



COMMONWEALTH OF KENTUCKY
DEPARTMENT OF HIGHWAYS
FRANKFORT

HENRY WARD
COMMISSIONER OF HIGHWAYS

January 21, 1963

ADDRESS REPLY TO
DEPARTMENT OF HIGHWAYS
MATERIALS RESEARCH LABORATORY
132 GRAHAM AVENUE
LEXINGTON 29, KENTUCKY

D.1.7.

MEMO TO: A. O. Nesier
Assistant State Highway Engineer

As an outgrowth of our hydrological studies on rainfall runoff, and our companion studies on the hydraulics of culverts, which began about 1951, and which were more-or-less consummated in the Department's current Drainage Manual, certain questions arose concerning the relative effects of various types of entrances on the capacity of culverts. In 1954, equipment for making model studies of culverts was designed, built, and put into operation in the hydraulics laboratory at the College of Engineering, University of Kentucky. Information was sought in three general areas, which were as follows:

1. Effect of obliquity of wing-walls on the capacity of box culverts.
2. Effect of hooded entrances on hydraulic capacity.
3. Effect of drop inlets on the capacity of box-type and pipe-type culverts.

The first reports were concerned with item 1, and a report was made to the Department in 1956*. That report presented data obtained

* "Hydraulic Model Studies of Culvert Operations," by E. M. West, November, 1956.

on 30- and 45-degree wing-walls. Somewhat concurrently, the information from that report was combined with test results pertaining to item 2 (hooded inlets) and published in Bulletin No. 41, Engineering Experiment

Station, University of Kentucky, 1956*. For reasons which are now obscure

* "The Development and Use of Hydraulic Models in a Study of Culvert Performance," by E. M. West and R. D. Hughes.

perhaps, the model was disassembled and was not operable for several years. However, interest in drop inlets persisted; and, inasmuch as this was one of the areas which we were more-or-less committed to study, the model was restored and adapted for the study of drop-inlets. The tests on this phase were made over a year ago, and the model was then returned to storage. The reporting of this work has been delayed, so to speak, by other matters.

The study was made by R. D. Hughes, and his report, "A Study of the Hydraulics of Drop-Inlet-Type Culvert Models," is attached, hereto. His results and analyses are best summarized by the discharge coefficient presented in the report. These coefficients, which are dimensionless, may be used directly in the basic formulae to compute discharges in full-scale culverts; whereas, the similitude analysis merely yields a factor relating the discharge in the model to the discharge of its full-scale counterpart. The end results would be the same, but the similitude analyses seem to be somewhat cumbersome.

The information may prove to be helpful should a need arise where only a drop-inlet structure will suffice or otherwise be preferred. Should such a need arise, we would welcome an opportunity to assist in its design and to observe its performance.

Respectfully submitted,



W. B. Drake
Director of Research

WBD:d1

Enc.

cc: Research Committee Members
Bureau of Public Roads (3)

Commonwealth of Kentucky
Department of Highways

A STUDY OF THE HYDRAULICS OF DROP-INLET-TYPE
CULVERT MODELS

by

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Lexington, Kentucky

January, 1963

INTRODUCTION

In certain unique situations, a drop-inlet-type of culvert might be employed to a greater advantage than the standard box- or pipe-type culvert. A drop-inlet consists merely of a vertical drop-section at the upstream end of the culvert. The drop section is connected to the main barrel of the culvert by means of an elbow or merely a right-angle intersection. The entrance to the drop-section must collect and funnel the channel-water into the drop-section. Thus, intercepting dams and sidewalls may be needed to form the entrance. Figure 1 illustrates, in a general way, some of the options which may be considered in the design of a highway culvert. The diagram alludes to the particular case where the difference between inlet and outlet elevations is such that the slope-gradient is quite steep. Likewise, as envisioned in Fig. 2, drop-inlets may find some application in situations where there is not sufficient head-room beneath the pavement to permit the installation of a culvert to the desired slope -- that is, on the uniform grade between the upstream channel and the outfall channel.

Although the concept of drop-inlet culverts is not new, situations have arisen in which this type of culvert might have been preferred to other alternatives, but was not used because reliable design parameters and criteria for design were not available and appropriately stylized. Previous information pertaining to drop-inlet structures was reported by Kessler in 1934 (1) and by Huff in the early 1940's (2).

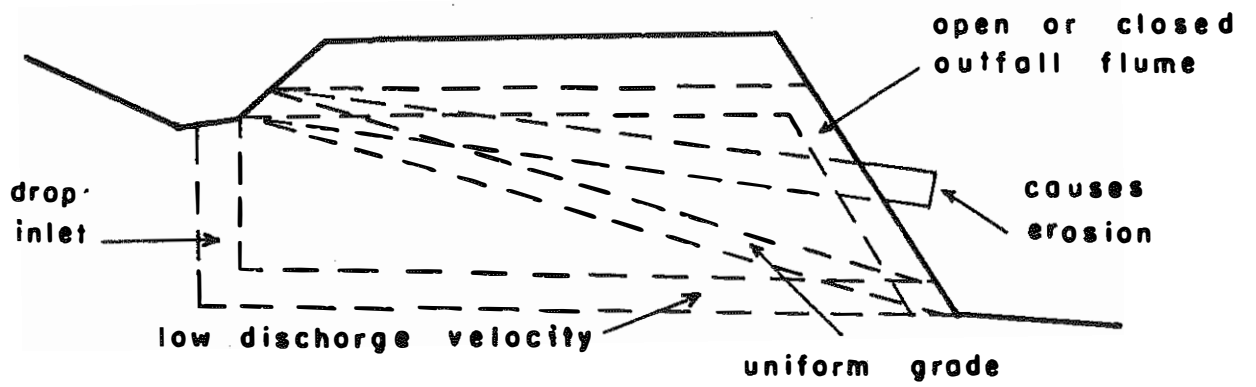


Fig. 1. Optional Routes of Culvert Under High Hydraulic Gradients

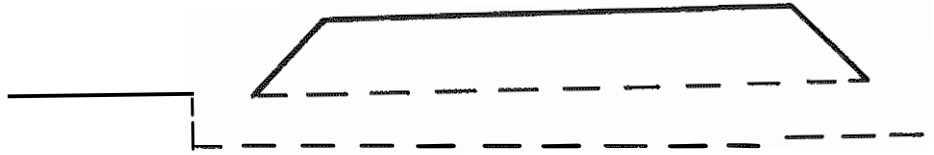


Fig. 2. Drop-Inlet Culvert in Low-Head-Room Situation.

Kessler's studies were concerned primarily with erosion-control structures; whereas, Huff's studies were concerned more specifically with the effects of the width of the approach channel and submergence of the entrance of flow-capacity. In 1951, Blaisdell and Donnelly (3)(4) reported data from 361 tests which involved free-flow, submerged-flow, and outlet-control conditions. The aforementioned reports deal with the hydraulics of drop-inlet structures, but none deals specifically with the design of drop-inlet structures for highway culverts.

The purpose of this report is to present the theories related to the hydraulic operation of drop-inlet culverts and present the results and analyses from a series of tests conducted on various models of highway culverts. The results are presented in the form of discharge coefficients which are recommended as interim design values and which, of course, are subject to confirmation by full-scale, field installations.

The design values presented herein are based on results of tests conducted on drop-inlets having drop-sections which were 4 by 4 inches in cross-section and 6-1/2, 9 and 12 inches in length. The barrel was 4 by 4 inches in cross-section and 72 inches in length. One of the transition sections consisted of a 91-degree, box-type intersection, and the other consisted of a modified box-type intersection. The inlets used in all tests were similar in that each had sidewalls parallel to the channel and a backwall -- all extending two inches above the crest of the approach channel. Other features of the models and the test procedures are included in the body of the report.

THEORETICAL CONSIDERATIONS

Whereas bridges merely span the natural channels of large streams, culverts usually constrict the natural channels of small streams. Constrictions, of course, tend to impede flow. If a culvert is to carry the same volume of water as an unstricted channel, the velocity through the culvert must be greater than the velocity in the approach channel. If the constriction is too abrupt, the water piles up at the entrance, but, even so, the barrel of the culvert may not flow full; hence, the capacity of the barrel exceeds the capacity of the entrance. This condition is commonly termed entrance-control, and is typical of flow through orifices. In each culvert, there is some dominant impedance or controlling factor, which largely governs its hydraulic capacity. For instance, backwater in the downstream channel creates back-pressure in the barrel and decreased velocity in the barrel. The control may thus be at the outlet rather than at the entrance. Likewise, the control would be in the barrel only when the barrel itself is the principal impedance.

Whereas Bernoulli's equation suggests that all impedences are accumulative and are equal to the total energy change, i.e.,

$$(HW - TW) + \frac{V_1^2 - V_2^2}{2g} = h_e + h_f + h_o + \dots,$$

some of the losses becomes negligible under certain conditions -- depending upon the location of the control. In the case where the out-fall

is free, the outlet-loss (h_o) is insignificant. Thus, either h_e or h_f governs. Moreover, if the barrel of the culvert changes direction abruptly, as in the transition from the drop-section of a drop-inlet-type culvert to the main barrel, it might be desirable to introduce a parameter, h_b , to describe the loss in the bend.

Inlet Acting as a Weir

The initial or low-head flow into a drop-inlet is typical of flow over a horizontal weir. The general flow pattern is shown in Fig. 3. The edge or top surface of the weir is called the crest, and it may be either flush with the bottom of the approach channel or at some elevation above it. Weirs are classified as either sharp-, broad-, square-, or round-crested. The over-falling stream is designated as the nappe and takes the form as shown in Fig. 3 for the square-crested weir. If the length of the weir is less than the width of the approach channel, the nappe will be contracted at each side. Directly upstream from the weir, there is a downward curvature of the water surface, known as the draw-down curve. Head, as measured at the crest, is slightly less than that actually causing flow; therefore, the velocity at some point upstream from the weir is taken to be the velocity of approach and is so used in all calculations.

Development of the basic weir formula is made in reference to Fig. 4. H_w represents headwater elevation above the crest of the weir; dh is the thickness of an elementary strip $dh \times L$; h is the headwater

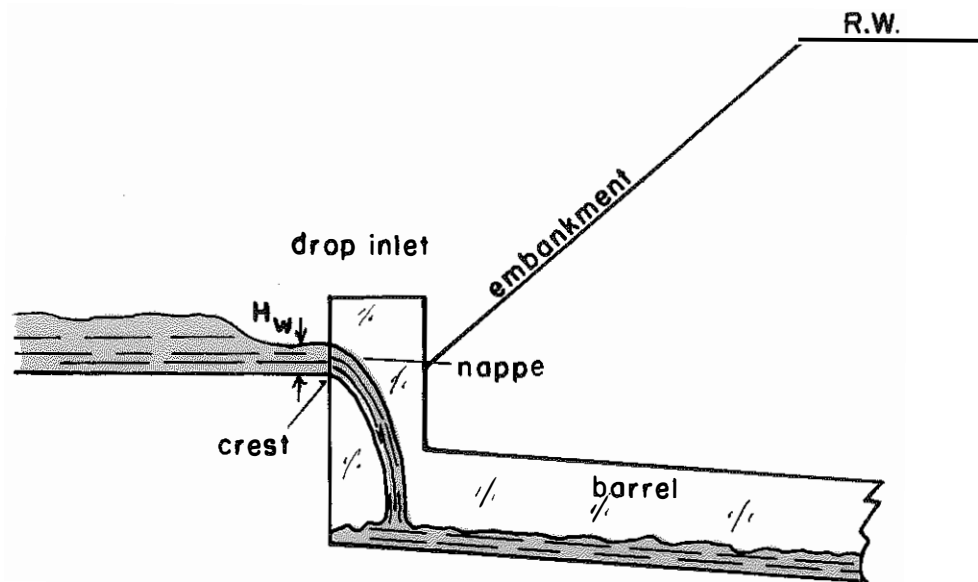


Fig. 3. Weir-Type Flow into a Drop-Inlet.

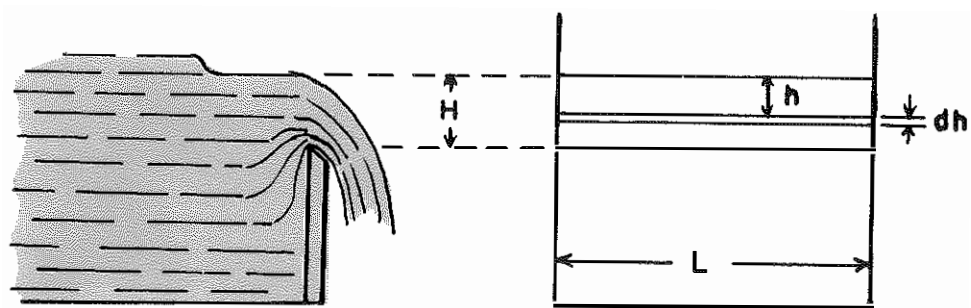


Fig. 4. Elemental Section of a Weir.

elevation above and αh is the approach velocity-head; and the effective head is $h + \alpha h$. The theoretical discharge through the elementary strip may be found as follows:

$$dQ_+ = L dh \sqrt{2g (h + \alpha h)} \quad 1$$

$$Q_+ = \int_0^{h_w} L \sqrt{2g} \sqrt{h + \alpha h} dh \quad 2$$

$$Q_+ = 2/3 \sqrt{2g} L [(H_w + \alpha h)^{3/2} - (\alpha h)^{3/2}] \quad 3$$

The approach-velocity head is relatively insignificant in comparison to depth of flow, thus the term αh may be omitted from equation 3. A more realistic value of discharge than that determined from equation 3 may be obtained through use of a coefficient with the theoretical formula. This coefficient may be combined with constants in the weir formula, and the resulting equation for actual flow over the weir is:

$$Q_w = C_w L H_w^{3/2} \quad 4$$

where C_w represents the product of the weir-coefficient and the constants in the theoretical weir formula. Assuming no head-loss during flow over the weir, C_w has a value of $2/3 \sqrt{2g}$ or 5.35. Since head-losses do occur, C_w has a value somewhat less than 5.35.

Inlet Acting as an Orifice

An orifice may be any closed perimeter through which a fluid flows. The inlet may act as an orifice if the headwater floods the weir. Flow through an orifice is governed by the area of opening as

well as the total head acting at the center of the orifice. Theoretically, discharge through the orifice is found to be the produce of the area of the opening and velocity. The general equation is:

$$Q_o = A_o V_o = A_o \sqrt{2g H_o} \quad 5$$

Where A_o , V_o and H_o represent the area of opening, velocity, and headwater elevation above the center of the orifice, respectively. Orifice flow is shown in Fig. 5.

As water passes the orifice, the area of flow is somewhat less than the area of the orifice: due to convergence of the flow-paths as they enter the orifice. The ratio of the cross-sectional area of the jet at the vena contracta to the area of the orifice is called the coefficient of contraction and is designated as C_c . The ratio of the actual velocity to the theoretical velocity is termed the coefficient of velocity and is designated as C_v . The actual discharge passing the orifice is found to be the product of the actual area of the jet times the actual velocity in the jet just beyond the orifice, or

$$Q_o = A_o C_c \sqrt{2g H_o} C_v \quad 6$$

The product of the coefficient of contraction, coefficient of velocity, and $\sqrt{2g}$ gives the orifice coefficient C_o . The general formula for actual flow through the orifice thus becomes:

$$Q_o = A_o C_o H_o^{1/2} \quad 7$$

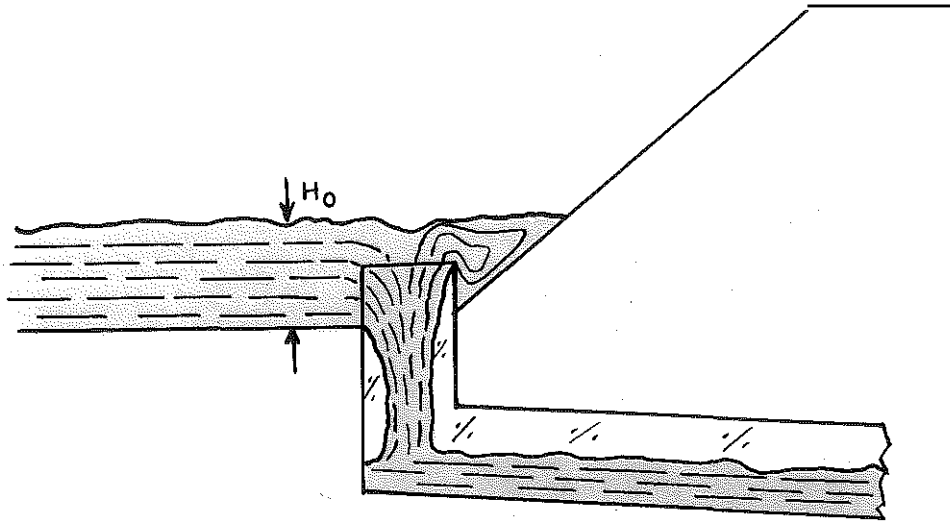


Fig. 5. Drop-Inlet Acting as an Orifice.

Short-Tube-Flow

In some instances flow may be great enough to fill the drop-section but not the barrel. This condition is likely to produce surging as control passes from the inlet to the barrel, and vice versa. When the drop section is running full, and the main barrel of the culvert is not running full, discharge is again found to be the product of the area of the vertical section and the velocity. Since the vertical section is assumed to be completely full, the coefficient of contraction is unity, and the short-tube coefficient is found to be the product of C_v and $\sqrt{2g}$. The basic equation for short-tube flow is:

$$Q_{st} = A_{st} V_{st} = A_{st} C_{st} H_{st}^{1/2}$$

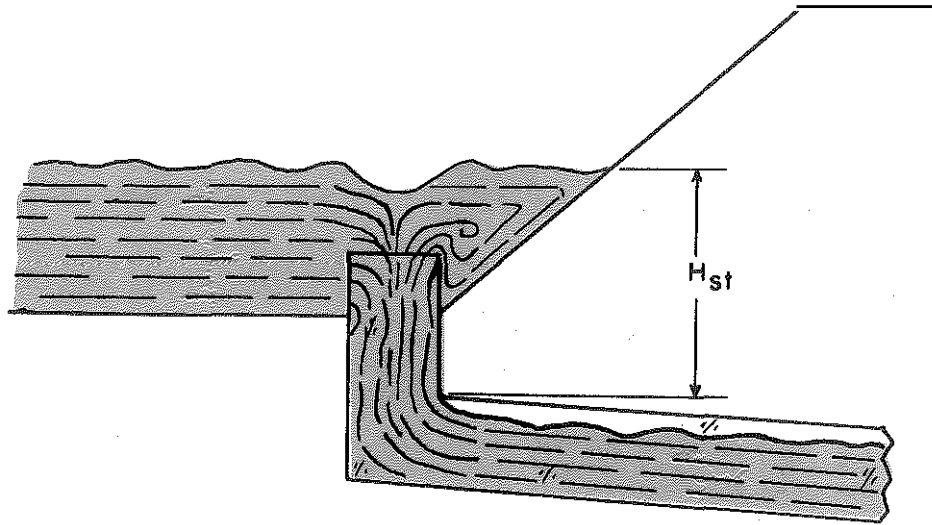


Fig. 6. Drop-Section Flowing Full; Short-Tube-Flow.

where H_{st} is as shown in Fig. 6. C_{st} is found to vary with slope of the approach channel, geometry, length and roughness of the drop-section.

Pipe Flow; Barrel Control

When the headwater elevation becomes great enough to cause full flow in both the drop-section and barrel, control switches to the barrel and to the domain of ordinary pipe-flow. The total head causing flow is assumed to be the difference between the headwater elevation and centerline of the outlet. Theoretically, the Head, H_p , should be the difference between headwater elevation and the point at which the

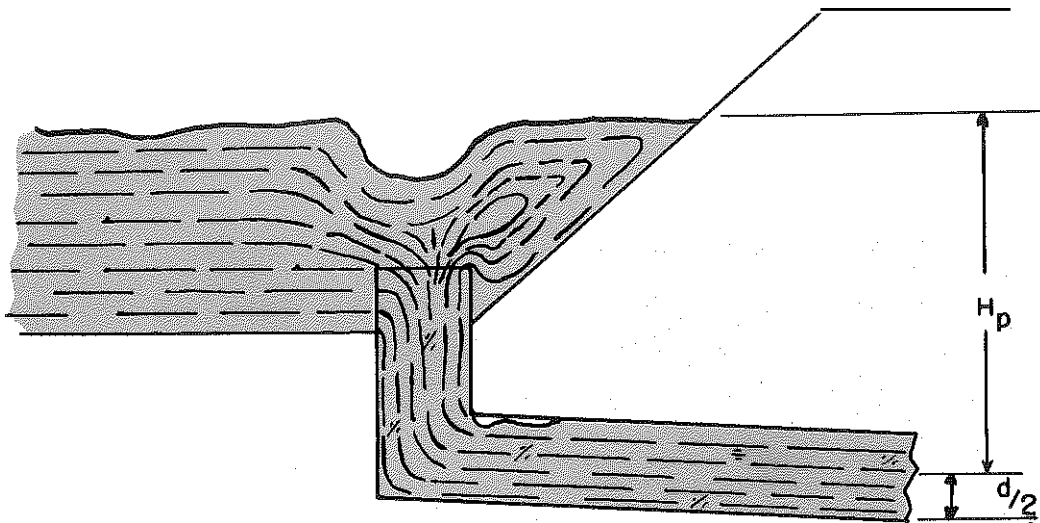


Fig. 7. Drop-Section and Barrel Flowing Full; Pipe Flow.

hydraulic grade-line pierces the plane of the outlet. This point is generally difficult to determine and is very nearly the same as the centerline of the outlet, thus little error is introduced by use of the assumption.

The total headwater elevation, as shown in Fig. 7, may be expressed in terms of the various head-loss coefficients as well as other factors governing flow. The equation thus becomes:

$$H_p = \frac{V^2}{2g} [1 + K_e + K_b + K_o + f_c \frac{L_c}{4 R_c} + f_r \frac{L_r}{4 R_r}] \quad 9$$

K_e , K_b and K_o represent the head-loss coefficients at the inlet, elbow, and outlet, respectively; V is the mean velocity throughout the structure. The factors, $f_c \frac{L_c}{4 R_c}$ and $f_r \frac{L_r}{4 R_r}$, represent losses which

are attributable to friction in the barrel and drop-section, respectively. The f 's represent the Darcy-Weisbach coefficient of friction; R 's represent the hydraulic radii; and L 's represent the lengths of the sections. The discharge may thus be computed in terms of: 1) area of flow (when areas of all sections are equal), 2) total head causing flow, and 3) head-loss coefficients. In this form:

$$Q = A \sqrt{\frac{2g}{1 + K_e + K_b + K_o + (f_c L_c / 4 R_c) + (f_r L_r / 4 R_r)}} \quad 10$$

APPARATUS AND TEST METHODS

The apparatus and test methods employed in this series of tests were essentially the same as those used in earlier studies and as reported previously (5)(6). A schematic and over-all-view are shown in Figs. 8 and 9.

Water was pumped from a pit and into the approach channel, from which it passed through the model culvert to the weir tank for gaging and was then returned to the pit for re-use. Turbulence of incoming water was stilled by means of an H-type outlet in a diffusing tank and by means of baffles placed perpendicular to the flow from the tank. Discharge was regulated by a valve above the H-type outlet.

The approach channel was constructed of marine plywood and was designed to simulate a trapezoidal stream-channel. A plexiglass end-section, having a 2:1 slope and representing a typical highway embankment was connected to the downstream end of the approach channel. Provision for affixing various inlets was made through use of a flanged opening in the plexiglass end-section.

Drop-Inlet Models

Each model consisted of four interchangeable units: the inlet, drop-section, elbow and barrel. All inlets were identical except for the flange-angles which were varied according to the channel slope for which each particular inlet was designed and in order that each

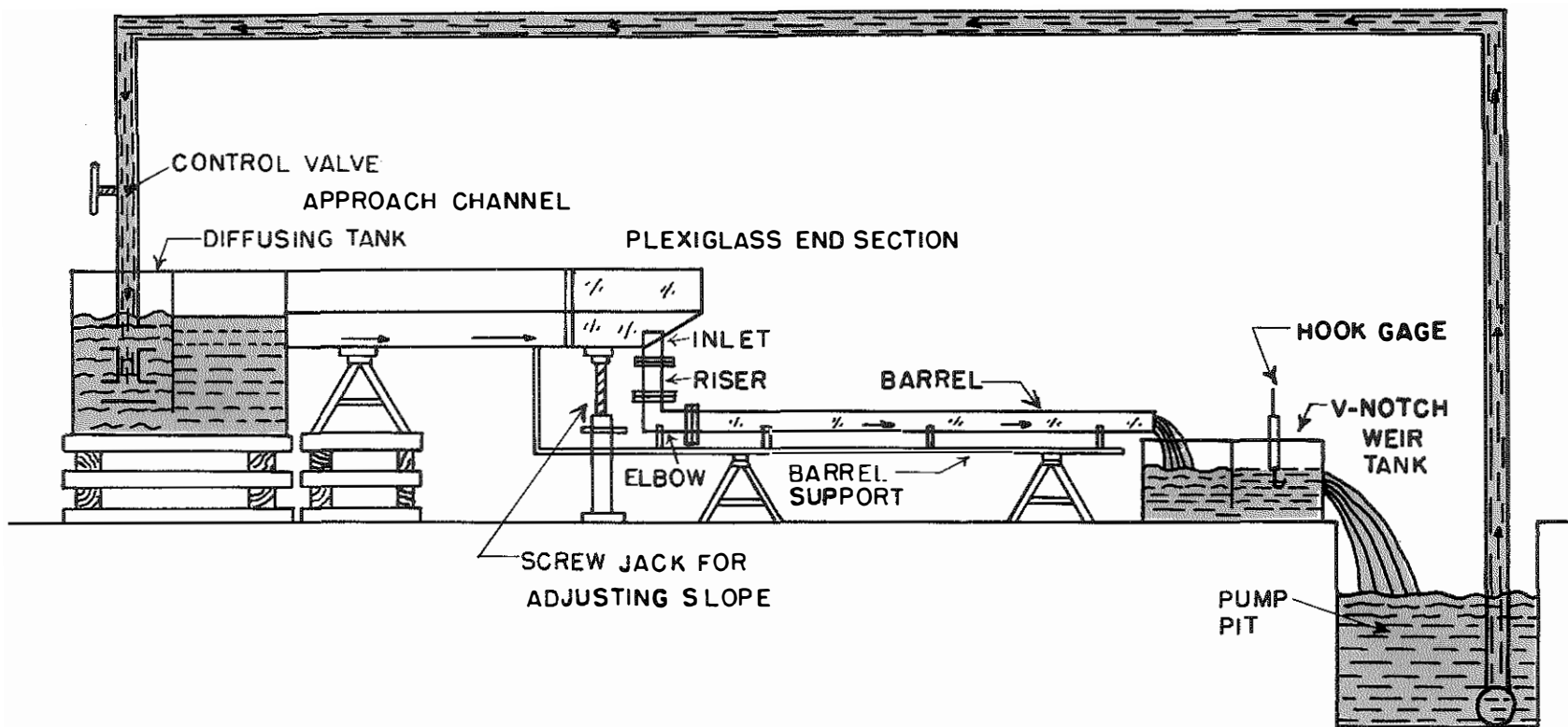


Fig. 8 - Model Apparatus

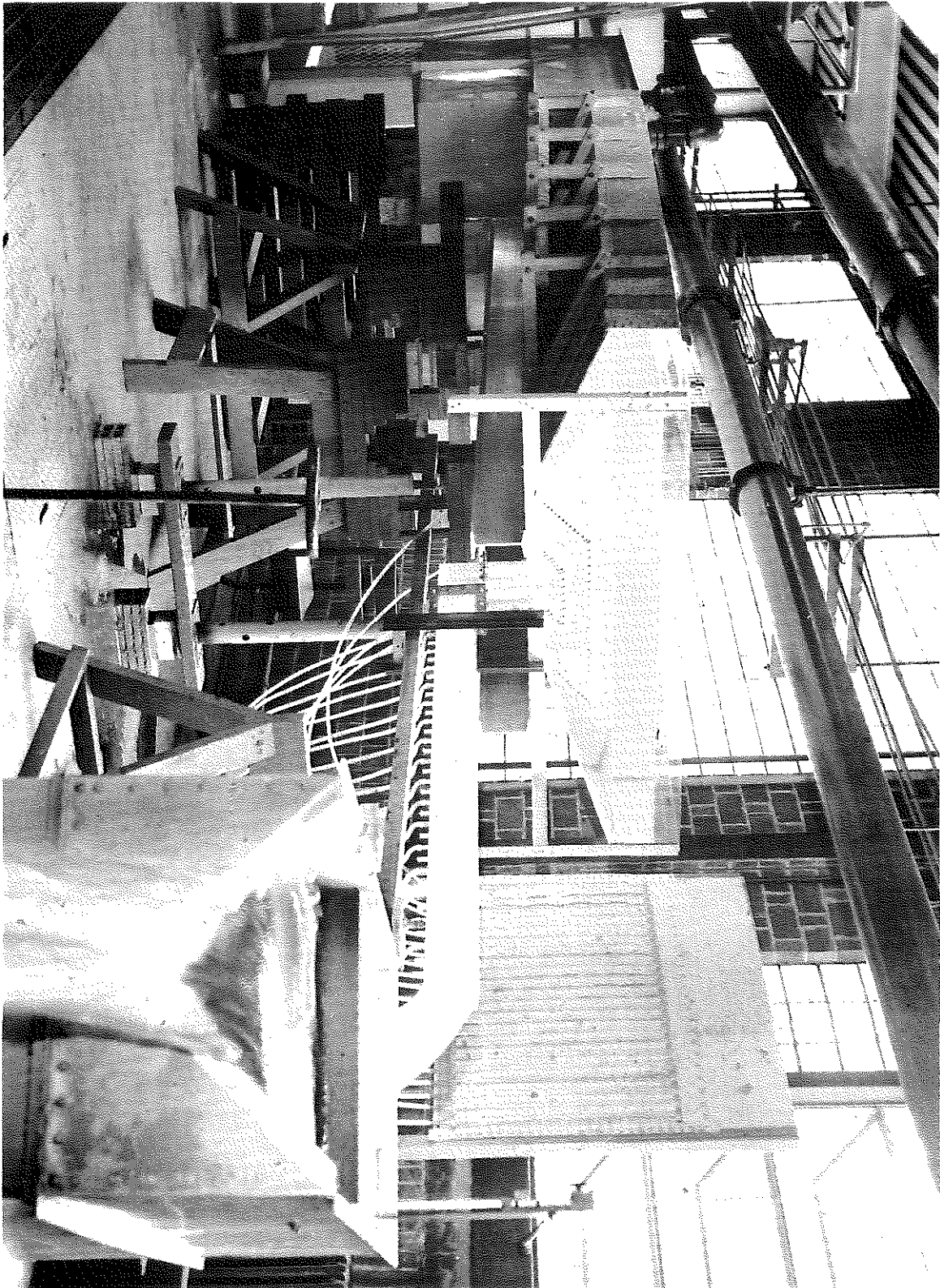


Fig. 9. Model in Operation.

drop-section would be vertical when attached to the approach channel at the designated slope. The inlets, one of which is shown in Fig. 10, were designed so that the upstream wall of the drop-section would be flush with the bottom of the approach channel. The headwall extended two inches above its juncture with the embankment portion of the end-section. The sidewalls were constructed to the same elevation as the headwall. Four inlets were constructed -- one each for channel slope 0, 3, 5 and 7 percent. The lower portion of each inlet was flanged for connection to an elbow or drop-extension. The elbows are shown in Fig. 11. The centerline-intersection in each was 91 degrees. The extension-sections are shown in Fig. 12. These extensions were simple, 4- x 4-inch sections, 2-1/2 and 5-1/2 inches in length -- giving a total drop of 9 and 12 inches, respectively. A drop of 6-1/2 inches was obtained with the elbow connected directly to the inlet.

The 4- x 4-inch barrel section, 72 inches in length, was used throughout these tests. Piezometer tubes were attached along the bottom of the barrel and elbow, at 2-inch intervals, and were connected to the manometer board. One tube was attached at the top of the elbow in order to measure the pressure at the bend.

Test Procedure

The desired slopes of the approach channel and barrel were set by screw jacks and checked by means of a level. Tests were conducted at headwater elevations ranging from approximately one inch to 15 inches

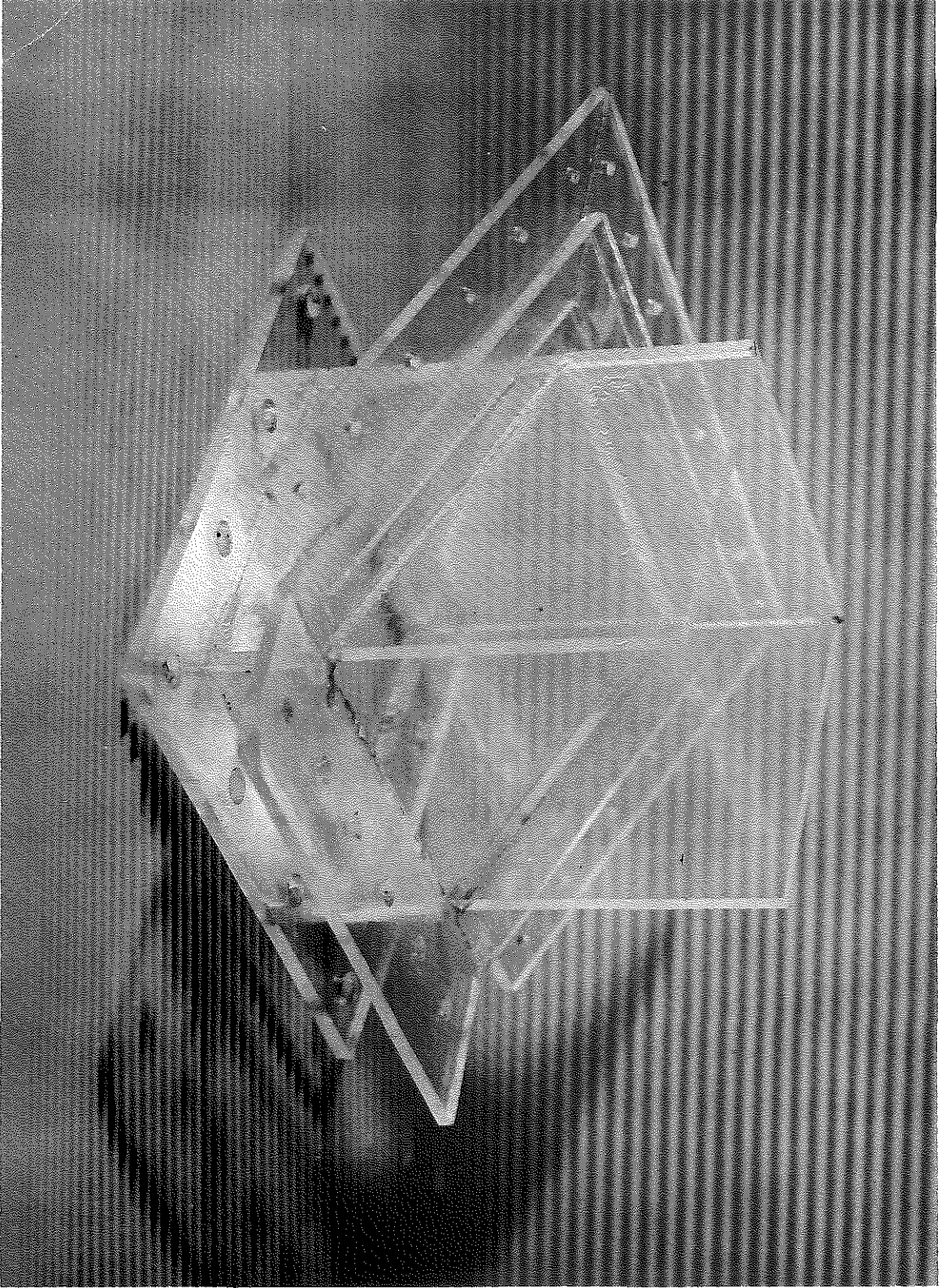


Fig. 10. Inlet Section.

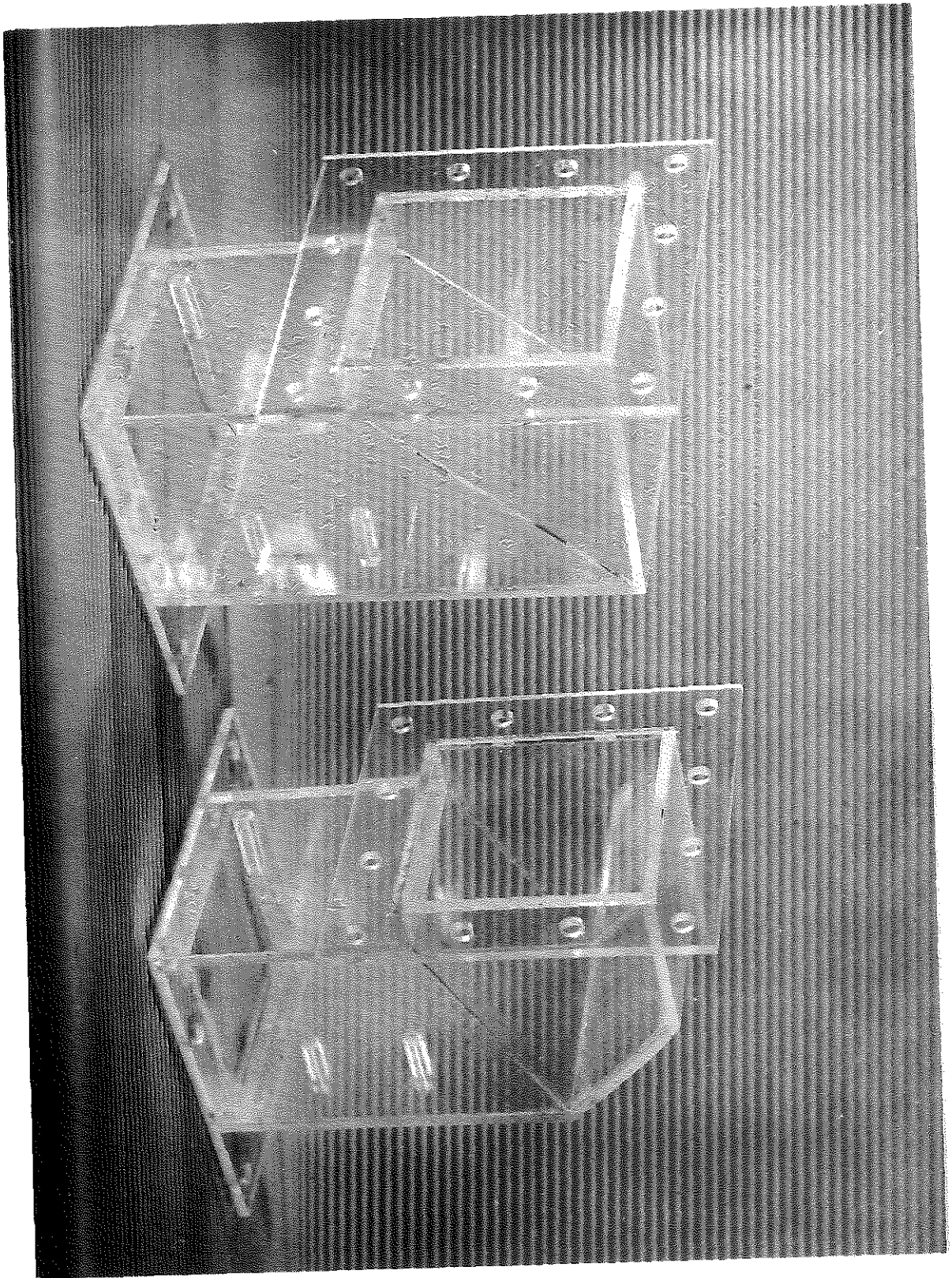


Fig. 11. Elbow Sections.

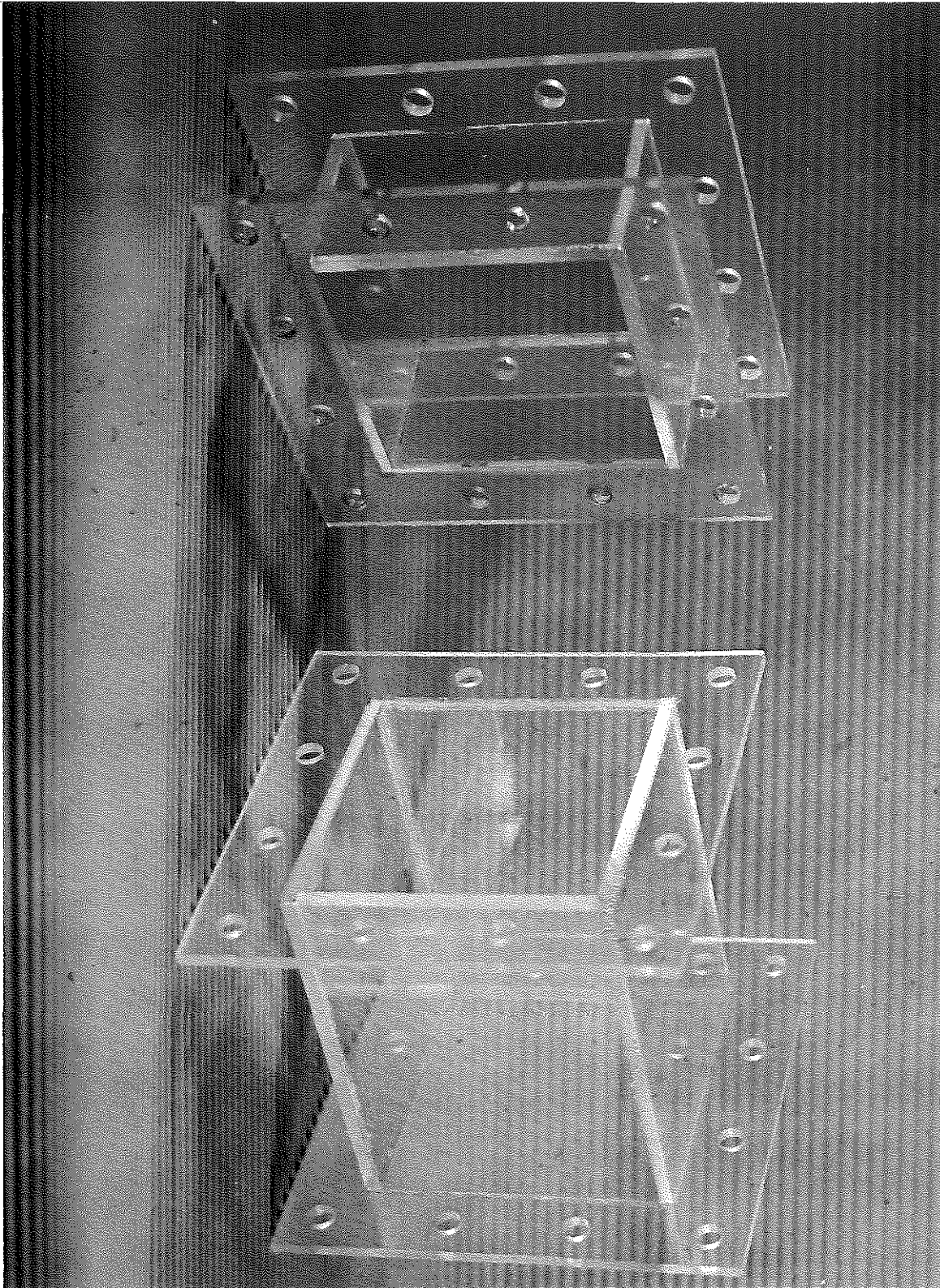


Fig. 12. Drop-Section Extensions.

and at intervals of approximately two inches. Headwater elevations were controlled by a valve in the piping system. A 15-minute period was allowed after final adjustment of the valve in order to permit the flow to reach a steady state. Headwater elevations were then measured at the inlet, and the discharge was determined by use of a hook-gage in the weir tank. Photographs were made of the manometer boards to record the attendant pressures. Notes were made as to type of flow in the approach channel and barrel.

RESULTS AND ANALYSES

The complete series of tests included model assemblies consisting of 6-1/2-, 9-, and 12-inch drops, approach-channel slopes of 0, 3, 5 and 7 percent, and the two elbows. Twenty-four installations were simulated through use of all combinations; thus comparative data were obtained for each variable as well as the specific data needed to calculate head-loss coefficients.

Head-discharge curves were plotted from the raw data. These curves proved to be insignificant inasmuch as the points were widely scattered for headwater elevations of three inches above the bottom of the approach channel. This was attributed to the control alternating from orifice to short-tube-flow to pipe-flow and thereby, altering the basic head-discharge relationships. Results were thus analyzed from the standpoint of the coefficients in the basic formulas for various types of flow through drop-inlets. Ranges in headwater elevation for control to occur in various sections were determined from observation of flow patterns during testing. These heads and their respective discharges were used in the calculations of coefficients for various types of control. The resulting coefficients are listed in the following table and are designated C_w , C_o , C_{st} and C_p for the weir-, orifice-, short-tube-, and pipe-flow coefficients respectively.

Flow was controlled by the inlet acting as a weir at headwater elevations of zero to three inches. Values of C_w were obtained

by substituting head-discharge values into equation 4. C_w was found to be constant for variable lengths of drop and for each elbow -- since control was upstream of the drop-section. Values of C_w were found to increase with the slope of the approach channel. This is attributed to an attendant increase in the approach velocity which was assumed to be negligible and thus not taken into account in the basic weir formula.

DISCHARGE COEFFICIENTS

Channel Slope (%)	Length of Drop (ins.)	Coefficients			
		Weir C_w	Orifice C_o	Short Tube C_{st}	Pipe C_p
0	6-1/2	2.78	3.61	3.50	2.62
0	9	2.78	3.62	3.66	2.59
0	12	2.79	3.62	3.46	2.46
3	6-1/2	3.04	3.71	3.64	2.59
3	9	3.03	3.71	3.83	2.53
3	12	3.05	3.72	4.06	2.43
5	6-1/2	3.30	3.78	3.76	2.56
5	9	3.31	3.80	4.04	2.50
5	12	3.30	3.79	4.30	2.43
7	6-1/2	3.55	3.86	3.93	2.50
7	9	3.57	3.87	4.23	2.43
7	12	3.56	3.86	4.51	2.37

Analysis for orifice-control was similar to that for weir-control. No appreciable variation in C_o was noted for the various lengths of drop. Values of C_o increased as the channel-slope increased -- again due to an increase in velocity-head, which is considered negligible in the basic orifice formula. The variation in C_o with changes in slope

were not as great as that for C_w because discharge under orifice-control occurs at greater headwater elevations and thereby, decreases the overall effect in variation of approach velocity.

The coefficients, C_{st} , as determined for short-tube-flow, vary more or less directly with the length of drop for a constant approach-channel slope. Increases in C_{st} with greater lengths of drop may be attributed to gain in head. The gain in head more than compensates for the increased friction.

Pipe-flow coefficients, C_p , decreases with increasing lengths of drop. In reference to equation 10, it may be seen that C_p decreases as Q increases; thus, C_p decreases with increasing drop, and this represents a gain in head over the loss due to friction.

Both C_{st} and C_p increased for increased slopes. Here, again, the increase is due to greater velocities of approach at the higher slopes.

In all cases the effect of the type of elbow was found to be negligible in that the head-discharge relationships for the two were almost identical. For this reason, data obtained from tests using each elbow were combined in the analysis for effect of slope and length of drop. The combined head-discharge curves are shown in Figs. 13, 14 and 15. The continuous portion of these curves were plotted from the test results; whereas, the dashed portions represent theoretical extrapolations which are based upon discharge coefficients determined from tests. The headwater elevations are shown as H_w and H_o . The datum for short-tube-flow was taken to be the break-point between the

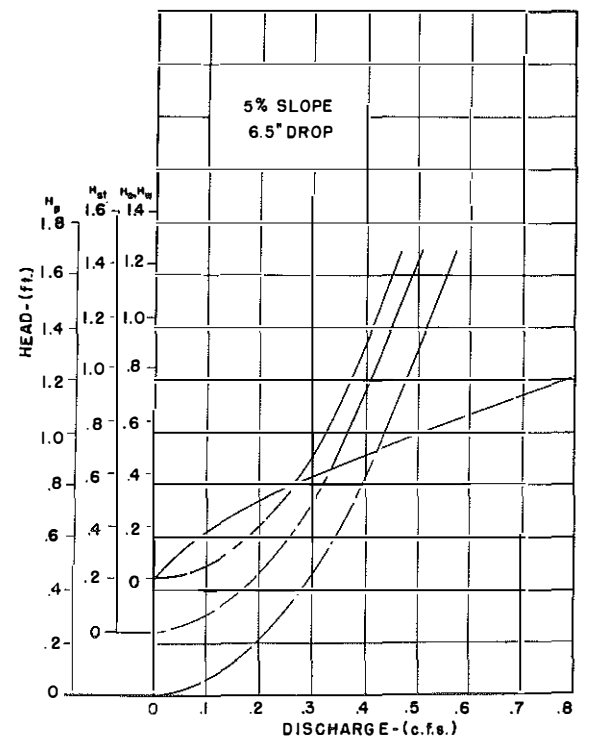
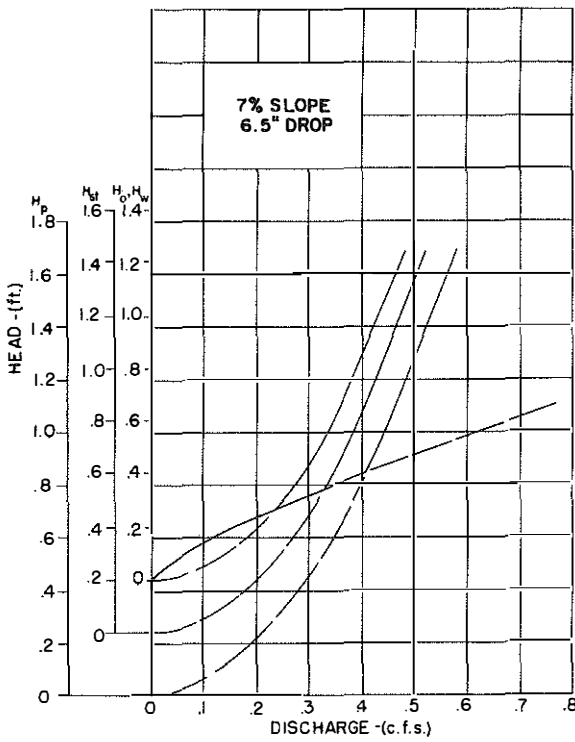
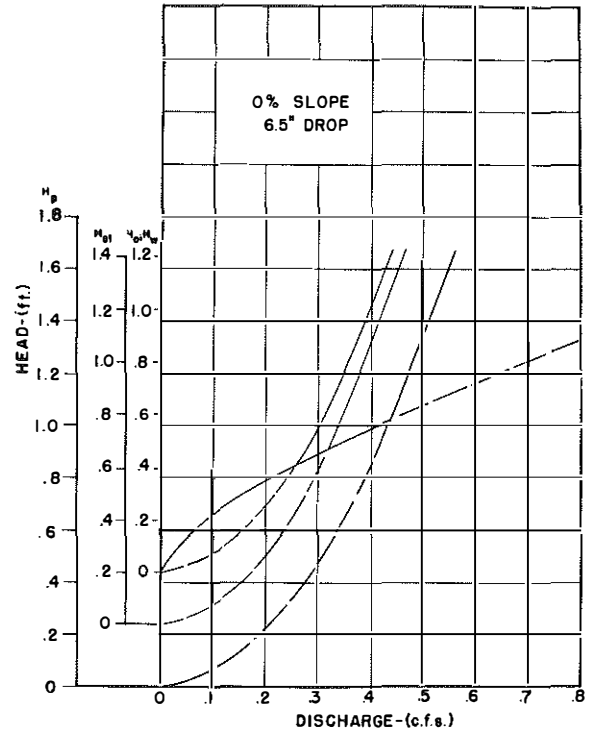
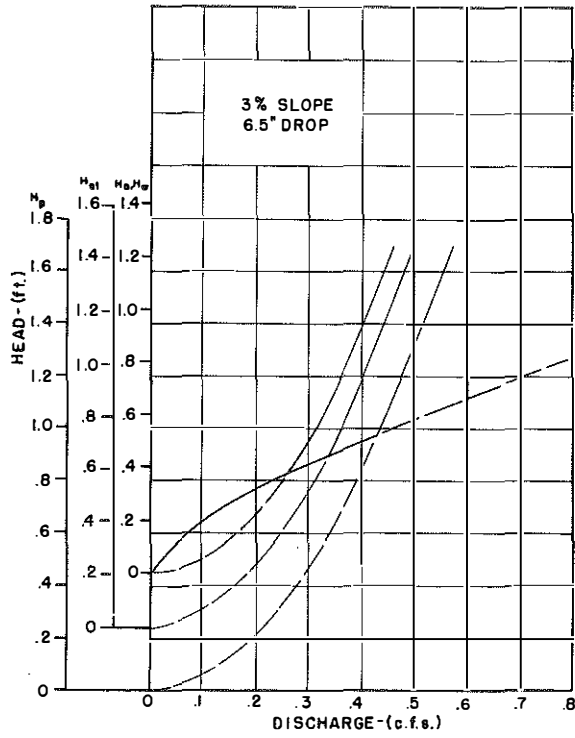


Fig. 13. Combined Head-Discharge Curve for 6.5-inch Drop.

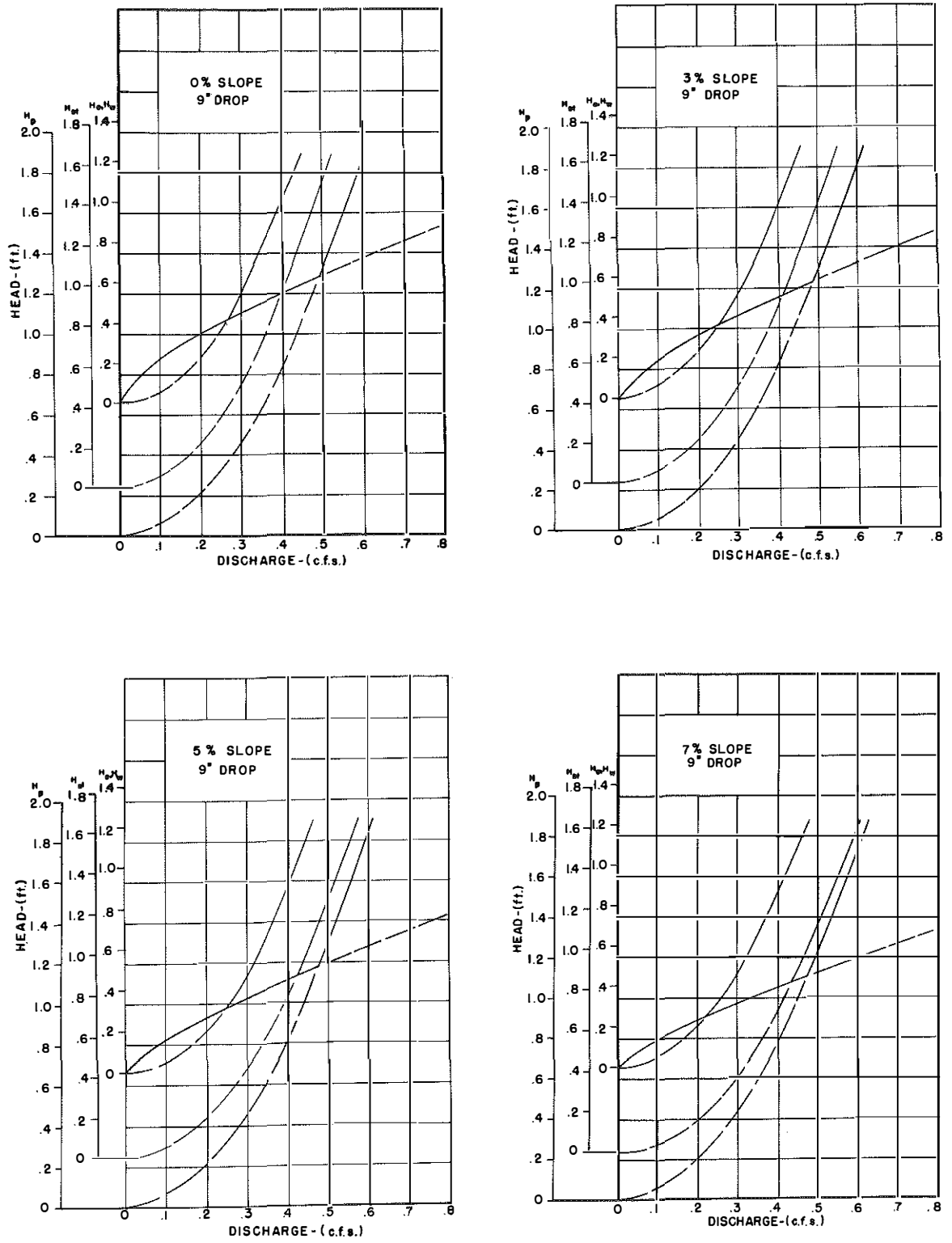


Fig. 14. Combined Head-Discharge Curves for 9-inch Drop.

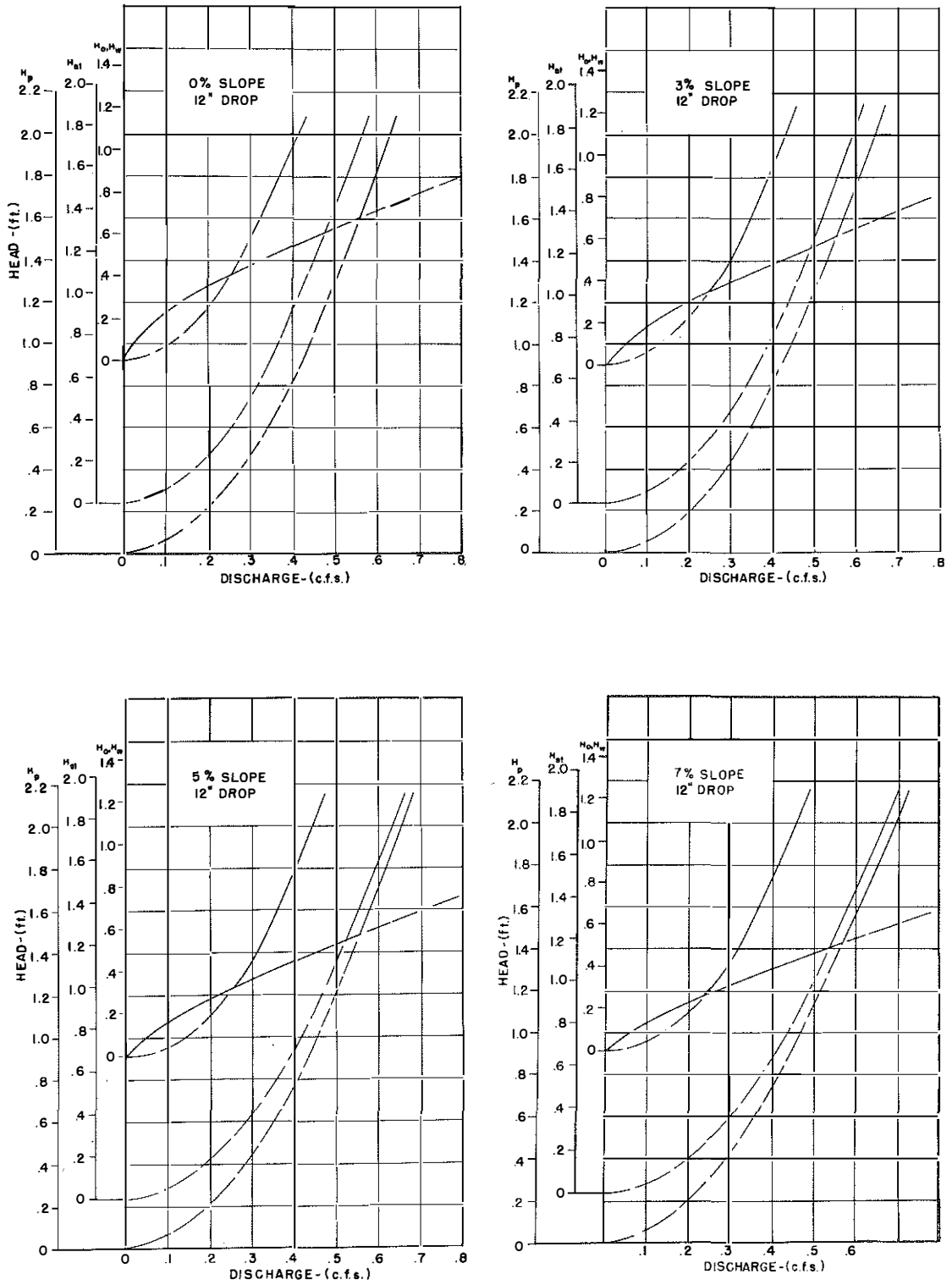


Fig. 15. Combined Head-Discharge Curves for 12-inch Drop.

drop-section and barrel and is below the weir- and orifice-datum, a distance equivalent to the length of the drop-section minus the height of the barrel. The datum for pipe-flow was taken to be the centerline of the barrel at the outlet and is thus plotted below the weir and orifice-datum by a distance equivalent to the difference in the elevation of the bottom of the approach channel and the centerline of the outlet.

Head-discharge curves for the 6-1/2-inch drop-section and channel slopes of 0, 3, 5 and 7 percent are shown in Fig. 13, and curves for the 9- and 12-inch drop-sections are shown in Figs. 14 and 15 respectively. Flow was observed to be controlled by the inlet acting as a weir at headwater elevations from zero to approximately three inches. Above three inches, the weir became flooded and the control switched to orifice-flow. Orifice-control persisted through heads of seven inches.

Short-tube-flow prevailed at headwater elevations of seven to eight inches. Short-tube-flow is a residual-type flow which occurs as a transition between orifice- and pipe-flow. Flow of this nature is most difficult to predict and generally persists for only a short period of time. Pipe-flow prevailed at headwater elevations of eight to 15 inches.

Figures 16 and 17 are head-discharge curves for weir- and orifice-flow, respectively, for the various slopes. Each curve, for a specific slope, represents the combined results from the three drop-lengths -- length of drop did not affect the head-discharge relationships.

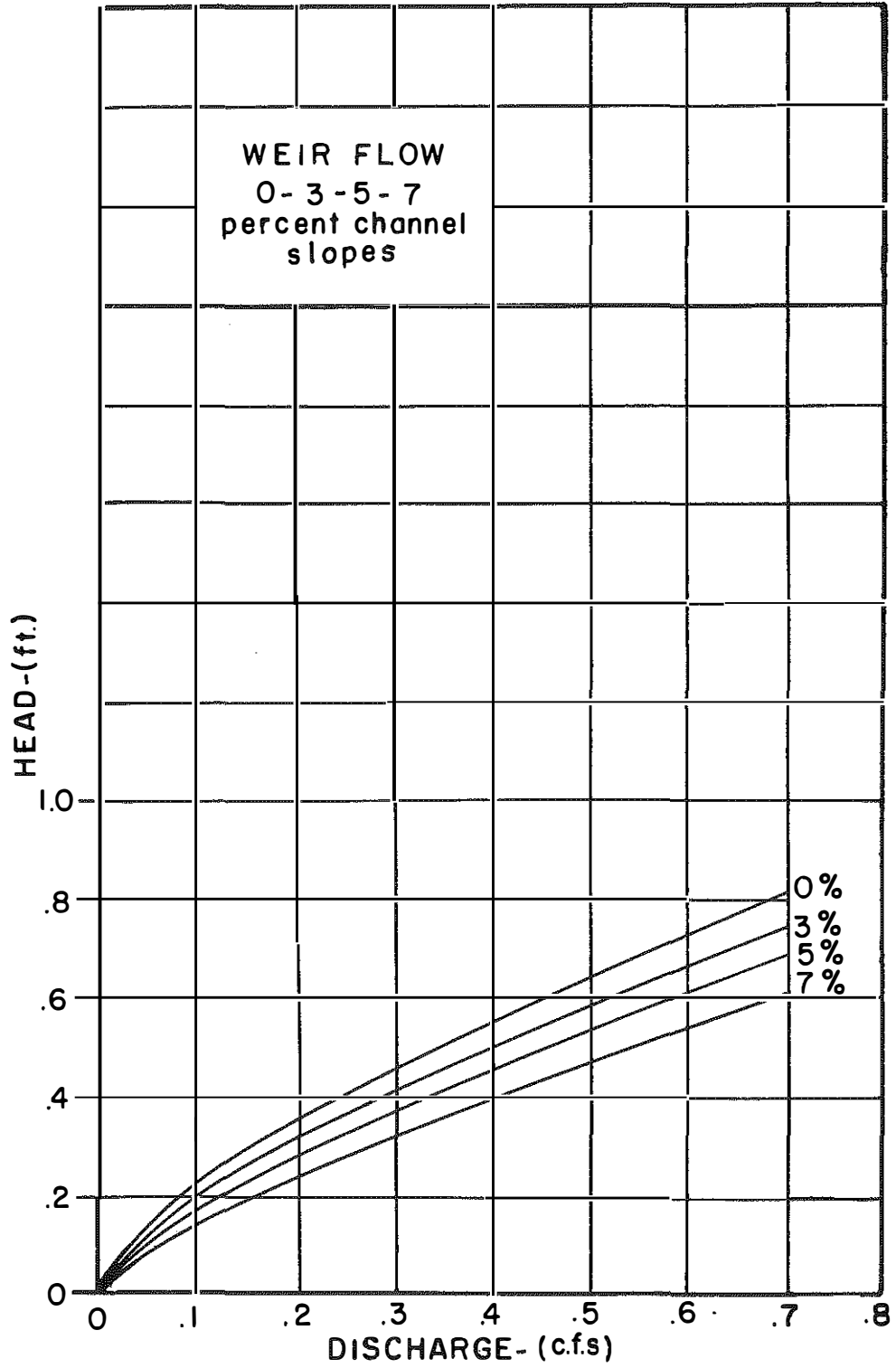


Fig. 16. Head-Discharge Curves for Weir-Flow.

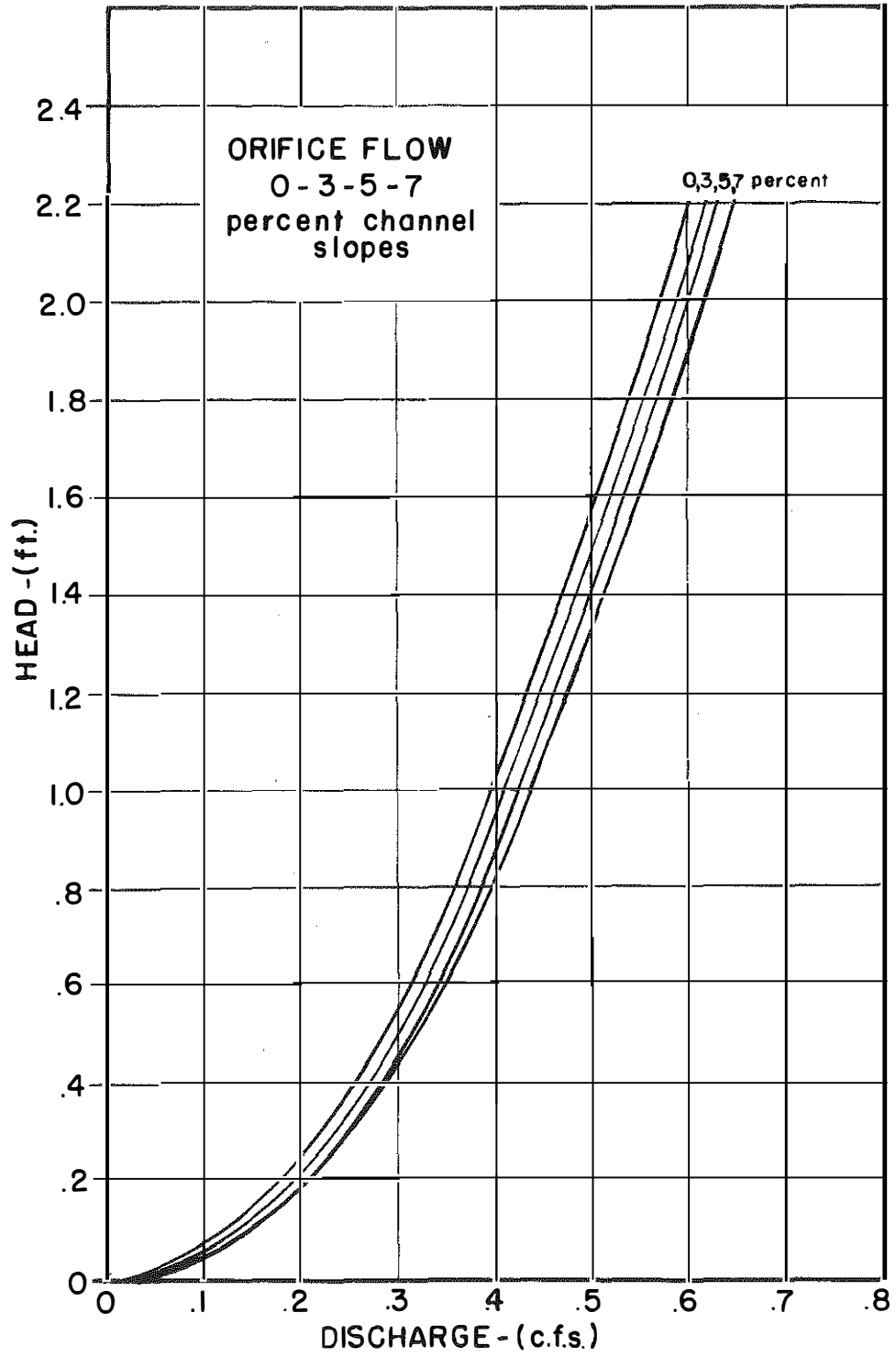


Fig. 17. Head-Discharge Curves for Orifice-Flow.

Increases in slope increased the approach velocity and the discharge at a given headwater elevation.

Head-discharge curves for short-tube- and pipe-flow are shown in Figs. 18 and 19, respectively. The curves are grouped to show effects of lengths of drop. It may be noted that the effect of length of drop is greater for short-tube-flow than for pipe-flow. This may be explained by the fact that the head-loss due to velocity is greater in the case of pipe-flow than in short-tube-flow.

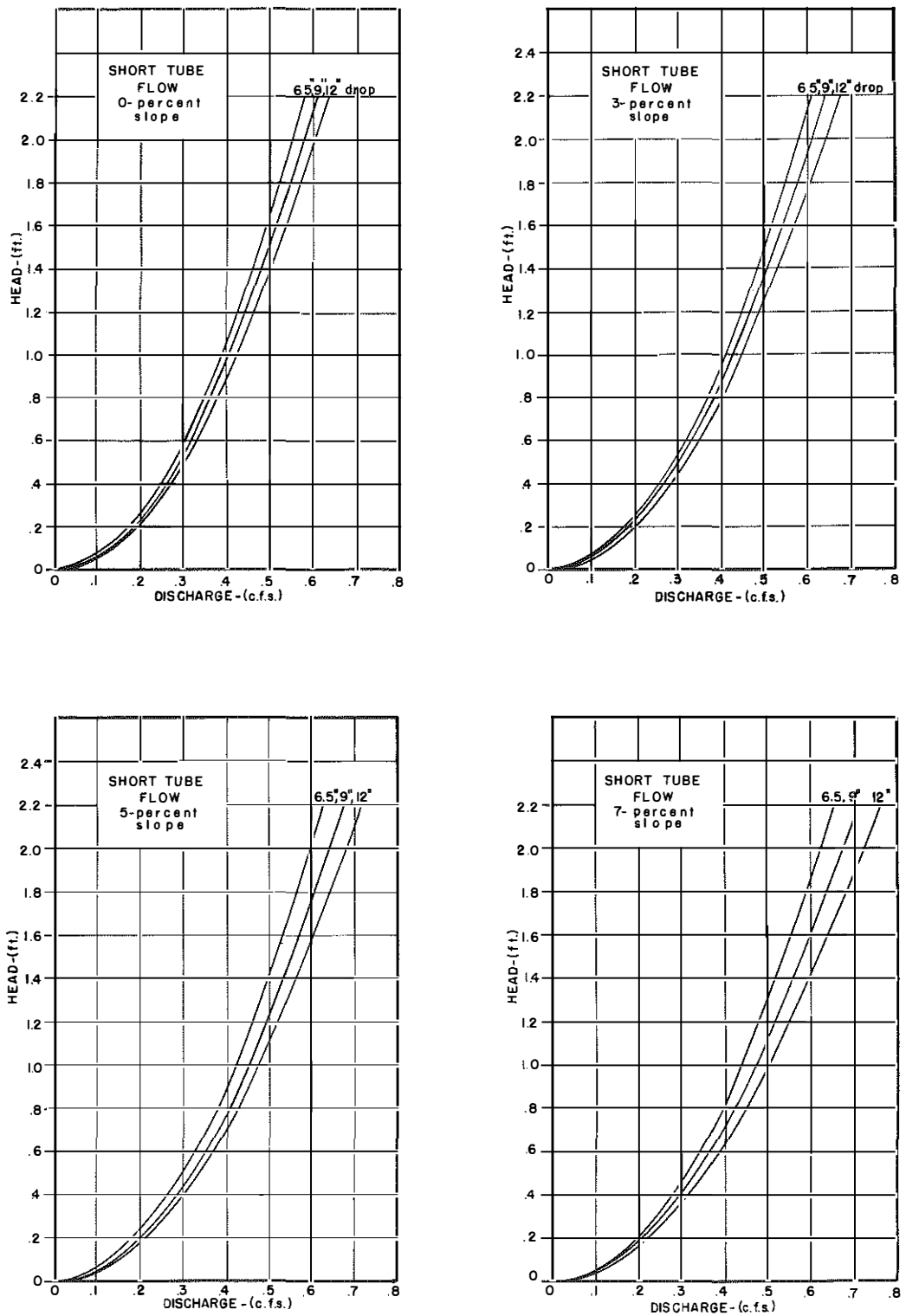


Fig. 18. Head-Discharge Curves for Short-Tube-Flow.

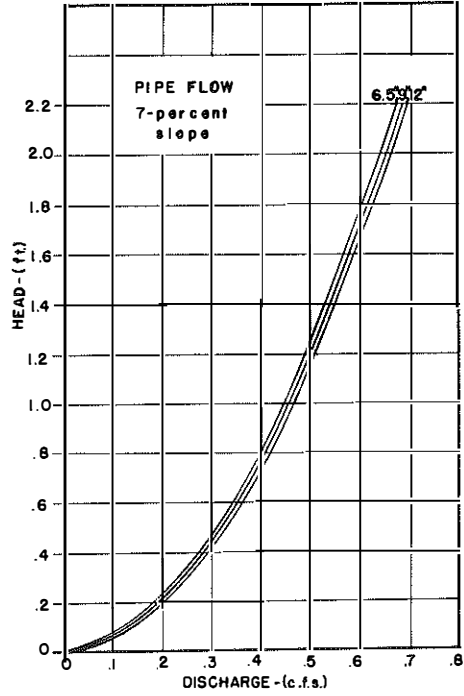
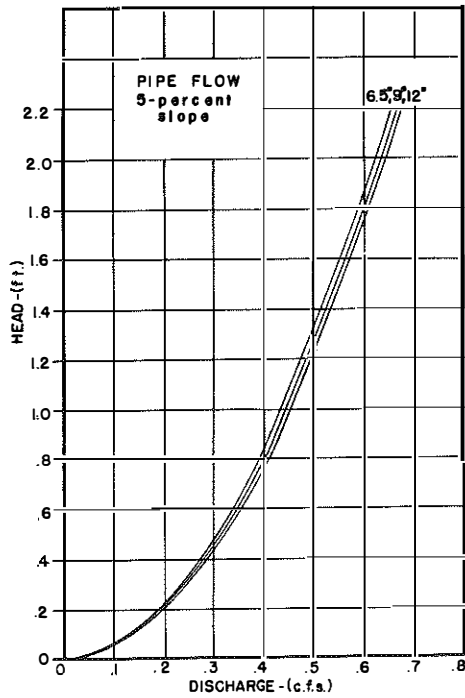
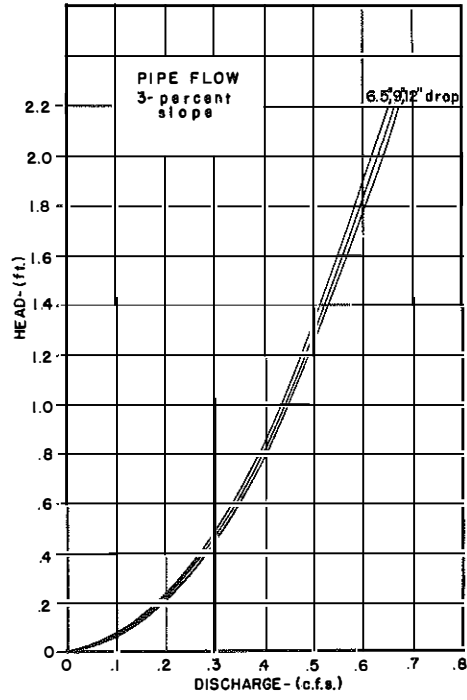
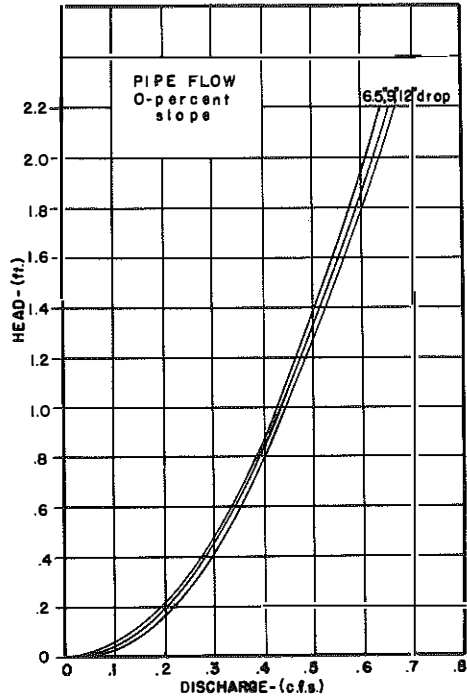


Fig. 19. Head-Discharge Curves for Pipe-Flow.

APPLICATION OF DATA

Prediction of flow through full-scale structures may be made by dimensional analysis and hydraulic similitude. Basic relationships used in predicting flow through the prototype culvert (the full-scale equivalent of the model) from model studies were reported previously (5) (6). There are three types of similarity to be considered: 1) geometric, 2) kinematic, and 3) dynamic; they refer to similarity of form, motion, and forces acting within and upon the structure.

Since it is seemingly impossible to control the forces acting upon the fluid masses, complete similarity is never attained; thus, dimensional analysis must be applied.

All dimensional units of the model are 1/12th those of its corresponding full-scale prototype, and thus the corresponding headwater elevations for the prototype are given by: 1 inch = 1 foot, since a scale modulus of 1:12 was used in construction of the model. An approximate conversion factor for determining the discharge through the prototype under conditions of weir-control may be found as follows:

$$\frac{Q_{(p)}}{Q_{(m)}} = \frac{C_{w(p)} L_{(p)} H_{w(p)}^{3/2}}{C_{w(m)} L_{(m)} H_{w(m)}^{3/2}}$$

where (p) and (m) subscripts denote prototype and model values respectively. The similitude relationships between $L_{(p)}$ and $L_{(m)}$, and $H_{w(p)}$ and $H_{w(m)}$ are: $L_{(p)} = 12 L_{(m)}$, and $H_{w(p)} = 12 H_{w(m)}$; thus, the

equation becomes:

$$\frac{Q(p)}{Q(m)} = \frac{C_{w(p)} 12 L(m) (12 H_{w(m)})^{3/2}}{C_{w(m)} L(m) (H_{w(m)})^{3/2}}$$

$$\frac{Q(p)}{Q(m)} = (12)^{5/2} \times \frac{C_{w(p)}}{C_{w(m)}}$$

The factor $\frac{C_{w(p)}}{C_{w(m)}}$ becomes unity since $C_{w(p)}$ and $C_{w(m)}$ are dimensionless and are nearly equal. $Q(p)$ is thus $(12)^{5/2}$ times $Q(m)$ or approximately 500 $Q(m)$. Similar relationships for orifice, short-tube, and pipe control may be developed from their respective basic equations.

As noted above, the coefficients, $C_{w(p)}$ and $C_{w(m)}$ were dimensionless and were assumed to be nearly equal; hence, the ratio of $Q(m)$ to $Q(p)$ is wholly dependent upon the scale modulus. Alternatively, then the use of the respective coefficients in the basic equations and in conjunction with full-scale dimensions would be a means whereby full-scale design calculations may be made from results of the study.

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