THESIS

"SIPHON SPILLWAY"

Presented as part of the requirement
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by

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CONCLUSIONS

A brief summary of conclusions drawn from this work is as follows:

1. The coefficient of discharge increases directly as the operating head on a siphon spillway while discharging into deep water, and decreases rapidly with increase of head when discharge is in shallow water and air.

The rate of discharge increases with the head, and approaches a nearly constant value for higher heads.

Similar tests made on the model without the air trap seal has the effect of increasing the rate of discharge when discharging into air, and a slight increase of the coefficient of discharge at lower heads.

2. Better means of measuring pressure must be sought, and until then, <u>accurate</u> means of obtaining losses from point to point are not available. A Pitot tube exploration seems the best suggestion. The maximum losses are in the entrance, throat, and bend over the crest. $Q = AV$ does not hold for ordinary piezometer tube measurements, and Bernoulli's theorem may not be applied.

3. The principles of similitude may be applied to the siphon, and the scale factor may be increased to 4, with all characteristics remaining the same. The discharge varies as the five halves power of the scale factor.

THE SIPHON SPILLWAY

The siphon spillway is a closed conduit of an inverted U shape, extending thru a dam, where a fall is available, which utilizes the siphonic principle to induce a high velocity of flow. The siphon is built in the masonry of the dam.

This siphonic principle, or the principle of the flow of liquids thru a tube of the inverted U type, was known to the ancients, but their use of this principle was confined to small tubes and very small quantities of liquids, and only in recent years has this principle been applied to large tubes and their use in discharging large quantities of water from a higher to a lower level. The first application of this principle, in the form of a siphon spillway, in the United States, was made by G. F. Stickney on the New York State Barge Canal in 1909, but it had previously been used in Europe as early as 1870, or before. The first spillways of this kind were of rather poor design, and were not used for large quantities of water, having a coefficient of discharge of about 50%. Only in the last fifteen years have they been applied to large quantities of water and the coefficient of discharge raised as high as 75 to 80%.

The siphon spillway cannot be used for every purpose where a spillway is needed, and only in certain places is its use practical or economical. In the siphon spillways so far constructed, the action has been almost entirely automatic, but it is believed by many engineers that they may be used to advantage with manually controlled priming and stopping devices. Some of the advantages of the siphon spillway over the "overflow" spillway with floodgate control are: reliability of action, simplicity and permanence of construction, freedom from maintainance and operating expense, close regulation of water level (automatically), and large discharge capacity for a minimum width of spillway. In the majority of places where the siphon spillway has been used, it was necessary to keep the water above the spillway at a nearly constant level with large fluctuations in the amount of flow, where an overflow spillway of sufficient width to care for the maximum flow was impractical with the allowable head, and control gates were too complicated for economy in operation and construction. There is also its economic length of crest, as, for small depths of overflow it will pass considerably more water than an open crest of the same length and thus insure greater storage capacity.

The advantage of the siphon spillway over the ordinary spillway for obtaining greater storage capacity can readily be seen by an example:

Consider an overflow spillway of width b. Then the overflow discharge will be $q = \frac{10}{3} bh^{3/2}$

A siphon spillway of equal width and a depth at the throat of one foot, operating under a head of thirty feet, will have a discharge as $q = 0.7 \; b \times 1 \times \sqrt{2g30}$

If these two spillways are to serve the same purpose, their rates of discharge must be equal. Then

$$q = \frac{10}{3} bh^{3/2} = 0.7\, b\sqrt{60g}$$

$$h^{3/2} = 92.4$$

$$h = 20.5 \text{ ft.}$$

Thus the siphon spillway in this case needs only one foot above the crest, as compared to an overflow spillway which requires 20.5 feet over the crest to obtain sufficient head for the same flow.

There are certain types of dams that have a limited height, due to topographic features, etc., so that a saving of twenty feet, or even a few feet, would greatly increase the storage of the reservoir. Such an increase amounted to 10,500,000 cu. ft. when a siphon spillway was installed on the Lagolungo Dam in Tripoli in place of an overflow spillway.

Before continuing with discussion of the siphon spillway, it will be best to have a clear understanding of the principles underlying siphonic action.

The head acting on the upper water surface is the atmospheric pressure of 14.7 lbs./sq. in. at sea level, which corresponds to a head of 33.9 ft. The atmospheric pressure decreases about 1/2 lb. for each 1,000 ft above sea level. Thus the siphonic head is decreased approximately 1 ft. for each 850 ft. elevation. This decrease must be taken into consideration in designing for the maximum available head on any spillway.

It will be well to understand in the beginning that the flow of water thru a siphon is due to the push of the atmosphere on the upper surface, and is not directly due to the pull of the water flowing out of the lower leg. Suppose both ends of the siphon to be closed to the admission of air and the lower leg be partly full or full of water. The water in this leg will fall, due to its position, and cause a partial vacuum in the crown. The pressure on the water surface then being greater than the pressure inside the siphon, will force the water up in the upper leg, and when the water flows over the crown, due to this difference in pressure, the siphonic action is complete and the water will continue to flow thru it until sufficient air is admitted to the crown to break the action.

The least pressure which may be obtained at the crown is plainly absolute zero, and thus the greatest difference in pressure between the upper water surface and the crown is the atmospheric pressure, and the greatest head available for forcing water thru the crown of the siphon is the head which corresponds to this pressure. To produce a perfect vacuum at the crown, the distance between the crown and the lower water surface must be equal to the "siphonic head," corresponding to the atmospheric pressure, plus all losses in head due to bends and friction in the lower leg.

In a siphon with vertical lower leg, the water rises above the air vent, sealing it against the admission of air, and spills over the crown of the siphon, forming a jet at the throat that jumps across the lower leg. This jet forms a diaphram of water across this leg, which seals it against admission of air from below. A certain volume of air is entrapped in the crown, which is gradually absorbed and carried away by the jet, thus reducing the pressure in the crown and allowing the water level in the upper leg to rise and start the siphonic action, which is broken when the upper water surface falls below the air vent, allowing the admission of air to the crown. A loss of about 3% of the air entrapped in the crown allows the water in the upper leg to rise one foot, which is usually enough to start the siphon, the time required for starting being ordinarily only a few seconds.

A siphon with a slightly inclined lower leg has a sealing basin at the bottom which seals the lower leg against the admission of air instead of the diaphram used in the former design. The jet, formed at the crown, carries the air out thru the sealing basin which is constructed as shown so the air bubbles will not rise inside the lower leg. This type also primes quickly.

In a siphon with the lower leg on a flat slope and a sealing basin, no jet is formed by the crown and this necessitates an auxiliary channel which forms a jet and seals the upper part of the leg against the admission of air in priming.

The efficiency of a siphon is taken as the ratio of the actual flow to the theoretical flow. The theoretical flow is computed by assuming that there are no losses in the conduit and the entire head was utilized as velocity head. Several siphon spillways built by the United States Reclamation Service on the Yuma Project in Arizona have efficiencies varying from 64.4% to 80.5% in the tests made.

It has been found with siphons that the depth of the center of entrance below the water surface should be such that it is not exceeded by the negative pressure head at the entrance. If this negative head at the entrance does exceed the distance to the center of the entrance, vortex may be formed on the water surface and will allow the siphon to suck air, which will lower the efficiency materially, if not stop the siphonic action entirely.

Mr. Arthur P. Cramer, at the Oregon State Agricultural College, found by use of the formula $Q = CA\sqrt{2gh}$ for siphons, a theoretical coefficient of about 0.65, which is a little small for siphons in operation by actual tests.

But before any definite coefficients can be theoretically arrived at, some data and experiments on rectangular and semi-rectangular conduits with fairly high velocities of flow should be obtained. With increased use the design should be improved appreciably and the coefficient of discharge raised as high as 75 or 85%.

Alton Chick, of Brown University, made a study of the angle of divergence, and it was interesting to note that the maximum flow occured with the discharge leg set at an angle approximately the same as that which gives the greatest recovery of velocity head and therefore the maximum efficiency in the venturi meter. About six degrees.

He found that the siphon primes more quickly if the discharge leg is completely submerged. Also, large radii of curves increase the efficiency, but should not be too large as water has to fall on opposite side to prime siphon.

WORK AT TECH

Work was first started on siphon spillways at the California Institute of Technology in 1916, by R. N. Allen. The Institute was then known as Throop College.

Allen gives in his thesis, "The Design and Test of a Model Siphon Spillway," very complete work on priming conditions of the model which he tested.

Allen's work shows that when the discharge lip is under water a higher depth of water over the throat is necessary for priming, and that the time of priming is reduced by increasing the depth over the throat. He also found that the model started more easily and quickly when discharging into air.

Kenneth Belnap and Leonard Ross in 1927 designed and constructed a new model siphon spillway, but were only able to make a few unenlightening tests, due to lack of pumping facilities.

During the following year, George Russell and Earl Peterson were still having trouble with the pumping conditions while taking further tests on the model the year before. To limit these difficulties the inlet leg of the model was lengthened so as to have a longer run before flow ceased by admission of air. This revision was made at the suggestion of Messrs. F. C. Scobey and A. T. Mitchelson, Irrigation Engineers for the Division of Agriculture, who visited the Institute and inspected the work then in progress. When the revision was completed, there was insufficient time for complete tests on the model.

The pumping problem has now been completely solved by addition of a deep well pump, and the work this year has been advanced by Gunner

Gramatky and Kenneth Robinson.

SIPHON SPILLWAY

Some of the details already known in the design of the Siphon Spillway are:

1. A mild curvature of the upper leg of the inlet of the siphon is recommended, not only for the priming of the siphon, but also for the quieter stopping of action when the siphon begins to take air. This curvature, however, should not be too large, as there will be difficulty in priming without some special built-in features.

2. There should be strong iron bars placed in front of the entrance of the siphon to protect it from knocks by ice and other floating bodies, and prevent them from entering the siphon.

3. The lower surface of the inlet leg may be changed, either up or down, within certain limits, without any particular diminution of efficiency.

4. The discharge leg must be made milder for greater efficiency, but not too large as to decrease the quickness of priming.

5. Shortening the discharge leg will not reduce the efficiency.

6. The efficiency is affected by the angle of divergence of the discharge outlet. The most efficient angle of this outlet is a divergence of six degrees. This happens to be the angle for the greatest

recovery of velocity head in a Venturi meter.

7. Sometimes where there is a small flow of water over the crest, it was found advantageous to assist the priming of the siphon by means of a small "nose" attached to the siphon immediately after passing the crest at the throat in order to cause the water to strike the other side of the siphon, thus forming a spray as seal to entrain air and create a partial vacuum, so that priming takes place in a shorter time.

8. It is found that siphons prime more quickly for a smaller head of water over the crest when discharging into water. Some types of siphon prime more quickly and easily when discharging into air.

9. When the submergence of the lower leg is increased, it is necessary to increase the depth of water over the crest for priming.

10. There is a great energy of discharging water from a siphon spillway must be removed in a quieting basin or some form of hydraulic jump or baffles, in order to protect the river bed from erosion by the whirlpools which form.

11. The interior surfaces of the siphon should be made as smooth as possible without large man-holes, to prevent the formation of eddies and the creation of large friction losses.

WORK NEEDED ON SIPHON SPILLWAYS

A thorough study of siphon spillway problems is of great importance in the economical development of many reservoirs and mill ponds by bringing the ordinary working level nearly or quite up to the flood level.

In view of this importance, considerable work has been done on siphon spillways all over the world, and as yet relatively little is known about their fundamental principles, so much more can be done to insure better design, such as:

1. A study of the total energy through a siphon, both experimentally and theoretically, with comparisons to similar tests and data on conduits and curved sections, would throw some light upon design.

2. A study of the action in the crown of a siphon might explain some of the difficulties encountered in operating siphon spillways.

3. A study of the coefficient of discharge as it varies with the head would give considerable aid in operation of siphons as well as design. This type of work has been carried on by Mr. F. C. Scobey in calculating values of this coefficient for various siphons now in existence and compared these with the actual coefficients they were operating under.

4. A study of the relation of the area of the air vent for stopping a siphon to the area of the throat has never been done, and would be of some help in operation and design.

5. A comparison of tests made on siphons with and without air seal at the outlet would also be useful.

6. John R. Freeman, in his "Hydraulic Laboratory Practice", gives the following suggestions for research with siphon spillways:

"Test the limitations of the hydraulic siphon spillway with a greater depth of throat, up to twelve feet in height, try to get a larger percentage of discharge, measured at the crest area, by treating the whole course of the siphon as a long curved venturi tube. Develop better forms for quick exhaust of air, and generally improve and standardize the design of siphon spillways, once for all time."

"A hundred siphon spillways or more have been built in various parts of the world with little knowledge as to the true theory of their operation. It is possible that by regarding this siphon as a long tube and gently curved and elongated Venturi tube, the capacity could be greatly increased, perhaps doubled for low falls. Also it is desirable to know if dangerous shocks can be caused by the opposing surges meeting within the tube, when acting at the extreme limits of atmospheric pressure with its outlet submerged, and if so how best they can be cushioned."

OUR PARTICULAR WORK

With the opening of the fifth year courses in the fall of 1928, and the plan for a research course extending throughout the year, the writers were assigned the task of carrying forward the work. A Peerless Deep Well Pump was installed (See Photograph #1) of the following type: 15 h.p., 220 volt, 1,000 R.P.M., 3 phase, 38 amperes. This gave an additional available quantity of about 1.5 sec. ft., and with the centrifugal pump which had a capacity of about 3 sec. ft., the siphon could be made to run steadily full for long periods of time.

The model is shown in detail by diagram #1, and the only modification made was to change the discharge to horizontal as thereon indicated. Plates I-VI show the model in position in the tanks and need some explanation which will be given.

The siphon was constructed of sheet metal, braced with stiffening angles, and although the constants of friction differ from those with concrete conduits, the relative losses would be the same. The inlet lip was comparatively long, and extended into tank A (see Fig. #2), which was covered. The long outlet leg of the siphon was braced horizontally out from the wall of the smaller tank, B, but the vertical thrust was transmitted up to the crown by the siphon itself. The long inlet leg gave us a high variation in head before the siphon would break, and therefore not much was done on priming conditions as this work had been well covered by Allen.

The particular studies which we set out to accomplish were four in number:

1. Study of efficiency, or coefficient of discharge.

2. Study of the relative and actual heads during discharge. (Also Allen's object).

3. Study of similitude, with respect to actually calculating the size of the actual siphon comparable to the model.

4. Study of air vents, with respect to the start and break of the siphon.

PROCEDURE

1. Nineteen runs were taken in all, and all of these could be used in plotting the efficiency curves, Figs. #3 and #4.

As the only comparative figure seems to be the efficiency or coefficient of discharge, we used it as our guide in working up the data. It is expressed by the short tube formula $Q = KA_t \sqrt{2gh}$

where Q = rate of discharge in sec. ft.

A_t = cross sectional area of siphon at throat, smallest area.

h = head under which siphon operates, in feet.

K = coefficient of discharge, sometimes called efficiency; ratio of $\frac{Q \text{ ideal}}{Q \text{ actual}}$

These runs were taken with varying heads and quantities, and equilibrium was reached in each case before readings were taken.

Water was pumped into tank A until the siphon primed, then the necessary adjustment for the head desired was made by reducing the speed on the centrifugal pump. The quantity on the deep well pump was measured by an orifice meter on the 8" line, read on a mercury manometer constructed especially for the purpose and calibrated by a standard orifice test, later checked more accurately by a Weir discharge test. The quantity of water discharging on the centrifugal was measured directly by a Venturi meter on the 8" line.

Much difficulty was experienced in obtaining and holding the prime of the centrifugal pump, and rather exasperating failures of it occasioned much pumping in order to obtain workable conditions again. It is hoped that this difficulty, probably caused by leaky foot valves, will be corrected in the future.

The head and tail water levels were measured from the floor as datum, with floats and long slender wooden rods. All readings were taken both before and after each run, and if there were any great variations the run was discarded. Starting and finishing conditions were averaged, and the average considered sufficiently accurate, as the runs were for a long enough time to reduce the inaccuracy of such assumptions. The temperature of water and air was recorded.

As soon as it was indicated that the efficiency was higher when discharging under water, several more runs were taken in order to place points on the curve. A discussion of the physical and economical significance of this discovery will appear in the section entitled "Discussion." (See also Figs. 3 and 4).

2. This study called for steady conditions of flow, and that the siphon be run at full capacity. When pumping conditions were equalized and steady flow had apparently been obtained the method was as follows:

The "U" mercury columns, measuring pressure from the nipples shown in Fig. 1, were carefully tested to see that no water remained in the rubber tube joining the nipples with the manometer. When it was ascertained that we were getting the true pressure at the point, it was read on the manometer scale. The inverted columns were taken from the T. W. and carefully tested for any water remaining in them, and then were lowered into the T. W. When all columns were in as nearly perfect condition as possible, the run was started, and as before, T. W., H. W., Q on both pumps, temperature, time, and all gage and manometer readings were taken.

The head losses were calculated as follows:

Q = discharge of siphon in sec. ft.

A = cross sectional area of any section as determined by measurement.

V = the velocity in ft./sec. at any section.

p = pressure head in feet at any point.

z = height of the section above some datum. Tube #11 = 0.00 ft.

H = total head acting on siphon.

The fundamental equations used were: $Q = AV$ and Bernoulli's.

$H_1 = H_2 + H'$ which in detail

$$P_2 + z_1 + \frac{V_1^2}{2g} = p_2 + z_2 + \frac{V_2^2}{2g} + \text{losses.}$$

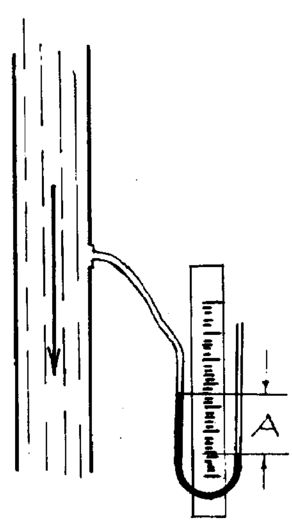
All quantities are in feet of water.

p is pressure head.

z is elevation head.

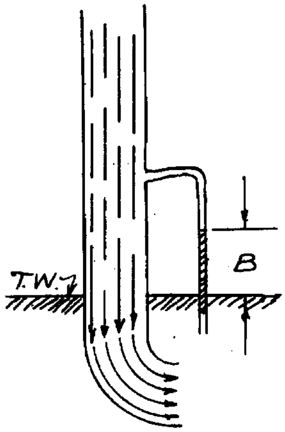
$\frac{V^2}{2g}$ is the velocity head.

THE "U" MERCURY TUBES



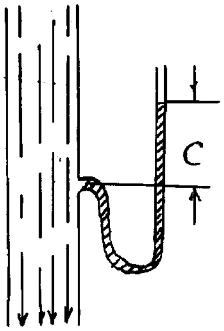
A single manometer tube was used for all tubes with a considerable vacuum, and those not in use were closed with clamps. The rubber tube was allowed to fill with air, and all the water was exhausted before taking any readings, hence the value of reading A, when converted into feet of water, is the pressure at this point.

$$p = \frac{13.57}{12} \times A$$



THE INVERTED T.W. TUBES

These were inserted directly into the T.W. and the direct reading, B, in feet measured the vacuum at that point.



"U" WATER COLUMN

These were used on the lower half of the outlet leg and measured positive pressures above the nipple directly in feet, as shown.

These readings were taken for twelve different runs with heads varying from 7.5 feet to 1.5 feet, and Bernoulli's equation applied for each point. It was discovered immediately that there was a lively discrepancy between the total energy gradient and the plotted values of the equation. Runs were more carefully checked and curves plotted for each of the twelve runs. The most typical of these, run #5 and #11 are shown on figure # 5. The rest are not included because they are exactly the same type, and obviously can have no physical significance, as they violate the fundamental conservation of energy principle, for in several places there appears a gain in energy, which is absurd. The total energy gradient, plotted from assumed losses, in entrance, bends, contractions and enlargements was taken from Daugherty's "Hydraulics," and Creager and Justin, "Hydro Electric Handbook."

It indicates that the maximum losses in this particular model are in the throat and first bend, and a better design might well improve this model. The reasons for this difference are explainable and are handled in the "discussion" of this work. The writers' analysis of the difficulties were corroborated by Professor R. L. Daugherty, and Mr. F. C. Scobey, Senior Irrigation Engineer, U. S. Department of Agriculture.

AIR VENTS

4. Inasmuch as only one model was available, the problem of the amount of air necessary to cause the siphon to break would have involved changes in the structure which it was not felt advisable to make. This study would have been worth while, and regret is felt at not being able to carry it on. Work has been done on this, however, and it was not as important a part of the research as 1, 2, and 3.

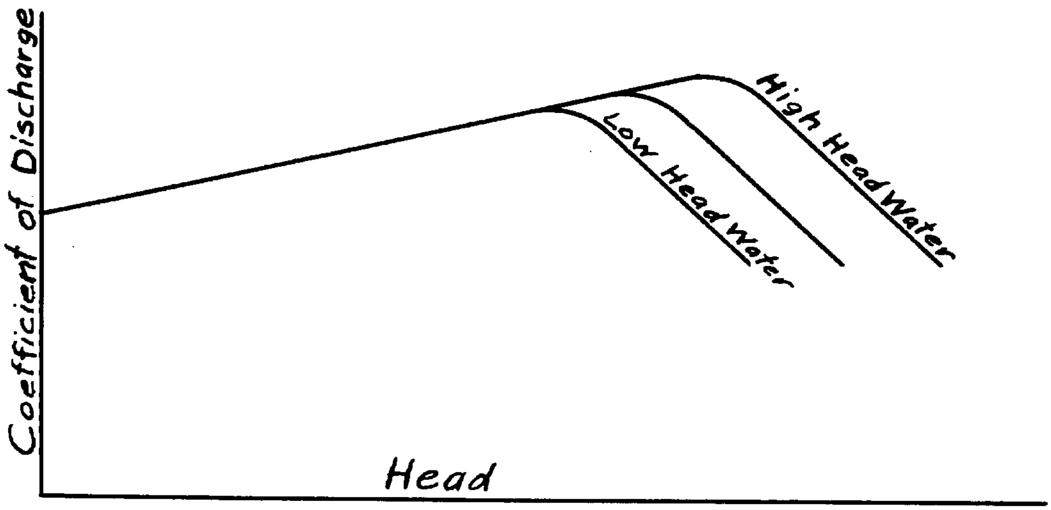
DISCUSSION

1. In the study of the coefficient of discharge, the formula for short tube orifices was used in computing the so-called efficiency, as this seems to be in accord with current practice in siphon spillways, and gives a rough method of comparison. The relation, as mentioned before, is $Q = KA_t \sqrt{2gh}$

From a curve drawn from a large number of runs it is found that the values of this coefficient, K, increase almost directly as the head, while the siphon is discharging into deep water. When the tail water is less than two feet above the center of the outlet, the coefficient starts dropping rather rapidly with increasing head and when discharging into air it has a much lower value and is practically a constant value.

These relations appear on the curves which accompany the report.

From these results it may be predicted that, by varying the tail water and keeping the head water at a constant level during a set of runs, and then using a different head water level for another series of test runs, and so on, a set of curves could be obtained similar to those shown on the figure herewith given.



From these curves it can readily be seen that the coefficient starts dropping off at shallow discharge which will be at a lower head when the head water is low. Thus shallow water is reached in the curve at different heads for different heights of head water, giving the peculiar breaks in the curve shown.

From a coefficient of discharge curve we might draw the conclusion that when a siphon is discharging into a certain depth of water above the center of the outlet, which was about two feet in our case, a higher efficiency or coefficient of discharge is obtained than when discharging into air with a lower coefficient of discharge and a thirty to forty per cent greater head and practically the same rate of discharge.

The rate of discharge is seen to increase with the head about as would be expected, approaching a nearly constant value for the higher heads.

As before stated, these tests were made both with and without the air seal or water trap, that is, with the discharge upward at about 45 degrees and with it horizontal. In thus revising the model to horizontal discharge we thought the losses in the bend at the bottom would be reduced, and the coefficient and rate of discharge would be increased for the same head.

Tests on this revised model show that it had the effect of increasing the rate of discharge slightly at high heads when discharging into air, and of increasing the coefficient of discharge slightly when discharging into deep water with lower heads.

2. In regard to the false energy gradient, there are several important facts to consider in its explanation.

a. In the first place, in using Bernoulli's equation, we used for the velocity head, the value given by $\frac{Q}{A}$. Now this assumes that each particle both moves in a straight line parallel to the axis and with a velocity which is constant throughout the section. Neither of these assumptions are true, especially in an irregular conduit, i.e., the velocity may have a direction nearly at right angles to the axis of the siphon, or in fact in some cases, might even be doubling back on itself. Furthermore the velocity may be very different, with a different pressure change on the inside of the siphon than on the outside. This is indicated by the results of tubes 5 and 5a, one located on the top and the other on the bottom of the siphon at the crest. An error of 25%, which might easily creep into the velocity at this point, would make a difference of nearly one foot of head. This same error, if applied to the higher readings on the tubes, would decrease the pressure perhaps a foot or a foot and a half. This might well smooth out the curve and make it a true value.

It should be noted that the average velocity squared, used in $\frac{V^2}{2g}$, is not the same as the average of the velocity heads taken at different points. The energy found in this manner is almost invariably greater than that computed from the average velocity.

b. Secondly, although leading directly out of the first, the available methods of measuring velocity head and particularly pressure are in error. In the model used it was quite a task to take one pressure reading on the center line of the side of the siphon at intervals of about 1.5 feet. It would be practically impossible to take enough pressure readings to accurately determine energy conditions. The most complete method, and one which might successfully be employed, would be to explore each section with a Pitot tube, measuring both pressure and velocity head. This would be fairly accurate, although some sort of device with a swivel joint would have to be constructed so that all parts of the section would be explored. The average velocity could then be determined as by any standard method given in texts on hydraulics. This would be superior to having a ring of tubes around the outside of each section, as in any case only the values at the siphon wall would be obtained.

The difficulties surrounding accurate pressure readings are very great, and Professor C. M. Allen, of Worcester Polytechnic, is now preparing a paper on this very subject which is to cover years of observation.

It is significant to note that the familiar equation of continuity, $Q = AV$ does not hold for the siphon spillway with respect to the average velocities, and consequently, when the pressure readings are correspondingly in error, Bernoulli's equation can hardly be expected to furnish accurate results.

The data shows that all readings were consistent, one nipple being steadily low or high throughout the runs. With some better means of measuring pressure, it would be possible to definitely assign and find the losses, and hence to improve the design. The writers regret exceedingly that the pressure readings are practically worthless, but feel somewhat repaid in discovering that the fundamental equations were at fault and not the experimenting.

SIMILITUDE

3. A study of the factors affecting similitude in siphonic and other hydraulic apparatus was made, and the notes prepared will be found in Appendix D. The work done on other siphons chose only circular channels, and correlation seems difficult between the two.

The velocity is given by the relation $V = \sqrt{\dfrac{2gH}{(1+e) + y\frac{L}{4m}}}$

Where $V =$ velocity in feet per second

$H =$ head on siphon

$e =$ entrance coefficient

$y =$ friction coefficient

$L =$ length of siphon

$m =$ hydraulic radius

Now if the scale is increased the relation is (the scale factor being $= n$): $V = \sqrt{\dfrac{2gHn}{(1+e) + y\frac{Ln}{4mn}}}$

From this relation it can be seen that V varies approximately as the $\sqrt{n}$, and while the friction factor is large for the model, it has been shown to be only a minor factor to be neglected (pages 515, 516 Hydraulic Laboratory Practice, R. Freeman).

Since $Q = AV$ approximately, and A varies as n^2

$Q_{actual} = Q_{model} \times n^2 \times n^{1/2} = Q_m \times n^{5/2}$

Now for our model, n can be made about 4, since the limitation placed on the head is that the pressure at the crown of the siphon must be equal to a greater than the vapor pressure of water if the siphon is to operate. Hence

$Q_{actual} = 3.35 \times 4^2 \times \sqrt{4} = 117.2$ sec. ft.

The possibility of building a siphon of much larger cross sectional area of course does not enter this discussion, as we are interested only in knowing if the characteristics obtained by test are applicable to the larger siphon. The same value of K would hold for the larger spillways, since on analysis

$$K = \frac{1}{\sqrt{(1 + e) + \frac{Ln}{4mn}}}$$

the n's cancelling out. It is a very helpful design factor to know what value of the discharge factor to expect, and with this sort of comparison, results are predictable within reasonable limits of accuracy.

CALCULATED DATA, APPENDIX "A".

Run No. 1. February 2, 1929. $Q = 3.16$ $h = 7.235$ $K = 0.730$

1. Tube No.	2. Area Sq.ft.	3. $Q = AV$ Vel.	4. $\frac{V^2}{2g}$	5. Z on No. 11.	6. on gage	7. gage reading	8. inches Hg.	9. Press. corr'd	10. H Energy
1.	.2500	12.65	2.48	5.10	2.30	3.65TW	[illegible]	-3.95	3.63
2.	.2290	13.80	2.96	5.60	1.80		5.00	-5.65	2.91
3.	.2083	15.19	3.58	6.10	1.30		5.13	-5.81	3.87
4.	.2083	15.19	3.58	6.76	0.64		4.75	-5.38	4.96
5.	.2083	15.19	3.58	6.80	0.60		7.75	-8.77	1.61
5a.	.2083	15.19	3.58	7.00	0.40		[illegible]	--	--
6.	.2083	15.19	3.58	6.75	0.64		3.50	-3.96	6.38
7.	.2083	15.19	3.58	6.00	1.40	[illegible]	6.00	-6.80	2.78
8.	.2413	13.10	2.67	4.50	2.90	5.00TW		-2.60	4.57
9.	.2743	11.51	2.06	3.00	4.40	6.60TW		-1.00	4.06
10.	.3073	10.30	1.65	1.50	5.90	4.95 U		1.00	4.15
11.	.3403	9.30	1.34	. 0	7.40	5.45 U		1.85	3.19
12.	.3403	9.30	1.34	-0.60	7.98	4.75 U		3.23	3.97

	Start	Finish	Av.			
HW	3'-4½"	3'-4¼"	3.365'	7.235	0.165	
TW	10¾'-	10.785'	10.81'	-0.21	7.60	
Orifice	3.7	3.8		"Hg	$Q = 0.933\ h^{\frac{1}{2}}$	
Meter	10.7	10.60	10.65	3.45	1.73	
Venturi			1.43		<u>1.43</u>	
Total Q					3.16 cu. ft. per second.	

Run No. 2. Feb. 2, 1929. $Q = 3.22$ $h = 7.25$ $K = 0.716$

1. Tube No.	2. Area Sq. ft.	3. $Q = AV$ Vel.	4. $\frac{V^2}{2g}$	5. Z on No. 11.	6. on gage	7. gage reading	8. inches Hg.	9. Press. corr'd	10. H Energy
1.	same	12.88	2.56	same	same	3.60		-4.17	3.49
2.	as	14.07	3.07	as	as		-4.12	-4.65	4.02
3.	before	15.40	3.68	before	before		-5.07	-5.73	4.05
4.		15.40	3.68				-4.69	-5.30	5.14
5.		15.40	3.68				-7.75	-8.76	1.72
5a.		15.40	3.68				--	[illegible]	--
6.		15.40	3.68				3.19	-3.60	6.80
7.		15.40	3.68				5.87	-6.66	3.02
8.		13.35	2.77			5.55		-2.22	4.55
9.		11.74	2.14			6.75		-1.02	4.12
10.		10.48	1.71			4.95		-0.95	2.26
11.		9.46	1.39			5.45		1.95	3.34
12.		9.46	1.39			4.75		3.23	4.02

	Start	Finish	Av.		
HW	3'-4½"	3'-4"	3.35'	7.25	0.15
TW	10'-11"	11'-0¼"	10.97'	-0.37	7.77
Orifice	-3.8	-3.8		"Hg	Q
Meter	10.6	10.6	7.40	3.70	1.79
Venturi			1.43		1.43
Total Q					3.22 cu. ft. per second.

Run No. 3. Feb. 2, 1929 Q = 2.42 h = 3,57 K = 0.766

1. Tube No.	2. Area Sq.ft.	3. Vel.	4. $\frac{V^2}{2g}$	5. Z on No. 11.	6. on gage	7. gage reading	8. inches Hg.	9. Press. corr'd	10. H Energy
1.	same	9.69	1.46	same	same	3.94		-1.03	5,53
2.	as	10.57	1.90	as	as		3.56	-4.03	3.47
3.	before	11.72	2.16	before	before		3.62	-4.10	4.16
4.		"	"				3.50	-3.96	4.96
5.		"	"				5.38	-6.09	2.87
6.		"	"				1.75	-1.98	0.94
7.		"	"				3.75	-4.24	3.92
8.		10.03	1.56			3.80		-1.17	4.89
9.		8.83	1.21			4.80		-0.17	4.04
10.		7.88	0.97			4.30		1.60	4.07
11.		7.12	0.79			4.65		2.75	5.54
12.		7.12	0.79			3.95		4.03	4.22
	Start	Finish	Av.						
HW	4'-7¼"	4'-7¼"	4.60'	6.00	1.40				
TW	8'-2"	8'-2¼"	8.17'	2.43	4.97				
Orifice	4.0	4.0	4.0	"Hg	Q				
Meter	10.4	10.4	10.4	3.5	1.74				
Venturi			0.68		0.68				
Total Q					2.42 cu.ft. per second.				

Run No. 4. April 5, 1929. Q = 3.825 h = 7.28 L = 0.720

1. Tube No.	2. Area Sq. ft.	3. Vel.	4. $\frac{V^2}{2g}$	5. L	6. on gage	7. gage reading	8. inches Hg.	9. Press corr'd	10. H Energy
1.	same	12.90	2.59	same	same	4.25		-3.97	3.73
2.	as	14.10	3.09	as	as		5.05	-5.71	2.98
3.	before	15.50	3.73	before	before		4.90	-5.53	4.30
4.		"	"				2.85	-3.22	7.27
5.		"	"				7.45	-6.42	2.11
5a.		"	"				5.60	-6.33	4.40
6.		"	"				4.00	-4.52	5.97
7.		"	"				5.47	-6.16	3.57
8.		13.40	2.78			5.90		-2.32	4.96
9.		11.78	2.15				1.20	-1.36	3.79
10.		10.49	1.71			4.70		1.20	4.41
11.		9.49	1.40			5.25		2.15	3.65
12.		9.49	1.40			4.60		3.38	4.20

	Start	Finish	Av.		
H.W.	3'-3½"	3'-5½"	3.38'	7.22	0.18
T.W.	11'-6½"	11'-3½"	11.42'	-0.82	8.22
				"Hg	Q
Orifice Meter	1.5 5.1	1.5 5.1	6.6	3.30	1.692
Venturi	1.54	1.525	1.5325		1.533
Total Q					3.825 cu. ft. per second.

Run No. 5. April 5, 1929. Q 3.237 h 7.27 K 0.720

1. Tube No.	2. Area Sq. ft.	3. Q = AV Vel.	4. $\frac{V^2}{2g}$	5. Z on No. 11.	6. on gage	7. gage reading	8. inches Hg.	9. Press. corr'd	10. H Energy
Inlet.	.667	4.86	0.367						
1.	.2500	12.96	2.600	5.10	2.30	5.70TW		-3.77	3.93
2.	.2290	14.80	3.40	5.60	1.80		5.05	-5.71	3.29
3.	.2083	15.55	3.75	6.10	1.30		4.80	-5.45	4.42
4.	.2083	15.55	3.75	6.76	0.64		5.40	-6.10	4.41
5.	.2083	15.55	3.75	6.60	0.60		7.50	-8.48	2.07
5a.	.2083	15.55	3.75	7.00	0.40		5.60	-6.33	4.42
6.	.2083	15.55	3.75	6.76	0.64		4.75	-5.37	5.14
7.	.2083	15.55	3.75	6.00	1.40		5.50	-6.21	3.54
8.	.2413	13.43	2.80	4.50	2.90	4.55TW		-[illegible].93	4.37
9.	.2743	11.81	2.16	3.00	4.40	6.80TW		-0.67	4.49
10.	.3073	10.55	1.73	1.50	5.90	4.75 U		1.15	4.38
11.	.3403	9.50	1.40	0	7.40	5.20 U		2.20	3.60
12.	.3403	9.50	1.40	-0.58	7.98	4.90 U		3.18	4.00

	Start	Finish	Av.		
HW	3'-4"	3'-4"	3.33'	7.27	0.13
TW	10'-8½"	10'-7½"	10.67'	-0.07	7.47

				"Hg	$Q = 0.958\ h^{\frac{1}{2}}$
Orifice Meter	1.50	1.50			
	5.10	5.10	3.60	3.30	1.692
Venturi	1.54	1.55	1.545		1.545
Total Q					3.237 cu. ft. per second.

Run No. 6. April 5, 193[illegible]. Q 3.24 h 6.01 . E 0.[illegible]

1. Tube No.	2. Area	3. Vel.	4. $\frac{V^2}{2g}$	5. Z	6. on gage	7. gage reading	8. inches Hg.	9. Press. corr'd	10. E Ener.
1.	same	12.97	2.62	same	same	3.4		-3.85	3.87
2.	as	14.17	3.12	as	as	4.95		-5.60	3.12
3.	before	15.55	3.75	before	before	4.65		-5.50	4.35
4.		"	"			4.55		-5.15	5.35
5.		"	"			7.25		-8.21	2.34
5a.		"	"			5.70		-6.45	4.30
6.		"	"			4.30		-4.86	5.65
7.		"	"			5.40		-6.11	3.64
8.		13.42	2.80				3.60	-2.41	4.89
9.		11.80	2.16				5.75	-0.26	4.90
10.		10.54	1.73				4.55	1.35	4.58
11.		9.52	1.40				4.90	2.50	3.90
12.		9.52	1.40				4.40	3.58	4.38

	Start	Finish	Av.		
HW	3'-2¼"	3'-2¼"	3.19'	7.41	-0.01
TW	9'-2½	9'-2½"	9.20'	1.40	0.00
				"Hg	Q
Orifice Meter	1.55 5.15	1.60 5.20	6.750	3.375	1.71
Venturi	1.53	1.53	1.53		1.53

Total Q 3.24 cu. [illegible] per second.

Run No. 7. April 6, 1929. Q = 3.15 h = 5.34 K = 0.7[illegible]

1. Tube No.	2. Area	3. Vel.	4. $\frac{V^2}{2g}$	5. Z	6. on gage	7. gage reading	8. "Hg.	9. Press. corr'd	10. H Energy
1.	same	12.60	2.46	same	same		2.70	-3.05	4.51
2.	as	13.77	2.94	as	as		4.10	-4.14	4.40
3.	before	15.12	3.54	before	before		3.90	-4.41	5.23
4.		"	"				4.05	-4.58	5.72
5.		"	"				5.90	-6.67	3.67
5a.		"	"				4.60	-5.20	5.34
6.		"	"				3.70	-4.18	6.12
7.		"	"				4.50	-5.09	4.45
8.		13.09	2.66			3.00		-2.29	4.87
9.		11.51	2.06			3.60		-1.69	3.37
10.		10.03	1.64			2.90		3.0	3.14
11.		9.26	1.33			3.20		4.2	5.52
12.		9.26	1.33			2.75		5.23	5.96

	Start	Finish	AV.		
HW	3'-1½"	3'-2"	3.15	7.45	-0.05
TW	8'-9"	8'-8"	8.79	2.11	5.29
				"Hg	Q
Orifice Meter	1.7 5.3	1.7 5.3	7.0	3.50	1.74
Venturi	1.40	1.41	1.405		1.41
Total Q					3.15 cu. ft. per second.

Run No. 8. April 6, 1929. Q = 2.28 h = 3.575 K = 0.721

1. TubeNo.	2. Area	3. Vel.	4. $\frac{V^2}{2g}$	5. Z	6. on gage	7. gage reading	8. inches Hg.	9. Press. corr'd	10. H Energy
1.	same	9.12	1.29	same	same		1.30	-1.47	4.92
2.	as	9.96	1.54	as	as		2.50	-2.83	4.31
3.	before	10.95	1.85	before	before		2.50	-2.83	5.13
4.		"	"				2.50	-2.83	5.79
5.		"	"				4.20	-4.75	5.91
5a.		"	"				3.10	-3.50	5.36
6.		"	"				2.40	-2.71	5.91
7.		"	"				2.70	-3.05	4.81
8.		9.55	1.42			3.00		-0.80	5.12
9.		8.31	1.07			3.60		-0.20	3.87
10.		7.42	0.85			2.90		3.00	5.35
11.		6.70	0.70			3.20		4.20	4.90
12.		6.70	0.70			2.75		5.23	5.33

HW	Start	Finish	Av.		
HW	3'-5"	3'-5$\frac{1}{4}$"	3.425	7.175	0.225
TW	7'00"	7'-0"	7.00	3.600	3.800

Orifice	0	
Meter	0	
Venturi		2.28

Total Q 2.28 cu.ft per second.

Run No. 9. April 6, 1929. $Q = 1.505$ $h = 1.46$ $k = 0.743$

1. Tube No.	2. Area	3. Vel.	4. $\frac{V^2}{2g}$	5. Z	6. on gage	7. gage reading	8. "Hg	9. Press. corr'd	10. H Energy
1.	same	6.02	0.56	same	same		0.40	-0.45	5.21
2.	as	6.57	0.67	as	as		0.40	-0.45	5.82
3.	before	7.22	0.81	before	before		0.70	-0.79	6.78
4.		"	"				1.10	-1.24	6.37
5.		"	"				1.90	-2.15	5.66
5a.		"	"				1.90	-1.13	5.42
6.		"	"				1.00	-0.68	5.68
7.		"	"				0.60	-0.22	4.63
8.		6.25	0.61			1.60		0	3.39
9.		5.49	0.47			1.82			3.47
10.		4.90	0.37						1.87
11.		5.41	0.46						0.46
12.		5.41	0.46						-0.12
	Start	Finish	Av.						
HW			3.56'	7.04	0.36				
TW			5.02'	5.58	1.82				
Venturi			1.505						
Total Q			1.505 cu. ft. per second.						

Run No. 10. April 12, 1929. $Q = 3.315$ $h = 7.85$ $K = 0.709$

1. Tube No.	2. Area	3. Vel.	4. $\frac{V^2}{2g}$	5. Z	6. on gage	7. gage reading	8. "Hg	9. Press. corr'd	10. H Energy
1.	same	13.26	2.64	same	before		3.8	-4.31	3.43
2.	as	14.04	3.06	as	same		5.3	-6.03	2.63
3.	before	15.91	3.94	before	as		5.2	5.90	4.14
4.		"	"		before		4.3	-4.87	5.83
5.		"	"				7.4	-8.40	2.34
5a.		"	"				5.9	-6.69	4.25
6.		"	"				4.8	-5.44	5.26
7.		"	"				5.8	-6.57	5.37
8.		13.73	2.94				2.1	-2.38	5.06
9.		12.09	2.27			6.7		-1.64	5.83
10.		10.79	1.81			5.0		0.90	4.21
11.		9.74	1.47			5.6		1.80	3.27
12.		9.74	1.47			6.4		1.58	1.00
TW	-1.07			-1.07	8.47				
HW	7.35			7.35	0.05				

Total Q 3.315 cu ft. per second.

Run No. 11. April 12, 1929. Q = 3.23 h = 5.92 K = 0.795

1. Tube No.	2. Area Sq.ft.	3. Q = AV Vel.	4. $\frac{V^2}{2g}$	5. Z on No. 11.	6. on gage	7. gage reading	8. inches Hg	9. Press. corr'd	10. H Energy
Inlet.	.667								
1.	.2500	12.92	2.59	5.10	2.30		3.5	-3.96	3.73
2.	.2290	14.11	3.09	5.60	1.80		5.0	-5.65	3.04
3.	.2083	15.50	3.73	6.10	1.30		4.9	-5.54	4.29
4.	.2083	15.50	3.73	6.76	0.64		4.5	-5.09	5.40
5.	.2083	15.50	3.73	6.80	0.60		6.7	-7.57	3.06
5a.	.2083	15.50	3.73	7.00	0.40		6.4	-7.24	3.49
6.	.2083	15.50	3.73	6.76	0.64		4.7	-5.31	5.17
7.	.2083	15.50	3.73	6.00	1.40		5.5	-6.21	3.52
8.	.2413	13.30	2.78	4.50	2.90		2.7	-3.05	4.23
9.	.2743	11.78	2.15	3.00	4.40	5.35TW		-0.97	4.18
10.	.3073	10.49	1.71	1.50	5.90	4.90 U		1.00	4.21
11.	.3403	9.49	1.40	0	7.40	5.30 U		2.10	3.50
12.	.3403	9.49	1.40	-0.58	7.98	5.40 U		2.58	3.40

	Start	Finish	Av.		
HW	3'-7"	3'-7½"	3.60'	7.00	0.40
TW	9'-6½"	9'-6"	9.53'	1.08	6.32
				"Hg.	$Q = 0.933\ h^{\frac{1}{2}}$
Orifice Meter	1.55 / 5.15	1.60 / 5.20	6.57	3.375	1.713
Venturi	1.52	1.52	1.52		1.52
Total Q					3.233 Cu. ft. per second.

Run No. 12. April 12, 1929. $Q = 2.285$ $h = 3.12$ $K = 0.775$

1. Tube No.	2. Area Sq. ft.	3. $Q = AV$ Vel.	4. $\frac{V^2}{2g}$	5. Z on No. 11.	6. on gage	7. gage Reading	8. inches Hg	9. Press. Corr'd	10. H Energy
1.	same	9.14	1.29	same	same		1.40	-1.59	4.80
2.	as	9.60	1.46	as	as		2.40	-2.72	4.34
3.	before	10.97	1.87	before	before		2.60	-2.95	5.02
4.		"	"				2.60	-2.95	5.68
5.		"	"				4.10	-4.65	4.02
5a.		"	"				2.90	-3.29	5.58
6.		"	"				2.70	-3.06	5.57
7.		10.97	1.87				2.60	-2.95	4.92
8.		9.48	1.39				2.60	-2.95	2.94
9.		8.34	1.08					0.67	4.75
10.		7.44	0.86			3.0		2.17	
11.		6.70	0.69			3.2		3.67	
12.		6.70	0.69			3.25		4.27	
	Start	Finish	Av.						
HW	3'-9½"	3'-10"	3.81'	6.79	0.61				
TW	6'-11½"	6'-10¾"	6.93'	3.67	3.73				
Venturi	2.30	2.27	2.285						

Total Q 2.285 cu. ft. per second.

TABULATED RESULTS. Runs # 1 - 6. Original Model
Discharge Upward.

Run#	#1	#2	#3	#4	#5	#6	
Date	Feb. 2	Feb. 2	Feb. 2	May. 5	May. 5	May. 5	May. 5
Head	7.25	7.25	3.57	7.22	7.27	6.01	
Dischge	3.16	3.22	2.42	3.22	3.24	3.24	
Coeffic.	0.73	0.72	0.77	0.72	0.72	0.79	
Disch in	Air	Air	Water	Air	Air	Water	

Tube General Equation $H = p + z + V^2/2g$

Tube	#1	#2	#3	#4	#5	#6
H. W.	7.23	7.25	6.00	7.22	7.27	7.41
1	3.63	3.49	5.53	3.72	3.93	3.87
2	2.91	4.02	3.47	2.98	3.29	3.12
3	3.87	4.05	4.16	4.30	4.42	4.36
4	4.96	5.14	4.96	7.27	4.41	5.36
5	1.61	1.72	2.87	2.11	2.07	2.34
5a	*	*	*	4.40	4.42	4.30
6	6.38	6.80	6.94	5.97	5.14	5.65
7	2.78	3.02	3.92	3.57	3.54	3.64
8	4.57	4.55	4.89	4.96	4.37	4.89
9	4.06	4.20	4.04	3.79	4.49	4.90
10	4.15	2.26	4.07	4.41	4.38	4.58
11	3.19	3.34	3.54	3.65	3.60	3.90
12	3.97	4.02	4.22	4.20	4.00	4.38
T. W.	- 0.21	- 0.37	2.43	- 0.82	- 0.07	1.40

Note; Datum chosen, Tube 11, z = 0.00';Bernoulli's Equation.

* No pressure readings were taken on Tube 5a until run #4.

TABULATED RESULTS. Runs # 7 - 12.

Runs # 7, 8, 9--Original Model, Discharge Upward.

Runs #10, 11, 12- Revised Model, Horizontal Discharge.

Run #	#7	#8	#9	#10	#11	#12
Date	Apr.6	Apr.6	Apr.6	Apr.12	Apr.12	Apr.12
Head	5.56	3.58	1.46	7.85	5.92	3.16
Disch'ge	3.15	2.28	1.52	3.31	3.23	2.28
Coeffic.	0.80	0.72	0.74	0.71	0.80	0.78
Disch in	Water	Water	Water	Air	Water	Water

Tube # General Equation $H = p + z + V^2/2g$

Tube #	#7	#8	#9	#10	#11	#12
H. W.	7.45	7.17	7.04	7.35	7.00	6.79
1	4.51	4.92	5.21	3.45	3.73	4.80
2	4.40	4.31	5.82	2.63	3.04	4.34
3	5.23	5.13	6.78	4.14	4.29	5.02
4	5.72	5.79	6.37	5.83	5.40	5.69
5	3.67	3.91	5.66	2.34	3.06	4.02
5a	5.34	5.36	5.42	4.25	3.49	5.58
6	6.12	5.91	5.68	4.26	5.17	5.57
7	4.45	4.81	4.63	3.37	3.52	4.92
8	4.87	5.12	3.39	5.06	4.23	2.94
9	3.37	3.87	3.47	3.63	4.18	4.08
110	3.14	5.35	1.87	4.21	4.21	4.53
11	5.52	4.90	0.46	3.27	3.50	4.36
12	5.98	5.33	-0.12	1.00	3.40	4.90
T. W.	2.11	3.60	5.58	-1.07	1.08	3.67

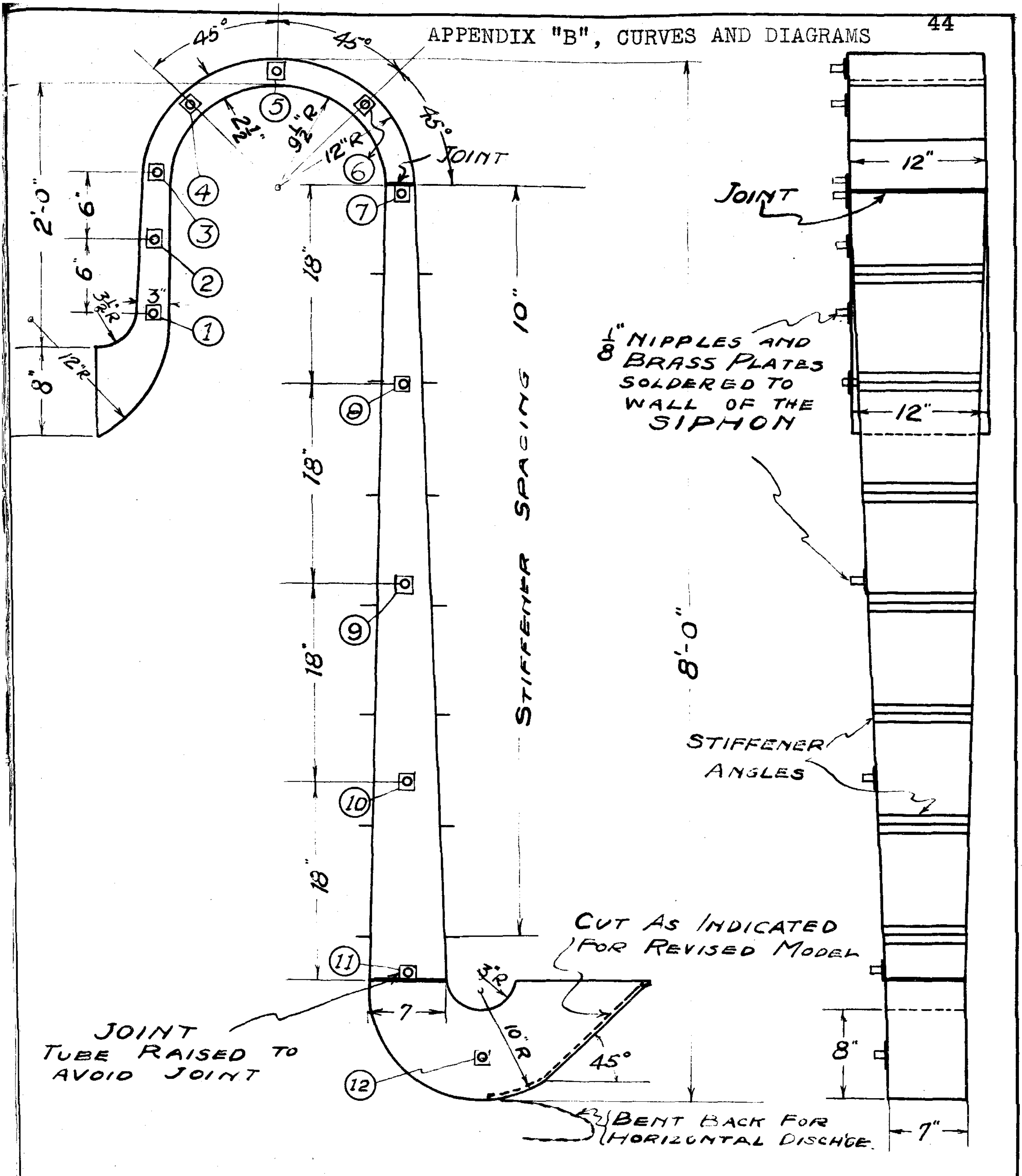
Note; Datum chosen, Tube 11, z = 0.00' Bernoulli's Equation...

TABULATED RESULTS Runs # 13 - 19

Revised Model. Horizontal Discharge.

Run #	#13	#14	#15	#16	#17	#18	#18
Data	May 7	May 7	May 7	May 7	May 7	May 7	May 7
Head	7.34	5.17	7.18	6.86	6.34	5.54	1.83
Disch'ge	3.31	2.95	3.27	3.27	3.24	3.20	1.71
Coeffic.	0.73	0.78	0.73	0.75	0.77	0.81	0.77
Disch in	Air	Water	Half way	Water	Water	Water	Water

Note; No pressure readings were taken on these runs as the primary object was to secure new values of K and to have these new values to plot on the curves.



MODEL SIPHON SPILLWAY

USED IN HYDRAULICS LABORATORY OF

California Institute of Technology - by

Gunner Grammatky   Kenneth Robinson

Sept. 1928   June 1929

Figure #1.

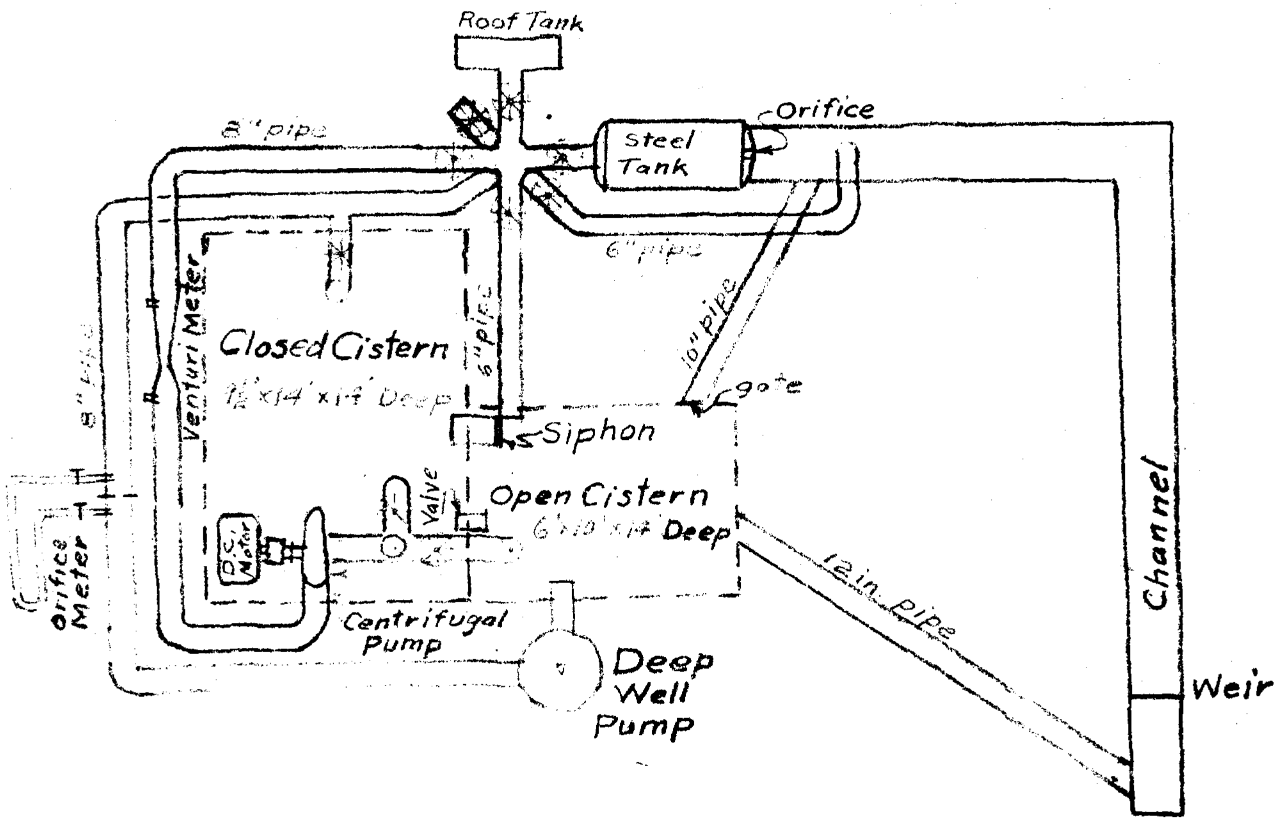
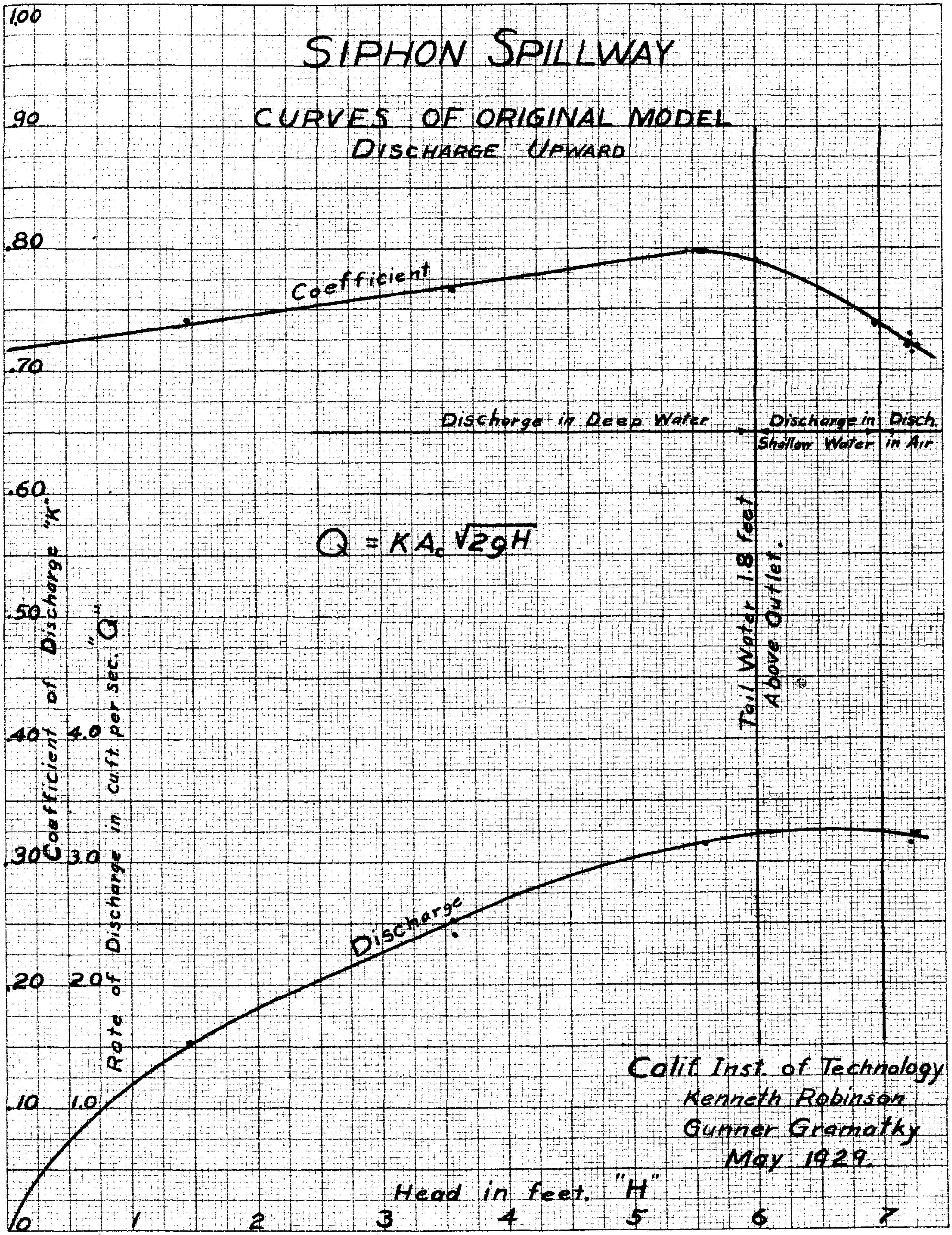


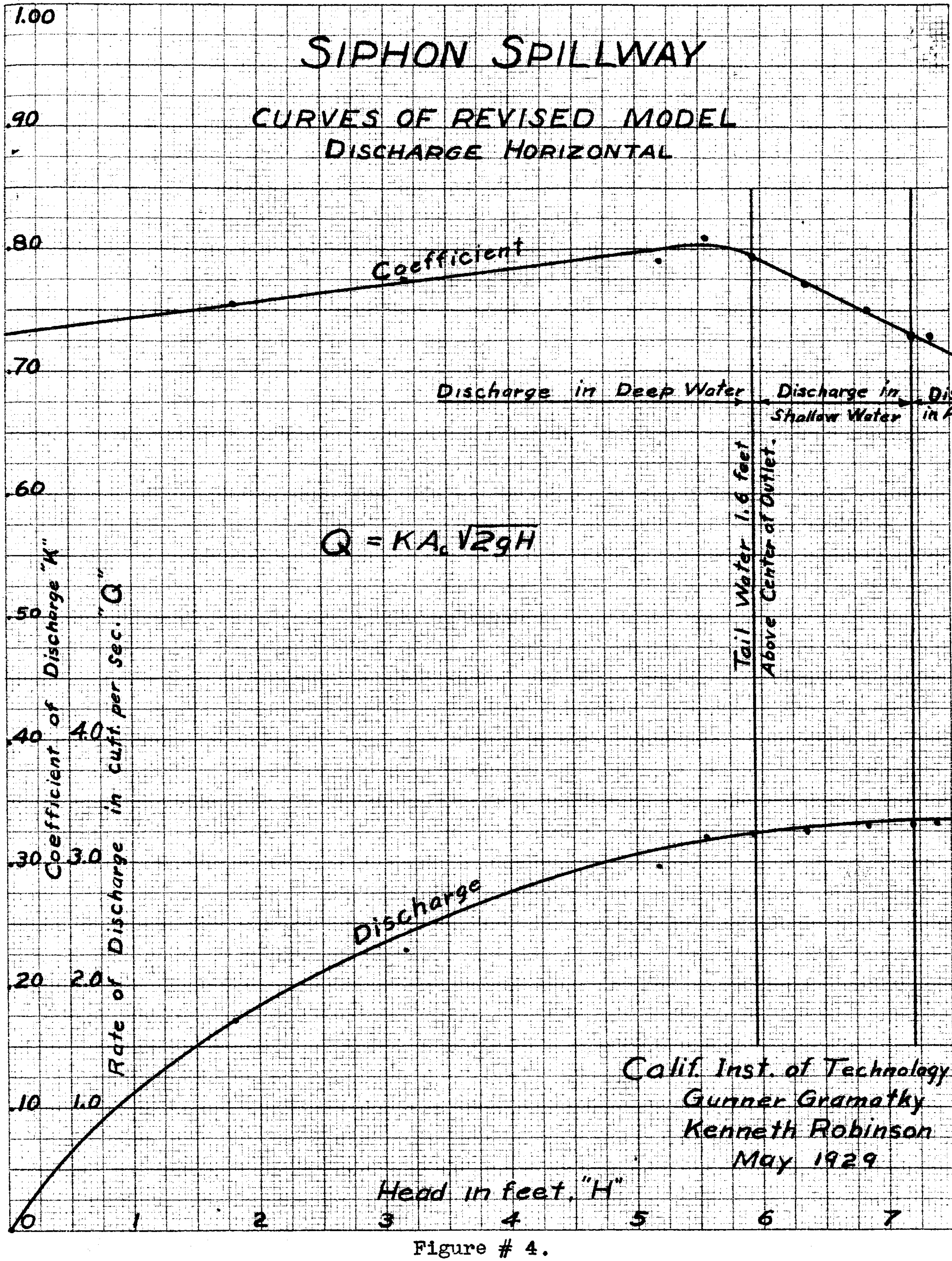
DIAGRAM OF HYDRAULIC LABORATORY

Fig. No. 2.



SIPHON SPILLWAY
CURVES OF ORIGINAL MODEL
DISCHARGE UPWARD
Coefficient
Discharge
Discharge in Deep Water
Discharge in Shallow Water
Disch. in Air
Q = KA_c √2gH
Tail Water 1.8 feet Above Outlet.
Coefficient of Discharge "K"
Rate of Discharge in cu.ft. per sec. "Q"
Head in feet. "H"
Calif. Inst. of Technology
Kenneth Robinson
Gunner Gramatky
May 1929.
1.00
.90
.80
.70
.60
.50
.40
.30
.20
.10
4.0
3.0
2.0
1.0
0
1
2
3
4
5
6
7

Figure # 3



SIPHON SPILLWAY
CURVES OF REVISED MODEL
DISCHARGE HORIZONTAL
Coefficient
Discharge in Deep Water
Discharge in Shallow Water
Disch in Air
Tail Water 1.6 feet Above Center of Outlet.
Q = KA$_c$ $\sqrt{2gH}$
Coefficient of Discharge "K"
Rate of Discharge in Cu.ft. per sec. "Q"
Discharge
Calif. Inst. of Technology
Gunner Gramatky
Kenneth Robinson
May 1929
Head in feet. "H"
1.00
.90
.80
.70
.60
.50
.40
.30
.20
.10
4.0
3.0
2.0
1.0
0
1
2
3
4
5
6
7

Figure # 4.

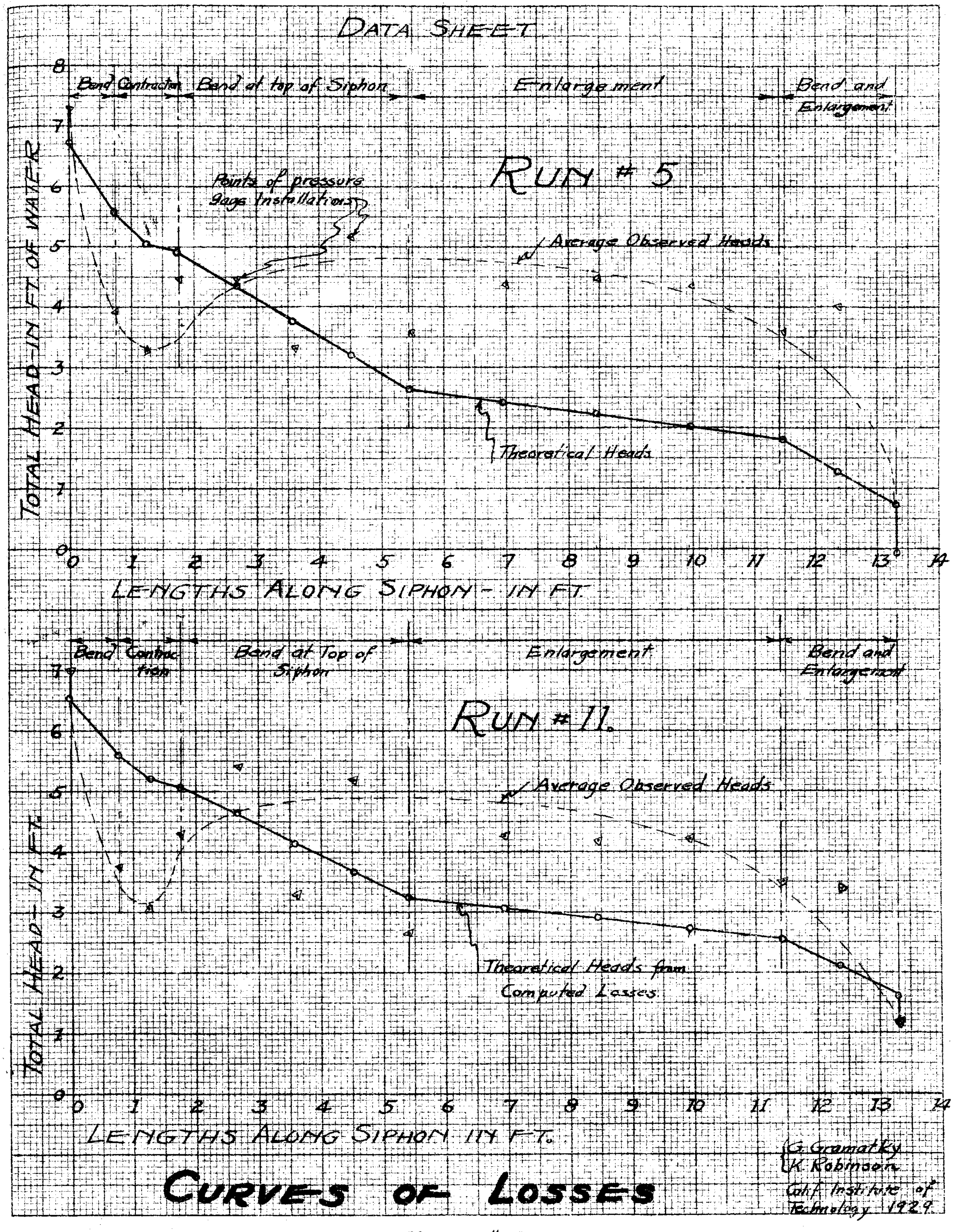
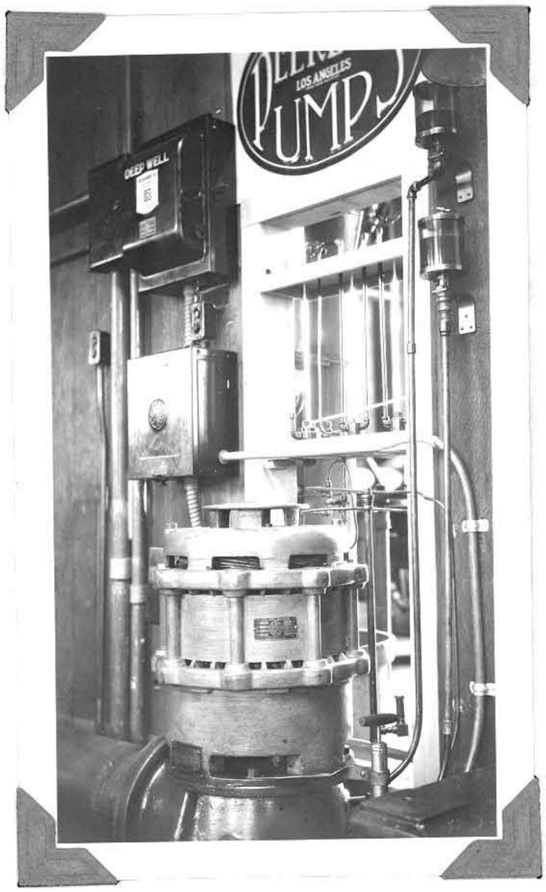

DATA SHEET
Bend Contraction
Bend at top of Siphon
Enlargement
Bend and Enlargement
RUN # 5
Points of pressure gage installations
Average Observed Heads
Theoretical Heads
TOTAL HEAD-IN FT. OF WATER
LENGTHS ALONG SIPHON - IN FT.
Bend
Contraction
Bend at Top of Siphon
Enlargement
Bend and Enlargement
RUN # 11.
Average Observed Heads
Theoretical Heads from Computed Losses
TOTAL HEAD- IN FT.
LENGTHS ALONG SIPHON IN FT.
G. Gramatky
K. Robinson
Calif. Institute of Technology 1929
CURVES OF LOSSES

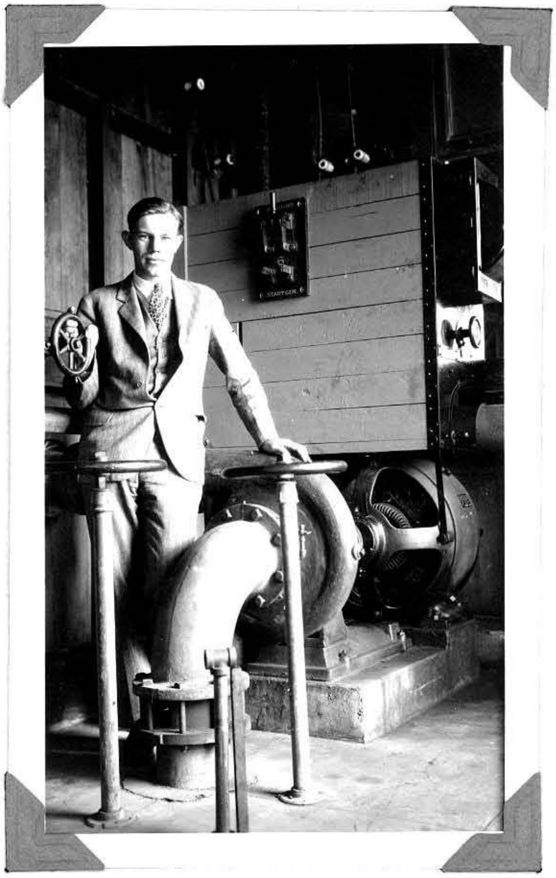
Figure # 5

SIPHON SPILLWAY



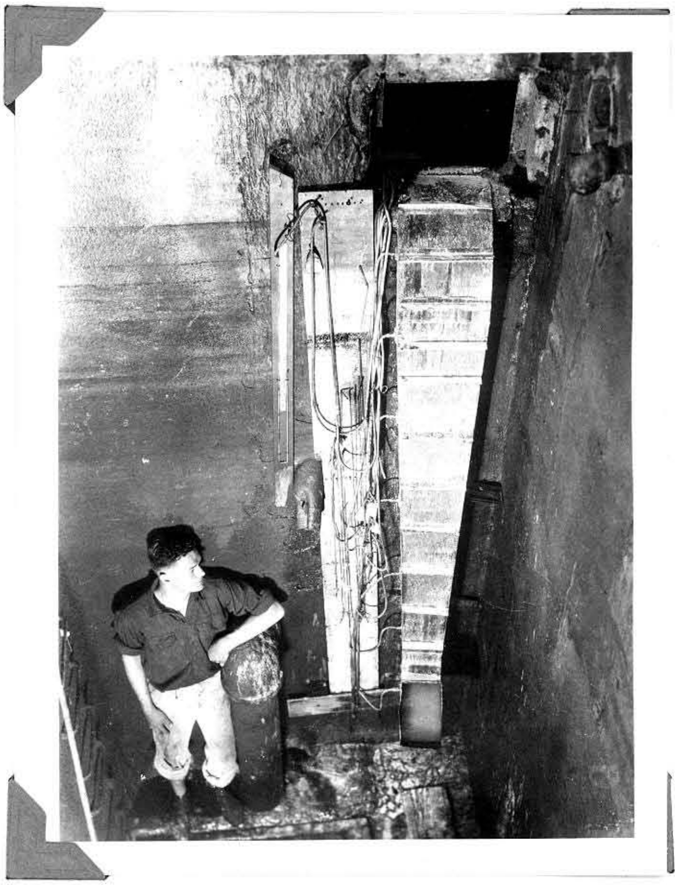
Photographof Peerless
Deep Well Pump, used
in experiment on the
spillway.

SIPHON SPILLWAY



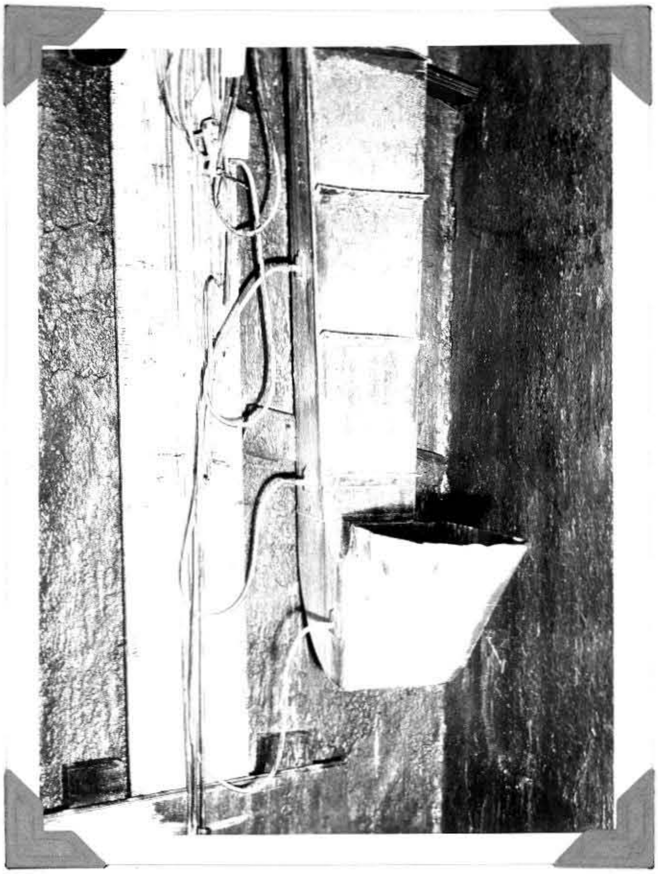
Centrifugal pump used as
the second means of getting
3.5 sec ft. Robinson, one of
the writers is seen here.

SIPHON SPILLWAY



Photograph of the outlet leg of
the original model before revision
showing pressure tubes and nipples.
Gramatky, one of the authors is
seen standing beside the model.

SIPHON SPILLWAY



Detail of discharge outlet of original model. The front face was cut out for revision so that horizontal discharge was possible.

SIPHON SPILLWAY

Original modēl discharging

upward. Q = 3.15 sec ft.

SIPHON SPILLWAY

Revised model in action with horizontal discharge. Q = 3.23 sec ft.

HYDRUALIC SIMILITUDE. Papers by KeNneth C Reynolds MIT
Alton C Chick Frm8ns A.

Notes by Kenneth Robinson, From..Freemans,"Hyd. Lab. Prac."

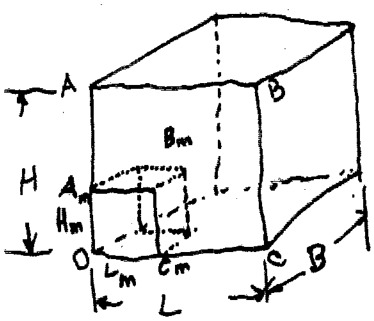
History and Men Making It.

Versatile Da Vinci began scientific stüdy of Hyd. 1500 19th Cent real progress, Darcy, Bazin Fargue, Dubuat French mostly. Bordeaux wanted to impbove waterway to the sea. Fargue made lab models and two yesrs studied. Scales assumed. 1885 Osborne Reynolds same prob at Liverpool Manchester Canal. He considered the time element. 1891 Germans got busy. Engels first man Dresden , a lab was built for River Hyd Exper. several more in short order.

Reasons for Study

In order to properly interpret the results on a small scale model and transfer these results to the full sized structure, the experimenter mus have a working knowledge of the laws of Hyd similarity, both as regards the construction of the model and as related to the flow in the model so that they are similar to the conditions occuring in the natural phemomena. Furthermore, he must know the limitations of experimental work and be able to properly evaluate the results of tests.

Simple cases



Take the cube for instance from which we can get relations of L, A, and Q.

Length

$\frac{L}{L_m} = n$ called the scale ratio or, scale of length.

$L;L_m = n$; 1**H ; $H_m = n$; 1 etc.

Area

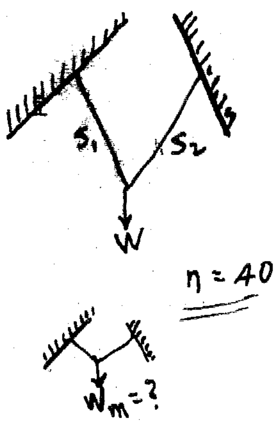
$$\frac{\text{Area} = (ABCD)\ H \times L}{\text{Area}_m = \quad H_m \times L_m} = \frac{H}{H_m} \times \frac{L}{L_m} = n^2 \ ; 1$$

Gemetrical Similarity

$A = A_m n^2$

$Q = Q_m n^3$ in a like manner, and so it merely becomes a metter of finding the dimensional relation between all of the quantities used in ordinary hyrdraulic practice as well as mechanics and physics and the like fields. Having explained the simplest kind of similarity let us turn to many of the common quantities to see how n varies with them and take a few illustrations.

Mechanical Similarity



In statics, a weight W of 128 kg is suspended as shown, If n = 40, det the weight W_m to be used for the k model. Case 1. Assume that it is desired to have $d = d_m$, and $E = E_m$.

$d = \frac{WL}{AE} \qquad d_m = \frac{W_m L_m}{A_m E}$ equating

$W_m = \frac{L}{L_m} \frac{A_m}{A} W = \frac{W}{n} = 3.2$ kg

Case 2. Assume $d = nd_m$
Then $W_m = \frac{W}{n^2} = 0.08$ kg

Case 3. $d = nd_m$, $E = n E_m$; $W_m = \frac{W}{n^3} = .002$ kg

Dynamic Stability $V = \sqrt{2gh}$, $V_m = \sqrt{2 g_m h_m}$ If experimaats are made at same elevation then g is the same and the eq

Velocity is $V; V_m = \sqrt{h}; \sqrt{h_m}$

$V = \sqrt{n}\, V_m$

This is called Froude's law and is applicable where only gravitational fprces are acting and the frictional one may be neglected.

Ex. A ship has a velocity of 20metres/sec. If N = 25 what is the veloc of the model $= \frac{20}{\sqrt{25}} = 4$ m/sec.

Length of ship = 120 meters what is L_m, $\frac{120}{25} = 4.8$ m.

Mass. $M = M_m n^3$ Model ship 100 gms what is M ship.
$M = 100 \times 40^3 = 6400$ kg.

Kinematic Similarity — In order for the model to be dynamically similar to the real, there must be a geometrical similarity at all times and with a corresponding scale of length. comstant.

Time — Time $T = \sqrt{n}\, T_m$ $h = 1/2\, gt^2$ relation.

Ex. If a certain q of water flows over a dam in 60 min how long will it take for a model n = 10.
$t_m = \frac{60}{\sqrt{10}} = 18.94$ min.

Acceleration — $A = A_m$ regardless of the scale ratio of modelto real.

Force. — $F = MA = F_m n^3$
Ex. If the force on a dam is 2,400,000# when overturning impends what is the force on a model if n = 20. $F_m = \frac{2.4 \times 10^6}{20^3} = 300\#$

Discharge/unit time Q Daugh. — $Q = A V = n^2 \sqrt{n}\, Q_m = n^{2.5} Q_m$.

Ex. Discharge over flodd spillway 4000 sec ft, what will be Q_m over a model of that spillway for a corresponding head over the crest? n = 20.
$Q_m = \frac{4000}{20 \times 20 \times \sqrt{20}} = 2.\ 23$ sec ft.

Power — P
$P = QH = n^2 \sqrt{n}\, n\, P_m = n^{3.5} P_m$

Ex. A model turbine develops 1.5 hp. How much power would a turbine 16 times as large develop, n = 16.
$P = 16^3 \times \sqrt{16} \times 1.5 = 24,556$.

Work — $W = Fs = n^3 \times n\, W_m = n^4\, W_m$.

Ex. 10,000 cu ft fall over a dam down 65' = 40.63×10^6 ft lb of work. Model dam n = 20 W_m 254 ft lb.

Energy KE Torque — same as work $E = n^4 E_m$
$M = n^4\, \text{Moment}_m$

Ex. A force of 350# acts on a ten ft lever arm. moment = 3500 ft/#. N = 10 m = .35 ft#

REYNOLDS LAW

Internal Friction   When the motions in the two systems are due to viscous resistances and the force of gravity does not appreciably affect the resistance, then another law applies called the Reynolds law. It is well known that liquids in certain states of motion will sustain shearing stresses, and for this reason are called viscous. The forces which tend to cause a gen motion of the liquid whether linear revolving, unif expanding unif contracting. involve no shearing stresses. Forces tending to change shape of body cause shearing stresses. Distortion expresses change in shape. Reynolds defines viscosity as the " shearing stress caused in the liquid while undergoing distortion, and the shearing stress divided by the rate of distortion is called the coeff of viscosity or commonly, the viscosity."

It is found that $n = \left(\frac{v}{v_m}\right)^{2/3}$ so that if complete similarity is wanted n should be chosen for this condition. Further it is found that

$$\frac{VL}{v} = \frac{V_m L_m}{v_m}$$

Where V and V_m are corr. velocities, L and L_m are corres. lengths

Pg 810   In other words the two systems, model and full size in which only the forces of viscosity are considered as being present and acting, and where the forces of gravity do not appreciably affect the resistance, are dynamically similar if corresponding velocities are directly proportional to the kinematic viscosities and inversely proportional to the linear dimensions.

Reynolds was the first to make use of this ratio and to determine values for it. It is a dimensionless quantity, It now bears the name Reynolds number.
Value is this--pg 815 table which allows the friction factor for pipes to be expressed in terms of the Reynolds no. $= \left(\frac{Vd}{v}\right)$. Hence with this one set of values you can get curves for oil, molasses or anything you want, thu pipes. Then other tables are made giving

Pg. 816   all these values for Kutters Curves and open channel Flows of f . Thus with the laws of similitude one can predict nearly exactly the relations of one smooth pipe to another. Roughness is uncertain and in some cases even with geom similarity and same Reynolds number the effect of roughness overshadows that of the factors in the Reynolds number. Non uniform flow it is impossible to obtain dynamical similarity.

Interesting table is developed... from the fact that it is found that

$$N_m = \frac{1}{\sqrt[6]{n}} N$$

showing that the wetted surfaces in the model must be smoother than the corresponding surfaces in the full sized structure when water is used. Varies quite a bit as the size of the model indicates.