T H E S I S.

DESIGN OF A CONSTANT ANGLE ARCH DAM AT DEVIL'S GATE IN CONNECTION WITH THE FLOOD CONTROL OF LOS ANGELES COUNTY.

GENE HEYWOOD.

Class of Nineteen Hundred Eighteen

Department of Civil Engineering.

THROOP COLLEGE OF TECHNOLOGY.

Pasadena, California.

1 9 1 8.

TABLE OF CONTENTS.

INTRODUCTION

The need of flood control and flood prevention have long been felt in Southern California and several investigations have been instigated. The most noteworthy and commendable report on this work is that of the Board of Engineers of Flood Control submitted to the Board of Supervisors of Los Angeles County in 1915.

As a resident of Pasadena, and as an engineering student, the writer has been particularly interested in the control of flood waters in the Arroyo Seco and especially in the plans submitted by the members of the aforementioned Board of Engineers and by Mr. J. W. Reagan, Engineer of the Los Angeles Flood Control District.

The principal features of Mr.Reagan's plan are as follows:

A dam, 130 ft. high, gravity type, arched up stream to a radius of 400 ft., is to be built of cyclopean masonry. The dam will serve also to carry the La Canada-Verdugo Boulevard over its top, a 20 ft. roadway and sidewalks on either side being provided.

The storage capacity of the reservoir formed by this dam will be about 6600 acre feet. The spillway proper will consist of a channel excavated in the east embankment, the waters to be confined between two side walls of reinforced concrete. The spillway tunnel, curved in plan, will be about 300 feet in length and have a capacity of 7000 sec. ft. The regulating works call for hydraulicly operated gates, large enough to take care of a discharge of that magnitude. The combined spillway capacity will be equal to about 15,000 sec.ft., considering the maximum flood level at elevation 1065.

A low overflow dam, of the arched type, will be constructed at the narrow outlet to Devil's Gate, to provide a water cushion for the spillway discharge and to deaden the erosive effect, at the foundations of the dam, caused by the high velocity of the spillway waters.

The roadway will cross the spillway proper on a reinforced concrete bridge of the beam and girder type.

The east approach of the roadway and the section between the main dam and the spillway will be earth embankment and ~~a~~ the west approach will be in excavation.

A map of the reservoir is shown on plate 3, the profile, cross section and plan views of plate 4, and the emptying and filling curves on plate 5 of Mr.Reagan's report.

Two features of the foregoing plan particularly attracted the attention of the writer: the selection of the gravity type of dam for a site ideal for an arched dam, and the choice of a velocity of 50 ft. per second, which woulc be necessary to discharge 7000 sec. ft. through the tunnel section chosen.

These considerations, and the fact that the auxiliary structures had only been drawn in outline and had not been designed in detail, led the writer to investigate an alternative plan, and to undertake the design of the auxiliary structures as the subject of the thesis submitted by him to Throop College of Technology. The report, calculations and designs herein contained are the result of that investigation.

The general plan is very similar to that suggested by Mr.Reagan, in fact, was purposely made so as to form a better and more direct comparison between the estimates of cost of a gravity dam in one case and of an arched dam in the other. The main divergence between the two plans consists of: the substitution of a dam of the so-called constant angle arch type in place of one of the gravity type;

the construction of a gravity abuttment to take the thrust of that portion of the arch dam coming above the belt of sound granite walls on the east bank; the use of a spillway tunnel, straight in plan, with a maximum velocity of 30 ft. per sec.; the provision for a spillway capacity equal to that of the estimated maximum flood that has ever occurred, namely, one 70% greater than that of 1914; and the carrying of the roadway across the canyon proper on a reinforced concrete arch bridge down stream from the dam.

SELECTION OF THE TYPE OF DAM TO BE USED.

The drainage area of the Arroyo Seco above the dam site is 35 sq. miles, and the outlet to this drainage area is through a rocky gorge some 700 ft. in length, known as Devil's Gate. The site chosen for the dam is about 200 ft. below the present La Canada-Verdugo highway bridge. The section of the canyon, at the dam site chosen, approximates an irregular V. But the bottom has been considerably rounded, forming a U. The walls of the canyon are of dense white granite, with felspar and quartz predominating, although some biotite is found in it in places. The west wall rises high above the top of the proposed dam and forms a perfect abutment for an arch dam. The east wall of the canyon is composed of sound rock to about elevation 1053. Above this contour the granite shows signs of weathering. Since the crest of the dam adopted not only comes several feet above this belt of rock between the sound and the weathered granite, but actually is above the natural surface of the ground, it is apparent that some form of masonry ábutment must be used to take the thrust of that portion of the arch dam which comes above solid rock.

There are four types of masonry dams which could be selected for this site: gravity, single center arch, multiple arch, and constant angle arch.

The gravity type of dam will be discussed later in comparing it with the arch dam finally chosen, and needs no mention at this point.

A single center arch dam could be economically used where the walls of the canyon are nearly vertical, but as before stated, the walls at this site are by no means vertical or nearly so. An economical design of a dam of this type would be much too light to carry a 20 ft. roadway and sidewalks on either side, and would contain 50% more masonry than the dam selected.

The cost of the separate bridge would, of course, be identical in both projects. The total cost of the bridge and dam would be greater in this case than if the dam were designed with a section heavy enough to carry the roadway and sidewalks. If designed with this heavier section, the volume of masonry would be slightly more than double that of the dam finally chosen, the cost would still be greater than the combined cost of the dam and bridge selected, and no advantages can be claimed for the single center arch dam which are not likewise applicable to the constant angle arch, while on the other hand many disadvantages of the former are done away with in the latter type.

An estimate of cost on materials, excavation, form work, labor, plant, overhead and 10% for engineering, show these figures:

Single center arch dam with lightest possible section; abutment and separate bridge $190,000

Single center arch dam with section heavy enough to support roadway; abutment $175,000

Constant angle arch dam, abutment and separate bridge $152,000

The third type, the multiple arch, is adapted to sites where a wide valley must be closed by a relatively low masonry dam. The writer believes that a height of 137 feet is too great for the economical design of a multiple arch dam, and that, in a narrow canyon like Devil's Gate, some other type of arch is better adapted to meet the requirements.

The advantages and disadvantages of each type of arch dam have been fully weighed, and the writer believes that the so-called constant angle arch offers the greater number of advantages without many of the disadvantages of the other types. The advantages claimed for the type of dam chosen over other arch dams are:

1. Greater economy of material. The thickness required of any section is directly proportional to the radius, and by changing the center at intervals, the thickness is reduced proportionately to the decrease in radius.

2. Due to the larger central angle near the foundations the arch action is more complete. The deflection of an arch is proportional to the square of the upstream radius. By making the radius half as large at the foundations as at the crest, the deflection would be but one-fourth as much as before and the load which could be carried by the arch, using the same unit stresses, would be four times as great as in the constant radius type.

3. The average crown deflection due to a decrease or increase in length caused by temperature changes is about half as much as in the constant radius type, and therefore temperature cracks are much less likely to occur than in the latter type.

It is believed that the only disadvantages that can be claimed against this type are the greater difficulties of design, and the greater cost of the field work of setting form points. The design is more complicated, and is a process of cut and try until the final section is fitted to the site. However, the saving in material more than compensates for the extra labor in design. The form carpenter gets his points every 10 ft. and the difficulties of building forms are not any greater than in the ordinary arch dam, but the computations and the field work of setting those points are much more complicated. The total cost of engineering, however, would not be any greater because less time would be required to construct a dam of this type and a consequent saving would be made in the necessary expense of maintaining an engineering force during the period of construction.

The advantages claimed for the type of dam selected over a gravity section are:

1. Economy of material. Actually in the dam chosen there is but 30% as much material as in the gravity section proposed.

2. Greater factor of safety. A factor of safety twice as great as in the gravity section is attained.

3. More favorable distribution of stresses. In a gravity dam the stresses are a maximum at the toe and zero at the heel, while in an arch they are practically uniform throughout the section.

4. In an arch dam, uplift at the foundations does not affect the stability of the structure, while in a gravity dam it is of considerable importanty and must be carefully designed for.

5. Vertical temperature cracks are not serious in an arch dam, for the pressure of the water tends to close them when the reservoir is full, while in a gravity dam vertical cracks cause serious leakage and temperature changes must usually be taken care of by providing expansaion joints.

6. Horizontal shear planes in an arch dam, although objection-

able as regards leakage, only tend to produce more complete arch action, while in a gravity dam the percolation of water through the seams and cracks increases uplift and seriously endangers the stability of the dam.

7. An overflow of an arch dam would not seriously reduce its stability, for it depends on the soundness of the abutments rather than on the foundations for support, while in the gravity dam any erosion at the toe is disastrous, and any overflow must be carefully guarded against unless an adequate apron on the downstream side is built.

While it is true that an arch dam is not capable of exact analysis, because of the combined action of arch, cantilever, and curved beam near the foundations, the fact remains that there is not on record a failure of an arch dam, although many have been built with working stresses greatly exceeding those used by the writer in his design. It is also true that in the gravity dam, the section is of sufficient size to carry a roadway across its top, while with the arch dam selected, a separate highway bridge must be built and an abutment must be provided to take the thrust of that part of the dam coming above the natural surface of sound rock. However, as it has before been stated, and as will be clearly shown in the detailed estimates of cost which follow, the combined cost of the arch dam, abutment and bridge is but 60% of the cost of the gravity dam alone.

In making these estimates of cost, the writer has endeavored to be as fair as possible, and it is believed that any errors made or approximations used have been favorable to Mr. Reagan's plans rather than to his own. Unit prices have been determined according to present costs of material which are higher than when the Flood Control Board's plans were first made public, and it is believed that all estimates are well on the safe side, and that the project could actually be built at a smaller figure in normal times. The cost of each structure includes excavation, materials, labor, plant, overhead and all other charges exclusive of engineering. Engineering was estimated at 10% of the sum of the prices of the individual structures and put down in a lump sum. Detailed estimates and the determination of unit prices are given later in the accompanying computations, and only the final figures arrived at are given here for purposes of comparison.

Cost of constant angle arch dam	$74,900
Cost of gravity abutment	3,200
Cost of concrete arch bridge	60,000
Total	$138,100
Cost of overflow dam	2,100
Cost of spillway	20,100
Cost of regulating works, including tunnel, gate house, valves, etc.	15,000
Cost of girder bridge crossing spillway	8,000
Cost of approaches, road material, drainage and incidentals	10,000
Engineering at 10%	19,000
Total cost of project	$212,300

Cost of Gravity Dam

34,000 cu. yds. of masonry @ $6.10	207,400
12,000 cu. yds. of rock excavation @ 1.50 . . .	18,000
6,000 cu. yds. of gravel excavation @ 60¢ . .	3,600
Total	$229,000

The above quantity figures were arrived at by calculating volumes from the drawings submitted by the office of the Flood Control District of Los Angeles County. Figuring the auxiliary structures at the same prices as above, this would give a total of $302,200 for Mr.Reagan's plan. This is slightly higher than his estimate of $290,000 submitted over a year ago, but this increase can be accounted for, partly by the increase in cost of materials, and partly by the fact that a larger spillway capacity was provided for than in his plan. This naturally calls for a larger amount of excavation, more concrete, and a longer bridge over the spillway. It might be said in passing that the seemingly high figure for the spillway is caused mainly by the large amount of excavation, and the figure quoted includes the placing of earth and rock in the east approach, for the material excavated can be placed in this embankment nearly as cheaply as it could be wasted.

Also, it will be noted that the item of engineering was figured on a basis of 10% of the cost of the plan as suggested by the writer, and that this same figure was used in the estimate of cost of Mr.Reagan's plan. It is believed that the figure for this latter plan would more nearly be 10% of the total price of that plan, but the two figures for engineering expense have been made identical so that no question could be raised as to the fairness of a comparison between the two estimates.

The writer has felt justified in using the same unit costs for masonry in both dams and while the cost of erection of forms may be slightly greater in his plan, the overhead charges in Mr.Reagan's plan would be much greater in items such as interest on investment, etc. because of the greater length of time required to build a gravity dam. The figures show a net saving of $91,000, which, figuring on a basis of the arch dam, gravity abutment and arch bridge, against the gravity dam, would amount to a saving of 40%; figured on a basis of the total cost of one project against the other, a saving of 30% would result.

The main features of the design of each of the structures as well as the typical calculation will now be given.

MAIN DAM

Mr. Lars R. Jorgensen, Assoc. Mem. A.S.C.E., in his excellent paper printed in the transactions of the American Society of Civil Engineers, vol. LXXVIII, in which he first advocates the use of a constant angle arch dam, has mathematically proved that the most economical theoretical central angle that can be adopted in arch dams is $133^\circ 34'$. Figure 2, in the same paper, also shows that for any angle between 150° and 115° there is not more than 1% variation in the amount of materials, and Mr. Jorgensen points out that an angle of 120° would be about the economical limit in practice. Actually, however, this angle will be limited by the peculiar configurations of the typography at each site, and a smaller angle than 120° may have to be used. This was the case at the site in question. A central angle of 120° would, in the first place, give thrusts too nearly tangential instead of normal to the rock walls, in which case only the shearing strength of the granite instead of its full compressive strength could be developed; and then, too, by using the larger angle, the dam would not follow the natural high ridges and the volume of masonry would thereby be increased. The central angle at the crest finally adopted was 114°.

It is easily apparent, and Mr. Jorgensen has ably discussed it in the aforementioned paper, that if this angle could be kept constant at the successive elevations between crest and base by shortening the radius, a great economy of material would result and greater arch action could be developed near the foundations than if the radius were kept constant.

If the slopes of the canyon walls at any point are such that the ratio of decrease in length of the upstream radius is greater than the ratio of increase in water pressure, a decrease in thickness of the dam at this elevation will result, and the upper portions will be overhanging. This, of course, is impractical, and either the thickness must be increased or the radius lengthened. In Mr. Jorgensen's paper, he advocates the use of a different center for the upstream and downstream radii, making the arch thicker at the crown than at the abutments. Actually, however, the writer, in his design, found that this overhang would always occur on the downstream face, and that a greater economy of material would result by making the downstream face vertical, (throwing all of the increase in thickness on the upstream side) and by gradually decreasing the central angle from crest to base, ~~and~~ keeping the thickness of each slice the same from crown to abutment. This was made necessary because of the very irregular surface of the rock, which in places was nearly horizontal, while in others nearly vertical.

As can readily be seen from the topographical map, the west canyon wall has a portion 35 ft. long which is nearly flat. To have kept the central angle constant would have resulted in a large amount of extra excavation and masonry at this point, and the adoption of different centers for downstream and upstream radii would make a thicker section at the crown than at the abutments. The writer does not believe that this is justifiable in this case, and a comparison shows that decreasing the central angle, using the same center for upstream and downstream radii, will give a more economical dam than would result in the other case.

It is a well known fact that there is a restrainment of the arch near the foundations, due to the relatively unyielding rock, and because of this, full arch action can not develop, and there will be a combination of arch, cantilever, and curved beam action. Near the crest the deflection of an arch of the dimensions used would be much less than the deflection of a curved beam of similar proportions or of a vertical cantilever. At this point the action would be almost entirely that of a perfectly loaded arch. Nearer the foundation, the deflection of a vertical cantilever would be less than that of the arch or curved beam. In this case, cantilever action will develop before full arch action can do so. Still further down, near the solid rock, it may be that the curved beam would have a smaller deflection than the cantilever, and in this case this type of action would develop and there would be but partial cantilever and partial arch action.

The weight of the masonry of the mass above any point will put into compression the masonry beneath it. The intensity will, of course, vary according to the distribution of the mass above. Arch action will put all of the masonry in compression and theoretically, if the section is small, the intensity will be uniform at any one section. Curved beam action will develop compression on the upstream side and tension on the downstream side. Cantilever action will develop tension on the upstream side and compression on the downstream side. Temperature changes will develop compressive or tensile stresses according to whether the change is an increase or a decrease in temperature. It can readily be seen that the actual stresses developed in an arch dam are very complex, and that the stresses developed by the different types of action are sometimes compensating and sometimes cumulative.

The late Mr. R. Shirreffs, Mem. Am. Soc. C.E., in his discussion on the paper entitled, "Lake Cheesman Dam and Reservoir," develops an empirical formula for the deflection of an arch dam, taking into account all of these factors. Mr. Jorgensen uses this formula in the investigation of the stresses developed in a constant angle arch dam and by an ingenious method, distributes the proportion of the water pressure taken by arch, cantilever and beam, according to the ratios of the deflection given by this formula to the deflection which would occur if each of these types of action acted alone and unrestrained. A glance, however, at the deflection curves of the Barren Jack Creek Dam in Australia will show that the many indeterminate factors, such as temperature changes and changes in moisture content, may practically nullify the assumptions on which the investigation is made. In fact, the deflection curves of dams built, as far as the writer has been able to ascertain, do not even closely approximate the curves which would result from an application of the aforementioned formula, and the writer is convinced of the utter futility of developing an empirical formula for deflections of an arch dam which would be applicable in all cases.

Mr. Jorgensen, in his investigations, has also taken into account the effect of Poisson's ratio on the stresses in an arch dam. The weight of masonry above any section will tend to decrease the vertical thickness of any horizontal layer, and this will cause an expansion in a horizontal direction. This lengthening will be partly resisted by the unyielding abutments and by the water pressure on the upstream side when the reservoir is full. This would set up an initial axial compression in the concrete and a certain proportion of the head would be held in equilibrium by this force before any shortening of the length of the arch would result and only the remainder would have to be taken care of by arch, cantilever and beam action. It is also true, and capable of proof, that this effect will be much greater in a dam of the constant angle type than in the ordinary upstream radius arch dam. However, Mr. Jorgensen seems to have overlooked the fact, in his main paper, that the upstream and downstream swelling of the masonry is only restrained on the upstream side and that the masonry is left free to expand in the downstream direction. The writer can not see how the proportion of head which will be neutralized by the initial axial compression is capable of such an exact solution as Mr. Jorgensen has used, especially when the solution is based on a seemingly erroneous assumption.

A theoretical dam designed purely as an arch, neglecting the effect of cantilever and curved beam action, if investigated by the many intricate and complex formulae evolved by some designers, will usually show a decrease in arch stresses near the foundations and an increase near the crest. This value near the crest would never exceed twice the assumed value, and indeed, the cantilever and beam action being small near the crest, it is not believed that the assumed values at the crest, if the arch is thin, will be greatly affected. Any practical dam, moreover, has an appreciable thickness at the crest, and the stresses would be considerably less than the allowable limit. Near the foundations the arch stress will be reduced and cantilever and beam action are of considerable importance. The net result would be to have the maximum compressive stresses at the toe and the minimum at the heel.

The writer, in making his design, has felt that the intricate and complex formulae which have been proposed by various designers and which are based on very approximate and questionable assumptions, do not justify the expenditure of time and labor made necessary by their use. Sections designed by these intricate theories do not vary materially from those designed on the simpler theory, treating each section as a rigid cylinder subjected to external water pressure, the thickness being determined independently of any restraint at the base. As has before been stated, there is not on record a failure of an arch dam, although the majority have been designed on this simpler theory and have used working stresses greatly exceeding those proposed by the writer.

The formula used is $q = \frac{62.5\ h\ r_u}{t}$, where q = average compression in pounds per sq. ft., h = the head of water in feet on the section in question, r_u = the upstream radius in feet. The writer has felt justified in using this formula as the basis of his design, but the sections so found have been altered according to his judgment to meet the peculiar conditions of the site.

The mean radius $R_m = \frac{W_m}{2 \sin \theta}$, where W_m = width of canyon and $\theta = \frac{1}{2}$ of the central angle.

The process of design was one of selection of a section according to the above formulae fitting it to the site and altering it to meet the particular requirements at that section. It was a cut and try process and the central angle theory could not be carried out to its full extent, but it is believed that the final design is economical of material and well on the safe side.

The dam as finally built called for a 137 ft. dam, containing 11,200 cu. yds. of 1:3:6 mass concrete. The mean crest length will be 342 ft. 1:3:6 concrete, after 6 months' curing, has an ultimate strength of 380,000 lbs. per sq. ft. A factor of safety of 9 has been provided, using a maximum allowable working stress of 42,200 lbs. per sq. ft. The crest of the dam has been placed at elevation 1067 and the stresses given in the table are calculated on a basis of the reservoir level at the crest of the dam.

It will be noticed that the arch stresses have been materially reduced near the base. This was done by putting a batter on the downstream face, which, up to this point, had been vertical. This extra thickness was added, and the arch stresses consequently reduced, in order to better distribute to the foundations the weight of the masonry above the base and to provide a larger section for cantilever and curved beam action near the bottom where the deflection is small and arch action incomplete.

The plan of the dam as shown represents the actual base after excavation and not the intersection of the natural surface of the ground with the dam faces. Actually, the masonry has been carried into solid rock for a distance of 12 ft. at all points. In the design, sections were taken at intervals of 10 ft., the first 7 ft. being struck from the same center. The warping of the faces was made very gradual, there being no sharp intersections, and a person not knowing that the centers were changed would not notice it. The design gives a light and graceful arch with a large factor of safety as to strength.

Sec.	Height Below Crest	Contour	Upstream Radius	Central Angle	Mean Width of Canyon	Thickness	Arch Stress Pounds per Sq. Ft.
1	0	1067	174.0	114	288	4.0	0
2	7	1060	174.3	114	288	4.33	17600
3	17	1050	172.5	109	277.5	5.0	36600
4	27	1040	167.5	88	227	7.0	40400
5	37	1030	162.5	86	215	9.0	41700
6	47	1020	156.5	84	201	11.0	41800
7	57	1010	148.5	83	188	13.0	40700
8	67	1000	141.5	82	176	15.0	39500
9	77	990	136.5	75	156	16.5	39800
10	87	980	131.5	77	136	17.5	40800
11	97	970	124.5	67	126	19.0	39800
12	107	960	122.0	58	109	20.5	39800
13	117	950	118.0	49	89	22.5	38300
14	127	940	114.5	42	73	25.0	36300
15	137	930	106.0	37	59	28.0	32400

Section	Mean Radius	Mean Width	Sin θ	½ Central Angle	Central Angle	Mean Length of Arch
1	172.0	288	.837	56.8	113.6	341
2	172.2	288.3	.837	56.8	113.6	342
3	170.0	277.5	.816	54.7	109.4	325
4	164.0	227	.692	43.8	87.6	251
5	158.0	215	.680	42.8	85.6	236
6	151.0	201	.666	41.8	83.6	220
7	142.0	188	.662	41.5	83.0	206
8	134.0	176	.657	41.1	82.2	192
9	128.25	156	.608	37.5	75.0	168
10	122.75	136	.555	33.7	67.4	144
11	114.75	126	.550	33.4	66.8	133.7
12	111.75	109	.488	29.2	58.4	113.7
13	106.75	89	.417	24.6	49.2	91.6
14	102.0	73	.358	21.0	42.0	74.8
15	92.0	59.0	.321	18.7	37.4	60

GRAVITY ABUTMENT

The gravity abutment has two functions to perform: to resist the thrust of the arch dam and to form a water-tight connection between the spillway and the main dam.

The crest of the gravity dam is at elevation 1069. A wing wall was put between the main dam and the gravity abutment, and the crest of the gravity dam was made two feet higher, so that, in case the main dam was ever overtopped, an overflow of two feet could be taken care of without washing out the foundations at the toe of the gravity abutment. The dam base was carried to sound rock and a cut-off wall in the solid rock provided to prevent excessive seepage. The details of design need no mention except that the intensity of the uplift was assumed to vary uniformly from two-thirds of the head at the heel to zero at the toe. Investigations were made at two sections, one where the downstream batter began and the other at the base. The specific gravity of the masonry was taken as 2.3.

Stability— Section 4 ft. from top.

Clockwise moment $= w\left\{(8)\ (1.333) + (5.53)\ (1.383)\right\} = 18.32w$

Counter clockwise moment $= w\left\{(36.8)\ (.617) + (1.36)\ (2.67)\right\} = 26.3w$

Section at the Base.

Clockwise moment $= \left\{\frac{(6)\ (18)^2}{2} + \frac{(18)\ (13.33)\ (4.44)}{3}\right\} w = 1328w$

Counter Clockwise moment $= 2.3w\left\{(4)\ (18)\ (6.22) + \frac{(8.67)\ (14)\ (1.333)}{2} + \frac{(.667)\ (18)\ (8.44)}{2}\right\} = 1332w$

The total area of the section equals 139.7 sq. ft.

The statical moment about the toe equals 1201.3

Location of resultant with reservoir empty equals $\frac{1201}{139.7} = 8.6$ ft. from toe.

Limit of middle third $= \frac{(13.33)\ (2)}{3} = 8.9$ ft. from toe.

These calculations show that the resultant falls inside the middle third of the base for reservoir both full and empty.

Sliding— Weight of concrete per foot of length = 321.w

Uplift of water per foot of length . . . . = 80w

W per foot of length . . . . = 241w

P per foot of length . . . . = 162w

$f > \tan\theta = \frac{P}{W} = \frac{162w}{241w} = .672$

A coefficient of friction equal to 0.75 provides for a factor of safety of two against sliding, if the shearing strength of the concrete is neglected. This is considered good practice and the value of tan θ should not exceed this value. The height of the dam is so small that vertical and inclined stresses in the concrete are negligable and need no investigation.

The thrust of the arch dam above elevation 1053 must be taken by the gravity abutment and the weight of that structure was designed heavy enough to resist that thrust due to its weight alone.

Area of section 1 of arch dam = 29.3 sq. ft.

Area of 7/10 of section 2 of arch dam = 32.0 sq. ft.

Average thrust on section1 = 8800 lbs. per sq. ft.

Average thrust on section 2 = 19800 lbs. per sq. ft.

Total thrust on gravity dam = (8800) (29.3) + (19800) (32) = 892,000 lbs.

Total weight of gravity dam = (241) (62.5) (82) = 1,235,000 lbs.

Tan θ equals $\frac{892{,}000}{1{,}235{,}000} = 0.722$

The weight is then sufficient to resist the thrust.

SPILLWAY CAPACITY

The maximum flood discharge of February-1914 was 14,140 second feet at the Colorado Street Bridge about 3 miles below Devil's Gate, and indications show that in 1884 and 1889 floods of about 70% greater magnitude occurred. This will give a discharge of 24,000 second feet. Although the channel below has a safe capacity of but 7000 second feet, it was thought advisable to provide adequate spillway capacity to take care of any flood which might occur. It was decided to have the regulating tunnel carry 4000 second feet and the open spillway 20000 second feet. It would have been preferable to have a tunnel capacity of 7000 second feet, but the enormous size of the tunnel capable of carrying that capacity at velocities within reasonable limits, practically excluded its use. The storage and tunnel capacity selected would be capable of reducing a flood as great as that of 1914 to a flow of 4000 second feet. The open spillway would not come into use except in case of a flood of greater magnitude.

The level of the reservoir at times of a prolonged flood, equal to 24,000 second feet flow, would be at elevation 1067. This would correspond to an elevation of 1064 at times of a flood equal to that of 1914. This is figured on a basis of a reservoir being full at the beginning of a flood.

OPEN SPILLWAY

The open spillway is partly in excavation and the flow of the water will be confined between two reinforced concrete walls. The floor of the spillway north of the bridge is not lined, but south of that point is lined with a 6-in. floor of concrete. A cut-off wall, sunk well into impervious rock, runs from the arch dam under the upstream face of the gravity abutment, across the upstream edge of the spillway floor, to the core-wall in the east earth embankment. Weep holes should be provided under the concrete floor to drain any seepage which may take place and to consequently reduce uplift on the floor.

The spillway was calculated as suppressed weir with a velocity of approach.

$$Q = 3.33\ b\left[(H + h)^{\frac{3}{2}} - h^{\frac{3}{2}}\right]$$

$h = \frac{u^2}{2g}$ where u equals velocity of approach

b = length of spillway crest equals 145 ft.

Considering the depth in the channel to be equal to 9.9 ft.,

$$u = \frac{20,000}{(140)\ (9.9)} = 14.43 \text{ ft. per second}$$

$$h = \frac{(14.43)^2}{64} = 3.2 \text{ ft.}$$

$$Q = (3.33)\ (145)\left[(9.9 + 3.2)^{\frac{3}{2}} - (3.2)^{\frac{3}{2}}\right] = 20100 \text{ sec. ft.}$$

The walls of the spillway were designed as a retaining wall subjected to water pressure both laterally and vertically. That is, the weight of the water acting on the floor, is sufficient to give stability to the wall subjected to water pressure. The top of the wall was carried to elevation 1067 and in the design the water was assumed at this level.

$$l = h\sqrt{\frac{1}{(1 + 2k - 3\ k^2}} \quad \text{in this case}$$

l = total length of base

h = head of water

k = some constant, here taken as 0.2

Then l = .884 h = 13.27 ft.

The width was made 13 ft. 4 in.

x = width of short cantilever = k l = 2.65 ft. and was made equal to 2 ft. 8 in.

Upright cantilever--

$$\text{Maximum moment} = \frac{(62.5)\ (15)^3\ (12)}{6} = 422,000 \text{ in. lbs.}$$

Steel used was 3/4 in. Tw. rods spaced 4 in. c. c. placed $1\frac{1}{2}$ in. from the face of the slab.

$d = 19\frac{3}{4} - 1\frac{1}{2} = 18\frac{1}{4}$

$$f_c = \frac{(422,000)\ (2)\ (64)}{(12)\ (21)\ (18.25)^2} = 642 \text{ lbs. per sq. in.}$$

$$f_s = \frac{(422,000)\ (8)}{(7)\ (18.25)\ (3)\ (.562)} = 15650 \text{ lbs. per sq. in.}$$

The moment $7\frac{1}{2}$ ft. from top equals $\frac{(62.5)\ (7.5)^3\ (12)}{6} = 52,700$ in. lbs.

Thickness equals 8 in., so $d = 6\frac{1}{2}$ in.

$$f_c = \frac{(52700)\ (2)\ (64)}{(12)\ (21)\ (6.5)^2} = 633 \text{ lbs. per sq. in.}$$

Every other rod was stopped at this point, so

$$f_s = \frac{(52700)\ (8)\ (2)}{(7)\ (6.5)\ (.562)\ (3)} = 11,000 \text{ lbs. per sq. in.}$$

It is common practice to put in a steel percentage of .004 for thin walls and .002 for thick walls in order to take care of temperature cracks which would develop. With $p = .003$, the area of steel required in this case would be .378 sq. in. per ft. 7/16 in. Tw. rods spaced 6 in. c. c. were used for the horizontal steel, giving an area of .383 sq. in. per foot.

Short cantilever—

Weight of concrete per foot of retaining wall equals 2970 lbs.

Weight of water acting on floor of retaining wall equals

$(62.5)\ (15)\ (10.67) = 10,000$ lbs.

Intensity of upward earth pressure at the end of the short cantilever

$$= \frac{(12,970)\ (2)}{13.33} = 1945 \text{ lbs. per ft.}$$

Intensity at the joint of the short cantilever

$$= \frac{(1945)\ (12.31)}{13.33} = 1795 \text{ lbs. per ft.}$$

Intensity at the joint of the long cantilever

$$= \frac{(1945)\ (10.67)}{13.33} = 1560 \text{ lbs. per ft.}$$

Intensity at the end of long cantilever $= 0$

Maximum moment in short cantilever $= \frac{(1870)\ (12.25)^2}{(2)\ (12)} = 11{,}700$ in. lbs.

$$f_c = \frac{(11700)\ (2)\ (64)}{(12)\ (21)\ (4.5)^2} = 294 \text{ lbs. per sq. in.}$$

$$A = \frac{(11700)\ (8)\ (\quad}{(7)\ (16000)\ (4.5)} = .186 \text{ sq. in.}$$

This was figured on a basis of a thickness of 6 in. which is sufficient. However, it was deemed advisable to batter this thickness from 6 in. at the end to a thickness equal to that of the long cantilever. The area of the steel required is so small, that the steel necessary to prevent temperature cracks would be larger and will govern in this case. This steel would be the same as that required in the 6" spillway floor. Assuming a value of $p = .004$

$$A = (.004)\ (6)\ (12) = .288 \text{ sq. in. per ft.}$$

7/16 in. Tw. rods spaced 8 in. c. c. both ways fulfills the requirements, giving an area equal to .287 sq. in. per ft.

Long cantilever—

$$\text{Maximum moment equals } \frac{(12)\ (10.67)^2\ (62.5)\ (15)}{2} - \frac{(12)(1560)(10.67)^2}{6}$$

$$= 285{,}000 \text{ in. lbs.}$$

Using a thickness of 16.5 in., $d = 15$ in.

$$f_c = \frac{(285000)\ (2)\ (64)}{(21)\ (12)\ (15)^2} = 643 \text{ lbs. per sq. in.}$$

$$A = \frac{(285000)\ (8)\ (\quad}{(7)\ (16000)\ (15)} = 1.36 \text{ sq. in.}$$

Using 7/8 in. Tw. rods spaced 6 in. c. c. gives an area equal to 1.53 sq.in. The thickness used is sufficiently large so that the shearing stress in the concrete is well under 40 lbs. per sq. in.

Wing walls and core walls—

The core wall will be subjected to hydrostatic pressure on one side and earth pressure on the other. The net result would be to have partial hydrostatic pressure on the upstream side. The section has been made large enough and sufficient steel has been provided to take care of this unbalanced pressure by using 3/4 in. Tw. rods spaced 6 in. c.c. giving a steel percentage of .004 figured on the maximum section. The bending moment of the cantilever at the half-way point will be less than one-half of the maximum and every other rod has been stopped at this point. A percentage of steel equal to .002 has been used for the horizontal steel, one-half inch Tw. rods spaced 6 in. c.c. being provided.

The wing wall will be subjected to partial earth pressure on the downstream face with the reservoir empty and for this reason the reinforcement has been placed $1\frac{1}{2}$ in. from the battered face instead of the vertical face as in the previous case. When the reservoir is full there will be full hydrostatic pressure on both faces, no reinforcement being needed. The same amount of reinforcement and section have been used in both the wing walls and the core wall.

TUNNEL SPILLWAY

The tunnel section and slope have been selected so that a discharge of 4000 sec. ft. can be taken care of at a velocity not exceeding 30 ft. per second. This velocity is slightly high, but the tunnel is built straight in plan and it is not thought that excessive erosion will take place. The tunnel will be driven through solid rock and will probably not need reinforcement except at the intake and outlet. However, it is deemed advisable to adequately reinforce it at the point where it passes under the gravity abutment. No attempt has been made to design the reinforcement, for the amount needed would depend on the character of the rock found on excavation. The tunnel should be lined, however, and a 6 in. cement lining is thought to be sufficient unless it is found that reinforcement is needed; the thickness would then depend, of course, on the amount of pressure that would have to be resisted.

A gate-house will be provided just north of the gravity abutment, and hydraulically operated valves will be used. No attempt has been made to select the gate equipment, but in the estimate of cost, quotations on gates of the same capacity and apparatus used under approximately similar conditions have been used, and it is thought that the estimate is liberal.

The tunnel has been designed with the maximum discharge taking place when flowing free, 9/10 full. When the level of the reservoir is above elevation 1036, the flow will take place under pressure, and larger discharges could be obtained. However, it is recommended that the gates be so regulated as to keep down the discharge to 4000 sec. feet.

The section of the tunnel at entrance has an area of 190 sq. ft. and a hydraulic radius of 3.7 ft. This area has been made sufficiently large so as to cut down the entrance velocity and consequently reduce eddies, etc. The section changes gradually in 45 ft. to an area of 140 sq. ft. and a hydraulic radius of 3.1 ft. The tunnel is 220 ft. long with a drop of 3.4 ft. in that distance, giving a slope of .0154.

The well known formula $v = c\sqrt{rs}$ was used in the design; n was taken as .013 and the corresponding value of c calculated to be 138. This gives a value of $v = 138\sqrt{(3.1)(.0154)} = 30$ ft. per sec. The actual discharge then equals (30) (140) = 4200 sec. ft.

OVERFLOW DAM

The overflow dam has been provided to form a pond of comparatively still water between it and the main dam. The effect will be to deaden the erosive action caused by the high velocities of the water from the spillway and tunnel discharged which flow over the bare rock walls. The main reason for using this dam was to protect the piers of the proposed bridge which crosses the Arroyo below the main dam.

The site chosen for the dam is at the lower end of Devil's Gate just before the stream leaves the narrow canyon and widens out into the wider flats of the lower Arroyo. The granite walls are nearly perpendicular at this point, for a height of 30 ft. and the site is ideal for an arch dam of the constant upstream radius type. The dam was made 36 ft. high, the masonry being carried 5 ft. below the point where solid rock foundations supposedly lie. If solid rock should lie deeper than estimated, the batter of the downstream face should be continued, giving an increase in thickness at the bottom proportional to the increase in height. The width of the canyon at the crest is 51 feet. The upstream radius adopted is equal to 30.5 ft. giving a central angle of 122.6. The mean length of the crest is 62 ft. At the base the central angle is 102 degrees and the mean length 49.8 ft.

If a flood of 24,000 second feet should ever occur again during the life of the proposed works, and all of this should have to pass the upper dam at one time, this dam would be subjected to an overflow of 24 ft. The ordinary maximum of 7000 second ft. would give an overflow of 10 ft.

Air vents should be provided so as to allow full atmospheric pressure on the under side of the overflowing sheet of water. These have not been shown in the drawings. As additional safety, however, the dam was designed for the above mentioned overflow of 24 ft. plus full atmospheric pressure. Air vents must be provided, however, to prevent vibration which might be set up by the alternate making and breaking of the vacuum formed on the downstream side of the dam.

It was, of course, impossible to design the downstream face to fit the curve of the falling sheet of water, because of the slender section used, but the upstream crest was designed to fit the curve of the sheet of water with an overflow of 7000 second feet. The upstream face was made vertical for a height of 30 ft. and then assumed a backward batter equal to that of the downstream face, giving the same thickness at the crest as 6 ft. below that point. The downstream face has a straight batter.

Atmospheric pressure = (14.7) (144) = 2,120 lbs. per sq. ft. The arch stress then equals $p = \frac{(62.5 h + 2120) R_u}{t}$

At the crest this stress is 40700 lbs. per sq. ft. and at the base is 35,800 lbs. per sq. ft.

GIRDER BRIDGE CROSSING SPILLWAY

The bridge is of the ordinary continuous girder type and is 144 ft. long consisting of nine spans of 16 ft. each. It will carry a 20 ft. roadway and 4½ ft. sidewalks on either side. A loading equivalent to that of a 20-ton truck was used in the design.

Although the drawings do not show it, the bridge should be built on a slight gradient, and the top of the floor slab placed at elevation 1069 at the west end and 1070 at the east end. This is to provide for drainage, and vents, covered with grillage, should be placed at frequent intervals near the curb throughout the length of the bridge.

A 20-ton truck has a distance of 5 ft. between wheels and 10 ft. between axles; the loading on each rear wheel is 14,000 lbs. and on each front one 16,000 lbs. To assume this heavier concentrated load acting on a 1-ft. strip of slab would give a slab thickness greatly in excess of that used on any modern highway bridge. The writer considers that the slab on each side aids in the support of this concentrated load, and for his purposes of design has considered the 40,000 lb. load as uniformly distributed over the area enclosed by the truck wheels. This gives a loading of 800 lbs. per sq. ft. This will give a slab section corresponding to that adopted by designers of first class modern highway bridges and is believed to be safe for the slab design. When used for the beams, it gives a greater bending moment than the concentrated load of 14,000 lbs.

Slab design—

a 6-ft. span was chosen between beams.

Dead weight of a 6-in. slab = 75 lbs. per sq. ft.

Total load on slab = 875 lbs. per sq. ft.

$$\text{Moment} = \frac{(875)\ (6)^2\ (12)}{12} = 31{,}500 \text{ in. lbs.}$$

$$f_c = \frac{(31{,}500)\ (2)\ (8)\ (8)}{(12)\ (3)\ (7)\ (5)^2} = 640 \text{ lbs. per sq. in.,}$$

assuming a value of 7/8 for j and 3/8 for k, which corresponds with common practice.

$$M_s = A\ f_s\ j\ d$$

½" Tw. Rods spaced 6 in. c.c. were used for the transverse steel.

$$f_s = \frac{(31{,}500)\ (8)}{(.5)\ (7)\ (5)} = 14{,}400 \text{ lbs. per sq. in.}$$

¼" Tw. rods spaced 12" c.c. were used for longitudinal steel. Over supports steel 4' long, of same size as above, was used to take care of the negative moment.

Beams—

The beams were designed as T beams with a width of flange not greater than 4 t, namely, 32 in. in this case.

slab and live load per foot of beam = (875)(6) = 5,250 lbs.

dead weight of the beam per foot = $\frac{(18)(8)(150)}{140}$ = 150 lbs.

Total load per ft. 5,400 lbs.

Moment = $\frac{(5,400)(12)(16)^2}{12}$ = 1,380,000 in. lbs.

An investigation of values of j and k show these quantities not to vary materially from the ordinarily assumed values of 7/8 and 3/8 respectively.

$$f_c = \frac{M_c}{\left(1-\frac{t}{2kd}\right)btjd} = \frac{(1,380,000)(8)}{\left\{1-\frac{(6)(8)}{(2)(3)(21)}\right\}(32)(6)(7)(21)} = 632 \text{ lbs. per sq. in.}$$

Using 4— 1-1/8 in. tw. rods

$$f_s = \frac{(1,380,000)(8)}{(5.06)(7)(21)} = 14,850 \text{ lbs. per sq. in.}$$

Reaction equals (5,400)(8) = 43,200 lbs. = maximum shear.

Intensity $v = \frac{V}{bjd} = \frac{(43,200)(8)}{(8)(7)(21)}$ = 294 lbs. per square in.

Concrete takes 40 lbs. per sq. in., steel takes $\frac{254}{294}$ of shear = 37,300 lbs.

$P = \frac{vs}{d} = \frac{(37,300)(4)}{21}$ = 7,100 lbs. considering stirrups spaced 4 in. apart.

$A = \frac{7,100}{(4)(16,000)}$ = .111 sq. in.

3/8 in. round rods were used for stirrups and were bent as shown in the drawing. The other details of design are clearly shown in the drawings and need no explanation. Two of the 1-1/8 in. rods were bent up 4 ft. from the center to take care of negative moment over the columns.

Column Walls. ----

The walls were made 14 in. thick. The 12 in. by 24 in. portions coming directly under the beams, and enclosed by the reinforcing steel, were designed as columns with sufficient strength to carry the applied loads. The walls were made homegenious throughout their length and only the reinforcement was changed at the point before mentioned.

Total load on a column = (43,200)(2) = 86,400 lbs.

Axial compression = $\frac{86,400}{(12)(24)}$ = 300 lbs. per sq. in.

This is much less than the allowable 450 lbs. per sq. in. and the walls would stand without reinforcement if it were not for ~~an~~ eccentric loading. Reinforcement as designed calls for 1/2 in. rods, spaced 6 in. c.c. under the beams and 12 in. apart in between. Vertical rods should be tied together by a band of 1/4 in. round steel, spaced every 12 in. Horizontal steel was made 1/2 in. tw. rods spaced 12 in. c.c. In the bottom of the spillway floor under the columns, rods 4 ft. long, with the spacing and size used in the floor slab, were put in. As the floor is excavated in rock, no other footings are required to support the column loads.

CONCRETE ARCH BRIDGE.

The design of the concrete arch bridge was undertaken by Mr. Kenneth Harrison of Throop College as the subject of his thesis. The drawings are here included, and for the details of design, the reader is referred to Mr. Harrison's report. It will suffice here to say that the central arch has a span of 155 ft. with a rise of 40 ft. The east approach is about 90 feet in length and the west approach about 100 ft. The total cost, ready for traffic, is conservatively estimated at $60,000.00. Notice is called to the very favorable topography for a structure of this type and the economical as well as light and graceful bridge attained by Mr. Harrison in his design.

UNIT COSTS.

In determining unit costs, the following items have been taken into consideration: Concrete, falsework and forms, steel and finishing. Each of the above may be further subdivided into materials, labor, plant, overhead and engineering.

No attempt has been made to design a construction plant layout and to figure the exact cost of each item that goes into the making up of the plant. The damsite is but a few hundred yards from the tracks of the S.P.L.A. & S.L.R.R. and good paved roads connect the railroad with the damsite. There are then no transportation difficulties and materials not available at the site could be cheaply delivered. This applies especially to the machinery, etc., of the construction plant. It is planned to place all concrete by derrick or from a central tower, preferably the latter. Four years ago, masonry could be placed in large masses such as in a dam, by mechanical methods and under less favorable conditions for 80 cents per cu. yd. An increase of 25% has been provided for, in the writer's estimate, to take care of the increase in cost of power, freight, materials, etc., and it is estimated that the net cost of the plant for this job would be about $13,000.00 or $1.00 a cubic yd. of masonry.

Other unit prices have been figured on a basis of the cost of Materials in Pasadena at the present time.

Cement ---

Cement	$ 2.10	net including credit for sacks, freight, etc.
Cost of testing.............	.03	
Cost of unloading...........	.03	
Cost of teaming.............	.05	
Cost of handling, bundling, sacks, etc.,.incidentals...	.04	
	$ 2.25	a barrel.

The ordinary cost of sand is $1.00 per cu. yd. washed, screened, delivered on the job. Suitable sand, however, is available at the damsite and it is estimated that it could be dug for $.30 per cu. yd. Screening and washing, delivering into the hoppers ready for use at the mixer, would bring this up to about $.50 a cu. yd. for sand.

Crushed stone can be bought at $1.00 a cu. yd. in cargo at the present time in this vicinity. It is thought since there is suitable rock at the damsite, that the rock couldbe crushed, screened, graded, handled and stored ready for use for $.80 a cu. yd.

Concrete - With a mechanical plant it is thought the following labor prices will apply:

Floor slabs and beams............	$1.00 a yd.
Columns, walls of spillway.......	1.50 " "
Arch overflow dam................	.70 " "
Main dam and gravity abutment....	.60 " "

Main dam and gravity abutment ~~and~~ overflow dam ---

Cement 1.2 bbls. @ $2.25.........	$2.70
Sand 1/2 yd. @ $.50..............	.25
Stone 1 yd. @ $.80...............	.80
Labor............................	.60
Plant............................	1.00
	$5.35 a cu. yd.

Spillway walls, core wall, wing walls, column walls of bridge ---

Cement 1-2/3 bbls. @ $2.25.......	$3.75
Sand 1/2 yd. @ $.50..............	.25
Stone 1 yd. at $.80..............	.80
Labor............................	1.50
Plant............................	1.00
	$7.30 a cu. yd.

Floor of spillway & bridge, and beams of bridge ---

Cement 1-2/3 bbls. @ $2.25.......	$3.75
Sand 1/2 yd. @ $.50..............	.25
Stone 1 yd. at $.80..............	.80
Labor............................	1.00
Plant............................	1.00
	$6.80 a cu. yd.

Steel ---

Costs of steel are very unfavorable at the present time, but are here considered at present stock mill rates plus freight. If it were possible to obtain steel in a large mill order the base price could probably be cut in two.

Labor costs on steel work ---

Cost of unloading and piling steel on job ----------------------------	$.02 per 100 lbs.
Cost of teaming....................................	.01
Cost of cutting and placing steel in floors...	.25
Cost of cutting, bending, placing steel and stirrups in beams............	.50
Cost of cutting and placing steel in walls including tieing, etc..............	.65

Steel costs ---
3/4 in. or larger.

Mill price	$2.90
Freight and transportation tax	1.02
	$3.92 per 100 lbs.

5/8 in.

Mill price	$2.95
Freight, etc.	1.02
	$3.97 per 100 lbs.

1/2 in.

Mill price	$3.00
Freight, etc.	1.02
	$4.02 per 100 lbs.

3/8 in.

Mill price	$3.15
Freight, etc.	1.02
	$4.17 per 100 lbs.

1/4 in.

Mill price	$3.60
Freight, etc.	1.02
	$4.62 per 100 lbs.

Forms ---

Lumber prices at the present time are high and variable, ranging from $36.00 to $42.00. These are list prices and can be reduced by $5.00 to $7.00 when bought in large quantities. The cost of form lumber, bracing, studs, etc., would perhaps come to about $60.00 per 100 sq. ft. of form surface. It is believed that all lumber could be used twice, bringing the lumber cost down to $30.00 per 100 sq. ft. The cost of nails, wire, oil, etc., would come to perhaps $10.00, making a total of $40.00 per 100 sq. ft., or 4 cents per sq. ft. of form surface.

Main Dam, Gravity Abutment and Overflow Dam forms ---

Lumber, nails, wire, etc.	$ .04
Labor making, erecting, plumbing, wiring, etc.	.10
Labor stripping	.01
	$ .15 a sq. ft.

Spillway walls, core walls, wing walls, column walls of bridge ---

Lumber, etc.................	$ .04
Labor making and erecting...	.08
Labor stripping............	.01
	$ .13 per sq. ft.

Floor of bridge ---

Lumber, etc.................	$ .04
Labor making................	.03
Labor erecting..............	.04
Labor stripping.............	.01
	$ .12 per sq. ft.

Beams of bridge, ~~1~~--

Lumber, etc.................	$ .04
Labor making...............	.04
Labor erecting.............	.04
Labor stripping............	.01
	$ .13 per sq. ft.

Finishing ---

For surfaces which are to be finished a cost of 4 cents per sq. ft. can be added for materials, labor, etc.

Form work in main dam ---

Number of Sq. Ft. of Form Work per Cu. Yd. of Masonry:

Main dam --- $\frac{52,500}{11,220}$ = 4.68 or 70 cents a cu. yd.

Gravity dam --- $\frac{3,160}{460}$ = 6.9 or $1.03 a cu. yd.

Overflow dam --- $\frac{3,000}{270}$ = 11 or $1.65 a cu. yd.

$$\frac{(.70)(11,220)}{11950} = .658$$

$$\frac{(1.03)(460)(}{11,950} = .040$$

$$\frac{(1.65)(270)}{11,950} = .037$$

$.735 per cu. yd. of masonry.

A cost of 75 cents a cu. yd. of masonry was given for form work in the main dam, gravity abutment and overflow dam. This gives a total unit cost of $6.10 for masonry for these three structures.

It is not thought that there is any need of showing the calculations by which the masonry volumes, excavation, area of form work, amounts of steel, etc., were determined, as their solution presents nothing new and their method of calculation is self evident. It might be pointed out, however, that the method employed in obtaining the masonry volume in the main dam was the following: The area of each 10 ft. section was calculated and multiplied by the average of the mean lengths of the dam at the top and bottom of that section. The sums, of course, gave the total volume.

The final estimates of cost follow and include everything except engineering, this being added at the end in a lump sum.

ESTIMATES OF COST.

Main Dam ---

11,220 cu. yds. of masonry	@ $6.10.....	$68,500.00
3,100 cu. yds. of gravel	@ .60.....	1,860.00
3,000 cu. yds. of rock	@ 1.50.....	4,500.00
		$74,860.00

Gravity Dam ---

460 cu. yds. of masonry	@ $6.10.....	$ 2,810.00
290 cu. yds. of rock	@ 1.50.....	435.00
		$ 3,245.00

Overflow Dam ---

270 cu. yds. of masonry	@ $6.10.....	$ 1,710.00
110 cu. yds. of rock	@ 1.50.....	165.00
220 cu. yds. of gravel	@ .60.....	130.00
2,040 lbs. of 1 in. steel	@ .0392...	80.00
		$ 2,085.00

Open Spillway ---

Concrete ---		
517 cu. yds. of concrete	@ $7.30.....	$ 3,780.00
304 cu. yds. of concrete	@ 6.80.....	2,070.00
Forms ---		
13,390 sq. ft.	@ $ .13.....	$ 1,740.00
Steel ---		
7/8 in. rods. - 16,750 lbs.	@ $ .0392...	660.00
Labor	@ .0028...	48.00

3/4 in. rods -	25,550 lbs.	@	$ .0392...	$ 1,000.00
	Labor	@	.0068...	174.00
7/16 in. rods-	40,400 lbs.	@	.0417...	1,680.00
	Labor	@	.0028...	112.00
3/8 in. rods -	2,310 lbs.	@	.0417...	100.00
	Labor	@	.0028...	6.00
Finishing ---				
30,000 sq. ft.		@	.03.....	900.00
Excavation ---				
7,850 cu. yds.		@	1.00.....	7,850.00
				$20,120.00

Girder Bridge ---

Concrete ---				
155 yds. at $7.30......................				$ 1,133.00
140 yds. @ 6.80......................				952.00
Railing..............................				350.00
Forms ---				
7,440 sq. ft. @ $.13..................				966.00
3,200 sq. ft. @ .13..................				416.00
4,350 sq. ft. @ .12..................				522.00
Steel ---				
1-1/8 in. rods -	20,900 lbs.	@	$.0392..	812.00
	Labor	@	.0053..	111.00
1/2 in. rods -	19,050 lbs.	@	.0402..	766.00
	Labor	@	.0068..	130.00
	12,750 lbs.	@	.0402..	512.00
	Labor	@	.0028..	36.00
	1,480 lbs.	@	.0402..	60.00
	Labor	@	.0028..	4.00
3/8 in. rods -	7,120 lbs.	@	.0417..	296.00
	Labor	@	.0053..	38.00
1/4 in. rods -	750 lbs.	@	.0462..	35.00
	Labor	@	.0068..	5.00
	1,310 lbs.	@	.0462..	60.00
	Labor	@	.0028..	4.00
Finishing ---				
9,000 sq. ft. @ $.03..................				270.00
Road material, surfacing, sidewalk fill, drains and incidentals............				500.00
				$7,985.00

Tunnel Spillway ---

Excavation @ $5.00 per cu. yd..........	$ 26.80 per ft.
Lining and reinforcement, forms, etc...	13.20
	$ 40.00 per ft.
Total cost of tunnel..................	$8,800.00
Cost of gate, house, valves, regulating apparatus, etc.	6,200.00
Total cost of regulating works....	$15,000.00

The total costs have been given before in comparison with the figures of Mr. Reagan's plan and will not here be repeated. The drawings enclosed are complete and need no explanation, and details of the design not included in this report are clearly shown on these drawings.

S U M M A R Y.

In conclusion the writer wishes to state that he believes all the ~~suc~~assumptions and approximations made have been on the safe side and that an economical design of the various structures has been obtained. The estimates of cost are liberal and while they may not represent the exact expenditures that would be made if the plan as advocated were to be actually constructed, still the figures form a fair and equal basis on which the two plans as outlined can be compared.

The writer claims the following points of superiority in his design over that submitted by the office of the Engineer of Los Angeles County Flood Control District:

1. A saving of 30% on the cost of the total project and 40% on the dam alone.

2. A factor of safety twice as large in the main dam as well as the other advantages previously described of an arch dam over a gravity dam.

3. Elimination of the erosive action in the spillway tunnel by the reduction of grade and velocities to a reasonable value.

4. A spillway capacity large enough to take care of the maximum flood recorded, even if the regulating works of the tunnel spillway fail to work.

5. The provision of a separate highway bridge, making the means of communication between the two sides of the Arroyo

independent of the dam structure.

6. Better aesthetic treatment. The light, graceful arch bridge and dam are more pleasing to the eye and capable of better treatment than the rather heavy and cumbersome looking gravity dam.

The writer realizes the following limitations to his plan: That the capacity of the spillway tunnel is smaller than that planned on by Mr. Reagan, and that the full capacity of the open spillway may so seldom be used as not to warrant the expenditure necessary to provide for such a large discharge. However the large discharge calculated for the tunnel spillway of Mr. Reagan's plan was only obtained by the use of excessive velocities, the use of which cannot be justified. If it is thought, on further study, that the likelihood of the full capacity of the open spillway's ever being used were too small to warrant the extra expense necessary to provide for it, and that the tunnel spillway must be able to take care of a flow of 7,000 sec. ft., the writer would suggest the construction of a pair of tunnels, each with a section of 120 sq. ft., and the reduction of the open spillway section, by narrowing the width, so that its capacity would be equal to a flow of 14,000 sec. ft. The expenditure would not be greatly in excess of the present estimate and a better regulation of the reservoir storage would be obtained.

The writer does not in any way wish to depreciate the value of Mr. Reagan's plan and does not say that the design of each structure submitted by himself is the most economical or the best that is capable of design, but he firmly believes that the study made by him and here submitted conclusively shows that a material saving and a better dam can be obtained by the use of a constant angle arch dam.

B I B L I O G R A P H Y.

Constant Angle Arch Dam Design,
By Mr. Lars Jorgensen,
Engineering News, Volume 68, Page 155.

Influence of Poisson's ratio on stresses in Arch Dams,
By Mr. Lars Jorgensen,
Engineering News, Volume 68, Page 208.

Neglected First Principles of Masonry Dam Design,
By Mr. George H. Moore,
Engineering News, Volume 70, Page 442, 623, 832.

Uplift Pressure,
By Mr. Edward Godfrey,
Engineering News, Volume 70, Page 371.

Stresses in Masonry Dams with Discussions,
By Mr. William Cain,
Transactions Am. Soc. C.E., Volume LXIV, Page 208.

The Action of Water under Dams and Discussions,
By Mr. J. B. T. Colman,
Transactions Am. Soc. C.E., Volume LXXX, Page 421.

Provisions for Uplift and Ice Pressure in the Design of Masonry Dams,
By Mr. C. L. Harrison,
Transactions Am. Soc. C.E., Volume LXXV, Page 142.

The Ultimate Dam,
By Mr. J. S. Eastwood,
Western Engineering, September, 1913.

Huacal Dam, Sonora, Mexico,
By Mr. H. Hawgood,
Transactions Am. Soc. C.E., Volume LXXVIII, Page 564.

Constant Angle Arch Dam,
By Mr. Lars Jorgensen,
Transactions Am. Soc. C.E., Volume LXXVIII, Page 685.

Concrete Arch Dam near Cheyenne, Wyoming,
By Mr. M. V. Moulton,
Engineering Record, February 8th, 1913, Volume 67.

Construction of a Pathfinder Dam,
By Mr. E. H. Baldwin,
Engineering Record, November 7th, 1908, Volume 58.

A remarkable Concrete Arch Dam at Crowley Creek, Oregon,
Engineering News, August 24th, 1911, Volume 66.

Salmon Creek Dam - First Constant Angle Arch,
Engineering News, March 11th, 1915, Volume 73.