THE DESIGN OF A GRAVITY DAM
WITH SIPHON SPILLWAY
FOR THE
EL CAPITAN DAMSITE

by Carl F. Beed

Thesis

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BIBLIOGRAPHY

Proceedings of the American Society of Civil Engineers, "Siphon Spillways" , volume XLVII-No. 2

Paper by G.F.Stickney, M.Am.Soc.C.E.

Discussions in volume XLVIII-No. 4

By Charles F.Ruff

Edward Wegmann

W.P.Creager

H.F.Dunham

Karl R.Kennison

Joel D.Justin

H.B.Muckleston

Charles G.Darrach

" The Design and Test of a Model Siphon Spillway"

By Robert N. Allen . Thesis 1916

THE EL CAPITAN DAM PROJECT

The El Capitan Dam Site is located about thirty miles from San Diego, near Lakeside. In carrying out San Diego's water development program $4,500,000 was voted in November 1924 to construct the first development of the El Capitan Dam on site number 2. The first development was to be a dam 150 feet high, the completed dam is to be 200 feet high with a storage capacity of forty-three billion gallons or about 132,000 acre feet.

There has been considerable discussion as to the relative merits of the El Capitan damsite as compared with the Mission Valley damsite which is located about seventeen miles down the San Diego River. These sites are shown on the location map of plate III . The main disadvantage of the Mission Gorge sites is the large amount of valuable farm land which would be flooded. Also the large surface area would cause considerable loss due to evaporation, since the loss is about 60 inches per year. The El Capitan Dam, on the other hand, would be a deeper and more narrow reservoir, and would not flood so much farm land. Practically the only farm land which will have to be bought belongs to a small Indian reservation, about 15 miles above the damsite. In general the land which must be bought is not very valuable.

There are two El Capitan damsites, number 1 and number 2. Core drilling has revealed the fact that site number 2 is more desireable. This can be seen from the results of the core drilling shown on plate IV.

The data for the problem was obtained at the city engineer's office in the City Hall at San Diego. It consisted of the results of the core drilling at the two sites together with profiles of the two locations. A contour map of damsite #2 was obtained, and a location map of the different damsites on the San Diego River. This latter map shows the proposed route for the flume from El Capitan to the city of San Diego, emptying into the University Heights Reservoir. This route is proposed by Mr. Savage, who was hydraulic engineer for the city for a number of years. Since the proposed dam is to be 200 feet high it was decided to design a dam of this height in order that comparisons might be made with the design of the city engineers.

The object of this thesis is simply to design a dam for the El Capitan damsite which would best suit the conditions, at a reasonable cost. Although the initial object did not include the design of a siphon spillway, it later developed that this was the most suitable type, in the author's estimation. This design was therefore incorporated in the problem and this was the most difficult part due to the very small amount of work which has been done on siphon spillway design. No design showing placing of steel could be found, although I learned that old steel rails were used in the siphons at the Sweetwater Reservoir.

In choosing the type of dam to design, multiple arch, constant angle arch, arched gravity, and straight gravity dams were considered. The main disadvantage of the multiple arch dam for this site is the extreme height for which the dam must be designed. Since the advisability of a multiple arch dam over 100 feet high is somewhat questionable, this type was not used.

The question of a constant angle arch dam was also considered but the extreme length of the dam made this type unsuitable. This type would require at least an angle of 100° for an economical section. This would increase the length of the structure very considerably and would require probably as great an expenditure as a gravity section and with less dependability.

The arched gravity was discarded in favor of the straight gravity for practically the same reasons as mentioned in the preceding paragraph. It was thought that the additional length of dam required to form the arch would offset any advantage gained through arch action. Also, the fact that the sides of the canyon are not steep is against any type of arch, since a very small amount of slipping of the arch abutments would be a very serious condition. A gravity section was therefore designed taking into account the water pressure without the consideration of uplift. The resultant was kept within the middle third throughout.

PRELIMINARY CONSIDERATIONS OF A SIPHON SPILLWAY

A siphon spillway was decided on for the following reasons:

First, close regulation of the water seemed desireable due to the high value placed on water in this vicinity.

Second, a weir spillway was out of the question due to the extreme length of crest required with a reasonable depth of flow over the weir. In order to pass a flow of 50,000 sedond feet over a weir with a depth of 4 feet it would require a length of 1880 feet. With a depth of 8 feet it would require a length of 663 feet, and with a depth of 10 feet a length of 475 feet. The length available for spill is limited to about 400 feet.

Third, it was decided that a siphon spillway could be constructed considerably cheaper than a high-pressure gate type, and that siphons would be more dependable.

The argument against siphon spillways is that very little is known of the stresses set up when siphonic action takes place, especially if there is any tendency toward vibration within the siphon. However, due to the special type of siphon used in the design very little vibration is to be expected, if any at all. This design is different from other siphons in that the water is allowed to leave the siphon without the usual upward bend at exit which would tend to reflect back into the siphon any vibratory wave of water pressure. In the design of the siphon an additional factor of safety was allowed to take care of any vibration which might occur, and as a factor of ignorance.

PRIMING OF SIPHON

The priming of a siphon occurs as soon as the siphon is sealed at the the top and at the bottom with a resulting rarification of the air within the tube. This rarification is caused by the water flowing through the siphon carrying with it bubbles of air from the inner chamber. This action is cumulative so that as soon as the pressure within is reduced, due to this rarification, more water begins to flow through which carries with it additional air. This continues until the siphon is flowing full.

The usual method of obtaining a seal at the bottom of the siphon is by means of a distinct upward bend just before exit. This type of seal has two disadvantages. First, it tends to increase vibration in the siphon, as already mentioned; and second, in the gravity section of the dam as designed it would require considerable additional concrete. It also decreases the efficiency of the siphon somewhat. For these reasons different means of obtaining a seal were used.

PRIMING OF SIPHON

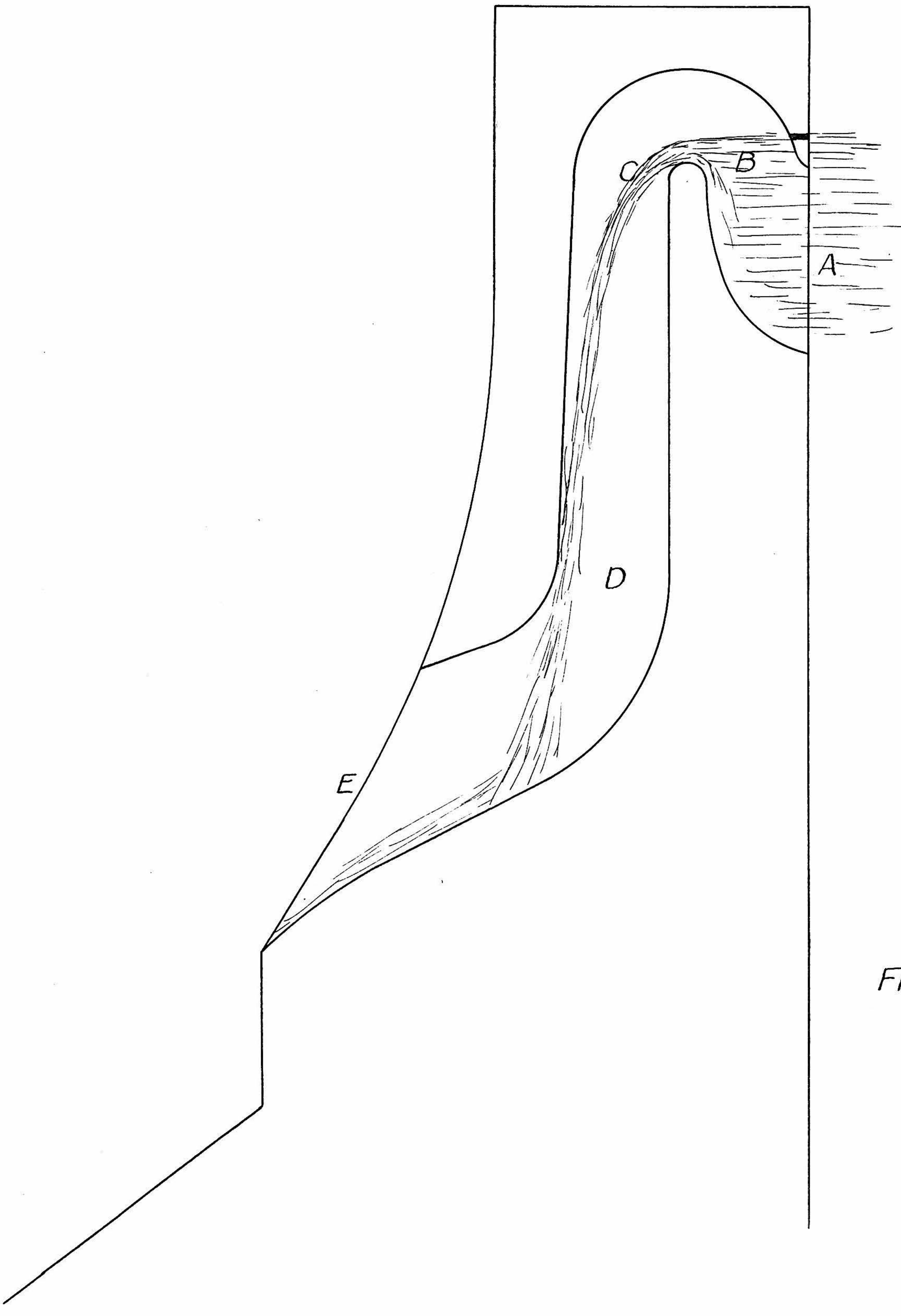

Fig 1.

Refering to figure 1. it can be seen that when the water flows with sufficient velocity over the lip of the siphon to strike the opposite side of the siphon wall, a seal of the lower end will be obtained. In this type of siphon a slightly greater head above the lip is required but this is of small consequence in comparison with its advantages. In computing the head required to accomplish this it is merely necessary to figure the head required on a weir to produce a velocity sufficient to get the water to the opposite side before the force of gravity gets the water to the bottom of the siphon.

TO COMPUTE THE HEAD REQUIRED TO PRIME THE SIPHON

$$H = 1/2\ gt^2$$

Time required for water to fall 30 feet

$$t = \sqrt{\frac{2H}{g}}$$

$$= \sqrt{\frac{2 \times 30}{32.16}} = 1.87 \text{ seconds}$$

And average horizontal velocity

$$v = 3.33\ h^{1/2} \quad \text{where } h = \text{head on weir}$$

Then $3.33\ h^{1/2} 1.87 = 7$ ft.

and $h = 1.26$ feet

It is estimated that the siphon will prime with a head between 1 and 1.26 feet.

PRELIMINARY COMPUTATIONS FOR SIPHON SPILLWAY

The head acting in the siphon is taken from the lip of the siphon to the top of the siphon at discharge end. This is a conservative measurement as there will be a small head above the lip of the siphon. The efficiency for preliminary computations was taken as 60% which is a safe assumption according to engineers who have written on the subject. A more accurate determination of the efficiency is made on the next few pages. The cross-section at the throat of each siphon is 4 by 6 feet.

The discharge for each siphon is equal to

$$Q = c\ A\sqrt{2gh}$$

$$= .60 \times 24\sqrt{2 \times 32.2 \times 32}$$

$$= 1640 \text{ sec.ft. per siphon}$$

Using 20 siphons the total available discharge capacity would be

$$\text{Total } Q = 20 \times 1640 = 32{,}800 \text{ sec. ft.}$$

Since the maximum flow recorded is 38,000 sec. feet it was decided that a spill of at least 50,000 sec. feet should be provided for. The excess spill over the capacity of the siphons is to be taken care of by an ordinary weir type spillway.

The length of weir required to take the remaining 17,200 sec. ft. assuming an 8 foot depth overflow.

$$L = \frac{17200}{3.33 \times 8^{3/2}} = 236 \text{ feet}$$

THEORETICAL DETERMINATION OF LOSSES IN SIPHON

The losses which occur in a siphon are:

Friction loss, $h_f = \frac{L\ v^2}{12000}$

Entrance velocity loss. $h_v = \frac{v^2}{2g}$

Loss due to bends, $h_b = c\frac{v^2}{2g}$,where c is some constant depending on the angle of bend.

These losses were determined using as a guide Mr. Creager's discussion of losses in siphons published in the proceedings of the A.S.C.E. for April 1922.

Referring to Fig. I

The losses from A to B are:

$$h_f = \frac{2xv^2}{12000} = .00017\ v^2$$

$$h_v = \frac{v^2}{2g} = .0155\ v^2$$

$$h_b = 1/8\frac{v^2}{2g} = .00194\ v^2$$

The losses from B to C are:

$$h_f = \frac{9xv^2}{12000} = .00075\ v^2$$

$$h_b = .9\frac{v^2}{2g} = .0139\ v^2$$

Loss from C to D are:

$$h_f = 28\frac{v^2}{12000} = .00234\ v^2$$

Losses from D to E are:

$$h_f = 15\frac{v^2}{12000} = .00125\ v^2$$

$$h_b = .2\frac{v^2}{2g} = .00311\ v^2$$

Total head = $.0390\ v^2$ = 32 ft.

DETERMINATION OF EFFICIENCY OF SIPHON

Since the total head = $.039\ v^2 = 32$

Therefore the velocity = 28.7 ft/sec.

The theoretical velocity = $\sqrt{2xgx32}$ = 45.4 ft/sec

Therefore the efficiency = 28.7/45.4 = 63.2 %

Taking the average section as 6.5 x 10 the discharge per siphon,

$$Q = .632.x65\sqrt{2gx32} = 1860 \text{ sec. ft.}$$

Total Q = 20 x 1860 = 37,200 sec. ft.

This figure is probably more accurate than the previous one. Since the maximum recorded flood recorded is 38,000 sec. ft. it is possible that no weir spillway would be necessary. However, due to the short period for which records are available it is thought that a weir spillway would be added to insure safety to the structure. The additional cost would be quite negligible in comparison with the total cost and the added insurance against flood to towns below the structure would fully warrant such construction.

A glareing example of inadequate spillway capacity is found only about twenty -five miles from the El Capitan site. This was the Sweetwater Reservoir which supplies the southern part of San Diego with water. It was originally constructed with a spillway capacity of 1500 sec. ft. which was the maximum flood recorded at that time. It was necessary to enlarge this at three different times until its present spillway capacity is about 35,000 sec. ft. At this reservoir are located six siphons with a capacity of 12,000 sec. ft.

COMPUTATIONS TO DETERMINE THE STRESSES DUE TO FLOW IN SIPHON

In order to allow for any additional head which might come on the siphons in time of maximum flood the stresses will be determined assuming a flow of 2000 sec. ft. per siphon instead of 1860 sec. ft.

The entrance velocity would then be

$$V_1 = \frac{2000}{12x10} = 16.7 \text{ ft/sec.}$$

The velocity at the throat

$$V_2 = \frac{2000}{6x10} = 33.4 \text{ ft/sec.}$$

The dynamic action of the water due to the first bend and acting horizontally on the front cantilever of the siphon

$R_x = \frac{W}{g}(V_1 - V_2 \cos \theta)$ where θ is the angle through which the water is turned, taken as 60°.

$$R_x = \frac{2000x62.4}{32.16} (16.7 - 33.4x1/2)$$

$$= 0 \text{ force due to dynamic action}$$

The reason that this force is so small is due to the fact that the water velocity is increased at the same time that it is turned.

The only two forces acting on this front cantilever are the force due to water pressure and the force due to atmospheric pressure. The reason that this latter force must be taken into account is due to the partial vacuum within the siphon. For purposes of computation the pressure within the siphon is taken as zero at the throat and uniformly varying down to the discharge. This distribution of pressure is borne out approximately by pressure tests in a siphon which were made at the California Institute of Technology in 1916 by Robert N. Allen.

With these assumptions the bending moment acting on the front cantilever

$$M_x = \frac{62.4 x 25^2 x 25}{2x3} + \frac{14.7}{2} x 25 x 12^2 x 25 x 2/3$$

$$= 162,500 + 441,000 = 603,500 \text{ ft.lbs.}$$

Considering the front of the siphon to act purely as a cantilever the steel required

$$A_s = \frac{603,500 \ x \ 8}{16000x \ 7 \ x \ 9} = 4.8 \text{ in}^2 \text{ per foot}$$

The force acting per square foot on the face of the dam 12 feet below the lip of the siphon

$$p = 62.4 \ x \ 12 + \frac{13}{25} x 14.7 x 144$$

$$= 749 + 1110 = 1859 \text{ lbs per ft}^2$$

Considering this force to be taken into the 4 foot solid sections between siphons the bending moment at the chosen section is

$$M = \frac{1}{8} 1859 x 14^2 x 12 = 655,000 \text{ in.lbs.}$$

Then steel required $= \frac{655,000}{16000x9x12} \ x \frac{8}{7} = .43 \text{ in}^2$ per ft.

The steel computed above is vertical steel and the latter is placed horizontal. Both are not necessary to the full amount so about two thirds of each will be used. The steel distribution is shown on Plate III

The upward reaction due to 150° bend is

$$R_y = \frac{W}{g}(V_1 \cos 0 - V_2)$$

$$= \frac{2000x62.4}{32.16} \cancel{x}\ 33.5 (.866 + 1)$$

$$= 243,000 \text{ lbs.}$$

Moment acting = 243,000x6x12 = 1,750,000 in.lbs/ft.

Steel required assuming about a 6 foot section

$$A_s = \frac{1,750,000x8}{16000x6x12x7} = 1.74 \text{ in}^2$$

At the 4 foot section the moment will be about 1/4

And the steel required $= \dfrac{1/4 \text{ x } 1,750,000}{16000\text{x } 7/8 \text{ x}4\text{x}12} = .651 \text{ in}^2$

These figures are total steel per siphon.

Bending moment acting on the back cantilever of siphon

$$M_x = \frac{14.7}{2} \text{x } 25 \text{ x } 12^2 \text{x} \frac{25}{3} = 221,000 \text{ ft.lbs per foot}$$

$$A_s = \frac{221,000}{16000\text{x } 7/8 \text{ x}6} = 2.63 \text{ in}^2 \text{ per foot.}$$

The shear in the concrete due to forces on the front cantilever near the top where shear is maximum

$$v = \frac{14.7 \text{ x } 10}{3.5 \text{ x } 2} = 21 \text{ lbs per in}^2 \text{ O.K.}$$

Total weight above each siphon

$$W = 340 \text{ x } 150 \text{ x } 14 = 715,000 \text{ lbs.}$$

Compressive stress in concrete when siphon is not running, F_c =715,000/ 24x4x144 = 51.7 lbs per in^2 O.K.

When the siphon is running full this stress will be reduced.

For the amount of steel used and the placing consult Plate III

Each siphon has two vent pipes located one foot above the lip of the siphon to prevent vibration. If this is not provided the siphon will begin to flow before there is sufficient head to keep it running, and the result is that the siphon will take in a gulp of air and then stop. This is where the greatest danger from vibration occurs, and should be guarded against.

The siphons are arranged as shown on plate I, there being ten siphons on each side with a 200 foot weir spillway in the center. It was thought that this arrangement would tend toward greater stability in case of vibration in the siphons. Every other siphon is lowered 6 inches so that five siphons will go into operation at the same time on either side of the dam.

The total number of cubic yards of concrete required is about 560,000. which is 4.25 cubic yards of concrete for each acre foot of capacity. The cost of concrete at the damsite is $8 per cubic yard. This would mean a cost of $4,480,000 for the concrete alone. To this must be added the cost of the steel required for reinforcement. Also the cost of excavation would be additional and the cost of formwork for the siphons. The water to be taken from the reservoir would be obtained by means of four valves located at elevations 590, 630, 670, and 710. From these valves the water could be run through a chlorinator, as is done at the Sweetwater Dam, and then through the conduit to the city.