UNIVERSITY OF MINNESOTA
ST. ANTHONY FALLS HYDRAULIC LABORATORY
LORENZ G. STRAUB, Director

Technical Paper No. 4, Series B

Hydraulic Tests on Concrete Culvert Pipes

by
LORENZ G. STRAUB
and
HENRY M. MORRIS

July, 1950
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C O N T E N T S

LIST OF ILLUSTRATIONS

HYDRAULIC TESTS ON CONCRETE CULVERT PIPES

I. INTRODUCTION

Included in an experimental program conducted at the St. Anthony Falls Hydraulic Laboratory of the University of Minnesota on full-scale culverts was a series of tests on concrete pipes up to 3 ft in diameter. The primary purpose of these tests was to obtain pipe friction and entrance loss coefficients which would be more accurate and dependable than those currently recommended in culvert design literature. The studies were begun in 1946. This paper is confined to a discussion of the concrete culvert test program and the results of the studies.

The test series included three concrete culvert pipes, 18 inches, 24 inches, and 36 inches in diameter, respectively. Each pipe was 193 ft long and laid on a slope of 0.20 per cent, except that the 24-in. pipe was on a slope of 0.224 per cent. The pipes tested were all manufactured by the cast-and-vibrated process. Details of the pipe sections are shown on page 22.

Friction and entrance loss coefficients were established for the culverts under the usual conditions of field operation. With this objective in view, each pipe was tested for the following conditions:

(a) Full flow with submerged inlet and outlet.

(b) Part-full flow at uniform depth.

The 18-in. and 36-in. diameter pipes were tested for each of the two types of flow with two different entrance conditions; namely, (a) pipe projecting 2 ft into the headwater pool, (b) pipe entrance flush with the headwall. The 24-in. pipe was tested with the projecting entrance only.

II. RESUME OF EXPERIMENTAL PROGRAM

A. Full-Flow Tests

In pipe hydraulic design, the Manning formula is of most frequent use. In this formula,

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2} \tag{1}$$

The Manning roughness coefficient $\underline{n}$* is a measure of the influence of wall roughness in causing head losses. The primary objective of the tests described in this report was to determine the Manning coefficient for typical new concrete culvert pipe. The results of these tests are summarized in Table I.

*All symbols are defined in the Glossary on page 21.

TABLE I

MANNING COEFFICIENTS FOR FULL FLOW

Pipe Diameter	No. of Tests	Range of Values for Coefficient n		
		Maximum	Minimum	Average
18 in.	12	0.0108	0.0091	0.0097
24 in.	9	0.0104	0.0093	0.0100
36 in.	11	0.0108	0.0103	0.0106
	32	0.0108	0.0091	0.0101

A more detailed study of the data reveals a slight, but systematic, decrease in n for increasing discharge and temperature, that is, for increasing Reynolds numbers. It is believed, therefore, that a value as low as 0.0100 can be recommended for n for new concrete culvert pipe of the type tested, although a more accurate design analysis would take into account the small variation of n with water temperature and velocity.

Coefficients of entrance loss were also computed for each run for application in the entrance head loss formula,

$$H_e = K_e \frac{V^2}{2g} \tag{2}$$

These coefficients are summarized in Table II.

TABLE II

ENTRANCE LOSS COEFFICIENTS FOR FULL FLOW

Pipe Diameter	Range in K_e for Projecting Inlet				Range in K_e for Flush Inlet			
	No. of Tests	Maximum	Minimum	Average	No. of Tests	Maximum	Minimum	Average
18 in.	4	0.12	0.09	0.10	7	0.13	0.05	0.08
24 in.	8	0.19	0.07	0.11	-	-	-	-
36 in.	6	0.21	0.12	0.16	5	0.12	0.05	0.10
	18	0.21	0.07	0.12	12	0.13	0.05	0.09

The foregoing entrance loss coefficients apply for a culvert entrance consisting merely of the groove end of a normal length of concrete pipe with tongue-and-groove joints, which is the customary orientation of concrete pipe culverts. The losses are very small, and the range in coefficients obtained is attributable mostly to random experimental variation. The recommended entrance coefficient for pipe projecting into a headwater pool is 0.15, and the recommended entrance coefficient for a flush headwall inlet is 0.10.

B. Part-Full Flow Tests

Roughness and entrance loss coefficients were also obtained for two of the pipes flowing partly full at various uniform depths. It was not possible to do this for the 36-in. pipe because the slope of the pipe was so near the "critical slope" for the pipe that critical flow, with attendant water surface waviness and instability, was always established near the entrance and through most of the barrel, making a uniform depth determination meaningless.

However, the other two pipes afforded adequate data for design for this type of flow. The Manning roughness coefficients obtained are indicated in Table III.

TABLE III

MANNING COEFFICIENTS FOR UNIFORM TRANQUIL FLOW IN PIPE

Pipe Diameter	No. of Tests	Range of Values for Coefficient <u>n</u>		
		Maximum	Minimum	Average
18 in.	10	0.0110	0.0102	0.0107
24 in.	6	0.0108	0.0102	0.0104
	16	0.0110	0.0102	0.0106

The variation in the above data is mostly random experimental variation. The average value of $\underline{n}$ was 0.0106; the maximum value obtained in the tests was 0.0110. The latter may be recommended as a conservative value for $\underline{n}$ for general use with part-full flow in new concrete pipes of the kind tested. It should be noted that these values apply only to uniform flow at tranquil (subcritical) velocities. However, in most cases of culvert design, if supercritical flow exists, the design will be dependent upon inlet geometry rather than barrel friction, so that roughness coefficients for supercritical flow would be unnecessary.

Entrance loss coefficients for the uniform tranquil flow condition are given in Table IV.

TABLE IV

ENTRANCE LOSS COEFFICIENTS FOR UNIFORM TRANQUIL FLOW IN PIPE

Pipe Diameter	Range in K_e for Projecting Inlet				Range in K_e for Flush Inlet			
	No. of Tests	Maximum	Minimum	Average	No. of Tests	Maximum	Minimum	Average
18 in.	8	0.20	0.13	0.16	2	0.15	0.06	0.10
24 in.	6	0.23	0.02	0.08	-	-	-	-
	14	0.23	0.02	0.12	2	0.15	0.06	0.10

The same values of K_e as for full flow, 0.15 and 0.10, are recommended for part-full tranquil flow, for projecting and flush inlets, respectively.

III. EXPERIMENTAL METHODS

All three pipes were tested in the main testing channel of the St. Anthony Falls Hydraulic Laboratory. This channel is about 300 ft long overall, 9 ft wide, and 6 ft deep. At the upstream end of the channel is an electrically operated sluice gate, which controls the amount of water entering the channel. Above the sluice gate is a pressure tunnel leading to the headwater pool on the Mississippi River above St. Anthony Falls. The entrance from the pool to the tunnel is controlled by an electrically operated weir gate.

For large discharges, the combination control afforded by the sluice gate and weir gate made possible the accurate maintenance of constant flows. For small discharges, flows were controlled by a valve in an auxiliary 8-in. pipe leading into the test channel, through which small rates of flow could be supplied without use of the pressure tunnel and sluice gate.

Each pipe was installed in the central region of the test channel, with upstream and downstream bulkheads to form headwater and tailwater pools, respectively. The upstream bulkhead was located approximately 56 ft from the sluice gate, with the pipe projecting back into the headwater pool to form a re-entrant inlet. The pipe sections were always laid with the groove end upstream so that the entrance functioned as a partly rounded inlet.

Figure 1 shows the 18-in. pipe as laid in the channel, the bulkhead being a bolted steel frame supporting aluminum plates and a center plywood panel.

A false bulkhead of wooden construction was fitted over the pipe entrance lip when it was desired to simulate a flush headwall entrance. This false bulkhead is shown in place on the 36-in. pipe in Fig. 2.

The downstream bulkhead was similar to the main upstream bulkhead. It was set approximately 25 ft from the tailgate, leaving about 17 ft of pipe projecting into the tailwater pool. The elevation of the tailwater pool was controlled by the electrically operated tailgate over which the water flowed into a channel leading to the outside volumetric measuring tanks.

A side-wall diversion gate in the tailwater pool could be opened, permitting flow into the Laboratory's inside weighing tanks. When this was done, the tailgate was raised above the tailwater elevation, and the latter was controlled by vertical stop logs in the diversion gate.

Discharge measurements were usually made in the large volumetric tanks. The tests on the part-full flow condition in the 18-in. pipe were made during freezing weather, which was too cold for operation of these outside tanks. Therefore, the discharges, which were small, were determined by means of the weighing tanks. These flows were admitted through the auxiliary inlet pipe in which an elbow meter had been installed and calibrated. Discharge readings from the elbow meter very closely agreed with the values obtained from weighing-tank measurements.

Pressure readings in the pipe were obtained by means of flush piezometer openings located at intervals along the bottom centerline of the pipe. For the 24-in. pipe, these piezometer taps were installed at 3, 33, 63, 93, 123, 153, and 183 ft downstream from the inlet. For the other two pipes, the taps were located at 3, 9, 15, 27, 45, 75, 105, 136, 166, 184, and 190 ft from the inlet.

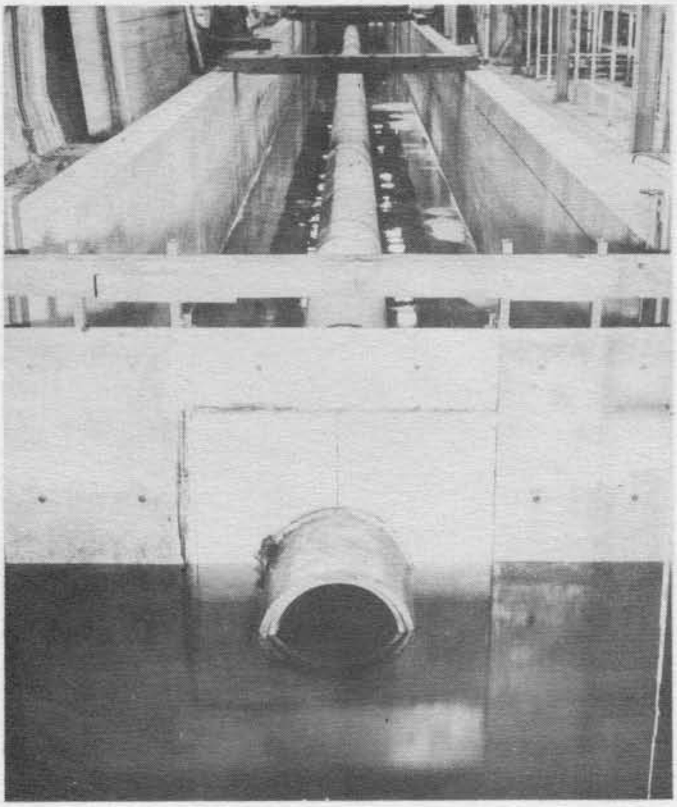


Fig. 1 – 18-in. Concrete Culvert Test Installation

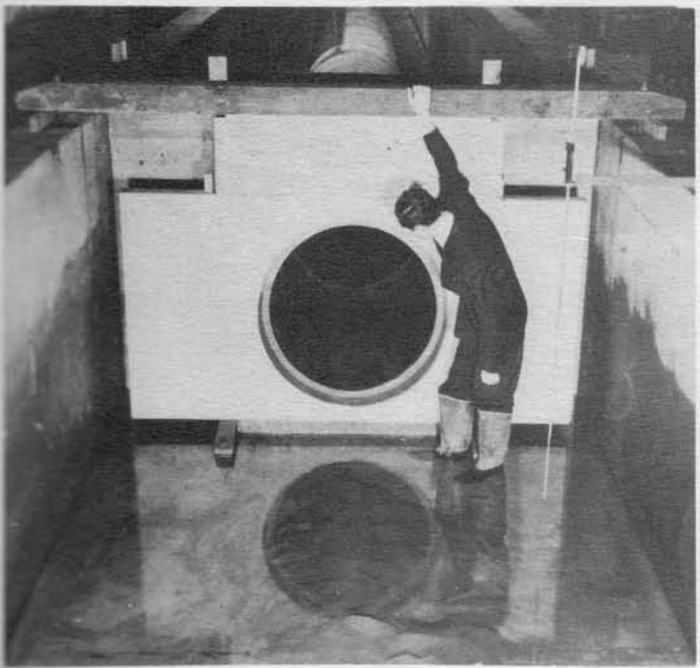


Fig. 2 – Flush Headwall on 36-in. Concrete Culvert

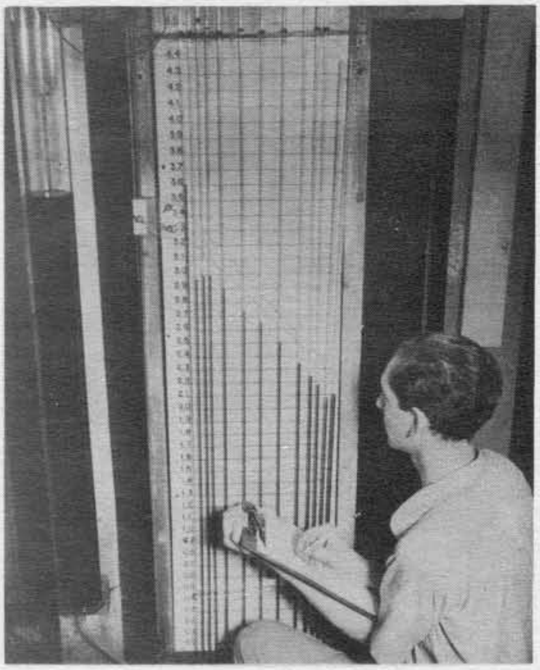


Fig. 3 – Manometry Apparatus

The pressure at each opening was transmitted through a 1/2-in. copper pipe to a glass piezometer tube. All of these piezometer tubes, including one connected to the headwater pool and one to the tailwater pool, were attached to a common manometer board on which a scale of elevations was also placed. The horizontal placement of the tubes on the manometer board was geometrically similar to the positions of their respective pressure taps in the pipe, so that water surface or pressure slopes could be more easily adjusted. All air was expelled from the manometer lines prior to each series of runs. Manometer readings were recorded to hundredths of a foot.

Readings of the headwater pool elevation also could be obtained by means of a point gage in the headwater pool and by an electric point gage in a stilling well mounted beside the manometer board. The manometer board and stilling well were both mounted in the observation pit beside the glass-walled portion of the test channel. The manometry apparatus is shown in Fig. 3.

IV. METHODS OF COMPUTATION

A. General

The experimental data for each run consisted of the measured discharge, the corresponding hydraulic grade line, and the water temperature. The discharges were determined by means of the volumetric tanks, the weighing tanks, or the supply-line elbow meter, depending upon the circumstances. The hydraulic gradients were obtained by simultaneous readings on the piezometers.

The method of reducing and analyzing these basic data was dependent upon whether the condition of flow in the pipe was full or partly full. Consequently, the methods of computation employed will be briefly explained under these two categories.

B. Full Flow

When a culvert is flowing full, the total head producing flow through the culvert is given by

$$H = \text{Headwater Elevation} - \text{Tailwater Elevation} \qquad (3)$$

If the approach velocity head is large, it should be added to the headwater elevation in this equation. Similarly, the tailwater pool velocity head could be added to the tailwater elevation. In the experimental installation, these velocity heads were small and were neglected. By application of the energy equation, the head, H, is equated to the sum of the various energy losses in the culvert as follows:

$$H = K_e \frac{V^2}{2g} + K_f \frac{V^2}{2g} + K_o \frac{V^2}{2g} \tag{4}$$

The three terms on the right represent head losses resulting from the pipe entrance, barrel friction, and pipe outlet, respectively. The three coefficients K_e, K_f, and K_o can be evaluated from the measured hydraulic gradient for a given pipe and discharge.

The Darcy formula for barrel friction loss in a long uniform reach of pipe is applied first:

$$H_f = K_f \frac{V^2}{2g} = f \frac{L}{D} \frac{V^2}{2g} \tag{5}$$

The friction head loss, H_f, divided by the corresponding length of pipe, L, is the slope of the hydraulic gradient. In the central region of the pipe, where this slope was practically constant, its value could readily be determined at least within a range of ± 0.00005. Because of influence from the entrance and outlet conditions and from the changing cross-sectional distribution of velocities through a part of the pipe length, the gradient was linear over only the central region of the pipe. However, the distance in which the gradient was a straight line was always more than 120 ft, so that the friction slope could be determined with good accuracy. Several typical hydraulic gradients are shown on Fig. 4, which illustrate the essential linearity of the hydraulic gradient. Experimental rating curves showing the relation of measured discharges and hydraulic slopes appear in Fig. 5.

The Darcy friction factor f was then computed from a rearrangement of Eq. (5), as follows:

$$f = \frac{DS}{\frac{V^2}{2g}} = \frac{39.6\ D^5\ S}{Q^2} \tag{6}$$

Similarly, the Manning coefficient was computed as follows, replacing R by 1/4D:

$$n = \frac{0.59\ D^{2/3}\ S^{1/2}}{V} = \frac{0.463\ D^{8/3}\ S^{1/2}}{Q} \tag{7}$$

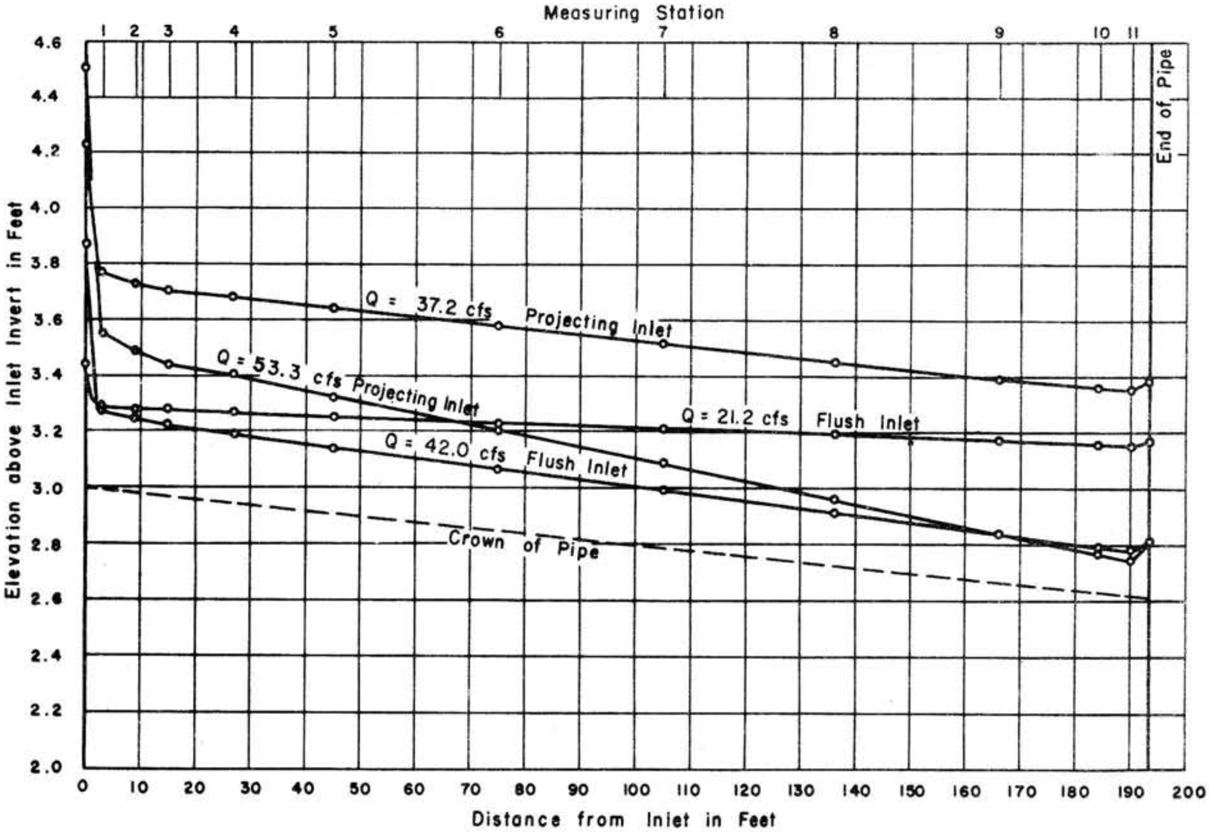


Fig. 4– Typical Hydraulic Grade Lines
(36-in. Concrete Pipe Flowing Full)

The entrance loss was computed by extending the linear portion of the hydraulic gradient to the plane of the entrance, adding $V^2/2g$ to the elevation thus obtained, and subtracting this total from the headwater elevation. This procedure attributes all energy loss in excess of the normal barrel friction loss, in the region near the entrance where the hydraulic grade line is nonlinear, to the effect of the entrance. It also neglects the velocity distribution factor α in the expression for kinetic energy head. The factor α varies with different conditions, but it is always only slightly greater than unity. For the purpose of obtaining practical design data, the method is satisfactory and the results are quite adequate.

The outlet loss was determined in a similar manner by extending the straight-line portion of the hydraulic grade line to the outlet, adding $V^2/2g$ to the resulting elevation, and then deducting the measured tailwater elevation.

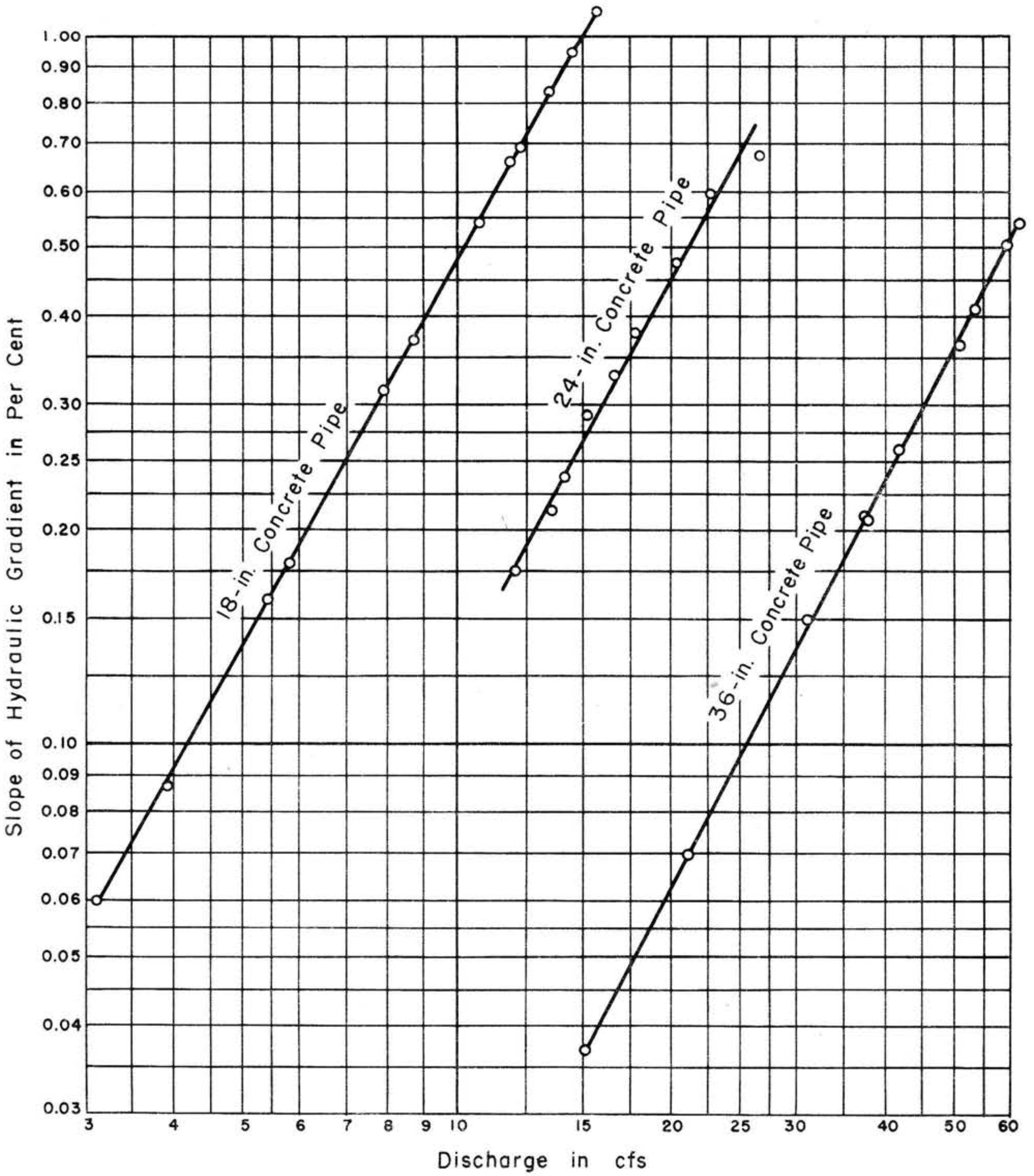


Fig. 5- Experimental Rating Curves
(Concrete Culverts Flowing Full)

Another important quantity that was computed for each run was the Reynolds number,

$$Re = \frac{DV}{\nu} = \frac{1.27Q}{\nu D} \tag{8}$$

The kinematic viscosity of the water, ν, was determined from measured water temperatures and viscosity tables.

C. Part-Full Flow

In the part-full flow tests, a condition of uniform or near-uniform flow was established for each run by adjusting the tailwater to the proper elevation for maintaining flow at approximately uniform depth and velocity through most of the pipe barrel. For some discharges, the flow as established was slightly nonuniform, but the water surface slope was linear over a sufficiently long central reach of the pipe to permit an accurate determination of roughness coefficient. In all cases, the actual value of the hydraulic slope was used in the computations. The average depth of flow in the region of uniform slope was taken as the depth from which to calculate flow area and hydraulic radius.

Rating curves for the part-full flow condition are shown in Fig. 6. The normal discharges shown were computed on the basis of the pipe slope of 0.002, by multiplying the measured discharges by the factor $(0.00200/\text{measured slope})^{1/2}$, since discharge is proportional to the square root of the hydraulic gradient.

In the case of the 18-in. diameter pipe, the effect of nonuniformity of the pipe cross section made the establishment of perfectly uniform flow virtually impossible, particularly at small depths. These effects were further aggravated by the fact that the flow, though subcritical, was not far from the critical flow regime, causing a tendency for the water surface to be wavy and unsteady with very slight changes in total energy.

Consequently, for this pipe, the energy gradient rather than the hydraulic gradient was used to compute the friction slope. The energy gradient was computed with reference to the line of specific energy for the pipe, which was made smooth by trial-and-error adjustment of the pipe invert elevations.

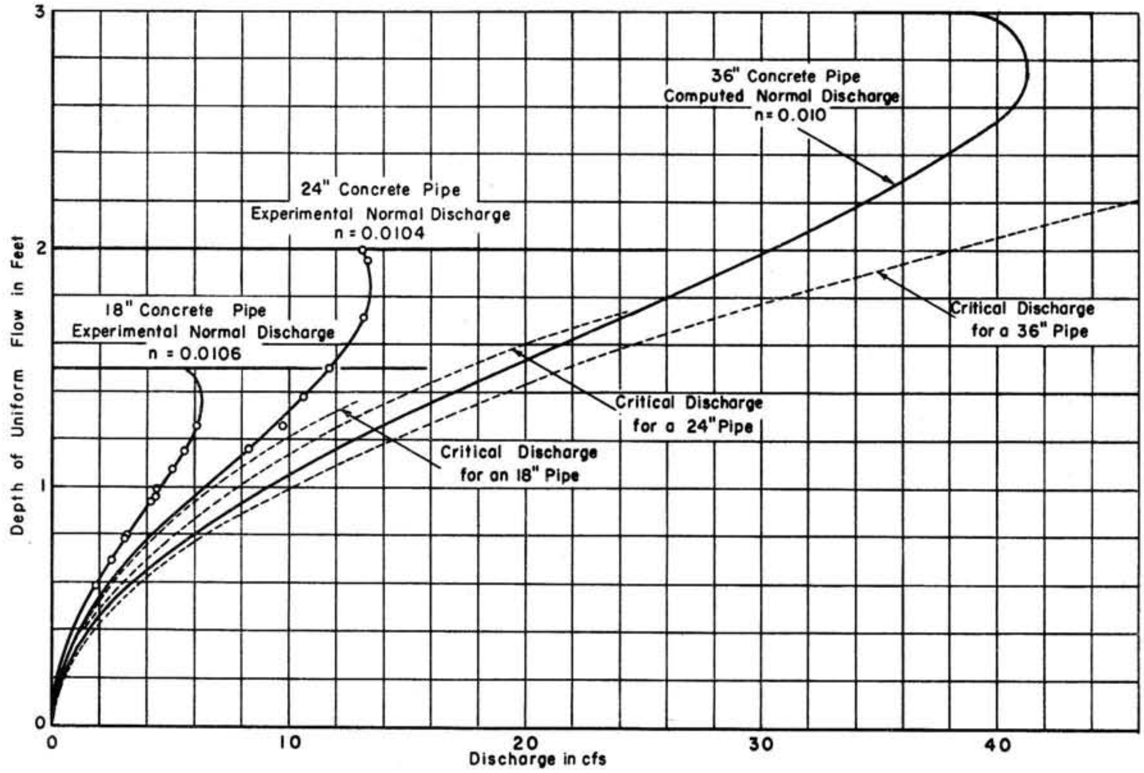


Fig. 6— Experimental Rating Curves
(Concrete Culverts Flowing Partly Full)

For the larger diameters, the effect of nonlinearity of pipe slope was not so pronounced. However, the influence of the near-critical condition of the flow was more marked. The effect of entrance contraction actually seemed to make the flow pass through critical depth near the entrance, with resulting waviness and instability of surface for a considerable portion of the barrel length, in many cases the entire length.

In the 24-in. pipe, this phenomenon was significant only at small depths. At larger depths, it was possible to establish stable, uniform flow conditions.

However, it proved impossible to obtain stable uniform flow at any stage in the 36-in. pipe. The pipe slope of 0.002 was very near the critical slope for most possible stages in the 36-in. pipe. A computed rating curve for the 36-in. pipe has been shown on Fig. 6, based on an assumed value of 0.010 for Manning's coefficient, together with the critical flow curve for a

36-in. diameter section. It is apparent that the normal and critical flow curves are so near each other over most of the range of flow stages that the actual flow in the pipe is practically critical at all except the highest stages, a situation which is aggravated by the critical flow phenomena induced near the entrance by jet contraction.

Similar critical flow curves have been drawn for the 18-in. and 24-in. pipes and are also shown on Fig. 6. Percentagewise, it is evident that these curves are farther separated from their corresponding normal flow curves than is the case with the curves for the 36-in. pipe.

Entrance loss coefficients were also computed for the part-full subcritical flows in the same manner outlined for full flows, by extending the computed energy gradient to the plane of the inlet and deducting the resulting elevation from the measured headwater elevation. The entrance loss thus obtained was then expressed as a coefficient times the pipe velocity head, the latter being based on the mean depth of flow in the reach of uniform slope.

Outlet loss coefficients were not computed for the part-full flow tests because of the variety of tailwater positions that were necessary to establish uniform flow in the pipe. The outlet velocity head bore an irregular relation to the uniform flow velocity head on which the other computations were based.

V. ANALYSIS AND DISCUSSION OF RESULTS

A. General

It will be noted that the values of the Manning coefficient for concrete pipe average about 0.010 in these tests, which is considerably less than the values 0.013 and 0.015 previously recommended.

Several factors may have contributed to the unusually small roughnesses indicated. In the Laboratory, the pipes were laid as straight as possible with a minimum of flow disturbance due to protrusions at the joints and other causes. Various modern methods of pouring and finishing concrete pipe, including the vibration process by which the test pipes were made, result in an exceptionally smooth surface. It is known that open channels lined with smooth cement also have a Manning coefficient as low as 0.010.

Experimental control and accuracy were of as high or higher degree of precision than other previous friction tests on concrete pipe. It is of interest to note that tests on several corrugated pipes have been conducted by the same personnel, using the same methods, installation, and instrumentation as for the concrete pipes. These tests yielded considerably higher values for Manning's n for corrugated metal pipe than have heretofore been recommended. Thus, it is believed that the low values obtained for concrete pipe cannot be attributed to experimental inaccuracies.

The roughness values obtained in the tests, of course, represent rather the idealized conditions which ordinarily might not exist in the field. On the other hand, the tongue-and-groove type of pipe, with reasonably careful installation procedure, could give equivalent results in the field. In choosing the n-value, however, one must recognize that the alignment might not be as good as laboratory conditions; there might be openings in the field for inlets or branch pipes (especially in sewers), debris of various kinds might accumulate in the pipe, and the walls themselves could be expected to undergo some deterioration. Also, some processes of manufacture produce rougher surfaces than the ones tested. The latter conditions, of course, are not within the scope of the tests nor do the tests offer a basis for increasing roughness with age or under various field conditions.

B. Friction Losses for Full Flow

The flow of water in commercial pipe is usually assumed to be fully turbulent, an assumption which is implicit in the use of pipe-flow formulas such as those of Scobey, Hazen-Williams, Manning, and others whose particular roughness terms are taken to be independent of viscous shear and to depend on wall roughness only. Actually, however, the flow will often be in the transitional range from partly turbulent ("smooth pipe") flow to fully turbulent flow, and thus will depend on viscous action as well. The parameter usually employed as a measure of the relative importance of viscosity in the flow pattern is the Reynolds number, DV/ν.

The Darcy formula is commonly used as a general pipe flow formula, since its friction factor, unlike the roughness terms of other formulas, is dimensionless and can conveniently be defined to cover all types of flow. The Darcy friction factor is a function of only the Reynolds number and the relative roughness of the pipe wall with respect to its diameter. Experimental curves showing the variation of friction factor with Reynolds number for each pipe are shown on Fig. 7. Similar curves showing the variation of the Manning coefficient with Reynolds number are shown on Fig. 8.

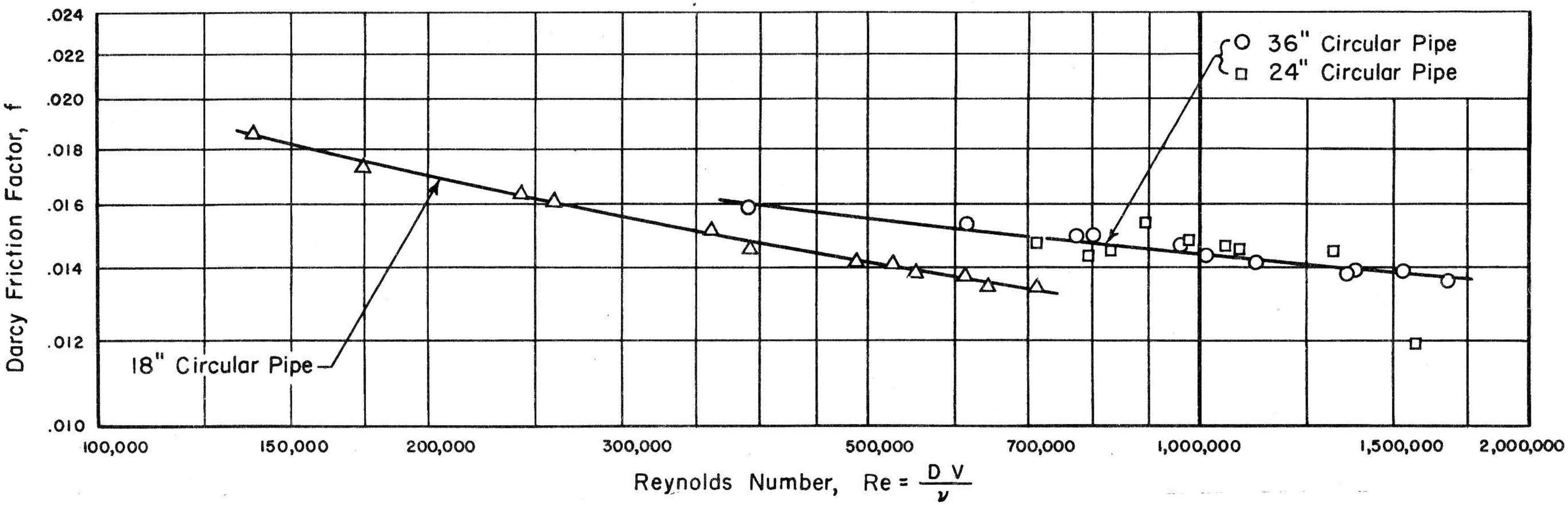


Fig. 7– Variation of Friction Factor with Reynolds Number (Concrete Pipes Flowing Full)

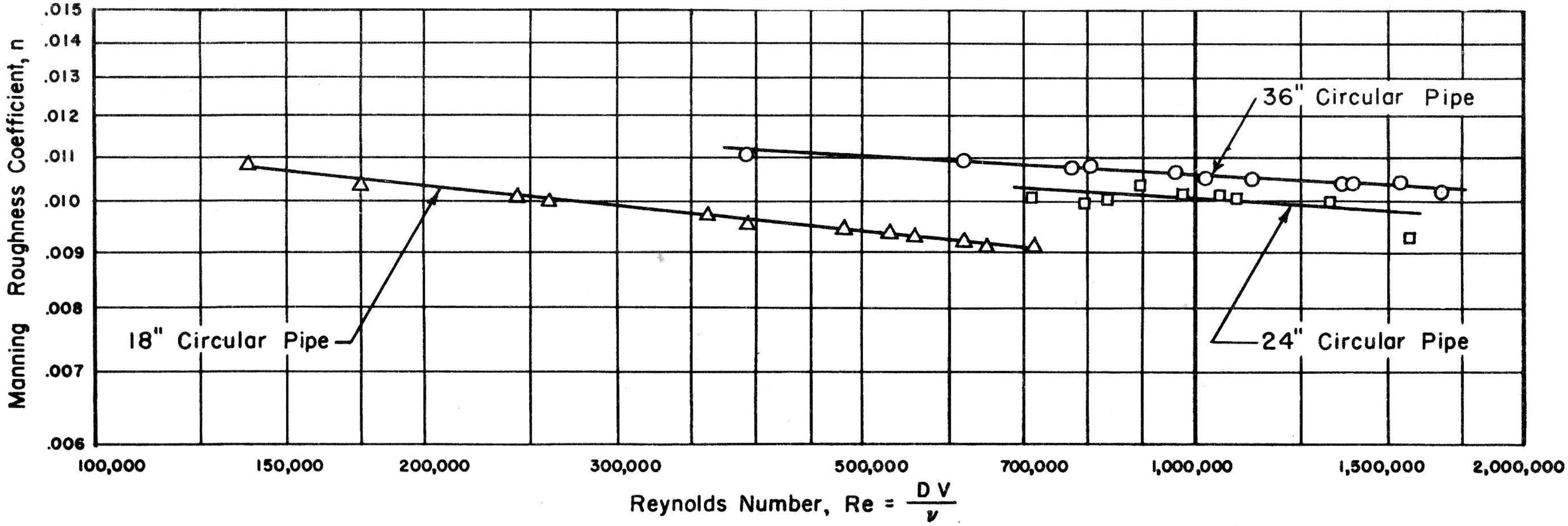


Fig. 8– Variation of Manning Coefficient with Reynolds Number (Concrete Pipes Flowing Full)

C. Friction Losses for Part-Full Flow

It has been noted already that the Manning coefficients obtained for part-full uniform flow were in close agreement for the two pipe sizes tested. Furthermore, there was little individual variation from the mean of 0.0106. The range from the mean was only ± 0.004 or ± 3.8 per cent. This variation exhibited no systematic pattern. It apparently was essentially random and is attributable mostly to experimental variation. It appears reasonable to recommend 0.0110 for n for uniform, subcritical flow in new concrete pipes of the type tested.

D. Entrance Losses

The most important factor influencing entrance loss is the geometry of the inlet itself. When the jet of entering water contracts and then re-expands, much of the high kinetic energy of the contraction is lost through intense turbulence generated in the re-expansion.

Thus the degree of jet contraction is directly related to the magnitude of entrance loss, and the inlet geometry (particularly the relative sharpness of the entrance lip) determines the amount of jet contraction. Theoretical re-expansion losses for pipes flowing full from a relatively quiescent headwater pool are as follows:

(1)	Sharp-edged, re-entrant inlet	$1.00\ V^2/2g$
(2)	Sharp-edged, flush headwall inlet	$0.41\ V^2/2g$
(3)	Rounded inlet (radius of rounding $>1/7\,D$)	$0.00\ V^2/2g$

It is known that for re-entrant pipes with finite wall thickness, the theoretical coefficient of loss rapidly reduces from 1.00 to 0.41 as the ratio of wall thickness to diameter increases. When this ratio becomes greater than about 1/20, the inlet approaches the condition of a flush headwall inlet with a sharp-edged entrance.

Since all commercial concrete pipes have wall thicknesses in excess of 1/20 of their diameters, the theoretical re-expansion loss could never exceed $0.41\ V^2/2g$. This would presumably be the loss if the pipe were laid with the spigot end upstream, in the case of pipes with bell-and-spigot joints, or with the tongue upstream in the case of pipes with tongue-and-groove joints.

However, it is universal practice to lay these pipes with the bell end (or groove end) upstream. The contraction, therefore, is from an initial diameter equal to the pipe diameter plus twice the thickness of the tongue or spigot.

All of the pipes tested were of the tongue-and-groove type and gave average entrance loss coefficients of 0.12 for the projecting inlet and 0.09 for the flush inlet. These values indicate the desirable entrance conditions obtained with concrete pipes. A well-rounded entrance with a radius of rounding greater than one-seventh the pipe diameter, would practically eliminate all entrance loss. This condition is approached by the standard concrete pipe entrances, so that entrance losses for the latter are not much greater than for the ideal inlet.

The recommended values, believed to be conservative, are 0.15 and 0.10 for re-entrant and flush inlets, respectively. It may be noted that the end face of the groove is usually less than 1/20 D in thickness. This means that for a re-entrant groove opening the jet is controlled by the back flow along the outer surface of the projecting pipe and springs clear of the inner faces of the groove and wall.

There are other factors that may have some effect on the entrance loss coefficient, but their effects are so small as to be masked by experimental variations. For most design purposes, the average recommended values of 0.15 and 0.10 for K_e will be found quite adequate for re-entrant and flush inlets.

These coefficients are also recommended for part-full, subcritical flow on the basis of the experimental results. Since part of the contraction is eliminated when the headwater surface drops below the inlet crown, it is obvious that the coefficient should be somewhat reduced for the part-full condition. However, since the coefficient is quite small for full flow, it is possible that this reduction is of the same order of magnitude as the experimental variations and, therefore, does not show a significant effect on the data.

E. Outlet Losses

When a pipe discharges into a quiescent tailwater pool, the kinetic energy of the pipe flow is dissipated in the pool. This is the limiting case of loss due to a sudden expansion, and the head loss is theoretically equal to the kinetic head of flow in the pipe at exit.

When the tailwater pool is not quiescent and particularly if it is confined within a relatively narrow channel, some of this kinetic energy may be converted to useful head rather than being entirely dissipated.

The outlet loss was determined as explained previously for the full-flow condition in the 18-in. and 36-in. pipes. The outlet loss coefficient, K_o, in the equation

$$H_o = K_o \frac{V^2}{2g} \tag{9}$$

was found to average 1.00 for flow in the 18-in. pipe and 0.90 for the 36-in. pipe (Table V).

Since the exact value of K_o would depend upon the geometry of the tailwater channel, it is conservative practice in design to use $K_o = 1$ for all pipe diameters, assuming a submerged outlet. No determinations of K_o were made for part-full flow, but it is obvious that the outlet loss for this condition would be very closely equal to the actual velocity head of flow at the pipe exit. This would not usually be the same as the head of uniform velocity in the central region of the pipe barrel and would have to be determined from knowledge of the tailwater elevation at the particular discharge.

The above discussion applies to a straight pipe, without flaring of the outlet, or any special transition to channel dimensions. All the tests were made for this condition. However, the advantages of a properly designed, prefabricated, flared outlet should not be overlooked. For the concrete pipes, the friction and entrance losses were relatively small, especially for the large pipe, as compared to the outlet loss. If the latter could be materially reduced, a substantial saving in pipe size might often be affected for a given headwater position, or else a substantial lowering of headwater for a given pipe size.

Acknowledgment

The experimental program described in this report was sponsored by the American Concrete Pipe Association and the Portland Cement Association. All experiments were conducted at the St. Anthony Falls Hydraulic Laboratory of the University of Minnesota, under the supervision of Dr. Lorenz G. Straub, Director. The experimental observations and the calculations on the 18-in. and 36-in. pipes were made by Thomas Timar. The 24-in. pipe had been tested previously by Owen P. Lamb and William Dingman. Leona Schultz and Lois Fosburgh edited and prepared the manuscript; illustrative material was arranged by Loyal A. Johnson.

TABLE V

SUMMARY OF TEST RESULTS

FULL FLOW

D (in.)	Inlet	Q (cfs)	H (ft)	S (%)	Re (DV/ν)	f	n	K_e	K_o
18	Flush	3.91	0.25	0.087	175,000	0.0173	0.0103	0.09	0.92
		5.42	0.48	0.160	243,000	0.0164	0.0101	0.13	1.03
		7.91	0.94	0.315	361,000	0.0152	0.0097	0.07	1.00
		10.72	1.65	0.540	490,000	0.0142	0.0094	0.05	1.00
		12.23	2.13	0.690	558,000	0.0139	0.0093	0.07	1.00
		13.49	2.60	0.830	616,000	0.0137	0.0092	0.07	1.02
		15.59	4.37	1.080	712,000	0.0134	0.0091	0.07	-
					Average	0.0149	0.0096	0.08	0.99
18	Proj.	3.11	0.17	0.060	139,000	0.0187	0.0108	-	1.04
		5.80	0.53	0.180	260,000	0.0161	0.0100	0.12	0.96
		8.75	1.13	0.370	392,000	0.0146	0.0095	0.09	1.00
		11.82	2.05	0.660	530,000	0.0142	0.0094	0.10	1.01
		14.48	3.81	0.940	649,000	0.0135	0.0091	0.09	-
					Average	0.0154	0.0098	0.10	1.00
24	Proj.	12.11	-	0.175	711,000	0.0148	0.0101	0.07	-
		13.55	-	0.213	796,000	0.0144	0.0100	0.19	-
		14.21	-	0.237	835,000	0.0145	0.0101	0.12	-
		15.20	-	0.290	894,000	0.0155	0.0104	0.09	-
		16.61	-	0.330	976,000	0.0148	0.0102	0.07	-
		17.91	-	0.380	1,052,000	0.0147	0.0102	0.11	-
		20.04	-	0.475	1,080,000	0.0146	0.0101	0.10	-
		22.56	-	0.595	1,326,000	0.0145	0.0100	0.14	-
		26.61	-	0.675	1,565,000	0.0118	0.0093	-	-
					Average	0.0144	0.0100	0.11	-
36	Flush	21.18	0.27	0.070	778,000	0.0150	0.0108	0.11	0.86
		37.96	0.87	0.215	1,018,000	0.0143	0.0106	0.11	0.90
		41.95	1.06	0.260	1,125,000	0.0142	0.0105	0.11	0.91
		50.71	1.52	0.365	1,359,000	0.0138	0.0103	0.12	0.90
		61.93	2.51	0.540	1,690,000	0.0136	0.0103	0.05	-
					Average	0.0142	0.0105	0.10	0.89
36	Proj.	15.10	0.15	0.037	391,000	0.0158	0.0111	0.20	1.00
		23.85	0.37	0.090	618,500	0.0153	0.0109	0.21	0.88
		31.00	0.58	0.150	804,000	0.0150	0.0108	0.12	0.85
		37.15	0.85	0.210	963,000	0.0147	0.0107	0.14	0.90
		53.33	1.70	0.410	1,383,000	0.0139	0.0104	0.12	0.90
		59.35	2.40	0.505	1,539,000	0.0138	0.0104	0.15	-
					Average	0.0147	0.0107	0.16	0.91

TABLE VI

SUMMARY OF TEST RESULTS

PART-FULL FLOW

D (in.)	Inlet	Q (cfs)	y (ft)	R (ft)	A (ft^2)	S (%)	Re DV/ν $33^{\circ}F$	f	n	K_e
18	Flush	3.06	0.79	0.39	0.945	0.210	271,000	0.0193	0.0109	0.06
		4.31	0.99	0.43	1.24	0.205	318,000	0.0183	0.0109	0.15
							Average	0.0188	0.0109	0.105
18	Proj.	1.81	0.59	0.315	0.65	0.200	184,000	0.0210	0.0110	0.17
		2.41	0.695	0.355	0.80	0.205	228,000	0.0202	0.0110	0.10
		3.07	0.795	0.39	0.95	0.210	270,000	0.0193	0.0109	0.20
		4.13	0.94	0.425	1.16	0.200	318,000	0.0173	0.0106	0.15
		4.34	0.96	0.43	1.19	0.205	333,000	0.0167	0.0104	0.17
		5.05	1.075	0.445	1.36	0.210	356,000	0.0165	0.0104	0.13
		5.54	1.145	0.45	1.45	0.195	357,000	0.0159	0.0102	0.16
		5.03	1.26	0.455	1.59	0.195	358,000	0.0163	0.0104	0.20
							Average	0.0179	0.0106	0.16
24	Proj.	8.27	1.16	0.54	1.90	0.224	919,000	0.0147	0.0102	0.02
		9.68	1.26	0.57	2.28	0.204	901,000	0.0164	0.0108	0.23
		10.6	1.38	0.59	2.31	0.213	1,032,000	0.0144	0.0102	0.02
		11.65	1.50	0.61	2.52	0.217	1,078,000	0.0150	0.0103	0.02
		13.3	1.95	0.55	3.12	0.198	862,000	0.0157	0.0105	0.12
		13.08	1.71	0.61	2.86	0.225	1,090,000	0.0151	0.0104	0.05
							Average	0.0152	0.0104	0.08

GLOSSARY

A = Cross-sectional area of flow, sq ft

D = Pipe diameter, ft

f = Darcy friction factor

g = Acceleration of gravity = 32.16 ft/sec/sec

H = Total head on culvert, ft

H_e = Entrance loss, ft

H_f = Head loss due to pipe friction, ft

K_e = Entrance loss coefficient

K_f = Barrel friction loss coefficient

K_o = Outlet loss coefficient

L = Length of culvert, ft

n = Manning roughness coefficient

Q = Rate of flow, cfs

R = Hydraulic radius, ft

Re = Reynolds number = DV/ν

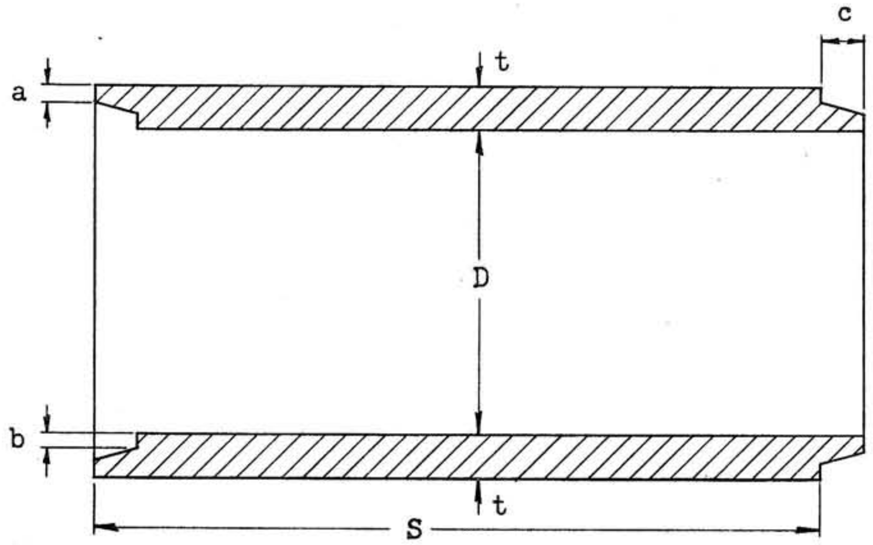
S = Slope of hydraulic gradient

V = Average velocity of flow, fps

α = Velocity distribution factor

ν = Kinematic viscosity, sq ft/sec

DETAILS OF EXPERIMENTAL PIPE SECTIONS



D	t	a	b	c	S
18	2 1/2	1 1/8	1	2	72
24	3	1 3/8	1 1/8	2 7/8	72
36	4	1 7/8	1 1/2	3 3/8	72

All dimensions in inches

All pipes were manufactured by the cast-and-vibrated process, with non-pressure rubber ring joints. The joints of the 24-in. and 36-in. pipe were also filled with cement mortar, applied internally; this was not done on the 18-in. pipe because of its small size, but very good joints were obtained by careful alignment and assembly of sections.