 3400, but that above that level the value can and should be increased to 600,000 p.s.i.

III. TRIAL LOAD ANALYSIS

The results of the trial load analysis studies completed to date are reassuring in that they definitely indicate that the desired results are attainable.

The methods of analysis, the controlling criteria, and the results obtained have been reviewed to the extent possible for them to be disclosed in the summary contained in the Technical Data Book.

The criteria are the same as those approved by the Board on May 7, 1957. The procedures appear rational and the results are satisfactory, except for a limited over-stress area at the top of the abutments. There is no doubt that this difficulty can be remedied.
To this end, the increase in the modulus of elasticity of the rock above elevation 3400, as recommended above, may be helpful.

Also a careful review of recent data reveals that 0.08 is a more realistic value for Poisson’s Ratio for the foundation rock than the value of 0.06 previously approved. Change to the higher value, which is recommended, would be helpful.

There are other design procedures which can be utilized to redistribute these stresses and reduce the high values.

IV. THERMAL CONTROL AND CONTRACTION JOINTS

On page 18 of the Technical Data Book supplied to the Board it is stated that temperature control studies which have been completed indicate the temperature rise for the contemplated concrete mix in 7½ foot lifts need not exceed 25°F. The details of this study have not been made available but we presume this applies to a starting temperature of about 50°F. which would lead to a maximum temperature of about 75°F. After the concrete has acquired satisfactory strength, the temperature would be reduced to the low value of 40°F. below elevation 3450 and 40 to 50°F. above that elevation, when grouting would be performed. The Board merely wishes to reaffirm its general approval of the plans for post cooling and grouting, all as covered in Item 3 of its report dated May 7, 1957.

The results of the complete trial load analysis given in the Technical Data Book show higher than desirable computed arch stresses at the top of the abutments, with lower arch stresses below. One way in which the high stresses could be reduced would be to cool this upper portion to a temperature higher than 50°F. specified before grouting.

One of the questions that has been raised is that of the possibility of the formation of longitudinal vertical cracks in long blocks which, with the single longitudinal joint shown in the drawings, will have a maximum length of about 210 feet. In the opinion of the Board such length of block should be no cause for concern if proper construction controls are exercised. One natural factor which is very favorable to reduction in cracking tendency is the extremely low modulus of elasticity of the foundation rock.

Further to reduce the cracking tendency the Board recommends that shallow lifts, say 2½ feet thick, be employed for an average depth of say 10 feet next to the foundation rock and that depressions in the rock be filled so as to provide a surface at the top of these shallow lifts which over the area of each block will be substantially plane. By the cooling pipe installation, temperature of the concrete of these shallow lifts should be so controlled during the period of
primary cooling as to insure minimum stress differentials between rock and concrete, between the shallow lifts, and between the top of these lifts and the concrete of the regular 7½ foot lifts above, which should be placed only after the foundation bed of shallow lift concrete has hardened. The condition would be most favorable when the temperature of the foundation bed of concrete at the time of placement of concrete above, was not much below the maximum of 75°F. so that as the new concrete is cooled that below will be at nearly the same temperature as that above.

Finally before the first stage of grouting, the cooling to 40°F. of the long blocks should take place slowly so that the temperature gradient may be as flat as practicable.

The Board has no change to suggest in the contraction joint layout as shown in the plans.

AGGREGATE PROCESSING

When the members of the Board visited the aggregate plant, experimentation with heavy media separation was in progress with material in the size range No. 4 to No. 8. According to those conducting the work, sink material was being produced which met the specification requirements that not more than 2 per cent should be of specific gravity less than 2.5. From the results of earlier trials it seems evident that the coarse aggregate will need to be processed in two sizes, fine and coarse, and that properly to remove low specific gravity material with a single feed covering the whole range of sizes from the minimum of No. 8 is impracticable.

A visual examination of the float material (as compared with the sink) shows it to be largely composed of chert and sandstone particles.

The Board was favorably impressed with this method for the removal of objectionable low specific gravity material.

The Board wishes to acknowledge with thanks the valuable assistance of staff members of the Denver, Salt Lake City and Kanab offices of the Bureau.
Respectfully submitted

Members of the Board

Edward B. Burwell, Jr.

Raymond E. Davis

John J. Hammond

John W. Vanderwilt

Julian Hinds, Chairman

APPROVED: JUN 2 1958

[Signature]

Assistant Commissioner and Chief Engineer
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

TECHNICAL DATA
FOR THE USE OF THE
BOARD OF CONSULTANTS
GLENCANYONDAM
COLORADORIVERSTORAGEPROJECT

PREPARED BY THE OFFICE OF THE
ASSISTANT COMMISSIONER AND CHIEF ENGINEER

Denver, Colorado
May 23, 1958
FOREWORD

The data presented herein has been prepared by the Divisions of Design and Engineering Laboratories from studies and investigations conducted mostly within the past year in the Denver office. The summary of construction progress on the dam was prepared by the Division of Construction.
CONTENTS

Status of Construction as of May 1, 1958--Glen Canyon Dam and Powerplant .............................. 1

Glen Canyon Dam Foundation Tests--Summary of Strength Grouping and Zoning of Foundation Rock Properties ..................... 3

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Design of Glen Canyon Dam and Supporting Data ...................................................... 13

Contraction Joint Layout .................................................................................................. 17

Thermal Control of Concrete .............................................................................................. 18
STATUS OF CONSTRUCTION AS OF MAY 1, 1958
GLEN CANYON DAM AND POWERPLANT

Right Abutment Excavation for Dam

Excavated to elevation 3600±.
Excavation of 5-foot by 7-foot foundation tunnel at elevation
3630 advanced a total distance of 54 feet.

Left Abutment Excavation for Dam

Excavated to elevation 3730±.

Left Diversion Tunnel

Length excavated, May 1 2,200 feet
Distance to excavate after May 1 763 feet
Total length 2,963 feet
Excavation of keyways for tunnel plug completed.
Excavation of open cut at the downstream portal in progress.

Right Diversion Tunnel

Excavation completed by another contractor.
Construction of invert form and the screed rail and the
placement of reinforcement steel for the tunnel lining
in progress.
Excavation in open cut at the downstream portal in progress.

Left Spillway

Excavation in open cut in intake channel completed except for
haul ramps and fine grading.
Excavation for spillway tunnel started from the top.
A 8-foot by 9-foot pilot hole for the spillway tunnel raise
excavated from the diversion tunnel for distance of 15 feet.

Right Spillway

Excavation in open cut in intake channel completed except for
haul ramps and fine grading.
Excavation for spillway tunnel started from the top.
A 7-foot by 14-foot pilot hole for the spillway tunnel raise
excavated from the diversion tunnel for distance of 56 feet.
Powerplant Service Road Tunnel

Length excavated, May 1 6,900 feet
Distance to excavate after May 1 2,604 feet
Total length 9,504 feet
Adits excavated concurrently with tunnel.

Service Roads to Dam

Excavation in progress.

Control Cable Tunnel from Powerplant to Switchyard

Excavation completed.

Contractor's Plant and Equipment

Excavation for the main cableway runways preparatory to placing concrete for the thrust rails in progress.
Temporary concrete batching and mixing plant erected on the right abutment to supply concrete for the diversion tunnels and miscellaneous concrete structures.
Erection of pilot plant for light and heavy media aggregate separation completed and plant being tested. Adjustments were being made in an attempt to get proper separation and gradations.
Construction of aggregate haul road from the dam site to the Wahweap aggregate deposit completed except for final surfacing.
Excavation along the canyon wall upstream of the right abutment keyway for the permanent concrete mixing plant in progress.
Cableway for service to diversion tunnels in operation.
Suspension foot bridge completed.
Two inclined cableways or "monkey-slides" in operation; both on left canyon wall.
INTRODUCTION

During 1957, the laboratory conducted tests on rock core from Glen Canyon to determine the physical properties of the foundation. Rock core tested in the laboratory was from 10 angle holes on a line extending across the canyon, 2 vertical holes in the abutments, and 6 vertical holes in the canyon floor.

The results of tests on rock from individual drill holes have been reported in previous memoranda.

To gain a more comprehensive knowledge of the physical properties of the foundation, a statistical type analysis has been made of the test results which are considered to be representative of the rock in the foundation at the dam site.

In making the analysis, the 1,300 feet of rock core received in the laboratory was first visually inspected and divided into about 40 different classes whose difference in physical characteristics was noticeable to the unaided eye. The extent of similar classes was noted by depth limits within each drill hole, and sufficient test specimens were selected to give a representative sampling. Physical properties of the 40 visual classes were correlated using frequency distribution charts and the core ultimately divided into four main groups according to strength.

The average compressive strength and relative amount of each strength group are:

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

The distribution of the strength groups in the abutments and foundation varied with depth in such a way that the dam site was then divided into three different zones. The location of these zones and the relative distribution of the strength groups are shown in Figure 1. The boundaries of these zones are approximately between the following elevations:
(1) Upper zone  Right abutment—3,400 feet to 3,700 feet  
Left abutment—3,300 feet to 3,700 feet  
(2) Middle zone  Right abutment—3,200 feet to 3,400 feet  
Left abutment—3,200 feet to 3,300 feet  
(3) Lower zone  Foundation—2,950 feet to 3,200 feet  

CONCLUSIONS  

1. The variation in properties of the rock is such that the average strength of the foundation increases with depth. Tabulated below for each zone is the average unconfined compressive strength weighted according to the percentage of each strength group in the zone.  

<table>
<thead>
<tr>
<th>Zone</th>
<th>Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper zone</td>
<td>2,870 psi</td>
</tr>
<tr>
<td>Middle zone</td>
<td>3,640 psi</td>
</tr>
<tr>
<td>Lower zone</td>
<td>3,830 psi</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Stress range</th>
<th>0-200 psi</th>
<th>0-400 psi</th>
<th>0-600 psi</th>
<th>0-800 psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load cycle</td>
<td>First</td>
<td>Second</td>
<td>First</td>
<td>Second</td>
</tr>
<tr>
<td>Upper zone</td>
<td>0.49</td>
<td>0.54</td>
<td>0.58</td>
<td>0.64</td>
</tr>
<tr>
<td>Middle zone</td>
<td>0.43</td>
<td>0.49</td>
<td>0.50</td>
<td>0.56</td>
</tr>
<tr>
<td>Lower zone</td>
<td>0.36</td>
<td>0.41</td>
<td>0.40</td>
<td>0.46</td>
</tr>
</tbody>
</table>

3. The effect of the angle of bedding planes (measured from the core axis) on strength or modulus is slight but is noticeable in the lateral strain or Poisson's ratio. Specimens with bedding angles of 0° to 60° had lateral strains about 40 percent less than specimens with angles of 60° to 90°.  

DISCUSSION OF RESULTS  

Distribution of Strength Groups by Zones  

A diagrammatic cross section of the dam site with zone boundaries and percentage distribution of the strength groups within each zone is shown in Figure 1. The percentage distribution of the strength groups is shown by circle graphs. The low strength group (average 770 psi) and high strength group (average 6,250 psi) were found only in the upper zone where the low strength group comprised about 18 percent and the high strength group about 8 percent of the rock. The occurrence of the medium low strength group (average 1,770 psi) decreased with depth, comprising about 30 percent in the
upper zone and 2 percent in the lower zone. The medium strength group (average 3,870 psi) is the most prevalent, comprising about 45 percent of the upper zone and 98 percent of the lower zone.

The location of the drill holes from which cores were tested is also shown in Figure 1. The blacked-out portions of the holes show those areas sampled.

**Location of Strength Groups in Drill Holes**

The data in Table 1 present the occurrence of strength groups in terms of total footage and relative percent in each drill hole.

Location of strength groups in each drill hole is shown in Table 2. The portions listed are those that were considered as samples in making the analysis. The footage not recovered from Drill Holes 56-H, I, and J was assumed to be in the low strength group for analysis purposes.

**Physical Properties of Glen Canyon Sandstone**

Presented in Table 3 for each of the strength groups are:

a. Average and range of unconfined compressive strength and tensile strength

b. Principal stress relationship and related Mohr's envelope equations

c. Secant modulus of elasticity at 200, 400, 600, and 800 psi for the first and second load cycles

d. Poisson's ratio at 200, 400, 600, and 800 psi for the first load cycle

e. Percent set for first load cycle

f. Average specific gravity, percent absorption by weight, and percent porosity by volume

The unconfined compressive strength, tensile strength, and properties indicated by the principal stress relationship and Mohr's envelope decrease in magnitude from the high to low strength group. As a matter of interest, the tensile strength in the medium and medium-low strength groups was 3 percent of the compressive strength; whereas, it was 6 percent in the low strength group. This difference is believed to be due to the variation in grain size and degree of cementation.

Triaxial and tension tests for the high strength group were not performed because of the limited quantity of core pieces long enough for testing.
The principal stress relationship (Figure 2) shows that the strength of the medium strength group is doubled at a confining pressure of 500 psi and the strength of the low strength group is tripled at a confining pressure of 250 psi.

Mohr's envelope equation, Figure 3, was determined by two methods:

1. Analysis of triaxial test data (SP-23)
   In this method, a tangent to Mohr's circles is computed from the principal stress relationship.

2. Analysis of unconfined tensile and compressive strength (SP-39)
   This method simply computes the tangent to the tension and compression circles.

The true cohesive strength and tan $\phi$ are actually between the values determined by these two methods.

The secant modulus of elasticity is tabulated in Table 3 at stresses of 200, 400, 600, and 800 psi for the first and second load cycles for each of the strength groups in the three zones. It may be noted from the variations in modulus with stress that the stress-strain characteristic of the sandstone is curvilinear.

First-load cycle stress-strain curves and stress-modulus curves for each of the groups in the different zones are shown in Figures 4 and 5, respectively. Referring to Table 3, it is noted that in any particular zone the highest modulus occurs in the highest strength group; however, within a group, the modulus is highest in the upper zone and decreases with depth.

The modulus is only slightly affected by the angle of bedding. It may also be noted that in the continued portion of Table 3, Poisson's ratio increases with increasing stress, increases slightly with depth, and increases appreciably with decreasing strength. The effect of increasing bedding angle on Poisson's ratio occurs rather abruptly at about 60° with about 40 percent increase in the ratio; the results, therefore, were divided into two ranges of bedding angle, 0° to 60° and 60° to 90°, for each strength group in each zone.

The percent set, which is the ratio of unrecovered strain to applied strain after the first load cycle, is also presented in Table 3. Within any zone, the percent set increased with decreasing strength, and within a strength group, it increased with depth. About 25 percent more set occurred in specimens with a bedding angle of 0° to 60° than in those with a bedding angle of 60° to 90°.

The specific gravity of the rock increased with strength, and the absorption and porosity decreased with strength. These values did not change significantly with depth.
<table>
<thead>
<tr>
<th>Drill Hole</th>
<th>High strength 6,250 psi</th>
<th>Med strength 3,870 psi</th>
<th>Med low strength 1,770 psi</th>
<th>Low Strength 770 psi</th>
<th>Total L.F. examined</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>L.F.</td>
<td>%</td>
<td>L.F.</td>
<td>%</td>
<td>L.F.</td>
</tr>
<tr>
<td>56-A</td>
<td>38.5</td>
<td>48.1</td>
<td>22.5</td>
<td>28.1</td>
<td>19.0</td>
</tr>
<tr>
<td>56-B</td>
<td>59.5</td>
<td>70.0</td>
<td>21.0</td>
<td>24.7</td>
<td>4.5</td>
</tr>
<tr>
<td>56-C</td>
<td>104.0</td>
<td>94.5</td>
<td>6.0</td>
<td>5.5</td>
<td></td>
</tr>
<tr>
<td>56-D</td>
<td>66.4</td>
<td>73.8</td>
<td>23.6</td>
<td>26.2</td>
<td></td>
</tr>
<tr>
<td>56-E</td>
<td>97.0</td>
<td>97.0</td>
<td>3.0</td>
<td>3.0</td>
<td></td>
</tr>
<tr>
<td>56-F</td>
<td>80.0</td>
<td>100.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>56-G</td>
<td>95.3</td>
<td>94.4</td>
<td>5.7</td>
<td>5.6</td>
<td></td>
</tr>
<tr>
<td>56-H</td>
<td>43.0</td>
<td>39.1</td>
<td>40.4</td>
<td>36.7</td>
<td>26.6</td>
</tr>
<tr>
<td>56-I</td>
<td>16.5</td>
<td>17.4</td>
<td>45.5</td>
<td>47.9</td>
<td>33.0</td>
</tr>
<tr>
<td>56-J</td>
<td>45.2</td>
<td>53.2</td>
<td>31.5</td>
<td>37.1</td>
<td>8.3</td>
</tr>
<tr>
<td>56-K_n</td>
<td>56.4</td>
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<td>Drill Hole</td>
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<td>Low strength 770 psi</td>
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<td>248.0-258.5</td>
<td>246.0-248.0</td>
<td>258.5-270.0</td>
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<td>252.0-256.5</td>
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<td>238.5-250.0</td>
<td>250.0-238.5</td>
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<td>257.0-263.5</td>
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<td>263.5-270.0</td>
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<tr>
<td>56-C</td>
<td>90.0-170.4</td>
<td>170.4-175.4</td>
<td>185.0-200.0</td>
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<td>175.4-184.0</td>
<td>184.0-185.0</td>
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<td>56-D</td>
<td>60.0-72.3</td>
<td>72.3-89.0</td>
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<td>89.0-135.6</td>
<td>135.6-142.5</td>
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<td>147.0-150.0</td>
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<td>56-F</td>
<td>80.0-160.0</td>
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<td>56-G</td>
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<td>105.0-110.7</td>
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<td>110.7-166.0</td>
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<td>56-H</td>
<td>85.0-93.5</td>
<td>75.0-85.0</td>
<td>93.5-98.5</td>
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<td></td>
<td>98.5-105.0</td>
<td>105.0-106.5</td>
<td>119.4-120.0</td>
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<td>106.5-109.5</td>
<td>109.5-119.4</td>
<td>124.0-125.0</td>
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<tr>
<td></td>
<td>120.0-124.0</td>
<td>144.0-155.0</td>
<td>155.0-175.0</td>
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<tr>
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<td>125.0-144.0</td>
<td>177.0-185.0</td>
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</tr>
<tr>
<td></td>
<td>175.0-177.0</td>
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<td>56-I</td>
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<td>75.0-82.5</td>
<td>105.0-138.0</td>
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<tr>
<td></td>
<td>96.0-105.0</td>
<td>90.0-96.0</td>
<td>138.0-170.0</td>
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<td>56-J</td>
<td>80.0-106.0</td>
<td>106.0-116.5</td>
<td>75.0-80.0</td>
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<tr>
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<td>129.0-132.2</td>
<td>117.5-129.0</td>
<td>116.5-117.5</td>
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<tr>
<td></td>
<td>134.5-135.0</td>
<td>135.0-144.5</td>
<td>132.2-134.5</td>
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</tr>
<tr>
<td></td>
<td>144.5-160.0</td>
<td></td>
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</tr>
<tr>
<td>56-K_n</td>
<td>10.0-50.0</td>
<td>50.0-52.6</td>
<td>68.0-70.0</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>52.6-67.0</td>
<td>67.0-68.0</td>
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</table>

**Table 2**

GLEN CANYON FOUNDATION ROCK INVESTIGATION
Location of Strength Groups
Depth in Drill Hole -- F<sub>n</sub>
<table>
<thead>
<tr>
<th>Drill Hole</th>
<th>High strength 6,250 psi</th>
<th>Medium strength 3,870 psi</th>
<th>Med low str 1,770 psi</th>
<th>Low strength 770 psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>56-Ln</td>
<td>41.0-100.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>56-15</td>
<td>120.0-125.0</td>
<td>140.0-145.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>160.0-165.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>56-17</td>
<td>45.0-50.0</td>
<td>65.0-70.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>85.0-90.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>56-18</td>
<td>95.0-100.0</td>
<td>125.0-130.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>155.0-160.0</td>
<td>185.0-190.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>56-19</td>
<td>45.0-50.0</td>
<td>65.0-70.0</td>
<td>85.0-94.0</td>
<td></td>
</tr>
<tr>
<td>56-26</td>
<td>70-75</td>
<td>90-93</td>
<td>93-95</td>
<td>275-280</td>
</tr>
<tr>
<td></td>
<td>131-135</td>
<td>130-131</td>
<td>254-255</td>
<td>401-402</td>
</tr>
<tr>
<td></td>
<td>155-160</td>
<td>210-215</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>185-190</td>
<td>250-254</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>325-330</td>
<td>300-305</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>430-435</td>
<td>350-355</td>
<td>377-382</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>400-401</td>
<td>402-405</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>460-465</td>
<td>495-500</td>
<td></td>
</tr>
<tr>
<td>56-30</td>
<td>200-205</td>
<td>95-97</td>
<td>97-100</td>
<td>121-126</td>
</tr>
<tr>
<td></td>
<td>226-231</td>
<td>150-151</td>
<td>331-336</td>
<td>148-150</td>
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<td></td>
<td>275-280</td>
<td>152-153</td>
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<td>151-152</td>
</tr>
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<td>370-375</td>
<td>250-255</td>
<td>300-305</td>
<td>175-180</td>
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<tr>
<td></td>
<td></td>
<td>348-353</td>
<td>395-400</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>420-425</td>
<td>470-475</td>
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<tr>
<td></td>
<td></td>
<td>495-500</td>
<td>530-535</td>
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</table>
Table 3
GLEN CANYON FOUNDATION ROCK INVESTIGATION
Summary of Physical Properties
(75 percent saturated)

<table>
<thead>
<tr>
<th>Percent of total core</th>
<th>High strength</th>
<th>Medium strength</th>
<th>Medium low strength</th>
<th>Low strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength (psi)</td>
<td>4 6,250</td>
<td>71 3,870</td>
<td>17 1,770</td>
<td>8 770</td>
</tr>
<tr>
<td>Range</td>
<td>5,350 to 7,360</td>
<td>2,260 to 5,390</td>
<td>1,290 to 2,230</td>
<td>310 to 950</td>
</tr>
<tr>
<td>Tensile strength (psi)</td>
<td>No tests</td>
<td>120</td>
<td>80</td>
<td>50</td>
</tr>
<tr>
<td>Range</td>
<td>50 to 250</td>
<td>40 to 120</td>
<td>20 to 70</td>
<td></td>
</tr>
<tr>
<td>Principal stress relationship</td>
<td>No tests</td>
<td>(S_1 = 7.36S_3 + 3.880)</td>
<td>(S_1 = 8.99S_3 + 1.810)</td>
<td>(S_1 = 9.03S_3 + 750)</td>
</tr>
<tr>
<td>Mohr's envelope (triaxial)</td>
<td>No tests</td>
<td>(Y = 1.17X + 720)</td>
<td>(Y = 1.34X + 300)</td>
<td>(Y = 1.34X + 120)</td>
</tr>
<tr>
<td>Mohr's envelope (tension-compression)</td>
<td>No tests</td>
<td>(Y = 2.56X + 340)</td>
<td>(Y = 2.22X + 190)</td>
<td>(Y = 1.80X + 100)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Load Cycle</th>
<th>First</th>
<th>Second</th>
<th>First</th>
<th>Second</th>
<th>First</th>
<th>Second</th>
<th>First</th>
<th>Second</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper zone</td>
<td>0.66</td>
<td>0.73</td>
<td>0.53</td>
<td>0.59</td>
<td>0.46</td>
<td>0.51</td>
<td>0.35</td>
<td>0.37</td>
</tr>
<tr>
<td>Middle zone</td>
<td>0.44</td>
<td>0.50</td>
<td>0.32</td>
<td>0.38</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lower zone</td>
<td>0.36</td>
<td>0.41</td>
<td>0.24</td>
<td>0.29</td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tbody>
</table>

| Secant modulus of elasticity Million psi Stress Stress Stress |
|-----------------|-----------------|-----------------|-----------------|
| 800 psi 600 psi 400 psi 200 psi | 0.73 0.81 0.63 0.70 | 0.53 0.63 0.53 0.63 | 0.46 0.47 |
| Upper zone | Middle zone | Lower zone | Upper zone | Middle zone | Lower zone | Upper zone | Middle zone | Lower zone |
| 0.80 0.89 0.72 0.78 0.66 0.73 0.53 0.57 0.41 0.52 0.32 0.37 0.46 0.47 0.53 0.57 |
| 0.88 0.98 0.78 0.86 0.73 0.79 0.68 0.68 0.55 0.68 0.44 0.50 0.53 0.58 0.44 0.50 |
### Table 3 (Continued)

**GLEN CANYON FOUNDATION ROCK INVESTIGATION**

Summary of Physical Properties
(75 percent saturated)

<table>
<thead>
<tr>
<th>Bedding angle</th>
<th>High strength</th>
<th>Medium strength</th>
<th>Medium low strength</th>
<th>Low strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0° - 60°</td>
<td>60° - 90°</td>
<td>0° - 60°</td>
<td>60° - 90°</td>
</tr>
<tr>
<td>Upper zone</td>
<td>0.06</td>
<td>0.03</td>
<td>0.08</td>
<td>0.08</td>
</tr>
<tr>
<td>Middle zone</td>
<td>0.05</td>
<td>0.08</td>
<td>0.11</td>
<td>0.06</td>
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<tr>
<td>Lower zone</td>
<td>0.05</td>
<td>0.11</td>
<td>0.11</td>
<td>0.14</td>
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</table>

<table>
<thead>
<tr>
<th>Poisson's ratio</th>
<th>First load cycles</th>
<th>Stress 200 psi</th>
<th>Upper zone</th>
<th>0.07</th>
<th>0.06</th>
<th>0.11</th>
<th>0.08</th>
<th>0.14</th>
<th>0.20</th>
<th>0.37</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Stress 400 psi</td>
<td></td>
<td>Middle zone</td>
<td>.08</td>
<td>.14</td>
<td>.08</td>
<td>.14</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Stress 600 psi</td>
<td></td>
<td>Lower zone</td>
<td>.08</td>
<td>.14</td>
<td>.08</td>
<td>.14</td>
<td></td>
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<td></td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Poisson's ratio</th>
<th>Stress 600 psi</th>
<th>Upper zone</th>
<th>0.08</th>
<th>0.08</th>
<th>0.12</th>
<th>0.11</th>
<th>0.17</th>
<th>0.27</th>
<th>0.42</th>
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<td>Stress 800 psi</td>
<td>Middle zone</td>
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<td>.15</td>
<td>.12</td>
<td>.16</td>
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<tr>
<td></td>
<td></td>
<td>Lower zone</td>
<td>.09</td>
<td>.16</td>
<td>.09</td>
<td>.16</td>
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</table>

<table>
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<tr>
<th>Percent set</th>
<th>Upper zone</th>
<th>9</th>
<th>11</th>
<th>9</th>
<th>11</th>
<th>10</th>
<th>17</th>
<th>12</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Middle zone</td>
<td>12</td>
<td>9</td>
<td>18</td>
<td>14</td>
<td>10</td>
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<td></td>
<td>Lower zone</td>
<td>14</td>
<td>10</td>
<td></td>
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<table>
<thead>
<tr>
<th>Specific Av gravity Range</th>
<th>2.12</th>
<th>2.03</th>
<th>1.97</th>
<th>1.92</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.06 to 2.18</td>
<td>1.95 to 2.16</td>
<td>1.91 to 2.01</td>
<td>1.86 to 1.97</td>
<td></td>
</tr>
</tbody>
</table>

| Absorption | 9.5 | 11.8 | 13.0 | 14.6 |
| percent by weight | 8.7 to 11.0 | 9.5 to 13.9 | 12.0 to 15.0 | 13.3 to 16.1 |

| Porosity | 20.1 | 23.9 | 25.7 | 28.1 |
| percent by volume | 18.1 to 22.7 | 20.2 to 27.0 | 24.3 to 28.6 | 26.1 to 30.0 |
GLEN CANYON FOUNDATION ROCK INVESTIGATION
PERCENT DISTRIBUTION OF STRENGTH GROUPS BY ZONES

KEY
CIRCLES SHOW PERCENTAGE OF STRENGTH
GROUPS IN EACH ZONE
AVG. COMP. STR

HIGH STRENGTH
MEDIUM STRENGTH
MEDIUM LOW STRENGTH
LOW STRENGTH

6500 PSI
5870 PSI
1770 PSI
770 PSI

DH. 56-30
(30 DOWNSTREAM)
DH. 56A
DH. 56B
DH. 56C
DH. 56D
DH. 56E
DH. 56F
DH. 56G
DH. 56H
DH. 56I
DH. 56J
106 LIN. FT. OF CORE
225 LIN. FT. OF CORE
1295 LIN. FT. OF CORE
365 LIN. FT. OF CORE
364 LIN. FT. OF CORE
(60 DOWNSTREAM)
(160 DOWNSTREAM)
(105 DOWNSTREAM)
(225 UPSTREAM)
(125 UPSTREAM)
(260 UPSTREAM)
(60 DOWNSTREAM)

CREST OF DAM
MIDDLE LEFT ZONE
MIDDLE RIGHT ZONE
LOWER ZONE
UPPER LEFT ZONE
UPPER RIGHT ZONE

ELEVATION
3800'
3700'
3600'
3500'
3400'
3300'
3200'
3100'
3000'

DRILL HOLE STATION
0 1 2 3 4 5 6 7 8 9 10 11 12
GLEN CANYON FOUNDATION ROCK INVESTIGATION
PRINCIPAL STRESS RELATIONSHIP

AXIAL STRESS ($S_1$) p.s.i. vs LATERAL STRESS ($S_3$) p.s.i.

- MEDIUM STRENGTH GROUP
  $S_1 = 7.36S_3 + 1380$

- MEDIUM-LOW STRENGTH GROUP
  $S_1 = 8.99S_3 + 1810$

- LOW STRENGTH GROUP
  $S_1 = 9.03S_3 + 750$

( ) REPRESENTS NUMBER OF SPECIMENS
GLEN CANYON FOUNDATION ROCK INVESTIGATION STRESS-STRAIN CURVES (AXIAL) BY STRENGTH GROUPS FIRST LOADING

AXIAL STRESS-PSI
FIGURE 5

VARIATION IN MODULI OF ELASTICITY BY STRENGTH GROUPS

GLEN CANYON FOUNDATION ROCK INVESTIGATION

LOWER ZONE
MEDIUM ZONE
UPPER ZONE

LOW STRENGTH

MEDIUM STRENGTH

HIGH STRENGTH

MEDIUM-LOW STRENGTH

AXIAL STRESS-psi

MODULUS OF ELASTICITY-IN MILLION psi.
MODULUS OF ELASTICITY AND SUSTAINED ELASTIC MODULUS OF FOUNDATION CORES--GLEN CANYON DAM

April 22, 1958

Tabled below are the instantaneous modulus of elasticity and sustained elastic modulus values obtained on Navajo sandstone cores from the Glen Canyon Dam site foundation. Five minutes elapsed between the first and second loadings. Twelve days elapsed between the second and third loadings. The third loading was the sustained load in the creep frames. The sustained modulus values are calculated for total strain from the original condition to 1 year under sustained load.

Each loading condition is the average of three 6-inch cores taken from Holes Ka and La which are vertical holes in the foundation.

Table 1
CORES SATURATED

<table>
<thead>
<tr>
<th>Load, psi</th>
<th>Modulus of elasticity, (psi)</th>
<th>Sustained modulus at 1 year</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1st loading</td>
<td>2nd loading</td>
</tr>
<tr>
<td>200</td>
<td>592,000</td>
<td>665,000</td>
</tr>
<tr>
<td>400</td>
<td>610,000</td>
<td>700,000</td>
</tr>
<tr>
<td>625</td>
<td>649,000</td>
<td>738,000</td>
</tr>
</tbody>
</table>

Table 2
CORES OVEN DRY

<table>
<thead>
<tr>
<th>Load, psi</th>
<th>Modulus of elasticity, (psi)</th>
<th>Sustained modulus at 1 year</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1st loading</td>
<td>2nd loading</td>
</tr>
<tr>
<td>200</td>
<td>703,000</td>
<td>849,000</td>
</tr>
<tr>
<td>400</td>
<td>698,000</td>
<td>853,000</td>
</tr>
<tr>
<td>625</td>
<td>682,000</td>
<td>889,000</td>
</tr>
</tbody>
</table>
DESIGN OF GLEN CANYON DAM
AND SUPPORTING DATA

May 14, 1958

1. Introduction

Following the May, 1957 meeting of the Board of Consultants on Glen Canyon Dam, the layout of the dam, Design A-18, and the design criteria on which it was based were reviewed for conformity with the Board's conclusions and for possible improvements. The revised design, A-19, including the design criteria and the results of stress analyses are illustrated and discussed in the following pages.

It should be noted that the stresses in Design A-18 presented to the Board in the 1957 meeting were based on a trial-load adjustment of radial deflections between the crown cantilever and crowns of the arches only. The stress results presented with Design A-19 include, in addition to a radial adjustment at the arch and cantilever elements indicated on the drawings, the effects of tangential shear and twist action. This results in a much more accurate estimate of stresses than was previously shown.

2. Design A-19

A plan and maximum section of Glen Canyon Dam, Design A-19, are shown on Drawing No. 557-D-435. It should be noted that the maximum section is the same as that of Design A-18; also the abutment thicknesses are approximately the same as those of Design A-18. By changing and rearranging the fillets and tangents on the downstream face of the dam, the rate of divergence of this face near the abutments was reduced. In addition to improving the appearance of the dam, this change tends to reduce principal stresses and stresses parallel to the face of the dam.

3. Design Criteria

Studies of Design A-19 were based on the following loading conditions and assumptions:

a. Top of dam, elevation 3715.

b. Normal reservoir water surface, elevation 3700.

c. Top of fill on downstream face, elevation 3158.

d. Minimum tailwater surface, elevation 3142.

e. Temperatures used in analysis are changes between average arch temperatures at time of joint closure and minimum operating temperatures. Operating temperatures are assumed to vary linearly from upstream to downstream faces.
f. The effects of a construction and grouting program are included as follows:

(1) Concrete placed to elevation 3480; reservoir water surface, elevation 3240; joints ungrouted; no arch action.

(2) Concrete cooled to 40° F. and contraction joints grouted to elevation 3480; concrete placed to elevation 3715 and water surface raised to elevation 3490. In the analysis, arch action is assumed below elevation 3490 but cantilevers only carry the loads above this elevation.

(3) Contraction joints grouted from elevation 3480 to 3715 after concrete has been cooled to temperatures varying from 40° F. at elevation 3480 to 50° F. at elevation 3715; reservoir water surface raised to elevation 3700 and the effects of earthquake, earth embankment and tailwater included. In the analysis arch action is assumed throughout the dam.

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

g. Earthquake was assumed to move the dam upstream and downstream horizontally in the direction of the plane of centers with an acceleration of 0.1 gravity and a period of vibration of one second. The increased water pressure was assumed to act equally on all cantilevers. Effects of vertical acceleration were not included.

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

i. Modulus of elasticity of foundation rock, 500,000 pounds per square inch.

j. Poisson's ratio of concrete, 0.20.

k. Poisson's ratio of foundation rock, 0.06.

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

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

4. Arch, Cantilever and Principal Stresses

Arch and cantilever stresses parallel to the faces of the dam are shown on Drawing No. 557-D-669. The maximum compressive stress computed at the arch abutments is 760 psi at the intrados, elevation 3715. A
stress of 681 psi at the arch abutment extrados occurs at the same elevation. With these exceptions all arch stresses computed at the abutments were less than 500 psi. On Drawing No. 557-D-670 are plotted the stresses at the extrados and intrados of the arch abutments and their variation from a stress of 500 psi. Drawing No. 557-D-671 shows the arch elements analyzed and the stresses normal to their crowns and abutments. The dashed lines across the stress diagrams at the abutments indicates the average stress at these locations.

The maximum compressive stress found at the bases of cantilever elements is 521 psi parallel to the downstream face of the crown cantilever. All other stresses computed at the bases of cantilevers were less than 500 psi. An area of tensile stresses was found on the downstream faces of the cantilevers above elevation 3475 near the abutment. The maximum tensile cantilever stress calculated is 131 psi at the downstream face of cantilever A, elevation 3625.

Principal stresses at the upstream and downstream faces of the dam's abutments are shown on Drawing No. 557-D-672. The maximum compressive principal stress at the upstream face occurs at elevation 3715 and was found to be 681 psi. A compressive principal stress of 504 psi was computed at elevation 3030 at the upstream face. All other principal stresses at the upstream face are less than 500 psi.

The maximum compressive principal stress at the downstream face was found to be 760 psi at elevation 3715. Compressive principal stresses of 518, 543, 541 and 521 psi were computed at the downstream face at elevations 3325, 3100, 3030 and 3010, respectively. All other principal stresses at the downstream face are less than 500 psi. The maximum principal tensile stress of 217 psi was computed at elevation 3100 at the downstream face of the abutment.

5. Summary

In order to clarify the construction and grouting program, Drawing No. 557-D-673 is included to illustrate this program and the resulting deflections at the crown cantilever. A better understanding of the stress results is possible by studying the load distribution in the dam. Therefore Drawings Nos. 557-D-674, 557-D-675, 557-D-676 and 557-D-677 were prepared to illustrate the load distributions and movements at arch and cantilever elements.

Other than the stresses at the abutments of the top arch, the stresses resulting from the trial-load analysis appear to meet the requirements of the Board of Consultants. If the modulus of elasticity of the foundation rock in place was found to have a value greater than 500,000 psi, this would result in a decrease in stresses near the top of the dam and an increase in the lower part of the dam. The relatively high tensile stresses occurring on the downstream face of the dam are also a result of the low modulus of the foundation rock, and would reduce if the modulus was found to be greater.
Laboratory tests indicate that after the "permanent set" has been taken up in the rock an increase in modulus of elasticity occurs. In the bottom portion of the dam the weight of concrete alone will produce this "set" and result in a higher value of modulus of elasticity. If other measures are taken to eliminate the "set" in the rock in the upper part of the dam before the water load is applied, it is possible that the high stresses at the top of the dam would be reduced substantially.
ASSUMPTIONS AND LOADING CONDITIONS

Tongue of dam, elevation 3715.

Geostatic water surface, elevation 3700.

Top of 50-ft. upstream face, elevation 3550.

Tailwater surface, elevation 3412.

Temperature changes between upstream and downstream faces are

Effects of construction and grading program

1. Concrete placed to elevation 5800, reservoir water surface, elevation 3540, pre-grouted

2. Concrete placed to elevation 4800, pre-grouted to elevation 3715 and other surfaces raised to elevation 4200

3. Construction joints for elevation 3680 to 3715 after concrete has been cooled to a temperature not less than 0.97 of the filling ambient temperature.

Concrete assumptions: Denotes upstream and downstream non-symmetry in the direction of the plane of cracks with an acceleration of 0.1 gravity and a period of vibration of 1.0 second. Incorporates water pressure 15 ft. on all cantilevers. Effects of vertical oscillation not modeled. Efforts to control all and units not included.

Material of similarity of concrete, 3,000,000 pounds per square inch.

Modulus of elasticity of foundation rock, 50,000,000 pounds per square inch.

Hypotenuse ratio of foundation rock, 0.06.

Coefficient of thermal expansion of concrete, 5,000,000,000,000 per degree Fahrenheit.

All cracks analyzed as symmetrical with symmetrical loading.

SUPPORTS AND REINFORCEMENT: Cantilever stresses are acting parallel to the edges of the cantilevers.

SYMBOLS

**F** Stress at base of arch

**I** Stress at interface of arch

**L** Maximum arch shear stress.

**m** Indicate downstream shear.

**u** Stress at upstream edge of cantilever.

**o** Stress at downstream edge of cantilever.

**L** Maximum cantilever Moment shear.

**s** Crown of arch.

**c** Structure adjacent arch points.

**W** All stresses are in pounds per square inch.

**P** Compression. **T** Tension.

SCALE

Design A/3

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER PROJECT

GLEN CANYON DAM
CONCRETE TRUNK LINE SEWER No. 1
ARCH AND CANTILEVER STRESSES

May 13, 1968

397-0-002
Arch stress of 500 pounds per square inch
- Computed arch stress
For constants, assumptions and loading conditions
see Drawing No. 557-D-669
The first principal stress (σ1) makes with the vertical, positive angle measured in a clockwise direction on the left side of the dam, and in a counterclockwise direction on the right side of the dam.

σ2: Second principal stress.

Maximum horizontal shear stress at rock abutment planes. (+) indicates downstream shear.

1 = Compression, 2 = Tension.

For constants, assumptions and loading conditions see Drawing No. 507-D-069.
CONTRACTION JOINT LAYOUT

May 8, 1958

The layout and details of the contraction joints for Glen Canyon Dam have changed slightly from those presented in the technical data for the Board of Consultants, dated April 26, 1957. These changes were made necessary by the contractor's decision to use a 7-1/2-foot placement lift instead of the usual 5-foot lift, and by a change in the design of the dam from that shown in the specification which increased the thickness of the dam at the abutments.

The grouting lifts are now shown 60 feet high rather than the 50-foot previously contemplated. Keyways on the transverse joints are the same as before, but the keyways on the longitudinal joints have been revised as to dimensions and position so as to fit a 7-1/2-foot lift. Details of the transverse and longitudinal contraction joints are shown on Drawings Nos. 557-D-534 and 557-D-536.

The longitudinal joints in the central portion of the dam are located the same as before, that is, 130.0 feet and 161.5 feet downstream from the upstream face. In the blocks to the left of Block 8 and in the blocks to the right of Block 18, the longitudinal joints have been shifted downstream so that in these blocks the longitudinal joints are now 179.5 feet and 151.0 feet downstream from the upstream face of the dam. This change in location of the longitudinal joints was made to decrease the length of the blocks in the area where the abutments were increased in thickness. The layout of the transverse and longitudinal joints is shown on Drawing No. 557-DC-117.

The longitudinal joints are terminated within 15 to 25 feet of the downstream face, and the specifications provide that no concrete is to be placed in that block above the termination of the longitudinal joint until all concrete in the upstream and downstream blocks separated by that longitudinal joint has been cooled to 50°F. Drawings Nos. 557-DC-118 and 557-DC-119 show elevations of the dam through alternate joints, and show the elevations where the longitudinal joints terminate. Drawing No. 557-D-536 shows the detail of the treatment at the top of each of these longitudinal joints.

The block directly above the termination of the longitudinal joint is somewhat longer than the lower blocks. The actual length and shape of these longer blocks are shown by the dotted lines on Drawing No. 557-DC-117. As indicated on this drawing, the maximum size block now is 60 feet wide by 210.88 feet long on one side and 191.28 feet long on the other side. No structural cracking is expected to occur in these blocks because of the 50-degree placement temperature and because, at the elevations where these longer blocks exist, foundation restraint has been reduced materially from that which occurs at the rock-concrete contact surface.
NOTES
Metal seals shall be placed across all contraction joints to prevent water or sediment from entering and preventing the joints from working as intended. All header systems are 1/2" self-draining valveless equivalent. Refer to details at surfaces, linings, embeddments, etc., in Shop 49-D-244.

EXPLANATION
Contraction joint surface

TYPICAL ELEVATION OF CONTRACTION JOINT

CONCRETE FINISH
Supply, burial, outlet return, contact area, water return, metal seal, grout area, and recessed...
THERMAL CONTROL OF CONCRETE

May 18, 1958

Temperature control studies have been completed using an estimated adiabatic temperature rise and laboratory mix. These studies indicate that the temperature rise after placement of 7-1/2-foot lifts can be limited to 25 degrees by the use of cooling pipe and 40°F cooling water. The maximum temperature drop will therefore be limited to 35 degrees in the dam.

The longest block in the dam at the foundation has a length of 190 feet. This will be the most severe condition for any block in the dam because of a 35 degree temperature drop with full foundation restraint. The blocks in the dam which are longer than this, that is, those blocks immediately above where the longitudinal joint terminates and where the maximum block dimensions are as large as 210.88 feet on one side of the block and 191.28 feet on the other side, occur away from the foundation. At the elevations where these longer blocks exist, the effect of foundation restraint has been reduced materially. The closest any of these longer blocks are to the foundation is 115 feet.
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 13-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 \( \text{lb/in}^2 \) at a cost of 4,600 \( \text{lb/in}^2 \) 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 exclude 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.
FIGURE 2
Figure 3. Example of distress in abutment rock presumed to be due to thrust loads.
Memorandum

Denver, Colorado
June 4, 1957

Informational Routing

Mr. Louis G. 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 $20^\circ F$ 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 $3\text{ in.}$ 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.
Denver, Colorado
May 21, 1957

Memorandum

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

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

From: H. Boyd Phillips
     Ira E. Allen

Subject: Photoelastic study of abutment stresses—Glen Canyon Dam—
         Colorado River Storage Project

Introduction

This memorandum presents the results of a photoelastic determination of the stresses that would exist in the abutment rock of Glen Canyon Dam for various arch loads and dimensions. The arch load and dimension assumptions studied were: (1) five linearly varying loads acting normal to a full radial abutment, one load of which is a preliminary estimate furnished by the Trial Load Analysis Unit of arch forces and dimensions for Design A-7, elevation 3665; (2) arch load acting through an abutment pad separate from the arch and with the abutment pad integral with the arch; in both, the pads rest on full radial abutments; (3) preliminary estimate of arch forces and dimensions for Design A-11, elevation 3400, with various dimensions of downstream pad extension; and (4) Design A-11 with a 50-foot radius fillet downstream (with and without earthquake loading).

Results

The abutment stresses for the preliminary estimate for Design A-7, elevation 3665, are given in Figure 1. In computing the vertical stress, $q_v$, an overburden of 105 feet was assumed.

The maximum principal stress versus the minimum principal stress for this loading condition, together with the loading conditions of the preliminary estimate for Design A-11, is plotted on Figure 7. Also shown on Figure 7 is the failure line for horizontally drilled, vacuum saturated 6-inch by 12-inch drill cores, reference (i).

The effect on the foundation reaction of varying the linear distribution of the same total load from uniform to triangular is shown on Figure 2. Figure 3 shows the effect of various thicknesses of pad with the arch acting against the pad and also acting integrally with it.
The effect on the abutment stresses of various dimensions of downstream pad extension was determined for the preliminary estimate of Design A-11, see Figures 4 through 6.

Figure 8 is a photograph of the loading mechanism used with the model of Design A-7.

Conclusions

For the load varying linearly, through an 82-foot length, from 500 psi upstream to 750 psi downstream (Design A-7, elevation 3665), the maximum foundation stress was 1,000 psi. This increase of one-third occurs at the downstream edge of the load and is due to the usual corner concentration. By inserting an 11-foot-thick pad between the arch and abutment, extending 12 feet upstream and 24 feet downstream beyond the arch, as shown in Figure 5, the maximum abutment stress is reduced to about 600 psi. If the arch is integral with the pad, however, the maximum abutment stress is shifted to the upstream edge of the pad and is reduced only to about 750 psi.

For Design A-11, the longest and thinnest footing extension tried gave the best abutment stress distribution, giving a maximum principal stress of about 600 psi. It should be noted that the stress curves in the region of high stress are very steep, making the stress area in this region relatively small.

These tests indicate that considerable reductions in stress intensities in the abutment rock can be accomplished by the use of properly designed abutment pads. For conditions such as those being encountered at Glen Canyon Dam, where the stresses in the rock abutments rather than those in the dam itself are the governing factor in design, use of abutment pads would permit the selection of a more economical dam section than would be required if the entire dam is made heavier in order to provide adequate bearing area at the abutments.

Basic Data

Dimension data for Design A-7 were obtained from Drawing No. 557-DC-2, dated May 10, 1956, and for Design A-11 from a drawing of arch sections furnished by the Trial Load Analysis Unit. Since the trial load analyses for Designs A-7 and A-11 were not completed when this study was started, the arch abutment stresses used were an estimate also furnished by the Trial Load Analysis Unit.
The data used in this study from reference (i) were:

a. The failure line on Figure 7 (Reference (i), Figure 19)

b. The modulus of elasticity of the abutment rock, $E_A = 0.6 \times 10^6$ psi

c. Poisson's ratio, $\mu = 0.10$

d. The specific gravity of dry abutment rock = 2.06

The modulus of elasticity of the pad concrete, $E_P$, was assumed to be $3 \times 10^6$ psi.

The ground surface elevation was assumed to be 3770.

**Technical Details**

Since the same photoelastic material (Columbia Resin, CR-39) was used for the arch, pad, and abutment, it was necessary to increase the pad thickness scale to compensate for the difference in the modulus of elasticity of the concrete and abutment rock. To get the same relative deflection of the model pad as would occur in the prototype with $E_A/E_P = 0.2$, the model pad thickness scale factor was 1.7 times the length scale factor of 0.02. The thickness factor was determined by use of the following equation, reference (ii):

$$\lambda = \sqrt[4]{\frac{K}{4E_PI}}$$  \hspace{1cm} (1)

where

$\lambda$ represents the characteristic of the system which includes the flexural rigidity of the pad as well as the elasticity of the abutment,

$K$ the modulus of the abutment,

$E_P$ the modulus of elasticity of the pad,

$I$ the moment of inertia of the cross section of the pad.

$K$ was computed using the following equation, reference (iii):

$$K = 0.71 \left[ \frac{E_A b^4}{E_P I} \right]^{\frac{1}{3}} E_A$$  \hspace{1cm} (2)
where

\[ E_A \text{ represents the modulus of elasticity of the abutment rock, and } \]
\[ b \text{ one-half the width of the cross section of the pad.} \]

For the model to have the same characteristic as the prototype, \( \lambda_{model} = \lambda_{prototype} \). If the value of \( \lambda \) is computed in terms of the pad thickness, the relation of model to prototype thickness can be determined.

After determining the principal stresses in the load plane, \( \sigma_1 \) and \( \sigma_2 \), by the photoelastic interferometer, the principal stress normal to the load plane \( \sigma_3 \), was computed by the theory of elasticity equation

\[ \sigma_3 = \sigma_w + \mu(\sigma_1 + \sigma_2) \]

where \( \sigma_w \) is the intensity of vertical load of the overburden.

The load for Design A-11 includes a shearing force, which would require cementing the model loading shoe to the abutment. Instead of doing this, an integral model was used. To determine if these two methods would give the same results, a vertical load only was applied to a nonintegral model with a plaster of paris joint and to an integral model. The stresses along the same line in both models were almost identical.

**Personnel**

This study was made under the general supervision of W. T. Moody. R. P. Kotelly assisted in the experimental work, and H. E. Willmann prepared the drawings.

**References**

(1) Structural Laboratory Report No. SP-30, Part I, Strength and Elastic Properties of Navajo Sandstone Core from Glen Canyon Dam Site, Mile 15\(\frac{3}{4}\), Colorado River Storage Project, Arizona, October 1, 1951.


COLORADO RIVER STORAGE PROJECT
GLEN CANYON DAM
ABUTMENT STRESSES
VARIOUS LINEAR DISTRIBUTIONS OF LOAD

For principal stress convention, see Figure 1.
Tables show principal stresses of points of maximum $\sigma_1$.
$\sigma_1$ = maximum principal stress in plane of load.
$\sigma_3$ = minimum principal stress in plane of load.
$\sigma_2$ = principal stress normal to plane of load
due to 105 feet of overburden.
--- stress parallel to load.
--- stress normal to load.

MAY 20, 1957

557-PEL-15
DIMENSIONS AND PRINCIPAL STRESSES

\[ \sigma_x \text{ STRESS} \]

\[ \sigma_y \text{ STRESS} \]

\[ \tau_{xy} \text{ STRESS} \]

Notes: Stresses are plotted on the Y-projection of the plane of contact. For principal stress convention, see Figure 1.

COLORADO RIVER STORAGE PROJECT
GLEN CANYON DAM
ABUTMENT STRESSES
PRELIMINARY ESTIMATE OF ARCH FORCES
FOR DESIGN A-11 - ELEV. 3400

MAY 20, 1957
COLORADO RIVER STORAGE PROJECT
GLEN CANYON DAM

ABUTMENT STRESSES
PRELIMINARY ESTIMATE OF ARCH FORCES - DESIGN A-11 AT ELEV. 3400
FOR VARIOUS DOWNSTREAM PAD EXTENSIONS

MAY 20, 1957

557-PEL-18
Notes:
Excluding earthquake:
F_x = 1,864 Kips
F_y = 10,215 Kips
M = 103,760 Ft. Kips

With earthquake:
F_x = 2,452 Kips
F_y = 12,257 Kips
M = 134,730 Ft. Kips

Stresses are plotted on the Y-projection of the plane of contact.

COLORADO RIVER STORAGE PROJECT
GLEN CANYON DAM
ABUTMENT STRESSES
PRELIMINARY ESTIMATE OF ARCH FORCES - DESIGN A-11 AT ELEV. 3400
FOR 3 LENGTHS OF DOWNSTREAM PAD EXTENSION
WITH PROPOSED 50 FT. RADIUS FILLET

MAY 20, 1957
Notes:
X Drill core failure points.
O Plotted from principal stresses in Figures 1, 2, and 4.

COLORADO RIVER STORAGE PROJECT
GLEN CANYON DAM
ABUTMENT STRESSES
MAXIMUM PRINCIPAL STRESS VS. MINIMUM PRINCIPAL STRESS

MAY 20, 1957
Memorandum

Chief Designing Engineer

Chief, Division of Engineering Laboratories

Effect of lateral restraint on the compressive strength of Navajo sandstone--Glen Canyon Dam--Upper Colorado River Storage Project

Introduction

Tests of HX and H2 cores drilled from the foundation of Glen Canyon Dam site indicate that some areas exist, particularly in the left abutment, where moduli of elasticity and strength are below the original design assumptions. Since the previous tests were made in the unconfined or partially confined condition, further testing was requested verbally by L. G. Puls to determine the comparative strength of Glen Canyon sandstone under conditions of lateral restraint approaching those in an infinite mass of rock.

The tests reported herein were planned and conducted in collaboration with Robert Glover who performed load bearing tests on the Dakota sandstone foundation rock underlying the capitol building in Lincoln, Nebraska.

Conclusions

These tests indicate that the compressive strength of Navajo sandstone can be increased to about 3.8 times the unconfined compressive strength, when the load is applied to only a small central area of the rock which is laterally restrained by a ring of reinforced material.

Description of Tests

As shown in Figure 1, an 18-inch-diameter by 12-inch-high cylindrical mold and 1/4-inch steel rod reinforcement were used to cast a saturated 6- by 6-inch Glen Canyon sandstone core (Specimen No. 56 Kk-33-51) in a material which had elastic properties similar to the sandstone.

Both mortar and molding plaster ("Red Top") cylinders were previously investigated to determine modulus of elasticity and compressive strength. The plaster was selected for use because the elastic properties were approximately the same as the sandstone within the time allowed to complete the test.
Two NK size cores were drilled adjacent to the 6- by 6-inch core for determination of modulus of elasticity and strength in the unconfined condition. Two 2- by 4-inch companion cylinders of plaster were also tested for modulus of elasticity and compressive strength. The plaster mixture used was 1 part water to 1.69 parts plaster by weight.

The sandstone surface of the composite specimen was first loaded at the center with a high-strength concrete cylinder (see Figure 2), which failed in this test. The sandstone was then loaded at the center, to failure with a 2-inch diameter by 1/2-inch thick steel disc (see Figure 3) topped by a 1- by 1-inch steel cylinder to allow deflection of the disc.

Test Results

Modulus of elasticity and compressive strength of the unconfined sandstone and molding plaster are shown in Table 1.

The composite sandstone-plaster specimen was tested at age 3 days. The first compression test (Figure 2) resulted in the failure of the concrete cylinder at 8,450 psi; 2.27 times the unconfined compressive strength of the sandstone. This test caused indentation of approximately 1/32 inch in the sandstone.

During the second compression test (Figure 3) the steel disc was pressed into the sandstone a total of 1/2 inch with an ultimate load of 18,000 psi or 3.77 times the unconfined compressive strength of the sandstone. The first appreciable penetration of the sandstone occurred at 5,030 psi with a drop in load at this point. The load then increased after successive partial failures to the ultimate load.

Failure of the specimen occurred by relief of stress in the direction of least resistance which was upward causing approximately horizontal cracking at successively increasing depths. Figures 3, 4, and 5 show the method of failure. Figures 6, 7, and 8 show sectional views of the failure pattern.

Enclosures

Copy to: L. G. Puls
E. R. Dexter
W. R. Irwin
W. Y. Holland
O. J. Olsen
R. E. Glover
S. Davidson

U.S. G.S. Bldg. 25
### Table 1

ELASTICITY AND COMPRESSIVE STRENGTH RESULTS

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>First Loading Load (psi)</th>
<th>Second Loading Load (psi)</th>
<th>Compressive Strength (ksi)</th>
</tr>
</thead>
</table>

**Red Top Molding Plaster**

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>First Loading Load (psi)</th>
<th>Second Loading Load (psi)</th>
<th>Compressive Strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (age 60 hr)</td>
<td>100</td>
<td>.52</td>
<td>.11</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>.48</td>
<td>.15</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>.41</td>
<td>.19</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>.37</td>
<td>.23</td>
</tr>
<tr>
<td>2 (age 92 hr)</td>
<td>100</td>
<td>.62</td>
<td>.12</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>.53</td>
<td>.16</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>.49</td>
<td>.19</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>.45</td>
<td>.22</td>
</tr>
</tbody>
</table>

**Olen Canyon Sandstone (saturated)**

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>First Loading Load (psi)</th>
<th>Second Loading Load (psi)</th>
<th>Compressive Strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ka-33B-51.5</td>
<td>100</td>
<td>.37</td>
<td>.03</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>.38</td>
<td>.04</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>.36</td>
<td>.08</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>.42</td>
<td>.12</td>
</tr>
<tr>
<td>Ka-33J-51.5</td>
<td>100</td>
<td>.41</td>
<td>.07</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>.39</td>
<td>.11</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>.40</td>
<td>.13</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>.42</td>
<td>.15</td>
</tr>
</tbody>
</table>
Figure 1. View of 18-inch diameter by 12-inch mold with reinforcement in place; 6- by 6-inch Glen Canyon sandstone core is in position for placement of plaster in mold. Handles shown at bottom were not used.
Figure 2. View after first compression test during which the concrete cylinder failed at 8,450 psi. Note 1/32-inch indentation at center of sandstone surface.
Figure 3. Ultimate failure of sandstone with steel disc imbedded 1/2 inch.
Figure 4. Top view of composite specimen after removal of steel disc and the larger fragments of sandstone displaced during failure. The larger fragments were moved radially from their original position for this picture. Note pulverized material remaining. Small cracks in the plaster are visible at the right and left of the sandstone specimen.
Figure 5. Closeup angle view of composite specimen shown in Figure 4.
Figure 6. Sectional view of composite specimen. One-quarter of the cylinder was removed by sawing. Plaster was placed in the cavity of the top of the sandstone specimen before sawing. The void below this plaster cap was caused by loss of pulverized material during sawing.
Figure 7. Closeup view of sectioned composite specimen after failure.
Figure 8. Closeup view of the one-fourth section of the composite specimen. This corresponds to the left one-fourth section shown in Figure 7.
Memorandum

Chief Designing Engineer

Chief, Division of Engineering Laboratories

Denver, Colorado
March 1, 1957

Effect of lateral restraint on the modulus of elasticity and compressive strength of Navajo sandstone--Glen Canyon Dam--Upper Colorado River Storage Project

INTRODUCTION

This memorandum presents results of special tests on Glen Canyon sandstone requested verbally by L. G. Puls.

A comparison has been made of strength and modulus of elasticity of laterally unrestrained and partially restrained Glen Canyon, Navajo, sandstone cores. Specimens were taken from a 6- x 12-inch core (No. 31-5-103) which had been vertically drilled at elevation 3159 from Hole No. 31. Triaxial tests of core from this drill hole were conducted in 1950 and reported in Structural Laboratory Report No. SP-30, Part I. Figure 2 of above mentioned report shows the location of the drill hole with respect to the proposed dam and Figure 16 shows the principal stress relation at failure of this material in the saturated condition. The tests of core No. 31-5-103 reported herein, were also conducted in the saturated condition.

CONCLUSIONS

These limited tests indicate that the compressive strength of Navajo sandstone is increased when the load is applied on only a portion of the area of the material due to the effect of lateral restraint imposed by the material surrounding the loaded area. In this case where a core 5.9 inches in diameter and 3.76 inches in length was loaded at the center with a steel disc 2.1 inches in diameter (HX size) leaving 1.9 inches from the edge of the disc to the outer edge of the core, the compressive strength was increased about 2.6 times that of the unconfined HX Core.

A comparison of secant modulus of elasticity between the restrained and unrestrained conditions shows similar values after consolidation of the sand grains. As shown in Table 1 however, the modulus of elasticity during the first loading in
the 5.9-inch partially restrained core is considerably lower than that of the unrestrained NX core. It should be noted that these are computed from average strain and average stress, whereas, actually the strain and stress vary from point to point in the restrained core.

TEST PROCEDURE AND RESULTS

Core No. 31-5-103 was sawed perpendicular to the axis into three pieces approximately 5.9 diameter by 3.7 inches high. Bedding planes in all specimens were 67.5° from the loading axis. Three NX (2-1/8-inch diameter) cores (No. 1B, 2B, and 3B) were drilled from the bottom third, parallel to the axis. Modulus of elasticity and strength were obtained on each of these NX cores by the usual unconfined method. Modulus of elasticity measurements were made with frames containing unbonded resistance wire strain gages. The strength results were corrected to L/D ratio of 2.

The top third (Specimen No. 4) of the 6 x 12 core was loaded at the center with an NX size loading disc thereby introducing partial lateral restraint. This loading arrangement is shown in Figure 1.

The middle third of the 6 x 12 core (Specimen No. 5) was loaded in the same way as the top third and an attempt was made to approximate total lateral restraint by surrounding the specimen with a steel ring of standard weight steel pipe and filling the void between the specimen and ring with iron cement ("Smooth-On No. 1"). SR-4 gages were attached to the outer surface of the steel ring to measure axial and lateral strain of the ring. This specimen is shown in Figure 2. The attempt to approximate total lateral restraint failed because the iron cement did not harden except at the exposed surfaces after 12 days' age.

Results of elasticity and compressive strength tests are presented in Table 1. For the 6-inch cores, the axial stress was calculated on the NX loaded area and the gage length was the total length of the core. The axial stress on the NX cores was calculated the same way and the gage length was governed by the elasticity frame, 2 inches.
Figure 3 shows one of the partially restrained specimens (No. 4), after failure. The other partially restrained specimen (No. 5) is shown in Figure 4 after failure, with the steel ring removed. The 6-inch cores failed, first by shearing along a cylindrical shape directly under the NX loading head similar to punching action, and then by splitting of the core radially.

Enclosures

Copy to: L. G. Puls
E. R. Dexter
O. J. Olsen
W. H. Irwin
W. Y. Holland
R. E. Glover
S. Davidson
Figure 1  Loading and strain measuring arrangement used on 5.9- x 3.7-inch cylinder to determine modulus of elasticity and strength of Glen Canyon sandstone with partial lateral restraint.
Figure 2. View of 5.9- x 3.7-inch specimen of Glen Canyon sandstone in steel ring after loading at center with NX disc to failure.
Figure 3. View of Specimen No. 4 after failure.
Figure 4. View of Specimen No. 5 after failure. Steel ring has been removed. Bedding planes were marked on the fractured surfaces.
Table 1

EFFECT OF LATERAL RESTRAINT ON GLEN CANYON NAVAJO SANDSTONE
ELASTICITY AND COMPRESSIVE STRENGTH TEST RESULTS

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Axial stress (psi)</th>
<th>Modulus of elasticity (E x 10^-6)</th>
<th>Poisson's ratio</th>
<th>Compressive strength, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylindrical:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>No.</th>
<th>size (inches)</th>
<th>disc, psi</th>
<th>First loading, psi</th>
<th>Second loading, psi</th>
<th>Third loading, psi</th>
<th>Fourth loading, psi</th>
<th>First correction, L/D ratio</th>
<th>Second correction, L/D ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-B</td>
<td>1.000</td>
<td>2.09: 3.67</td>
<td>1,000</td>
<td>--</td>
<td>--</td>
<td>0.55</td>
<td>0.57</td>
<td>0.14</td>
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<tr>
<td></td>
<td>2,000</td>
<td>2.09: 3.67</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>0.75</td>
<td>0.77</td>
<td>0.18</td>
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<tr>
<td>2-B</td>
<td>1.000</td>
<td>2.10: 3.66</td>
<td>1,000</td>
<td>0.52</td>
<td>0.95</td>
<td>--</td>
<td>--</td>
<td>0.15</td>
</tr>
<tr>
<td></td>
<td>2,000</td>
<td>2.10: 3.66</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>0.71</td>
<td>0.76</td>
<td>0.18</td>
</tr>
<tr>
<td>3-B</td>
<td>1.000</td>
<td>2.10: 3.71</td>
<td>1,000</td>
<td>0.52</td>
<td>0.57</td>
<td>--</td>
<td>--</td>
<td>0.16</td>
</tr>
<tr>
<td></td>
<td>2,000</td>
<td>2.10: 3.71</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>0.76</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
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<table>
<thead>
<tr>
<th>Partially Restained</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>4</td>
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<td></td>
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<td></td>
</tr>
<tr>
<td>5</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

*With steel ring.*
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, Formal 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>
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 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 VII</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, Mousner, 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 resonance 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, Housner, 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>Storage building with reinforced concrete frame and floor and hollow tile walls</td>
<td>0.14</td>
<td>USC&amp;GS--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>
<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>0.33</td>
</tr>
<tr>
<td>El Centro, California</td>
<td>E-W</td>
<td>May 18, 1940</td>
<td>0.23</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>
<td>S 45 E</td>
<td>February 9, 1941</td>
<td>0.07</td>
</tr>
<tr>
<td>Location</td>
<td>Direction</td>
<td>Magnitude</td>
<td>Depth</td>
</tr>
<tr>
<td>----------------------------------</td>
<td>-----------</td>
<td>-----------</td>
<td>-------</td>
</tr>
<tr>
<td>Ferndale, California</td>
<td>N 45 E</td>
<td>0.25</td>
<td>0.11</td>
</tr>
<tr>
<td>October 3, 1941</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ferndale, California</td>
<td>S 45 E</td>
<td>0.50</td>
<td>0.12</td>
</tr>
<tr>
<td>October 3, 1941</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Santa Barbara, California</td>
<td>N 45 E</td>
<td>0.20</td>
<td>0.23</td>
</tr>
<tr>
<td>June 30, 1941</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Santa Barbara, California</td>
<td>S 45 E</td>
<td>0.32</td>
<td>0.24</td>
</tr>
<tr>
<td>June 30, 1941</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hollister, California</td>
<td>S 01 W</td>
<td>0.20</td>
<td>0.12</td>
</tr>
<tr>
<td>March 9, 1949</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hollister, California</td>
<td>N 89 W</td>
<td>0.26</td>
<td>0.23</td>
</tr>
<tr>
<td>March 9, 1949</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Olympia, Washington</td>
<td>S 10 E</td>
<td>0.60</td>
<td>0.18</td>
</tr>
<tr>
<td>April 13, 1949</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Olympia, Washington</td>
<td>S 80 W</td>
<td>0.50</td>
<td>0.31</td>
</tr>
<tr>
<td>April 13, 1949</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seattle, Washington</td>
<td>N 88 W</td>
<td>0.46</td>
<td>0.080</td>
</tr>
<tr>
<td>April 13, 1949</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seattle, Washington</td>
<td>S 02 W</td>
<td>0.47</td>
<td>0.070</td>
</tr>
<tr>
<td>April 13, 1949</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Taft, California</td>
<td>S 69 E</td>
<td></td>
<td>about 0.17</td>
</tr>
<tr>
<td>July 21, 1952</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Taft, California</td>
<td>N 21 E</td>
<td></td>
<td>about 0.16</td>
</tr>
<tr>
<td>July 21, 1952</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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.

[Signature]

10
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, Irwin, 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. Joint-
ing 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 tri-axial 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 unrestrained 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-18 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 waterload 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-18. 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. M. McClellan
Assistant Commissioner
and Chief Engineer
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
Correlation of deformations observed when loads are applied locally to a surface of Glen Canyon sandstone with the deformations observed on conventional companion specimens.

Introduction

An important factor in the Glen Canyon design studies is the amount of abutment deformation to be expected. These deformations are a result of the thrusts applied to the abutments by the dam. Because the loads are applied locally, a three dimensional state of stress is produced in which the stress decreases rapidly in intensity with increasing distance from the loaded area. The abutment deformations to be expected are generally computed by Vogt's formulas or expressions similar to them. These formulas were obtained by integration of Boussinesq's solutions of the elastic equations and therefore imply that the abutment material has elastic properties. Test data indicate that the Glen Canyon sandstone departs materially from a truly elastic behavior and it is therefore important to evaluate the effect of these departures on the estimated abutment deformations.

To obtain data which could be used for these purposes a test was made on the plane face of a sample of Glen Canyon sandstone. The sample was of cylindrical form 5.9 inches in diameter and 3.76 inches in length and was loaded through a 2.1 inches diameter steel disc pressed against the circular face of the test specimen. The steel disc was applied at the center of this face. The test area is therefore surrounded by an unloaded region.

Analyses of test results

The stress distribution appropriate to this case is treated on an elastic basis in paragraph 106 of the "Theory of Elasticity" by S. Timoshenko., First Edition 1934. The depression w of the loaded area under a rigid die in the form of a circular cylinder of radius a pressed against the plane boundary of an elastic solid having a Young's modulus E and a Poisson's ratio μ is, for the total load P:

\[
w = \frac{P(1-\mu^2)}{2aE}
\]

Although this formula implies the existence of elastic properties, and we have reason to believe that the Glen Canyon sandstone departs from these characteristics, we can obtain useful results by using it to find the Young's modulus which
would be needed to produce the observed deformations. If the Young’s moduluses so obtained are used with the abutment deformation equations an improved result should be obtained. The results of an analysis made in this way are shown in tables 1 to 4 below.

**Table 1**

Equivalent modulus $E_1$ which will yield the same deformations as observed on the Glen Canyon sandstone.

**Specimen No 4. First load application.**

<table>
<thead>
<tr>
<th>Average pressure (lb/in²)</th>
<th>Total load $P$. (lb)</th>
<th>$w$ (in)</th>
<th>$E_1$ (lb/in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>---</td>
</tr>
<tr>
<td>200</td>
<td>687</td>
<td>.00919</td>
<td>35000</td>
</tr>
<tr>
<td>400</td>
<td>1372</td>
<td>.011905</td>
<td>54000</td>
</tr>
<tr>
<td>600</td>
<td>2060</td>
<td>.013655</td>
<td>70500</td>
</tr>
<tr>
<td>800</td>
<td>2750</td>
<td>.015545</td>
<td>82800</td>
</tr>
<tr>
<td>1000</td>
<td>3430</td>
<td>.01744</td>
<td>92000</td>
</tr>
</tbody>
</table>

*Note: Loaded area 2.09 in dia, Area 3.43 in²*

**Table 2**

Equivalent modulus $E_1$ which will yield the same deformations as observed on the Glen Canyon sandstone.

**Specimen No 4. Third load application.**

<table>
<thead>
<tr>
<th>Average pressure (lb/in²)</th>
<th>Total load $P$. (lb)</th>
<th>$w$ (in)</th>
<th>$E_1$ (lb/in²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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<td>0</td>
<td>---</td>
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<tr>
<td>1000</td>
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</table>
Table 3

Equivalent modulus $E_1$ which will yield the same deformation as observed on the Glen Canyon sandstone.

Specimen No 5. First load application.

<table>
<thead>
<tr>
<th>Average pressure $\text{lb/in}^2$</th>
<th>Total load $P$, lb</th>
<th>$w$, in</th>
<th>$E_1$, $\text{lb/in}^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>-----</td>
</tr>
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<td>200</td>
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<td>115400</td>
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<td>800</td>
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<td>132000</td>
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<tr>
<td>1000</td>
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<td>151300</td>
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</table>

Table 4

Specimen No 5. Fourth load application.

<table>
<thead>
<tr>
<th>$p$, lb/in²</th>
<th>Total load $P$, lb</th>
<th>$w$, in</th>
<th>$E_1$, $\text{lb/in}^2$</th>
</tr>
</thead>
<tbody>
<tr>
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<td>500</td>
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<td>.00809</td>
<td>198300</td>
</tr>
</tbody>
</table>
Check from test in tunnel.


Refer to fig. 10. F.W. Test.
Load 45 tons per square foot (Olsen thinks he used 2000 lb tons) Deflection 0.0092 inches as read from chart.

\[ w = \frac{P(1-\mu^2)}{2aE} \]

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

With \( P = 90000 \text{ lb} \quad 2a = 13.56 \quad (1-\mu^2) = 0.9775 \)

\( \mu = 0.15 \quad \text{Loading} \ 625 \text{ lb/in}^2 \)

\[ E = \frac{(90000)(0.9775)}{(13.56)(0.092)} = 705000 \text{ lb/in}^2 \]

Jones gets 701000 lb/in\(^2\) from the same data

Refer to fig 11.
Load 15 tons/ft\(^2\) 30000 lb
Deflection .0029 inches as read from chart
Test area circular 13.56 inches diameter
Test area circular 1 square foot or 144 square miles

\[ E = \frac{(30000)(0.9775)}{(13.56)(0.029)} = 745000 \text{ lb/in}^2 \]