HEADWALL ANGLE EFFECT
ON
BOX INLET DISCHARGE

By

GILBERT ARTHUR EKERDT

A Thesis Submitted in Partial Fulfillment of
the Requirements for the Degree of

MASTER OF SCIENCE
(Civil Engineering)
at the
UNIVERSITY OF WISCONSIN

1951

CONTENTS

Appendixes

SYNOPSIS

This thesis is the report of an investigation to determine the effect the upstream wingwall position has on the discharge characteristics of the rectangular spillway when used on a "box inlet culvert".

Tests were conducted over a wide range of model size ratios with wingwall positions ranging from 45 to 90-deg. from the longitudinal axis of the conduit. The relationship was determined for both free flow and submerged flow conditions at the spillway section.

The test results indicate that the position of the wingwall does affect the discharge characteristics for heads below the maximum for which the spillway crest functions as a long weir. When the spillway crest is submerged the discharge characteristics are independent of the wingwall position.

Design recommendations based on the model studies are presented.

INTRODUCTION

The purpose of this thesis has been to investigate the effect of the position of the upstream wingwall on the discharge capacity of the rectangular spillway when used as a box inlet discharging through a closed top outlet. The common name for this type of structure is the "box inlet culvert".

The rectangular spillway is a structure that is being used extensively for erosion control work, particularly where large quantities of water are involved with relatively small heads available. The rectangular spillway is a structure consisting of three weirs at right angles to each other, forming three sides of a rectangular box. The fourth side of the box is open below the weir-crest elevation and with a bulkhead above the crest elevation extending to each side of the channel. Defining the terms as used in this thesis, that portion of the bulkhead over the outlet end of the box will be referred to as the headwall and that portion extending beyond the sides of the box will be called the wingwalls. The discharge over the three weirs drops into the box and passes out the open end.

The open end consists of a closed rectangular conduit or culvert to carry the water beneath a fill or roadbed and to discharge it at the elevation of the bottom of the conduit. The controlled head drop is thus equal to the difference between the spillway crest and the outlet floor elevations.

Considerable work has already been done on determining the hydraulic characteristics of the rectangular spillway when used as a box inlet or as a U-type entrance for concrete flumes. A limited number of experiments were made by Kessler at the University of Wisconsin 1/ in 1933 on "Head Spillways". A more extensive investigation of the rectangular spillway was carried out by Huff for the United States Soil Conservation Service at the St. Anthony Falls Hydraulic Laboratory 2/ and these results published in 1944. The most recent investigation reported 3/ was also carried out under the research program of the U. S. Soil Conservation Service at the St. Anthony Falls Hydraulic Laboratory.

All of the experiments to date have been limited to conditions in which the bulkhead extended above and to either side of the channel and was normal to the axis of the conduit.

The University of Wisconsin tests were made with a closed top outlet and 90-deg. wingwalls. The tests by Huff 2/ were made with both closed top and open top outlets and 90-deg. wingwalls. The St. Anthony Falls tests 3/ were made with open top outlets. The report by Huff 2/ has been the criteria for the design of all rectangular spillways with closed top outlets built by the U. S. Soil Conservation Service.

It is becoming increasingly apparent to design engineers that in many instances it is ecomonical to use wingwall angles other than 90-deg. Frequently it has been necessary to attach rectangular spillways to existing road culverts that already have been constructed with wingwalls varying from 45 to 90-deg. with the longitudinal axis of the culvert. Numerous cases have also arisen wherein the use of wingwall angle of less than 90-deg. would have been desirable for new construction work. Inasmuch as no published data were available to determine the wingwall effect, the design engineer has been uncertain as to the design coefficients that apply. As a result spillways, when attached to existing culverts, are overdesigned to be on the safe side and use of wingwall angles of less than 90-deg. is avoided in new construction.

The study of submergence of the weir crest caused by a rise in the downstream tailwater level is of little value when the rectangular spillway discharges into a closed conduit. Structures of this type usually carry large flows beneath roadways as compared to the box inlet drop spillway which has an open top outlet and frequently acts as an "island type dam" 3/. Inasmuch as submergence causes a rise in the headwater level, which in the case of a roadway would threaten to overflow the road and cause an added flood hazard, the designer should take all possible precautions to assure that the hydraulic design of the structure and channels will be such as to prevent conditions of submergence.

The extent of the submergence studies for this thesis will be to ascertain what effect, if any, the wingwall position would have were submergence to occur. No attempt will be made to predict the overall effects of submergence because of its relative unimportance to this type of structure.

In field installations the length of wingwall, whether in place or to be constructed, is dependent upon the size of the angle and the fill slope required. For this reason a study of the effect of the wingwall length on the discharge capacity was included in the project.

To answer a definite need for hydraulic designers engaged in erosion control work, this investigation was undertaken for the purpose of obtaining a set of design criteria that would be applicable to structures having different wingwall positions.

NOMENCLATURE

W = Inside width of rectangular box at crest elevation in feet.

B = Length of spillway crest from upstream face of headwall to the upstream weir face in feet.

D = Vertical drop from spillway crest to floor of spillway in feet.

L = Total inside length of spillway crest ($2B - W$) in feet.

h = Depth of water above the spillway crest as measured in the laboratory experiments in feet.

h_v = Velocity head in approach channel at head measuring section in feet.

H = Total head on spillway ($h - h_v$) in feet.

g = Acceleration of gravity (32.2 ft./sec^2)

Q = Discharge in cubic feet per second.

θ = Angular position of the wingwall measured radially outward from the plane of the "B" dimension.

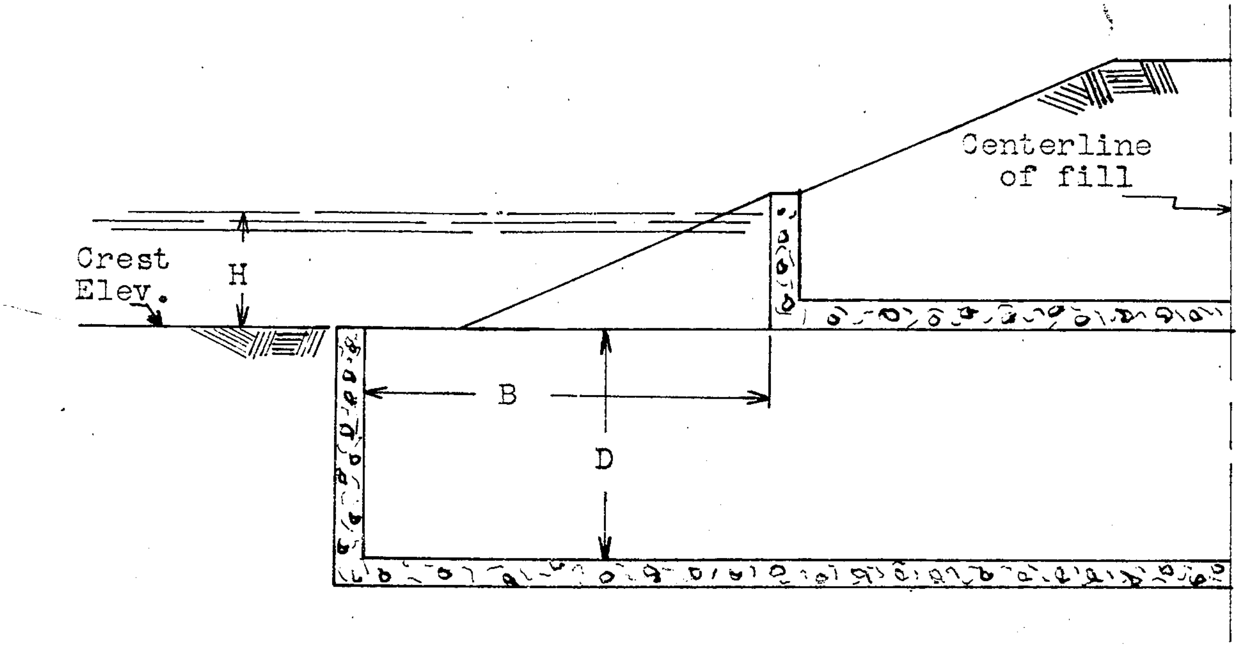
C_1 = Coefficient in the formula $Q = C_1 L H^{1.60}$

C = Coefficient in the formula $Q = C L H^{1.50}$

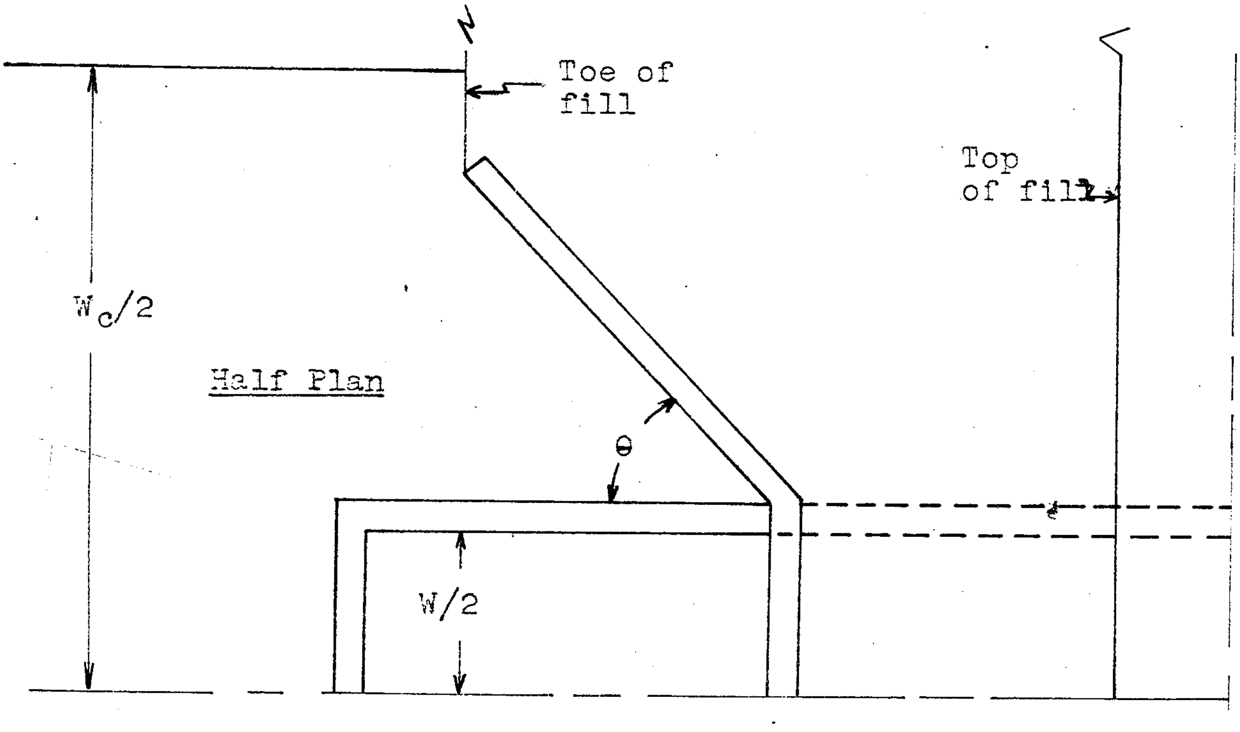
W_c = Width of channel approaching spillway in feet.

H_1 = Tailwater elevation with respect to spillway crest.

Figure 1.



Cross Section at Centerline



Half Plan

BOX INLET CULVERT

THEORETICAL CONSIDERATIONS

Hydraulic model studies have already proven to be the most accurate means of predicting the hydraulic design characteristics of the prototype. This model prototype relationship is based on the principal of "hydraulic similitude". The analysis of the basic relationships of the various physical properties involved in the motion and in the dynamic action of the fluid is called "dimensional analysis".

Geometrical similarity exists between two objects if the ratios of all corresponding linear dimensions are equal. This relationship is independent of any kind of motion and involves only similarity in form. Inasmuch as practical reasons limit the number of model tests that could be made, it was necessary to conduct these tests so that the results determined would be applicable over a large range of prototypes. The ratio of homologous lengths in model and prototype is expressed as

$$L_m / L_p = L_r \qquad (1)$$

where L_m = length of model, L_p = prototype length and L_r = length ratio.

The tests were conducted with certain relationships existing between the W, B, and D dimensions of the model and consequently these same relationships as modified by the length ratio (L_r) will exist on the prototype. The head causing discharge through the weir is also a linear dimension and can be expressed in the form of equation 1.

The discharge through the structure involves similarity of motion or "kinematic similarity". By the incorporation of Froude's law, the ratio of homologous discharge in model and prototype is expressed as

$$Q_m / Q_p = (L_r)^{5/2} \qquad (2)$$

where Q_m = model discharge and Q_p = prototype discharge. Equation 2 is the dimensionless expression of the weir formula

$$Q = C\,L\,H^{1.50} \qquad (3)$$

Since the discharge ratio is directly a function of the linear dimensions involved, the discharge coefficient (C) is identical for both model and prototype, provided the discharge equation for both is as given in equation 3.

The principal of dynamic similarity will be neglected in this analysis inasmuch as the friction losses in the size and shapes of models chosen can be assumed to be very small and it is reasonable to assume that these losses will also be small in the prototype. Because the fluid flowing is water and flow is turbulent, in both model and prototype it may be assumed that the Froude law holds true.

Comparing the discharge characteristics of a certain size spillway operated with various angles of wingwall, with the characteristics for a wingwall position normal to the axis of the conduit, the effect of the wingwall positions can be predicted for the prototype. The effects would be similar for both model and prototype.

The manner in which the principal of similitude is applied will be illustrated in the section of this thesis describing the method of analysis.

DESCRIPTION OF APPARATUS

All of the models used for making the tests of the rectangular spillway were built of lucite. Actually only one model was constructed but the design was such that parts could be removed or added as desired to give different values of B, D, and θ in relation to the fixed width W. (Figure 1.)

A constant rate of flow was obtained by taking water from the 220,000-gallon, 60-ft. constant head tank adjacent to the laboratory. Free flow from the tank was controlled by the gate valves in the lines leading to the 4" x 2" and 8" x 4" calibrated Venturi meters used for measuring the discharge.

The size of model that could be investigated was limited by two conditions; first, the range of discharge over which an accurate flow measurement was attainable, and second, the maximum width of approach channel that could be constructed in the concrete tank assigned to the experiment.

An approach channel of five feet in width was selected. Following the recommendations of the St. Anthony Falls report 2/ the ratio of the channel width, W_c, to the length, L, of the box should be a minimum of 1.5 when the B dimension of the box is equal to 2W. The width of box, W, was set equal to 8 in. and B, when equal to 2W, is 16 in. resulting in a total length, L, of 40 in. A 60 in. approach channel width, W_c, would satisfy the above criteria. The recommended W_c / L ratio for B = W is given as 2.5 which again is fully satisfied when W = 8 in. and W_c = 60 in. The same agreement with the design recommendation exists when B = 0.5W for W = 8 in.

To assure that adequate data would be obtained for predicting the wingwall effect over a wide range of dimensions, three ratios of width of spillway (W) to length of spillway (B) were selected so that B = 0.5W, B = W, and B = 2W. For each of these lengths three depths (D) of spillway were selected so that W = D, W = 2D and W = 4D. A hinged arrangement was used for attaching the wingwalls so that any desired angle (θ) could be used in connection with any ratio of B, W and D.

The width W equal to 8 in. remained constant for all tests. The (B) values used were 4 in., 8 in. and 16 in. When necessary, (D) values for each of these three lengths were 8 in., 4 in. and 2 in. Preliminary calculations indicated that models constructed with these dimensions would have discharge values within the calibrated range of the Venturi Meters, already installed in the flume. The actual tests proved the adequacy of the two meters.

The 5-ft. wide approach channel was filled with sand to crest elevation over the entire area upstream from the wingwalls. Downstream from the wingwalls to the channel bulkhead, the sand was banked to simulate fill conditions. In order to prevent erosion, adjacent to the weir, a sandsurfaced plaster mat ¼-in. to ½-in.

thick was extended out at crest elevation around the three sides of the box. This precaution was taken to assure that whatever discharge relationships existed between the boxes would be a function of the selected box dimensions and would not be affected by different amounts of erosion.

Corrugated metal baffles were placed 5 ft. upstream from the upstream edge of the spillway crest. The use of these baffles resulted in a smooth water surface upstream from the spillway crest.

The observed depth of water (h) was determined by the use of a point gage installed 28 inches upstream from the upstream edge of the spillway crest. This distance was selected after several trials had determined that depths recorded there would not be affected by the drawdown profile of the spillway. At this point the slight turbulence caused by the baffles was no longer discernable. The total head (H) at this point is the sum of the observed depth and the velocity head.

For the purpose of studying the effect of the wingwall position for condition of submergence, the outflow conduit discharged under free-flow conditions into a 5-ft flume separated from the 5-ft test channel by a watertight bulkhead downstream from the wingwalls. A hinged elevating gate was placed in the outlet section of the downstream flume for regulating the tailwater level. The elevation of the water surface above spillway crest elevation in this flume was measured with a point gage.

The zero reading for both the headwater and tailwater point gages with reference to the spillway crest elevation was determined with a surveyors level and rod. The entire model was rigidly attached to the channel floor so that uplift forces would not affect the crest elevation. The zero readings were checked at intervals throughout the testing period.

Figures 2 through 4 are photographs of the model.

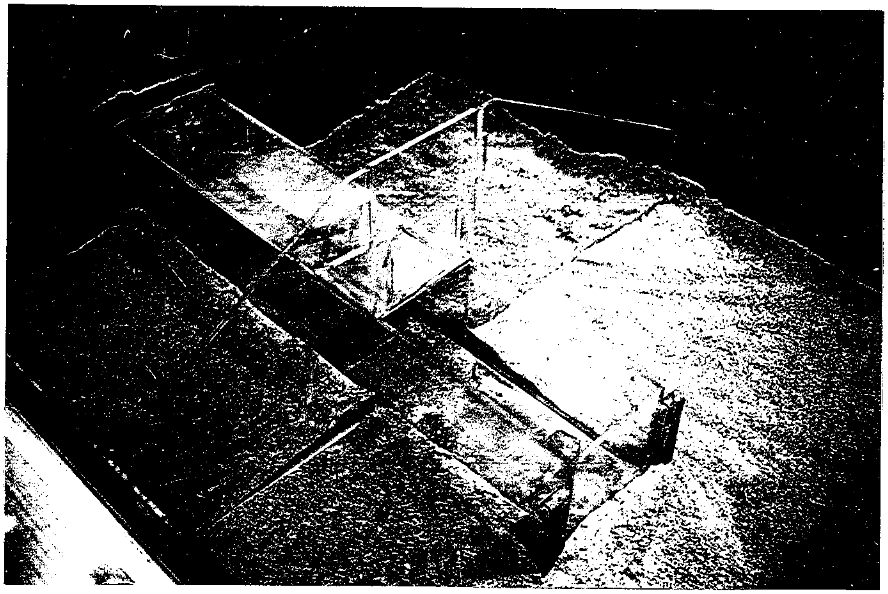


Fig. 2. Model prior to placing sand fill.

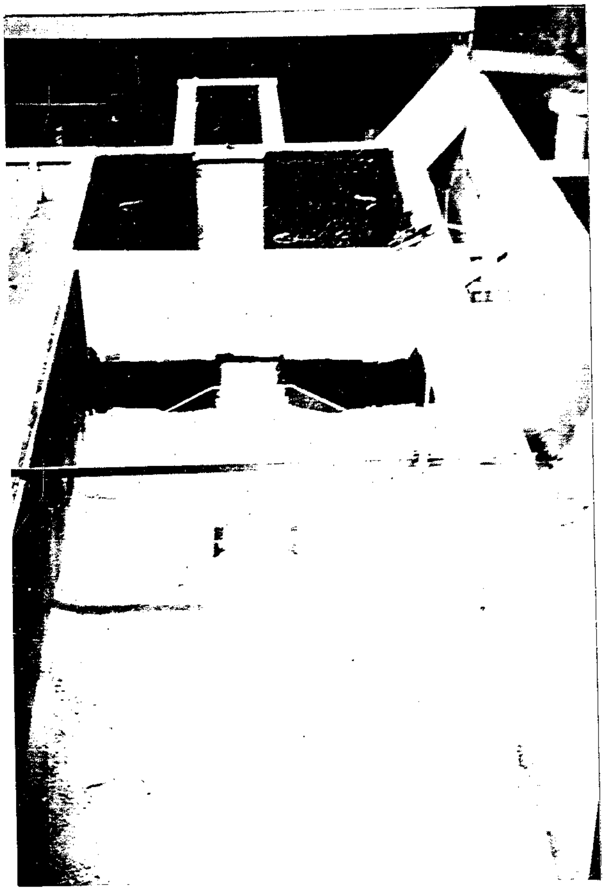


Fig. 3. Test channel looking downstream

Fig. 4. Model in operation. (B = W) ($\theta = 45^\circ$)

LABORATORY TECHNIQUES

Because the Venturi meters were upstream from the test section, it was necessary to make certain that all water leaving these meters would be discharged out of the test section over the rectangular spillway. The degree of watertightness was tested by filling the test channel with water prior to placing the model. The test channel was considered to be sufficiently watertight after a 26-hr. test showed an elevation drop of 0.011 feet. With the model in place, observations failed to indicate any leakage around the model. The actual testing proceeded in the sequence listed in Table No. 1.

After model changes had been made, a plaster mat was formed around the weir sides and allowed to harden for 24-hrs. The approach channel was then leveled to crest elevation and the sand downstream from the wing-wall was sloped up towards the channel bulkhead to simulate an actual fill condition.

Water was then allowed to enter the section at a rate just sufficient to cause flow over the spillway for a minimum period of ten minutes. This procedure was adopted to assure complete saturation of the sand bed of the channel.

The series of test runs began after this ten minute period. The initial discharge (Q) and water depth (h) readings were taken. Then the flow was gradually increased and for each increase in discharge, Q, a new

TABLE No. 1.

Model Sizes Tested

Series of Runs No.	B Inches	D Inches	θ Degrees	L Feet
1	16	8	90	3.33
2	16	4	90	3.33
3	16	2	90	3.33
4	16	2	45	3.33
5	16	4	45	3.33
6	16	8	45	3.33
7	16	8	67½	3.33
8	16	2	67½	3.33
23	16	8	52½	3.33
22	16	8	60	3.33
11	8	8	90	2.00
10	8	4	90	2.00
9	8	2	90	2.00
13	8	8	45	2.00
25	8	8	52½	2.00
24	8	8	60	2.00
14	4	8	90	1.33
15	4	2	90	1.33
16	4	4	90	1.33
19	4	8	45	1.33
17	4	4	45	1.33
20	4	8	60	1.33
21	4	8	75	1.33
26	8	8	90	2.00
27	8	8	45	2.00
28	8	8	67½	2.00
29	8	8	90	2.00
30	8	8	45	2.00
31	8	8	67½	2.00
32	8	8	90	2.00
33	8	8	45	2.00
34	8	8	67½	2.00
35	8	8	45	2.00
36	8	8	45	2.00

W = 8 in. for all series tested.

Numbers 26 through 34 submergence data.

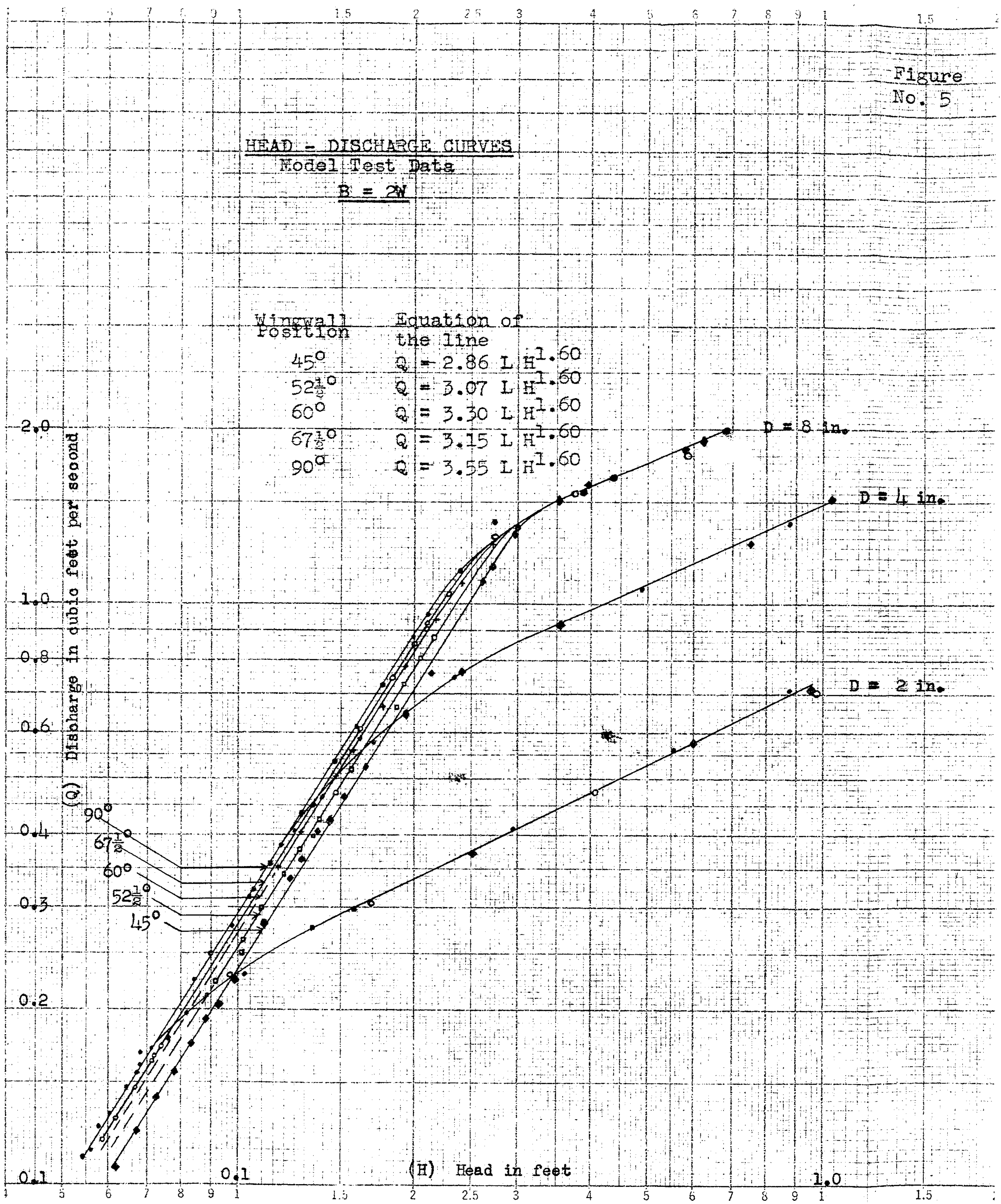
Numbers 35 and 36 wingwall length data.

value of h was recorded. For each new value of Q it was found necessary to wait several minutes until the head on the spillway became stabilized before determining h.

The curves showing the relationship between Q and H, Figures 5 to 7 inclusive, show a definite break in the head-discharge curve. This represents the point at which the spillway crest is flooded from the effect of the outlet depth at the bulkhead. Another way of stating this is that the box inlet weir crest is the control section at the lower flows while at higher flows the control section shifts to the headwall. Many attempts were made to determine the exact Q and H values at which this break would occur on the different models. This work met with no success so it was decided to space the runs closer when it was evident that this flooding was imminent. The result was that sufficient data was then available to plot the transition curve between the two conditions of free and submerged flow. Beyond the transition region the time required for the head to stabilize increased considerably over that under free flow conditions. The flow for each model was continued beyond the transition until the head was a minimum of twice that recorded immediately prior to the beginning of the transition.

This procedure was repeated for each test with the exception that when only the depths of the models were changed it was not necessary to replaster. The velocity

Figure No. 5



HEAD - DISCHARGE CURVES
Model Test Data
B = 2W

Wingwall Position	Equation of the line
45°	$Q = 2.86\ L\ H^{1.60}$
52½°	$Q = 3.07\ L\ H^{1.60}$
60°	$Q = 3.30\ L\ H^{1.60}$
67½°	$Q = 3.15\ L\ H^{1.60}$
90°	$Q = 3.55\ L\ H^{1.60}$

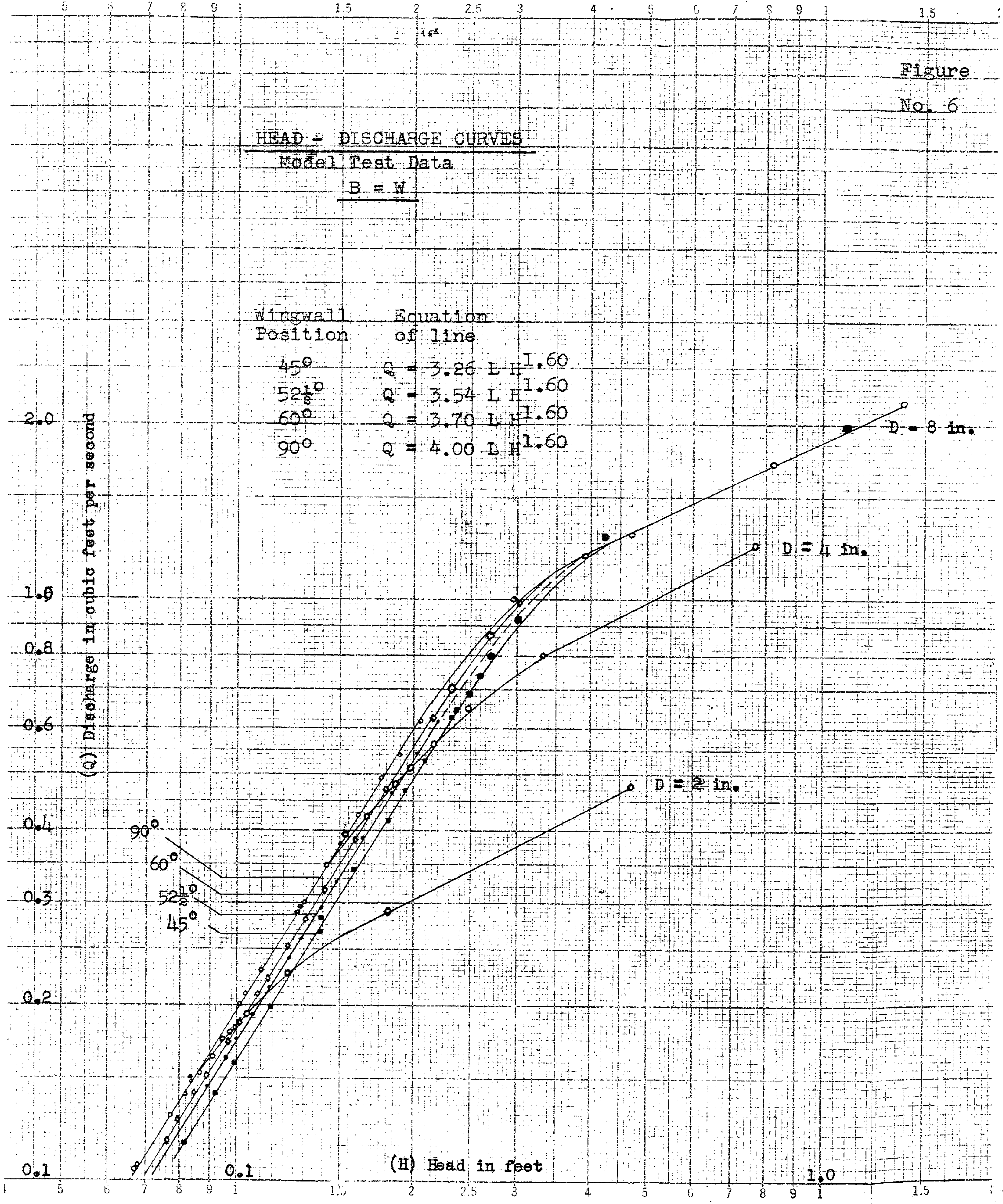


Figure No. 6

HEAD - DISCHARGE CURVES

Model Test Data

B = W

Wingwall Position	Equation of line
45°	$Q = 3.26\ L\ H^{1.60}$
52½°	$Q = 3.54\ L\ H^{1.60}$
60°	$Q = 3.70\ L\ H^{1.60}$
90°	$Q = 4.00\ L\ H^{1.60}$

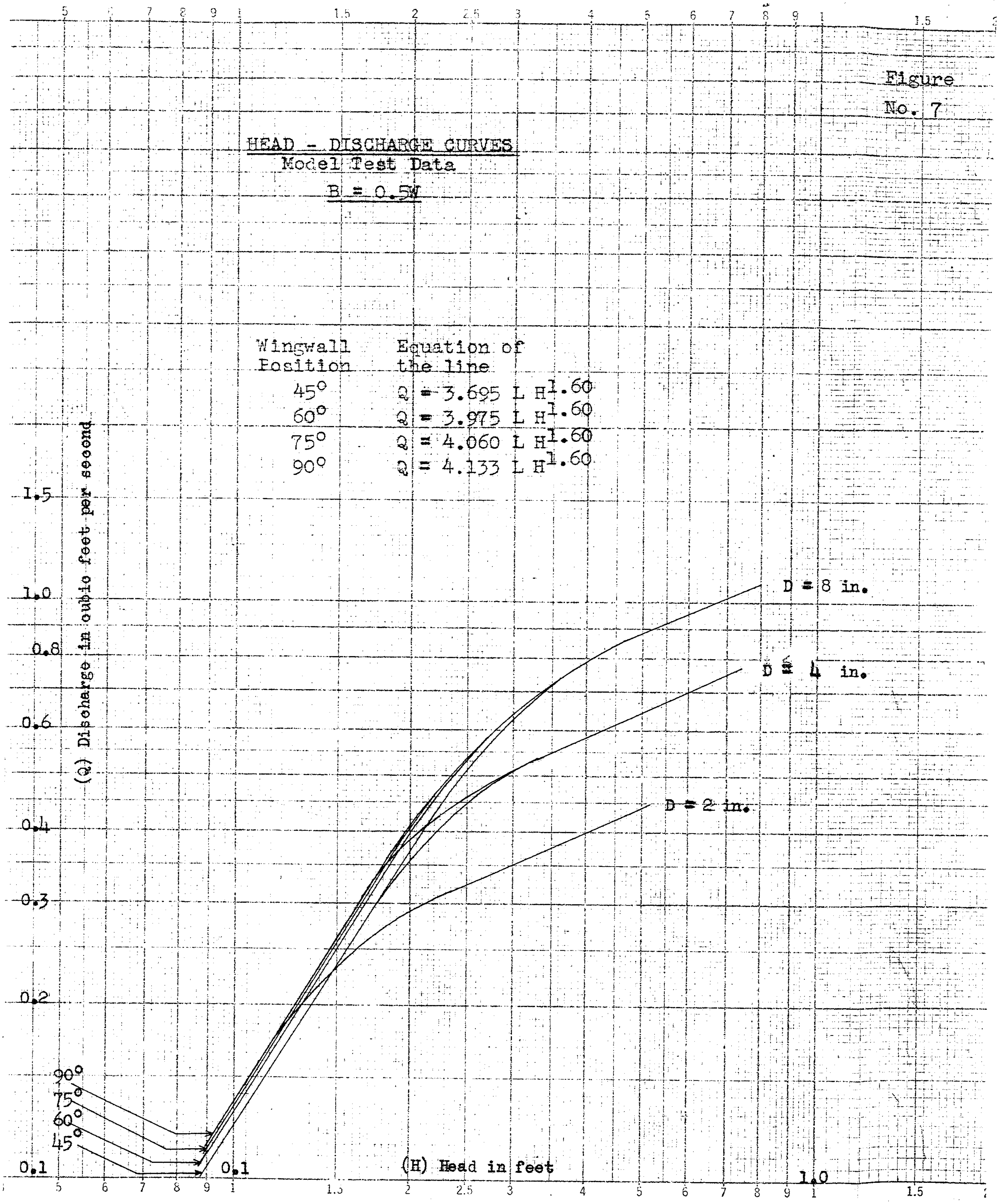


Figure No. 7

HEAD - DISCHARGE CURVES

Model Test Data

B = 0.5W

Wingwall Position	Equation of the line
45°	$Q = 3.695\ L\ H^{1.60}$
60°	$Q = 3.975\ L\ H^{1.60}$
75°	$Q = 4.060\ L\ H^{1.60}$
90°	$Q = 4.133\ L\ H^{1.60}$

head for each run was computed and added to the observed depth to get the total head H. The velocity head was determined using the depth and width of channel flow at the head measuring gage. Each series of runs were plotted on field curves so that it was possible to observe the nature of the results being obtained and to be able to select the dimensions for the next series. This manner of selective determination of what tests to make was very valuable in that no time was taken up in obtaining duplicate or unneccessary data.

The submergence tests were conducted on only one size of model in which W, B, and D were all equal to 8 in. The wingwall angle (θ) was the only variable dimension. A constant discharge, Q, was set to flow through the spillway and the free-flow head, h, for this discharge was determined. The level of the water in the downstream channel was gradually increased in steps. Upstream and downstream water depths were recorded for each step. For each angle of wingwall tested these depths were recorded for three different discharges.

Upon completion of the first series of runs for the first wingwall angle investigated,the ratio of the free-flow head,H, to the length of box, L, was computed. For the remainder of the angles tested the fixed discharge was adjusted so nearly similar H/L values would exist for the three flows tested at each position of wingwall. This procedure was followed to simplify the determination of the effect of the wingwall position upon the discharge under submerged conditions.

The final laboratory tests were concerned with the determination of the effect of length of wingwall with all other variables constant. For this investigation B, W, and D were set equal to 8 in. and the wingwall angle, θ, at a 45 deg. angle. The base length of wingwall at crest level was reduced to 2/3 and then to 1/3 of the original length used in all preceeding tests. Tests were run through the full range of heads.

METHOD OF ANALYSIS

The first step in the method of analyzing the data was to plot on log-log paper the Q versus H values for each run. The reasoning accompanying this is as follows:

The discharge is a function of the head, that is

$Q = f(H)$ or $Q = K H^n$

The plot of this equation on log-log paper presents a graphic solution for the unknown values "K" and "n" for each series of runs. The curves for each size of box, identical B/W ratios, tested were plotted on the same sheet to illustrate the change in discharge characteristics as a result of varying the depth of box (D) and the wingwall angle θ. The curve sheets for the different B/W ratios of 2.0, 1.0 and 0.5 are Figures 5, 6 and 7 respectively. It will be noticed that a spillway having a given length and width functions as a weir until a head is reached where the spillway crest is flooded out from the effect of the outlet depth at the headwall. This is the point on the curve where the head-discharge curve breaks away on a different slope. It will also be noticed that the head at which the spillway begins to be flooded out is much higher for the deep outlet than for the shallow outlet.

Inasmuch as three different sizes of models were tested, no direct comparison can be made in the discharge capacity for identical heads on the spillway for all tests.

However, since the same width and length of model were used for different outlet depths and wingwall angles, the comparative effect on the discharge capacity due to the spillway depth and the wingwall position is apparent.

The logarithmic plots of all tests had an exponent of 1.60 for the total head, H, for low heads when the weir crest was the control section. The coefficient, C_1, in the weir formula $Q = C_1LH^{1.60}$ for each group of tests having similar length to width ratios but different wingwall angles indicates, in general, the effect of the wingwall angle on the discharge capacity. The coefficient, C_1, is also a function of the length-width ratio on the discharge capacity for the groups of tests involving different length-width ratios.

In addition to determining the value of the discharge coefficient, C_1, by graphical means a numerical analysis was also made. The procedure followed was:

$$Q = C_1L\,H^{1.60} \quad \text{or} \quad C_1 = Q/\,L\,H^{1.60}$$

For each run in a given series, C_1, was computed using the known values of Q, L and H. An observation of the C_1 values indicated which of the runs were obviously in error and these runs were not averaged in to find the mean C_1 value for the series. In all cases close agreement existed between the graphical and analytical C_1 values.

The results of the free flow tests for C_1 in the formula $Q = C_1 L H^{1.60}$ are given in Table No. 2 and plotted on Figure 8 . The coefficient of discharge for weir flow conditions is independent of the depth. This is proven by virtue of the fact that the free flow coefficient for a given length-width ratio was the same for each value of depth tested regardless of the chosen length-width ratio or the wingwall angle.

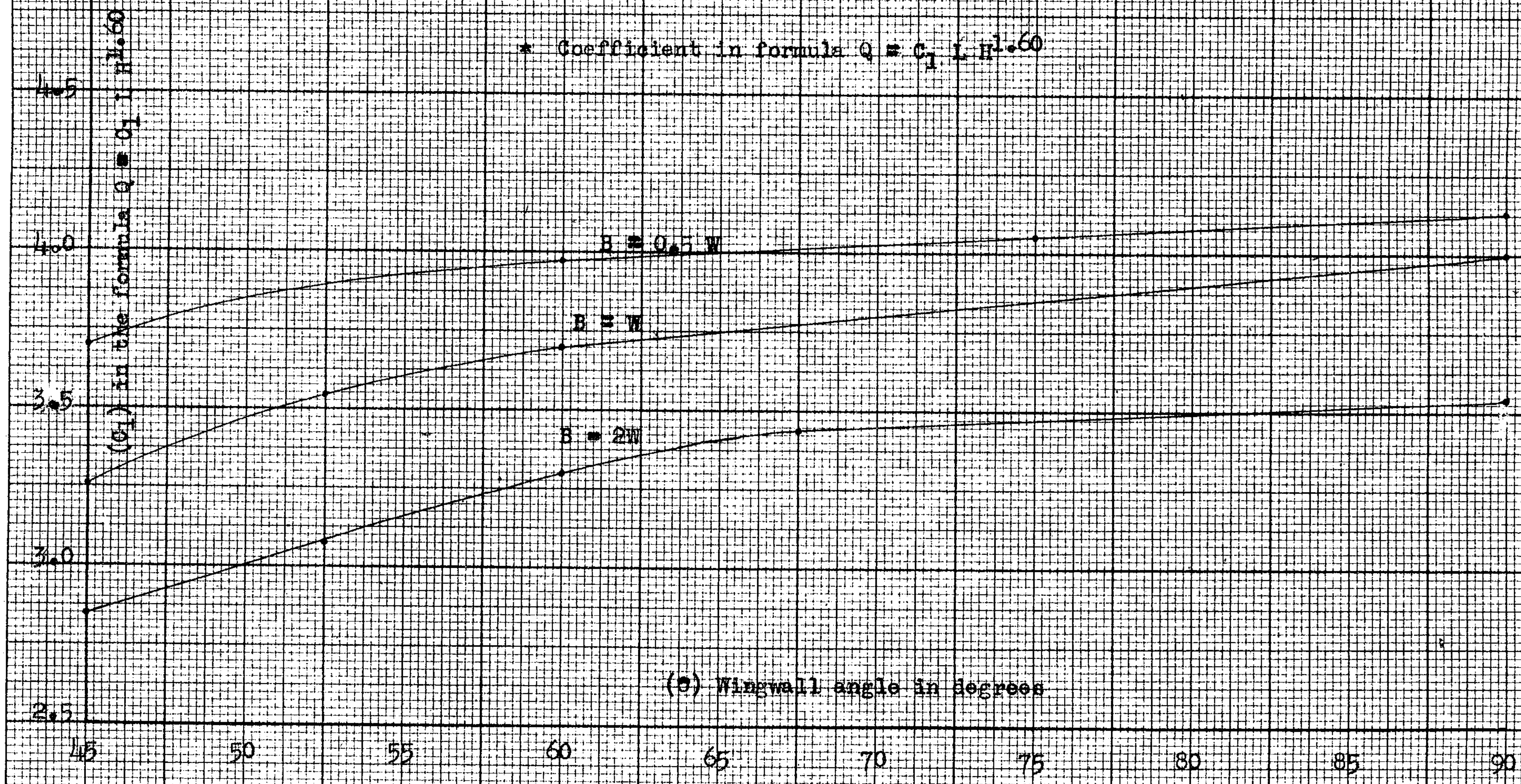
TABLE NO. 2

Test values of the coefficient C_1
in the formula $Q = C_1 L H^{1.60}$.
Weir crest not submerged.

Wingwall Angle θ (degrees)	B = 0.5W W = 8 in. B = 4 in.	B = W W = 8 in. B = 8 in.	B = 2W W = 8 in. B = 16 in.
45	3.695	3.260	2.860
52½		3.540	3.070
60	3.975	3.700	3.300
67½			3.145
75	4.060		
90	4.133	4.000	3.550

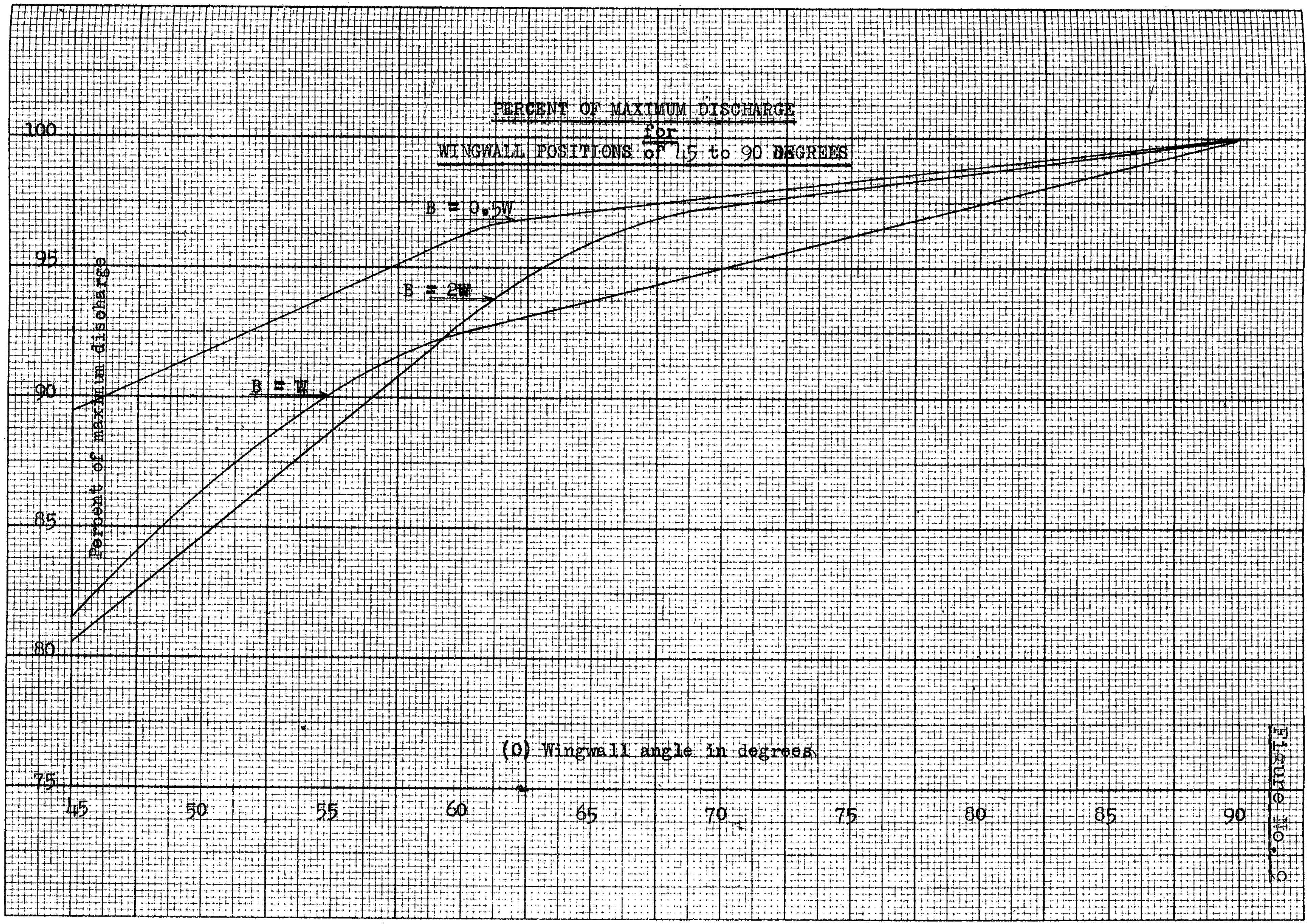
It is evident that the discharge capacity of the rectangular spillway is influenced by the position of the wingwall. Figure 9 was then plotted to illustrate these results on a percentage- wise basis.

The manner in which the depth of box affects the discharge is clearly illustrated on the head-discharge curves Figures 5 through 7 . This method of plotting



COEFFICIENT OF DISCHARGE *
for
Wingwall Positions of 45 to 90 Degrees

Figure No. 8



PERCENT OF MAXIMUM DISCHARGE for WINGWALL POSITIONS of 45 to 90 DEGREES

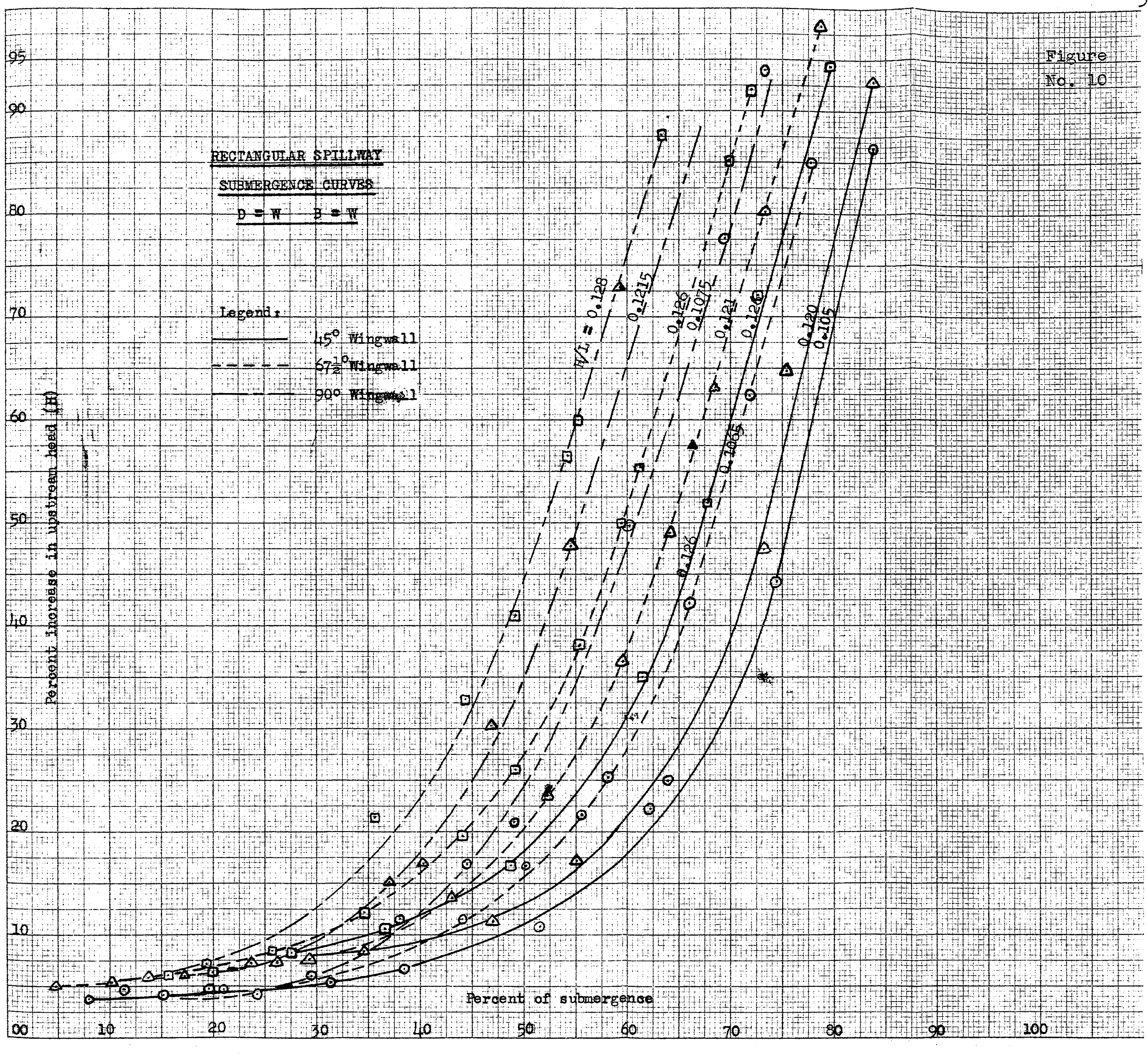
Figure No. 9

brought out several interesting facts which will be discussed in connection with the experimental results.

The submergence data was next analyzed and this too was found to be adaptable to graphical analysis. This data was analyzed in the same manner as that reported in the St. Anthony Falls report 2/. The submergence as used may be defined as the ratio, in per cent, of the water depth in the downstream channel above the spillway crest to the actual depth over the crest in the upstream channel. The increase in head for a given submergence is the difference between the actual head with submergence and the free-flow head for the same discharge. The plot of the head-submergence percentage curves, Figure 10, using nearly identical head-length (H/L) ratios is presented to illustrate the effect the wingwall position has on the discharge characteristics of the rectangular spillway.

In the determination of the effect of wingwall length on the coefficient C_1 for a given size box and angle of wingwall, a logarithmic plot was made of the Q versus H values obtained for the full length of wingwall. The values obtained for Q and H for the shortened lengths of wingwall were compared with this curve.

Figure No. 10



RECTANGULAR SPILLWAY

SUBMERGENCE CURVES

D = W B = W

DISCUSSION OF RESULTS

The main objective of this investigation was to determine exactly what relationship the angle of the wingwall had on the performance characteristics of the rectangular spillway, when used as a box inlet culvert. This investigation was carried out with the purpose of relating the discharge to head causing flow over a wide range of variable model dimensions so that the end result would be a set of design criteria that could be used for prototype construction, or could be used as a supplement to existing criteria. In all of the tests run, the model groups having wingwall angles of 90-deg. can be assumed to have the same discharge characteristics as those already presented by previous investigators for standard rectangular spillways with bulkheads normal to the axis of the conduit.

Reference is made to Figure 8 in which the various coefficients of discharge C_1 for the different length-width ratios (B/W) are plotted against the corresponding wingwall angle. The exponent for the total head (H) was found to be 1.60 for weir flow conditions. It is clearly evident from Figure 8 that the discharge coefficient C_1 and consequently the discharge, is maximum for the 90-deg. wingwall position and minimum for the 45-deg. position. A line drawn connecting these points and the intermediate points from the tests at other angles shows the variation in C_1 for wingwall positions between 45 and 90 degrees.

Previously it was shown that, to satisfy the conditions of similitude, the discharge equation for both model and prototype must have the same exponent if the model data is to be applicable to design of the prototype. To be able to apply the coefficient, as determined by the laboratory model to a structure having similar dimensions, the exponent of H must have a value of 1.50. However, could this formula, with an exponent of 1.60, have been used to design the prototype structure, it would be applicable only for heads on the structure below which the spillway begins to be submerged. Notwithstanding the fact that even though the discharge increases at a slower rate with increase in head above the point of submergence, it is sometimes economical to use design heads somewhat above the point of flooding. This is particularly true in instances where it is known that the peak discharge will sustain for only a short period of time.

Prior to continuing the discussion of similitude and the manner in which the test data can be presented for prototype design it is necessary to bring out what occurs at and beyond the point of flooding. For this purpose reference is made to Figures 5 through 7 .

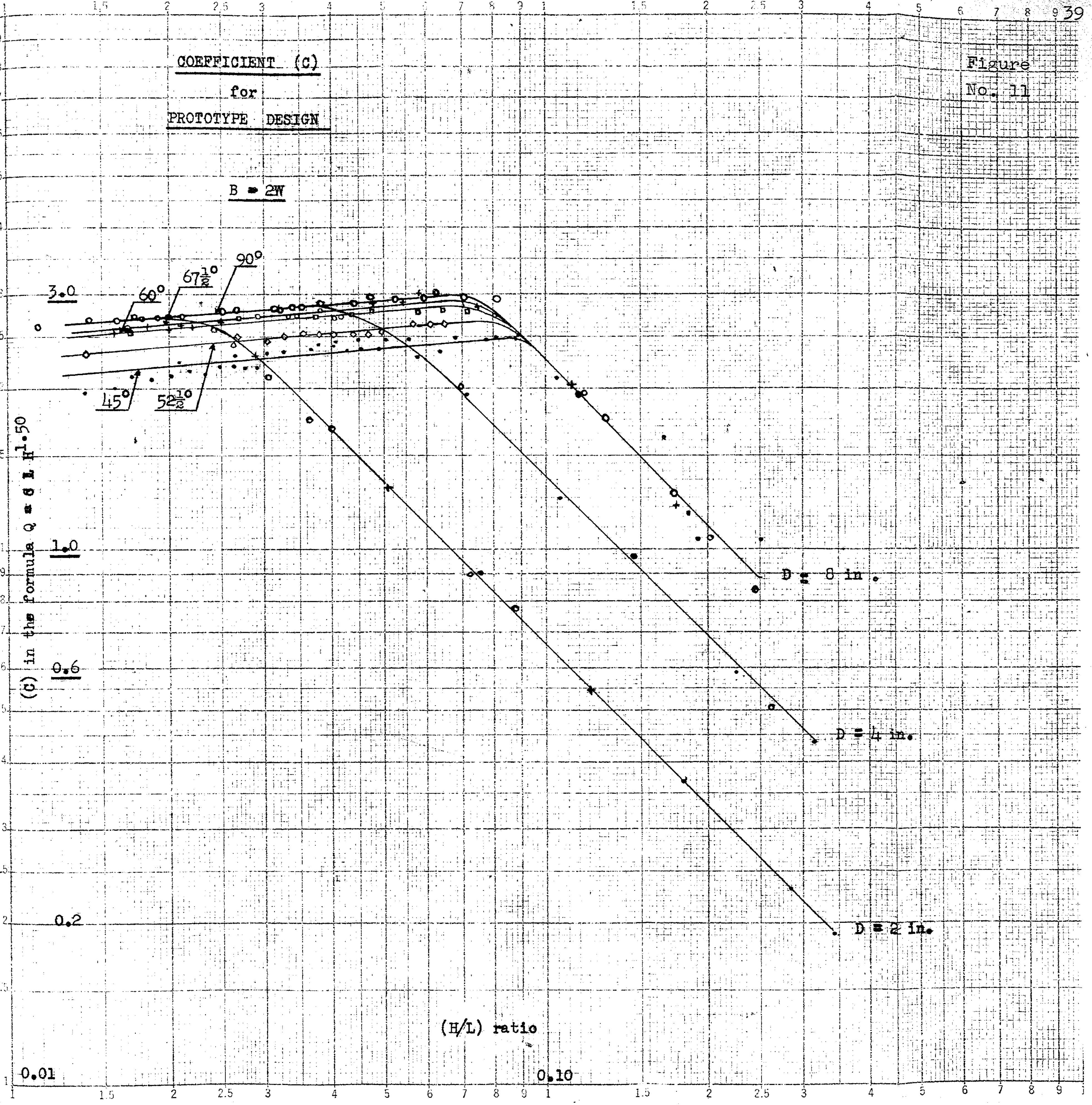
These three head-discharge curves have identical appearance. Each of the angles tested plots as an individual straight line up to the point at which the depth of the box causes the spillway crest to be submerged. For higher heads with all box depths, the head-discharge curves are seen to merge into one line indicating that once the weir is submerged, the discharge at a given head is no longer a function of the wingwall angle. Although it will be observed that the slope of the head-discharge curves beyond the point of submergence appears to be identical for the three different values of D, no attempt was made to determine the equations of the lines.

It may be noted from these curves that when the wingwall is at 90-deg., the crest will become submerged at a lower head than that for the lesser angles. This is true only in the transition section from free overfall to full submergence conditions. It is logical that this would occur because of the relatively higher heads required to discharge the same quantity of water under weir flow for the smaller wingwall angles.

In order that the test data could be adapted to prototype design, it was necessary to prepare coefficient curves using the weir formula $Q = C\ L\ H^{1.50}$. The theory in this case being that using the total head (H) with an exponent of 1.50 and a variable coefficient C instead of a constant coefficient C_1, the equation could be used for designing any size of prototype structure if the value of (C) for similar geometric dimensions were used. In order that the proper value of C could be selected, it was plotted against the ratio of head to spillway crest length (H/L). The value of H/L is a dimensionless number which gives the same value of the coefficient for similar heads on any size structure. The test data plotted on logarithmic paper showed a straight line for that portion of the curve where the flow over the spillway was not influenced by the depth of the outlet. On these curves, as in the original head-discharge curves, each wingwall position plotted as parallel individual straight lines up to the point of flooding after which all points merged into a single straight line. The curves are plotted on Figures 11 through 13 . The three groups of curves represent the three different B/W ratios tested.

As can be seen the lines are close together and not easily read. An alternate method of determining the value to use for C for a given H/L ratio, when the wingwall angle is less than 90-deg., is to utilize the curve sheet, Figure 9 , where the percent of discharge is plotted against the wingwall angle. The method outlined below will hold only for conditions of weir flow over the spillway crest.

The fact has already been proven that spillway discharge capacity is a function of the wingwall angle and that the discharge is maximum for the 90-deg. wingwall position. This is true for all heads less than those for submergence. Therefore the C value for a



COEFFICIENT (C)

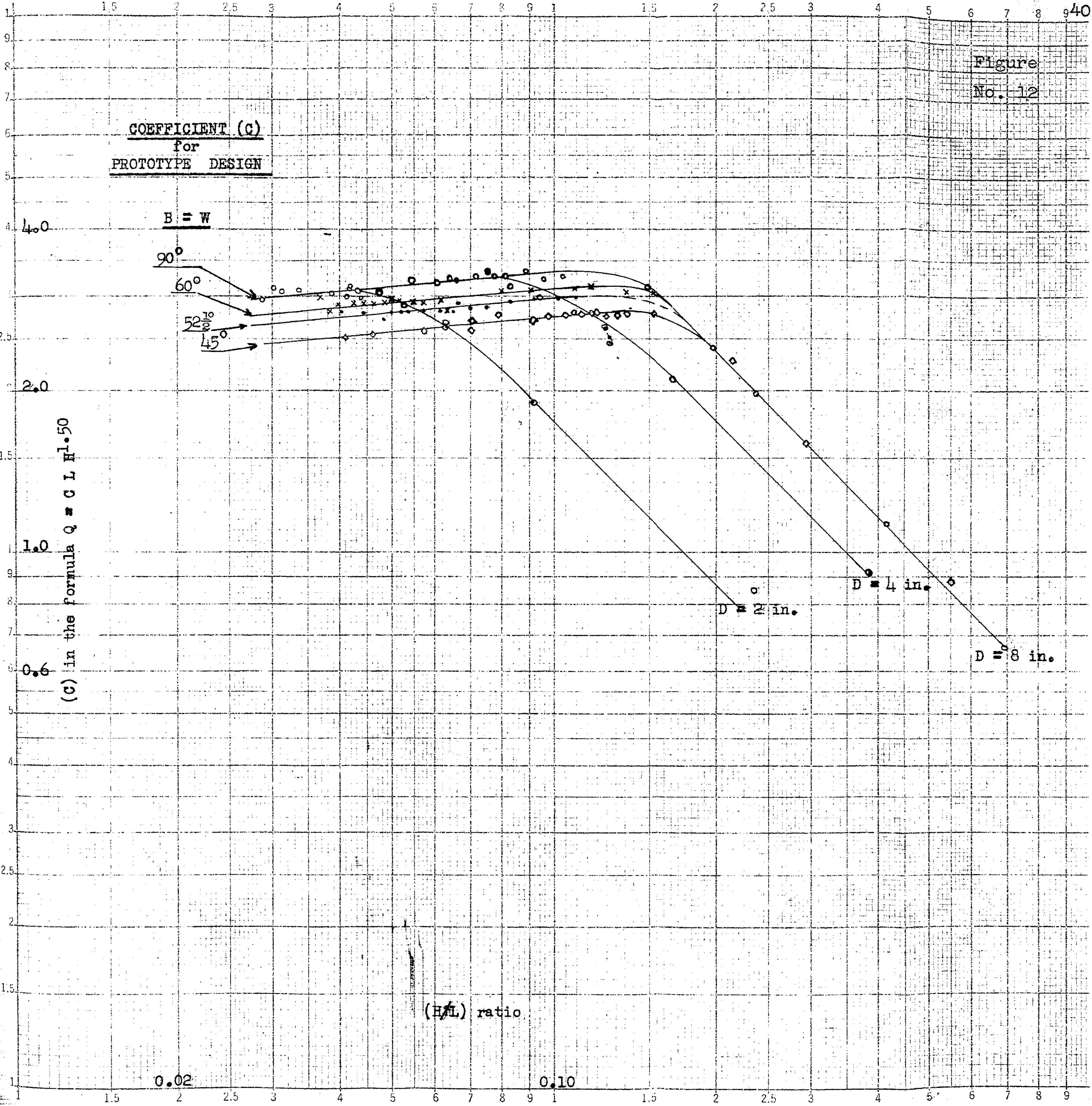
for

PROTOTYPE DESIGN

Figure No. 11

Figure No. 12

COEFFICIENT (C) for PROTOTYPE DESIGN



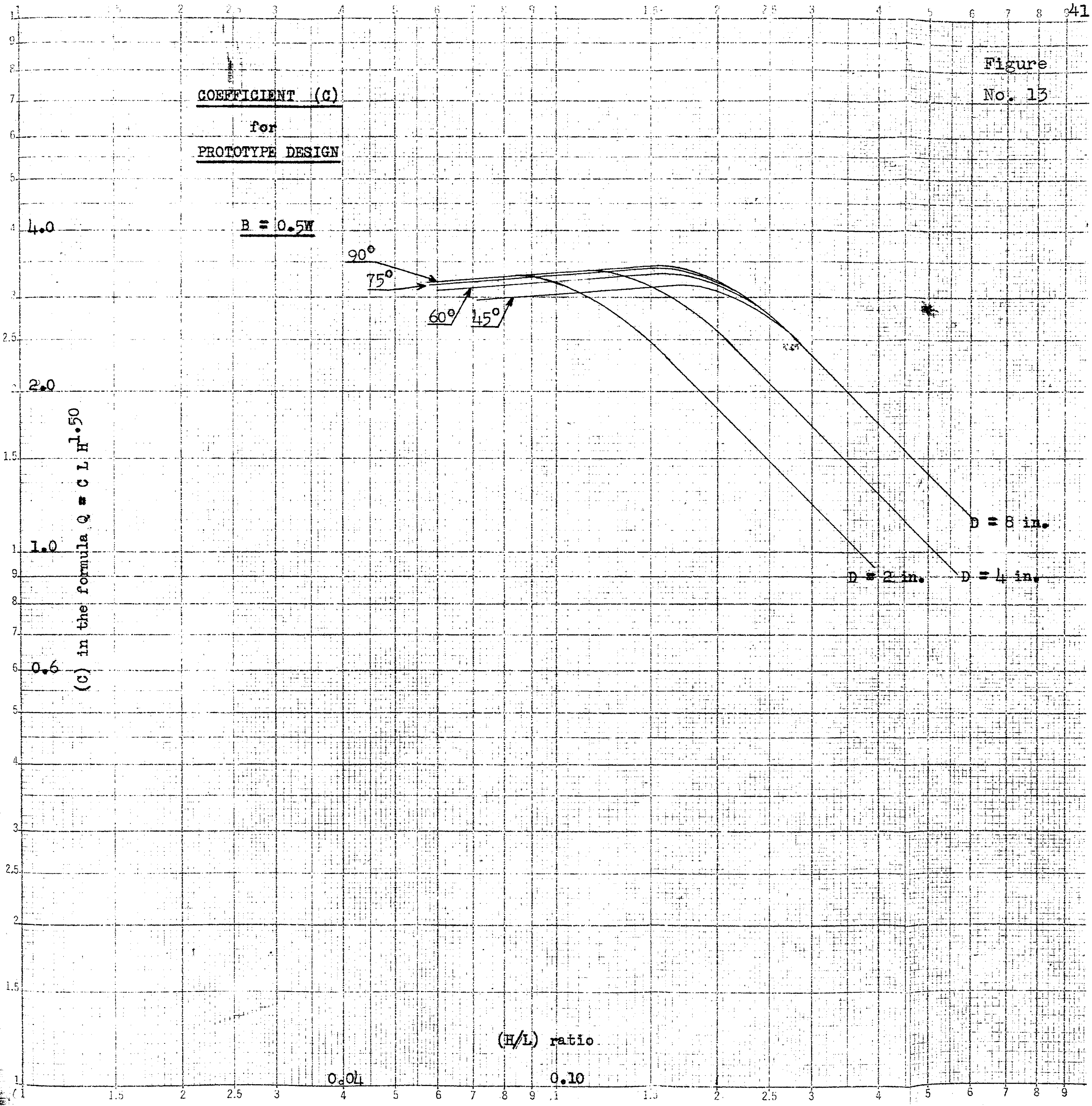


Figure No. 13

COEFFICIENT (C) for PROTOTYPE DESIGN

lesser angle can be obtained by multiplying the 90-deg. C value by the percent of maximum discharge as taken from Figure 9 . For H/L values in the transition range of the curve and beyond, it is necessary to obtain the C value directly from Figures 11 through 13.

Submergence.

The thesis investigation is also concerned with the problem of the effect of the wingwall angle on submergence. Three similar (H/L) values were tested for three different positions of the wingwall and the results are shown on Figure 10. It will be noted that there is a very marked increase in head water level (H) with increase in submergence, there being a greater effect of submergence with the higher heads. This pattern is true for the three angles tested.

Considering nearly similar H/L values, such as 0.128 for 90-deg., 0.126 for 67½-deg., and 0.126 for 45-deg., it can be seen that for a given submergence, say at 50%, the percent increase in head is less for the 90-deg. wingwall than for the 67½-deg,, which in turn is less than the 45-deg. value. This much smaller increase in head for the lesser angles is due to the fact that the head for the original free-flow for the lesser angles is higher for the same discharge than for the large angles. A reasonable assumption then is that if the position of the wingwall is such as to cause a higher head for free-flow conditions, any effect of submergence is going to have less influence.

The relation that the position of the wingwall has to the total head, H, required to discharge a given amount of water for conditions of submergence is shown by the following sample computations. The percent increase in head for the different percents of submergence are taken from Figure 10.

Assuming a free-flow head, H, of 0.215 for the wingwall position of 90-deg., the H/L ratio is 0.1075. The free-flow discharge for this head for the three angles is:

	90°	$67\frac{1}{2}^\circ$	45°
Q(cfs) =	0.684	0.642	0.550

At 50% submergence (H) increases as follows:

	90°	$67\frac{1}{2}^\circ$	45°
% Increase (H)	24.5%	16.1%	11.5%

For a discharge of 0.684, the head for each case is:

	90°	$67\frac{1}{2}^\circ$	45°
(H) Head rqd. to discharge 0.684 cfs	0.215	0.224	0.242
Δ H Based on % increase	0.053	0.036	0.028
Total Head H when crest is submerged	0.268	0.260	0.270
Percent variation from mean head 2.67 is	+ .075%	-2.25%	+ 1.5%

None of the many computations such as these indicated a head variation greater than 5% plus or minus and the majority of the head values for a given discharge were within 3% plus or minus of the mean value.

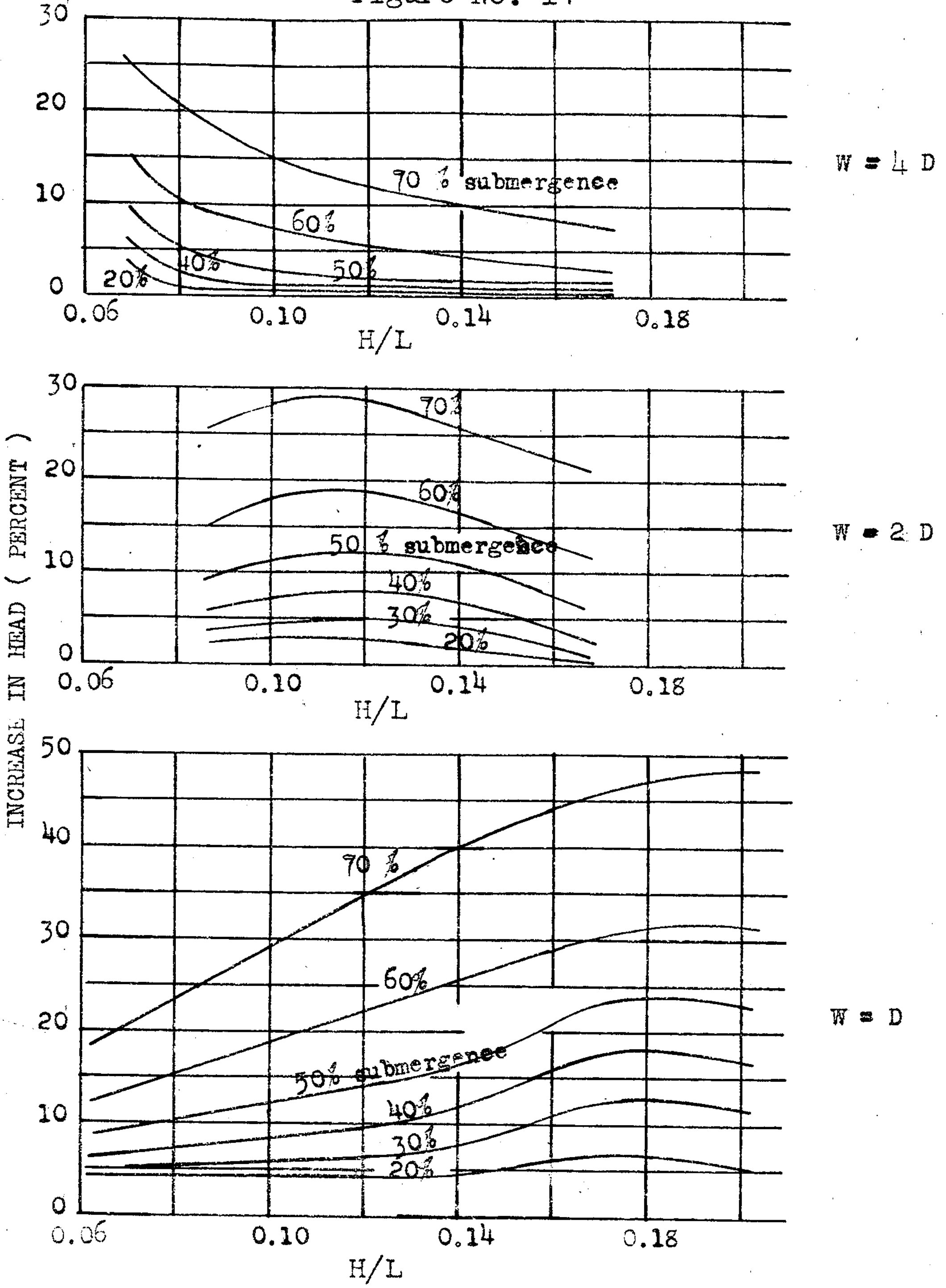
These computations indicate that, within the limits of experimental error, the total head for submerged flow will be the same regardless of the position of the wingwall. That this condition is true was shown previously, Figures 5 through 7 , on which the corresponding head-discharge values for all angles tested plot as a single straight line once the spillway crest is submerged.

Since the angle of wingwall has little effect on the head at a given flow for submerged conditions, it is recommended that Figure 14 taken from the St. Anthony Falls report 2/, be used for design purposes.

Length of Wingwall.

The analysis of the data plotted on Figure 15 shows that the length of the wingwall does not affect the hydraulic characteristics of the rectangular spillway. The corresponding H and Q values for one third, two thirds and full length of wingwall are similar.

Figure No. 14



RECTANGULAR SPILLWAY

Head Correction for Submergence (W=B)

SCS Project MN R3 Sept. 1943 (reprint)

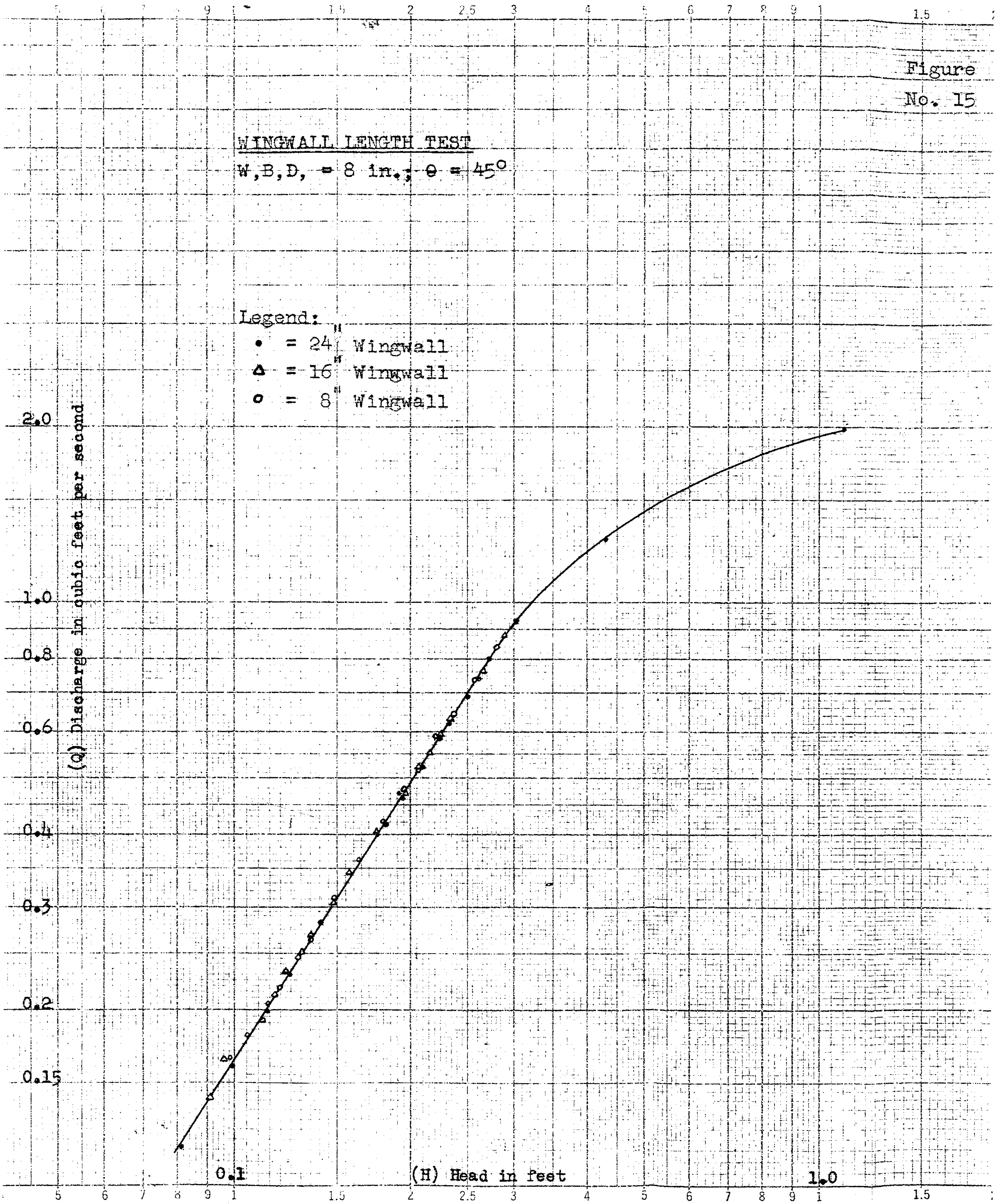


Figure No. 15

WINGWALL LENGTH TEST

W,B,D, = 8 in.; θ = 45°

EXAMPLE

Assume that is is desired to attach a rectangular spillway to a six foot square conduit having 45-deg. wingwalls. The conduit is already in place. The maximum allowable head over the structure is 2-0 (2 ft-0 in.), and the maximum discharge is 200 cfs. Free flow conditions must exist for this discharge.

The width and depth of spillway are equal to the width and depth of the conduit. Assume that the length of box required is equal to 2W.

$D = 6\text{-}0 \qquad W = 6\text{-}0 \qquad B = 2W = 12\text{-}0$

$L = 2B \neq W = 30\text{-}0 \qquad H = 2\text{-}0$ or less

$H/L = 2 / 30 = 0.0667$

Substituting in the formula $Q = C\ L\ H^{1.50}$

$200 = C \times 30 \times H^{1.50}$

$C\ H^{1.50} = 6.667$

Enter the curve, Figure 11, for θ = 45-deg. and $B = 2W$ for an H/L value of 0.0667 to find the value of $C = 2.42$.

$H^{1.50} = 6.667 / 2.42 = 2.76$

$H = 1.968$ (since H must be less than 2 this is satisfactory. The value of H/L changes to 1.968 / 30 or 0.0657. The new value of D is 2.415. Substituting this into the equation:

$Q = C \times L \times H^{1.50} = (2.415)\ (30.0)\ (2.76) = \underline{199.8 \text{ cfs}}$

This is close enough to the desired 200 cfs and therefore the size of box that will carry the expected discharge safely is when W = D = 6-0 and B = 12-0.

As this flow was required to be weir flow the alternative method is also applicable for determining the box dimensions. Using the same base assumptions, Figure 9 shows that when B = 2W for a 45-deg. wingwall position the discharge is 80.5 percent of that for a 90-deg. wingwall position.

The relative value of Q for designing the box by this method is 200 / 80.5% or 249 cfs.

$249 = C \times 30 \times H^{1.50}$

$C\,H^{1.50} = 8.30$ (C is found to be 2.97 from the 90-deg. line on the curve sheet.)

$H^{1.50} = 2.795$

H = 1.985 less than 2, so satisfactory.

The value of H/L has changed to 1.985 / 30 or 0.0662 and the corresponding C value is 2.96.

$Q = C \times L \times H^{1.50} = (2.96)\ (30)\ (2.795) = 248$ cfs

The reduction in this discharge due to the 45-deg. angle results in 248 x 0.805 = 200 cfs which is the design discharge. The W, B, and D dimensions have not changed.

CONCLUSIONS AND RECOMMENDATIONS

The model tests conducted for this thesis make it possible to predict the effect of the upstream wingwall position on the discharge characteristics of the " box inlet culvert". A limiting condition for the use of this data is that the angle of the wingwall must be the same on both sides of the box, a usual condition.

The discharge is a function of the wingwall position for all heads on a given size structure below the head at which the spillway crest becomes submerged. The percent of the maximum discharge for a 90-deg. wingwall is expressed as a function of the different wingwall positions on Figure 9 . When the crest becomes submerged, the discharge (Q) for a given head (H) is identical for all wingwall positions between 45 and 90 degrees.

The effect of submergence is identical whether caused by an excessive rise in the upstream headwater level or by an increase in the downstream tailwater level.

The length of the wingwall has no effect on the discharge characteristics of the spillway crest and consequently the wingwall can be built to conform solely to the site conditions.

The design of a prototype structure for ratios of lengths within the limits tested can be made directly from the curves plotted on Figures 11 through 13. The method used is shown in the example. It is also felt that the test results are applicable for use with existing criteria, based on tests using only 90-deg. bulkheads. The discharge that will occur for a given head for angles within the limits tested can be determined from Figure 9 .

For heads above those at which the weir crest begins to submerge, the position of the wingwall need no longer be considered. The determination of the discharge for a given upstream head or for the changed value of the upstream head due to a rise in the downstream tailwater level can be figured as if the wingwalls were normal to the axis of the conduit.

ACKNOWLEDGEMENTS

This project was under the direction of Dr. Arno T. Lenz, Director of the Hydraulics Laboratory, who offered many valuable suggestions. Dr. James R. Villemonte, Assistant Professor of Civil Engineering, also was very helpful.

Mr. Edwin F. Freyberger, Chief Regional Engineer, for Region 3, of the U. S. Soil Conservation Service suggested the need for this investigation.

Mr. J. D. Wilde, Laboratory Mechanic, gave considerable assistance in the construction of the apparatus.

BIBLIOGRAPHY

1. Kessler, Lewis H., Experimental Investigation of the Hydraulics of Drop Inlets and Spillways for Erosion Control Structures. University of Wisconsin Engineering Experiment Station Bulletin # 80. (1934).
2. Huff, Albert N., The Hydraulic Design of Rectangular Spillways. University of Minnesota, St. Anthony Falls Hydraulic Laboratory, U. S. Soil Conservation Service. -TP-71, (Revised copy 1948).
3. Blaisdell, Fred W., and Donnelly, Charles A., Capacity of Box Inlet Drop Spillways. University of Minnesota, St. Anthony Falls Hydraulic Laboratory, Technical Paper No. 7, Series B. (January 1951).
4. Rouse, Hunter, Engineering Hydraulics. (State University of Iowa, 1950).
5. King, Horace W., Wisler, Chester O., and Woodburn, James G. Hydraulics., 5th. ed., (New York 1950).

APPENDIX A

OBSERVED and COMPUTED DATA

Run No. 1.			Run No. 2.		
B = 2W	D = W	$\theta = 90^\circ$	B = 2W	D = W/2	$\theta = 90^\circ$
(Q) cfs	(H) ft.	(C)	(Q) cfs	(H) ft.	(C)
0.114	0.056	2.595	0.112	0.054	2.690
0.171	0.071	2.715	0.169	0.068	
0.250	0.090	2.775	0.226	0.084	2.790
0.314	0.104	2.810	0.251	0.090	2.790
0.384	0.118	2.845	0.279	0.098	2.720
0.438	0.128	2.865	0.325	0.107	2.790
0.539	0.147	2.865	0.359	0.113	2.830
0.619	0.158	2.960	0.408	0.125	2.770
0.721	0.177	2.905			
0.878	0.200	2.945			
0.965	0.210	3.015			
			submergence		
1.140	0.238	2.945	0.450	0.135	2.720
1.390	0.273	2.920	0.469	0.139	2.715
1.562	0.387	1.945	0.579	0.170	2.475
1.660	0.432	1.753	0.756	0.233	2.018
1.860	0.583	1.262	1.070	0.487	0.962
1.995	0.684	1.073	1.385	0.880	0.503
2.065	0.820	0.834			

OBSERVED and COMPUTED DATA

Run No. 3.			Run No. 4.		
B = 2W	D = W/4	θ = 90°	B = 2W	D = W/4	θ = 45°
(Q) cfs	(H) ft.	(C)	(Q) cfs	(H) ft.	(C)
0.0512	0.0332	2.560	0.7690	1.1383	0.189
0.0655	0.0384	2.620	0.7100	0.9483	0.231
0.0933	0.0477	2.690	0.5680	0.5986	0.368
0.1259	0.0578	2.720	0.3850	0.2534	0.905
0.1608	0.0679	2.730	0.2660	0.0985	2.190
0.1303	0.0594	2.700	0.2040	0.0923	2.180
0.1465	0.0642	2.710	0.1910	0.0880	2.200
0.1565	0.0673	2.700	0.1748	0.0830	2.200
			submergence		
0.1780	0.0749	2.600	0.1550	0.0780	2.135
0.1982	0.0811	2.575	0.1402	0.0725	2.155
0.2120	0.0884	2.415	0.1232	0.0672	2.120
0.2295	0.1023	2.100	0.1070	0.0620	2.085
0.2480	0.1221	1.745	0.0958	0.0570	2.110
0.2790	0.1347	1.690	0.0810	0.0526	2.005
0.2990	0.1573	1.715	0.0654	0.0463	1.958
0.4150	0.2962	0.770			

OBSERVED and COMPUTED DATA

Run No. 5.			Run No. 6.		
B = 2W	D = W/2	$\theta = 45^\circ$	B = 2W	D = W	$\theta = 45^\circ$
(Q) cfs	(H) ft.	(C)	(Q) cfs	(H) ft.	(C)
0.137	0.0696	2.250	0.344	0.1197	2.500
0.151	0.0822	1.920	0.411	0.1363	2.450
0.204	0.0886	2.230	0.476	0.1499	2.455
0.250	0.1011	2.335	0.567	0.1682	2.470
0.281	0.1090	2.340	0.660	0.1865	2.455
0.337	0.1222	2.356	0.760	0.2126	2.325
0.362	0.1266	2.410	0.922	0.2310	2.490
0.397	0.1348	2.400	1.087	0.2599	2.460
0.426	0.1430	2.350	1.162	0.2706	2.480
0.465	0.1514	2.370	1.335	0.2969	2.470
0.524	0.1614	2.370	1.360	0.2991	2.500
submergence					
0.644	0.1926	2.280	1.600	0.3930	1.950
0.761	0.2397	1.945	2.195	0.5520	1.605
0.925	0.3563	1.245	2.695	0.8435	1.045
1.275	0.7558	0.581	1.515	0.3506	2.219
1.525	1.0390	0.432	1.920	0.6280	1.160
			2.165	0.6380	1.028
			2.650	0.8050	0.989

OBSERVED and COMPUTED DATA

Run No. 7.			Run No. 8.		
B =2W	D = W	$\theta = 67\frac{1}{2}^{\circ}$	B = 2W	D = W/4	$\theta = 67\frac{1}{2}^{\circ}$
(Q) cfs	(H) ft.	(C)	(Q) cfs	(H) ft.	(C)
0.392	0.1196	2.840	0.1305	0.0611	2.595
0.609	0.1595	2.870	0.1470	0.0664	2.578
0.745	0.1820	2.880	0.1642	0.0707	2.620
0.866	0.1960	2.995	0.1652	0.0710	2.620
0.932	0.2083	2.940	0.1722	0.0737	2.580
1.040	0.2295	2.840			
0.609	0.1595	2.870			
0.079	0.0470	2.320			
submergence					
1.310	0.2749	2.725	0.1983	0.0820	2.535
1.552	0.3751	2.022	0.2280	0.0962	2.295
1.795	0.5880	1.190	0.3020	0.1690	1.303
			0.4705	0.4078	0.542
			0.7000	0.9723	0.219

OBSERVED and COMPUTED DATA

Run No. 9.			Run No. 10.		
B = W	D = W/4	$\theta = 90^{\circ}$	B = W	D = W/2	$\theta = 90^{\circ}$
(Q) cfs	(H) ft.	(C)	(Q) cfs	(H) ft.	(C)
0.0914	0.0600	3.105	0.0812	0.0573	2.960
0.0990	0.0640	3.055	0.1057	0.0667	3.070
0.1293	0.0770	3.020	0.1500	0.0831	3.140
0.1410	0.0820	3.000	0.2310	0.1090	3.210
0.1533	0.0860	3.040	0.2660	0.1202	3.195
			0.3015	0.1305	3.200
			0.3810	0.1502	3.275
			submergence		
0.1765	0.0943	3.045	0.424	0.1663	3.130
0.1640	0.0911	2.980	0.480	0.1864	2.980
0.1805	0.0972	2.980	0.515	0.1974	2.940
0.1940	0.1042	2.890	0.567	0.2165	2.820
0.2280	0.1222	2.670	0.653	0.2475	2.650
0.2910	0.1816	1.880	0.805	0.3317	2.105
0.4810	0.4697	0.747	1.249	0.7697	0.925

OBSERVED and COMPUTED DATA

Run No. 11.			Run No. 13.		
B = W	D = W	$\theta = 90^\circ$	B = W	D = W	$\theta = 45^\circ$
(Q) cfs	(H) ft.	(C)	(Q) cfs	(H) ft.	(C)
0.425	0.1615	3.280	0.640	0.2358	2.800
0.613	0.2059	3.280	0.625	0.2327	2.780
0.540	0,1903	3.260	0.587	0.2235	2.780
0.292	0.1265	3.245	0.525	0.2081	2.770
0.055	0.0459	2.755	0.469	0.1938	2.750
0.151	0.0830	3.155	0.417	0.1814	2.700
0.212	0.1028	3.215	0.345	0.1571	2.770
0.297	0.1285	3.225	0.283	0.1396	2.710
0.351	0.1420	3.280	0.230	0.1242	2.635
0.393	0.1534	3.280	0.199	0.1140	2.585
0.492	0.1761	3.330	0.160	0.0987	2.575
			0.142	0.0916	2.560
			0.116	0.0813	2.500
		submergence			
1.008	0.2960	3.120	0.689	0.2490	2.765
1.190	0.3939	2.410	0.740	0.2602	2.760
1.290	0.4727	1.985	0.800	0.2716	2.820
1.720	0.8297	1.138	0.929	0.3021	2.795
2.195	1.3896	0.670	1.280	0.4277	2.285
			1.980	1.1030	0.857

OBSERVED and COMPUTED DATA

Run No. 14.			Run No. 15.		
B = W/2	D = W	θ = 90°	B = W/2	D = W/4	θ = 90°
(Q) cfs	(H) ft.	(C)	(Q) cfs	(H) ft.	(C)
0.396	0.1937	3.485			
0.378	0.1886	3.460			
0.347	0.1784	3.460			
0.316	0.1673	3.465			
0.267	0.1510	3.420			
0.176	0.1174	3.280			
0.160	0.1103	3.270			
0.142	0.1033	3.210			
0.125	0.0951	3.200			
0.113	0.0871	3.300			
0.093	0.0769	3.240			
submergence					
0.438	0.2088	3.440	0.152	0.1063	3.280
0.456	0.2149	3.430	0.183	0.1215	3.245
0.480	0.2240	3.395	0.212	0.1366	3.142
0.518	0.2370	3.360	0.242	0.1546	2.980
0.573	0.2611	3.220	0.316	0.2361	2.262
0.842	0.4374	2.180	0.450	0.5100	0.926
1.092	0.8021	1.140			

OBSERVED and COMPUTED DATA

Run No. 16.			Run No. 17.		
B = W/2	D = W/2	$\theta = 90^\circ$	B = W/2	D = W/2	$\theta = 45^\circ$
(Q) cfs	(H) ft.	(C)	(Q)	(H)	(C)
			0.1320	0.1031	2.990
			0.1775	0.1253	3.000
			0.1920	0.1314	3.025
			0.2167	0.1425	3.025
			0.2340	0.1496	3.030
			0.2670	0.1626	3.050
			0.2840	0.1698	3.045
			0.3070	0.1798	3.045
submergence					
0.332	0.1782	3.310	0.3360	0.1910	3.020
0.376	0.1905	3.390	0.4040	0.2191	2.955
0.433	0.2263	3.020	0.4320	0.2361	2.825
0.465	0.2551	2.705	0.4720	0.2650	2.595
0.517	0.3127	2.222	0.5310	0.3376	2.030
0.659	0.5190	1.320	0.7820	0.7367	0.928

OBSERVED and COMPUTED DATA

Run No. 19.			Run No. 20.		
$B = W/2$	$D = W$	$\theta = 45^\circ$	$B = W/2$	$D = W$	$\theta = 60^\circ$
(Q) cfs	(H) ft.	(C)	(Q) cfs	(H) ft.	(C)
0.1690	0.1193	3.080	0.1500	0.1063	3.240
0.1890	0.1294	3.050	0.1695	0.1154	3.245
0.2000	0.1344	3.045	0.1845	0.1225	3.230
0.2140	0.1395	3.080	0.1980	0.1285	3.220
0.2230	0.1435	3.080	0.2090	0.1336	3.210
0.2310	0.1476	3.060	0.2310	0.1422	3.235
0.2480	0.1546	3.060	0.2810	0.1619	3.240
0.2580	0.1587	3.040	0.3120	0.1730	3.250
0.2745	0.1658	3.050			
0.2830	0.1688	3.060			
0.2960	0.1748	3.040			
0.3160	0.1800	3.100			
0.3290	0.1870	3.050			
0.3670	0.1972	3.145			

submergence

0.4100	0.2124	3.140	0.4070	0.2026	3.345
0.4650	0.2306	3.150	0.4770	0.2269	3.310
0.5380	0.2577	3.085	0.5330	0.2489	3.220
0.6450	0.3087	2.820	0.5930	0.2750	3.080
0.8060	0.3964	2.420	0.7290	0.3606	2.525
0.9500	0.6493	1.360	0.9410	0.5995	1.520

OBSERVED and COMPUTED DATA

Run No. 21.			Run No. 22.		
B = W/2	D = W	$\theta = 75^\circ$	B =2W	D = W	$\theta = 60^\circ$
(Q) cfs	(H) ft.	(C)	(Q) cfs	(H) ft.	(C)
0.3780	0.1879	3.485	0.250	0.0904	2.760
0.3600	0.1827	3.460	0.285	0.0997	2.720
0.3325	0.1753	3.400	0.338	0.1115	2.720
0.3150	0.1706	3.350	0.356	0.1157	2.710
0.3000	0.1651	3.350	0.405	0.1253	2.740
0.2835	0.1590	3.350	0.454	0.1358	2.720
0.2650	0.1519	3.355	0.504	0.1454	2.725
0.2510	0.1469	3.345	0.560	0.1560	2.725
0.2382	0.1418	3.345	0.666	0.1777	2.670
0.2193	0.1337	3.360	0.780	0.1931	2.755
0.1985	0.1256	3.345	0.923	0.2150	2.775
0.3015	0.1661	3.340	0.407	0.1254	2.750
			0.581	0.1602	2.720
			submergence		
			1.080	0.2401	2.760
			1.263	0.2693	2.715

OBSERVED and COMPUTED DATA

Run No. 23.			Run No. 24.		
$B = 2W$	$D = W$	$\theta = 52\frac{1}{2}^\circ$	$B = W$	$D = W$	$\theta = 60^\circ$
(Q) cfs	(H) ft.	(C)	(Q) cfs	(H) ft.	(C)
0.2235	0.0902	2.485	0.1170	0.0756	2.810
0.2635	0.1016	2.440	0.1283	0.0787	2.910
0.2990	0.1092	2.489	0.1415	0.0839	2.915
0.3435	0.1188	2.510	0.1528	0.0880	2.925
0.3780	0.1271	2.510	0.1633	0.0920	2.925
0.4250	0.1376	2.500	0.1747	0.0961	2.935
0.4695	0.1461	2.520	0.1844	0.0992	2.960
0.5150	0.1564	2.500	0.1945	0.1033	2.930
0.5770	0.1672	2.540	0.2090	0.1084	2.930
0.6370	0.1796	2.510	0.2225	0.1135	2.910
0.7300	0.1902	2.640	0.2530	0.1227	2.940
0.8055	0.2039	2.620	0.2810	0.1320	2.925
0.8670	0.2161	2.600	0.3160	0.1422	2.950
			0.3880	0.1589	3.065
			0.4690	0.1795	3.080
			0.6240	0.2164	3.100
			submergence		
			0.7030	0.2340	3.105
			0.8690	0.2708	3.085
			1.0120	0.3023	3.050

OBSERVED and COMPUTED DATA

Run No. 25.
(L = 2 ft)
B = W   D = W $\theta = 52\frac{1}{2}^{\circ}$

(Q) cfs	(H) ft.	(C)
0.1160	0.0727	2.96
0.1265	0.0797	2.81
0.1455	0.0878	2.80
0.1609	0.0958	2.72
0.1760	0.0990	2.82
0.1860	0.1031	2.81
0.1940	0.1061	2.82
0.2150	0.1134	2.82
0.2420	0.1225	2.82
0.2600	0.1286	2.82
0.2940	0.1389	2.84
0.3280	0.1485	2.87
0.3920	0.1637	2.96
0.4670	0.1823	3.00
0.5440	0.2027	2.98
0.6140	0.2191	2.99

APPENDIX B

SUBMERGENCE DATA

$B = W \quad D = W \quad L = 2$ ft

Run No. 26.

$H = 0.215$ ft
$Q = 0.679$ cfs
$H / L = 0.1075$
$\theta = 90^\circ$

H. ft.	H_1 ft.
0.215	0.000
0.513	0.412
0.480	0.367
0.466	0.359
0.418	0.304
0.382	0.264
0.322	0.193
0.261	0.127
0.251	0.111
0.238	0.090
0.228	0.067
0.225	0.047

Run No. 27.

$H = 0.210$ ft
$Q = 0.517$ cfs
$H / L = 0.105$
$\theta = 45^\circ$

H ft.	H_1 ft.
0.210	0.000
0.218	0.017
0.220	0.043
0.221	0.068
0.224	0.086
0.233	0.120
0.262	0.167
0.257	0.165
0.303	0.225
0.392	0.327

Run No. 28.

$H = 0.213$ ft
$Q = 0.626$ cfs
$H / L = 0.1065$
$\theta = 67\frac{1}{2}^\circ$

H ft.	H_1 ft.
0.213	0.000
0.394	0.305
0.345	0.246
0.303	0.199
0.268	0.155
0.259	0.143
0.249	0.125
0.237	0.104
0.232	0.056
0.222	0.034

SUBMERGENCE DATA

B = W D = W L = 2 ft

Run No. 29.		Run No. 30.		Run No. 31.	
H = 0.243 ft Q = 0.763 cfs H / L = 0.1215 θ = 90°		H = 0.240 ft Q = 0.623 cfs H / L = 0.120 θ = 45°		H = 0.242ft Q = 0.750 cfs H / L = 0.121 θ = 67½°	
H ft.	H_1 ft.	H ft.	H_1 ft.	H ft.	H_1 ft.
0.243	0.000	0.240	0.000	0.242	0.000
0.496	0.385	0.252	0.012	0.257	0.044
0.450	0.262	0.254	0.035	0.260	0.068
0.421	0.249	0.258	0.075	0.275	0.118
0.359	0.195	0.260	0.090	0.298	0.156
0.317	0.148	0.267	0.125	0.330	0.196
0.284	0.114	0.281	0.154	0.358	0.229
0.280	0.103	0.354	0.257	0.381	0.251
0.261	0.062	0.396	0.297	0.395	0.269
0.256	0.026	0.463	0.386	0.436	0.318
				0.470	0.367

SUBMERGENCE DATA

B = W   D = W   L = 2 ft

Run No. 32.

H = 0.256 ft
Q = 0.825 cfs
H / L = 0.128
θ = 90°

H ft.	H_1 ft.
0.256	0.600
0.480	0.302
0.436	0.254
0.409	0.225
0.401	0.216
0.361	0.176
0.340	0.151
0.311	0.110
0.274	0.053

Run No. 33.

H = 0.252 ft
Q = 0.672 cfs
H / L = 0.126
θ = 45°

H ft.	H_1 ft.
0.252	0.000
0.264	0.030
0.268	0.053
0.273	0.075
0.279	0.102
0.296	0.144
0.340	0.208
0.383	0.258
0.434	0.313
0.490	0.388

Run No. 34.

H = 0.252 ft
Q = 0.800 cfs
H / L = 0.126
θ = $67\frac{1}{2}$°

H ft.	H_1 ft.
0.252	0.000
0.482	0.345
0.466	0.324
0.414	0.257
0.378	0.224
0.347	0.191
0.318	0.156
0.301	0.132
0.282	0.111
0.273	0.070
0.270	0.042

APPENDIX C

WINGWALL LENGTH DATA

$B = W \quad D = W \quad L = 2\ \text{ft} \quad \theta = 45^\circ$

Run No. 35

16 inch Wingwall (2/3 full size)

(Q) cfs	(h) ft.	(h_v) ft.	(H) ft.
0.1407	0.089	0.0016	0.0906
0.1660	0.093	0.0019	0.0949
0.1770	0.099	0.0020	0.1010
0.1930	0.110	0.0020	0.1115
0.2145	0.114	0.0022	0.1162
0.2313	0.120	0.0024	0.1224
0.2520	0.127	0.0025	0.1295
0.2720	0.133	0.0026	0.1356
0.3075	0.145	0.0028	0.1478
0.3475	0.153	0.0032	0.1562
0.4090	0.171	0.0036	0.1746
0.4850	0.191	0.0040	0.1950
0.5270	0.202	0.0042	0.2062
0.5560	0.210	0.0044	0.2144
0.5960	0.219	0.0046	0.2236
0.6390	0.229	0.0049	0.2339
0.7620	0.257	0.0055	0.2625
0.8830	0.284	0.0060	0.2900

Run No. 36

8 inch Wingwall (1/3 full size)

0.0997	0.069	0.0013	0.0703
0.1685	0.096	0.0019	0.0979
0.1865	0.103	0.0020	0.1050
0.2080	0.112	0.0021	0.1141
0.2210	0.117	0.0022	0.1192
0.2470	0.125	0.0025	0.1275
0.2720	0.134	0.0026	0.1366
0.3115	0.146	0.0028	0.1488
0.3660	0.160	0.0033	0.1633
0.4210	0.175	0.0036	0.1786
0.4790	0.191	0.0040	0.1945
0.5260	0.201	0.0042	0.2052
0.5880	0.216	0.0046	0.2206
0.6400	0.231	0.0048	0.2358
0.7450	0.250	0.0056	0.2556
0.8470	0.274	0.0060	0.2800

The range of the velocity head (h_v) values as given above are representative for this project.

APPROVAL

The foregoing thesis is hereby approved as a creditable study of an engineering project, carried out and presented in a manner sufficiently satisfactory to warrant its acceptance as a prerequisite to the degree for which it is submitted. It is to be understood that by this approval the undersigned does not necessarily endorse or approve any statements made, opinions expressed, or conclusions drawn therein, but approves this thesis only for the purpose for which it is submitted.

Arno T. Lenz
Name

Prof. of Civil Engr.
Title

Aug. 9, 1951
Date