Mr Cox: Will you please have this article typed here to ask you for it again. L.F.

THE MULTIPLE ARCH DAMS
on
Rush Creek, Mono Co. Cal.

INTRODUCTION.

During the last few years several multiple arch dams have been built in various parts of the country, and they have so far been very successful in operation. Many more would undoubtedly have been constructed had a thorough knowledge of their design and construction been more general among dam building engineers. There are places where rock or earthfill dams, or a combination of the two, are now built, where multiple arch dams could have been constructed cheaper and more substantial.

While a rock or earthfill dam, can, under ordinary conditions, be constructed with a sufficient, ~~although~~ unknown factor of safety, such dams are absolutely unsafe under abnormal conditions, such as when water accidentally passes over their crest. It is a well known fact that nearly all failures in the past have been due to this cause. Water passing over the crest of a multiple arch dam would not destroy the dam, and for a more or less limited time would not hurt the foundation if this was otherwise at all safe for such a structure. A multiple arch dam requires a good foundation, as the load is concentrated on buttresses and settlement of these would be liable to cause the collapse of adjoining arches. Whenever the foundation is solid rock, however, a multiple arch dam can be constructed as substantial as any type of dam, and more substantial than most types. The stresses and dimensions can be calculated with a large degree of accuracy. The factor of safety of such a structure is therefore known within narrow limits, assuming first class construction work, and

precedence should, on that account, not be given so much consideration as with rock and earthfill dams. These latter cannot be subjected to calculation of stresses, and therefore have to be built mostly along lines dictated by precedence. In general, it can be said that a multiple arch dam of small and medium height (less than 100 feet high) will cost less to construct than a rockfill dam, and especially a rockfill provided with something better than a wooden upstream face for the watertight cut-off. Only perhaps in rare cases would there be occasion for comparing the relative cost of a multiple arch dam and the cost of a strictly earthfill dam, because if there is enough suitable earth to construct an earth dam, a sufficient good foundation for a multiple arch dam and sufficient good building material for the same is not likely to be found at the same place, and vice versa.

For multiple arch dams above, say 130 feet high, the amount of building material required, and therefore the cost of such a structure, increases quite rapidly, due mainly to the fact that the buttresses become very large and require more bracing. A limit of height is therefore eventually reached where it will be more economical to build one single arch across the canyon, unless the canyon be very wide. Where this limit of height for any dam lies can only be found by trying all the types possible of application, as the shape of the dam site has also quite an influence in the matter.

THE DESIGN OF THE MULTIPLE ARCH DAM.

The first thing to be determined is the length of each

individual span. Unless there should be strong reasons for using different span lengths for the several arches making up the complete structure, all spans should be made the same to facilitate the form work. Theoretically the shorter the spans are chosen, the less material is required for the arches. The material required for the buttresses remains theoretically the same, no matter what length of span is chosen. While a dam consisting of small spans takes less material than one where larger spans are used, it may not necessarily be cheaper to construct. The form work becomes more extensive and it is more difficult to place the concrete and reinforcing steel in the resulting narrow space between the form boards than in a wider space. Thin arch walls are more liable to collapse than thicker walls, and thin buttresses would require elaborate bracing to prevent collapse of same before anywhere near crushing strength had been reached. It is the arch that holds the water back, and therefore the watertightness of the dam is to some extent a function of the thickness of this wall, although to a much larger extent it depends upon the quality of the building material (concrete) used.

Taking all these facts into consideration it can be said that the practical, most economical span, lies somewhere between the limits of 30 feet and 50 feet. For high dams the economical span is near the upper limit, and for low dams, near the lower limit. A 40 foot span would be a good average value for ordinary cases, and is chosen in the present instance.

Next to be determined is the length of the upstream radius. It is known that the most economical arch** is the one that subtends an angle of $133\frac{1}{3}^{\circ}$ and that for variations of about 10% on either side of this angle, the difference in economy is very small. For the dams to be described later the subtended central angle at the upstream face has been chosen 120°, or to be exact, 119° 57'. The volume of the arch has thereby been increased approximately 1% above the theoretical minimum, but the thickness has, at the same time, been increased 6%, thereby decreasing the ratio of thickness of arch to length of arch, which is a desirable feature for structural reasons, at least towards the crest where the thickness is small compared with the length. This also decreases the tendency of percolation by decreasing the area of the wetted surface, and by increasing the thickness of the wall.

With the subtended angle (120°) and the span (40 Ft.) decided upon, the length of the upstream radius is calculated to be 23.1 Ft. To facilitate formwork the length of this radius is kept constant from crest to foundation, except as noted later. Incidentally this gives also the most economical arch as the subtended angle is thereby kept practically constant. The arch is given a slope with the horizontal of 50°, in order that the water pressure, acting upon the vertical projection of this slope, may tend to cut down the shearing stress on the buttresses to zero or to some insignificant value.

** For further information see Page 689 and 690, Transactions American Society Civil Engineers, Volume 78, Paper #1322.

The arch carries the total waterload, and a large part of the load due to its own weight. A preliminary arch thickness may be found by means of the simple formula $P \times Ru = qxt$ -- (1) in which P equals the water pressure in pounds per square foot. Ru equals the length of the upstream radius in feet, q equals the average stress in pounds per square foot of the area of the dam section under consideration, and t equals the thickness of the dam at any given horizontal elevation. ***

After the thickness t has been determined, at as many points as thought necessary, say at every 10 feet apart in elevation, the weight of the arch can be calculated and the additional arch stress due to this load determined. If the total stress is found to be excessive a new thickness will have to be decided upon.

Due to the inclination of the arch, the water pressure is not uniformly distributed between the crown and the abutments on any imaginary arch slice perpendicular to the slope. The crown is located at a higher elevation, and therefore sustains less water pressure than corresponding points at the abutments.

For the case under consideration, where the length of the upstream radius is 23.1 Ft., the subtended angle 120° and the inclination of the arch 50° with a horizontal plane, 40° with a vertical plane, a point at the crown will always be $\frac{Ru}{2} \times \sin 40^\circ = 11.55 \times 0.64379 = 7.394$ feet above a corresponding point at the

*** The thickness of any horizontal arch slice (elliptical with major axis $= \frac{Ru}{\sin 50^\circ}$ and with minor axis = Ru) is made constant from abutment to abutment and therefore the circular arch perpendicular to the slope must have its thickness increase from the crown towards the abutments.

springing line. Towards the crest this difference in elevation makes a large difference in the distribution of the load on the face of the arch, and therefore also in the location of the line of pressure in the arch ring. Take, for instance, an arch slice 1 foot wide with the middle of the crown at elevation 5, and with the middle of this slice at the springing line at elevation 5 + 7.394 = elevation 12.394. With reservoir full to the crest, the waterload at the middle point of the crown would correspond to 5 X 62.5 = 312.5 pounds per sq. ft., whereas the water pressure at the corresponding point at the springing line would correspond to 12.394 X 62.5 = 774.6 pounds, or nearly 2.5 times more than at the crown.

At lower elevations this large difference (2.5 times) is fast disappearing and becomes unimportant with crown elevations below elevation 15. In the upper portion of the dam the radial component of the arch weight borne directly by the arch itself tends towards neutralizing the large difference in water pressure between the crown and the corresponding abutment points, as this weight component has its largest value at the crown, and its smallest value at the abutments. This can most clearly be shown graphically, and in Fig. 1 and Fig. 2, two arch slices are drawn, assumed to be one foot wide. One (Fig. 1) with the crown at elevation 5 (The middle of the crown at elevation 5, and then six inches on either side), and the other (Fig. 2) with its crown at elevation 10. The arch is divided up into 20 voussoir's and the forces acting on these voussoirs are calculated. These forces, while not uniformly

distributed, are of course symmetrical on both halves of the arch with respect to the centerline between two buttresses.

On the right half of the diagrams are shown the weights of the different voussoirs dissolved into their components. The weight of a voussoir is partly transmitted to the base through the lower portion of the arch, and partly supported by the arch. The last component is equal to the total weight of the voussoir X Cos. 50°, and is set off vertically in the diagram, acting through the center of gravity of each voussoir. This vertical load is dissolved in one radial component and one perpendicular thereto. The radial component can now be directly **** added to the waterload on the same voussoir, which is of course also radial. All the last named (perpendicular) components added together geometrically give the increased axial stress towards the abutments.

On the left half of the diagrams, the total radial forces acting on the different voussoirs are indicated. A polygon of forces is drawn, and by means of this polygon the line of pressure of the arch can be determined by drawing lines parallel to the proper rays in the polygon of force. It is plainly seen that the line of pressure lies decidedly outside the centerline of the arch, especially is this true of the arch with its crown at elevation 5 (Fig. 1.).

By means of these diagrams the correct shape of the wooden arch trusses supporting the formwork for these upper elevations

**** For greater accuracy this force should be multiplied by the ratio $\frac{\text{length of mean radius}}{\text{length of upstream radius}}$. It has been neglected in the present case as this ratio is very close to unity.

can be ascertained. The outside members of these trusses are made elliptical above elevation 15, as shown in detail on Fig. 3 in such a way as to bring about coincidence between the line of pressure and the center line of the arch. At elevation 15 the center line of the arch and the line of pressure do not exactly coincide, but they are close enough together to allow the circular shape of the arch to be used with entire safety. Below elevation 15 the arch is made circular in a plane perpendicular to its sloping axis, and above elevation 15 it is made elliptical in accordance with the diagrams Fig. 1 and Fig. 2, or a slight modification thereof, depending upon how far below the crest the maximum water level is to be. It might not be out of place to state here that Mr. J. S. Eastwood, Member American Society Civil Engineers, who has been very prominent in bringing into actual use the multiple arch type of dam, builds the top portion of the arches vertical, whereby the circular shape can be used for the entire arch.

STRESSES DUE TO TEMPERATURE CHANGES.

As dams are generally built during the Summer season, it is only logical to assume that after their completion the individual arches are under tensile stress most of the time when the reservoir is empty, and decidedly so if it is empty during the cold season. The reinforcement in the arch has therefore been placed with the sole purpose of taking up these tensile stresses, which reach their maximum value near the downstream face at the crown, and near the upstream face at the abutments, under the above stated conditions. From Fig. 4 it will be seen that the reinforcing

steel is located at a distance equal to one fourth the total arch depth from the respective faces subject to tensile stresses at the crown and at the abutments. The amount of steel put in the arch (See Fig. 4 and Fig. 4a) is perhaps not entirely sufficient to take care of the maximum condition of temperature drop, but it is believed that if tension cracks do develop, the presence of the reinforcement will cause these to be minute and well distributed, and that when the structure becomes loaded the cracks will close tight. It was not deemed advisable to put more steel in the arches than shown, for the reason that it is of comparatively little use when the reservoir is full. It was also kept in mind that a large change in temperature is not liable to occur suddenly, but a time element of perhaps weeks or months is generally interposed between the occurance of maximum and minimum temperature in a dam body. This time factor can to some extent be depended upon to prevent or minimize temperature cracks. It gives the modulus of elasticity time to adjust itself to the new condition (colder or warmer concrete).*****

THE STABILITY.

After the buttresses have been given some preliminary dimensions the stability of the whole structure can most conveniently be investigated graphically as shown on Fig. 6 representing a section through the crown of the arch. To facilitate the investigation the dam is divided up into horizontal sections 10 feet apart

*****That the deformation of concrete under sustained load increases has been shown by tests and in practice. Two papers on the subject were read at the Twelfth Annual Convention of the American Concrete Institute in Chicago, one by Professor A.H.Fuller and Professor C.C.More, entitled "Tests showing continued deformation under con-

stant load", and one read by Mr. Carl B. Smith entitled "The Flow of Concrete Under Restrained Load". This was abstracted in the ENGINEERING RECORD for March 4th, 1916, Page 329. See also "The University of Minnesota" "Studies in Engineering #3" by F. R. McMillan."

in elevation, and the forces acting on and above each section are shown to scale in the location and direction in which they act.

The most important force acting on the structure is the water pressure. This force is, as usual, assumed concentrated in a horizontal plane, located two thirds of the total depth below the water surface.

The horizontal plane in which the water pressure is assumed concentrated intersects the upstream face along an elliptical curve. The point of application of a single force representing the water pressure on one total span of 40 feet coincides with the center of gravity of this ellipse, at least as long as the water pressure does not penetrate the upstream face skin, such as plastering, etc.

Due to the fact that the arches have been given a slope of 50° with the horizontal, the water pressure will have a vertical component = $\frac{\text{Horizontal Component}}{\text{tg. } 50^{\circ}}$. The stability of the dam depends to a very great extent upon the presence and action of this component, as it tends to hold the structure firmly down upon its foundation.

The point of application of the water pressure is, as already stated, taken at the center of gravity of the ellipse. From the crown this is located a little over one third the total

distance (1/3 + 5% for present conditions) between the crown and springing line.

Considering first the upper 10 feet of the dam, the horizontal plane in which the water pressure can be assumed concentrated is at a distance of $2/3 \times 10$ from elevation 0 downward with the water surface at elevation 0, and the point of application "a" of the single force is $(1/3 + 5/100) \frac{11.55}{\sin 50^\circ} =$ 5.78 Ft. from the crown measured horizontally.

The horizontal water pressure due to the 10 feet of water on the 40 foot span is $\frac{0 + 625}{2} \times 10 \times 40 = 125{,}000$ Lbs. or 62.5 tons. The vertical water pressure is equal to $\frac{62.5}{\text{tg } 50^\circ} = 52.5$ Tons. This latter force is now combined with the portion of the weight of the arch acting vertically. The weight of the arch above elevation 10 is equal to its volume in cubic feet times the weight of one cubic foot. The center of gravity of the section - a trapezoid - is found, and the center of gravity "b" of the whole arch is then taken to coincide with the center of gravity of an ellipse through the center of gravity of the section in the same manner as explained above for the water pressure. The weight of the arch is now combined with the vertical component of the water pressure and the location of their resultant is found by taking moments around either point "a" or "b". The numerical value of the resultant is equal to the sum of the two forces $52 + 52.5 = 104.5$ Tons. Taking moments around point "a" and scaling the distances, preferably on the sloping line between "a" and "b" for greater accuracy, we have

the equation $\frac{52 \times 2.75}{104.5} = X = 1.367$ Ft., giving the location of the resultant at a point 1.367 Ft. from "a" along the line between "a" and "b". The weight of the buttresses (above elevation 10) assumed concentrated in the center of gravity "d" is calculated to be 14.5 Tons; it is combined with the vertical load on the arch of 104.5 Tons in the same manner as shown above by taking moments around any point, say "c". The location of the resultant is found to be at "e", and the value of the same is $104.5 + 14.5 = 119$ Tons. This represents the total vertical force both as to size and location.

This force is now combined with the horizontal water pressure of 62.5 Tons and the resultant drawn. It is the purpose of this diagram Fig. 6 to give the value of the resultant of all forces and to establish the point of intersection "g" between this resultant and the base, in this case at elevation 10. For convenience the distance from e to f representing the total vertical force of 119 Tons may be measured, and the distance f - g representing the 62.5 Tons horizontal water pressure set off on the base to the same scale.

Considering next the portion of the dam above elevation 20 as a whole. The place of application of the concentrated water pressure is located at a distance of $2/3 \times 20$ Ft. below elevation 0, and the point of application of a single force representing the water pressure on the 40 Ft. span is located the same as before, 5.78 Ft. from the crown towards the springing line, measured horizontally. The vertical component of the water pressure =

$\frac{\text{horizontal component}}{\text{tg } 50^{\circ}}$ acting on the arch between elevation 0 and elevation 20 is now combined with the weight of the arch lying between Elevation 0 and elevation 20 and their resultant drawn in the correct location, found by taking moments as shown above. This resultant is again combined with the weight of the portion of the buttress lying between elevation 0 and elevation 20. The shape of the buttress is taken as that of an obelisk, and its volume and the location of the center of gravity is found from ordinary rules applying to such bodies. The weight of the struts and counterforts are to be added to the weight of the buttress, whereby the location of the center of gravity might be slightly changed.

The resultant of all vertical forces acting on the dam above elevation 20 can now be found, both as to size and location, and combined with the horizontal water pressure, whereby the point of intersection of the resultant with the base at elevation 20 is determined.

The same method of procedure is followed out for the remaining portion of the dam, taking each time the base 10 feet lower than in the preceeding calculations, and the whole diagram Fig. 6 is completed. If now the centerline of the buttress, and also the two lines representing the middle thirds, are drawn, this diagram will point out very clearly whether the load is distributed economically on the buttress or not. For maximum economy ~~and safety,~~ the resultant of all forces should intersect the base in the centerline of the buttress, then the load will be uniformly distributed over

the whole base. Towards the top this is not quite possible, and is not important, as the material here cannot be stressed very highly at any rate, but towards lower elevations the downstream slope of the buttress should be shaped such as to conform with this condition, viz: The resultant intersecting the base in the centerline, or approximately in the centerline of the buttress. The total vertical load on the section shown in Fig. 6 is seen to be 6332 tons, and the horizontal water pressure 5062 tons, both on a 40 foot span. If the coefficient of friction is taken at 0.75 it is seen that the actual shear along the base amounts to only 5062 - 6332 X 0.75 = 313 tons. There is considerable steel in the section to help take up this shear, and therefore it was not deemed necessary to eliminate the shear entirely. There is no hydrostatic uplift to amount to anything acting on this type of dam, and water could hardly find its way to lubricate the surfaces of possible cracks in the buttresses. Wherever it would be desirable to eliminate the shear entirely, the face slope should be made more flat, say 45°, instead of 50°. This, of course, adds to the material required for construction, but is the cheapest and best way of accomplishing the result.

Some difference of opinion may well exist as to what is the actual stress per square unit of area of the buttress at any horizontal elevation. The loads per buttress are given on diagram Fig. 6. Take, for instance, at elevation 80, the vertical load is seen to be 4977 tons, and the horizontal load 4000 tons. As the horizontal area is 360 square feet, the unit vertical stress should

be $\frac{4977}{360} = 13.82$ tons per square foot, or 192 pounds per square inch. The shear would be a little more than friction alone would take care of, but steel is provided for the remaining. The resultant of the two (horizontal and vertical) forces intersects the base 2 feet downstream relative to the centerline, but on account of having the counterfort (20 sq. ft.) on this side the centerline also, the stress is actually uniformly distributed. Now if instead of the two principal forces, their resultant is used, assuming it acting on a number of steps perpendicular to the direction of this force (the resultant), the apparent unit stress will be much higher. Thus the resultant (6385 tons) acting on an area equal the sum of all the steps (360 X sin 51^0- 13' = 280 Sq.) will produce a compression equal to $\frac{6385}{280}$ = 22.8 tons per square foot or 317 pounds per square inch.

There is a large difference between 192 pounds per square inch and 317 pounds per square inch. The actual unit compression will be lying somewhere in between, depending undoubtedly upon the relative value of the modulus of elasticity of the concrete for compression and for shear. It is seen that the first method, using the principal forces, gives the maximum possible shear that could occur, and that the last method gives the maximum possible unit compression that could occur. None of these methods are very satisfactory as they do not fix the absolute value of stress within narrow enough limits, but the writer knows of no better at present.

The reinforcing steel embedded in the buttress is put there for different purposes. Along the upstream slope the triangular

steel construction shown in Fig. 4 and Fig. 5 ties the adjacent arches into the buttress. This is desirable on account of the fact that in order to facilitate construction, the buttresses are built first, and later the arches. The hooping that interconnects the different bars is simply left protruding through the concrete of the buttesss at the time this is built. Should one arch fail this triangular girder would immediately take up the unbalanced thrust and prevent adjacent arches from collapsing, and this is its principal duty. Besides this, however, the steel is active in preventing cracks in the buttess and in taking up some shear. Towards the downstream edge of the buttess four vertical rods are embedded in the concrete for the purpose of stiffening the same, preventing cracks and taking care of wind pressure. Towards the middle portion reinforced counterforts are constructed for the same purpose. These are still more effective in accomplishing this, due to the greater distance between the pairs of steel rods. All material in the counterforts supports load the same as the buttress itself, but they are at the same time most effective in stiffening the large flat slab (buttress) so that fewer struts are required.

Towards the top of the buttress (at elevation 15) two horizontal struts are tied to the vertical reinforcement. (Fig. 4 and Fig. 5) These two struts are so designed that besides their main purpose of holding the upper portion of the buttress in place, they are capable of supporting a light roadway above them. At

elevation 45 another strut is located near the upstream face, mainly in order to support the triangular girder, should this latter ever be called upon to support any unbalanced arch pressure. All struts are able to withstand tension as well as compression, as can be judged from an inspection of the details shown on Fig. 4 and Fig. 5.

The design shown in the foregoing illustrations was made by the writer for the Pacific Power Corporation, of Bodie, California, to be used for the construction of their Gem and Agnew Lake dams, situated on Rush Creek, Mono County, California. Fig. 7 is a plan view of the Gem Lake dam and dam site, and Fig. 8 shows the Agnew Lake dam site. On Fig. 7 is also indicated the outlet works, consisting of a concrete chamber provided with a row of iron bars in front, and a 48" diameter steel pipe laid through a tunnel for a distance of approximately 300 feet. This pipe line terminates in a power house located 1808 feet below (in elevation) the top contour of the lake. The maximum height of any individual arch of this dam is 84 feet, and the vertical distance from the deepest point in the foundation to the crest is 112 feet. The length across the dam site at crest elevation 9053 is 700 feet. The reservoir created is of 17,000 acre feet capacity, capable of regulating Rush Creek to yield an estimated average flow of 45 second feet. The drainage area above the dam is 22½ square miles, and is located on the Eastern slope of the Sierra Nevada mountains, between elevations 9,000 feet above sea level at the dam, and 12,000 feet along the crest of the mountains. The precipitation at these

elevations is still high, but decreases very rapidly from the foot of the steep mountains out over the desert situated immediately East at elevations ranging between 6,000 and 7,000 feet above sea level.

The rock at the dam site was, for the most part, exposed bedrock worn clean by glacial action. Some excavation was, however, necessary in the stream bed and through a rock slide at the northerly abutment. The building material for the dam was found near by; the sand along the shore of the natural lake, while the rock had to be hauled a short distance on a tramway, first from the outlet tunnel dump (limestone), and later from a large rock slide (granite) about 2,500 feet away. All available material in the neighborhood, especially the different sand deposits, were tested before any particular material was decided upon for construction. It was found that the sand deposit along the shore of Gem Lake was good, and therefore used. This sand was first pumped, and later shoveled from the lake and transported to a storage pile near the mixing plant. This lake sand, which contained 3½% of clay and 1% of dirt, was mixed with the sand from the rock crusher (all particles below ¼" size) in the proportion of about three fourths of lake sand to one fourth of crushed rock sand. This gave a very good combination both as to strength and watertightness.

Compression tests on 6" cylinders were made as construction work progressed, using Bear Brand of Portland cement, Gem Lake sand, and crushed rock in the proportions 1:2:4, and averaged about 900 pounds per square inch for crushing at the age of 14 days.

A 1:2:4 mix was adopted for the arches and struts, and a 1:2½:5 mix for the buttresses. The actual proportion, however, was sometimes changed, but 1½ barrels of cement for the arches, and 1¼ barrels for the buttresses was used always. The rock was crushed in a gyratory crusher, and separated into three sizes through a revolving screen having 1½", ¾" and ¼" meshes. The rejects from the screen went into a jaw crusher, the jaws of which were set to give a 2" maximum size rock.

The distribution of the concrete to the different arches and buttresses was done by means of two wheeled push carts and short shutes.

During the construction period in the Summer of 1915 and 1916, tension tests were made of cement briquettes in a small field laboratory. Much attention was also paid to the sand used, and 1:2 and 1:3 mortar briquettes were tested frequently, and silt analysis were made as the work progressed.

The reinforcing placed in the dam consists of high carbon steel bars, either corrugated or twisted. The position and details of these bars is fully shown in the different views, and no further explanation need be given.

For form lumber the trees standing on the reservoir site were cut down and sawed into lumber in a mill erected on the ground by the contractors. Large cone bearing trees can be found in this neighborhood up to elevation 11,000 in fairly large quantities, but not of the best quality. The lumber proved to be good enough for form work, was used green, and on that account 3/4" x 4" battens

were placed on the under side of the sloping arch to avoid leakage of grout between boards when shrinkage took place. These battens also helped to give an even circular shape to the outer ¾" x 12" boards, constituting the form for the downstream face. This sheeting was supported on 2" x 6" studs spaced 2' 6" apart toe-nailed on to the wooden arch trusses shown in Fig. 3 and Fig. 9. These trusses were spaced 5 feet apart in elevation, and a little over 6½ feet apart measured along the arch slope. The formwork for the upstream face consisted also of ¾" x 12" boards nailed to 2" x 6" studs, which in turn were nailed to circular boards at spaced intervals. This outside form was held away from the inside form (downstream side) by means of wooden distance pieces of the proper length, which pieces were removed just before the concrete reached them.

A 1:2 plater coat of cement mortar was put on the upstream face by means of a cement gun, ¼" thick at the crest, and increasing to ¾" thick at elevation 80 feet below.

At the Southerly abutment the two last arches are provided with spillway openings. These spillway openings can be closed with loose flashboards. This it is proposed to do towards the end of the wet season so as to fill the reservoir to within, say one foot from the crest of the dam, thereby gaining two feet of water over an area of nearly 300 acres. The spillway is shown in detail on Fig. 10.

COST:

The Gem Lake dam contains 8537 cubic yards of concrete and 82 tons of reinforcing steel. The contract price was $22.00

per cubic yards including cement, forms, platering the upstream face, all tools and materials except the reinforcing steel, which was paid for extra at the rate of $110.00 per ton in place. The excavation, of which there was only a limited amount, was also paid for extra. The high cost is explained by the fact that freight rates were high, and further that the distance from the railroad to the power house site at Silver Lake, at the foot of the steep mountains, was nearly 60 miles over desert roads with heavy grades. From the power house site, a tramway approximately 4,500 feet long took all supplies up the mountain side to a lake (Agnew Lake) situated 1250 feet above. At the outlet of this lake the Agnew dam, a smaller structure 30 feet high, and 280 feet long, similar in design to the Gem Lake dam, with the exception of having only one strut, was built at the same time and at the same unit prices, in order to increase the capacity of the natural lake. Across this lake all material for use in the Gem Lake dam construction was brought on a barge for a distance of about 2,000 feet to the foot of another tramway terminating at Gem Lake dam site 550 feet higher in elevation. The cost of these tramways were not included in the $22.00 per cubic yard of concrete, but were paid for extra. They were also necessary for the construction of the pressure pipes, one from Gem Lake, and one from Agnew Lake to the power house, and would have had to be built independent of the dams.

For the long haul across the desert from the Southern Pacific railroad station at Benton, California, to the power house at Silver Lake, six 75 HP C. L. Best Tracklayers, burning distillate,

were used, each hauling three trailers. The net load was as close to 20 tons as practical, and the time necessary to make one round trip was about six days of twelve hours, including loading, unloading, and ordinary delays. The speed of these tracklayers was 2½ miles an hour on high gear, and 1½ miles per hour on low gear. The contractors figured the cost to them for hauling this distance was at the rate of $13.50 per ton. The hauling of the cement and materials for the dams were not paid for extra, but were included in the $22.00 per cubic yard of concrete in place.

A rock fill dam on the same site would have to be built for $2.15 per cubic yard (construction cost), including the water-tight face and hand laid rock, in order to be on equal terms as to cost, with the multiple arch dam built. In the writer's opinion this would not have been possible in this place. In any locality where cement can be laid down cheaper than in the above case, the relative cost of a multiple arch dam and a rock fill dam will be still more in favor of the multiple arch dam.

Besides furnishing the designs, the writer also supervised the construction of the dams.

Mr. C. O. Poole was Chief Engineer for the whole development, Mr. E. J. Waugh, A.M.A.S.C.E., was Resident Engineer, Mr. L. B. Curtis, M.A.S.C.E., Field Engineer, and Mr. F. O. Dolson, Superintendent of Construction.

Messrs. Duncanson Harrelson Co. of San Francisco, were the contractors on the dams.

Ed Fletcher Papers

1870-1955

MSS.81

Box: 47 Folder: 10

Business Records - Water Companies - Volcan Land and Water Company - San Dieguito System - San Dieguito Mutual Water Company - Carroll Dam (Lake Hodges) - Typed copy of article re. Multiple Arch Dam at Mono, California