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REPORT OF TEST
ON
STEVENSON CREEK DAM

By
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REPORT OF TEST ON STEVENSON CREEK DAM.

By Willis A. Slater.

I. GENERAL DESCRIPTION.

11. Introduction. This report described and gives the results of load tests made on an experimental concrete arch dam. This project was initiated by a committee of Engineering Foundation for the purpose of securing information on the behavior of arch dams under load, and with the expectation that it would be an important step in establishing rational standards for the design of arch dams. The site for a test dam had been chosen, the dam had been designed, and its construction and testing had been financed, when, in 1925, the National Bureau of Standards was invited to cooperate in the carrying out of the test.

About December 1, 1925, the writer arrived in Los Angeles to take charge of the experimental work in the carrying out of this test as representative of the Bureau of Standards. At that time rock excavation for the foundation of the dam was in progress. The dam was constructed during the spring, and the load tests occupied the latter part of the summer, of 1926. The working up of the data and the preparation of this report have been carried out in Los Angeles since October 1, 1926, under the direction of the writer.

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The members of the testing crew, most of whom were with the work throughout most of the construction and all of the testing, were:

G. H. Barry

R. W. Carlson

A. W. DeYoe

K. M. Fenwick

Inge Lyse

L. J. Marchand

J. J. Stout

B. E. Wilson.

All of this crew, except Messrs. Fenwick and Wilson, have participated in the preparation of this report. All of these men have worked intelligently and loyally to make the test and the report successful, and the fullest appreciation of their support is herewith expressed. Special recognition is due Messrs. Carlson and Lyse for taking a high degree of initiative in the interpretation of the results, and to Mr. Lyse for his assistance in formulating the report.

Just as important as the proper carrying out of the test was the proper construction of the dam, which was carried out by the construction department of the Southern California Edison Company. Unlimited pains were taken to

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secure exactness of detail in the excavation and construction, and to secure uniformity in strength of concrete and in the curing of the completed structure.

It is far more important in such investigations to secure a dam which will represent the behavior of an ideal structure than to secure one which conforms to the ordinary limitations of the construction job. Although it may never be possible in practice to build a structure as symmetrical and as true to the designed dimensions, etc., as this one, the uncertainties which would enter the results if the ordinary limitations of the construction had been allowed to control, would be so great that neither practical nor ideal results would be secured.

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12. General Description of Test Dam. Stevenson Creek Test Dam is located about 60 miles east of Fresno, California, on Stevenson Creek, a tributary of the San Joaquin River. It is located in a small gorge of such shape as to give a very small reservoir capacity. This is shown in Fig. It is close to a large conduit of the Southern California Edison Company's hydroelectric system. Water from this conduit was used in carrying on the tests.

Fig. 2 shows the plan, vertical section, and the developed vertical surface of the dam, which is of the constant radius type, with vertical upstream face. The radius for the upstream face is everywhere 100 ft. The maximum angle included between the radii is 80 deg. 37 min. at the 60-ft. elevation. The abutment lines on the developed surface are straight and have an inclination of 45 deg. from the 60-ft. down to the 7.5-ft. elevation. At the latter elevation the abutment line is tangent to a circular arc of 25-ft. radius which forms the profile of the developed surface below the 7.32-ft. elevation.

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A vertical radial section anywhere in the dam gives for the downstream face a vertical straight line down to the 30-ft. elevation. At all points of the 30-ft. elevation this straight vertical line is tangent to a circular arc of 84.568 ft. radius, which forms the profile of the downstream face below the 30-ft. elevation. The thickness of the dam, therefore, is 2 ft. everywhere, from the top down to the 30-ft. elevation. Below this elevation the thickness increases by the ordinates to the circular arc, until at the bottom the thickness becomes 7.5 ft. This gives a constant thickness for all parts of any horizontal element. The thicknesses at the 5-ft. contours are shown in Fig. 2.

The rock at the site of the test is a strong granite. The gorge is of a natural V-shape and fits closely the outline of the dam as designed.

The dam is provided with an undersluice, shown in Fig. 3. This undersluice occupies a natural fissure in the rock It is capped by a slab having an average thickness of about 4 ft., and reinforced heavily with steel rails. Observations made with the level bar and the clinometer during the testing indicated that the slab over the undersluice was as stationary as any other part of the foundation. The throat of the undersluice is 4 by 6.5 ft. and was provided with stop-logs which could be removed when it was desired to let all of the water pass the dam. It was found, however, that even with the stop-logs out, a heavy storm during the winter of 1926-27 caused the undersluice to clog up and the reservoir to fill to the top of the dam.

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A 24-in. gate valve was provided for ordinary emptying of the reservoir so that it was not necessary to remove the stop-logs during the period of testing. For more exact control of the water, a 6-in. gate valve also was provided. Fig. 2 shows the arrangement of the undersluice and the gate valves.

An oversluice consisting of a 24-in. wood stave pipe permitted the carrying of all of the ordinary flow of Stevenson Creek over the top of the dam. This oversluice was necessary for the purpose of diverting the water during the time of work on the installation of the stop-logs and gate valves, and was useful at other times for diverting the flow. The oversluice is shown in Fig. 4.

For convenience in taking observations, framework on both the upstream and the downstream sides of the dam, supported platforms 3 ft. wide and extended the length of the dam at elevations 54 ft. 9 in.; 44 ft. 9 in.; etc. (10 ft. apart), down to elevation 24 ft. 4 in., on the upstream side, and 4 ft. 9 in. on the downstream side.

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II. CONSTRUCTION.

21. General Procedure. Excavation for the foundation of the dam began during August, 1925. Although not a a great deal of excavating was required, the space available for work was small and a high degree of care to bring the final profile to close conformity with the design without damage to the foundation rock was necessary. These conditions, together with the fact that there was serious delay due to storms, made the excavation require a long time. The excavation was finished by January 20, 1926, except for that which was necessary to provide a foundation for the deflection towers. The profile at the site ofthe dam along the upstream face is given in Fig.5. The profile of the final excavation is shown as a rough, solid line. The theoretical profile of the dam is shown by the lighter, smooth, solid line. During excavation operations some of the rock below the theoretical profile of the foundation was loosened. This loose rock was removed and where the variation from the profile appeared to be of importance, the cavities were filled with concrete. This concreting of the over-cut was done during the concreting of the dam, but several days ahead of the concreting of the dam at these elevations. The concreted portions of the over-cut, also the slab over the undersluice, are cross-hatched in Fig. 5. The dotted line shows the original profile of the rock along the upstream face of the dam.

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The slab over the undersluice was concreted on January 25. Storms immediately following the placing of this slab caused much damage and delayed the progress for nearly two months before the concreting of the dam began. The storms referred to washed the fine material from the tunnel muck, thus dislodging larger rock which in turn did much damage in falling. Efforts were made to protect the work against such damage by means of heavy barricades, but these barricades were ineffectual and it was finally concluded that it would be necessary to remove from above the dam site the entire bank of excavated material which had been deposited there in previous years. A 4-inch stream of water under a head of about 100 ft. was used for washing down the excavated material. It is estimated that in all about 20,000 cu.yd. of rock were thus washed down into the canyon. This stream of water was not sufficient to do more than to wash the rock into the bottom of the canyon. From there it was dislodged by a flood of water produced by opening the gates of the Shaver Lake reservoir of the Southern California Edison Company. About 840 ac.ft. of water passed down the canyon in less than two hours, effectually cleaning out the bottom of the canyon and clearing the site for construction. This washing out was done on March 17.

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From this date on, the preparations for concreting went ahead without delay, except for the loss of some time during rainy weather. Some slight amount of excavation was still necessary, mainly in order to provide bases for the towers which were intended for use in measuring deflections by means of the resistance micrometers. These tower bases were concreted on April 16, and the concreting of the dam proper began on April 19.

With each day's pouring the height of the dam was increased 5 ft. Each pouring was made up of four lifts of 15 in. each. This division of the day's pouring into four lifts avoided having a depth of fresh concrete so great as to cause appreciable bulging of the forms.

A height of 5 ft. of concrete was added every three or four days until June 4, when the dam was completed to a height of 60 ft.

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22. Materials. The cement was furnished without charge by the Colton Cement Company. At the time when it was expected that the construction would begin in the fall of 1925, the supply of cement was shipped. The cement was received at Big Creek on October 12, 1925, and immediately placed in a cement storage house at the site of the test dam. Owing to delays mentioned in a previous section, the cement had been in storage about six months before construction of the dam actually began. Examination of a number of bags of cement showed a considerable number of lumps. It is probable that the lumpiness was due entirely to the length of time during which the cement had been in storage. Although it was stored under a tight roof, it is to be expected that enough moisture would penetrate to have some effect on the cement during this length of time.

The aggregates were of three sizes, all of which consisted of crushed granite. The coarse aggregate was divided into two sizes, that which passed through a $1\frac{1}{2}$-in. screen with round holes and was retained on a screen with $\frac{3}{4}$-in. square holes, and that which passed a screen with $\frac{3}{4}$-in. square holes and was retained on a screen with $\frac{3}{8}$-in square holes.

The fine aggregate was taken from that which passed the $\frac{3}{8}$-in. screen. The unwashed aggregate passing the $\frac{3}{8}$-in. screen contained about 26 per cent of material finer than the No. 100 sieve. This was believed to be too much fine

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material, so the fine aggregate was washed to remove the fine material. Fig. 6 gives the sieve analyses for all the aggregates, including the fine aggregate, both before ans after washing. While the purpose of the washing was to remove the material finer than No. 100 sieve, it will be seen from Fig. 6 that considerable quantities coarser than that sieve were removed also. Table 2 gives the sieve analyses made of the aggregate on successive days of pouring concrete.

In considering the question of gradation of the fine aggregate, tests were made by Prof. R. E. Davis of the University of California, to determine whether the large amount of fine material would cause considerable shrinkage in the concrete. The specimens used were 3 in. square and 40 in. long and the change of length due to shrinkage was measured from end to end. Two independent series of tests were run, and in each case two specimens of a kind were made. For all specimens the water-cement ratio was 1.0. In the first series the mix, stated as ratio of cement to mixed aggregate by volume, was 1:4. In the second series using aggregate having 25, 6, and 0 per cent sand finer than the No. 100 sieve, the mixes were 1:4.45, 1:4.82, and 1:5.15, respectively. For the second series these mixes were designated in Fig. 6 by A, B, and C, in the order given above. The specimens were kept covered by wet burlap at a temperature of about 12.8 deg. C. for a period of four days after casting. During this time

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no shrinkage was found. From this time on, the specimens were subjected to a higher temperature and allowed to dry for a certain length of time. They were then returned to the constant-temperature room and brought to the initial temperature of 12.8 deg. C. before measuring and weighing to determine the shrinkage and loss of weight. Finally they were oven dried to a constant weight at a temperature of 40.5 deg. C. and then returned to the initial temperature of 12. 8 deg. C. before the final measuring and weighing. This period of oven drying is indicated in Fig. 6. Although in general the specimens having the most fine material showed the least shrinkage, the difference was slight, and in both series the shrinkage appeared to be practically independent of the amount of fine material. In neither case was the shrinkage unusually high.

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23. Measurement of Materials for Concrete. The materials were proportioned by volume. The cement was measured in bags assumed to contain 1 cu.ft. each. Medium and coarse aggregates were measured in hoppers and the fine aggregate and water were measured in a Blaw-Knox inundator.

For the purpose of proportioning, the weights of the fine aggregate by loose measure, exclusive of moisture, were taken at 91 lb. per cu.ft. The weight of the medium-size aggregate was nearly the same as that of the coarse aggregate and the mixture of the two by loose volume, exclusive of the moisture, was taken at 93 lb. per cu.ft. The results of several weighings under different conditions of compactness are shown in Table 9.

Hoppers for measuring the medium and the coarse aggregates were provided by arranging the lower parts of the chutes from the aggregate bins so that filling them to a predetermined elevation measured out a known quantity of aggregate. At the bottom of the chute a hinged gate was provided to permit the discharge of the aggregate directly into the batch hopper which was mounted on a car for hauling to the mixer. The upper part of the hopper was narrowed down to a small cross-section to secure accuracy of measurement. The determination of the capacity of the hopper was based on weighing tests of the contents made in advance of construction. During construction additional weighing

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tests were made on the medium and the coarse aggregates from one batch during each day's run.

The inundator is a device for measuring sand in water. The advantage to be expected from its use is uniformity of quantities of both sand and water which go into batches of concrete.

A series of tests was carried out in 1922 at the U. S. Bureau of Standards* to test the reliability of the method. The tests indicated that a reasonably close control of both water and sand might be secured by the inundation method of measurement. Since that time the inundator has been manufactured and placed on the market by the Blaw-Knox Company of Pittsburgh, and this company furnished without expense to the committee an inundator for use in making concrete for the construction of the test dam.

* G. A. Smith and W. A. Slater, "Inundation Methods for Measurements of Sand in Making Concrete". Proceedings Am. Con. Inst. Vol 19, p. 222 (1923).

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24. Mixing and Placing of Concrete. The concrete was mixed 1 $\frac{1}{2}$ minutes in a $\frac{1}{2}$-yd. Ransom mixer at the elevation of the Edison Company's road, which is about 300 ft. above the base of the dam. From the mixer it was discharged into the chute 18 in. wide which was constructed at a slope of approximately 1 vertical to 2.5 horizontal. The stiffness of the mix was such that the concrete would just flow down this slope. The range in consistency is fairly well illustrated in Fig. 7 and 8, showing slump tests. The concrete was carried in this sloping chute to the approximate elevation of the top of the dam. For the lower 20 ft. of the dam, the concrete was carried to its final position through auxiliary chutes of still greater slope. For the portion of the dam above the 20-ft. level, the concrete was carried in the chutes to the elevation where it was to be used and was there carried in buggies to the point where it was deposited.

Fig. 9 is a photograph which shows the location of the mixing plant and the upper and the lower portions of the chute, described above. This shows the condition of the work when concreting had progressed to an elevation of 15 ft.

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25. Construction Joints. There were no vertical construction joints in the dam. Horizontal joints were necessitated by the fact that it was not practicable to pour the concrete continuously from the bottom to the top of the dam. These horizontal construction joints were placed at vertical intervals of about 5 ft. and were located about 6 in. below the even 5-ft. contours. This location for the construction joints was determined by the fact that large numbers of strain-gage, telemeter, and clinometer stations were located at the 5-ft. contours, and it would be undesirable to have them intersected or interfered with by construction joints. This location of the joints also permitted covering the instruments or inserts shortly after the beginning of the day's pouring, and left them free from disturbance while placing the rest of the concrete.

In general, about four days elapsed between pourings. During this time the surface concrete had generally become quite dry. In preparation for making a construction joint, the surface of the concrete of the previous pouring was removed to a depth of about one inch by means of sharp-pointed tools operated with an air hammer. The loosened material was then washed away with water and the excess water blown off by an air hose. This operation was performed in the evening when the next concrete was to be placed on the following morning. At this time all form work

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for the next pouring had been completed and all instruments at the stations except the telemeter cartridges had been placed.

The surface so prepared was clean, rough, and composed of sound concrete. After the concreting operations began on the following morning, the surface had usually dried to the extent that additional moisture appeared to be needed. This was furnished by spraying water upon the surface in sufficient quantities to make it moist, but standing water was avoided as far as possible. The first two batches from the mixer were of mortar. On the top of the mortar the concrete was deposited to a depth of 15 in., and as the concrete pushed along from one end to the other, the mortar was crowded ahead of it, furnishing a mortar bed all along the construction joint. By the pushing along of the concrete and mortar any excess of water (if by chance any remained), was crowded out of the depressions in the old concrete. This procedure was carried out carefully and appeared very effective in producing good construction joints.

A lift of 15 in. of concrete was thus placed from end to end of the dam. The next lift was started at such time that at all points the concrete would have at least an hour for some set to take place between lifts. In the second pouring from the 3.2 to 9.5-ft. elevation, an exception to this procedure was made. Less time was allowed between lifts and it showed up in a slight bulging of forms. This bulging

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appears in the greater divergence, shown in Table 3, between designed and measured thicknesses of the dam for the second, than for any other pouring. Even there the maximum divergence was only 0.03 ft.

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26. Forms and Form Removal. The form ribs were horizontal and 5 ft. apart. Studs 2 by 6 in. spaced 18 in. apart, spanned vertically from rib to rib, and 1-in. ship-lagging spanned the studs horizontally. On the upstream side the studs were 16 ft. long, hence were continuous across supproting ribs. On the downstream face the forms were built up as the construction progressed, that is, 5 ft. at a time, hence the studs were simple beams from horizontal rib to rib. During the pouring from elevation 35 to elevation 40 ft., measurements of the bulging of the forms were taken in several places from a point on the lagging to a strip nailed to the ribs above and below the point. All the points were midway between ribs and approximately midway between studs, hence the deflections measured were the sums of stud and lagging deflections relative to the ribs.

Table 4 shows the effect of the continuity of the upstream studs in keeping down the deflection on the upstream below those on the downstream face.

From these results it appears that the maximum variation from the nominal thickness is probably about $\frac{1}{4}$ in. greater than that shown in Table 3.

During the period of construction, that is, up to June 4, the forms were left in place four weeks after the pouring of the concrete. At that time a change in the program

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was made in order to hasten the completion of the construction operations, and all forms which had been in place two weeks or more were removed. From then on, forms were removed whenever they had been in place two weeks, thus completing the removal of forms on June 18.

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27. Control Specimens. The control specimens for the compressive strength of concrete were made regularly during the concreting operations. The results of the tests of these specimens are given in Table 5. These specimens were 6 by 12-in. cylinders and were selected as follows: On each day of placing concrete 24 cylinders were made. Eight cylinders were taken from each of the first three 15-in. lifts of concrete. From the fourth (and last) lift, no specimens were taken. Of the eight specimens taken from any lift, the first four were from one batch and the other four were from another batch. These eight specimens were numbered consecutively, beginning with the number following a multiple of eight. For example, (see Table 5) for the third lift placed on April 30, specimens 89 to 96 were taken. Specimens 89 to 92 were made from one batch and specimens 93 to 96 were made from another batch. The specimens having odd numbers were stored in damp sand at a temperature of about 70 deg. F in a curing room in the adit to water tunnel No. 3 of the Southern California Edison Company. The temperature in this adit was very nearly constant, but it was necessary to apply artificial heat to raise the temperature high enough for proper curing of the specimens. The temperature at all times was recorded with an autographic thermometer. The specimens having even numbers were stored at the dam under conditions as nearly like those of the curing of the dam as appeared feasible. Of these eight specimens, the first two were tested at seven days, the second two at twenty-eight days, the third two after three months, and the remainder are being held for later tests.

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27. (Cont.)

Some exceptions to the above procedure has been made due to the fact that certain specimens were sent to the university of California at Berkeley for the determination of the modulus of elasticity. Details are given in a later paragraph.

During the construction of the dam a beam 12 in. square and 8 ft. long was made for determination of the modulus of elasticity. A known load was applied to the beam. The deflection due to the applied load was measured and from the relation between the load and the deflection, the modulus of elasticity was computed. Eight control culinders were made with the beam, four of which were cured at the site of the dam along with the other control cylinders taken from the dam. These cylinders were No. 281 to 288, inclusive, Table 5. A hose connected to the same source of water supply as that which furnished water for curing the dam, was carried to the test beam so that when water was turned upon the dam it was turned upon the beam also.

On June 16, twelve days after the completion of the dam, samples of the cement used in the greater part of the construction of the dam were sealed for future use. Two sacks of cement were mixed together and from the mixture seventeen 2-qt. pails were filled and sealed thoroughly with solder. At that time the cement had been in the construction warehouse

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since about October 12, 1925, or approximately eight months. It was already about four months old at the time of placing in the warehouse. These samples of cement were sealed for the purpose of future study in case occasion for such study should arise.

Eight sacks of the cement used in the construction of the dam were shipped to the University of California at Berkeley, California, for use in the carrying out of the tests of various kinds on concrete made with similar materials and of the same proportions as those which went into the test dam. This series of tests was planned in May, 1926, but owing to lack of finances for carrying out the tests, the program was abandoned until later in the summer. This cement was kept in a dry place all summer and shipped to the University of California on September 7, arriving at Berkeley about September 17.

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28. Proportions and Strength of Concrete. Proportions and water-cement ratio for the concrete on the different days of pouring are shown in Tables 5 and 6. Table 5 gives also the strength and slump of the concrete as determined from the test specimens. The proportions given in this table are those determined by weighings made on one batch during each day of pouring. The results of these weighings are shown in Table 6.

It will be noted that except for the first and the last two days pouring, the proportions were approximately 1:3:2. Within the same period the water-cement ratio varied from 0.87 to 1.03. At the beginning of the construction the proportions were aimed to give a 28-day strength of 1800 lb. per sq. in. Before the time came for the second pouring it was found from tests of 6 by 12-in. cylinder specimens that the proportions used on the first day gave a strength of only about 1200 lb. per sq. in. Accordingly, the proportions were changed and the mix decided upon was used until at the 54-ft. elevation the supply of cement became exhausted, necessitating a change in cement.

Fig. 10, prepared from the data of Table 5, shows the strength of the curing-room specimens distinct from those cured at the dam site for the three different ages, and for the various heights from which they were taken. It shows also the average of the strengths for all ages. The strength

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of the specimens from the first pouring was markedly lower than from any other pouring for each age. This is evidently due to the fact that a leaner mix was used there than in all the portions above. For the most part, the strengths were quite uniform with the exception of the lowest specimens and those from heights of 52 ft., 56 ft., and 59 ft. The lack of uniformity in the specimens near the top of the dam is probably due in some manner to the change in cement which was made at about the 55-ft. elevation. The strengths for the 52-ft. elevation represent the last of the cement in the bin, and it is not unlikely that it consisted largely of cement from the outer portions of the pile where the greatest deterioration should be expected with a correspondingly low strength. The strengths for the 56 and 59-ft. elevations represent the new cement, and although an effort was made to change the mix in such a way as to hold the strength constant, it seems evident that this effort was not successful.

A marked similarity of form is to be noted in all the graphs of this figure, indicating that the same kind of variation ran through all the sets of specimens, and showing that the variations represent actual variations in the strength of the concrete in the structure, and not merely variations in the manipulation of the laboratory specimens.

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The 28-day strengths show an average a little more than 10 per cent greater than the designed strength of 1800 lb. per sq. in. With the exception of the extreme top and the extreme bottom, at which places the cause for variations in strength is fairly well understood, the uniformity of the strength seems to be very good.

By averaging the strengths for all ages, as shown in Fig. 10, the effect of accidental variations is reduced to a minimum. Therefore, Fig. 10 indicates best the degree of uniformity attained.

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29. Thickness and Alignment of Dam at Various Heights. The measured thicknesses of the dam at various elevations and the elevations of the construction joints are shown in Table 3.

The next day after each pouring of concrete, except the first, the elevation of the top of the concrete was determined accurately at each even 10-ft. station along the dam, and the distance between upstream and downstream forms was measured at these stations. These measurements were taken at the 10-ft. elevation and at even 5-ft. contours from there to the top of the dam. These, it is believed, may fairly be taken as the thickness of the concrete at these contours, though the concrete was generally from 5 to 10 in. below the contour on which the thickness was measured.

For the first pouring, the thickness was determined at the surface of the concrete. The exact elevation at the points at which these thicknesses were measured was not determined, and, due to the taper at this elevation, slight variations in thickness would accompany the variation in height. This is sufficient to account for the variations in thickness at the average elevation of 3.2 ft., shown in Table 3.

The variation from the designed thickness shown in Table 3 is generally not greater than $\frac{1}{8}$ in. In one instance (at the 10-ft. elevation) the thickness was $\frac{3}{8}$ in. greater

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than the designed thickness. This was the greatest variation found anywhere in the dam. In that pouring the height of the concrete was increased approximately 6 ft. in one day, and this may account for the greater variation in thickness at the 10-ft. elevation than elsewhere, since less time than usual elapsed between the pouring of the successive 15-in. lifts.

It is difficult to determine the movements of the dam at each elevation, beginning with the time immediately after the pouring of the concrete. Theodolite readings were taken on the morning after each pouring of the concrete. These readings were for the purpose of comparing the location of the upstream face of the dam with the position required by the design. The position of the downstream face was determined by measuring the thickness at the positions at which the location of the upstream face was determined. Theodolite readings gave no information on the amount of deflection after the concrete was placed. Generally, the upstream form was placed so that at the top of a new lift it was about $\frac{1}{8}$ in. downstream from the position called for by the design, while the downstream form was placed as specified by the design. If the theodolite readings showed the upstream face to be in the position called for by the design, it indicated a spreading of the forms.

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The first clinometer readings were taken when the dam was 15 ft. high. Thereafter the first reading on any 5-ft. lift of concrete was taken on the morning after the lift in question had been poured. However, since the clinometer brackets were generally about 4 ft. below the construction joint, the highest readings at this elevation represent, principally, deflection of the portion of the dam below the top lift.

It was, therefore, generally five days or more after a given lift of concrete was poured before the first reading was available which would give information on the deflection at that elevation. Before these five days had elapsed, the chemical heat of the concrete had been developed and disappeared, and the clinometer readings would be of no value in interpreting the effect of the chemical heat.

A study has been made to determine as nearly as possible from the clinometer readings the actual position of successive points on the downstream face of the dam during the construction period. The clinometer readings themselves were of use only for determining deflections of points defined by the clinometer holes in the brackets which were at fixed distances from the face of the dam. In order to determine the positions of points on the surface of the concrete, the distances from the concrete to the hole and from the hole to the ball were measured on the brackets with a

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scale. The dimension F was determined by means of the clinometer. The clinometer brackets were placed so that the center of the ball for each bracket fell on an even 5-ft. elevation. Calling the elevation of the center of the ball the station, the horizontal position of each station at any elevation H relative to the highest station was computed directly.

The positions of the various points on the downstream face of the dam for the various clinometer lines on June 5, with the construction completed, are shown in Fig. 11. At this time readings were taken simultaneously with the theodolite and the clinometer, on clinometer lines 2, 4, and 6. Simultaneous readings were not taken on clinometer lines 3 and 5 or on any other previous date on lines 2, 4, and 6. The position for the station at the top on lines 2, 4, and 6, was therefore taken as the theodolite reading, and the measured thickness of the dam, indicated that it should be. The points on the graph in Fig. 11 below the top show the variation of the positions of the stations from the position indicated by the design. In the absence of theodolite readings for lines 3 and 5, the upper points for those lines were assumed to have deflected from the correct position indicated by the design the same amount as was found for lines 2, 4, and 6, that is, 0.17 in.

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In Fig. 11 the movements are shown for successive days at various elevations. It will be seen that this offers no reason for thinking that there was an appreciable deflection of the dam, either upstream or downstream, during the construction period, except as changes in temperature may have caused it to go either way temporarily.

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30. Curing of Concrete. To assist in retaining the moisture in the concrete, forms were left in place for a period following the concreting from a minimum of 14 days to a maximum of 32 days. For the lower 30 ft. forms were wet with a hose three times a day in an effort to keep them damp. In spite of these precautions, a separation of the dam from the foundation at an elevation of about 12 ft. was observed. In the hope that the use of more water would prevent shrinkage which seemed to be responsible for this separation, more water was supplied through pipes perforated at intervals of about 18 in. One pipe was carried along the upstream and another along the downstream face for the full length of the dam at approximately the top of the concrete already in place. As the height of the concrete was raised, the height of the pipe was raised by the same amount. The water was then kept flowing on the forms all night and at such times during the day as did not interfere with construction operations and taking of instrument readings on the dam. This pipe was first put into operation at the 35-ft. elevation. After completion of the concreting it was placed on the top of the dam and the water was kept flowing whenever it did not interfere with the work, except that in the latter part of june, the water was discontinued for about 7 days in order to determine the effect on the dam of a certain amount of drying.

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III. GENERAL METHOD OF TAKING MEASUREMENTS.

31. Purpose in Testing. The purpose of the test was mainly to determine the distribution of the load, and all measurements were aimed as directly as possible at giving information on this subject. In addition, some information on shrinkage and temperature effects was secured. All of the measurements taken may be grouped under four headings: deflections, strains*, change in width of cracks, and change in temperature.

* Wherever in this report the word "strain" is used, it denotes a change of length per unit of length. The expression "temperature strain" where used refers to expansion or contraction due to temperature change. The expression "total strain" denotes the total change of length within the entire gage length or whatever length is under consideration. Wherever the word "stress" is used, it designates intensity per unit of area of the internal force. This usage is consistent with the recommendations of Committee E-1 of the American Society for Testing Materials. See Proceedings A. S. T. M., Vol. 23, part 1, p. 937 (1923). Also Vol. 26, part 1, p. 502 (1926).

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32. General Procedure. During the construction, accurate observations were made to determine changes of temperature which accompanied the setting of the cement and the corresponding strains in the concrete. Deflections of the dam were observed also. During the subsequent curing period, measurements were taken to determine the effect of changes of temperature and moisture on the deflections, and the strains. These measurements were taken at various times during the day. During their progress it became evident that temperature effects during the day were so great that the agreement of readings taken even a few minutes apart was affected by the condition of shadow or sunshine at the station under consideration. This made it evident that for subsequent tests it would be necessary to take readings at such a time that a uniform temperatute would prevail over the entire dam and that comparatively little change would take place during the entire time required for making the test. Approximately such conditions prevailed at night and at no other time.

The wiring and other arrangements for the lighting system around the dam were completed on June 22, 1926. From June 24 on until the conclusion of the testing program in September, 1926, practically all observations on the dam were taken at night or early in the morning before the sun

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had risen high enough to shine on any part of the dam. The readings taken on June 24 were to serve as initial readings for the study of temperature effects. Additional sets of readings without water in the reservoir were taken on the nights of June 30 and July 3. The load testing began on July 12 and was completed on September 23. Generally the observations occupied from about 12 p. m. to 7 or 8 a.m. The temperature during this period was very nearly constant as will be seen from Fig. 46.

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IV. INSTRUMENTS.

41. Strain Gage The strain gage used is a variation from the Berry strain gage. Before deciding on the type of strain gage and the gage length to be employed, a study of strain gages was made. It might be expected that the use of the 20-in. gage length would give greater accuracy than that of the 10-in. gage length. The total strains to be measured in the former case would generally be greater than those of the latter, and if the total errors are the same in both cases, the percentage of error would be correspondingly smaller with a longer gage length. It was found, however, that the increase of inaccuracy due to the greater difficulty of handling an instrument of greater gage length, was sufficient to over-balance the advantage of the greater gage length. For this reason a 10-in. gage length was decided upon. Fig. 12 is a sketch of the instrument adopted. The principle upon which it operates is that of a parallelogram with one leg of the instrument attached to one corner of the parallelogram and the other leg attached to the diagonally opposite corner. The two sides of the parallelogram are connected by short leaf springs. These springs are flexible enough to permit the necessary amount of movement and to serve as hinges between the two sides of the parallelogram. The idea of this type of gage was suggested by

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H. L. Whittemore of the Bureau of Standards. The design of the instrument, shown in Fig. 12, was worked up by G. S. Binckley, a member of the Sub-Committee on Test Dam.

The error of a reading with the strain gage may be said to be made up of the error inherent in the micrometer gage mounted upon it and the error of observation with the strain gage as a whole. Owing to the difficulty of manipulating the strain gage so as to duplicate readings on any gage line, the latter error is more than merely that of reading the dial on the gage.

In order to determine the error inherent in the Ames gage which formed the micrometer of the strain gage, an Ames gage graduated to ten-thousandths of an inch was calibrated at the Bureau of Standards. In this calibration it was found that there were errors at certain points in the travel of the instrument which were larger than the errors at other points. The calibration curves for selected portions of the range are shown in Fig. 13 and Fig. 14.

It will be seen that the curves of Fig. 13 and 14, showing the total error, are not drawn through the observed points, but have been smoothed out by connecting averages for pairs of consecutive points. This is justified on the ground that in calibrating the instrument, the readings were taken to the nearest one hundred-thousandth inch, whereas, the gage is graduated only to ten-thousandths

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of an inch. Part of the error must be in estimating the reading and this is allowed for by drawing the curve through the averages for all pairs of consecutive values. The method used for correcting the error of reading the Ames gage is arbitrary, but it seems to give reasonable results. The use of individual values instead of the modified curve would give a maximum percentage of error for Fig. 13 almost four times as great, and for Fig. 14 about twice as great, as the maximum errors they show.

As a basis for determining percentage of error, average and maximum values of strain are here introduced. The strain of 0.00005 is about the average value of the strain observed during the test, and the largest strains found were not often greater than 0.0004. The total strains in 10 in. for the two cases would be ten times the above values. The calibrated range shown in Fig. 13 and Fig. 14 for each figure is about twenty times the average strain and two and one-half times the maximum. It seems fair, then, to assume that the calibrated range covers the largest errors likely to be found in the Ames gage.

The percentage of error for a strain of 0.00005 at various positions in the range of the gage is shown in Fig. 13 and Fig. 14. For strains larger than 0.00005 , the

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percentage of error would, in all cases, be smaller than the error shown. For strains smaller than 0.000050, the percentage error may be larger than that shown, but it is not likely to be much larger. As shown in Fig. 13 and 14, the maximum error for a strain of 0.000050 was about 10 per cent. For the strain of 0.00040, the maximum error from Fig. 13 and 14 is found to be about 5 per cent.

In order to determine the portion of the error which is due to inability to repeat readings on a given gage line, twenty gage lines were selected for a study of the instrument. The readings used were taken in the course of carrying out Tests No. 11 and No. 12. Each of three observers took two readings on each gage line for each test and the probable error of the mean of each of the two readings was computed. The probable error of the mean of two observations is .954v, where v is the variation of either reading from the average. This is so close to the value v itself that the constant of .954 has been dropped and the residuals v have been used as the probable error.

The probable errors so obtained have been arranged in the order of their magnitude and plotted for each of the three observers for Tests No. 11 and 12 in Fig. 15.

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It will be seen from Fig. 15 that the range of errors is about the same for Test No. 11 as for Test No. 12 and that the errors for the three observers lie reasonably close together.

In the testing procedure the maximum permissible variation between any two readings on a single line was fixed at 0.0001 in. If any two readings differed from each other by more than this amount, additional readings were taken until two consecutive readings agreed within 0.0001 in. The variation of either of the two readings from the average could not then be greater than 0.00005 in. Dividing by 10 (the gage length) to reduce to strain, it will be seen that no error in strain could be found in Fig. 15 greater than 0.000005. Evidently, the number of gage lines selected for this study (twenty) is just about sufficient to include the maximum error of 0.0001, since this error is not repeated in these figures more often than any other single error.

Taking the maximum error in strain as five millionths (0.000005) this is found to be 10 per cent of the average strain of fifty millionths, and one and one-fourth per cent of the maximum strain of four hundred millionths.

In the above paragraphs, the maximum error of the Ames gage and the maximum error of observation with the strain gage have each been found to be 10 per cent of the

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average strain of fifty millionths. The combined error may be taken as the square root of the sum of the squares of these errors, or 14 per cent of the average strain. Based on the maximum strain, the maximum errors of the Ames gage and of reading with the strain gage are approximately 5 per cent and 1.25 per cent respectively. Taking the combined error again as the square root of the sum of the squares of these errors, gives approximately 5.1 per cent of the maximum strain.

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42. Reference Standard for Strain Gage. Although the strain gages were designed to compensate for change of length due to change of temperature, reference standards were used as a means of detecting and correcting for other changes in the instrument. Two of the reference standards were invar steel bars, 1 in. square, having a total length slightly greater than the 10-in. gage length. A third standard bar was constructed with invar steel and brass, and the parts combined in such a way that the expansion or contraction of one metal was designed to compensate for the expansion or the contraction of the other. An approximate determination of the coefficient of expansion of the standard bars was made. With standard bars and strain gages at a temperature of 21.7 deg. C., observations were taken on each standard with each of the four strain gages. The temperature of the standard bars was then changed to 48.9 deg. C. and observations were again taken on all standards with each of the four strain gages, the latter still having the former temperature of 21.7 deg. The difference of temperature of the standard was, therefore, 27.2 deg. C.

The total expansion measured for standard bar No. 1 was 0. 00051 in. in a total length of 10 in., or 0.00000187 in. per in. per deg. C. For standard bar No. 2, the corresponding total expansion was also 0.00051 in. and the expansion per in. per deg. C., was as above, 0.00000187 in.

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The compensating standard bar, on the other hand, gives for this increase of 27.2 deg. C. in temperature, a shortening of 0.00022 in. This corresponds to a negative coefficient expansion of 0.00000081 in. per in. per deg. C. Evidently, too much compensation had been provided, so that the error in the compensating standard bar was about one-half that in the plain invar steel standard bar. The invar steel standard bar had a coefficient of expansion of 0.0000019 in. per in. per deg. C., or about one-fifth that of ordinary steel. Evidently, for most cases, the expansion would be small enough to be negligible in its effect on strain-gage observations.

In taking of strain-gage observations, duplicate readings on twelve gage lines generally intervened between two successive readings on the reference standard. The entire twelve readings were first taken and then in the same order duplicate readings were taken. This gave sufficient time between duplicate readings to prevent the observer from being prejudiced in any reading by the reading immediately preceeding it.

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43. Electric Telemeter. The electric telemeter is an instrument for measuring microscopic changes of length and temperature by means of varying electric resistances. By installing lead-wires the observations may be taken at any convenient distance from the telemeter cartridge The electric telemeter used was designed by Burton McCullom and O. S. Peters, of the Bureau of Standards. The principle by which the changes of length are measured is described in detail in Technologic Paper 247 of the Bureau of Standards. As stated by Mr. McCullom and Mr. Peters, the electric telemeter "depends upon the well-known fact that if a stack of carbon plates is held under pressure, change of pressure will be accompanied by a change of electric resistance and also a change of length of the stack, both of which are reversible, the stack of plates under change of pressure performing like an elastic body."

In this test that portion of the instrument which was embedded in the concrete has been termed the cartridge. The cartridge consisted of a stack of carbon plates enclosed in a cylindrical steel casing 6 in. long, and having end-bearing plates of steel in contact with the stack of carbon plates so that the strain in 6 in. of concrete was transmitted to the stack of carbon plates. See Fig. 16.

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Fig. 16 shows the details of this cartridge. In Technologic Paper 247 it has been shown that by using two stacks of carbon plates in the cartridge they can be so arranged that the direction of the pressure is opposite for the two stacks. The purpose of this arrangement is to rectify the curve of resistance with the change of length of the stack. This double stack of carbon discs was not used in the cartridge employed in this test and the resistance-strain curves were not straight lines. Fig. 17 gives typical resistance-strain diagrams for several of the cartridges.

There was no means of checking the telemeters against a reference standard during the carrying out of the test. Therefore dependence had to be placed upon the calibration curves after the cartridges had once been embedded in the concrete.

The resistance measured in a telemeter is a function of the sum of the distances between the carbon discs in a stack. These distances will be affected, not only by strains in the concrete, but also by expansion or contraction of the carbon discs themselves, or of the metal frame and capsule which carries the stack of discs. It is essential for other reasons that all moisture be kept out of the instrument, and temperature changes will be the principal, if not the only, cause of expansion and contraction. Therefore

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in order to make possible the interpretation of the strains observed in the telemeter, it is necessary to observe the temperatures within the cartridges, and, knowing the coefficient of expansion of the carbon discs and the metal frames, compute the corrections to be applied to the observed strains. The temperatures in each cartridge were determined from measured resistances of a coil of No. 40 enameled copper wire wound on a bakelite spool. The resistance of this wire at 20 deg. C. was specified to be 512 ohms. A change of one deg. C in the temperature of the wire, caused a change in resistance of two ohms. Such a device, known as a resistance thermometer, was contained within the cartridge of each telemeter. It was found that a rise or fall in temperature of 1 deg. C. corresponded to a change in length of the instrument of 0.000015 in. per in., and this value was used as a coefficient for making temperature corrections to the telemeter readings. (See Sec. 61.)

In all, 140 telemeter cartridges were embedded in the dam. Each cartridge had three lead-wires, one from the stack of carbon discs; one from the temperature coil, and one common to both stack and coil. All the leads were carried to a switchboard where there was a terminal for each lead. This switchboard, and the Wheatstone bridge used for measuring the resistances, are shown in Fig. 18.

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44. Radius Meter. The so-called radius meter used in this test is an instrument which measures the change in mid-ordinates of a 40-in. arc of the flexure curve of a horizontal rib of the arch. Its usefulness depends upon the fact that the bending moment in a member is inversely proportional to its radius of curvature. Expressed algebraically $M = \frac{EI}{r}$, where r is the radius of curvature, E is the modulus of elasticity, I is the moment of inertia, and M is the bending moment. If E, I, and r vary throughout the gage length, their average values will give the moment at the center of the gage length with only slight error. The instrument consisted of a 40-in. arm with contact points at either end and a gage measuring the change of mid-ordinates to ten thousandths of an inch, mounted at the center. The contact at one end consisted of a conical-shaped leg seated in a $\frac{1}{8}$-in. hole drilled into a steel plug which was screwed into a cast-iron socket set in the concrete. The contact at the other end consisted of a similar cone-shaped leg seated in a $\frac{1}{8}$-in. groove in a similar steel plug. The groove permitted the extension of the gage length relative to the instrument without affecting the measurement of the change of mid-ordinate. The contact point at the center consisted of a bearing of the plunger of the Ames gage on a flat steel surface. The plugs used for the end points of the stations were adjustable as to depth so that they could be set to

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give a reading within the range of the instrument. Fig. 19 is a photograph of the radius meter. Fig. 20, (a), (b), (c), (e) and (f), show the details of the inserts used with the radius meter.

The reference standard for the radius meter consisted of a heavy steel angle about 48 in. long with contact points arranged for the radius meter and spaced so that deflections may be read on either side. This steel angle was mounted on a concrete pedestal and at frequent intervals a reading was taken on each side of the steel angle. Any deflection of this standard would have opposite signs when read from opposite sides, therefore, the algebraic sum of the deflections should be zero and any resultant deflections appearing from the readings would indicate a change in the instrument itself.

Each series of radius-meter observations was begun with a set of readings on the reference standard. Then readings on 12 radius-meter stations were taken. This was followed by another set of readings on the reference standard, then by readings on the other 11 radius-meter stations, and finally by a third set of readings on the reference standard.

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45. Clinometer. The clinometer consisted of a vertical staff provided with first, a level bubble used to bring the axis of the staff into the vertical position; second, a means of bringing the lower end of the staff to the same position relative to the dam time after time, and, third, a micrometer at the top for measuring variation in distance from a fixed point on the dam to the axis of the vertical staff. The clinometer is 4 ft. 10 in. in length, and is used as shown in Fig. 21. The conical point at the lower end of the clinometer seats in a $\frac{1}{8}$-in. hole in the lower bracket shown there. The plunger of the Sterrett gage bears against a $\frac{1}{2}$-in. steel ball which is inserted in the end of the upper bracket. The vertical distance from the bottom bearing of the clinometer to the center of the $\frac{1}{2}$-in. steel ball below is 2 in. This gives a vertical distance of 5 ft. between stations. By means of the micrometer head the upper end of the clinometer is moved in a horizontal direction until the bubble iscentered in the level vial. The micrometer is then read to an estimated ten-thousandth of an inch. The instrument is then moved to the next span below, as shown in Fig. 21 (a) in dotted lines, and a similar reading is obtained. In this way readings are taken from the top to the bottom of the dam. Such a set of readings taken at any time may be referred to a similar set taken at a previous time and the differences at corresponding positions give the

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deflection in the corresponding 5 ft. By adding the deflections cumulatively from the bottom to the top, the total deflection at any point in the height of the dam is obtained.

If the instrument were always kept in the same adjustment, it would not matter whether the reading for the exact vertical position of its axis were known or not, since the differences would be the same regardless of the verticality of the axis. However, since the adjustment may not remain the same, it is necessary to provide a means of determining the reading which will bring the end of the plunger to a line vertically over the cone point at the bottom of the staff when the bubble is in its central position. Fig. 21 (b) illustrates the manner in which this is done. Two brackets the same distance apart vertically as the clinometer brackets on the test dam are provided. The steel ball is inserted in the upper clinometer bracket in such a way that readings may be taken on opposite sides of the ball. The instrument is placed as shown in solid lines. The bubble is brought to this central position and the reading of the clinometer is taken. The clinometer is then rotated 180 deg. to the position shown in dotted lines. The bubble is again leveled and a second reading of the clinometer taken. The average of the readings minus one-half the diameter of the ball is the reading at which the end of the plunger of

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the micrometer is vertically over the cone point at the bottom of the clinometer when the level bubble is centered. One-half the difference of the readings is the distance of the center of the ball from the vertical line through the center of the hole in the lower bracket.

In the process of using the clinometer, frequent readings on the reference standard, as shown in Fig. 21 (b), were taken. In the early part of the test it was not feasible to have the reference standard close to the test dam because of danger of damage to the standard during construction operations. For this reason a reference standard was constructed in the tunnel adit at the camp. While this standard was in use, a reading on it was taken before going down to the test dam. The next reading was taken after the completion of the observations at the test dam. As considerable time elapsed between the two readings, there was likely to be a considerable variation between the successive readings on the standard. As soon as the construction of the test dam was completed, a reference standard was placed on the rock of the south abutment at the 30-ft. level. Readings on the reference standard were thereafter taken at intervals during each series of observations frequently enough to eliminate all appreciable error due to variation in the adjustment of the clinometer.

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In order to obtain a measure of the accuracy of the readings taken with the clinometer, a study of probable errors for deflections on all stations was computed. These are shown in Fig. 22. Although the clinometer measures deflections, the deflections so measured have been used for the computation of strains.

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46. Level Bar. The level bar is an instrument which was designed to determine the change in inclination of the foundation rock and of the lower portions of the dam. A photograph of the instrument is shown in Fig. 23. It consists of a flat bar a little more than 10 in. long which forms the distance piece and on which is mounted, with a hinge at one end and a coil spring bearing at the other end, an auxiliary bar carrying a level vial. The vial is graduated to indicate 4.5 sec. change of inclination for each change of one division in the position of the bubble. To measure the change of inclination, a bracket attached to the distance piece carried a Sterrett micrometer whose plunger had its bearing on the end of the auxiliary bar. To the flat distance piece bar a conical pointed leg was attached at each end. In using the level bar the conical point at one end had its bearing on a circumference of the hole drilled by a No. 54 drill (.055 in diameter). The point of the leg at the other end of the instrument bore upon the edges of a groove of rectangular cross-section cut longitudinally with the gage length of the instrument.

To take observations with the level bar, the instrument is placed on the contact points of the station and the level bubble is brought to the center of the vial by turning the micrometer screw up or down. The coil spring always

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keeps the auxiliary bar in position against the lower end of th plunger of the gage. The difference of readings on the Sterret gage for different observations indicates the change of inclination between the observations. Since the pointed leg at eith end of the instrument may be used either in the drill hole or i the groove, the instrument may be reversed in direction for successive observations. This compensates for changes in the instrument, obviating the necessity of a reference bar.

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47. Resistance Micrometer. In addition to the determination of deflections by means of the clinometer, provisions were made for measuring the deflections relative to steel towers erected a few feet from the dam on the downstream side. The tower bases were placed just previous to the construction of the dam. The material for the towers was received during the period of construction of the dam. The towers were designed to be 5 ft. square and to be protected by a wooden sheath built 1 ft. away from the tower on all sides. For convenience of construction, the erection of the towers was postponed until after the completion of the construction of the dam. At the time of the completion of the dam, the finances of the committee did not permit the construction of the towers and by that time it had been found that the deflections as measured with the clinometer were more accurate than seemed reasonable to expect from the towers. Therefore, up to the present time, there probably has been little loss to the test due to the failure to erect the towers. It is possible that for continuing the test it may be desirable to erect the towers in order to make feasible the observation of deflections at times when it may not be safe to take readings with the clinometers.

An instrument for measuring the deflection relative to the steel towers was developed by Col. G. S. Binckley Though this instrument , the resistance micrometer, has not been used for measuring the deflections of the dam, it has been adapted to the under-water measurement of the change in width of a crack.

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The resistance micrometer is an instrument which consists essentially of a variable resistance with a sliding contact so arranged that the distance through which the sliding contact moves is equal to the deflection and approximately proportional to the change in resistance. The coil of resistance wire is wound around a core and each end of the wire is connected with an instrument for measuring the resistance. A circular contact slides along the resistance coil, as shown in Fig. 24. A third wire is carried from this circular contact to the resistance-measuring instrument. As the deflection increases, the contact moves away from one end of the coil toward the other end, increasing one resistance and decreasing the other. Since the resistance of each part of the entire coil can be measured at any time, it is possible to determine the proportionate part of the entire range which has been traversed by the contact at any time. Although the resistance in the lead-wire may vary with the temperature, the proportional effect will be nearly the same on all three resistances so that the deflections may be determined without correcting for temperature, length of lead-wire, etc. The length of lead-wire will thereby affect the proportionate resistances, but with wires of the size used in the test, this effect would be negligible.

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A calibration curve for the instrument is shown in Fig. 25. This calibration was made just before attaching the instruments to the dam. As with the telemeters, no reference standard could be used during the test and dependence had to be placed upon the calibration curve.

In adapting the instrument for the measurement in the change of crack width, the essential feature was its protection against leakage of water into the resistance coil and into the leads from the instrument to the Wheatstone bridge. The instrument operated satisfactorily for the purpose of measuring the crack width. (See Fig. 75.

After the completion of the load tests, the resistance micrometer was removed from the place of attachment at the base of the dam, and after a time it was found that water had penetrated the envelope in considerable quantity. It is possible that water had been inside the cover during the taking of the measurements, but it is not likely that it had been in contact with the resistance coil.

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V. LOCATION OF STATIONS FOR OBSERVATIONS.

51. Basis for Arrangement of Stations. The stations for observation of deflection and strains were laid out in such a way that curves of deflection and strain would be obtained for various vertical and horizontal sections. These curves did not necessarily give the maximum strains, but they furnished a convenient system for studying the data, and the strains and moments on sections selected may be compared directly with the analytical results for the same sections.

To adopt a system which involves only the sections giving the maximum strains would be next to impossible, but with the data secured it is feasible (theoretically at least) to determine the amount and direction of the maximum strains. This is true, both for the strains determined from the deflections and for those determined from the direct measurement of strains with the strain gage.

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52. Arrangement of Strain-Gage Stations. The number and arrangement of the strain-gage stations are shown in Fig. 26. The first one or two digits in any strain-gage station number indicates the elevation at which the station is located. Where a station involves more than one gage line, the letter A is appended as the designation for the vertical gage line and the letters B, C, and D for the gage lines making clockwise angles of 45, 90, and 135 degrees, respectively, with the vertical. The stations are arranged in vertical and horizontal lines all over the downstream face and above the 30-ft. elevation on the upstream face of the dam. The horizontal rows of stations are generally five feet apart. On the downstream face, alternate stations at even 10-ft. elevations in all vertical rows have four gage lines each, 45 deg. apart. These four-gage-line stations were designed to be used in determining the magnitude and direction of the maximum strains. They also were of use in determining the amount of the torsional moment.

In addition to these stations 5 ft. apart, vertically, a continuous row of stations 10 in. apart, was placed on the vertical center-line and on a horizontal line at the 30-ft. elevation.

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A few special strain-gage stations not included in the system described above were used for showing the movement of the dam relative to the abutments and for showing the variation in the width of the crack which occurred at the center-line in the upper portion of the dam. These are stations 1D to 4D on the downstream face; 1U to 3U on the upstream face, T1, T2, and E1 to E4 on the top of the dam at the ends, and 45C and 50C on the downstream face of the dam at elevations 45 to 50 ft. across the crack at the center-line. Most of these stations have three gage lines each, which form the three sides of an equilateral triangle. One corner of the triangle is in the rock and the other two are in the concrete. The three gage lines are designated as A, B, and C, of which gage lines C is entirely in the concrete and the others span from the concrete to the rock. The stations referred to are shown in Fig. 26 with the other strain-gage stations.

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53. Arrangement of Telemeter Stations. The numbers and locations of telemeter stations are shown in Fig. 27. Arabic numbers from 1 to 30, followed by initials showing the position and direction of measurement, identify the various telemeter cartridges. Of the initials which follow the Arabic numeral, the first indicates whether upstream, downstream, or central in the dam. These initials are U, D, and C, respectively. The second initial indicates the direction of measurement, whether longitudinal (extending in the direction of a horizontal tangent to the dam at the point under consideration) vertical, or diagonal. Initials representing these directions are L, V, and D, respectively. There are no diagonal telemeter cartridges on the downstream face. On the upstream face there were, in some cases, diagonal cartridges bisecting an angle made by vertical and longitudinal cartridges. Midway between the upstream and the downstream faces there were vertical and longitudinal cartridges, also diagonal cartridges in vertical radial planes, but there were no diagonal cartridges in a vertical longitudinal plane except at the upstream face. A diagonal cartridge with its upper end closer to the upstream than the downstream face is indicated by the initial D1, and a diagonal with its upper end closer to the downstream face is indicated by D2.

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It was the general plan to employ telemeter cartridges in the upstream face opposite each point where one was used in the downstream face. In general, this was followed out. It is now believed that better use would have been made of the cartridges available if, for the most part, they had been placed in the dam close to the connection with the foundation, in order to determine the boundary conditions.

Eight telemeters were placed at five stations in the foundation rock to measure temperatures and strains. Three of these were close to the upstream face of the dam and were embedded partly in the rock and partly in the dam. The other five were near the downstream face and were embedded entirely in the foundation rock. Fig. 27 shows the location and the manner of embedding these telemeters.

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54. Arrangement of Radius Meter Stations. The radius meter stations 40 inches in length cover the entire length of the dam at the 30-ft. elevation. It was originally planned to have 30 additional stations located as shown in Fig. 28, and the inserts for these stations were placed, but they were not used in the test. The reasons for their disuse were, first, the difficulty in the use of the radius meter, and, second, the effectiveness of other measurements which made the radius meter less necessary than otherwise would have been the case. Although the radius meter was difficult to use, it gave information that has been useful in the test.

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55. Arrangement of Clinometer Stations. The clinometer stations are arranged on vertical lines, 10, 30, and 50 ft. from the vertical center line on either side of it as shown in Fig. 26. Lines at 20 and 40 ft. from the center line were omitted, as it was expected that deflections on those lines would be measured with the resistance micrometer from the deflection towers. The stations were placed at elevations of zero, 5, 10, and 15 ft., etc., except on the lines 10 ft. either side of the vertical center line. Since the bottom of the dam is about 2.5 ft. higher at this point than at the vertical center line, the lowest observed deflection on these two lines was at an elevation of 7.5 ft. instead of 5 ft. This gives an overlapping of the two lower clinometer spans. All the other stations were placed at elevations which were a multiple of 5 ft. in order to obtain deflection curves for horizontal sections.

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56. Arrangement of Level Bar Stations. There were twelve stations for measurement of change of inclination with the level bar, of which five were on the foundation rock near the bottom of the dam, and seven were in the concrete of the dam near the bottom. There were no stations above the 20-ft. elevation. The numbers and the locations of these stations are shown in Fig. 28. Inclination of the foundation rock was observed upon I-beams embedded in the rock, and in Fig. 28 the locations of the stations are shown by the letter I. Inclinations of the dam were measured on clinometer brackets and the symbol used in Fig. 26 to represent the clinometer stations is used in Fig. 28 to designate these level bar stations.

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57. Arrangement of Stations for Resistance Micrometer. The locations designed for the measurement of deflections by means of the resistance micrometer are shown in Fig. 26. For reasons given in a previous paragraph, this micrometer has not been used in tests already made, except for the measurement of the width of a crack, as described in Sec. 83.

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58. Placing of Instruments and Inserts. Figs. 29 to 32 illustrate the appearance of the interior of the forms during the construction of the dam, and show how the instruments and inserts were held in place during construction.

Steel plates about $\frac{1}{16}$ inch thick were used as templates to hold in place, temporarily, the cast-iron inserts into which the strain-gage plugs were inserted. The templates used for the 4- and 8-point stations were 14 in. square. The templates used for continuous rows of gage lines were 3 in. wide and 44 in. long, thus permitting four complete 10-in. gage lengths per template. The inserts were drilled and tapped as shown, in Fig. 20 (f) to receive the cap screws and strain-gage plugs shown in Fig. 20 (d). The taper shank on the cap screw and on the plug fitted into the corresponding taper in the recess of the inserts. The cap screws were used to assemble the templates and inserts into units for placing in the concrete, as shown in Figs. 29 and 32. The taper in the insert assisted in drawing the insert to an accurate center. After the concrete had hardened, the cap screws and templates were removed and the gage-hole plugs were inserted for taking strain-gage observations. The taper shank on the plug assisted in centering the plug in the insert and in drawing it to a bearing. The large recess in the upper end of the insert furnishes access with socket wrench for

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inserting the plugs and permits the placing of a protecting cap over the end of the plug when tests are not in progress. The gage hole in the plug is drilled with a No. 55 drill. After drilling the gage hole, the plug was hardened and the hole was lapped inside to smooth the surface and remove all burr. Under a magnifying glass the edges of the hole appeared sharp.

This method of placing the plugs resulted in satisfactorily accurate gage lengths. The variation from station to station was generally within 0.03 in.; occasional variations were slightly greater.

Fig. 20 shows the cast-iron inserts which were set into the concrete to receive the steel strain-gage plugs. The plug is shown in Fig. 20, also the brass cap screw used to hold the inserts in place during the pouring and the hardening of the concrete.

The manner of placing and retaining the telemeter cartridges in the correct position before concreting is shown in Fig. 30 and 31. Most of the cartridges were placed near the surface so that as nearly as possible the axis of the cartridge lay parallel to and two in. from the inside face of the form. They were supported and held in place by twisted wire brackets tacked to the form, as shown in Fig. 30 and 31. Fig. 29 shows quite clearly the

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method of mounting the telemeter cartridges in the mortar blocks for use in the interior of the dam. The block was slightly less than 4 in. square and permitted each end of the telemeter to project from the block a little more than one inch. All movement of the cartridge is transmitted from the concrete through the end plates of the cartridge, and this arrangement leaves the ends free to receive the pressure. Only slight force is required to operate the parts of the telemeter, and the projections shown give sufficient bond between the end plate and the concrete to develop either tensile or compressive movements.

It will be seen in Fig. 30 that two short pieces of cast-iron pipe project upward from the surface of the concrete and that two ½-in. bars passing entirely through the mortar blocks enter these pipes and hold the block in position. After the block had been placed, it was held from moving horizontally by wedges placed in the pipe to fill up the space around the ½-in. bars. During concreting very close supervision was maintained to avoid displacement of the telemeters. This feature was more important with the telemeters than with other instruments because there was no means of determining their actual position after the concrete had been poured.

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For each radius meter gage line, three plugs were required, one at each end and one at the center. Those at the ends were of the same form as the strain-gage plugs, and used the same kind of inserts. On account of the small range of the gage used with the radius meter, it was necessary that one of the three plugs (the center one) be made adjustable as to depth so that the plugs could be set to give a reading within the range of the instrument. This required room for a lock-nut on the thread of the plug and necessitated a special insert to accompany this middle plug. The insert and the plug are shown in Fig. 20.

The templates used for holding the inserts for the radius-meter plugs in place during concreting were the same as one of the types used with the strain-gage inserts, that is, 3 by 44 in. by about $\frac{1}{16}$-in. Cap screws were used to attach the inserts to the templates. The templates were tacked to the forms in position before the concreting, as illustrated in Fig. 30 and 31. After the concrete had been placed and had hardened, the cap screws and templates were removed and the plugs were inserted and adjusted.

The clinometer brackets were steel bars 0.5 by 2.5 in. in cross-section, extending about 12 inches into the concrete. The length of the projecting portion, 11

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illustrated in Fig. 21, depended upon the elevation in the dam. It was just long enough that the extreme point of the ball at the end of one bar stood, as nearly as possible, vertically above the bearing hole of the bar next below it in which the conical point of the clinometer staff rested. The ball on which the micrometer plunger bore is a hardened steel ball, $\frac{3}{8}$-in. in diameter. It was inserted in a recess in the end of the bar and was retained by peening around the circumference of the hole. The lower bar is $\frac{3}{8}$-in. in diameter by $\frac{1}{2}$-in. deep.

The clinometer brackets were held in place in the dam during construction by means of rigid frames made up from small structural steel channels, placed back to back. The lower end of the frame was attached to a clinometer bracket which had been placed during the previous pouring. Into the upper end of the frame (5 ft. above its lower end) the bracket to be placed was clamped into the position it was to occupy in the dam. This frame held the upper bracket at the proper elevation during the succeeding pouring of the concrete. It was necessary to maintain continuous inspection during concreting in order to prevent its displacement laterally. The frames were left in position until after the concrete had thoroughly hardened. Then the frame was removed and raised 5 ft.

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engaging the new clinometer bracket at the bottom of the frame and in a like manner holding the one above in position for the next pouring of the concrete.

Changes in inclination of foundation rock were measured on the upper surface of 6-in. I-beams which were embedded in the foundation. For placing the I-beams, trenches in the rock were prepared which extended across the foundation along a radius of the dam. The trenches were about 6 in. wide and 8 in. deep. Within the dam about one foot from either face, a small area in the bottom of the trench was left slightly higher than the rest of the bottom to afford definite bearing places for the I-beam. The I-beam was placed in the trench. Anchor bolts were set in the rock on each side of the I-beam opposite the bearing points. The strap across the I-beams at each of the two bearings was drawn down as firmly as possible by means of nuts on the anchor bolts. Rather thin cement mortar was then placed in the trench and worked around the I-beam in the effort to secure a uniform bearing. One end of the I-beam extended to within a few inches of the upstream face of the dam, and the other end projected about 12 in. beyond the downstream face of the dam, entirely free from contact with the rock. The level bar station occupied about 10 in. of the 12 in.

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projection. The station consisted of a small hole drilled for the bearing of the pointed leg at one end of the instrument and a groove for the bearing of the pointed leg at the other end.

The hole was drilled on the center line of the bar with a No. 54 drill and generally near the face of the dam. The groove was rectangular in cross-section, about $\frac{1}{16}$-in. wide and $\frac{1}{2}$-in. long and extended in a longitudinal direction of the I-beam. The groove and the hole were both about $\frac{1}{8}$-in. deep. For measuring the change of inclination of the dam above the foundation with the level bar, the projecting ends of the clinometer brackets were used for placing the level bar stations as described above. It had been planned to use the level bar data in connection with the clinometer data to determine strains at the bottom of the dam, but it was not feasible to do so. This is more fully discussed in Sec. 87.

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VI. RECORDS OF TEST DATA

61. Specimens of Original and Reduced Data. The form in which the data were taken is indicated in Fig. 33, 34, and 35. To a considerable extent, the sheets are self-explanatory. For the strain-gage, clinometer, level bar, and the radius meter, all readings are duplicated and the average of the two readings was taken as one observation. From the original data sheet, the average reading was copied to a reduction sheet, samples of which are shown in Fig. 36 to 38 for the strain gage, clinometer, and level bar. For the strain gage, clinometer, and radius meter, an arbitrary zero reading on the reference standard was assumed, and the corrections shown in the reduction sheets of Fig. 36 and 37 are the differences between the actual reading on the reference bar and the assumed zero reading. The corrected readings are the algebraic sum of the average reading transferred from the original data and the correction. The corrected difference is the difference between the corrected reading for the series under consideration and the corrected reading for the series with which the comparison is being made. For the strain gage the elongation or shortening is in hundred-thousandths of an inch in 10 inches.

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To obtain the strain (that is, the elongation or shortening in inches per inch) it is necessary to divide the corrected difference by 10, since the gage length was 10 inches. Therefore, the strain is given in millionths.

For the clinometer the average readings and the corrections are obtained in a manner similar to that for the strain gage. Likewise the corrected readings are obtained by adding algebraically the average reading and the correction. The difference between the corrected reading for any series and the corrected reading for the series with which comparison is being made, gives the deflection at the point under consideration relative to a point 5 ft. below it. Therefore, the deflection at any point is obtained by adding to the difference for that point the sum of the differences for all the points below it in the same vertical radial plane. On the reduction sheets, the stations in the same vertical radial plane are so arranged that the cumulative sum from left to right gives the total deflection for the corresponding stations.

For the radius meter, a method of reduction similar to that of the strain gages was used. Here the data for each station are proportional to strain, and to obtain deflections from them a double instead of a single integration (cumulative addition) would be necessary.

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Telemeter data were recorded and reduced in a manner somewhat different from other data. Since temperature corrections for parts of the instrument, the resistance of the stack, and the resistance of the temperature coils are factors in the reduction of the data, a different kind of form was necessary.

Fig. 39 gives a sample of the original and reduced data for test No. 12.

For each telemeter cartridge a calibration curve was prepared before the instruments were embedded in the concrete. A specimen curve is given in Fig. 17. On the form used there is a column for each telemeter row, for each series of observations. The resistance of the carbon stack is recorded to the nearest tenth ohm as "stack ohms." The resistance of the copper coil is recorded to the nearest half ohm opposite the "coil ohms."

The resistance under "coil ohms" is used to determine the temperature by means of the constant furnished with the calibration curves for the telemeters.

The indicated strain is the value of the strain which in the calibration curve corresponds to the resistance "stack ohms." The temperature correction is that portion of the change of length recorded by the stack which is due to change of temperature of the carbon and the mountings. This correction has been found to be

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proportional to the temperature change and the same constant was applied to all of the telemeters. The value found to be most nearly consistent for all of the data was 0.000016 inches per inch per degree C. in the 6-inch cartridges used. The corrected strain is found by adding the temperature correction (a strain) to the indicated strain with the proper sign. The temperature corrections were referred to 20 deg. C., as a zero correction. The "corrected strain" shown on the data sheet represents an arbitrary value for the study of strain and is without meaning until compared with similar "corrected strain" for the same telemeter under other conditions, that is, the net strain is found by subtracting the "corrected strain" for one series of readings from the corrected strain for the series with which it is to be compared.

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62. Filing of Data. The original data and the reduced data will be filed with the Engineering Foundation and will be available for reference to any who wish to examine them.

Table 7 outlines the method of filing the data with the Engineering Foundation for purposes of reference. Items 0, 1, 3, etc., of data, whether in the form of tabulations or of graphs, are bound in suitable form and are indexed according to the numbers given in Table 7.

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VII. BEHAVIOR OF DAM DURING CONSTRUCTION AND CURING PERIOD.

71. Temperature During Hardening and Curing of Concrete. On April 18, 1926, before any concreting had been done on the dam, readings were taken on telemeters 3U and 3D before they had been grouted into the foundation rock. They were grouted in on the evening of April 18 and the next readings were taken just before concreting began on April 19. From then on frequent readings were taken. All the temperatures for telemeters 3U and 3D are shown in Fig. 40. The difference of about 4 deg. C. between the first two readings in Fig. 40 probably represents partly the daily variation in temperature and partly the difference between air temperature (before grouting) and the temperature of the rock and grout. After the concrete had been placed, the temperature increased rapidly for about two days and then decreased slowly. Air temperatures for the same time are shown in Fig. 45. The data for Fig. 45 were taken from daily temperature graphs recorded by an autographic thermometer stationed at the dam site. Previous to the completion of the dam, the autographic thermometer was kept in a partly enclosed hoist house about 100 ft. downstream from the dam and at the approximate elevation of the top of the dam. On June 27, after the removal of the forms from the dam, the thermometer was moved to a position about 5 ft. downstream from the dam

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at an elevation of about 35 feet. The two upper graphs of Fig. 45 give the maximum and the minimum daily temperatures. The difference between the maximum and the minimum temperatures was generally about 15 deg. during the construction and the curing period and decreased to about 10 deg. during the close of the testing period. The lower curve shows the air temperatures for the same times of day that the telemeter observations in Fig. 40 to 44 were taken.

Fig. 41 shows the temperature rise at station 6 which is on the center line of the dam at an elevation of 4 ft. The thickness of the dam at this station is about 6 ft. 2 in. At station 6, the temperature rose to a maximum of 47 deg. C. in the center of the dam in about two days. From then on the temperature gradually decreased, reaching a steady temperature of about 17 deg. after about three weeks. No other station had so great a temperature rise as this, though others were nearly as great. Near the surface of the concrete the temperature rose to a maximum of about 35 deg. C. within one day and from then on decreased at about the same rate as at the center, reaching a steady temperature of about 11 deg. after two weeks time.

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Station 6 is taken to be typical of the behavior of the lower (that is, the thicker) portion of the dam, and all the data for that station are included in Fig. 41. In Fig. 42, 43, and 44 envelopes of the temperatures in the concrete at elevations 10, 20, 40, 50 and 60 ft. are given. In general, temperatures at the center of the dam were higher than those near the surface during the period in which the temperature level was affected by chemical heat. From that time on, the highest temperature may have occurred either at the surface or in the interior of the dam, depending on whether the air temperature at the time was higher or lower than the temperature in the interior. This is shown in Fig. 41. The same statement applies to Fig. 42 although it is not apparent on inspection, since only the highest and the lowest temperatures on any date are given in this figure. At elevation 30 and above, there were no telemeters in the center of the dam and consequently, all the temperatures shown are for positions near the surface of the concrete. The average temperatures at various elevations from June 6 to September 22 are shown in Fig. 46.

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72. Strains and Relation Between Strain and Temperature. The strains shown by the telemeters of stations 1 to 5, with no water in the reservoir, are shown in Fig. 47. Those for station 6 are shown in Fig. 48. The strains for representative strain-gage stations are shown in Fig. 49. Fig. 47 and 48 show that in some places an increase in temperature was accompanied by an increase in length and in others by a decrease in length. In the figures it is not easy to see any direct relation between cause and effect.

Fig. 50 shows the relation between temperature change and strains observed at elevations 30, 40, and 50, and includes also the relation of the deflection to the strains. The ordinates of the points on the curves marked "computed from temperature changes", represent the strains computed by multiplying the temperature differences by the coefficient of thermal expansion, found to be 0.0000077 in.per in per deg. C. Thus, the ordinates, though shown as strains, are proportional to temperature changes. The strains computed from the deflections were obtained by solving for the average strain, $\frac{\Delta L}{L}$, the approximate formula:

$$D = \frac{3}{16}\frac{L}{h}\Delta L,$$

. . . . . .

* Fred A. Noetzli, "Gravity and Arch Action in Curved Dams," Transactions A.S.C.E., Vol.LXXXIV. Eq. 11, p. 10 (1921).

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That is, $\varepsilon = \frac{\Delta L}{L} \frac{D}{\frac{3}{16}\frac{L^2}{h}}$. The plotted strains from the telemeter data are in all cases the averages of the downstream and the upstream strains, and averages of strains measured at different locations along the arch ring. It will be seen that the strains in the vertical direction were in all cases quite close to those computed from the temperature changes, while those in the horizontal direction appear to bear little relation to the temperatures. The principal exception to this statement is shown in Fig. 50 for the 50-ft. elevation. Here the strains in the horizontal direction computed from the deflections are a much smaller proportion of the measured strains than elsewhere, and the agreement between the measured horizontal strains and the strains computed from the temperature changes (same for horizontal as for vertical) is much better than elsewhere. This is as should be expected if (as may have been the case) the dam was broken loose from the abutment at the 50-ft. elevation. The tensile strains across the abutment joint were large enough on the downstream face as early as June 6 that the dam may then have been cracked loose. On June 12, a crack was observed on the downstream side. A crack was first observed on the upstream side on June 6. The strains above the 50-ft. elevation across the abutment joint are shown in Fig. 70, 71, and 72.

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It was found that during the period of the hardening of the concrete, the vertical strain in the dam at a point was greater, in general, than the horizontal strain at the same point. It was thought at first that this might be in the nature of a continued flow added to the shrinkage. Later studies, however, showed that the vertical strains were related to the temperature changes and the lateral strains to the deflections in such a manner as to indicate that the restraint of the dam at the abutments against free movement was responsible for the difference between the vertical and the horizontal strains. The excess of the vertical over the horizontal strains was so general that there is no question as to its existence.

Fig. 51 and 52, prepared in a study of the calibration of the telemeters, give information also on the behavior of the dam under temperature changes when the concrete at points under consideration was from 17 to 36 days old. The strains in the vertical direction, as measured by the strain gage, are shown in the figures by the curves marked 504A, 504UA, and 508UA, and those in the horizontal direction at the same stations by 504C, 504UC, 508C, and 508UC. The rates of expansion with temperature change (the slope of the curves) for the vertical are nearly the same as for the horizontal direction, and in this respect conditions are different from those noted for the hardening period. The dam was free to expand or contract vertically, and since the horizontal

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was nearly the same as the vertical expansion (or contraction) it appears that the direct stress accompanying the expansion must have been slight. A further examination (including 35 stations distributed over the dam) of the effect of the temperature rise during the drying period, showed that the difference between the vertical and horizontal strains was too small to be detected except for the stations near the abutments. For the abutment stations below the 20-ft. elevation, the strains averaged about 0.000040 in. per in. greater in the vertical direction than the strains in the horizontal direction. For the higher elevations, the difference between vertical and horizontal strains was only about 0.000010 in. per in.

The slopes of the curves correspond quite well with the coefficient of expansion (0.0000077 in per in. per deg. C.) determined in the laboratory tests at the University of California. Between tests 2 and 3 there was at the 60-ft. elevation an average change of temperature of 4.97 deg. C. and an average increase in strain (including both vertical and horizontal direction) determined by the strain gage, of 0.00003 in per in. Between tests 3 and 4, there was an average drop in temperature at the 60-ft. elevation of 7.77 deg. c., and the average decrease in strain of 0.000071 in. per in. The corresponding coefficients of expansion were 0.0000060 and 0.0000092 for the two cases, respectively,

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giving an average value of 0.0000076 in. per in per deg. C.

At the 60-ft. elevation, there are only three telemeter stations. At the 50-ft. elevation there are five stations and the coefficient determined for that elevation may be expected to show less variation from time to time than that for the 60-ft. elevation. For the 50-ft. elevation between tests 2 and 3, the temperature rose 2 deg. C., and the average expansion was 0.000018 in. per in. Between tests 3 and 4 there was a decrease in temperature of 6.5 deg. and a contraction of 0.000048 in. per im. and the corresponding coefficients of expansion of 0.0000090 and 0.0000074, respectively, with an average of 0.0000082 in. per in. per deg. C. The average coefficient for the 50 and the 60-ft. elevations was 0.0000079 in. per in. per deg. C.

Additional information on the coefficient of expansion is furnished by tests made at the University of California on three 6 by 12-in. control specimens from the dam, and on several 3 by 3 by 40-in. specimens made from concrete of the same mix, and using the same aggregate as was used in the dam. The cylinders had telemeters embedded in them and strains were measured both with the telemeters and with dial extensometers attached externally. As the telemeters themselves are affected by the change in temperature, the values obtained by the extensometer were used in determining the thermal coefficient of expansion. The specimens were tested both

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Dry and saturated, and the average coefficients found were 0.0000080 and 0.0000074, respectively, with an average of 0.0000077 in. per in. per deg. C. The 3 by 3 by 40-inc. specimens at an age of 7 weeks gave an average coefficient of 0.0000074 in. per in. per deg. C., with the specimens successively dry and water soaked.

The agreement of the various values quoted is reasonably good, and the average coefficient was about 0.0000077 in. per in. per deg. C. Additional discussion of the relation between temperatures and strains is found in Sec. 76, "effect of Drying."

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73. Deflections and Relation between Deflection and Temperature. Deflections which occurred during construction, as nearly as they may be determined from the data, are shown in Fig. 11 and are discussed in Sec. 29. After the completion of construction it was possible to determine more exactly what the movements were. On the morning of June 5, 1926, one day after completion of the pouring of concrete, a complete set of deflection readings was taken. These readings were used as the datum for plotting the isometric diagrams of deflections for several subsequent dates with the dam under no water pressure. See Fig. 53 to 60. Fig. 53 shows that the deflection which occurred between June 5 and June 14 was mainly at the top of the dam. Comparison of the temperatures for the various heights shown in Fig. 46 and 44 with the deflections in Fig. 53, shows that the deflections from June 5 to June 14, practically all of which were downstream, were approximately proportional to the fall in temperature for the various heights. From June 14 to June 24, the temperatures rose slightly, and Fig. 54 compared with Fig. 53 shows that there was a slight decrease in deflection in this period, but not as much as would be expected if it be assumed that the deflection was due entirely to temperature change. On June 24, in order to study the effect on the dam of allowing it to dry for a period of time, the sprinklers used to keep the dam saturated were shut off and no water was applied until July 2.

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Fig. 46 and 44 show that from June 24 to June 30 the temperature within the concrete rose about 8 deg. in the upper part of the dam, and Fig. 55 shows that on June 30 the dam had almost everywhere deflected upstream by an amount which corresponded quite closely to the temperature rise. Evidently, the deflection was due mainly to the temperature rise, and since the air temperature, as shown by Fig. 45, was nearly constant during this time, the temperature rise in the concrete must have been caused by the drying out.

On July 3, about 12 hours after the water had again been turned on the dam, the deflections of Fig. 56 were observed. Fig. 46 and 44 show that the temperature dropped rapidly at the top of the dam, but only slightly at the lower elevations. Likewise, Fig. 56 shows that the downstream deflections were much greater at the top than at the lower elevations. The irregularity in the curves at the extreme right of this figure is probably due to an erroneous reading at elevation 50, though there is no way of knowing positively that such an error was present.

Fig. 57 was plotted from the zero readings for the first load test, that of July 12, compared with the reading of June 5. On July 12 deflections at the various elevations relative to those of June 24, were about as would be expected from the difference of temperature for these two dates.

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Considerable variation in temperature occurred between load tests 2,3, and 4, and the temperature differences have been used in computing the coefficients of expansion given in Sec. 72. Fig. 58, 59, and 60 show that the variation in deflection for this period is in reasonable agreement with the variation in the internal temperature of the dam. Data are available for showing similar no-load deflection curves corresponding to the time of each of the remaining load tests, but it is believed that those already given are sufficient to illustrate the behavior of the dam. Fig. 61 shows the crown deflection at elevations 30 and 60, from the date of pouring to the end of the testing program.

Fig. 62 (a) gives a comparison between the difference of the crown deflections observed from June 24 to June 30 and from June 24 to July 3, with the deflections computed by Cain's * formula for the support of arch rings at various heights from the 30 to the 60-ft. elevations. The change in temperature was determined from the average temperatures observed with the telemeters at each elevation.

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* William Cain, "The Circular Arch Under Normal Loads", Transactions, A. S. C. E., Vol. LXXXV p. 233 (1922).

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The coefficient of expansion used was 0.0000077 in. per in. per deg. C. The observed deflection from June 24 to June 30 was less than the deflection computed for either hinged or fixed arch at any elevation considered. The observed deflection from June 24 to July 3 was between the deflections computed for hinged and fixed arches for most of the arch elements between the 30-ft. elevation and the top of the dam.

Fig. 62 (b) gives the coefficients by which the product of the temperature change and the coefficient of expansion must be multiplied to obtain the crown deflection at any elevation for any given conditions. Fig. 62 (b) shows that a constant temperature change above the 30-ft. elevation will give nearly a constant deflection. Therefore, the deflection at any elevation should be nearly proportional to the temperature change at that elevation. It has been pointed out in the preceding paragraphs that this was approximately the case with the test dam.

During the curing period, it was found that the deflections varied considerably during the day. It seemed probable that this was due to the temperature variation. This temperature variation was due partly to the fact that during the day the sun shone on one face of the dam and not on the other face at the same time, and partly to the fact

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that the water which kept the dam saturated was necessarily shut off during the taking of the observations. In Fig. 63 the crown deflections at various elevations which occurred between 6 a.m. and 2 p.m. on June 24 are shown, also the change of temperature and the vertical bending strains at various elevations, as measured with telemeter and strain gage. These bending strains are algebraic differences between the total strains measured on the upstream face and the downstream face of the dam. The strains measured with the telemeter were available only from the bottom of the dam to the 30-ft. elevation and those measured with the strain gage were available only from the 30-ft. elevation to the top. At the 30-ft. elevation both were available, and the difference between the two values is a large percentage of either one, but it is not larger than the sum of the probable errors of the instruments. The deflections measured with the clinometer and those computed from the strains are shown both in the same diagram for comparison. The computed deflection curve, assumes as the strain at the 30-ft. elevation, the average of those found by the telemeter and the strain gage. The upper and the lower dotted lines from the 25 to the 35-ft. elevations show the deflections which would have been obtained by using the strains from the telemeter and strain gage separately.

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The agreement of the curve of measured deflection with the curve of deflection computed from the strains is fair, and both indicate quite a large deflection, about $\frac{1}{16}$ in. at the top of the dam. Since all of the strains measured were so small that the probable error of the instruments would be a considerable percentage of the measured strain, close agreement may have been partly accidental, and not too much weight should be attached to this agreement.

Temperature changes near both faces and in the interior of the dam wherever observations were obtained, are shown in Fig. 63. The number of points in the diagram is too small to give full information on the distribution of the change of temperature, but in general it seems that the change was greatest on the upstream face at the top of the dam and on the downstream face at elevation 30 and lower. The sun shone on the upstream face during most of the time and this probably accounts for the greater change in the upstream face at the top. It is not clear why, for the lower elevations, the change was not about the same on the downstream face as on the upstream face, since the platforms and framework shielded the dam from the direct rays of the sun on both faces at the lower elevations. Temperature changes in the interior of the dam were observed only at the 10 and the 20-ft. elevations, and it is clear that there the temperature change was quite small.

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74. Apparent Loads Computed from Temperature Deflections. For the two periods, June 24 to July 30 and July 15 to July 19, the temperature deflections were reduced to apparent moments and loads on the vertical center line element and on the uppermost horizontal element. The apparent bending loads were determined by differentiation of deflections by methods similar to those described in Sec. 132. The apparent direct load is the load which if applied to the dam would, according to the cylinder formula, cause the same direct strain as the measured temperature strains. The deflections, strains, moments, and loads, both bending and direct, determined in this study are shown in Fig. 64 to 67. By the term apparent load is meant the load which would be required to bring back to the position occupied before the temperature change took place, the elements of the dam under consideration. The apparent moments are the moments produced by the apparent bending load. It is not probable that the internal stresses are as great as the apparent moments indicate, since part of the strains are merely temperature expansions and contractions. However, since the dam is restrained against free expansion and contraction, deflections must set up stresses. It is not known what proportion the actual moment is of the apparent moment, but since the deflections and the temperature changes were much the greatest at the top of the dam, whereas the

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apparent moments and loads were much the greatest near the bottom, it seems likely that the actual moments were not far short of the apparent moments. The temperature changes at the top and at the bottom of the dam from June 24 to 30 were about 8 deg. and 3.5 deg. C., respectively. The highest apparent bending stress resulting from the temperature changes was about 100 lb. per sq. in.

Since the dam is free to expand and contract in the vertical direction, it is assumed that the vertical direct stress due to temperature change may be neglected. In Fig. 66 and 67 the plotted points show the apparent bending loads, and the cross-hatched area shows the apparent direct loads on the uppermost horizontal element of the dam. The actual bending moment and bending stresses probably are about equal to the apparent bending moment and the stresses computed therefrom. However, for reasons given in Sec. 72 it is believed that the dam was nearly free to expand horizontally without introducing large direct stress. Therefore, the stress corresponding to the apparent direct load probably is negligible.

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75. Cracking Away from Foundation During Curing Period. The first evidence of cracking of any kind was given by telemeters 2U and 4U. These telemeters are located at an elevation of about 12 ft. and are about 24 ft. north and south respectively of the vertical center-line of the dam. They extend from the foundation into the dam. The strains measured by these telemeters from April 28 to May 3 are shown in Fig. 47. It appears likely that the concrete cracked away from the foundation between April 29 and May 1 at Station 4U and between May 2 and May 3 at Station 2U. After these dates the resistance in the telemeters was far beyond their calibrated range and it seems certain that at least by that time a crack had occurred. Evidently, however, the crack width became about stationary, since the resistance did not continue to increase appreciably. The later resistances are not shown in Fig. 47 as they are beyond the range of the calibration curve. Examination at one of these stations about May 12 disclosed a crack which appeared to be about 0.01 in. wide. The upstream telemeter cartridge at station 3 (passing from the foundation into the dam at the center line) had shown no similar movement up to the time that it went out of commission on July 23, during test 4, when, under a bending load due to a 30-ft. head of water, a crack formed between the dam and the foundation on the upstream side of the dam.

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At elevation 30, the strain gage stations were placed in such a way that at each end of the dam two gage lines crossed from the dam into the foundation rock on the downstream face of the dam and one on the upstream face. The concrete was placed to a height which included these stations on May 13, and initial readings were taken on May 14. In Fig. 68 the strains observed are shown. At both ends of the dam the strains on the upstream face increased quite sharply on May 19 and on May 22, a crack was observed at the south end. On May 22 the temperature began to fall and continued falling until the minimum temperature was reached on May 25, the date on which the maximum opening of the crack is noted.

Although Fig. 68 indicates considerable elongation on the downstream face, no crack was found at either end of the dam on that face at the 30-ft. elevation until about June 30 during the period referred to in Sec. 76 as the drying period. However, the principal feature of significance during this period seems to have been the temperature change. The sharp rise which occurred for gage lines A and B and stations 1D and 2D, beginning June 24, corresponds perfectly with the date of shutting off the sprinkling water from the dam.

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Fig. 68 shows that at both ends of the dam, both the upstream and the downstream gage lines across the abutment joint elongated with the fall in temperature from May 18 to 26. The upstream elongation was considerably larger than the downstream elongation, indicating a rotation corresponding to a downstream deflection of the dam. Later on during the drying period, a sharp rise in the internal temperature of the dam occurred, and correspondingly, a shortening of both upstream and downstream gage lines might be expected. However, only the upstream gage lines shortened and the downstream gage lines elongated. This anomalous behavior may be accounted for if it be assumed that the dam was not entirely free to expand and that the pressure at the abutments produced an upstream deflection resulting in a rotation of the dam at the abutments, as indicated by the strains shown in Fig. 68. Fig. 61 which gives the crown deflections at the 30-ft. elevation shows that some such deflection as that here outlined took place as the temperatures changed. After the high internal temperature of June 30, the upstream gage lines elongated and the downstream gage lines shortened consistently with the above explanation.

Fig. 69 to 72 show the movements at the abutment joints at elevations 44, 53, 55 and at the top of the dam respectively. A comparison of Fig. 69 to 71 shows sudden

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elongation of the gage length on the upstream face at station 3U, elevation 55, within two days after pouring, compared with a steady change of gage length up to June 24, for stations 3D and 4D on the downstream face at elevations 44 and 53, respectively.

From the diagram for station 3U in Fig. 71, it appears that the dam cracked loose from the abutment at the south end of the dam between June 4 and June 6. In fact, on June 6 the crack was observed, but on June 5 it could not be detected by the unaided eye.

Comparison of Fig. 68 to 72 with Fig. 46 shows that the dates on which large elongation or shortening at the abutment joint occurred are generally the same as the dates on which large temperature changes took place. On the same dates also the large strains within the dam and large deflections have elsewhere been shown to occur.

The crack at the abutment joint on the top of the dam occurred at each end on June 6 about 36 hours after the completion of the dam. The width at the center of the thickness of the dam, as measured with the strain gage was about 0.008 in. at each abutment. The cracks when first observed were widest at the upstream face, and extended to within about one or two inches of the downstream face. Within a few days the cracks had

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extended entirely across the dam. Fig. 72 shows the measured extensions and shortenings on all of the gage lines on the top of the dam across the abutment joints. Although when the crack was first found, it could not be traced entirely to the downstream face, the movement shown in Fig. 72 for the north abutment was greater for gage line A than for gage line B, although gage line A was slightly closer to the upstream face. At the south abutment the movement was greater in gage line B than in gage line A. Gage lines A and B are so close together where they cross the crack that the differences of their readings cannot be expected to tell much about the rotation of the dam at the abutment. In order to detect more exactly this rotation, gage lines E1 to E4, inclusive, were placed in the positions shown in the sketches in Fig. 72 the movements were shown by the graphs in these gage lines.

It is likely that before the load tests began on July 12, cracks extended all the way from the top of the dam down to within about 10 feet of the bottom on the upstream face at the abutment joint, though they were not detected at all points in this course. On the downstream face similar cracks had been found to extend down

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to about the 50-ft. elevation before the drying period of June 24 to July 2. During this drying period they were found to extend down to slightly below the 30-ft. elevation. No other cracks than those which occurred in and along the abutment joints were detected before the first load test.

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76. Effect of Drying. In order to obtain some information on the effect on the dam of allowing it to dry for a period of time, the sprinkling water was shut off from June 24 to July 2. The effect was to cause a sharp rise in the internal temperature of all parts of the dam when the cooling effect of the water was removed. The temperatures varied from about 8 deg. C. in the upper part to about 3 deg. C. near the bottom of the dam. The other effects of the drying out seem to have been much the same as should have been expected if the temperature rise had been caused by any other agency than drying out. In other words, the drying out seemed to have no appreciable effect, except as it permitted the temperature to rise.

The effect produced during the drying period on strains at representative positions on both faces of the dam is shown in Fig. 49. Averaging strains in all four directions at a given 8-point station, it was found that an expansion of from 0.000040 to 0.000080 in. per in. occurred on the downstream face and about 0.000060 to 0.00010 in. per in. on the upstream face.

After the water was again turned on the dam, the strains returned to approximately the same values as before the drying period.

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The deflections during this period are seen in Fig. 54, 55, and 56. The maximum deflection was about 0.1 in. upstream. It seems from the diagrams that the deflections did not entirely disappear for a considerable length of time after the water was again turned on. In this respect they correspond very well with the slow disappearance of the interior temperature effect.

The change in widening of the cracks along the abutments is shown in Fig. 68 to 72 and a discussion of the cracks is given in Sec. 75.

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VIII. GENERAL DATA OF TESTS.

81. Description of Load Tests. Thirteen load tests were made. Table 8 gives a list of these tests. The general procedure in carrying out the tests was to turn off the water from one to four hours before the testing, and to begin zero readings about 12 a. m. In general, the zero readings required about two hours, though there were considerable variations from this length of time. The water was admitted as soon as the zero readings were completed, and with the earlier tests the reservoir was filled first to a height of 10 ft. below the maximum height for the test being carried out. Readings were taken at certain key positions for comparison with previous tests under the same head of water. About one-half hour was allowed for the taking of these key readings. Immediately after completion of the key readings, the water was again admitted and the head increased to the maximum head for the test.

The first three tests used only the flow of water from Stevenson Creek, but it was found that so long a time was required to fill the reservoir that with the 30-ft. head and greater, there would not be sufficient time during one night to take the no-load readings, fill the reservoir, and take the load readings unless the filling of the reservoir could be speeded up. For all the succeeding tests water was taken from water tunnel No. 3 of the Southern California Edison Company by opening the 6-in. outlet valve in the bulkhead in the adit.

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In test No. 8, (made on September 4, with only the telemeter readings taken at a head of 50 ft., and all other readings at 40 ft.) the length of time required for filling the reservoir was so great, owing both to the requirement of a larger quantity of water than in previous tests, and to difficulty in operating the 6-in. valve in the tunnel, that it was necessary to omit the key readings in order to complete the test within the time available. In tests No. 9, 10, and 11, it was necessary also to omit the key readings at heads of 40 and 50 ft. It was the aim in all of the tests to complete all of the observations by 8 a.m., taking as many as time permitted after the emptying of the reservoir. In the errlier tests made in July, this was essential because the sun rose upon the dam at about eight o'clock and introduced temperature strains and deflections. For the later tests it was not necessary for this reason, but after eight hours of continuous work throughout the night, the strain upon the observers was very great, and it seemed desirable not to prolong the test, except for the most urgent purposes.

For heads of 40 ft. and under, there was nothing to indicate that provision for safety in case of failure of the dam need be made, because there seemed to be no possible danger of failure. With the approach of the test to a head of 50 ft., it appeared that some precautions should be taken for the safety of the men who operated the gate valves during the filling and emptying of the reservoir. The valve

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stems rose on the upstream side of the dam to a platform located about 10 ft. north of the center of the length of the dam, as shown in Fig. 73. A rope life-line was stretched across the canyon above this platform and attached at both ends to anchor bolts in the rock. During the test the operators secured themselves by means of auxiliary ropes around their waists and looped over the life-line. During this test it seemed for a few minutes that the precautions taken were very timely, as a crack occurred and in the darkness it was uncertain as to how serious it was. This crack is described in Sec. 83.

At the time of the occurrence of the crack, the water in the reservoir stood at an elevation of approximately 50 ft. The crack extended entirely across the thickness of the dam, but it seemed unsafe to go below the dam to make an examination to see how far down the face of the dam it extended, or to take the strain-gage and clinometer readings. The station for taking the telemeter observations, shown in Fig. 4, was in a place free from danger, and a complete set of telemeter readings was taken. These readings required about one and one-fourth hours and as soon as they were completed the water was lowered to the 40-ft. level. As the dam had carried the head of 50 ft. of water for this length of time without further indication of failure, it was considered safe to proceed with the observations, and this was done.

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The readings for test No. 9 were taken under a head of 50 ft. In order to establish the safety of taking readings under this head, the water was first raised to an elevation of about 54 ft. and held there for about one-half hour. It was then lowered to the 50-ft level and the readings were taken. For test 10, in which the readings were again taken under a head of 50 ft., it was not necessary to establish again the safety of taking these readings. Hence the water was raised to the 50-ft. level as rapidly as possible, and the readings were taken at once.

For test 11, the first test with a 60-ft. head, it was not possible to establish safety by raising the water to an elevation greater than that at which the readings were to be taken, but the same result was accomplished by filling the reservoir during the day and leaving it under the head of 60 ft. from about 4:45 p.m. until 10:45 p.m., when the taking of the readings began.

Although the safety of taking the readings under the 60-ft. head had been sufficiently established by the above procedure, still the same procedure was followed in the succeeding tests, No. 12 and 13, which also were under a 60-ft. head. This was done because so long a time was required to fill to the 60-ft. elevation that there would not have been time to take zero readings long enough after sunset that a temperature equilibrium had been established, then to fill the reservoir, and finally to complete the load readings before the sun was upon the dam on the following morning.

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82. Leakage. In the testing, there was extremely little leakage. For holding the forms together during construction, form bolts extended entirely through the dam from upstream to downstream face. Around the downstream ends of about one-quarter of these bolts moist spots appeared when the dam was under a head of water, due evidently, to a little water following the surface of the bolts through the dam. Some leakage also occurred through the abutment joints of the dam, and much passed through fissures in the rock entirely away from the dam. With a head of less than 50 ft. no leakage of any kind through the concrete of the dam or through a construction joint in the dam was discovered. Under a head of 50 ft. a small moist patch about 4 by 7 in. appeared at the horizontal construction joint 15 ft. above the base of the dam, due apparently to seepage through this joint. Under a head of 60 ft. enough moisture came through the construction joint at an elevation of 20 ft. to wet an area about 7 ft. long and 4 ft. high. Also in the same test, four or five small patches of moisture appeared at the construction joint 55 ft. above the base of the dam. These patches were not larger than one foot in diameter. Under the head of 50 ft. rough measurements were made from which it was estimated that the total leakage was about 6 gal. per min. Of this it seems fair to estimate that 95 per cent came

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through fissures in the rock. Under the head of 60 ft. the leakage was considerably larger, but the percentage which came through the dam appeared to be even smaller than under the 50-ft. head.

On September 24, after the horizontal crack at the 30-ft. elevation, described in Sec. 83, had formed, some seepage through this crack was discovered. The area covered by this seepage was quite extensive. The amount of water coming through was very small, but was greatest near the abutment.

The leakage was not serious at any time, and it is believed that even the leakage which did occur would in time have been stopped by the filling up of pores and cracks with sediment, unless under the continued load the cracks continued to open.

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83. Cracking During Testing Period. In order to facilitate the detection of cracks, a fillet of cement mortar was placed in the angle at the junction of the dam with each abutment, on the upstream side from the bottom to the top of the dam. The smooth surface thus afforded gave a good opportunity for observation of cracks when they occurred. This fillet was placed after the formation of the first cracks due to shrinkage or temperature changes and before the first load test, and afforded a good opportunity for the determination of subsequent changes in the behavior of the dam. Up to the completion of the load tests, there had been very little increase in the extent of the cracking at the abutment, and in much of the region where early cracks were formed, and later covered by the fillet, no crack was then in evidence. Near the top of the dam a distinct crack was to be found at each end on the upstream side for a depth at the south end of about 10 or 12 ft. At the north end the dam seems to be free from the rock on the upstream side for the upper 4 ft. Therewas then an interval of about 3 ft. with no sign of a crack. Below this point a crack appeared intermittently down to an elevation of about 36 ft. Below that point to elevation 30 there was no crack except the old one which appeared soon after the completion of the dam. Where this crack was covered with fresh mortar the mortar did not crack. The first crack due to the water pressure occurred at the bottom of the dam, in the joint between the

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concrete and the rock on the upstream side.

The first indication of the imminence of this crack was given by the telemeter reading in test No. 4 with a 30-ft. head of water. After the first indication of marked elongation on the upstream side frequent readings were taken during the remainder of the test, and the record of those readings is given in Fig. 74. A continued elongation on the upstream side greater than the concrete could be expected to resist without cracking is noted. The largest reading indicates an elongation within the 6-in. gage length of about 0.0015 in. Corresponding to this elongation on the upstream face, there was an increase in the compression in the telemeter embedded in the rock on the downstream side. After the reservoir had been emptied, the resistance in this telemeter returned to the previous no-load value. There is no doubt that a crack occurred before the 40-ft. head of water was reached and after the reservoir was emptied the resistance did not return to a value even within the range of the calibration curve.

After noting the possibility of a crack at this location, and realizing that the width of the crack under higher heads would probably be out of range of measurement with the telemeter, one of the resistance micrometers, designed to be used in connection with the towers for the

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measurement of deflections, was equipped for the measurement of further opening of this crack. This micrometer is essentially an instrument which consists of a coil of resistance wire, a portion of which is cut out with a decrease of the gage length over which the instrument operates. The instrument is calibrated for movement in inches against resistance of the coil. The calibration curve for the instrument used in the measurement of this crack width is shown in Fig. 25. In adapting the instrument to the measurement of the crack under water the essential feature was its protection against leakage of water into the resistance coil and into the lead-wires extending from the instrument to the observation platform. The instrument was protected from water by means of an envelope made from a portion of an automobile inner tube, and it operated satisfactorily for the purpose of measuring the crack widths. Fig. 75 shows the amount of opening of this crack for tests 6 to 12.

Although the agreement in different tests under the same head of water is not perfect, it is sufficiently good to show that the measurements are significant of the opening of this crack. It is not unlikely that differences in temperature for successive tests would be sufficient to cause the cracks between the dam and the abutment near the top of the dam to vary in width before the application of pressure. Such a variation in the width of the upper cracks would cause an opening of the crack at the bottom to start

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earlier or later depending on whether the upper cracks were wider or narrower. For these reasons, the variations, shown in Fig. 132, of the width of the crack at the bottom do not seem greater than might be expected.

In test No. 8, a vertical crack occurred very nearly on the exact center-line from the top of the dam to approximately elevation 49 on the downstream face. As measured with a scale and magnifying glass, the crack width on the top was found to be $\frac{1}{60}$-in. on the upstream face and $\frac{1}{40}$-in. on the downstream face after the water had been lowered to an elevation of 40 ft. The strain observed with the telemeter during the rise of the water was 0.000065 in. per in. (corresponding stress about 240 lb. per sq. in.) when the elevation of the water was about 47 ft. It is not known at just what instant the crack appeared, but it does not seem likely that it occurred before the water reached a head of 50 ft. At this time the static tensile stress due to the water pressure could not have been much above 300 lb. per sq. in. It is not unlikely that there was some initial stress in the dam at the beginning of the test, although the dam had been kept wet by means of continuous streams of water from a sprinkler pipe up to within about two hours of the beginning of the test. When the water level reached 50 ft. it was necessary to open the 24-in. valve to control the water level. There was evidence of considerable vibration

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in the dam when the valve was opened, and it is not improbable that the crack occurred during this vibration and partly on account of it. In test No. 9 instruments used for measuring the change in width of the crack during filling and emptying of the reservoir, indicated vibration with an amplitude of 0.0005 in. when the 24-in. valve was opened.

At the close of test No. 8, as the water was being discharged, strain gages held on the concrete across the crack indicated a closure of the crack on the downstream side of 0.013 in. and on the upstream side of 0.0031 in. With removal of the water after test No. 8, the crack remained open about 0.01 in. and closed rather slowly until in the middle of the day it was almost entirely closed. After removal of the water in test No. 9 the crack immediately closed almost completely. That there was no seepage through this crack confirms the observation that the crack did not extend all the way through the dam, down as far as the 50-ft. elevation.

In later tests measurements were taken showing the variation in width of this crack with various heads of water against the dam. Fig. 76 shows that the crack began opening under a head of a very few feet and continued to open until a head of from 45 to 50 ft. had been reached, and from that time on until the water reached the top of the dam the crack closed progressively. Evidently, the cause of the closure of the crack was the direct compression (arch action) which

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apparently extended only a short distance above the water level. The indications of a negative opening of the crack on the upstream side of the dam under the full head (60 ft.) may be satisfactorily explained by assuming that the crack was open when the zero reading on the instrument was made. Therefore the diagram would indicate that the crack was further closed at the top of the dam with the reservoir full than with it empty. By the end of the testing program, this crack could be traced down to an elevation of 41 ft. on the downstream face.

During the first test to a head of 60 ft. the highest tensile strain observed, based on a modulus of elasticity of 3,600,000 lb. per sq. in. was about 520 lb. per sq. in. This was in a horizontal direction at station 6 across the centerline near the bottom of the dam. At the time of completion of the filling of the reservoir the telemeter readings indicated that there was no crack at this place. The full head of 60 ft. was reached about 4:45 p.m., and this head was held until 10:45 p.m., when the deflection, strain, and other observations began. By this time the strain at the place where the stress of 520 lb. per sq. in. had been observed had gone beyond the calibrated range of the telemeter, and it was evident that a crack had occurred. Visual inspection disclosed this crack almost exactly on the center line. At this time the crack could be traced from about 2 in. above the bottom of the dam to a height of about 9 ft. Before the close of the testing

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program this crack had extended so that it could be traced to a height of 13 ft.

This crack did not extend entirely through the dam, at least at the time of the last telemeter readings, for the telemeters near the upstream face at station 6 showed a considerable compression in horizontal direction at the center-line of the dam. However, there was a slight amount of seepage through this crack. Water may have seeped through a horizontal construction joint from the upstream face into the interior of the dam whence, meeting an open vertical crack, it would travel to the downstream face through the latter crack. This is the more likely, since there was considerable tension in the vertical direction within this region on the upstream face. An attempt was made to search for cracks on the upstream face of the dam at this elevation, but an effective examination could not be made because of the large amount of falling water which could not be diverted through the oversluice. The width of the crack on the downstream face was not measured, but it appeared to be not more than 0.01 in. wide at the widest part.

Between tests 5 and 6, a thin mortar patch was plastered smoothly across each construction joint on the downstream and also on the upstream face of the dam in such position that it was crossed by the vertical center-line. Most of the patches were about 4 in. square but the one at the 30-ft. elevation extended throughout the full length of the dam.

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These patches were provided to furnish a means of detecting more accurately the time of the formation and the location of the cracks.

After the program of load testing had been completed, the dam was left under the pressure of a 60-ft. head of water from September 23 to 25. Under this pressure horizontal cracks were discovered at the 30 and 35-ft. elevations along the construction joints. At elevation 30, the mortar patch covering the construction joint was cracked all the way from the abutments to within a few feet of the center. At the 35-ft. elevation the mortar patch across the construction joint at the center of the dam was cracked. These were the only horizontal cracks found in the dam.

The extent of the cracks after the flood of November, 1926, is discussed briefly in Sec. 89.

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84. Temperature Variation During Load Tests. The temperature variations during load tests are shown in Fig. 46. Between load tests 2, 3, and 4, there was a maximum range of temperature of about 8 deg. C. For the rest of the load tests the variation was much smaller. However, the temperature variation from one load test to the next is of comparatively slight importance in its effect on the strains developed, since the observed effect of a load is practically independent of the temperature at which it is applied, on the condition that the difference in temperature between the no-load and the load readings is negligible. This condition was met in this investigation by the taking of a complete set of no-load readings for each load test, and not depending on zero readings taken at a previous time. Fig. 46 shows the difference in temperature between the times of taking the no-load readings and the load readings for each test, and it will be seen that the difference was less than 1 deg. C. for the great majority of cases. In one case, it was 3 deg. C. The significance of these temperature differences may be judged by the fact that the effect of temperature change on the strains is approximately equivalent to that of a stress of 30 lb. per sq. in. per deg. C. change of temperature.

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85. Movement of Bedrock. In the preliminary plans for the test, it had been planned that movement of the bedrock should be measured by triangulation methods. The results secured from triangulation did not give warrant for the expectation that sufficient accuracy could be secured to permit measurements of bedrock movement by this method. A method was, therefore, substituted by involving the use of $\frac{3}{8}$-in. invar steel bars extending between the points, the relative movement of which was to be measured. Fig. 77 shows the location and arrangement of the invar steel bars used for the measurement of movement of bedrock. Movements in two general directions were measured, namely, the movement of the bedrock in approximately a radial direction near the bottom of the dam, and the movement of the abutments in approximately the direction of a long chord, at several elevations.

For the radial movement of the bedrock near the bottom of the dam, an invar steel bar 17 ft. long was attached at one end to a steel plug set into the rock just below the downstream face of the dam at an elevation of about 1 ft. The bar was carried along the face of the canyon and at its downstream end it bore against the plunger of a one ten thousand-inch Ames gage which was attached rigidly to the face of the rock wall. The arrangement of the apparatus used is shown in Fig. 78. As shown in this photograph, the invar bar was supported at frequent intervals

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along the length of a 4by4" timber which spanned the gage length. While it is possible that both points of attachment to the rock moved, due to the pressure of the water, it seems likely that the greater part of this movement would be within the first few feet next to the dam, and that the amount of movement at the lower point of the attachment to the rock would be so small as to be entirely unmeasurable.

Readings were first taken with this apparatus in test No. 6, in which the maximum head of water was 40 ft. Although the bar used was of invar steel, the readings were somewhat affected by temperature changes during the test. During the release of the water from the reservoir, the time involved was so short that the temperature changes were very small. For this reason the readings taken during the emptying rather than the filling of the reservoir were used. Fig. 79 shows the measured movements for tests 7, 9, 10, 11, and 12. In test 6 only the total movement, due to the entire head of 40 ft., was measured and this amounted to 0.00076 in. The largest movement occurred with 60-ft. head and averaged about 0.0012 in. for the two tests in which measurements were taken. Fig. 79 shows considerable divergence of measured movements for different tests, but the agreement is sufficiently good to indicate approximately the magnitude of the movement for various heads of water.

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Movement of the abutments along the direction of a long chord, that is, the spreading of the canyon walls, measured at elevations of 17 and 33 ft., is shown in Fig. 80, with corresponding heads of water. For taking these measurements a $\frac{3}{8}$-in. invar steel bar was attached to a heavy steel plug, embedded in the bedrock at the north end of the dam at each of these elevations. Each bar extended along the direction of the long chord to a corresponding point at the south abutment where the end of the bar bore against the plunger of an Ames gage attached to another plug embedded in the bedrock. The method of supporting the invar bar throughout its length and the end connections with the rock, were similar to those illustrated in Fig. 78. Thus the total change of distance between the ends of the chord was transmitted to the Ames gage. The readings gave the sum of the movement at the two end points. It is this total movement along the chord length which is plotted in Fig. 80. The installation for these measurements was not made until just previous to the first test to a head of 60 ft., and the measurements were made only in tests 11 and 12. The agreement of the measured movements for the two tests with each other is very good and the movement at elevation 17 was about the same as that at elevation 33. The average movement for a head of 60 ft. was about 0.035 inches.

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At the 60-ft. elevation movements of points at the north and the south ends were measured independently. It was not feasible in this case to measure in the direction of the chord at both ends, nor was it feasible to measure in the direction of the tangent at both ends. Consequently, the measurement at the north end was approximately in the direction of a tangent to the dam at that point, while the measurement at the south end was approximately along the direction of the long chord. The method of measurement in these cases was substantially the same as for the measurement at the bottom of the dam. The movements of the bedrock found at elevation 60 are shown in Fig. 81. It will be seen that the movements were very small and in opposite directions at the two ends.

While the movements of bedrock have not been large, the amounts measured have been consistent and indicate unquestionably that movement was present. The measured spreading of abutments has been very helpful in determining from the deflections the amount of strain due to direct compression.

The measured spreading of the abutments has been compared with corresponding values determined analytically* for the 60-ft. head of water. In making these computations

* Dr. Fredrik Vogt. "Uber die Berechnung der Fundamentdeformation". Published by "Det Norske Videnskaps-Akademi" Oslo (1925).

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the equivalent loads carried by direct compression, as shown in Fig. 119, were used to determine the end reactions at various heights of the dam. The pressures, p, per unit area corresponding to these reactions are used in Dr. Vogt's formula:

$$\Delta z = \frac{4}{ab}\int_0^a\int_0^b \frac{m^2-1}{m^2}\cdot\frac{p}{\pi E}\left\{a\log_e\frac{b+\sqrt{a^2+b^2}}{a} + b\log_e\frac{a+\sqrt{a^2+b^2}}{b}\right\}db\ da$$

$$= \frac{m^2-1}{m^2}\ \frac{p}{E}\ \frac{2}{\pi}\left\{a\log_e\frac{b+\sqrt{a^2+b^2}}{a} + b\log_e\frac{a+\sqrt{a^2+b^2}}{b} - \frac{(a^2+b^2)^{1.5}-(a^3+b^3)}{3ab}\right\}$$

where:

Δz = tangential movement of the abutment due to the arch thrust.

$m = \frac{1}{\mu}$ = reciprocal of Poisson's ratio $= \frac{1}{0.15}$

E = modulus of elasticity of the bedrock, = 2,000,000 lb. per sq. in.

a = width over which the pressure is applied; in this case a is the thickness of the dam.

b = the length of the rectangle of width, a, over which the pressure , p, must be applied in order to give the movement Δz.

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The values of modulus of elasticity and Poisson's ratio given here, are based upon the tests of samples of granite taken from the foundation, as described in Sec. 96 and 97. This equation for Δz gives only the movement of the bedrock due to the direct compression. Dr. Vogt's article includes also an equation for the movement due to the bending moment at the abutment. This movement due to the bending moment was computed for the 30-ft. elevation and was found to be so small that the movement due to bending moment was neglected.

The results of the measured and computed movements of bedrock under a head of water of 60 ft. are shown in Fig. 82. The agreement between the computed movement and the observed movement is sufficiently good to indicate that Dr. Vogt's analysis is of value in determining the movement of bedrock.

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86. Deflections due to Applied Load. The deflections caused by the application of load are shown in Fig. 83 to 91. The maximum deflections, which generally occurred near mid-height of the dam, are shown in Fig. 83 for all tests. The lowest point at which a maximum deflection occurred was at an elevation of 25 ft. and the highest was at an elevation of 45 ft. The most prominent feature of this deflection curve is the sharp break in direction at the 30-ft. elevation. As discussed in Sec. 83, a crack between the dam and the foundation at the center line occurred when the head of water had risen to somewhere between 30 and 40 ft. This fact may appear to be the explanation of the sudden break in the direction of the curve in Fig. 83, but closer examination of Fig. 83 shows that this is not the explanation. This is clear from the fact that the maximum deflection under a 30-ft. head was practically the same as that for the tests made after the crack occurred, and after the application of a 60-ft. head of water. In test 13, the deflections were observed under 60, 50, 40, and 30-ft. heads successively, and the maxima are shown in Fig. 83. If the formation of the crack were responsible for the break in the curve, the break would not have been found for this test, made under decreasing heads. That there is, however, some relation between the width of crack and the amount of the maximum deflection, is indicated by the similarity in the shapes of curves in Fig. 83 and 75.

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The arrangement of the deflection curves in Fig. 84 and 85 for different vertical sections, permits a comparison of deflections at different places at which, for reasons of symmetry, the deflections should be equal if the dam is perfectly homogeneous. The agreement is good and indicates both a fair degree of accuracy in the measurement of the deflections and a fair degree of homogeneity in the dam. Though the differences in deflection were slight, generally the deflections for points south of the center-line of the dam were greater than for points north of the center-line. This difference may be due to slight lack of homogeneity in the dam, or to variation in the rigidity of the foundation rock at various places, or the variation of the rigidity of the connection between the dam and the foundation at various places. The degree of symmetry in the deflections is high and warrants the averaging of deflections and strains for symmetrical points in order to obtain more reliable values. This was done in the reduction of deflections and strains to loads.

In the study of the data of the test of Emigrant Creek Dam at Talent, Oregon, made under the direction of D. C. Henny of the Arch Dam Committee, it was found that there were indications of an upstream deflection. On account of certain inconsistencies in other parts of the data some doubt remained as to whether the upstream deflection actually

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occurred. In the Stevenson Creek dam test there were slight upstream deflections near the ends of the dam, first under a head of water of 30 ft. As the head increased, the amount of the upstream deflection increased, reaching as much as 0.1 in. with a 60-ft. head of water. It has now been reported that the celluloid model of the Stevenson Creek dam, being tested under the direction of Professor Beggs at Princeton University, shows upstream deflections very similar to those of the Stevenson Creek dam. With this double confirmation there need no longer be any reason for doubting the indication of an upstream deflection of the Emigrant Creek Dam. The region of downstream deflection at the top of the dam generally lay within the middle 50 to 60-ft. of the length of the da dam, and the remaining 45 to 40 ft. at either end deflected upstream. The measurements show that upstream deflection occurred at the 60, 55, 50, and 45-ft. elevations, and the indication is that it probably occurred at the 40-ft. elevation, but the deflection stations at the 40-ft. elevation were not so distributed as to make certain on this point. For the 30-ft. elevation and below, there was no upstream deflection.

Fig. 90 shows the deflections for the 40-ft. head of water before and after the formation of the vertical crack in the upper part of the center-line of the dam. It shows that the deflections, both upstream and downstream

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were considerably greater under the same head after the crack than before, but the points of zero deflections were not greatly changed by the formation of the crack.

The deflection as measured on January 5, 1927, is given in Sec. 89.

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87. Tilting of Dam. The level bar measures the change in inclination of the section to which it is applied. The clinometer measures the average change in inclination within the 5-ft. gage length. For certain places it is possible to use the level bar to check inclinations found by the clinometer. At these places the inclinations by the two methods are practically the same, indicating that both instruments were behaving satisfactorily. The degree of agreement at the 10-ft. elevation on the clinometer lines, 10 ft. either side of the center line, is shown in Fig. 92. With the clinometer only approximate inclination at the 10-ft. elevation can be determined, since only average inclinations at approximately 2.5, 7.5, 12.5 ft., etc., elevations were available.

If the change in angular inclination of a horizontal section of the dam at the bottom, due to the applied load, be deducted from the change in angular inclination of a section 5 ft. above the bottom, the difference will the the angle through which the dam has bent within a height of 5 ft. Also this difference in inclination is equal to $\frac{2M}{EI}$. The lowest clinometer gage line measured the change in inclination of the dam in the first 5 ft. at the bottom, and the level bar station was so arranged as to measure it with the level bar at the bottom.

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It had been planned to deduct the inclination measured with the level bar from that measured with the clinometer to determine the bending in the first 5 feet. It was found, however, even after deducting the change in inclination at the bottom that the remaining angular change was so large that it could not have occurred within the 5-ft. gage length without producing an unreasonably large stress. It was not possible, therefore, to use either the level bar or the clinometer data in deriving the moments and loads at the bottom of the dam. The manner of determining the moments and loads for this portion of the dam is shown in Sec. 122.

The use of the level bar data in the manner indicated above is dependent upon the assumption of conservation of plane section. If the sections do not remain plane, the projecting ends of the bars which form the brackets for the use of the level bar stations will have approximately the inclination of a portion of the section near the downstream face instead of the inclination of the section as a whole. Even the I-beams which were embedded solidly in the rock would be bent by the same forces which caused distortion of the section from the plane, thus throwing the level bar results into error. The clinometer brackets extended only about 12 in. into the concrete and where these brackets were used for placing the level bar stations they would be even more subject to the effect of distortion from a plane section

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than would the I-beams which were used in the foundation rock. Above the bottom, however, the distortion, due partly to the heavy bearing of the toe of the dam on the foundation, would gradually disappear and the agreement, noted in a previous paragraph, between the inclination of a section at the 10-ft. elevation, as determined by the clinometer with that from the level bar, should be expected.

In Fig. 93 is shown the change in inclination measured by the level bar at the bottom of the dam for different heights of water, also the differences between these inclinations and the average change of inclination in 5 ft. as determined by the clinometer. Also a scale is shown which, in connection with the curves for difference of inclination, shows the corresponding stress in the concrete. In these computations of stress the modulus of elasticity was taken at 3,6000,000 lb. per sq. in. (See Sec. 96), and a plane section was assumed to remain plane even at the bottom of the dam. These stresses are much greater than those determined from the strains measured on the surface of the dam with the strain gage. The strain-gage strains are shown in Fig. 43, 44, and 45.

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88. Strains Due to Applied Load. The lines of equal vertical and horizontal strain caused by the applied water pressure are shown in Fig. 94 to 101. These diagrams are largely self-explanatory and require little discussion. It is seen that the lines of equal vertical strain, in general, take a horizontal direction, suggesting some degree of uniformity in the bending moment at a given height in the various vertical elements. The lines of equal horizontal strain show considerable symmetry about the vertical center-line. They indicate that the points of greatest strain were at approximately the 25-ft. elevation and close to the abutments. They indicate a marked variation of strain along any horizontal and along any vertical element.

The locations of the vertical cracks on the center-line of the dam are shown in Fig. 94 to 99 and their influence on the strains may be seen in these figures.

From the strains measured in the four directions, 45 deg. apart on the downstream face of the dam, the amount and direction of the principal strains were computed. Any three of these four strains are sufficient for determining the amount and direction of the maximum strain. Thus four combinations are possible, and four different directions for the maximum strain will result if there is any error in the observations. The directions were computed for all four

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combinations at each point, and the average direction and amount are given in Fig. 102 for a head of 60 ft. of water. The extreme range of direction, as determined by the four independent computations, is shown in Fig. 103 for each point. The points to which the strains apply are the stations shown in Fig. 26 which consists of four gage lines, A, B, C, and D. These stations generally are at the elevations which are multiples of 10 ft. The arrow at any point in Fig. 102 shows the average of the directions for the individual values of the larger principal strain found for tests 11 and 12.

At any point there are, of course, two principal strains making a right angle with each other. For the sake of simplicity in Fig. 102, the directions and amounts of only the larger of the principal strains are shown. It will be seen from the figure that above the 20-ft. elevation, the direction of the larger principal strain is, in general, horizontal. Therefore, its amount should be nearly the same as the measured horizontal strain shown in Fig. 94. Comparison with Fig. 94 shows that this is the case. Below the 20-ft. elevation, the principal strains near the foundation were, in general, nearly perpendicular to the abutment line. At interior points they approach a horizontal direction. The principal exception to this statement is on the vertical center-line of the dam near the top and bottom. Here vertical cracks have occurred and the larger of the

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principal strains approach the vertical instead of the horizontal.

Since the larger of the principal strains generally took, approximately, a horizontal direction, the smaller must have taken, approximately, a vertical direction. Thus it will be seen that the division of the dam into imaginary vertical and horizontal elements for the purpose of analysis, not only serves the ends of simplicity, but also for the most part, furnished a means of determining the maximum stresses.

As a matter of fact, the largest principal strain shown in Fig. 102, is only 0.0002, while the largest strain in the horizontal direction, shown in Fig. 94, is 0.0003. The reason for this is that the horizontal strain of 0.0003, occurred at a station where only horizontal strains were measured, and the principal strains could not be determined at that point. The strain of 0.0003 corresponds to a stress of approximately 1100 lb. per sq. in. with a modulus of elasticity of 3,600,000 lb. per sq. in., and a Poisson's ratio of 0.15. Adding a probable stress of 100 lb. per sq. in. due to temperature change, as discussed in Sec. 74, indicates about 1200 lb. per sq. in. as a maximum stress present. Comparison of this stress with the strengths of concrete, shown in Table 5, and Fig. 10, shows that the concrete was nowhere near failure by compression.

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89. Effect of Flood. After completion of the program of tests the stop-logs closing the sluice beneath the dam were removed with the expectation that all water entering the reservoir would be drained through this opening. In November, 1926, a severe strom caused a flood which deposited so much rock, debris, and sand in the reservoir, that the undersluice was choked and the reservoir filled with with water. It is probable that the wrecking of the observation scaffolding helped to choke the undersluice. The depth of the deposit at the upstream face of the dam was generally about 40 ft. on January 5, 1927. Water has been running over the top of the dam from November, 1926 to July, 1927, the date of writing. At the height of the storm, the depth of water on the crest of the dam became as great as three feet. The water, together with the debris carried by it, washed out or battered down the observation platforms on both the upstream and the downstream faces of the dam. The stems of the 24-in. and 6-in. valves and the wiring of the electric telemeters close to the face of the dam were torn loose.

Fig. 104 shows the conditions at the dam shortly after this storm. It has not been possible to make any observations with clinometers or telemeters, but observations of deflections made with the theodolite on January 5, 1927 on points placed at the crest of the dam, indicate an increase in the downstream deflection, compared with that for the

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60-ft. head September 22, 1926, of about 0.1 in. at the center line and 0.2 in. at points 30 ft. away from the center-line.

An inspection of the dam on January 5, when about one-half in. of water was flowing over the crest, indicated that the cracks had not extended appreciably in length, nor had the crack in the upper part of the dam increased, noticeably in eidth. The crack near the bottom of the dam, however, was wider and whereas on October 1 it stopped about two in. from the bottom, on January 5 it had extended all the way to the bottom. On the whole, it may be said that the dam had been little affected by the severe test given it by this storm.

In February, 1927, another severe strom visited the vicinity of the dam. The water was again at a high elevation and it is most likely that the debris will be found to be considerably deeper than on January 5, but no detailed examination has been made since that time. Inspection from a distance, however, indicates no important change in conditions since the examination made on January 5.

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IX. FACTORS INFLUENCING REDUCTION OF DATA.

91. Sources of Error. In determining the moments from the observed data, the data are reduced, first, to the form of strains, and, then to stresses through the use of a term involving the modulus of elasticity and Poisson's ratio. Sometimes short cuts have been used in the operations, but these steps are involved directly or indirectly. In order to determine the moment in a member when the stress is known, use is made of the formula $M = \frac{fI}{z}$, in which M is the resisting moment in the member, f is the stress, I is the moment of inertia, and z is the distance from the neutral axis to the surface in which the stress f is known. This formula at best is an approximation, but a very close approximation for the case to which it applies; that is, a straight beam with parallel tension and compression faces. If the faces of the beam are not parallel or if the axis is curved, this formula does not apply and it becomes important to ascertain how great are the errors introduced by its use in determining the moments from the observed strains.

In the determination of moments from the deflection data of the clinometer, differentiations of the deflections are made in the direction in which the moment is to be determined. The deflection, however, includes whatever deflection there may be due to shear. Consequently, the resulting

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moment is in error by an amount which depends on the amount of the deflection due to shear.

In the determination of loads from the deflections and strains, differentiations performed along vertical and horizontal elements, will give bending loads which would be carried by these elements if there were no physical contact between them, that is, if the cantilevers were entirely separated from each other and from the arch ribs. Due, however, to the fact that the cantilevers and arch ribs are not free from each other, torsions are set up between adjacent cantilevers and adjacent arch ribs which transfer moment from one element to an adjacent element. Neglecting the possibility of this transfer of moment may introduce appreciable errors into the resulting loads. A study of the amount of torsion in the dam indicates, however, that for all places where information is available, the torsion is small and may be neglected in the interpretation of the data. This is discussed in Sec. 95.

The term involving modulus of elasticity and Poisson's ratio is an essential part of the determination of the stress and the moment at any point. However, it is almost impossible to determine exactly the value of the modulus of elasticity. Even if there were no chances for error in determining the stress-strain curve, because of the fact that the

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stress-strain curve is not a straight line, the modulus of elasticity is not constant and the assumption of a constant modulus introduces some error into the results. It is not practicable to make determinations of stress at all points taking account of the variability of the modulus of elasticity. As a matter of keeping the work within bounds, it is therefore, necessary to assume a constant modulus of elasticity and the determination of the most probable value for this modulus is a matter of the greatest importance in the complete interpretation of the test results. However, it is shown in a later paragraph how the modulus of elasticity may be determined from the test of the dam itself. These comments on the modulus of elasticity apply very largely to Poisson's ratio also.

The possible sources of systematic error mentioned above are discussed in the following paragraphs.

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92. Effect of Taper in Lower Part of Dam on Determination of Vertical Moments. It has been pointed out above that the use of the formula for beams with parallel surfaces in the determination of the moments from the strains introduces certain errors. An effort has been made to determine whether in the case of the test dam the error so introduced is sufficiently important to require that a correction for the taper in the lower part of the dam be applied. The error referred to is due to the fact that in the formula

$$M = \frac{fI}{z} \qquad (1)A$$

the stress f should be that in one of the two parallel faces of the beam. In the test dam the downstream face is not parallel to the axis of the beam and the strains measured with the strain gage and telemeter were observed on the sloping face. It should be noted, however, that the strains determined from the clinometers are strains normal to the horizontal section of the beam and no correction for the taper need be applied to the clinometer data.

It may be shown that with a beam of any homogeneous material in which stress is proportional to strain, the moment on any plane section which remains plane after bending is given by the expression:

$$M = \frac{1}{6} f_n bt^2 \qquad (2)A$$

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where f_n is the "normal" stress in the extreme fiber, that is, the stress normal to the section considered. However, the maximum stress at the surface must be parallel to the surface and both the shear and the stress at right angles to the surface must be zero*. For any particle under this condition of stress, the stress f_α at any angle α with the maximum stress f_m is

$$f_\alpha = f_m \cos^2\alpha$$

Therefore, if in equation (2)A the normal stress f_n makes an angle α with the maximum stress, that equation may be written

$$M = \frac{1}{6} f_m b t^2 \cos^2\alpha \qquad (3)A$$

In Professor Cain's article the section is taken so that the stress f_c makes an angle α with the normal to the section and there results the equation:

$$M = \frac{1}{2} f_c k j b t^2 \cos^2\alpha \qquad (4)A$$

Equation (4) is for a reinforced beam. For a homogenous beam:

$$k = \frac{1}{2} \text{ and } j = \frac{2}{3}$$

and this equation reduces to the form of (3)A.

* Wm. Cain, "Stresses in Wedge-Shaped Reinforced Concrete Beams". A.S.C.E. Vol. LXVII., p. 745, (1914)

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There seem to be inconsistencies in taking the section so that it makes an angle other than 90 deg. with the neutral axis, and a different assumption was made in which a circular cross-section was used which is normal to both surfaces of the beam and to the neutral axis, which lies in the center of the section. This section is shown in Fig. 105. The moment of the stresses everywhere normal to the circular section was considered. The maximum value of the angle θ occurs at the bottom of the dam and is about $\frac{3}{16}$ radians or about 10 deg. Neglecting terms involving the third and higher powers of θ, as being so small in proportion to the remainder of the resulting value of M that they may be neglected, the formula which results is:

$$M = \frac{1}{6} \frac{f_n b t^2}{\cos^2\theta} \left(1 + \frac{\theta^2}{6} \right) \qquad (5)A$$

where M is the moment at the center of gravity of the circular section, t is the thickness of the dam at the height of the downstream end of the section, f_n is the bending stress in the extreme fiber normal to the circular section, and θ is half the angle between the upstream face and the tangent to the downstream face, that is, it is half the angle α of equations (3)A and (4)A. The angle θ is in radians.

There is also a direct stress due to the inclination of the section assumed. The moment in terms of the total stress, f, the sum of the bending and direct stresses is given by the approximate equation:

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$$M = \frac{1}{6} fbt^2 \frac{1}{(1 - \frac{7}{6}\theta^2) + \frac{t}{6y}(\theta - \frac{2}{3}\theta^3)} \qquad (6)A$$

The y of this equation is the distance from the section under consideration to the point of application of the resultant force. It was determined by dividing the average moments found in tests 11, 12, and 13 at the various heights by the shears found from the same tests at the same heights.

The coefficient $\cos^2\alpha$ of equation (3)A and the coefficient

$$\frac{1}{(1 - \frac{7}{6}\theta^2) + \frac{t}{6y}(\theta - \frac{2}{3}\theta^3)}$$

of equation (6)A have been computed and are shown in Table 8.

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At the 5-ft. elevation of the dam, the angle θ is approximately .15 radians. It will be seen that for this elevation the correction for equation (3)A gives a value for the moment about 9 per cent less than that obtained by neglecting the effect of the taper. The correction to equation (6)A gives a value only about one per cent less than the uncorrected value. However, the center of gravity for the section to which the moment of equation (6)A applies, is lower than the section to which equation (3)A applies. This is illustrated in Fig. 105. The moments determined from the strains by means of equation (2)A, without correction for the taper, are shown as a solid line in Fig. 106. The corrected moments, as given by equation (3)A, are indicated on the same diagram by triangles. The corrected moments for equation (6)A are indicated by plus signs. It will be seen that the dotted line fits both sets of corrected moments equally well. The indication, therefore, is that the correction of the moments found by equation (6)A is equivalent, for this case at least, to the similar corrections given by equation (3)A, Cain's formula. While the corrections for equation (3)A are not large, they are easily made and have been applied to the moments determined from the strain gage data. See Fig. 106. They have not been applied to the moments determined by the clinometer data, because, as previously pointed out, the strains determined by the clinometer are, without correction, the strains in a vertical direction.

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93. <u>Effect of Curvature on Determination of Horizontal Moments.</u> The ordinary analysis of stresses and moments in a beam assumes that a plane section before bending remains plane during bending, so that the stress is proportional to the strain. Various analysis have been made on the stresses and moments in a curved beam. For the purpose of this examination, the analysis from Slocum and Hancock's "Strength of Materials", second edition, p. 191, is used. This analysis assumes for the curved, as well as for the straight member, that plane sections remain plane and that stress is proportional to strain. However, since, due to the curvature of the section, fibers on the concave side of the neutral axis are shorter than fibers on the convex side of the neutral axis, any given stress, although causing the same strain at all points will cause a smaller change of length in a given element on the inner side of the curve than on the outer side. Consequently, the neutral axis will be nearer the inside of the curved beam than the outside.

The resulting formulas, when applied to a curved beam of homogeneous material, and having rectangular section, may be reduced to the following approximate form:

$$M = \frac{1}{6} fbt^2 \frac{\frac{1}{2}\frac{t}{R} - 1}{\frac{1}{6}\frac{t}{R} + 1} \qquad (7)A$$

where R is the mean radius, and M, f, b, and t, are as in the preceding section.

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The coefficient $\dfrac{\frac{1}{2}\ \frac{t}{R} - 1}{\frac{1}{6}\ \frac{t}{R} \pm 1}$ is the coefficient by which the moment for a straight beam must be multiplied to obtain the moment for a curved beam which has the same extreme fiber stress.

The minus sign in the denominator of this equation applies to the extrados, and the plus sign applies to the intrados. In the test the strains were measured on the intrados, and, therefore, to determine the moment for the test dam, the plus sign should be used. The maximum value of $\frac{t}{R}$ for the test dam is at the bottom and is about.077. Using this value with the plus sign in the same equation:

$$M = .95\ \frac{1}{6}\ fbt^2 \qquad (8)A.$$

This indicates an error of from 5 to 3 per centwithin the lower 10 ft. of the dam, due to neglect of the curvature in determining the moments from the measured strains. Evidently, this is too small to be considered in view of much larger uncertainties below the 10-ft. level (See Sec. 122) and in view of the fact that the error is much smaller than this for all other positions. At the 30-ft. level, and higher, the error due to the use of the formula for straight beams is about 1 per cent.

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94. Effect on Stress Determinations in Dam of Neglecting Deflection Due to Shear. The tensile and compressive strains have been determined from the observed deflections on the assumption that all the deflection was due to bending. If some of the deflection was due to shear, then the strains found are larger than the true strains by an amount equal to the bending strain which would be computed from the deflection due to shear.

A possible method for approximating the errors introduced by the inclusion of the shearing deflection with the bending deflection may be outlined as follows:

(1) From the observed deflection curve, determine the moment and shear curves, assuming the deflection to be due entirely to moment.

(2) From the shear curve so obtained, determine the deflection due to this shear. Such shear and deflection curves are shown in Fig. 107.

(3) Deduct this shear deflection from the total deflection, obtaining the corrected deflection curve.

(4) Re-determine the shears from the corrected deflection, and if they differ from the shears obtained from the total deflections by an appreciable amount, determine a new shear deflection curve.

(5) When a shear deflection curve has been sufficiently established, assume temporarily that these are bending deflections, and deduce from it by double differentiation the corresponding moments and strains. These moments and strains will be

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the corrections to be applied to the moments derived on the assumption that the deflection was due entirely to bending.

These operations, which have been outlined for clearness, may be materially shortened due to the fact that, whereas, the shear curve is integrated once in obtaining the shear deflection, and this curve then is differentiated twice to obtain the moment, one differentiation of the shear curve gives the same result at once. This is illustrated by a specific case in which it is assumed that the entire load on a vertical element of the dam is carried to the foundation by that element. The shear at any height will then be the total horizontal pressure of the water above that height.

The shearing deflection dz within a height dy may be expressed as:

$$dz = \frac{Q}{A} \frac{dy}{G} \qquad (1)B$$

where,

Q= the total shear on the section
A= the area of the section
G = The shearing modulus of elasticity.

$$\frac{dz}{dy} = \frac{Q}{AG} \qquad (2)B$$

In the case of the test dam let:

y= distance downward from the top of the dam
t= thickness of dam at elevation y
w= weight of water per unit of volume.

Assume also that the variation in thickness from 24 in. at the 30-ft. height to 90 in. at the bottom follows a parabolic law.

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$$\frac{Q}{AG} = \frac{wy^2}{2tG} \quad \text{for a vertical element of unit width} \quad (3)B$$

From $y = 0$ to $y = 360$ in. $t = 24$ in.
From $y = 360$ in. to $y = 720$ in. $t =$

$$24 + \frac{1}{1963}(y - 360)^2$$

From $y = 0$ to $y = 360$ in.

$$\frac{dz}{dy} = \frac{wy^2}{48G} \quad \text{since} \quad \frac{dz}{dy} = \frac{Q}{AG} \qquad (4)B$$

$$\frac{d^2z}{dy^2} = \frac{1}{24}\,\frac{wy}{G} = \frac{M}{EI} = \frac{fI}{\frac{t}{2}EI} = \frac{f}{12E}, \text{ from which} \qquad (5)B$$

$$f = \frac{1}{2}\,\frac{E}{G}\,wy\Big]_0^{360} \qquad (6)B$$

From $y = 360$ in. to $y = 720$ in.

$$\frac{dz}{dy} = \frac{wy^2}{2G\left[24 + \frac{1}{1963}(y - 360)^2\right]} \qquad (7)B$$

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$$\frac{d^2y}{dy^2} = \frac{1}{2G} \frac{\left[24 + \frac{1}{1963}(y-360)^2\right] 2wy - wy^2 \frac{2}{1963}(y-360)}{\left[24 + \frac{1}{1963}(y-360)^2\right]^2} \quad (8)B$$

$$= \frac{M}{EI} = \frac{f}{E\frac{t}{2}} \quad (9)\ B$$

$$F = \frac{Et}{4G} \frac{2wy}{} \frac{\left[24 + \frac{1}{1963}(y-360)^2\right] y \frac{1}{1963}(y-360)}{\left[24 + \frac{1}{1963}(y-360)^2\right]^2} \quad (10)B$$

Since the denominator of the second term is t^2, f may be written

$$f = \frac{1}{2} \frac{E}{G} wy \frac{24 + \frac{1}{1963}(y-360)^2 - y \frac{1}{1963}(y-360)}{24 + \frac{1}{1963}(y-360)^2}, \text{ or} \quad (11)B$$

$$f = \frac{1}{2} \frac{E}{G} wy \frac{t - t - \frac{y}{5.45} + 90}{t} \quad (12)B$$

$$= \frac{1}{2} \frac{E}{G} wy \left. \frac{90 - \frac{y}{5.45}}{t} \right]_{360}^{720} \quad (13)\ B$$

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Equation (6) B gives the apparent tensile or compressive stress, f, corresponding to the computed shearing deflection between the top and mid-height of the dam. Equation (13)B gives similar values from mid-height to the bottom of the dam.

Equations (6)B and (13)B are valid only for certain assumed values of applied moment and of cross-section of the dam. In addition to this it is desirable to state the expression for f in general form so that it can be used with any applied moment and any section of the dam, regardless of whether the values for the moment are computed or observed.

As before, let z = shearing deflection in any height y.

Let Q = total shear on the cross-section.

From equation(1)B:

$$\frac{dz}{dy} = \frac{Q}{AG} \qquad (14)B$$

$$\frac{d^2z}{dy^2} = \frac{d\,\frac{Q}{AG}}{dy} = \frac{d\left\{\frac{\frac{dM'}{dy}}{AG}\right\}}{dy} \quad \text{since} \quad Q = \frac{dM'}{dy} \qquad (15)B$$

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Expressing the derivative indicated in (15)B:

$$\frac{d^2z}{dy^2} = \frac{AG\dfrac{d^2M'}{dy^2} - \dfrac{dM'}{dy}\,\dfrac{d(AG)}{dy}}{A^2\,G^2} \qquad (16)B$$

From general flexural relations:

$$\frac{d^2z}{dy^2} = \frac{M'}{EI} = \frac{2fI}{EIt} = \frac{2f}{Et} \qquad (17)B$$

If a section of unit thickness be considered, A of equation (16)B becomes t.

Therefore, from (16)B and (17)B:

$$\frac{2f}{Et} = \frac{t\dfrac{d^2M'}{dy^2} - \dfrac{dM'}{dy}\,\dfrac{dt}{dy}}{t^2G} \qquad (18)B$$

$$f = \frac{E}{2G}\left(\frac{d^2M'}{dy^2} - \frac{1}{t}\,\frac{dM'}{dy}\,\frac{dt}{dy}\right) \qquad (19)B$$

That equation (19)B gives the same results as does equation (13)B, for the condition assumed there, may be shown as follows:

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For that case below the 40-ft. elevation:

$$M = \frac{wy^3}{6} \quad \text{and} \quad t = 24 + \frac{1}{1963}(y - 360)^2$$

$$\frac{dM}{dy} = \frac{wy^2}{2}$$

$$\frac{d^2M}{dy^2} = wy$$

$$\frac{dt}{dy} = \frac{2}{1963}(y-360)$$

Substituting these values in equation (19)B:

$$f = \frac{E}{2G}\left\{wy - \frac{1}{t}\frac{wy^2}{2}\frac{2}{1963}(y - 360)\right\} = \qquad (20)B$$

$$\frac{E}{2G}\,wy\left\{\frac{t - \frac{y}{1963}(y - 360)}{t}\right\} \qquad (21)B$$

$$\frac{y}{1963}(y-360) = t - \left(90 - \frac{y}{5.45}\right) \qquad (22)B$$

From this equation (13)B resluts: that is,

$$\therefore f = \frac{E}{2G}\,wy\left\{\frac{90 - \frac{y}{5.45}}{t}\right\} \qquad (13)B$$

In the application of equation (19)B it is necessary to determine approximate values of $\frac{dM}{dy}$ and $\frac{d^2M}{dy^2}$ from the observed moment curves and to

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use $\frac{dt}{dy} = \frac{2}{1953}\left(y - 360\right)$ for the lower 30 ft. of the dam, (23)B

For the upper 30 ft. $\frac{dt}{dy} = 0.$ (24)B

As a typical case, the moment curve for the central vertical element with a head of water of 60 ft. has been determined from the strain data, and are shown in Fig. 106. Fig. 106 was prepared before the final values of moment on the section used in Fig. 126 had been determined. Any differences between the moments shown in Fig. 106 and 126 for this section are of no importance in this study. From the moments of Fig. 106 the shears have been obtained. From the shears the deflections due to shear were determined by approximate integration of equation (1)B. Also, from the moments the apparent bending stresses have been obtained from equation (19)B. The graphs of these values are shown in Fig. 107. It will be seen that the shearing deflections are everywhere small, and the apparent stresses due to the shearing deflections are small, down to the 2.5-ft. elevation. From the very nature of the shear deflection, however, the apparent stress must increase considerably below this point. In fact, assuming a perfectly rigid foundation it would reach an infinite value at the bottom. This will be clear if the value of $\frac{M}{EI}$ be thought of as proportional to the change in the slope of the tangent to the shear

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deflection ~~curve~~ within a unit distance. The shear deflection curve shows an abrupt angle at the bottom and this corresponds to an infinite apparent stress f. In the use of clinometer data, the shear deflection at the bottom is divided into so great a vertical distance that the error in the moments, due to the presence of shear deflection, is much less than might be expected from the above discussion. The errors are, in fact, so small that it does not seem likely that the accuracy of the resulting stresses will be appreciably increased by making the corrections indicated, and the errors so introduced have been neglected as being much smaller than the uncertainties due to lack of exact knowledge of modulus of elasticity, distortion from a plane section near the abutments, tilting of dam at abutments, etc.

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95. Torsion. In equation (3)C, there appears the term

$$2\frac{d^4 z}{dx^2 dy^2} EI.$$

This term enters because of the torsion, but it is not a load carried by torsion, although its form might indicate that it is. The load, w_x, carried in the x direction is the first derivative of the shear, V_x, for that direction. The shear, however, has a term which involves the derivative of the torsional moment, M_z, that is,

$$V_x = \frac{dM_x}{dx} + \frac{dM_z}{dy} \quad * \qquad (1)E$$

$$w_x = \frac{d^2 M_x}{dx^2} + \frac{d^2 M_z}{dx\, dy} \qquad (2)E$$

$$M_z = \frac{d^2 z}{dx\, dy} EI \qquad (3)E$$

$$\frac{d^2 M_z}{dx\, dy} = \frac{d^4 z}{dx^2\, dy^2} EI. \qquad (4)E$$

Equation (2)E shows how the expression involving torsional moment comes into the equation for the load, and equation (4)E shows how this portion of the load is related to the deflections. The same expression as that of equation (4)E enters the equation for load in the y-direction also, and since the total load is the sum of the x-load and the y-load, the torsion term in equation (3)C is twice the expression given in equation (4)E.

*Westergaard and Slater, "Moments and Stresses in Slabs". Proc. A.C.I. Vol. 17, p.426 (1921)

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Just as the loads carried by the horizontal element have been separated into parts called "equivalent load carried by direct compression", and "equivalent load carried by bending", this term may be called "equivalent load carried by torsion". It is, in fact, a correction, half of which should be applied to the load carried in the horizontal direction and half to the load carried in the vertical direction in order to obtain the total load, w.

The form of the expression for the equivalent torsional load shows that this load may be determined by successive differentiations of the deflection z, times EI. As long as the moment of inertia and the modulus of elasticity are constant, the order in which the differentiations with respect to x and y take place is immaterial; the result will be the same in either case. However, below the 30-ft. elevation the thickness of the dam varies with the elevation, y, and therefore the coefficient, EI, is not constant. In this portion of the dam, therefore, it is essential that the differentiations be performed alternately with respect to x and y. The equivalent torsional load determined by such differentiations of the deflections are given in Table 9, under the caption of Method Z. Because of the great horizontal distance between points at which the deflections were measured, it was necessary for this purpose to interpolate between the measured deflections to obtain deflections at closer intervals. The errors entering

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into such operations are large and the resulting torsional loads are correspondingly inaccurate. It will be seen, however, that none of the loads so determined are large.

The moments in the vertical and the horizontal directions may also be used to determine the torsional moments and the equivalent torsional loads. The equivalent torsional load may be written:

$$EI \frac{d^4 z}{dx^2 dy^2} = \frac{d^2}{dx^2}\left(EI \frac{d^2 z}{dy^2}\right) = \frac{d^2 M_y}{dx^2}$$

Since:

$$EI \frac{d^2 z}{dy^2} = M_y$$

The derivatives in the x-direction, of the bending moments, M_y, have been determined for the head of 60 ft. and the results are shown in Table 9, as equivalent torsional loads, under the caption of Method M_y. The moments, M_y, were determined from the strains measured with the strain gage and since the strain-gage measurements were taken at closer intervals in the horizontal direction than were the deflections, a correspondingly larger number of points were available for determination of the equivalent torsional loads by this method than from the deflections. Again, it will be seen that the loads so determined are quite erratic, but again, also, none of these loads are large.

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In the same manner the moments in the horizontal direction would give the equivalent torsional loads. Due to the possibility of some confusion between bending and direct strains in the horizontal direction, there is greater chance for error in this case than in the use of the vertical moments. The horizontal moments have not been used for this purpose.

The simplest and most direct source of information on the equivalent loads carried by torsion is found in the results of the strain-gage measurements taken in the diagonal directions. The torsional moment in a horizontal plane is * equal to one-half the difference of the bending moments in the two directions at 45 deg. with the horizontal. The strains measured in the 45-deg. directions contain components of the direct thrust in the horizontal direction, and, therefore, the total stress in the 45-deg. direction multiplied by the arm of the resisting couple would not give correctly the moment in the 45-deg. direction. However, since the moment desired is the difference between the moments in the two 45-deg. directions, and since the component of the thrust in one of the 45-deg. directions is the same as that in the other 45-deg. direction, the moment of the thrust will be eliminated completely in the difference of those moments. The torsional moments have been deter-

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* Westergaard and Slater, "Moments and Stresses in Slabs". Proceedings, Am.Conc. Inst. Vol. 17, P 426 (1921).

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mined in this manner from the bending moments in the two 45-deg. directions and differentiation with respect to the x axis and with respect to the y axis successively, has yielded the equivalent loads carried by torsion, shown in Table 9 under the caption of Method M_{xy}.

The equivalent torsional loads determined by the three methods above outlined, and their averages are given in Table 9, also the ratio of the average torsional load to the total water pressure for the points considered. In the column under the caption "Method", z indicates the determination of the torsion from the measured deflection. The Method M_y refers to the determination of the torsional loads from the moments in the direction of the y axis. The method M_{xy} refers to the determination of torsional loads from the bending moments in the two diagonal directions.

It will be seen from this table that by all of the methods the equivalent torsional loads are small. In determining loads due to bending, it is believed that the error involved may be as large as ± 100 lb. per sq. in. in most cases, and only a few values of the equivalent torsional loads are larger than this. The error of the equivalent torsional load in lb. per sq. ft. is of more significance than the error in terms of per cent of the total load. For example, close to the top of the dam a very small error in lb. per sq. ft. would be a very large percentage of the total pressure at the

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point considered and yet it would not be of great significance. However, it is of some interest to know the ratio of the equivalent torsional load to the total water pressure and these ratios are shown in Table 9. Generally, the ratios are small. Only a few large percentages occur and these are near the top of the dam where the total water pressure is small.

In view of the smallness of the equivalent torsional load, and in view of the difficulty of obtaining accurate values for it, this load has been omitted in the following study of the data of the test dam.

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96. Modulus of Elasticity. During the construction of the dam, control specimens were made at frequent intervals for the purpose of determining the modulus of elasticity. These control specimens consisted generally of 6 by 12 in. cylinders. In addition, however, a beam 12 in. square and about 8 ft. long was made for determination of the modulus of elasticity. The beam was tested by measuring the deflection under a known load and computing the modulus of elasticity from its relation to load and deflection. Most of the control cylinders used for the determination of modulus of elasticity were sent to the University of California at Berkeley, California, for the tests; however, telemeters were embedded in four of the cylinders and these were tested at the damsite.

The testing of the cylinders yielded a variety of results. The one thing common to all the tests was the shape of the stress-strain curve. In the effort to make the most out of these tests, the most probable values of the strain were determined from all cylinder tests, including those with telemeters embedded. A most probable stress-strain curve was thus arrived at, and is shown in Fig. 108. This curve gives an initial modulus of elasticity of 3,800,000 lb. per sq. in.; a modulus of elasticity of 2,700,000 lb. per sq. in. at the maximum stress of 900 lb. per sq. in.; and a modulus of 3,000,000 lb. per sq. in. at a strain of .0002. This latter strain represents approximately the highest found in the dam so that the range of modulus to be expected in the dam would be from about 3,800,000 lb. per sq. in. to 3,000,000 lb. per sq. in. if this curve is used as

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the criterion. The values of the modulus of elasticity are given in Fig. 108.

The test beam under a stress of 216 lb. per sq. in. gave a secant modulus of elasticity of 4,000,000 lb. per sq. in. for the earlier tests and of 6,000,000 lb. per sq. in. for the later tests. Because the modulus of 6,000,000 at the later ages seemed too high, check tests were made on the beam and this value was confirmed. It is difficult to reconcile the variations in modulus of elasticity between the beam and the test cylinders. However, as shown in Fig. 109, it was found from tests of the cylinders in which telemeters had been embedded that under a stress of 500 lb. per sq. in. the secant modulus determined immediately upon application of the load was about 5,000,000 lb. per sq. in., but that with the load maintained constant on the specimen, the modulus fell rapidly and consistently to a value of about 2,900,000 lb. per sq. in., within about four hours, as shown in Fig. 109. Maintaining the load constant for about an hour gave a modulus of about the same magnitude as that determined from the cylinders tested at Berkeley. In the testing of the beam, the deflection was determined almost instantaneously upon application of the load and it would be expected from the information obtained from the cylinders having telemeters embedded in them, that the modulus of elasticity determined from the beam would be higher than it was from the cylinders tested at Berkeley.

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Because of the importance of a knowledge of the modulus of elasticity in the interpretation of the results, a further effort was made to obtain information on its value. As shown by equation (5)C, Section 101, the load on the dam, as determined from the deflections and strains, is given in terms of the modulus of elasticity. Since the load actually applied by the water pressure is known, it is possible for any given case to use a modulus of elasticity which would make the computed load agree with that known to have been applied. However, if the wrong modulus is used in order to bring about this agreement at one point, this modulus will not give the correct value of the load at some other point. Accordingly, after determining the loads in terms of modulus of elasticity for a large number of points, it was found that the value which gave the closest agreement between computed loads and applied loads for a large number of points was 3,600,000 lb. per sq. in. with Poisson's ratio taken at 0.15. While this is not a perfect agreement with the results of the cylinder tests, it is sufficiently close to give confidence in the general concordance of the data.

This value was used for the reduction of all strains to stresses whether in compression or tension. No determination of modulus of elasticity in tension was made. Other investigations gave varying conclusions, but in general the modulus of elasticity in tension has been found to be equal to

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or slightly less than that in compression. There seems to be little or no greater reason for using different values of the modulus of elasticity in tension and compression, respectively, than for using a different modulus for each different degree of compression. The effects of using a variable modulus were studied, and they were so small that it was concluded that any possible gain in accuracy would be more than offset by the confusion and complexity attending the use of a variable modulus in working up the test data.

In order to interpret the measured yielding of foundation and abutments, it became necessary to know the modulus of elasticity of the granite of which they were composed. A block of granite was cut from the foundation rock at a place about 20 ft. downstream from the dam at an elevation of about 15 ft. It was sent to the University of California and there two 6 by 12-in cylinders were cut from it. The longitudinal axis was parallel to the bedment plane in one of the specimens and perpendicular to it in the other specimen. Fig. 110 gives the stress-strain curves, Poisson's ratio, and the modulus of elasticity for these specimens. The average value of the modulus of elasticity for the lower stresses was about 2,000,000; and for the higher stresses it was about 2,500,000 lb. per sq. in. The value of 2,000,000

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lb. per sq. in, was used in computing the movement of the bedrock in Sec. 85. The specimen of granite from which the cylinders were cut was sound and of good appearance in every respect. Although the stress was carried only to 3,000 lb. per sq. in., in determining the modulus of elasticity the specimen would undoubtedly have taken much greater load.

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97. Poisson's Ratio. Equation (9) page shows that Poisson's ratio, μ, the ratio of the lateral to the longitudinal strain in a member under longitudinal stress, enters, along with the modulus of elasticity, into the expression for load, as determined from deflections and strains. Values of μ were determined for all of the cylinders which were sent to the University of California for determination of modulus of elasticity. The value of Poisson's ratio varied from as high as 0.3 to as low as 0.1, but the variation showed little relation to the variation in modulus of elasticity. Fig. 111 gives specimen lateral and longitudinal stress-strain diagrams and corresponding diagrams of Poisson's ratio. Although separate diagrams are given for specimens cured in damp sand and those cured at the dam site, it is not supposed that the differences represent the effect of the method of curing. In one case the value of Poisson's ratio increased with load, while in the other case it decreased. This difference seems to have been accidental, for in some cases the same cylinder, tested at different times, showed in one instance increase in Poisson's ratio, and in the other a decrease, with the increase in load. For the low loads, both longitudinal and lateral strains are extremely small and even a small error in observed strains would have a large effect on Poisson's ratio, and it seems, therefore, that little weight should be given to the lower values.

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With a stress of 500 lb. per sq. in., the average value of Poisson's ratio for all the tests was 0.17. At the time in the progress of the preparation of the report when it became necessary to decide on a value of Poisson's ratio for the reduction of the data of the test dam, the available information on the lateral and longitudinal strains gave an average value for Poisson's ratio of 0.15 and this value was used. This value differed so little from the final average of 0.17, the difference had so slight an effect on the results, and the variation of individual values from each other were so great, that there was no reason for changing to the final average and recalculating the stresses and loads.

Fig. 110 gives Poisson's ratio for a sample of the granite taken from the foundation rock. The values vary rather uniformly from as low as 0.1 at low stress to as high as 0.2 high stress. The specimens on which the determinations were made are described elsewhere.

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X. GENERAL METHOD OF ANALYZING TEST DATA.

101. General Outline. In carrying the load, there are in the arch dam several distinct, though interdependent, units of structural action.

The dam may be assumed to be made up of a system of horizontal strips (arches) and a system of vertical strips (beams)(usually termed cantilevers) intersecting each other and acting at the same time and in combination with each other. If it were known how much load was carried by each element of each system, the design of the dam would be simple; therefore, it seems that the principal aim in analyzing the test data should be that of determining how much load is carried by each of the elements mentioned above. The conception of the behavior may be simplified as follows:

Assume a 1-ft. square element of the dam at a mean depth of 10 ft. The total pressure on this element will be approximately 625 lb. This total pressure must be carried in some manner to the abutments or foundation of the dam. Due to curvature of the horizontal element, a portion of the load will be carried horizontally towards the abutment by a direct thrust, as arch action.

Because the horizontal element is not free to move at the support, but does deflect at the center, there is, obviously, a bending moment and therefore a shear in the horizontal element. It is evident, therefore, that another part of the load from the 1-ft. square element is carried by shear toward the abutments.

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Similarly, in the vertical direction a portion of the load travels from the element under consideration by causing shear in the vertical element, and, consequently, a bending moment. These latter bending moments constitute the, so-called, cantilever bending moments.

Fig. 112 is intended to represent diagramatically the manner of distribution of load to the elements of the dam. Fig. 112 (a) represents an unstrained horizontal element in its original position with full water pressure applied to it. By the principle of superposition of loads it may be assumed that the portion of the load which causes direct compression is applied first, causing the uniform radial deflection shown in Fig. 112 (b), as would be the case if the dam were a part of a complete cylinder and free to deflect radially at all points. The uniform water pressure represented as causing this constant direct compression is also shown in Fig. 112 (b). The equivalent bending load which pulls the ends back to the original position and causes a bending deflection, as shown in Fig. 112 (c), may then be assumed to be applied independently of the load causing direct compression. Not all the load is accounted for by the equivalent load carrying direct thrust and that carried by bending. The remainder, carried by vertical bending, also is shown in Fig. 112 (c). Fig. 112 (d) represents an unstrained vertical section of the dam in its original position with the full water pressure applied to it.

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Fig. 112 (e) represents the same element deflected by the portion of the load carried by vertical bending, and shows the corresponding load causing the bending.

The remainder of the analysis is aimed at determining from the data of the test the values of the different components of the load illustrated in Fig. 112.

It is possible to separate completely from other portions of the load, the load which is carried by the bending in the vertical elements of the dam. It is not possible, however, to separate completely between the loads carried by the bending of the horizontal elements and the load carried by direct compression in the horizontal elements, since a load which at one point causes only a direct compression will in general cause a bending moment at another point and vice versa.

In the fundamentas analysis of a flat plate the equation of equilibrium for load on any element is:

$$\left\{\frac{d^4z}{dy^4} + 2\,\frac{d^4z}{dy^2\,dx^2} + \frac{d^4z}{dx^4}\right\}\frac{EI}{1-\mu^2} = w, \text{ where} \qquad (1)C$$

z is the deflection of the plate, that is, movement in the direction of the "z" axis.

y is the distance in the direction of the y axis.

x is the distance in the direction of the x axis, which makes a right angle with the y axis.

E is the modulus of elasticity of the material.

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I is the moment of inertia of the section.

μ is Poisson's ratio for the material.

w is the total unit pressure on the plate at any given point.

In equation (1)C the term:

$$\frac{EI}{1-\mu^2}\frac{d^4z}{dy^4}$$, represents the load carried by bending

in the y direction. The term:

$$\frac{EI}{1-\mu^2}\frac{d^4z}{dx^4}$$, represents the load carried by bending

in the x direction, and the term:

$$2\frac{EI}{1-\mu^2}\frac{d^4z}{dy^2\,dx^2}$$, represents the equivalent load

carried by torsion.

The arch dam may be considered to have the combined properties of a flat plate and an arch in direct compression. If it were possible to separate completely between the load carried by bending and the load carried by direct compression in the horizontal element, a new equation of equilibrium could be set up which would be the same as equation (1)C, except for the addition of a term representing the load carried by direct compression in the horizontal elements, and except for a correction in the load carried by bending in the horizontal elements due to the curvature of these elements.

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Although the load carried by bending and the load carried by direct compression in the horizontal elements cannot be entirely separated, it can be shown* that the sum of the two loads at any point in the horizontal element may be represented by the expression:

$$\left\{\frac{d\ z}{dx^4} + \frac{1}{R^2}\ \frac{d^2z}{dx^2}\right\}\frac{EI}{1-\mu^2} + \frac{P}{R} \quad \text{where} \qquad (2)C.$$

P is the total horizontal thrust per unit of height at the section of the element considered, and

R is the radius of curvature of the horizontal element.

This being true, a new equation of equilibrium for the total load carried on any small rectangular element of the arch dam whose width is dx and whose height is dy, may be represented by equation (3)C.

* See Hans Lorenz, "Technische Elastizitätslehre", p. 241 (1913), and John Prescott's "Applied Elasticity", p. 288 (1924).

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$$\left\{ \frac{d^4z}{dy^4} + 2\frac{d^4z}{dy^2\,dx^2} + \frac{d^4z}{dx^4} + \frac{1}{R^2}\,\frac{d^2z}{dx^2} \right\} \frac{EI}{1-\mu^2} + \frac{P}{R} = w. \qquad (3)C.$$

It will be convenient in determining from the test data the most probable value of the modulus of elasticity, to eliminate the modulus from all the terms derived directly from the test data and to place it on the right-hand side of equation (2)C.

It may be shown that the horizontal pressure P of equation (2) C is given by the expression:

$$P = \left(\frac{\varepsilon_x + \varepsilon_y\mu}{1-\mu^2} \right) EA, \text{ where,} \qquad (4)C$$

ε_x and ε_y are the horizontal and vertical strains, due to the axial force P, and A is the area of the section on which it acts. Substituting this value of P in the equation (3)C, and dividing by

$$\frac{EI}{1-\mu^2}$$

$$\frac{d^4z}{dy^4} + 2\frac{d^4z}{dy^2\,dx^2} + \frac{d^4z}{dx^4} + \frac{1}{R^2}\,\frac{d^2z}{dx^2} + \frac{A}{R\,I}\left\{ \varepsilon_x + \varepsilon_y\mu \right\} = w\,\frac{1-\mu^2}{EI} \qquad (5)C.$$

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From the test data, the properties of the section of the dam, and the head of water, all the terms of equation (5)C, except E are known. It is, therefore, possible to solve the equation for E and thus determine the value of E which best fits the conditions at the various sections of the dam. The values so obtained correspond very well with the modulus of elasticity determined from the test cylinders. Further discussion of the modulus of elasticity is given in other paragraphs.

The portion:

$$\left\{\frac{d^4z}{dx^4} + \frac{1}{R^2}\cdot\frac{d^2z}{dx^2}\right\}\frac{EI}{1-\mu^2}$$

of equation (3)C, may be said to represent the equivalent load from the element under consideration which is carried by bending in the horizontal direction.

Of this expression the term:

$$\frac{1}{R^2}\ \frac{d^2z}{dx^2}\ ^*$$

enters because of the curvature of the horizontal elements. In the test the maximum value of this term represents a load of only 2 lb. per sq. ft. This load is represented, in the portion of the dam having a thickness of 2 ft., by a strain of only three millionths and this is too small to be measured accurately.

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* See Timoshenko and Lessels, "Applied Elasticity", p. 230, (1925).

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The term $\frac{P}{R}$ may be said to represent the equivalent load carried by direct compression in the horizontal element. It will be desirable to define what the terms representing the equivalent bending load and the equivalent load carried by direct compression are, since it has been stated that they do not actually represent the bending load, and the load carried by direct compression respectively.

The equivalent bending load is an imaginary load, which, if integrated with respect to the distance along the arc, will give the moment at all points along the arc. Or, it may be stated as the load which, if distributed along a straight beam of the same length as that of the arc, will give the same moments at all points in the length of the straight beam as are found in corresponding positions along the arc.

The expression $\frac{P}{R}$ has been called the equivalent load causing direct compression. It will be seen from its make-up that it is the radial load per unit of length of arc, which, if applied to a complete relatively thin circular ring of a uniform cross-section, would cause the same direct compression in the ring as that which is actually found in the horizontal element under consideration.

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XI. EQUIVALENT LOAD CARRIED BY DIRECT THRUST.

111. Sources of Information. There are several sources of information regarding the amount of the load carried by direct thrust.

(1) The average strain for the upstream and downstream sides at any section, as given by the telemeter observations, is a measure of the direct strain, and since the thrust E. ε. t. is connected with the load by the equation:

$$w = \frac{E.\ \varepsilon.\ t}{R} \qquad (1)D$$

known as the "cylinder formula", it is possible to compute the load w resisted by the thrust where ε is the measured strain due to the thrust; t is the thickness of the dam, and R is the radius of curvature. Fig. 113 shows the average strains, ε for all parts of the dam where telemeter measurements were available in tests 11, 12, and 13, that is, with the water in the reservoir at an elevation of 60 ft.

(2) The difference between the total strain observed by the strain gages and the bending strain from the clinometers gives the strain ε due to direct thrust at various points of the arch, and from equation (1)D, the load w, may be determined.

Fig. 114 to 117, inclusive, show the total strain at the 30-ft. elevation as measured with the strain gage under different heads of water.

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They show also the bending strains as given by the clinometer, radius meter, and telemeter. Since the upper curves represent only bending strain and the lower curve represents the sum of the bending and direct strains, the difference between the two average curves (represented by the vertical distance between curves) is the strain due to the direct axial compression in the arch ring. The indication is that this direct compression was nearly uniform throughout the length of the arch rib. For later computations, a constant value of the compression at various portions of any arch ring has been assumed.

(3) The direct strain due to thrust may also be determined from the deflections by the methods outlined in Sec. 112.

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112. Determination of Thrust from Deflections.

Assume the arch to be straight between the points of measurement of deflection. Let Fig. 118 represent any segment of the arch. In general, the points A and B will not deflect radially. Assume that the line AB has deflected to some other position, $A_1 B_1$, Fig. 118, and then has been pushed back parallel to itself along a tangent at its midpoint until the point A lies at A' on the radial line AO, Fig. 118 (a) and 118 (b). This motion of translation does not in any way affect the strains in the segment and the analysis may be carried out as though the point A had deflected radially to A'.

Due to the yielding of the abutment, the point B' moves axially a distance DB' away from the line BO. Call this distance Δx. Rotate the line $A' B'$ about its center through the angle $\Delta\phi$ to the position $A'' B''$ parallel to the position of the line AB. The angle $\Delta\phi$ is so small that the arcs $A'A''$, $B'B''$, and $D D'$ will be considered straight lines making right angles with the line $A'' B''$.

$$\text{Then } A'' E = A' A'' \frac{\Delta \phi}{2} = \frac{A' B'}{2} \Delta\theta \frac{\Delta \phi}{2}$$

$$D'F = D D' \frac{\Delta\phi}{2} = \left(\frac{A' B'}{2} - \Delta x\right) \Delta\theta \frac{\Delta\phi}{2}$$

$$A''E - D'F = \Delta x \cdot \Delta\theta \frac{\Delta\phi}{2}$$

Since $\Delta x \Delta\theta \frac{\Delta\phi}{2}$ is a differential of the second order, it may be neglected, therefore $A'' E = D' F$ and $A' B' = E F + \Delta x$.

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From this it is seen that the change in the center-line length of a segment of an arch may be taken as the difference between the original length and the length of a chord parallel to the original position plus Δx, the change in length due to the movement of the abutment. This difference in length is $z\Delta\phi + \Delta x$.

Measurements of deflection were taken on the downstream face of the dam and the movement of the abutments parallel to a long chord were measured. Obviously, in determining the shortening due to direct compression, the change in width of cracks and the difference in changes of length on the center-line and the downstream face must be taken into account.

In Fig. 118(c) let s represent the original length of a segment of the arch on the downstream face, and let s' represent the length after the load has been applied. Assume that the loading has deflected the segment into a position not parallel to its original position and that after deflecting, the segment has been rotated about its central point as in Fig. 118(c)and moved tangentially as in Fig.(118)A, into a position parallel to its original position. It has been shown in the preceding paragraph that in determining the strains due to the loading, the segment may be treated as though it deflected parallel to its original position an amount just equal to the actual deflection of the center of the segment.

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Let Δl = the total change of length of the arch rib on the downstream face.

Let ϵ_a = the strain due to axial compression. This will be taken to be the uniform strain as the data of the test has shown that it is nearly so.

Let Δu = the total change in width of cracks on the downstream face in length s.

Let Δu_a = the change in length of the downstream face of a segment of length s due to axial opening or closing of cracks.

Let ϕ = the angle from the center to the point under consideration.

Let θ = the angle subtended by any complete segment s.

Let ϕ_1 = half the total central angle of the arch.

Let $\Delta\theta$ = total rotation of one end section of the segment relative to the other end, or the sum of the rotations of the abutments.

Let $\Delta\theta_c$ = rotation due to differences in change of width of cracks at the downstream face and the center-line.

Let $\Delta\theta_\epsilon$ = rotation due to bending strain, ϵ_b, in downstream face.

Then: $\frac{t}{2}\Delta\theta$ = change of length in the downstream face due to rotation $\Delta\theta$.

And: $\frac{t}{2}\Delta\theta_\epsilon$ = change of length of downstream face due to the rotation $\Delta\theta_\epsilon$.

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In Fig. 118(c) it is seen that:

$$s = s' + z\theta - \left(\Delta x + \frac{t}{2} \Delta\theta \right) \qquad (2)D$$

$$s - s' = z\theta - \left(\Delta x + \frac{t}{2} \Delta\theta \right) \qquad (3)D$$

$s - s'$ is the change of length within the distance s at the downstream face of the segment. It includes the change of the lengths of the "extreme fiber" due to rotation and to axial movement.

Both the axial movement and the rotation may be made up partly by strain and partly by opening or closing of cracks. It may, therefore, be written:

$$s - s' = \frac{t}{2} (\Delta\theta - \Delta\theta_c) + \frac{t}{2} \Delta\theta_c + \varepsilon_a s + \Delta u_a = \qquad (4)D$$

$$= \frac{t}{2} \Delta\theta + {}_a s + \Delta u_a \qquad (5)D$$

$$\Delta l = \sum_{-\phi_1}^{+\phi_1} (s - s') = \sum_{-\phi_1}^{+\phi_1} \frac{t}{2} \Delta\theta + \varepsilon_a l + \sum_{-\phi_1}^{+\phi_1} u_a =$$

$$\sum_{-\phi_1}^{+\phi_1} z\theta - \sum_{-\phi_1}^{+\phi_1} \left(\Delta x + \frac{t}{2} \Delta\theta \right) \qquad (6)D$$

$$\varepsilon_a l = \sum_{-\phi_1}^{+\phi_1} z\theta - \sum_{-\phi_1}^{+\phi_1} \left(\Delta x + \frac{t}{2} \Delta\theta \right) -$$

$$\sum_{-\phi_1}^{+\phi_1} \Delta u_a - \sum_{-\phi_1}^{+\phi_1} \frac{t}{2} \Delta\theta \qquad (7)D$$

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The term $\sum_{-\phi_1}^{+\phi_1} (\Delta x + \frac{t}{2} \Delta\phi)$ represents the integrated axial movement of the elements of the arch. Since the direct stress has been found to be very nearly constant, the rate of change of movement will be taken as the average, that is, $\frac{\Sigma (\Delta x + \frac{t}{2} \Delta\phi)}{l}$ The movement in the direction of the long chord within any differential distance dx will be

$$\frac{\Sigma (\Delta x + \frac{t}{2} \Delta\phi)}{l} dx \cos\phi =$$

$\Sigma \frac{(\Delta x + \frac{t}{2} \Delta\phi)}{l} R d\phi \cos\phi$, (since $dx = R d\phi$). The component along the long chord was measured as the spreading of abutments, Q, and $\frac{R}{l} = \frac{l}{2\phi_1}$.

Therefore:

$$Q = 2 \int_0^{\phi} \sum_{-\phi_1}^{+\phi_1} \frac{(\Delta x + \frac{t}{2} \Delta\theta)}{2\phi_1} \cos \phi \, d\phi =$$

$$\sum_{-\phi_1}^{+\phi_1} (\Delta x + \frac{t}{2} \Delta\theta) \frac{\sin \phi_1}{\phi_1} \qquad (8)D$$

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$$\sum_{-\phi_1}^{+\phi_1} \left(\Delta x + \frac{t}{2} \Delta\theta \right) = Q \frac{\phi_1}{\sin\phi_1} \qquad (9)D$$

By definition:

$$\Delta u_a = \Delta u - \frac{t}{2} \Delta\theta_c \qquad (10)D$$

Substituting in equation (8)D:

$$\sum_{-\phi_1}^{+\phi_1} \left(\Delta x + \frac{t}{2} \Delta\theta \right) = Q \frac{\phi_1}{\sin\epsilon\phi_1} \qquad (11)D$$

And

$$\Delta u_a = \Delta u - \frac{t}{2} \Delta\theta_c \qquad (12)D$$

$$\varepsilon_a t = \sum_{-\phi_1}^{+\phi_1} z\theta - Q \frac{\phi_1}{\sin\phi_1} -$$

$$\sum_{-\phi_1}^{+\phi_1} \left(\Delta u - \frac{t}{2} \Delta\theta_c \right) - \sum_{-\phi_1}^{+\phi_1} \frac{t}{2} \Delta\theta \qquad (13)D$$

$$= \sum_{-\phi_1}^{+\phi_1} z\theta - Q \frac{\phi_1}{\sin\phi_1} - \sum_{-\phi_1}^{+\phi_1} \Delta u -$$

$$\sum_{-\phi_1}^{+\phi_1} \left(\frac{t}{2} \Delta\theta - \frac{t}{2} \Delta\theta_c \right) \qquad (14)D$$

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$$\sum_{-\phi_1}^{+\phi_1}\left(\frac{t}{2}\Delta\theta - \frac{t}{2}\Delta\theta\right) = \sum_{-\phi_1}^{+\phi_1}\frac{t}{2}\Delta\theta \qquad (15)D$$

Therefore:

$$\varepsilon = \frac{1}{z}\left\{\sum_{-\phi_1}^{+\phi_1} z\theta - Q\frac{\phi_1}{\sin\phi_1} - \sum_{-\phi_1}^{+\phi_1}\Delta u - \sum_{-\phi_1}^{+\phi_1}\frac{t}{2}\Delta\theta\right\} \qquad (16)D$$

The expression in equation (16)D for ε_a is slightly in error due to the assumption (for the purpose of determining

$$\sum_{-\phi_1}^{+\phi_1}\left(\Delta x + \frac{t}{2}\Delta\theta\right)$$

that the rotation $\Delta\theta$ per unit of length along the arch is constant. The error from this source is probably less important than the fact that the rotation

$$\sum_{-\phi_1}^{+\phi_1}\frac{t}{2}\Delta\theta$$

due to the bending stresses is not known with any degree of certainty. The error from the former source may be eliminated by assuming that of the total movement along the axis,

$\sum_{-\phi_1}^{+\phi_1}\left(\Delta x + \frac{t}{2}\Delta\theta\right)$ only the portion $\sum_{-\phi_1}^{+\phi_1}\Delta x$ due to

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the axial compression has a constant rate of increase.

The portion $\sum_{-\phi_1}^{+\phi_1} \frac{t}{2} \Delta\theta$ is due to the rotation accompanying

the bending and change in crack width.

By a solution similar to the former, this assumption gives for the axial strain:

$$\epsilon_a = \frac{1}{t} \left\{ \Sigma z\theta - Q \frac{\phi_1}{\sin\phi_1} - \Delta u - \frac{t}{2} \Delta\theta - \right.$$

$$\left. \left(\Sigma \frac{t}{2} \Delta\theta - \frac{\phi_1}{\sin\phi_1} \Sigma \frac{t}{2} \Delta\theta \cos\theta \right) \right\} \qquad (17)D$$

It will be seen that this expression is the same as that of equation (16)D, except for the addition of the last two terms (in parenthesis). If the rotation $\Delta\theta$ be assumed to be uniform, the last two terms become equal, and therefore their sum becomes zero, and this checks equation (16) D.

The use of equation (17)D will possess no advantages over the use of the equation (16)D for the determination of axial strain. Either equation involves the separation of the rotation into the parts due to bending strain and to change in crack width. These parts are not known, and in fact the total rotation is known only approximately. The

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rotation due to bending strains has been determined approximately from a study of the combined data of the strain gage, the telemeters, and the clinometers, and has been used in the determination of the axial strains from the strain-gage data. This rotation may be used in equation (16)D to determine the axial strains and the reliability of the results must finally be judged by a comparison of the axial strains so obtained with those obtained from the strain-gage data. This comparison may be made directly by comparing the points in Fig. 119 to 122. The agreement between these two methods and with the axial strains determined as the algebraic sum of the strains from the telemeters in the upstream and the downstream sides of the dam (also shown in Fig. 113) is good enough to establish confidence in the results.

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113. Results. Fig. 119 to 122, inclusive, give the direct strains and the corresponding equivalent loads carried by direct compression for the 60, 50, and 40-ft. heads of water. The plotted circles represent the most probable values for the direct strains, giving weight to the individual values shown by other symbols and to the general shape of the curves. The strains were determined by the methods outlined in Sec. 111 and 112, and for those obtained from the clinometer only an average value for any elevation was available. For the strain gage and telemeter, values at various points along the rib might have been obtained, but the degree of uniformity of the average strains at the 30-ft. elevation (see Fig. 114 to 117) is sufficient to warrant using an average for these elevations also. This is fortunate, since the analysis of the data is simplified by assuming a constant direct strain in the horizontal ribs, and since, for other than the 30-ft. elevation, the data are not sufficiently complete to warrant using other than average strain. The agreement of the strains derived by different methods is generally fair, although in some instances erratic results were obtained.

The water pressure (or equivalent direct load) necessary to produce the direct strains, shown in Fig. 119 to 122, is shown in the right hand portion of each figure. For the 60-ft. head, shown in Fig. 119, the equivalent load

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carried by direct thrust is found to increase from a small value at the top to a maximum at about the 35-ft. elevation, and to decrease below that elevation to a minimum at the 15-ft. elevation. It then increases again reaching another and greater maximum at about the 5-ft. elevation. As a crack had formed at the bottom of the dam at this time, it seems that this may be responsible for the peculiar behavior near the bottom.

For the 50 and 40-ft. heads, shown in Fig. 120 to 122, the form of load curve is quite different from that in Fig. 119, especially for the lower elevations. In all three cases, the load increased from zero at the top to a maximum at about the 15-ft. elevation, and then decreased regularly to an assumed value of zero at the bottom.

Fig. 121 and 122 show the equivalent loads carried by direct thrust with a 40-ft. head of water against the dam, both after and before the crack occurred on the vertical center line of the dam in the upper 10 to 20-ft. of its height. Comparison of Fig. 121 and 122 indicates that at nearly all elevations more load was carried by direct compression after than before this crack occurred. This does not seem unreasonable, since the ability of the dam to carry a load by bending was decreased by the formation of the crack, making it necessary that a greater

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equivalent load be carried by direct compression after than before the formation of the crack. A comparison of the proportion of the total load carried by direct thrust with the proportion carried by bending in the vertical and horizontal directions may be seen in Fig. 130 to 133. This subject is discussed in Chapter XIV.

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XII. EQUIVALENT LOAD CARRIED BY HORIZONTAL BENDING

121. General Method. The bending strains in the horizontal arch ring may be determined from measurements with the clinometer, telemeters, and the radius meter, and may also be obtained from the total strains measured by the strain gages by subtraction of the direct strain. The radius meter, however, was used only at the 30-ft. elevation of the dam and will, therefore, not give any information on the bending except at this elevation. The telemeters were placed at every 10-ft. elevation, but so far apart in horizontal direction that detailed information regarding the bending along the arch ring cannot be obtained from them. They gave valuable information, however, regarding the bending at the places where they were located.

By double differentiation of the measured deflections of the horizontal elements, bending strains may be obtained for every 5-ft. elevation. But the strains obtained will be the average between any two stations and as the clinometer stations were placed rather far apart, 10 and 20 feet in a horizontal direction, the results from the clinometer data so obtained will give little information on the variation of the bending strains along the arch ring. However, in order to determine whether the strain gage and

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the clinometer data harmonized with each other, a study was made which is illustrated by Fig. 123 and 124. In the lower part of these figures the average deflections of half the dam for the 50 and 30-ft. elevations, with a head of 60 ft. of water, are shown as crosses. The horizontal strains measured with the strain gage at 10-ft. intervals along the horizontal element are shown in the upper part of the diagram as circles. They include both the direct (axial) and the bending strains. Since the direct strain has been shown to be nearly constant (see Section 111) the curve of bending strains must be a curve parallel to the curve of strains determined by the strain gage.

By a process of "cutting and trying" the intermediate deflections have been so selected that when the resulting deflection curve has been differentiated twice to obtain the bending strains, the strain curve will lie, as nearly as possible, parallel to the strain curve determined by means of the strain gage. The bending strains so obtained are shown in the upper part of Fig. 123 and 124. For the 60-ft. head this was done for each horizontal element whose elevation was a multiple of 10 feet. The fact that it was possible to find deflection curves which passed through the points determined by measurement and which

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gave a strain curve approximately parallel to the curve of strains measured by the strain gage, is a good indication of the agreement between strain gage and clinometer.

Fig. 123 and 124 show the deflections of the horizontal elements at the 50 and 30-ft. elevations, as measured with the clinometer, and the deflections determined by double integration of the measured strains at these elevations. Both deflections are for a height of water of 60 ft. The agreement between deflections arrived at by the two methods is much better for the 30-ft. elevation than for the 60-ft. elevation. This may be accounted for partly by the fact that at the 50-ft. elevation the strains were measured at stations 10 ft. apart horizontally, while at the 30-ft. elevation a continuous row of strain gage measurements only 10 in. apart was available. The greater length at the 50-ft. than at the 30-ft. elevation will also increase the error in the deflections obtained by integration of strains for the former elevation. On the whole, the agreement between the measured deflections and those determined from the strains seems fair.

For determining the loads the strain gage data were used in preference to the clinometer data. A smooth curve was drawn through the points of measured strain, and

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strains read from this curve, at intervals of 5 ft. along the element were used in a tabular differentiation such as is illustrated for the vertical elements in Table. 11. Since the direct compression was nearly constant, its effect on the values obtained in the differentiation would be negligible. The strains, shears, and loads were obtained in this way for the elements whose elevations are multiples of 5 ft., down to and including the 15-ft. elevation. As an illustration, the results so obtained are shown in Fig. 125 for the elements at elevations of 60, 50, 40, and 30 ft.

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122. Determination of Loads on Lower Part of Dam. In the lower part of the dam the strains in the vertical direction were so affected by such influences as the rotation of the toe, causing an excessive pressure at the downstream face of the dam for a considerable height, that assumption of conservation of plane section could not be made with reasonable expectation of determining with sufficient accuracy the loads carried by the vertical elements. The strains and deflections in the horizontal direction also were not taken close enough together for satisfactory use in determining the loads carried by the horizontal elements in bending. Besides this, the vertical crack which occurred at the bottom of the dam under the 60-ft. head before the strains had been observed, vitiated the data for use in this way.

It was necessary, therefore, to arrive at the loads in the lower part of the dam in another way. At the 10-ft. and the 5-ft. elevations the strain was observed on the upstream face and at the center of the thickness of the dam with telemeters. Strains on the downstream face were of no value in this study because of the presence of the crack above noted, and the depth of the crack was not known. If, in this case, the plane section remained plane during bending, the assumption of strain in the concrete

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at the base of the crack is equivalent to assuming a depth of the crack and, consequently, the depth of section remaining intact, that is, the depth of the "secondary arch" which is assumed to carry the load. With this information and by making use of the measured strains on the upstream face and at the center of the thickness of the dam, it is possible to compute the horizontal bending moment resisted by the section. The telemeter data give a basis for computing the bending moment and direct strain at the abutments. These data give the basis for determining the loads carried by the secondary arch.

So little is known as to how much strain may be resisted at the base of a crack that there is much uncertainty as to what strain should be assumed. As a first trial, the tensile strain at the 10-ft. elevation was assumed as 50 millionths, and in the second trial, as ten millionths. The loads per sq. ft. obtained for the 5 and 10-ft. elevations for the two different assumptions were as given in Table 10.

It will be seen from Table 10 that, although the variation in the assumed value of the strain at the base of the crack has an important influence on both the equivalent bending load and the equivalent load causing direct thrust, the differences compensate each other and the total loads carried are nearly the same for the two cases, and the average value was taken as the load carried by the horizontal element. In view of the approximate nature of the analysis, an

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assumption of the manner of variation of the load from crown to abutment would not be warranted, and so the load has been assumed to be uniform throughout the length of the arch. To obtain the loads carried by the vertical elements, the average loads given in Table 5 were subtracted from the total water pressure at corresponding elevations and these loads have been used for the vertical elements instead of the loads determined from the measured strains in the vertical elements.

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123. Results. In Fig. 125 specimen results for horizontal bending are given. This figure shows the distribution of strains and corresponding moments and the shears, and loads obtained therefrom by differentiation of the moments for the 60, 50, 40, and 30-ft. elevations, with a 60-ft. head of water against the dam. An idea of the confidence which may be placed in the results may be gained by observing the agreement between the observed strains and the strains obtained by double integration of the load curve. The agreement checks the differentiation by means of which the load was derived.

Although the dam was cracked through at the vertical center-line from the top down to about the 40-ft. elevation, it was carrying load even at the crack for all elevations shown in Fig. 53, except elevation 60. It may appear strange that bending load should be carried even at the crack, especially since, due to symmetry, it is evident that the shear at the location of the crack must be zero. However, the load is proportional to the curvature of the moment curve (since it is equal to the second derivative of the moment) and wherever the moment diagram is not a straight line, there must be a load.

In Fig. 125 positive loads are plotted downward and negative loads are plotted upward. A region of positive load is one in which the resultant of all the bending pressures acting on the arch ring causes a downstream pressure, and a region of negative load is one in which the resultant gives

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an upstream pressure. It is seen that for the 60, 50, and 40-ft. elevations the portion of the arch rings extending out about 30 ft. from the center was a region of positive load, that is, they were carrying load, and that the portion from there on to the abutments was a region of negative load, that is, they were applying load to the vertical elements. For the 30-ft. elevation, however, within the entire distance from the center-line to the abutment, load was carried by the arch ring.

The portion of the total load carried by direct compression, and by bending in the vertical and horizontal directions, may be seen in Fig. 130 to 133. This subject is discussed in Chapter XIV.

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XIII. LOAD CARRIED BY VERTICAL BENDING.

131. General Method. The deflections on certain vertical lines were determined by the clinometer in 5-ft. steps. The deflections so determined are proportional to the average slopes in the 5-ft. gage lengths. Due to symmetry of the arrangement of the sections on which the slopes were measured, each slope curve so obtained has its counterpart on the opposite side of the center-line and a more accurate slope curve was obtained by using the average for each symmetrical pair of slope curves. The average slope curve was integrated, giving the deflections shown as plotted points and dotted curves in Fig. 126 to 129. By a single differentiation of the slope curve the bending strains were obtained. The bending strain at each point was multiplied by the quantity $\frac{EI}{t}$ to give the moments in the vertical element one foot wide. The value used for the modulus of elasticity, E, was 3,600,000 lb. per sq. in. The thickness, t, and the moment of inertia, I, apply to the point at which the strain is measured. As indicated by the method of reducing strains to moments, plane sections have been assumed to remain plane, although this probably is not the true condition near the bottom, where foundation influences mentioned later will change the conditions considerably.

It is well known that in the differentiation of experimental curves, slight inaccuracies are magnified, and where several successive differentiations are performed, it is generally necessary to smooth out the curve at some stage

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in the operation. In this case the moments determined, as described above, were plotted and a smooth curve was fitted to the plotted values. In order to test the correctness of this modification in the moment curve, these moments were reduced back to deflections by a reversed process. That is, the moments were divided by $\frac{2EI}{t}$ to reduce to strains, and the strains were integrated successively to obtain the slope and the deflection curves. The graphs of strains, slopes, and deflections obtained by this integration, were then drawn in to test the agreement of the derived results with the original data. Having thus checked the substantial correctness of the modified moment curve, the shear and load curves were obtained from it by two successive differentiations.

All differentiations and integrations were performed by taking tabular differences or sums, as illustrated in Table 11, so there is no chance for inaccuracies to creep in due to errors in mechanical operation. Consequently, double integration of the load curve must give exactly the same moment curve as that from which the load curve was obtained by differentiation.

The shear, moment, and deflection curves, shown as solid lines in Fig. 126 to 129, are, therefore, successive integral curves obtained from the corresponding load curves and the substantial agreement of the deflection curve, shown as a solid line, with the dotted curve drawn through the

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plotted points satisfactorily checks the correctness of the mechanical operations in determining the loads from the observed deflections. The slope curves have been omitted from these figures because they have little direct bearing on the results. In all cases the solid lines represent the curves obtained by integration of the load curve. The plotted points for deflections were obtained by integration of the observed slopes. For strain and moment curves the plotted points which are shown as open circles were obtained by differentiation of the slope without modification of the observed values, and the points shown as closed circles were obtained from the strain-gage data. The plotted points for the load and shear curves represent values obtained by differentiation of the modified moment curve.

As stated above, the moment curve was modified by removing the inaccuracies. It will be seen that at and above the 15-ft. elevation this modification is very slight, while below this elevation it is considerable. At these lower elevations the influence from the foundation is so great that it makes the results very doubtful. Among these foundation influences may be mentioned the cracking loose of the dam from the foundation on the upstream side and the consequent extreme pressure on the toe, together with the yielding of the bedrock. The telemeter and the strain-gage measurements were so few that they do not give much information here (at the bottom there were no strain measurements at all), but at

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these elevations the strain curve obtained from the modified moment curve gives a better agreement with the strains measured with the telemeter and strain gage than with those obtained from the clinometer. See Fig. 126.

With so much room for selection of a moment curve to fit the points below the 15-ft. elevation, it is necessary to use some other basis for arriving at the correct moment curve than strains in the vertical direction. Since the total load at any elevation must be the sum of the loads carried by the vertical and the horizontal elements, and since, as shown in Chapters XI. and XII, the load carried by the horizontal elements has been determined fairly satisfactorily, the remaining load has been assumed to be carried by the vertical elements. At the very bottom all the load is assumed to be carried by the vertical elements, since the horizontal element cannot deflect radially, as would be necessary if it were to carry load, either by bending or by direct compression (arch action).

The strains and moments obtained from the strain-gage and telemeter data for the vertical center line of the dam are also shown in Fig. 126. Telemeter stations on both the upstream and the downstream faces indicate that there was little direct strain in a vertical direction in the dam; therefore, the vertical strains must be due principally to pure bending and this justifies the use of the strain-gage

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data for computation of the bending moments. The agreement between the bending moments obtained from the strain gages and those obtained from the telemeter is very good and adds confidence in the correctness of the modified moment curve determined from the clinometer data.

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132. Specimen Determination of Vertical Bending Load. In Sec. 91, the Method of Analysis of Test Data, the equation of the load on the rectangular element of the dam has been expressed in terms of the fourth derivative of the deflection, z. In order to determine the loads from this equation, a tabular method of differentiating the deflections has been used. Table 11 gives the average deflections found in tests 11, 12, and 13 on the vertical center-line of the dam for a head of 60 ft. and the various steps employed in reducing the deflections to loads.

If the vertical element is considered as a beam, and use is made of the fact that successive derivatives of the deflection, z, are equal to S, $\frac{M}{EI}$, $\frac{V}{EI}$, and $\frac{w}{EI}$, respectively, where:

S = slope.

M = moment.

ε = strain.

E = modulus of elasticity.

I = moment of inertia of the section.

V = total shear per ft. width for section considered.

w = load per unit of length at the point considered.

In order for the solution to be exact, deflections would have to be observed at points a differential distance, dy, apart. This, of course, was not done and could not be done. Therefore, instead of the limiting value, $\frac{dz}{dy}$, of the change

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of deflection per unit of distance along y, it is necessary to use the finite incfement Δz and Δy, and the ratios $\frac{\Delta z}{\Delta y}$, $\frac{\Delta^2 z}{\Delta y^2}$, $\frac{\Delta^3 z}{\Delta y^3}$ and $\frac{\Delta^4 z}{\Delta y^4}$ are used instead of the corresponding derivatives. If the deflection curve were made up of straight lines from point to point, the increment ratios, $\frac{\Delta z}{\Delta y}$, etc., would give exact values and the degree of approximation of the results may be judged by the variation of straight lines through measured deflections from smooth curves fitted to the same points. The following notation will, therefore, be used:

z = deflection at any point

Δz = measured deflection in any 60-in. gage length

$\Delta^2 z = \Delta(\Delta z)$ = difference between any two successive Δz's

$\Delta^3 z = \Delta(\Delta\Delta z)$ = difference between any two successive $\Delta^2 z$'s

$\Delta^4 z = \Delta(\Delta\Delta\Delta z)$ = difference between any two successive $\Delta^3 z$'s

Δy = gage length = 60 inches.

$\Delta y^2, \Delta y^3, \Delta y^4$ = 3600, 216,000, and 12,960,000 respectively.

The first line in Table 11 gives captions for the columns. The second line uses symbols, factors and column captions of line (1) to indicate the operations performed. For

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example, Σ_0^{60}(b) indicates that to obtain the deflections of column (a), the observed deflections for each 60-in., given in column (b), were added cumulatively, beginning at the bottom and ending at the top of the dam. The symbol Δ(b) in column (c) indicates that to obtain the values in that column, differences were taken between the values given in column (c). To obtain the values of moment given in column (e), the values from column (c) were multiplied by 1028I. To obtain the values given in column (a), a smooth curve was drawn to fit the values given in column (e) to eliminate inaccuracies and to take account of the fact that near the bottom the load must be determined by subtracting from the total load the load carried by the horizontal element. The moments thus obtained were used for succeeding operations, as indicated. To test the correctness of the moments shown by the smooth curve and tabulated in column (f) the deflections corresponding to these moments were determined by the operations indicated in columns (j), (k), (l), and (m).

In Fig. 126, the deflections, strains, and moments given in columns (a), (d), and (e) are shown as open circles. The shears shown as a solid line are one-fifth the values given in column (g). The loads, moments, strains, and deflections shown as solid lines are the values given in columns (i), (f), (j), and m.)

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133. Results. In Fig. 126 to 129, inclusive, the loads carried by vertical bending are shown for all portions of the dam on which the data gave information. The deflections, strains, moments, and shears were used in determining the loads, and these also are given in Fig. 126 to 129.

Positive loads are plotted to the right and negative loads to the left. The significance of the signs is the same as is stated in Sec. 123, that is, a region of positive load is one in which the resultant of the bending loads on the vertical element causes a downstream pressure and a region of negative load is one in which the resultant gives an upstream pressure.

In all cases the bending load on the vertical element was negative in the upper part of the dam. That is, the upper portion of the vertical element, instead of carrying load, was resting on the arch element, which at any elevation in the upper part of the dam was, therefore, carrying more than the total water pressure applied at that elevation. For all sections shown in Fig. 126 to 129, the load on the vertical elements changed from negative to positive somewhere between the 40 and 35-ft. elevations and was positive from there to the bottom. The region of negative load on the vertical elements extended, as shown in Fig. 126 to 129, from the center-line to at least 30 ft. from the center line, and it is not known how much

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133. (Contd.)

further it may have extended.

The portion of the total load carried by direct compression and by bending in the vertical and horizontal directions, may be seen in Fig. 130 to 133. This subject is discussed in Chapter XIV.

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TABLE 1. WEIGHTS PER UNIT OF VOLUME OF AGGREGATES FROM BINS AT TEST DAM.

Size	Per cent by volume	Weight by Loose Measure		
		Moist	Dry	Rodded Dry
		Lb. per cu. ft.		
0 to $\frac{3}{8}$	100	85.65		113.0
$\frac{3}{8}$ to $\frac{3}{4}$	100		84.96	93.1
$\frac{3}{4}$ to $1\frac{1}{2}$	100		82.77	93.2
$\frac{3}{8}$ to $\frac{3}{4}$	50			99.3
$\frac{3}{4}$ to $1\frac{1}{2}$	50			
0 to $\frac{3}{8}$	46			121.5
$\frac{3}{8}$ to $\frac{3}{4}$	27			
$\frac{3}{4}$ to $1\frac{1}{2}$	27			

NOTE: All measurements were made with cubical wooden measures, generally of 1.0 cu.ft. capacity but in a few cases of 0.5 cu.ft. capacity.

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TABLE 4. FORM DEFLECTIONS AT ELEVATION 37.5 FEET.

Station	Deflection		
	Downstream Form in.	Upstream Form in.	Total Spreading of Forms in.
Left 0+25	.13	.08	.21
Right 0+1	.14	.12	.26
Right 0+11	.15	.12	.27
Right 0+15	.17	.10	.27

NOTE: The deflections recorded are downstream deflection for downstream forms and upstream deflections for upstream forms.
The stationing is to right and left of the center-line of the dam.

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TABLE 8. COEFFICIENTS FOR CORRECTION OF MOMENTS DETERMINED FROM MEASURED STRAINS.

Elevation Feet	Angle α (Radians)	Angle α (Degrees)	$\cos^2 2\theta = \cos^2\alpha$	$\frac{1}{\left(1-\frac{7}{6}\theta^2\right)+\frac{t}{6y}\left(\theta-\frac{2}{3}\theta^3\right)}$
		Deg. Min.		
30	0	0 - 0	1	1
25	.0295	1 - 41.3	.99652	1.00177
20	.0588	3 - 22.1	.96625	1.00404
15	.0892	5 - 06.5	.96851	.99960
10	.1194	5 - 50.6	.94403	1.00503
5	.1502	8 - 36.2	.91248	1.01174
0	.1763	10 - 06.6	.88052	

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TABLE 9. EQUIVALENT TORSIONAL LOAD IN POUNDS PER SQUARE FOOT AND PER CENT OF TOTAL LOAD.

Elevation, y feet	Method	Horizontal Distance From Vertical Center-Line-Feet.										
		0	7.5	10	15	20	25	30	35	40	45	50
55	Z	0		+6		+20		+6		-17		
	M_y			-13	-16		-32	+24	+20		0	-144
	M_{xy}		-23			-36		-36		+54		-144
	Av.	0	-23	-4	-16	-8	-32	-2	+20	+18	0	-144
	Av. / Total		-.07	-.01	-.05	-.03	-.01	-.01	+.06	+.06	0	+.46
50	Z	0		+6		+20		+6		-17		
	M_y	-12		-1	-27		-60	-158	-28		+63	
	M_{xy}											
	Av.	-6		+3	-27	+20	-60	-76	-28	-17	+63	
	Av. / Total	+.01		-.05	-.04	+.03	-.10	-.11	-.04	-.03	-.10	
45	Z											
	M_y	0		+5	+12		+76	-192	+104		+27	
	M_{xy}	0	+34			+36		-36				
	Av.	0	+34	+5	-12	+36	+76	-114	+104		+27	
	Av. / Total	0	+.04	+.01	-.01	+.04	+.08	-.12	+.11		+.03	
40	Z	0		+32		-37		-35				
	M_y	-8		-6	-8		+48	-112	+79			
	M_{xy}											
	Av.	-4		+14	-8	-37	+48	-74	+79			
	Av. / Total	-.00		+.01	-.01	-.03	+.04	-.06	+.06			

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TABLE 9. (CONTD). EQUIVALENT TORSIONAL LOAD IN POUNDS PER SQUARE FOOT AND PER CENT OF TOTAL LOAD.

Elevation, y Feet	Method	Horizontal Distance From Vertical Center-Line-Feet. 0	7.5	10	15	20	25	30	35	49	45	50
35	Z											
	M_y			-32	-16		-56	-128				
	M_{xy}		+12			-72		+90				
	Av.		+12	-32	-16	-72	-56	-19				
	Av. / Total		+.01	-.02	-.01	-.05	-.04	-.10				
30	Z	-12										
	M_y	-12		0	0		-88	-24				
	M_{xy}											
	Av.	-12		0	0		-88	-24				
	Av. / Total	-.01					-.05	-.01				
	Z											
	M_y			-1	0	+144						
	M_{xy}		+46									
	Av.		+46	-1	0	+144						
	Av. / Total		+.02			+.06						
	Z											
	M_y	-60		-16	-88							
	M_{xy}											
	Av.	-60		-16	-88							
	Av. / Total.	-.02		-.01	-.03							

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TABLE 10. APPROXIMATE LOADS CARRIED ON HORIZONTAL ELEMENTS AT 5 AND 10-FOOT ELEVATIONS.

Elevation Feet	Assumed Strain at Base of Crack Millionths	Load Carried by Horizontal Element		
		Direct	Bending	Total
5	50	1575	132	1707
5	10	2135	382	1753
		Average	Total Load	1730
Feet	Millionths	Lb. Ft.Sq.	Lb. Ft.Sq.	L..Ft.Sq.
10	50	818	462	1280
10	10	1126	207	1333
		Average Total Load.		1306

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TABLE 4 DATA OF CONCRETE MIX

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FIG. 1. VIEW OF TEST DAM AND SURROUNDINGS FROM ABOVE.

FIG. 4. PHOTOGRAPH SHOWING GENERAL ARRANGEMENT, INLETS TO OVERSLUICE.

FIG. 7. SLUMP TESTS AND CONTROL SPECIMENS, SLUMP 2 INCHES.

FIG. 8. SLUMP TESTS AND CONTROL SPECIMENS, SLUMP 7 INCHES.

FIG. 2. PLAN, ELEVATION AND DEVELOPED UPSTREAM SURFACE OF DAM SHOWING ARRANGEMENT OF UNDERSLUICE AND GATE VALVES.

FIG. 5. PROFILE SHOWING CONFORMITY OF EXCAVATION TO DESIGN.

FIG. 9. VIEW SHOWING CONCRETE MIXING-PLANT AND PORTIONS OF CHUTE DELIVERING CONCRETE AT ELEVATION 15.

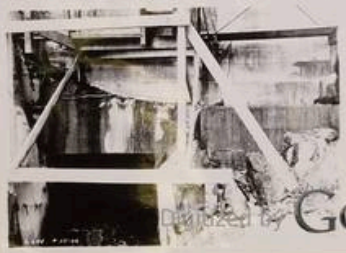

FIG. 3. VIEW SHOWING UNDERSLUICE OUTLET AND PROJECTING ENDS OF I-BEAMS USED FOR LEVEL BAR STATIONS.

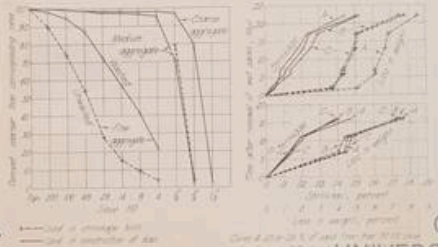

FIG. 6. SIEVE ANALYSES OF AGGREGATES.

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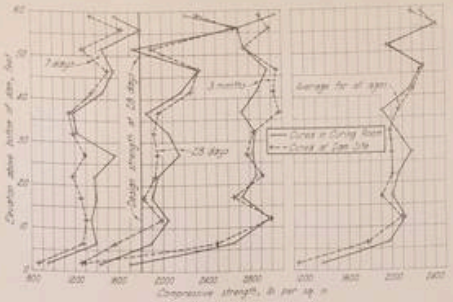

FIG. 10. STRENGTH OF CONCRETE AT VARIOUS ELEVATIONS.

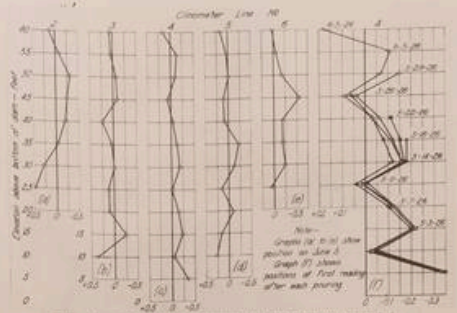

FIG. 11. POSITION OF DOWNSTREAM FACE OF DAM ON JUNE 5.

FIG. 12. VIEW OF STRAIN GAGE.

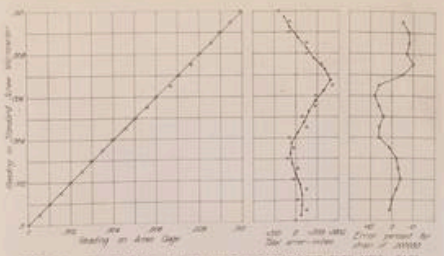

FIG. 13. CALIBRATION CURVES FOR TEN-THOUSANDTHS-INCH ANES GAGE MICROMETER FROM 0.150 to 0.165.

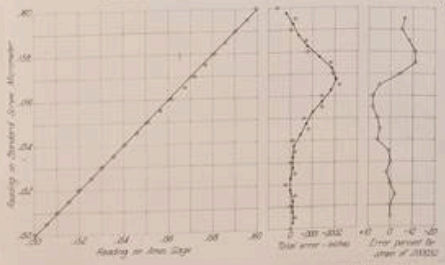

FIG. 14. CALIBRATION CURVES FOR TEN-THOUSANDTHS-INCH AMES GAGE MICROMETER FROM ZERO TO 0.015.

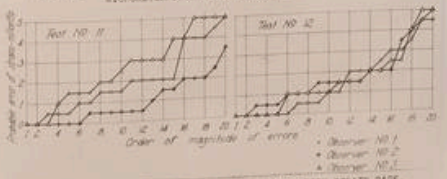

FIG. 15. PROBABLE ERROR OF AVERAGE READING WITH STRAIN GAGE. (A) TEST NO. 11. (B) TEST NO. 12.

FIG. 16. DRAWING OF ELECTRIC TELEMETER.

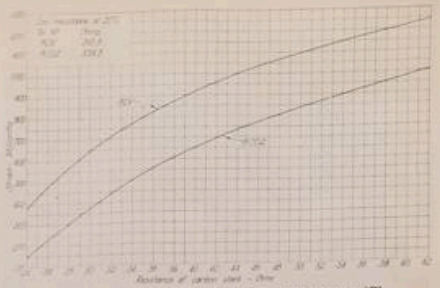

FIG. 17. RESISTANCE-STRAIN DIAGRAM FOR TELEMETERS NO. 52V AND 60DP.

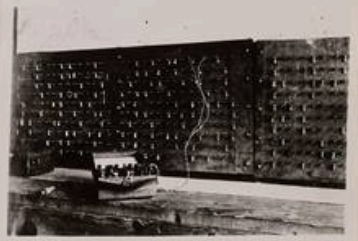

FIG. 18. VIEW OF SWITCHBOARD AND WHEATSTONE BRIDGE.

FIG. 19. PHOTOGRAPH OF RADIUS METER.

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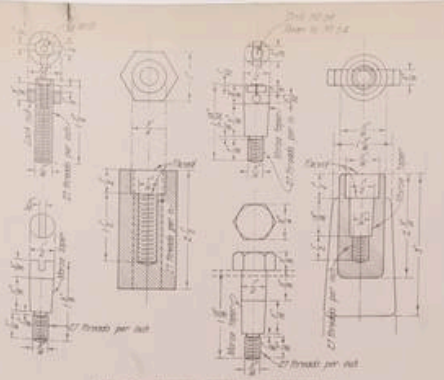

FIG. 20. INSERTS USED WITH RADIUS METER.

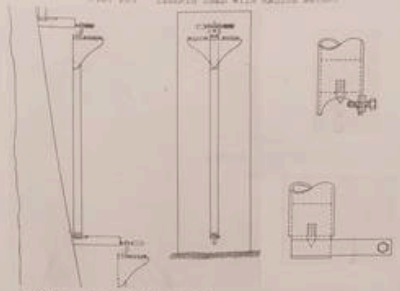

FIG. 21. SKETCH OF CLINOMETER.
(A) AS USED FOR MEASURING DEFLECTIONS OF DAM.
(B) AS USED ON REFERENCE STANDARD.

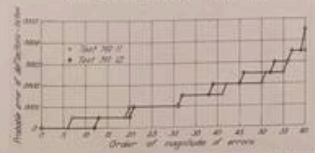

FIG. 22. PROBABLE ERROR FOR DEFLECTIONS DETERMINED WITH CLINOMETER.

FIG. 23. PHOTOGRAPH OF LEVEL-BAR.

FIG. 24. DRAWING OF RESISTANCE WIRGMETER.

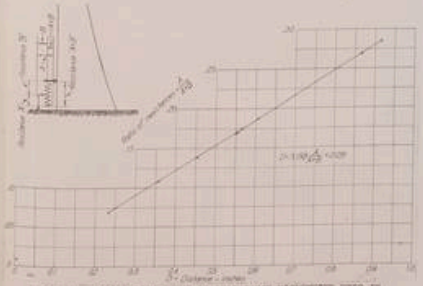

FIG. 25. CALIBRATION CURVE OF RESISTANCE WIRGMETER USED IN MEASUREMENT OF WIDTH OF FOUNDATION CRACK.

FIG. 26. LOCATION OF STATIONS FOR OBSERVATIONS WITH CLINOMETER, STRAIN GAGE AND DEFLECTOMETERS.

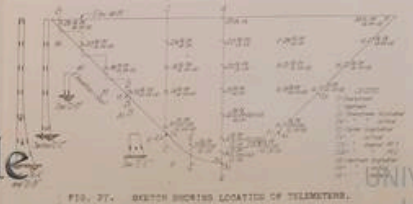

FIG. 27. SKETCH SHOWING LOCATION OF TELEMETERS.

FIG. 28. LOCATIONS OF STATIONS; RADIUS-METER, LEVEL-BAR, AND THERMOMETER.

FIG. 29. FOUNDATION AND INTERIOR OF FORMS BEFORE CONCRETING.

FIG. 30. CONSTRUCTION JOINT AT ELEVATION 20, PREPARED FOR NEXT POURING. TELEMETERS, CLINOMETER BRACKETS AND RADIUS-METER INSERTS IN PLACE.

FIG. 31. INTERIOR OF FORM AT SOUTH ABUTMENT, 40-FT. ELEVATION; TELEMETERS, CLINOMETER BRACKETS, AND RADIUS-METER INSERTS IN PLACE.

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FIG. 16. INSTALLATION OF BEAM GAGES FOR CLINOMETER, STRAIN GAGE AND GARDNER-WATER DEVICES AND CLINOMETER READINGS IN PLACE.

Records of Tests made in cooperation with U. S. Bureau of Standards — Form 1

ENGINEERING FOUNDATION — COMMITTEE ON ARCH DAM INVESTIGATION

STEVENSON CREEK DAM — ORIGINAL DATA OF TEST — Clinometer — Test No 12

Date 9-21-26; Series 1; Test No. A-2; Observer Berry; Observer Steel

| Time | Gage Reading | Device No. | Note | | |

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Records of Tests made in co-operation with U. S. Bureau of Standards — Form 1

ENGINEERING FOUNDATION
on
Committee on Arch Dam Investigation
on
STEVENSON CREEK DAM

GENERAL DATA SHEET

Subject: Level Bar
Test No. 12
Observer: Lyon Recorder: Dolfe
Date: 9-21-26 Sheet 1 of 2 Sheets

Time of Action Start	Series	Bar 20 W	Bar 20 E	Station 21 W	Station 21 E	Bar 30 W	Bar 30 E	Bar 31 W	Bar 31 E	Bar 32 W	Bar 32 E
0	3	10693	10531	11135	-0086	10022	10010	12492	-0068	0400	11446
60	2	10790	10601	11295	-0313	10080	10047	12580	-0018	-0102	11639
	Diff.	-1	+20	-163	+213	-48	+63	-141	+140	-298	+187
	Mean	-10'-5"		-135'-34"		-62'-11"		-142'-17"		-298'-149"	
Change in 60 m.		-0015		-0274		-0090		-0207		-0434	
60	1	10701	10606	11493	-0409	10082	10130	12570	-0162	-0002	11648
	Diff.	-2	+13	-246	+279	-43	+80	-143	+134	-372	+398
	Mean	-8'-4"		-274'-137"		-32'-16"		-190'-95"		-386'-193"	
Change in 60 m.		-0012		-0398		-0105		-0262		-0561	
		40 W	40 E	41 W	41 E	50 W	50 E	51 W	51 E	52 W	52 E
0	3	10875	10011	10599	-0680	10065	-0059	12240	-0235	10638	-040
60	2	10935	10983	10730	-0767	10069	-0064	12337	-0341	10812	-0896
	Diff.	-60	+28	-131	+147	+6	-5	-117	+106	-177	+319
	Mean	-44'-22"		-109'-64.5"		+6'-3.5"		-102'-56"		-238'-119"	
Change in 60 m.		-0064		-0188		+0009		-0163		-0434	
60	1	10918	10079	10765	-0762	10096	-0034	12390	-0341	12012	-1052
	Diff.	-43	+32	-166	+182	+8	-25	-140	+125	-379	+490
	Mean	-55'-19"		-174'-87"		+17'-8.5"		-138'-69"		-390'-195"	
Change in 60 m.		-0055		-0253		+0025		-0001		-0567	

FIG. 39. COPY OF PORTION OF REDUCED LEVEL-BAR DATA FOR TEST NO. 12.

Records of Tests made in co-operation with U. S. Bureau of Standards — Form 4

ENGINEERING FOUNDATION
COMMITTEE ON
ARCH DAM INVESTIGATION

TEST OF
STEVENSON CREEK DAM
TELEMETERING & RESISTANCE THERMOMETERS

INSTRUCTION BOARD NO. 17061
OBSERVER: Carlson
RECORDER: Barnwith
DATE: 9-21-26
SHEET

Time of Action Start																							
	Telemeter	20	21	30	31	32	40	41	50	51													
	North. Ohms	32.7	22.9	50.2	22.3	47.6																	
	Cell. Ohms	501	500	50.3	43.5	50.2																	
	Indicated Meter	300	140	192	242	458																	
	Temp. Correction	-96	-95	-80	-136	-96																	
	Corrected Meter	204	45	112	106	362																	
	Temperature	14.0	14.5	13.0	11.5	14.0																	
18 hr. to 40 min. P.M.	Telemeter	TE1	TE2	TE3	TE4	TE5	TE6	TE7	TE8	TE9	TE10	TE11	TE12	TE13	TE14	TE15	TE16	TE17	TE18	TE19	TE20	TE21	TE22
	North. Ohms	41.7		44.1	42.9	49.6	45.1	45.0	43.4	48.4	45.1	50.3	47.9			36.6	40.3	36.2	50.8	50.8			
	Cell. Ohms	501			498	498	502.5	500	500		498		497			502	500		500				
	Indicated Meter	738		627	630	702	1125	1020	970	812	652	558	488			858	530	834	670	655			
	Temp. Correction	-64		-64	-96	-104	-84	-88	-94	-96	-104	-104	-112			-88	-88	-88	-88	-88			
	Corrected Meter	674		563	534	598	041	932	874	716	548	454	376			314	442	746	582	567			
	Temperature	16.0			14.0	13.5	14.75	14.5	14.0		13.5		13.0			14.5	14.5		14.5				
	Telemeter	TE23	TE24	TE25	TE26	TE27	TE28	TE29	TE30	TE31	TE32	TE33	TE34	TE35	TE36	TE37	TE38	TE39	TE40	TE41	TE42	TE43	TE44
	North. Ohms	41.7	37.4	41.6	43.8	46.9	44.7	43.8	77.5	50.0	43.2	40.9	42.4	50.5	46.8	49.5	45.8	40.5	39.4				
	Cell. Ohms	495	491	502		500	499.5	501	498	501.5	499	498		498		501	500	502.5	502				
	Indicated Meter	625	425	628	748	710	652	738	240	832	615	407	537	605	680	620	435	470	520				
	Temp. Correction	-104	-104	-72	-72	-96	-96	-96	-104	-100	-96	-96	-96	-96	-96	-96	-96	-60	-64				
	Corrected Meter	521	321	556	676	614	490	642	136	732	519	311	441	506	594	524	339	510	456				
	Temperature		13.5	13.5		14.0	14.25	14.0	13.5	13.75	14.0	14.0		14.0		14.0	14.0	16.25	16.0				
	Telemeter	TE45	TE46	TE47	TE48	TE49	TE50	TE51	TE52	TE53	TE54	TE55	TE56	TE57	TE58	TE59	TE60	TE61	TE62	TE63	TE64	TE65	TE66
	North. Ohms	41.8	39.3	48.4	53.0	45.2	39.4	33.5		45.8	54.4	56.1	36.7	33.4	48.3	44.3	48.3	43.4	31.2				
	Cell. Ohms	502			501	499					500			500.5		504	502	498.5					
	Indicated Meter	808	580	708	125	712	500	435		781	615	475	94	25	620	664	620	664	212				
	Temp. Correction	-96	-96	-96	-96	-104				-96		-100			-100			-100	-100				
	Corrected Meter	712	484	612	029	592	500	435		685		375			520			564	112				
	Temperature	14.0			14.0	12.5					14.0				14.0			13.75					

FIG. 39. COPY OF PORTION OF ORIGINAL AND REDUCED TELEMETER DATA FOR TEST NO. 12.

FIG. 40. TEMPERATURE ON FOUNDATION ROCK DURING BETTING AND CURING OF CONCRETE.

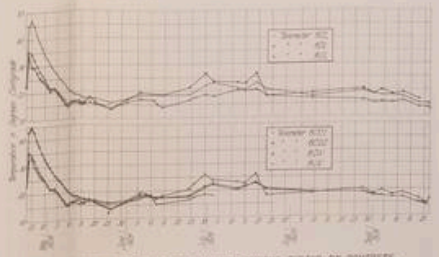

FIG. 41. TEMPERATURE AT STATION SIX DURING CURING OF CONCRETE.

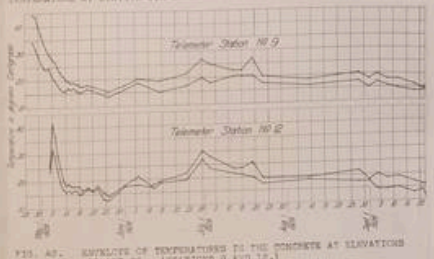

FIG. 42. ENVELOPE OF TEMPERATURES IN THE CONCRETE AT ELEVATIONS 10 AND 20. (STATIONS 9 AND 12.)

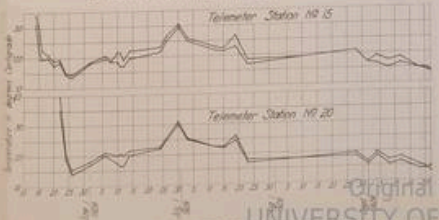

FIG. 43. ENVELOPE OF TEMPERATURES IN THE CONCRETE AT ELEVATIONS 30 AND 40. (STATIONS 15 AND 20.)

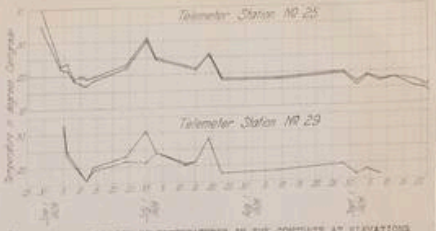

FIG. 44. ENVELOPE OF TEMPERATURES IN THE CONCRETE AT ELEVATIONS 50 AND 60. (STATIONS 25 AND 29.)

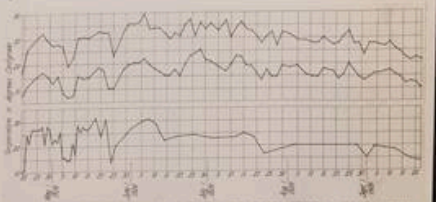

FIG. 45. AIR TEMPERATURES AT DAM DURING TEST PROGRAM. UPPER CURVE: MAXIMUM AND MINIMUM DAILY TEMPERATURES. TEMPERATURES IN LOWER CURVE SIMULTANEOUS WITH TELEMETER READINGS.

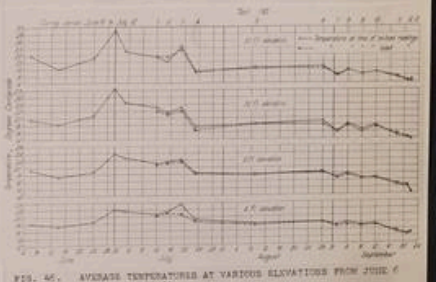

FIG. 46. AVERAGE TEMPERATURES AT VARIOUS ELEVATIONS FROM JUNE 6 TO SEPTEMBER 29.

FIG. 47. [illegible] TEMPERATURES AT TELEMETER STATIONS 1 TO 5, WITH NO WATER IN THE RESERVOIR.

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Records of Tests made in co-operation with U. S. Bureau of Standards — Form 1

ENGINEERING FOUNDATION
on
Committee on Arch Dam Investigation
on
STEVENSON CREEK DAM

GENERAL DATA SHEET

Subject: Level Bar
Test No. 12
Observer: Lyon Recorder: Dolfe
Date: 9-21-26 Sheet 1 of 2 Sheets

Time of Action Start	Series	Bar 20 W	Bar 20 E	Station 21 W	Station 21 E	Bar 30 W	Bar 30 E	Bar 31 W	Bar 31 E	Bar 32 W	Bar 32 E
0	3	10693	10531	11135	-0086	10022	10010	12492	-0068	0400	11446
60	2	10790	10601	11295	-0313	10080	10047	12580	-0018	-0102	11639
	Diff.	-1	+20	-163	+213	-48	+63	-141	+140	-298	+187
	Mean	-10'-5"		-135'-34"		-62'-11"		-142'-17"		-298'-149"	
Change in 60 m.		-0015		-0274		-0090		-0207		-0434	
60	1	10701	10606	11493	-0409	10082	10130	12570	-0162	-0002	11648
	Diff.	-2	+13	-246	+279	-43	+80	-143	+134	-372	+398
	Mean	-8'-4"		-274'-137"		-32'-16"		-190'-95"		-386'-193"	
Change in 60 m.		-0012		-0398		-0105		-0262		-0561	
		40 W	40 E	41 W	41 E	50 W	50 E	51 W	51 E	52 W	52 E
0	3	10875	10011	10599	-0680	10065	-0059	12240	-0235	10638	-040
60	2	10935	10983	10730	-0767	10069	-0064	12337	-0341	10812	-0896
	Diff.	-60	+28	-131	+147	+6	-5	-117	+106	-177	+319
	Mean	-44'-22"		-109'-64.5"		+6'-3.5"		-102'-56"		-238'-119"	
Change in 60 m.		-0064		-0188		+0009		-0163		-0434	
60	1	10918	10079	10765	-0762	10096	-0034	12390	-0341	12012	-1052
	Diff.	-43	+32	-166	+182	+8	-25	-140	+125	-379	+490
	Mean	-55'-19"		-174'-87"		+17'-8.5"		-138'-69"		-390'-195"	
Change in 60 m.		-0055		-0253		+0025		-0001		-0567	

FIG. 39. COPY OF PORTION OF REDUCED LEVEL-BAR DATA FOR TEST NO. 12.

Records of Tests made in co-operation with U. S. Bureau of Standards — Form 4

ENGINEERING FOUNDATION
COMMITTEE ON
ARCH DAM INVESTIGATION

TEST OF
STEVENSON CREEK DAM
TELEMETERING & RESISTANCE THERMOMETERS

INSTRUCTION BOARD NO. 17061
OBSERVER: Carlson
RECORDER: Barnwith
DATE: 9-21-26
SHEET

Time of Action Start																							
	Telemeter	20	21	30	31	32	40	41	50	51													
	North. Ohms	32.7	22.9	50.2	22.3	47.6																	
	Cell. Ohms	501	500	50.3	43.5	50.2																	
	Indicated Meter	300	140	192	242	458																	
	Temp. Correction	-96	-95	-80	-136	-96																	
	Corrected Meter	204	45	112	106	362																	
	Temperature	14.0	14.5	13.0	11.5	14.0																	
18 hr. to 40 min. P.M.	Telemeter	TE1	TE2	TE3	TE4	TE5	TE6	TE7	TE8	TE9	TE10	TE11	TE12	TE13	TE14	TE15	TE16	TE17	TE18	TE19	TE20	TE21	TE22
	North. Ohms	41.7		44.1	42.9	49.6	45.1	45.0	43.4	48.4	45.1	50.3	47.9			36.6	40.3	36.2	50.8	50.8			
	Cell. Ohms	501			498	498	502.5	500	500		498		497			502	500		500				
	Indicated Meter	738		627	630	702	1125	1020	970	812	652	558	488			858	530	834	670	655			
	Temp. Correction	-64		-64	-96	-104	-84	-88	-94	-96	-104	-104	-112			-88	-88	-88	-88	-88			
	Corrected Meter	674		563	534	598	041	932	874	716	548	454	376			314	442	746	582	567			
	Temperature	16.0			14.0	13.5	14.75	14.5	14.0		13.5		13.0			14.5	14.5		14.5				
	Telemeter	TE23	TE24	TE25	TE26	TE27	TE28	TE29	TE30	TE31	TE32	TE33	TE34	TE35	TE36	TE37	TE38	TE39	TE40	TE41	TE42	TE43	TE44
	North. Ohms	41.7	37.4	41.6	43.8	46.9	44.7	43.8	77.5	50.0	43.2	40.9	42.4	50.5	46.8	49.5	45.8	40.5	39.4				
	Cell. Ohms	495	491	502		500	499.5	501	498	501.5	499	498		498		501	500	502.5	502				
	Indicated Meter	625	425	628	748	710	652	738	240	832	615	407	537	605	680	620	435	470	520				
	Temp. Correction	-104	-104	-72	-72	-96	-96	-96	-104	-100	-96	-96	-96	-96	-96	-96	-96	-60	-64				
	Corrected Meter	521	321	556	676	614	490	642	136	732	519	311	441	506	594	524	339	510	456				
	Temperature		13.5	13.5		14.0	14.25	14.0	13.5	13.75	14.0	14.0		14.0		14.0	14.0	16.25	16.0				
	Telemeter	TE45	TE46	TE47	TE48	TE49	TE50	TE51	TE52	TE53	TE54	TE55	TE56	TE57	TE58	TE59	TE60	TE61	TE62	TE63	TE64	TE65	TE66
	North. Ohms	41.8	39.3	48.4	53.0	45.2	39.4	33.5		45.8	54.4	56.1	36.7	33.4	48.3	44.3	48.3	43.4	31.2				
	Cell. Ohms	502			501	499					500			500.5		504	502	498.5					
	Indicated Meter	808	580	708	125	712	500	435		781	615	475	94	25	620	664	620	664	212				
	Temp. Correction	-96	-96	-96	-96	-104				-96		-100			-100			-100	-100				
	Corrected Meter	712	484	612	029	592	500	435		685		375			520			564	112				
	Temperature	14.0			14.0	12.5					14.0				14.0			13.75					

FIG. 39. COPY OF PORTION OF ORIGINAL AND REDUCED TELEMETER DATA FOR TEST NO. 12.

FIG. 40. TEMPERATURE ON FOUNDATION ROCK DURING BETTING AND CURING OF CONCRETE.

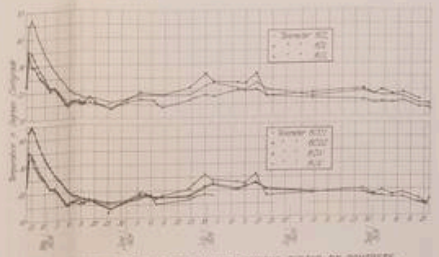

FIG. 41. TEMPERATURE AT STATION SIX DURING CURING OF CONCRETE.

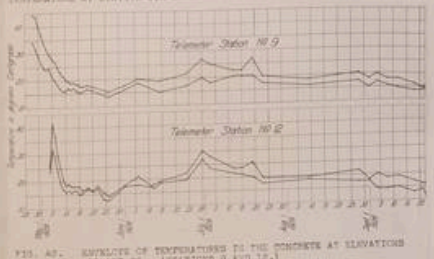

FIG. 42. ENVELOPE OF TEMPERATURES IN THE CONCRETE AT ELEVATIONS 10 AND 20. (STATIONS 9 AND 12.)

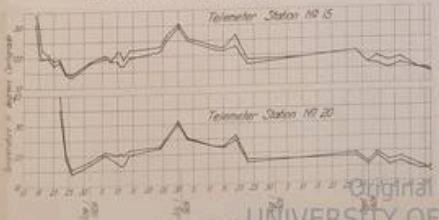

FIG. 43. ENVELOPE OF TEMPERATURES IN THE CONCRETE AT ELEVATIONS 30 AND 40. (STATIONS 15 AND 20.)

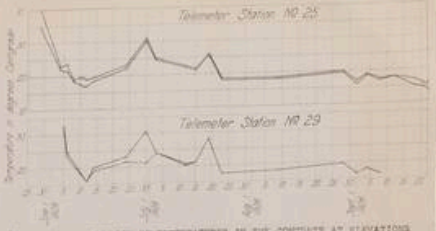

FIG. 44. ENVELOPE OF TEMPERATURES IN THE CONCRETE AT ELEVATIONS 50 AND 60. (STATIONS 25 AND 29.)

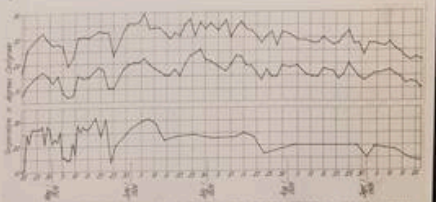

FIG. 45. AIR TEMPERATURES AT DAM DURING TEST PROGRAM. UPPER CURVE: MAXIMUM AND MINIMUM DAILY TEMPERATURES. TEMPERATURES IN LOWER CURVE SIMULTANEOUS WITH TELEMETER READINGS.

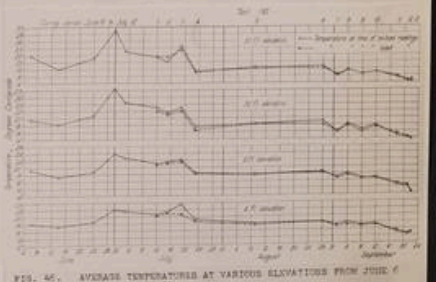

FIG. 46. AVERAGE TEMPERATURES AT VARIOUS ELEVATIONS FROM JUNE 6 TO SEPTEMBER 29.

FIG. 47. [illegible] TEMPERATURES AT TELEMETER STATIONS 1 TO 5, WITH NO WATER IN THE RESERVOIR.

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FIG. 48. STRAINS AT TELEMETER STATION 4, WITH NO WATER IN THE RESERVOIR.

FIG. 50. RELATIONS BETWEEN STRAINS MEASURED BY TELEMETERS AND STRAINS COMPUTED FROM TEMPERATURE CHANGES AND DEFLECTIONS.

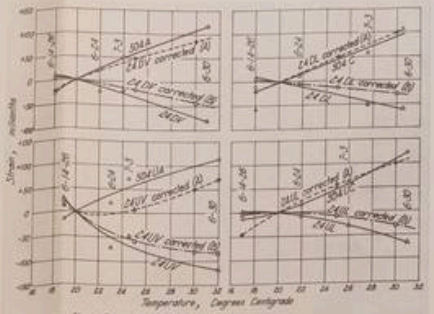

FIG. 51. RELATION OF STRAIN TO TEMPERATURE DETERMINED BY STRAIN GAGE AND TELEMETER FOR STATION AT 50-FT. ELEVATION, 24 FT. NORTH OF CENTERLINE.

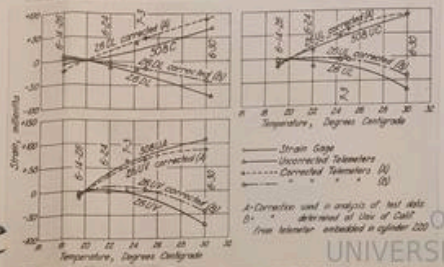

FIG. 52. RELATION OF STRAIN TO TEMPERATURE DETERMINED BY STRAIN GAGE AND TELEMETER FOR STATION AT 50-FT. ELEVATION, 24 FT. SOUTH OF CENTERLINE.

FIG. 49. NO-LOAD STRAINS AT REPRESENTATIVE STRAIN-GAGE STATIONS UP TO THE END OF THE TESTING PROGRAM.

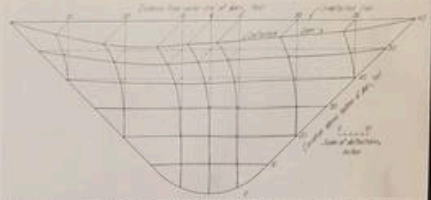

FIG. 53. ISOMETRIC DIAGRAM OF DEFLECTIONS, JUNE 5 TO JUNE 14.

FIG. 54. ISOMETRIC DIAGRAM OF DEFLECTIONS, JUNE 5 TO JUNE 24.

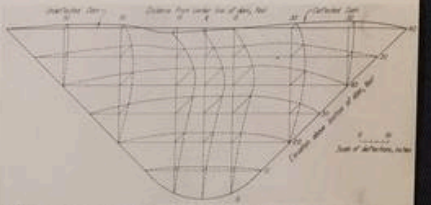

FIG. 55. ISOMETRIC DIAGRAM OF DEFLECTIONS, JUNE 5 TO JUNE 30.

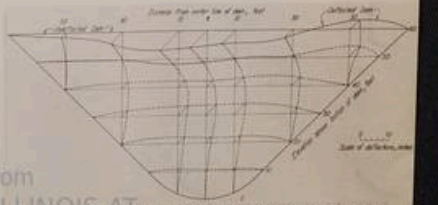

FIG. 56. ISOMETRIC DIAGRAM OF DEFLECTIONS, JUNE 5 TO JULY 3.

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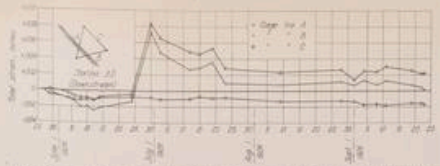

FIG. 69. TOTAL STRAINS IN GAGE LINES OF STATION 30 ACROSS ABUTMENT JOINT AT ELEVATION 44.

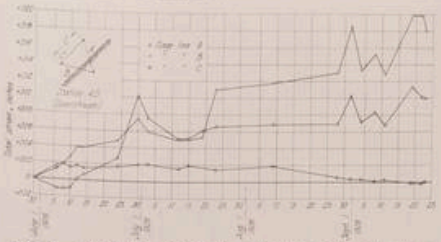

FIG. 70. TOTAL STRAINS IN GAGE LINES OF STATION 40 ACROSS ABUTMENT JOINT AT ELEVATION 53.

FIG. 71. TOTAL STRAINS IN GAGE LINES OF STATION 30 ACROSS ABUTMENT JOINT AT ELEVATION 55.

FIG. 72. TOTAL STRAINS IN GAGE LINES OF STATIONS 71 AND 72, E1 TO E4 ACROSS ABUTMENT JOINTS ON TOP OF DAM.

FIG. 73. PHOTOGRAPH OF DAM AND RESERVOIR.

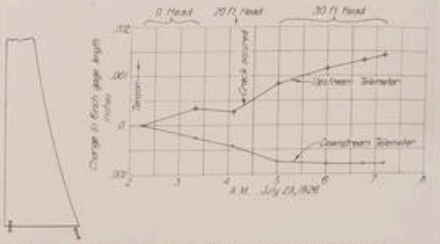

FIG. 74. TELEMETER RECORD OF CRACK AT FOUNDATION UNDER 30-FT. HEAD OF WATER.

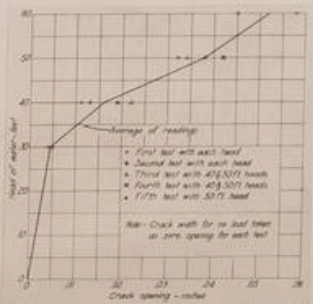

FIG. 75. CHANGES IN WIDTH OF CRACK BETWEEN DAM AND BEDROCK ON UPSTREAM SIDE AT ELEVATION 2.

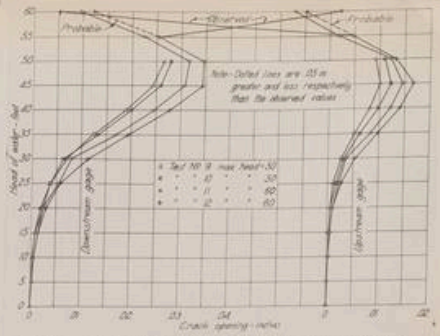

FIG. 76. CHANGES IN WIDTH OF CRACK IN DAM AT CROWN ELEVATION 60.

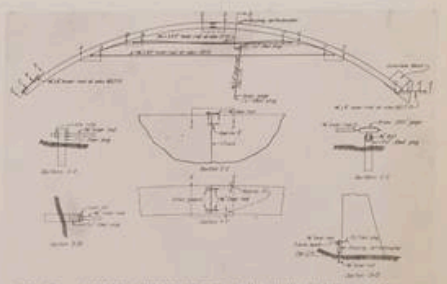

FIG. 77. LOCATION OF STATIONS FOR MEASUREMENT OF MOVEMENT OF BEDROCK AND CHANGES OF CRACK WIDTH.

FIG. 78. [illegible] APPARATUS USED FOR MEASUREMENTS OF RADIAL MOVEMENT OF BEDROCK NEAR BOTTOM OF DAM.

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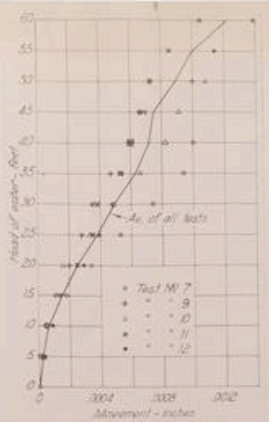


FIG. 79. RADIAL MOVEMENT OF BEDROCK AT ONE FOOT ELEVATION.

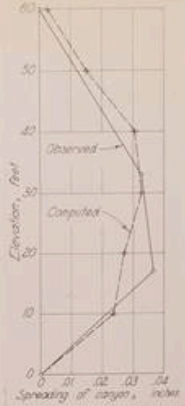


FIG. 80. COMPARISON OF OBSERVED AND COMPUTED MOVEMENT OF BEDROCK FOR 60-FT. HEAD.

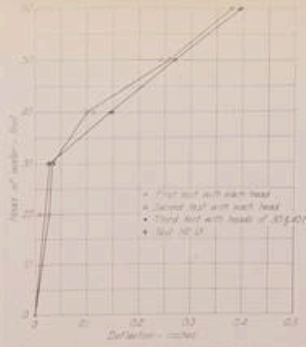
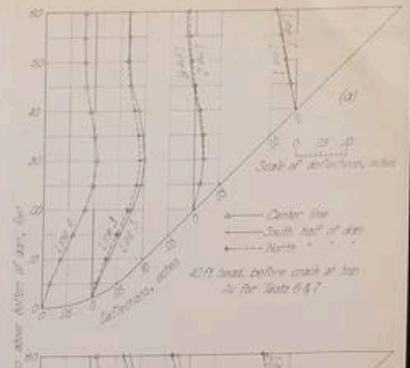


FIG. 83. MAXIMUM DEFLECTIONS FOR VARIOUS HEADS OF WATER.



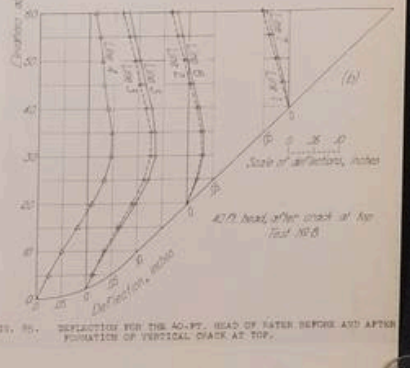


FIG. 86. DEFLECTION FOR THE 40-FT. HEAD OF WATER BEFORE AND AFTER FORMATION OF VERTICAL CRACK AT TOP.

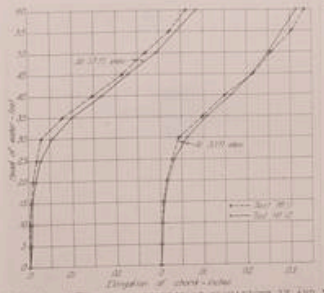
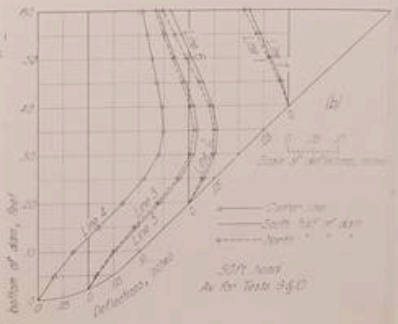


FIG. 80. MOVEMENT OF BEDROCK AT ELEVATIONS 17 AND 33.



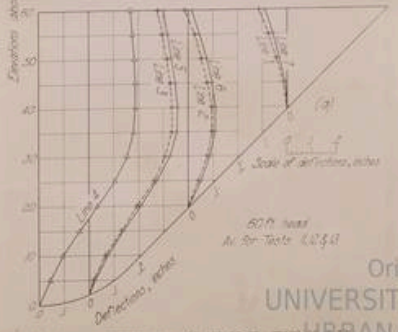


FIG. 84. DEFLECTIONS FOR 60 AND 50-FT. HEADS OF WATER.

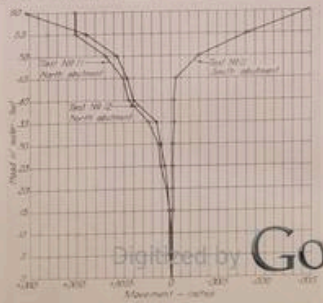


FIG. 81. MOVEMENT OF BEDROCK AT ELEVATION 60.

FIG. 87. GEOMETRIC DIAGRAM OF DEFLECTIONS FOR 60-FT. HEAD OF WATER

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FIG. 87. ISOMETRIC DIAGRAM OF DEFLECTIONS FOR 50-FT. HEAD OF WATER.

FIG. 88. ISOMETRIC DIAGRAM OF DEFLECTIONS FOR 40-FT. HEAD OF WATER AFTER CRACKING.

FIG. 89. ISOMETRIC DIAGRAM OF DEFLECTIONS FOR 40-FT. HEAD OF WATER BEFORE CRACKING.

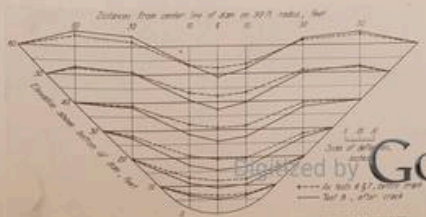

FIG. 90. COMPARISON OF DEFLECTIONS BEFORE AND AFTER OCCURRENCE OF CRACK; HEAD OF WATER 40 FT.

FIG. 91. ISOMETRIC DIAGRAM OF DEFLECTIONS FOR 30-FT. HEAD OF WATER.

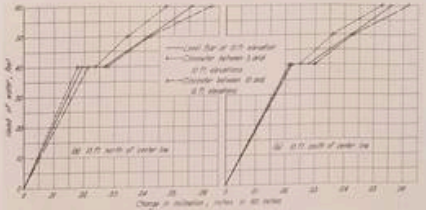

FIG. 92. COMPARISON BETWEEN CHANGES OF INCLINATION AT 10-FT. ELEVATION BY CLINOMETER AND LEVEL-BAR.

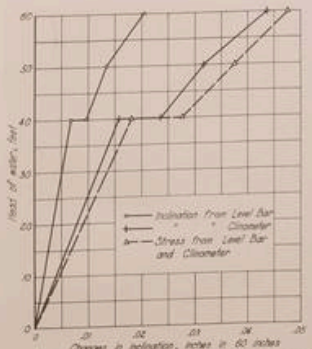

FIG. 93. TILTING AT BOTTOM OF DAM AND CORRESPONDING STRESS BASED ON CONSERVATION OF PLANE SECTIONS.

FIG. 94. LINES OF EQUAL HORIZONTAL STRAINS ON DOWNSTREAM FACE FOR 60-FT. HEAD OF WATER.

FIG. 95. LINES OF EQUAL VERTICAL STRAINS ON DOWNSTREAM FACE FOR 60-FT. HEAD OF WATER.

FIG. 96. LINES OF EQUAL HORIZONTAL STRAINS ON DOWNSTREAM FACE FOR 50-FT. HEAD OF WATER.

FIG. 97. LINES OF EQUAL VERTICAL STRAINS ON DOWNSTREAM FACE FOR 50-FT. HEAD OF WATER.

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FIG. 98. LINES OF EQUAL HORIZONTAL STRAINS ON DOWNSTREAM FACE FOR 40-FT. HEAD OF WATER AFTER VERTICAL CENTER CRACK AT TOP HAD OCCURRED.

FIG. 99. LINES OF EQUAL VERTICAL STRAINS ON DOWNSTREAM FACE FOR 40-FT. HEAD OF WATER AFTER VERTICAL CENTER CRACK AT TOP HAD OCCURRED.

FIG. 100. LINES OF EQUAL HORIZONTAL STRAINS ON DOWNSTREAM FACE FOR 40-FT. HEAD OF WATER BEFORE VERTICAL CENTER CRACK AT TOP HAD OCCURRED.

FIG. 101. LINES OF EQUAL VERTICAL STRAINS ON DOWNSTREAM FACE FOR 40-FT. HEAD OF WATER BEFORE VERTICAL CENTER CRACK AT TOP HAD OCCURRED.

FIG. 102. LINES OF EQUAL PRINCIPAL STRAIN AND DIRECTIONS OF PRINCIPAL STRAIN FOR 60-FT. HEAD OF WATER.

FIG. 103. EXTREME RANGE IN DIRECTION OF PRINCIPAL STRAIN FOR 60-FT. HEAD OF WATER.

FIG. 104. STEVENSON CREEK EXPERIMENTAL ARCH DAM AFTER THE FLOOD OF NOVEMBER, 1906.

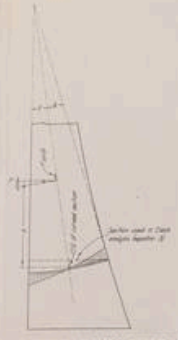

FIG. 105. CURVED SECTION USED IN DETERMINING EFFECT OF TAPER.

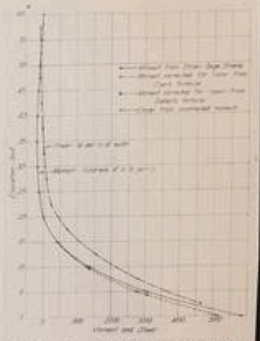

FIG. 106. MOMENT AND SHEAR FOR VERTICAL CENTER LINE UNDER 60-FT. HEAD; MOMENTS CORRECTED FOR TAPER OF VERTICAL ELEMENT.

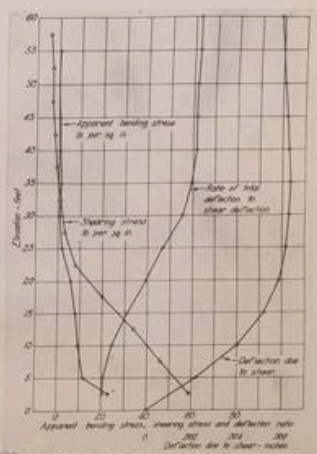

FIG. 107. INFLUENCE OF SHEAR ON DETERMINATION OF BENDING STRESSES.

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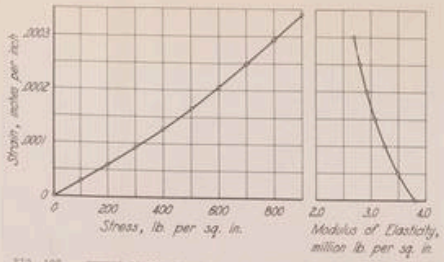

FIG. 108. STRESS-STRAIN RELATION AND MODULI OF ELASTICITY FROM CYLINDER TESTS.

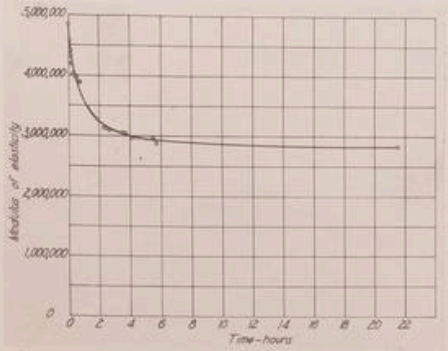

FIG. 109. EFFECT OF TIME UNDER LOAD ON MODULUS OF ELASTICITY.

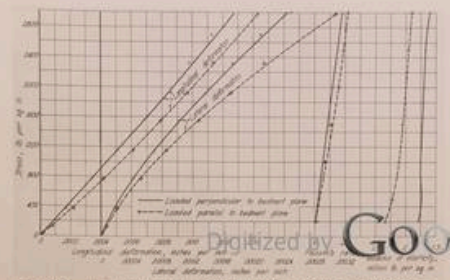

FIG. 110. LATERAL AND LONGITUDINAL STRAINS, MODULUS OF ELASTICITY, AND POISSON'S RATIO FOR GRANITE TAKEN FROM THE FOUNDATION

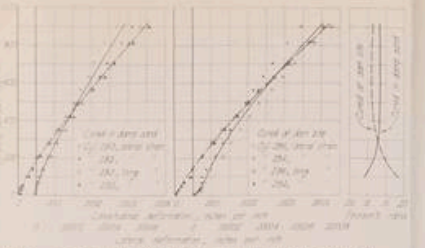

FIG. 111. LATERAL AND LONGITUDINAL STRAINS AND POISSON'S RATIO FOR CYLINDERS 283 TO 286.

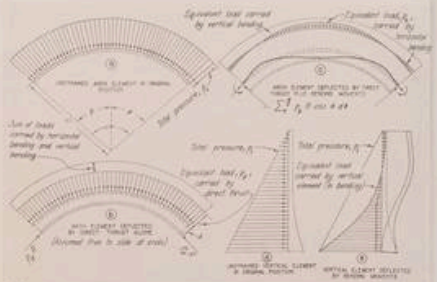

FIG. 112. DIAGRAMMATIC REPRESENTATION OF LOAD DISTRIBUTION ON ELEMENTS OF DAM.

FIG. 113. DIRECT STRAINS FOR 60-FT. HEAD; AVERAGE STRAINS AND DOWNSTREAM TELEMETER STRAINS.

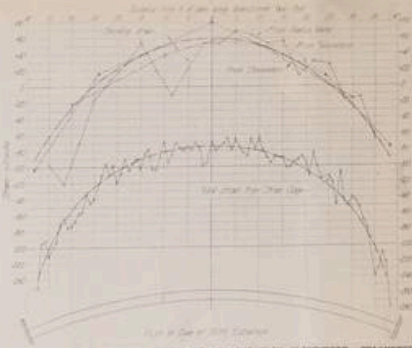

FIG. 114. COMPARISONS OF BENDING STRAINS FROM CLINOMETER, TELEMETER AND RADIUS-METER DATA WITH TOTAL STRAINS FROM STRAIN-GAGE, FOR HORIZONTAL ELEMENT AT ELEVATION OF 30 FEET, UNDER HEAD OF 60 FEET.

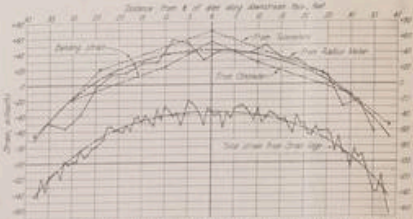

FIG. 115. COMPARISONS OF BENDING STRAINS FROM CLINOMETER, TELEMETER AND RADIUS-METER DATA WITH TOTAL STRAINS FROM STRAIN-GAGE, FOR HORIZONTAL ELEMENT AT ELEVATION OF 30 FEET, UNDER HEAD OF 50 FEET.

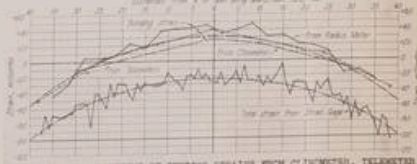

FIG. 116. COMPARISONS OF BENDING STRAINS FROM CLINOMETER, TELEMETER AND RADIUS-METER DATA WITH TOTAL STRAINS FROM STRAIN-GAGE, FOR HORIZONTAL ELEMENT AT ELEVATION OF 30 FEET, UNDER HEAD OF 40 FEET AFTER CRACKING.

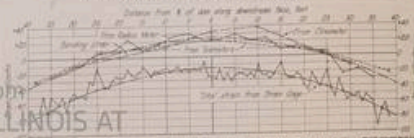

FIG. 117. COMPARISONS OF BENDING STRAINS FROM CLINOMETER, TELEMETER AND RADIUS-METER DATA WITH TOTAL STRAINS FROM STRAIN-GAGE, FOR HORIZONTAL ELEMENT AT ELEVATION OF 30 FEET, UNDER HEAD OF 40 FEET BEFORE CRACKING.

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FIG. 118. CHANGE IN LENGTH OF A SEGMENT OF THE ARCH AT THE NEUTRAL AXIS.

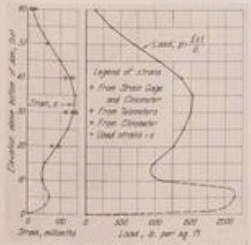

FIG. 119. AVERAGE DIRECT STRAIN AND CORRESPONDING LOAD FOR VARIOUS ELEVATIONS WITH WATER AT 60-FT. ELEVATION.

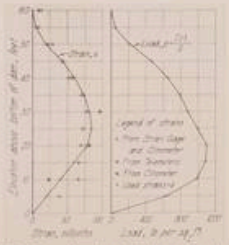

FIG. 120. AVERAGE DIRECT STRAIN AND CORRESPONDING LOAD FOR VARIOUS ELEVATIONS WITH WATER AT 50-FT. ELEVATION.

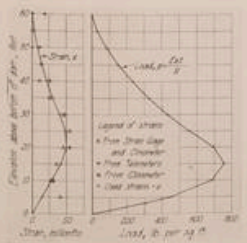

FIG. 121. AVERAGE DIRECT STRAIN AND CORRESPONDING LOAD FOR VARIOUS ELEVATIONS WITH WATER AT 40-FT. AFTER CRACKING.

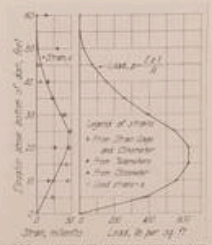

FIG. 122. AVERAGE DIRECT STRAIN AND CORRESPONDING LOAD FOR VARIOUS ELEVATIONS WITH WATER AT 40-FT. BEFORE CRACKING.

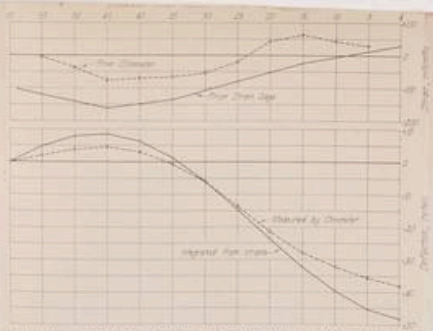

FIG. 123. COMPARISON OF MEASURED DEFLECTIONS OF HORIZONTAL ELEMENT AT 50-FT. ELEVATION WITH DEFLECTIONS FOUND BY INTEGRATION OF HORIZONTAL STRAINS; HEAD 60 FT.

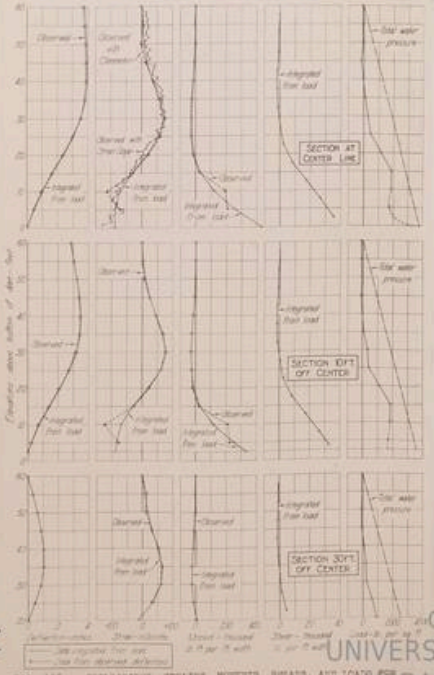

FIG. 126. DEFLECTIONS, STRAINS, MOMENTS, SHEARS, AND LOADS IN VERTICAL ELEMENTS AT CENTER LINE AND 10 AND 20 FT. FROM CENTER LINE. HEAD OF WATER 60 FEET. WITHOUT CRACKS; AT TOP AND BOTTOM FOR TESTS 11, 12, AND 13.

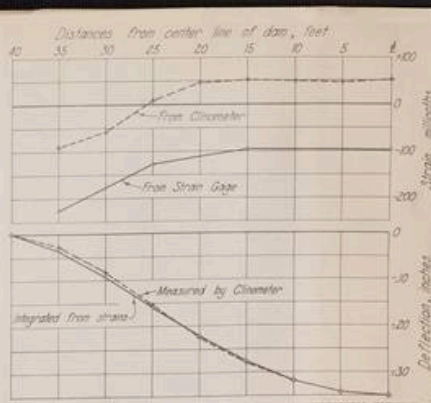

FIG. 124. COMPARISON OF MEASURED DEFLECTIONS OF HORIZONTAL ELEMENT AT 50-FT. ELEVATION, WITH DEFLECTIONS FOUND BY INTEGRATION OF HORIZONTAL STRAINS; HEAD 60 FT.

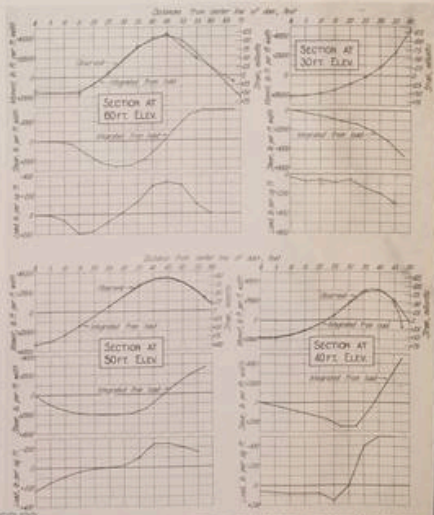

FIG. 125. BENDING STRAINS, SHEARS, AND LOADS IN HORIZONTAL ELEMENTS AT 70, 60, 50 AND 40-FT. ELEVATIONS UNDER A HEAD OF

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FIG. 118. CHANGE IN LENGTH OF A SEGMENT OF THE ARCH AT THE NEUTRAL AXIS.

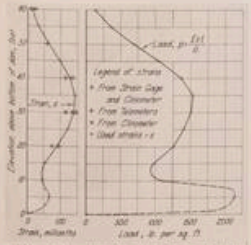

FIG. 119. AVERAGE DIRECT STRAIN AND CORRESPONDING LOAD FOR VARIOUS ELEVATIONS WITH WATER AT 60-FT. ELEVATION.

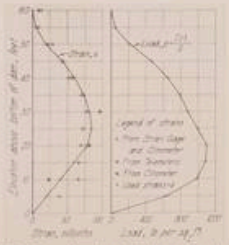

FIG. 120. AVERAGE DIRECT STRAIN AND CORRESPONDING LOAD FOR VARIOUS ELEVATIONS WITH WATER AT 50-FT. ELEVATION.

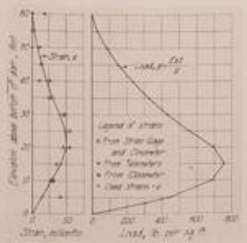

FIG. 121. AVERAGE DIRECT STRAIN AND CORRESPONDING LOAD FOR VARIOUS ELEVATIONS WITH WATER AT 40-FT. AFTER CRACKING.

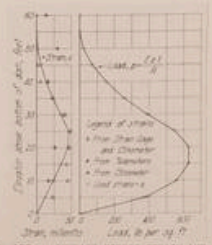

FIG. 122. AVERAGE DIRECT STRAIN AND CORRESPONDING LOAD FOR VARIOUS ELEVATIONS WITH WATER AT 40-FT. BEFORE CRACKING.

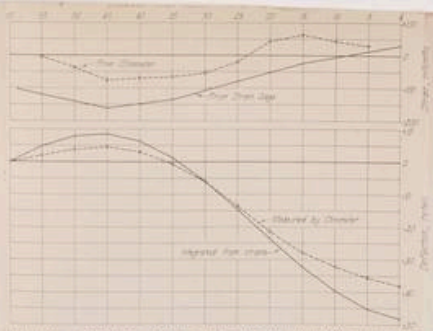

FIG. 123. COMPARISON OF MEASURED DEFLECTIONS OF HORIZONTAL ELEMENT AT 50-FT. ELEVATION WITH DEFLECTIONS FOUND BY INTEGRATION OF HORIZONTAL STRAINS; HEAD 60 FT.

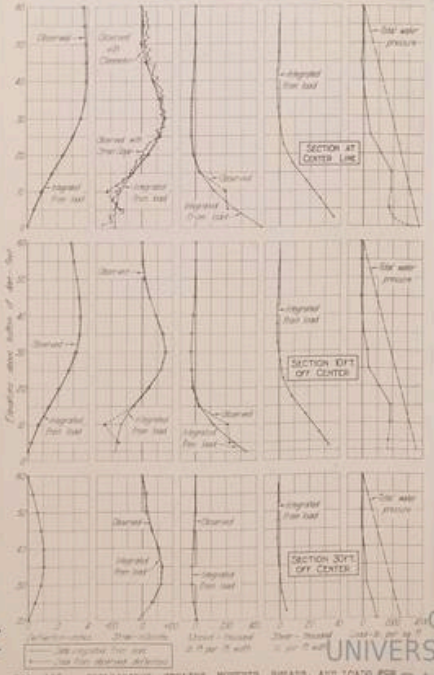

FIG. 126. DEFLECTIONS, STRAINS, MOMENTS, SHEARS, AND LOADS IN VERTICAL ELEMENTS AT CENTER LINE AND 10 AND 20 FT. FROM CENTER LINE. HEAD OF WATER 60 FEET. WITHOUT CRACKS; AT TOP AND BOTTOM FOR TESTS 11, 12, AND 13.

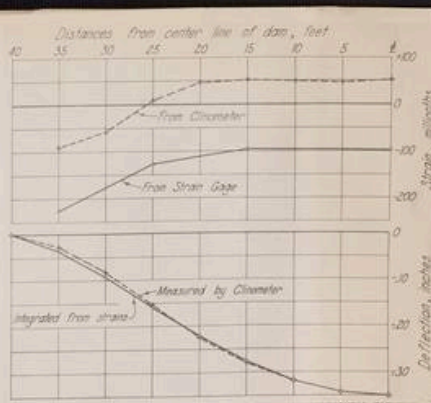

FIG. 124. COMPARISON OF MEASURED DEFLECTIONS OF HORIZONTAL ELEMENT AT 50-FT. ELEVATION, WITH DEFLECTIONS FOUND BY INTEGRATION OF HORIZONTAL STRAINS; HEAD 60 FT.

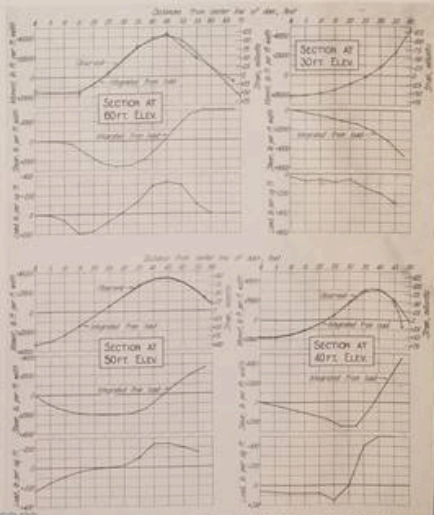

FIG. 125. BENDING STRAINS, SHEARS, AND LOADS IN HORIZONTAL ELEMENTS AT 70, 60, 50 AND 40-FT. ELEVATIONS UNDER A HEAD OF

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