FLOW CHARACTERISTICS AND PERFORMANCE OF BROKEN-BACK TRANSITIONS (OUTLETS)

LIDSTER

THESIS
1960

May 12, 1960

TO WHOM IT MAY CONCERN:

The thesis "Flow Characteristics and Performance of Broken-Back Transitions (Outlets)" is the result of my own work in substance, form and expression.

William A. Ludeke

I typed the above thesis, but did not participate in the collection or organization of the material or in the writing of the thesis.

Louise H Ball

FLOW CHARACTERISTICS AND PERFORMANCE OF BROKEN-BACK TRANSITIONS (OUTLETS)

by

William Albert Lidster

B.S., Missouri School of Mines and Metallurgy, 1955

A Thesis submitted to the Faculty of the Graduate School of the University of Colorado in partial fulfillment of the requirements for the Degree

Master of Science

Department of Civil Engineering

1960

This Thesis for the M.S. degree, by

William Albert Lidster

has been approved for the

Department of

Civil Engineering

by

_____

_____

Date_____

Lidster, William A. (M.S., Civil Engineering)

Flow Characteristics and Performance of

Broken-Back Transitions (Outlets)

Thesis directed by Assistant Professor J. Ernest Flack

Due to rising costs of design and construction it has become necessary to determine an economical yet effective means of providing a transition for flow of water from conveyance structures to canals and vice versa. A solution to this problem is to modify the complicated warped transition to a more easily constructed broken-back transition.

Hydraulic Laboratory studies were made on broken-back transitions with 12-inch diameter conduits to evaluate the flow characteristics and performance of this type of structure. It is anticipated that the performance of the model can be duplicated in prototypes of various sizes by applying the laws of similitude.

As a means of defining the flow characteristics, the coefficient of energy loss was used throughout this study. A comparison of this coefficient gave definite results as to the affect of the various conditions under which the broken-back transition might operate. It was found that the flow characteristics were influenced by: (1) the angle of divergence of the water surface with the center line of the structure; (2) upstream submergence of the inflow conduit; and (3) slope of the inflow conduit.

The results of this study will give the designers of hydraulic structures a more realistic value of energy losses through the broken-back transitions.

This abstract of about 210 words is approved as to form and content. I recommend its publication.

Signed_____
Instructor in charge of dissertation

CONTENTS

LIST OF FIGURES

LIST OF FIGURES--Continued

ACKNOWLEDGMENT

The author wishes to express his appreciation, to the Bureau of Reclamation, Denver, Colorado, for granting permission to use the material presented in this thesis; to James W. Ball and William P. Simmons, Jr., of the Bureau of Reclamation, Denver, Colorado, for their valuable suggestions and criticisms.

The work described in this thesis was performed by the author in the Hydraulic Laboratory of the Bureau of Reclamation, of the United States Department of the Interior, Denver, Colorado. The laboratory is under the supervision of Harold M. Martin, Head of the Hydraulic Laboratory.

FLOW CHARACTERISTICS AND PERFORMANCE OF BROKEN-BACK TRANSITIONS (OUTLETS)

INTRODUCTION

For several years the designers of water conveyance structures have been concerned that the coefficients used in the evaluation of energy losses through the outlets of siphons using broken-back transitions give losses less than actually occur. Use of low coefficients of loss for the design may result in the transition structure becoming a restriction in the canal system. Due to lack of accurate data on the broken-back outlet transition, present design practice assumes a head loss of 0.3 of the velocity head change that occurs as the flow passes from the conduit through the transition into the canal. This value is added to friction loss. Transition losses obtained in this manner are in close agreement with Hinds.[1] To obtain this head loss the transition must expand the stream flow so that the velocity distribution is near that which will exist in the canal. The transition must also be of such shape that the flow follows the walls and provides the necessary drop in free water surface to overcome friction, turbulence, and entrance losses. Tests of models by Benson[2] indicate that the redistribution of flow can be accomplished in a very short distance by placing a submerged hump or obstruction in the floor of the transition.

The determination of the flow characteristics and performance of the broken-back transition may be made by use of hydraulic model

---

[1]Hinds, Julian, "The Hydraulic Design of Flume and Siphon Transitions," Transactions of the American Society of Civil Engineers, 1928, pp. 1423-1459.

[2]Benson, M. H., "Model Tests of Outlet Transitions," Civil Engineering, Vol. 6, 1936, p. 760.

studies. The slope of the incoming pipe, degree of submergence of this pipe crown at the headwall, and the rate of divergence of the sidewalls will affect the flow characteristics and performance.

It is important to designers of water conveyance structures and the water users that the influence of these factors be accurately determined so that the most economical and practical structures may be built.

PURPOSE

This thesis covers hydraulic laboratory studies made to define the flow characteristics and performance of the broken-back type transition used as an outlet structure. The tests were performed on broken-back transitions with sidewalls diverging at angles of 20, 25, and 30 degrees relative with the center line, Figure 3. The information is intended for use as reference material in the hydraulic design of broken-back transitions.

THE BROKEN-BACK TRANSITION

The broken-back transition, Figure 3, was initially used for small pipe culverts of secondary importance. These structures were designed by adaptation from successful structures operating under similar conditions. The simplicity of design and low construction costs have resulted in the extensive use of the broken-back transition by the Bureau of Reclamation in water distribution systems. Through observations the performance of this type of structure seems to be quite satisfactory.

THE THEORETICAL ANALYSIS

At present there is no satisfactory theory for computing the coefficient of energy loss for water flowing through transitions. This coefficient can only be determined experimentally by using the difference between the total head at each end of the transition.[3] The coefficient of energy loss, K, was determined for each test by dividing the difference in the elevation of the energy grade line, ΔH_e, by the difference in velocity head, $\Delta V^2/2g$, as measured upstream and downstream from the transition.

For the material in this study to be utilized for transitions of various sizes, it was desirable to present it in dimensionless form. In this study the loss coefficients were based on the change in the elevation of the energy grade line from a point one diameter upstream from the transition to a point 3.8 channel surface widths downstream from the transition. This length was obtained by observation as the location where the velocity redistribution caused by the transition was essentially complete.

The coefficient of energy loss can be obtained in dimensionless form as follows:

---

[3] Formica, G., "Esperienze preliminari sulle perdite di carico nei canali, dovute a cambiamenti di sezione," in L'Energia Elettrica (Milano), Vol. 32 (7), July 1955, pp. 554-567.

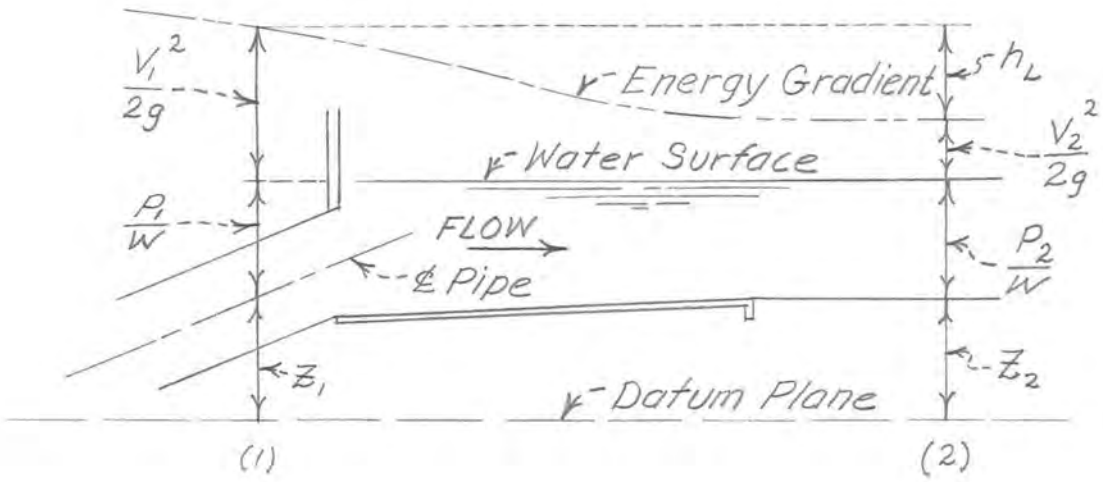


FIGURE 1

THE BERNOULLI EQUATION FROM (1) TO (2) THROUGH THE TRANSITION

$$Z_1 + \frac{P_1}{w} + \frac{V_1^2}{2g} = \frac{P_2}{w} + \frac{V_2^2}{2g} + Z_2 + h_L$$

$$Z + \frac{P}{w} + \frac{V^2}{2g} = \text{Energy head } H_e$$

$$H_{e_1} = H_{e_2} + \text{Loss}$$

$$H_{e_1} - H_{e_2} = \text{Loss}$$

$$\Delta H_e = \text{Loss}$$

This loss, in feet of water, may be expressed as some factor, K, times ΔH_v or

$$\text{Loss} = K\left(\Delta H_v\right)$$

$$\text{Loss} = K\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right)$$

then

$$H_{e_1} - H_{e_2} = K\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right)$$

or

$$K = \frac{H_{e_1} - H_{e_2}}{\frac{V_1^2}{2g} - \frac{V_2^2}{2g}} = \frac{\Delta H_e}{\Delta H_v}$$

It was concluded that the coefficient of loss hereafter referred to as K was influenced by the following variables as defined in Figure 4: (1) the angle of divergence of the sidewalls from the center line of structure, θ; (2) upstream submergence of inflow pipe crown in terms of pipe diameter, d/x; (3) slope of inflow pipe, α ; (4) the variation in velocity head between Points 1 and 2, $\Delta V^2/2g$, and (5) the mean velocity in the pipe, V. The variables, θ, d/x, α , $\Delta V^2/2g$, and V, were put into dimensionless parameters with this resulting relationship:

$$K = \phi\left(\theta, \frac{d}{x}, \alpha, \frac{\Delta V^2}{2g}, H_F\right)$$

The parameter, H_F, is the Froude number and is equal to:

$$H_F = \frac{V}{\sqrt{gd}}$$

where:

V = mean velocity in the pipe, ft/sec

d = nominal diameter of inflow pipe, ft

g = gravity, 32.2 ft/sec^2

The advantages of using N_F rather than V or Q is that it takes gravitional and inertia forces into consideration, and it is a convenient parameter to utilize the laws of similitude.

The coefficient of loss K determined by this investigation may be used in all size prototypes, provided the Froude number of the prototype is within the limitations of this investigation. This may be proved as follows:[4]

Geometric similarity between two systems or bodies exists when the ratios between all corresponding linear dimensions are equal. Length similarity can be expressed as follows

$$L_r = \frac{L_p}{L_m}$$

where L_r denotes the length ratio, L_p and L_m are corresponding linear dimensions in prototype and model.

Kinematic similarity exists in two geometrically similar systems when the ratios of velocity and acceleration of all homologous

---

[4] Liu, H. K. and Albertson, M. L., "Significance and Application of Froude and Reynolds Numbers as Criteria for Similitude." Paper presented at the Hydraulic Conference of the American Society of Civil Engineers, Fort Collins, Colorado, July 1959.

particles are equal throughout the systems. The flow will then be kinematically similar and the paths of the homologous particles will also be geometrically similar.

Dynamic similarity is the ratio of all homologous forces required to maintain the geometric and kinematic similarity in the two related systems, model to prototype, and must be the same at each homologous point. These active forces are: force due to pressure variation, force due to gravity, viscous shear force, force due to surface tension, and the force resulting from elastic compression. The forces due to surface tension and elastic compression are relatively small and can be safely neglected. Equating these active forces to the force of inertia may be accomplished by applying Newton's second law of motion. Newton's second law of motion can be written:

$$\text{Vector sum of forces} = -Ma$$

$$F_p + F_g + F_v = -Ma$$

or

$$F_p + F_g + F_v + F_i = 0$$

where

F_p = force due to pressure variation

F_g = force due to gravity

F_v = viscous shear force

F_i = the inertia force = $-Ma$

When similarity in two fluid motions occur, the ratio of the inertia forces between the model and prototype must equal the ratio

of the vector sum of the active forces between the model and prototype. This force ratio may be shown by the vector polygon, Figure 2.

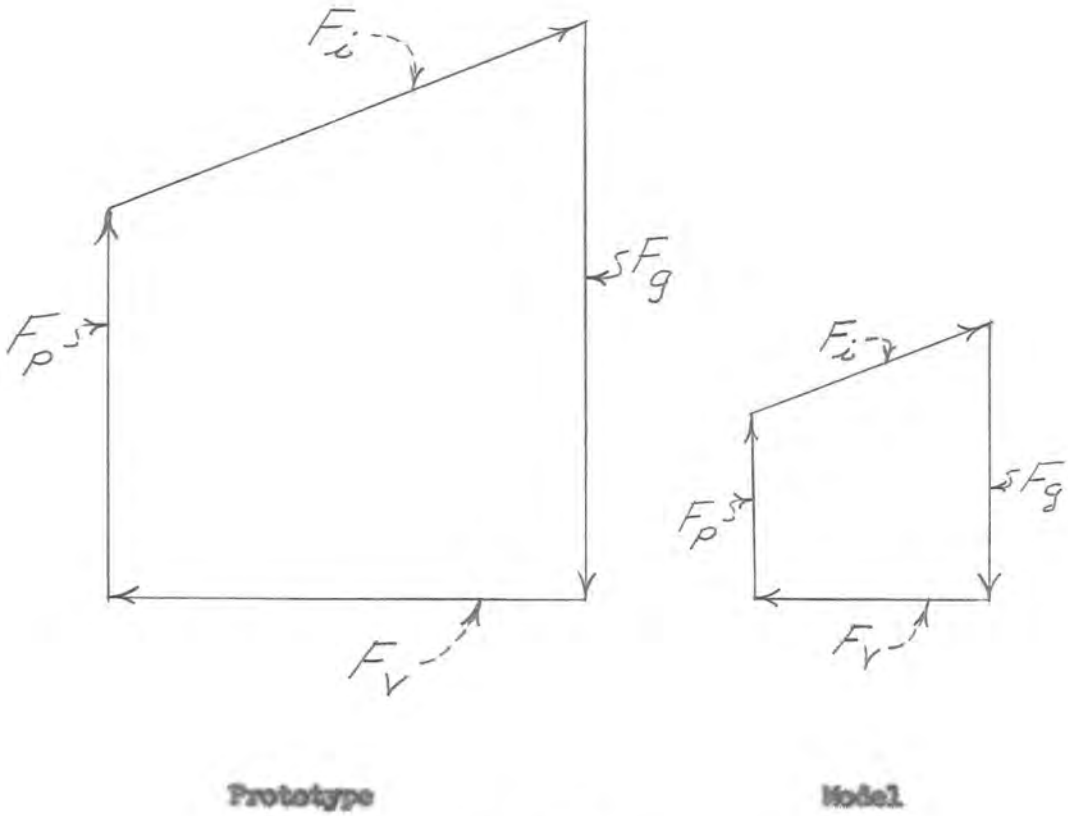


FIGURE 2

RATIO OF THE VECTOR SUM OF THE
ACTIVE FORCES BETWEEN THE MODEL AND PROTOTYPE

From the similar polygons in Figure 2 it may be shown that if the ratio between the force due to gravity, F_g, viscous shear force, F_v, and the inertia force, F_i, are the same, then the ratio of the force due to pressure variation must also be the same.

$$L_r = \frac{(F_g)p}{(F_g)m} = \frac{(F_v)p}{(F_v)m} = \frac{(F_p)p}{(F_p)m} = \frac{(Ma)p}{(Ma)m} = \frac{(F_i)p}{(F_i)m}$$

then

$$L_r = \frac{(V_1 - V_2)p}{(V_1 - V_2)m}$$

and

$$L_r = \frac{(H_{e_1} - H_{e_2})p}{(H_{e_1} - H_{e_2})m}$$

or

$$L_r(\Delta H_e)m = (\Delta H_e)p$$

$$L_r(\Delta V)m = (\Delta V)p$$

by definition

$$K = \frac{(\Delta H_e)p}{(\Delta H_v)p} = \frac{L_r(\Delta H_e)m}{L_r(\Delta H_v)m}$$

hence

$$K = \frac{(\Delta H_e)p}{(\Delta H_v)p} = \frac{(\Delta H_e)m}{(\Delta H_v)m}$$

INVESTIGATION

The Laboratory Installation

The broken-back transitions shown in Figures 4 and 5 were tested at the Bureau of Reclamation Hydraulic Laboratory, Denver Federal Center, Denver, Colorado. Flow was provided to the installation by a 12-inch, 100-horsepower centrifugal pump capable of delivering 4,750 gallons per minute and a pressure of 103 feet of water. The rate of flow was measured by a 6-inch Venturi meter located near the pump. To obtain the water pressure in the inflow conduit one diameter upstream from the transition, pressure taps (piezometers) were installed as shown in Figure 6. The resulting pressures were transmitted to a pot gage, Figure 7, by rubber tubing. Water levels in the pot gage could be read to 0.001 foot by means of a point gage equipped with a vernier scale. The water surface in the channel was measured by a point gage located 3.8 channel surface widths downstream from the outlet end of the transition. The point gages were correlated to the invert of the outlet end of the transition by use of a Dietzgen level. They were further checked by ponding the channel.

The three transitions used in this study were constructed using 3/4-inch plywood. All dimensions were in terms of pipe diameter, d. The conduit leading to the transition was a 12-inch steel pipe installed on a 2 to 1 slope, and horizontally, thus giving a good representation of the slopes on the prototype.

The channel bed consisted of ungraded sand with a base width of 1.667 d (20 inches) and side slopes of 1-1/2 to 1. The bed was

carefully shaped before each run by means of a template, Figure 8. Water level in the channel was kept at constant depths by means of a tail gate.

The Test Procedure

Water was pumped through the transition and the quantity of flow was adjusted and measured with a Venturi meter to give the desired velocity. The depth of upstream submergence was varied by adjusting the tail gate until the water surface in the canal was just in contact with the previously set point gage located downstream from the transition. Air was bled from the piezometer line that was attached to the conduit upstream from the transition. After allowing the flow to become steady the water surface in the pot gage was read by means of a point gage. Pot gage readings were made two or more times for each submergence and flow. The measurements obtained from the pot gage were correlated with the water surface in the canal to give a difference in pressure head. The respective velocity heads were added to the pressure heads thus giving energy heads. Dividing the difference in energy heads by the difference in velocity heads gave the coefficient of loss, Table 1. The piezometer lines were continually checked to guard against air pockets and possible leakage, and care was exercised to allow the flow to fully stabilize for each condition of flow and submergence.

The ranges of flow and submergence used for each transition and each slope of inlet pipe, i.e., horizontal and on a 2 to 1 slope, are as follows:

Velocity in Pipe	Upstream Submergence
2.00 fps	d/3
2.50 fps	d/3
3.00 fps	d/3
2.00 fps	d/6
2.50 fps	d/6
3.00 fps	d/6
2.00 fps	0
2.50 fps	0
3.00 fps	0

The coefficient of loss K was plotted against the Froude number for each condition, Figures 12 and 13, and also plotted against the change in velocity head ($\Delta V^2/2g$) for each condition, Figures 14 and 15.

The pipe velocities used in this investigation resulted in Reynolds numbers between 1.6×10^5 and 2.5×10^5. With Reynolds numbers of this magnitude the frictional resistance and turbulence in the model will be nearly the same as in the prototype and the model will predict prototype performance with reasonable accuracy by Froude criteria alone.

Influence of Angle of Divergence (θ)

The angle of divergence between the sidewalls and the axis of the structure of 20, 25, and 30 degrees resulted in a variation in the values of the coefficient of head loss, K. Where θ was equal to 25 degrees, low optimum values of K were obtained, Figures 14 and 15. From Figures 12 and 13 it was concluded that the value of K be 0.6

where the Froude number was greater than 0.45. Where the Froude number was less than 0.45, the value of K should be 0.5.

Influence of Outlet Submergence, d/x

The amount of submergence has a distinct affect on the values of K as seen in Figures 12 and 13. A submergence between zero and d/6 affects the value only slightly whereas a submergence of d/3 increases the value of K about 20 to 30 percent. In the design of the prototype a submergence of zero is commonplace.

Influence of Slope of Inflow Conduit, α

Values of the slope of inflow conduit, α, chosen for this investigation were zero and 2 to 1 with the horizontal. These values are within the usual range of conduit slopes found in the prototype. The effects of the slope of the inflow conduit are shown in Figures 12, 13, 14, and 15. From Figures 13 and 15 it can be seen that when the slope of the inflow conduit was zero, the flow followed the boundaries of the transition, thus showing distinct effects the angle of divergence of the sidewalls (θ) had on the values of the coefficient of energy loss K. From Figures 12 and 14 where the slope of the inflow conduit was 2 to 1 with the horizontal, the coefficient of energy loss K was concentrated about the value of 0.6, thus minimizing the effects of the angle of divergence of the sidewalls. This minimizing is due to the flow being lifted away from the transition boundaries.

It was observed that the distribution of flow was better in the model tests where the slope of the inflow conduit was 2 to 1 with the horizontal. This slope also gave less erosion in the streambed, Figures 9, 10, and 11.

The Effects of Inertia and Gravitational Forces

As stated in the theoretical analysis, the assumption was made that inertia and gravitational forces would be variables influencing K. For simplicity the mean velocity in the conduit was chosen to make the analytical comparison. This mean velocity was used to compute the dimensionless parameter N_F. The range of values of N_F used in this study were 0.35 to 0.55. Figures 12 and 13 show the influence of N_F on the values of K. It is noted that the slope of the K versus N_F plots are relatively flat, thus showing the values of K are constant for the given range of the Froude number.

CONCLUSION AND RECOMMENDATIONS

1. The coefficient of loss, K, obtained from the following relationship

$$K = \frac{\left(H_{e_1} - H_{e_2}\right)}{\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right)} = \frac{\Delta H_e}{\Delta H_v}$$

is influenced by four factors:

(a) K is affected by the angle of divergence of the sidewalls from the center line of the structure, θ. The value of K being optimum with θ equal to 25 degrees.

(b) K is affected by submergence of inflow conduit, d/x. A submergence between zero and d/6 affects the value of K only slightly, whereas a submergence of d/3 increases the value of K by 20 to 30 percent.

(c) K is affected by the slope of the inflow conduit, α. As the value of α increases from horizontal to a 2 to 1 slope the value of K converges on a value of 0.6.

(d) K is affected only slightly by the variation in inertia or gravitational forces as represented by the dimensionless parameter, the Froude number, in the range of field application, Figures 12 and 13, provided the above three parameters remain constant.

2. For design purposes the value of K may be taken from Figures 14 and 15 with an average value of K equal to 0.60. The energy loss equals

$$0.60 \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right)$$

3. The limitations of K are for values of Froude number between 0.35 and 0.55.

4. This type of structure may be used on large inverted siphons where saving of energy head is not a prime factor.

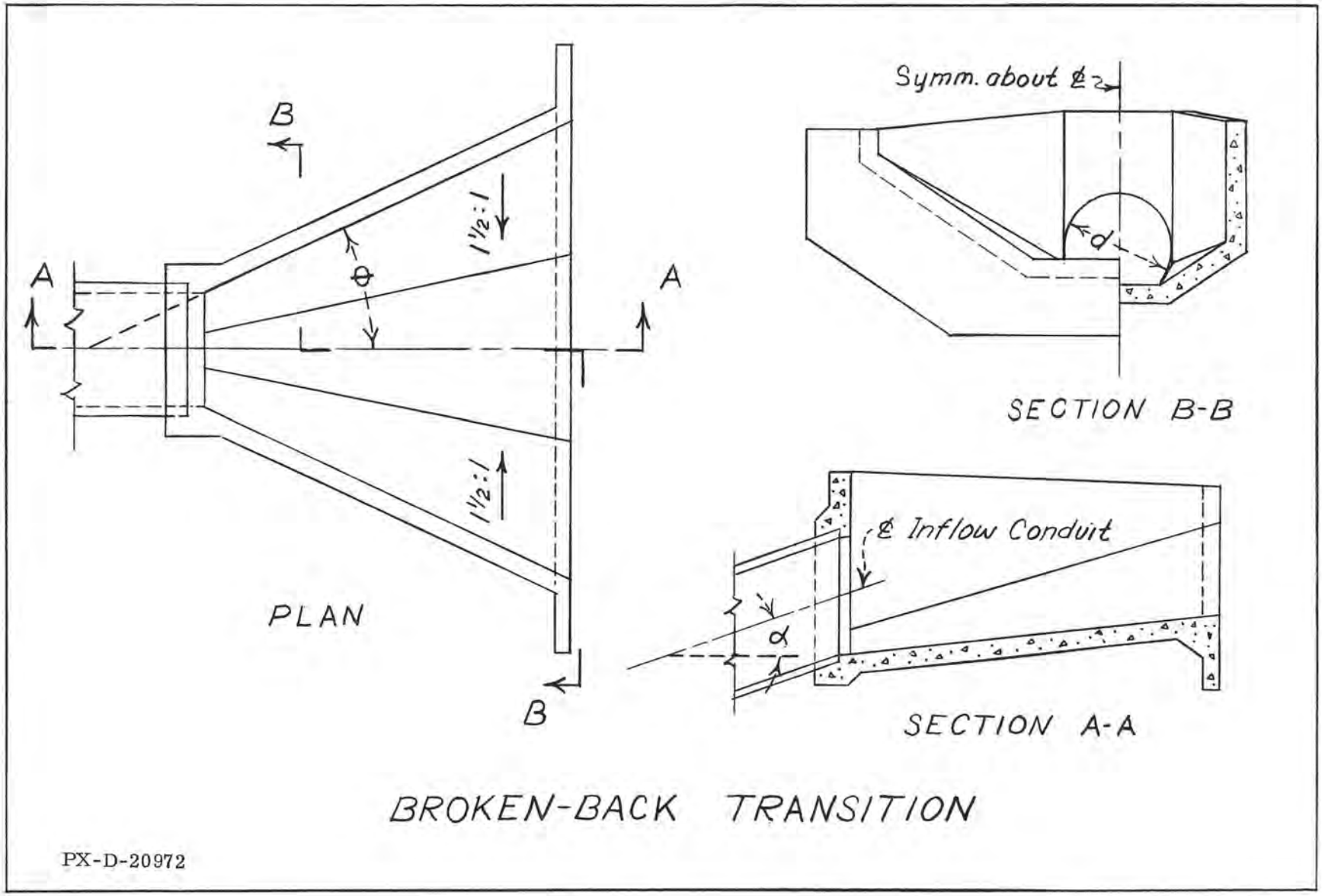
BIBLIOGRAPHY

Hinds, Julian M., "The Hydraulic Design of Flumes and Siphon Transitions," Transactions of the American Society of Civil Engineers, Vol. 92, 1928, pp. 1423-1459. New York: American Society of Civil Engineers, 1928.

Benson, M. H., "Model Tests of Outlet Transitions," Civil Engineering, Vol. 6, January-December 1936, p. 760. New York: American Society of Civil Engineers, 1936.

Kui, H. K. and Albertson, M. L., "Significance and Application of Froude and Reynolds Numbers as Criteria for Similitude." Paper presented at the Hydraulics Conference of the American Society of Civil Engineers, Fort Collins, Colorado, July 1959.

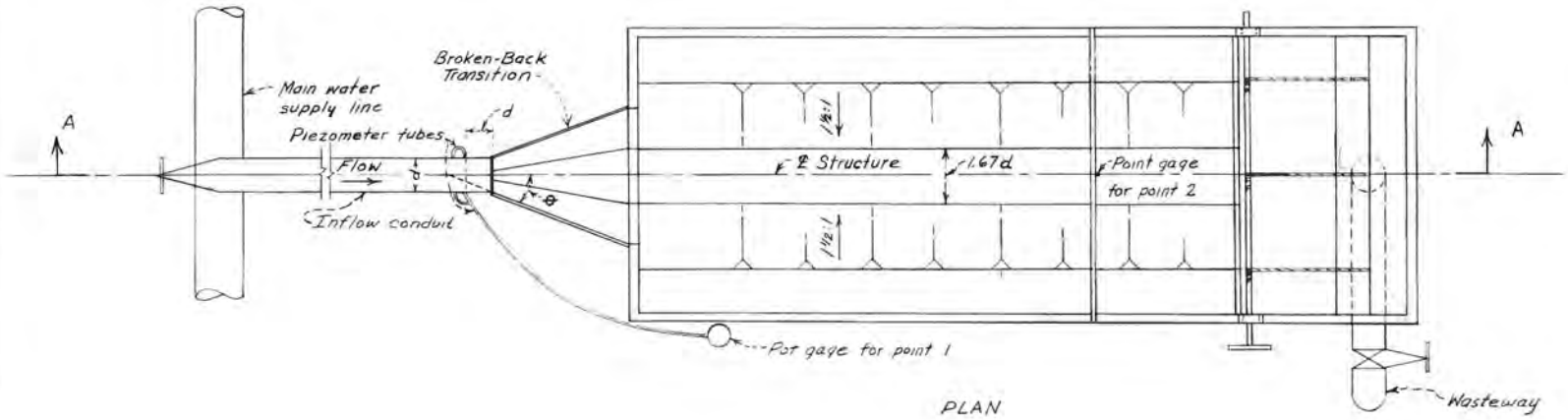
Fornica, G., "Esperienza preliminari sulle perdite de carico nei canali, dovute a cambiamenti di sezione," L'Energia Elettrica (Milano), Vol. 32 (7), July 1955, pp. 554-567. Milano, Italy: Official publication for the documents of the (Associazione Nazionale Imprese Produttrici E Distributrici Di Energia Elettrica) National Association of Industries for the Production and Distribution of Electric Energy and (Della Federazione Nazionale Imprese Elettriche) the National Federation of Electrical Industries.



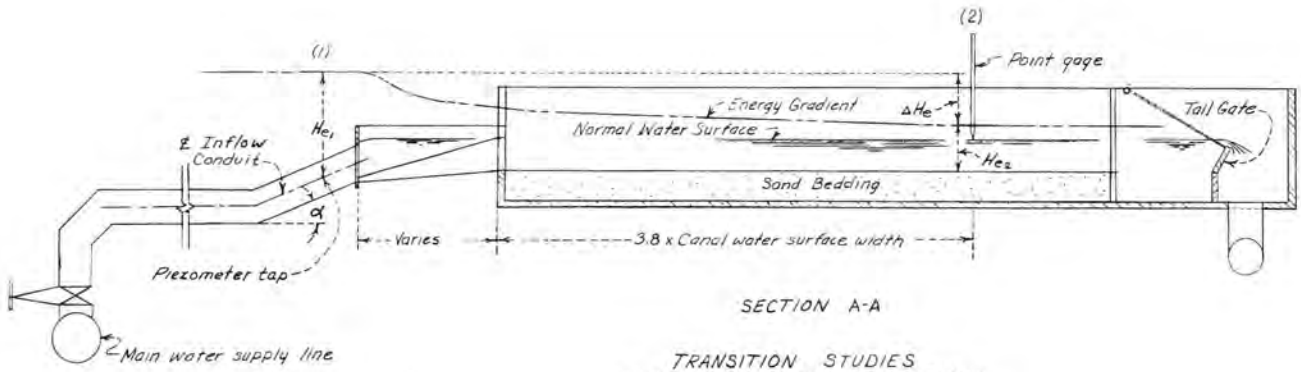
BROKEN-BACK TRANSITION

PX-D-20972

FIGURE 3



PLAN



SECTION A-A

TRANSITION STUDIES
LABORATORY INSTALLATION

PX-D-20973

FIGURE 4

Figure 5. General view of model

Figure 6. Location of piezometer taps in 12-inch inflow conduit

Figure 7. Pot gage installation

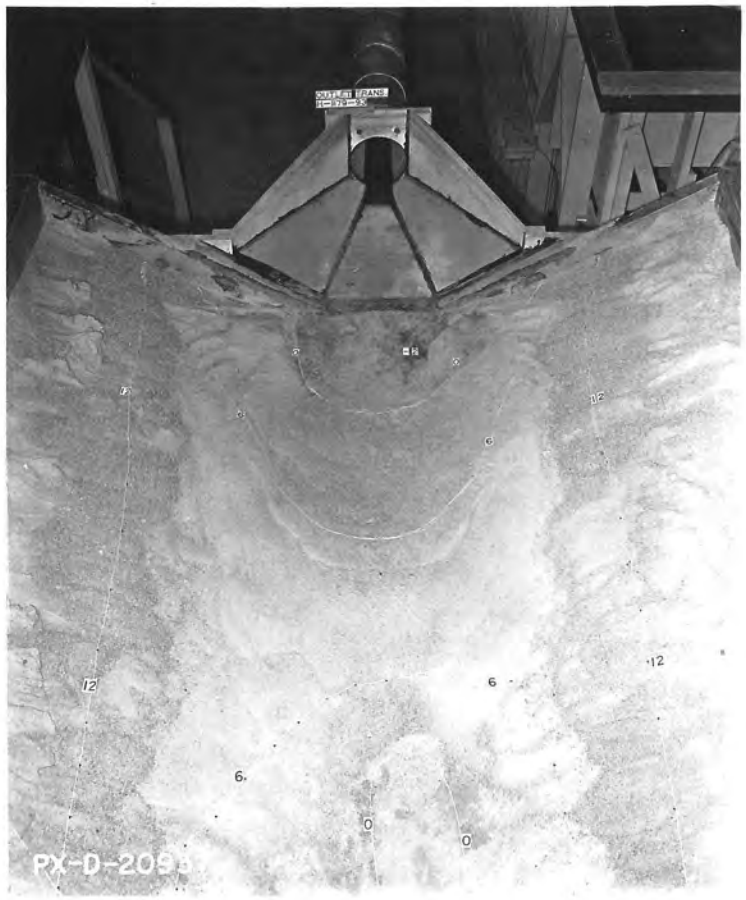
Figure 8. Shape of channel bed before each test

A. Inflow conduit horizontal

B. Inflow conduit sloped 2:1

SCOUR PATTERN OF CHANNEL BED AS A RESULT OF FLOW THROUGH
A BROKEN-BACK TRANSITION WITH θ EQUAL 20 DEGREES

Figure 9



A. Inflow conduit horizontal

B. Inflow conduit sloped 2:1

SCOUR PATTERN OF CHANNEL BED AS A RESULT OF FLOW THROUGH
A BROKEN-BACK TRANSITION WITH θ EQUAL 25 DEGREES

Figure 10

A. Inflow conduit horizontal

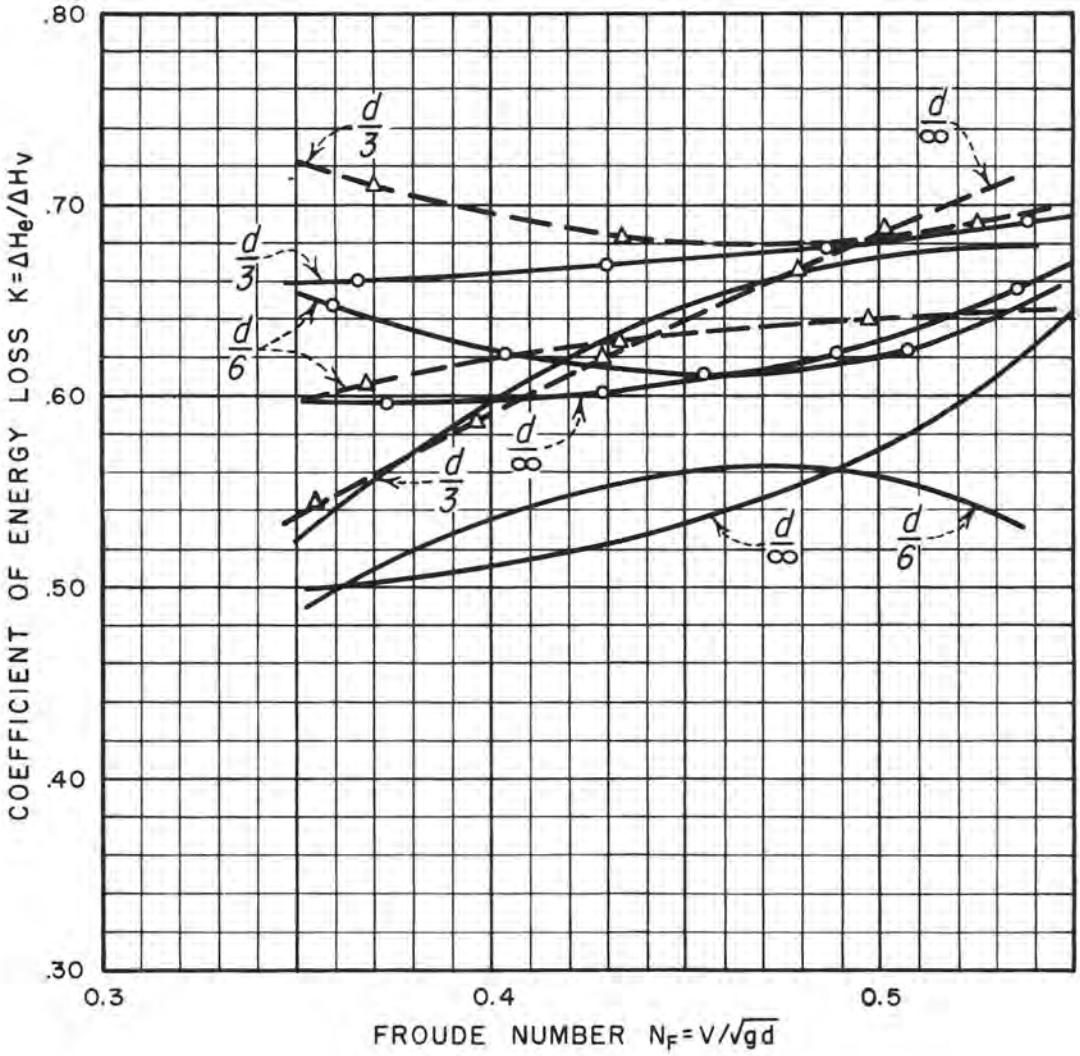
B. Inflow conduit sloped 2:1

SCOUR PATTERN OF CHANNEL BED AS A RESULT OF FLOW THROUGH A BROKEN-BACK TRANSITION WITH θ EQUAL 30 DEGREES

Figure 11

EXPLANATION

$\frac{d}{x}$	Submergence of inflow conduit where d is the diameter of conduit in inches and X is a constant to give desired submergence.
α	Slope of inflow conduit
θ	Angle of sidewall divergence
—o—	For θ equal to 20 degrees
——	For θ equal to 25 degrees
—Δ—	For θ equal to 30 degrees



TRANSITION STUDIES

K VERSUS N_F FOR α EQUAL 2:1 WITH THE HORIZONTAL

PX-D-20974

9/22/61
W.P.S.

Average Effect of Submergence

2:1 sloping pipe

from Figure 12, Sidstis thesis Photo PX-D-20974

20°	$\frac{D}{\infty}$ = 0.598 (K =)	$\frac{D}{6}$ 0.653	$\frac{D}{3}$ = 0.660
	.599	.622	.663
	.606	.611	.671
	.630	.620	.681
	.670	.660	.692
	5 | 3.103	5 | 3.166	5 | 3.367
	.621	.633	.673
25°	$\frac{D}{\infty}$ = 0.500	$\frac{D}{6}$ = 0.486	$\frac{D}{3}$ = 0.524
	.511	.535	.595
	.533	.560	.647
	.571	.558	.672
	.643	.518	.680
	5 | 2.758	5 | 2.657	5 | 3.121
	.552	.531	.624
30°	$\frac{D}{\infty}$ = 0.538	$\frac{D}{6}$ = 0.598	$\frac{D}{3}$ = 0.722
	.590	.619	.696
	.640	.631	.681
	.684	.640	.682
	.725	.644	.701
	5 | 3.177	5 | 3.132	5 | 3.482
	.635	.626	.696

$\frac{D}{\infty} = 0$ $\frac{D}{6} = .1666\ D$ $\frac{D}{3} = .3333\ldots D$

Submergence factor

FIGURE 13

EXPLANATION

$\frac{d}{x}$ Submergence of inflow conduit where d is the diameter of conduit in inches and X is a constant to give desired submergence.

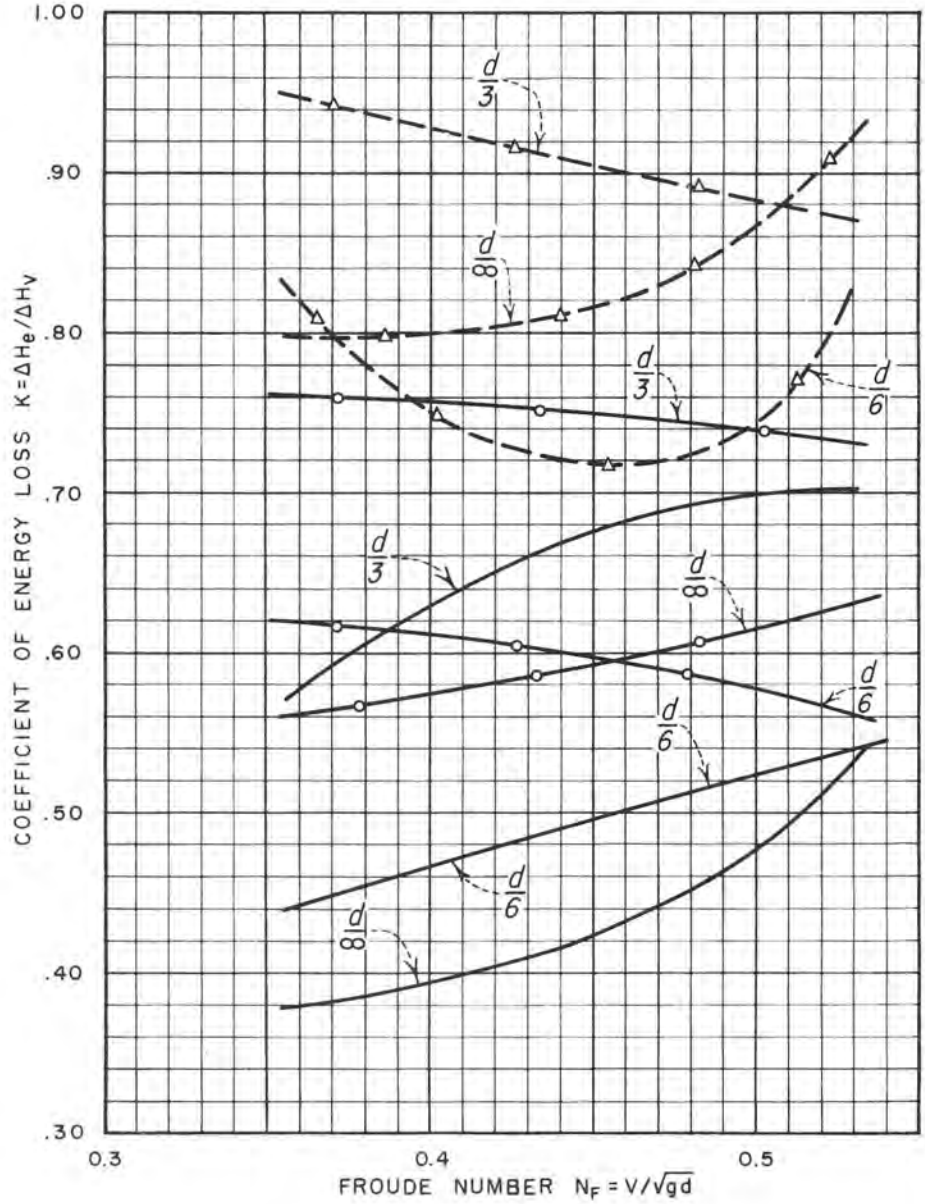
α Slope of inflow conduit

θ Angle of sidewall divergence

For θ equal to 20 degrees

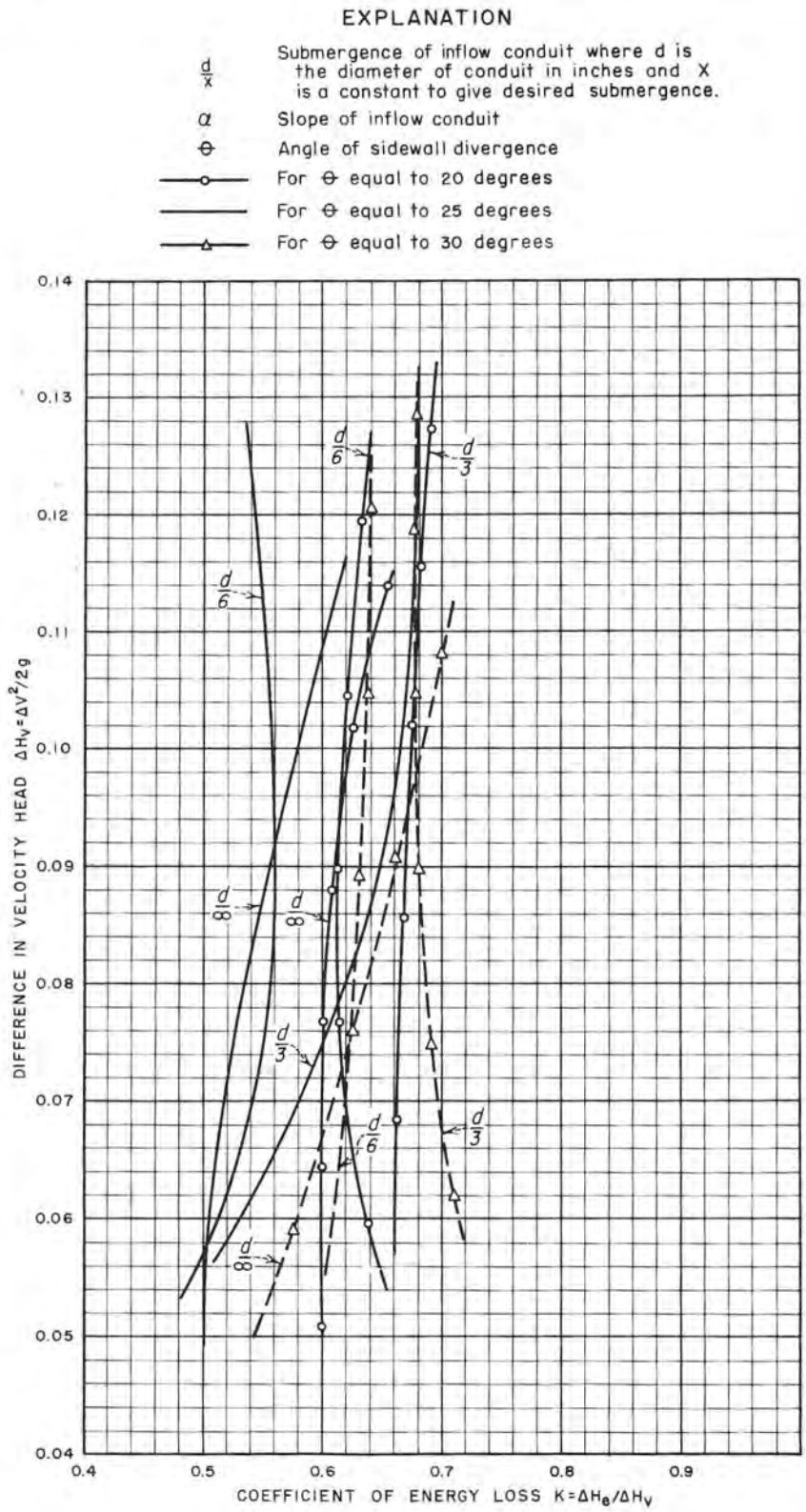
For θ equal to 25 degrees

For θ equal to 30 degrees



TRANSITION STUDIES

PX-D-20975 ***K VERSUS N_F FOR α EQUAL ZERO***

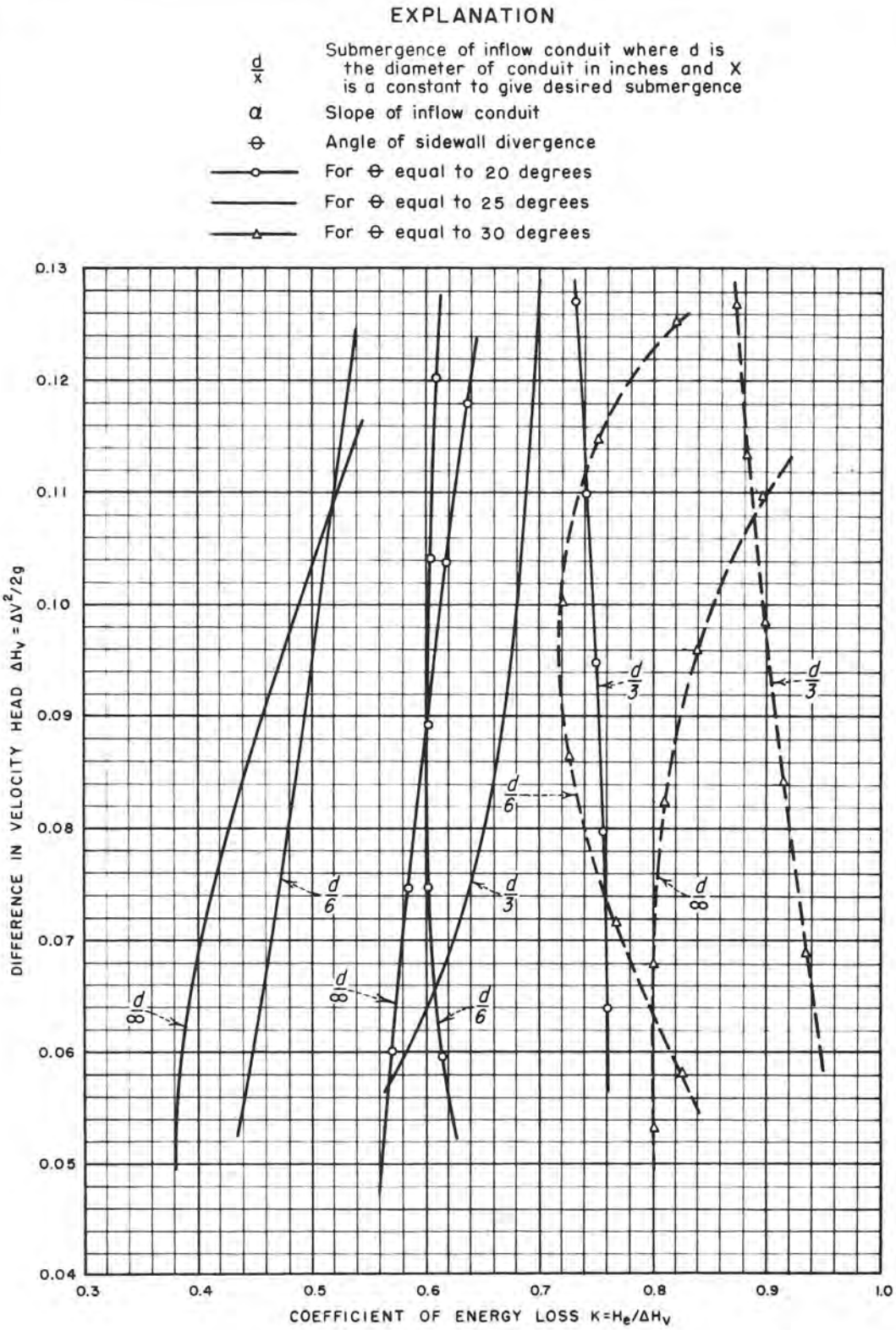


TRANSITION STUDIES

$\Delta V^2/2g$ VERSUS K

FOR α EQUAL 2:1 WITH THE HORIZONTAL

PX-D-20976



TRANSITION STUDIES

$\Delta V^2/2g$ VERSUS K
FOR α EQUAL ZERO

PX-D-20977

Submergence of Inflow Pipe	Depth of Water in Canal	Q	Velocity in Pipe	$V_p^2/2g$	Piezometer Head on Pipe	Relative Pipe Head	Velocity in Canal	$V_c^2/2g$	Canal Head	Total Pipe Head	Total Canal Head	Δ Head	$V_p^2/2g - V_c^2/2g$	$K = \frac{\Delta \text{ Head}}{V_p^2/2g - V_c^2/2g}$
d/3	12"	1.571	2.00	0.062	2.154	0.984	0.496	0.004	1.000	1.046	1.004	0.042	0.058	0.72
d/3	12"	1.964	2.50	0.097	2.141	0.971	0.620	0.006	1.000	1.068	1.006	0.062	0.091	0.68
d/3	12"	2.356	3.00	0.140	2.130	0.960	0.774	0.009	1.000	1.100	1.009	0.091	0.131	0.69
d/6	10"	1.571	2.00	0.062	1.981	0.811	0.647	0.007	.833	.873	.840	0.033	0.055	0.60
d/6	10"	1.964	2.50	0.097	1.971	0.801	0.809	0.010	.833	.898	.843	0.055	0.087	0.63
d/6	10"	2.356	3.00	0.140	1.958	0.788	0.970	0.015	.833	.928	.848	0.080	0.125	0.64
0	8"	1.571	2.00	0.062	1.813	0.644	0.833	0.012	.667	.706	.679	0.027	0.050	0.54
0	8"	1.964	2.50	0.097	1.807	0.638	1.101	0.019	.667	.735	.686	0.049	0.078	0.63
0	8"	2.356	3.00	0.140	1.803	0.634	1.324	0.027	.667	.774	.694	0.080	0.113	0.71

Table 1

Typical Laboratory Computation Sheet (θ = 30 degrees, α = 2:1)

Figure 16

PX-D-20978