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Hydraulics of Closed Conduit Spillways

Part VIII. Miscellaneous Laboratory Tests
Part IX. Field Tests

by

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ABSTRACT

The theory of the hydraulics of closed conduit spillways has been given in Part I of this report series. Parts II to VII, giving results of tests on several forms of the closed conduit spillway and a discussion of vortices, have also been published. Parts VIII and IX, presented in this paper, report the results of tests on a number of additional forms of the closed conduit spillway. In contrast to the general tests reported in prior Parts, the tests reported here are model tests of specific field structures and actual field tests of full size structures. The results have been presented in such a way that they have general application to the design of the type of structure they represent.

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HYDRAULICS OF CLOSED CONDUIT SPILLWAYS

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FORWARD

The theory of the hydraulics of closed conduit spillways has been given in Part I** of this report series. Parts II to VII,*** giving results of tests on several forms of the closed conduit spillway and a discussion of vortices, have also been published. Parts VIII and IX, presented in this paper, report the results of tests on a number of additional forms of the closed conduit spillway. Discharge coefficients and design recommendations are recorded.

The results described here are a product of an investigation conducted by the staff of the Agricultural Research Service, U. S. Department of Agriculture, located at the St. Anthony Falls Hydraulic Laboratory, University of Minnesota, Minneapolis. There the Agricultural Research Service, the Minnesota Agricultural Experiment Station, and the St. Anthony Falls Hydraulic Laboratory cooperate in the solution of problems concerning conservation hydraulics. Some of the experiments were made at the Laboratory, but many were made by others. The source of all information is acknowledged in the paper. All data were reanalyzed by the writer. The analytical methods and the computations have been checked by Robert V. Keppel.

PART VIII

Miscellaneous Laboratory Tests

INTRODUCTION

A number of laboratory tests that have been made on closed conduit spillways do not fall into any of the categories of Parts II through VI, yet they are of some interest. Therefore, discussed in this Part are the results of tests on an 18-in. steel pipe with vertical and horizontal legs and two field structures that were modeled and tested in the laboratory.

18-INCH STEEL PIPE

DESCRIPTION OF SPILLWAY

The spillway consisted of a flared entrance, a drop inlet pipe 18 ft long, a standard 4-piece long radius elbow made up in 30-degree segments, and a horizontal barrel 30 ft long. The total fall through the spillway was 21.93 ft.

The entrance consisted of a flare 11.04 in. (0.626D) high having a top diameter of 21.63 in. (1.23D). This flare was proportioned so that the contraction of the stream leaving the crest would fill the drop inlet.

The crest was about one foot above the approach channel floor. A false floor level with the crest was installed for the full width of the 8-ft wide approach and for a distance of 6 ft upstream. Originally the false floor consisted of 3 ft by 8 ft galvanized iron sheets. Part of the floor was replaced with concrete after one of the sheets was lifted out by the water, crumpled into a ball and carried through the pipe.

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*Agricultural Research Service Report No. 41-505-76.

**Fred W. Blaisdell, Hydraulics of Closed Conduit Spillways--Part I. Theory and Its Application, St. Anthony Falls Hydraulic Laboratory Technical Paper No. 12-B, revised February 1958.

***Fred W. Blaisdell, Hydraulics of Closed Conduit Spillways--Parts II through VII. Results of Tests on Several Forms of the Spillway, St. Anthony Falls Hydraulic Laboratory Technical Paper No. 18-B, March 1958.

An anti-vortex wall 4 ft high by 6 ft long (2.72D by 4.08D) was located tangent to the downstream side of the drop inlet. This wall was spiked to the false floor supports with 16d nails and was wedged against the tunnel roof. Nevertheless, the pressure behind the wall bent the spikes and forced the wall upstream over the inlet. The anti-vortex wall was then bolted to the concrete approach floor.

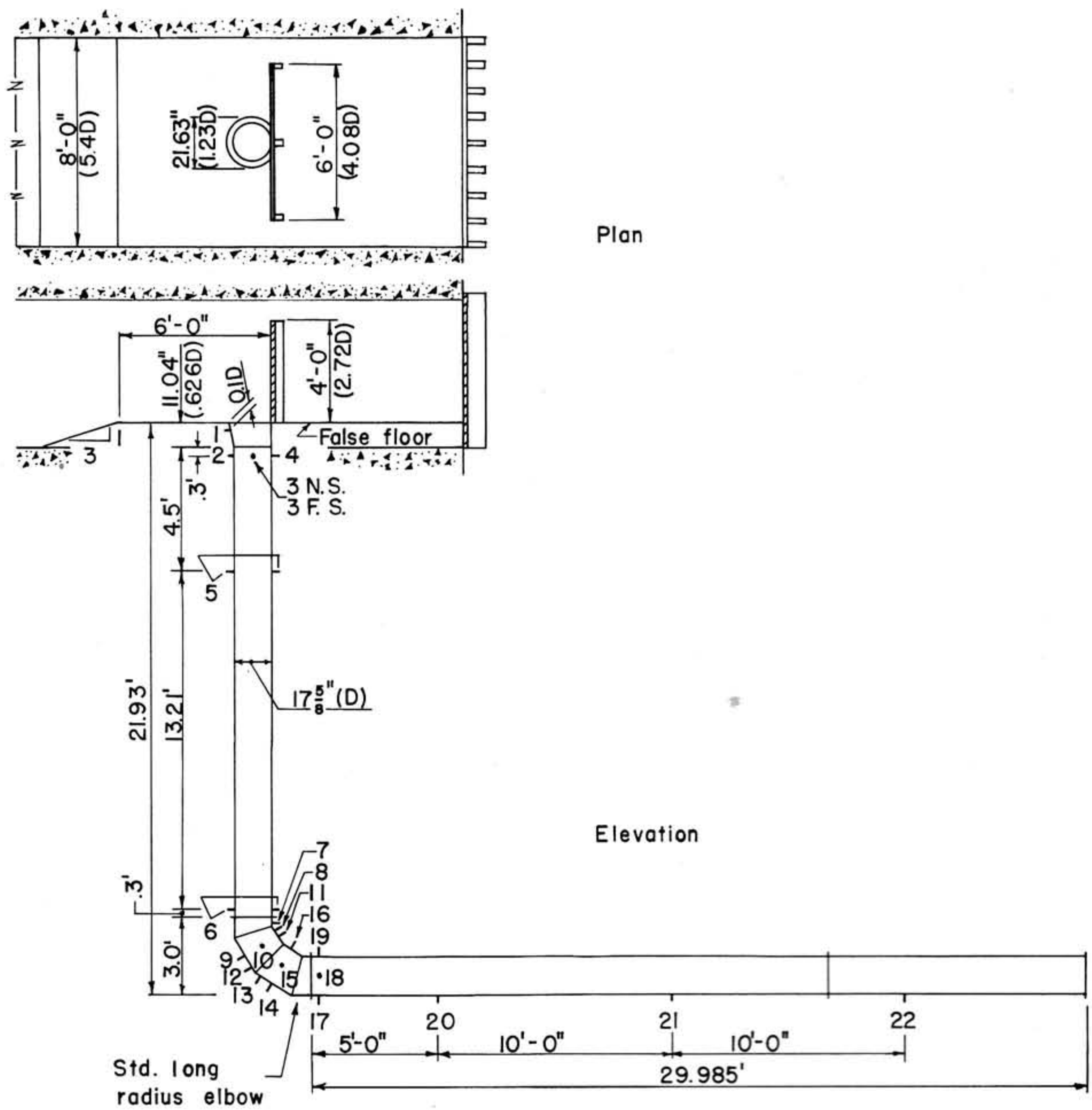


Fig. VIII-1 - 18-in. Steel Pipe Closed Conduit Spillway.

The form and dimensions of the drop inlet, the elbow, and the barrel are shown in Fig. VIII-1.

The pipe was made of spirally welded steel tubing having a nominal diameter of 18 in. and an actual inside diameter of 17-5/8 in. At the conclusion of the tests the pipe surface was covered with a uniform rust having the appearance and feel of sandpaper. This surface is shown in Fig. VIII-2.

APPARATUS AND PROCEDURE

The entrance to the closed conduit spillway was located near the end of the waste tunnel for the main test channel at the St. Anthony Falls Hydraulic Laboratory. This tunnel is 8 ft 0 in. wide by 5 ft 7 in. high. A greater width would have been desirable for the purposes of these tests. There was a bend in this tunnel about 35 ft upstream from the spillway entrance. What effect this had on the flow conditions is unknown since it was impossible to enter the tunnel during the tests to observe the flow. The tunnel exit was bulkheaded so all water was forced through the spillway. The drop inlet passed through a 3/4-in. steel plate covering a hole in the tunnel floor. Since the tunnel at this point crossed the Laboratory turbine room just under its roof, the drop inlet and the horizontal barrel were in the turbine room proper. This provided access to the outside of the spillway for the installation of the piezometers used to measure pressures in the spillway. A heavy concrete thrust block braced against the turbine room wall was placed at the elbow and a heavy anchor block was placed at the barrel exit to take care of the large forces anticipated and to prevent vibration. The discharge from the pipe was allowed to spill on the turbine room floor and to find its way into the tailrace.

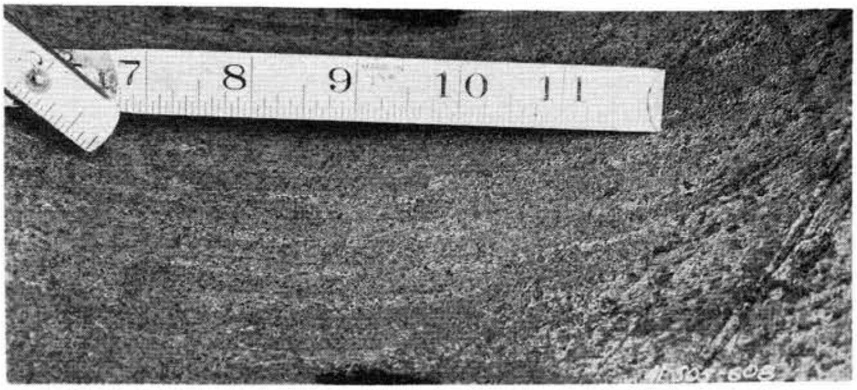


Fig. VIII-2 - Surface of 18-in. Steel Pipe.

The headwater level was determined at a point 12 ft upstream from the anti-vortex wall. Levels were read from a water manometer and a water level recorder was used to insure that a constant water surface elevation existed during each test. Piezometric pressures were obtained at the 22 points indicated in Fig. VIII-1. Pressures were read from mercury manometers.

Discharges were measured by using the tailgate at the end of the main test channel as a suppressed weir. This weir was calibrated after the tests had been completed.

Test procedures were similar to those used for previous tests.

DESCRIPTION OF FLOW

This description is being written 11 years after the tests were made. Time has therefore taken its toll of the writer's memory of the details of the events. Vivid, though, is the memory of standing on the stairs leading to the turbine room, hearing air being sucked into the pipe at certain flows, and feeling the accompanying vibration. This vibration was quite noticeable in spite of the fact that the turbine room is of heavy reinforced concrete construction and is founded directly on the underlying sandstone.

Notes obtained during the tests indicate that the flow phenomena are similar to those observed during the tests reported previously. As the flow increased, the depth at the outlet increased until the outlet flowed alternately full and partly full, the outlet finally flowing completely full all of the time. The full-flow condition is shown in Fig. VIII-3. The white appearance of the jet is believed to be a surface layer of a water-air mixture.

The lifting of the false floor and the movement of the anti-vortex wall noted previously are a direct result of the drop in the level of the water as the velocity increased in the vicinity of the inlet. In this region the pressure head was converted to velocity head. The drop in the water surface of course lowered the pressure on top of the false floor. The under side of the false floor had access to the higher upstream pressures where the velocity was low. The difference in pressure caused the false floor to lift out and pass through the spillway. In the case of the anti-vortex wall, the dropdown of water into the inlet under weir flow exposed part of the upstream face of the wall. The pressure on this side of the wall was atmospheric where the wall was exposed and was reduced below hydrostatic pressure in proportion to the velocity head where the surface of the wall was wetted. On the downstream side of the anti-vortex wall the velocity was nil and full hydrostatic pressure existed. The difference in pressure on the two sides of the anti-vortex wall caused its upstream movement. This is mentioned here because the designer should realize that there is a considerable force acting on this type of anti-vortex wall in an upstream direction.

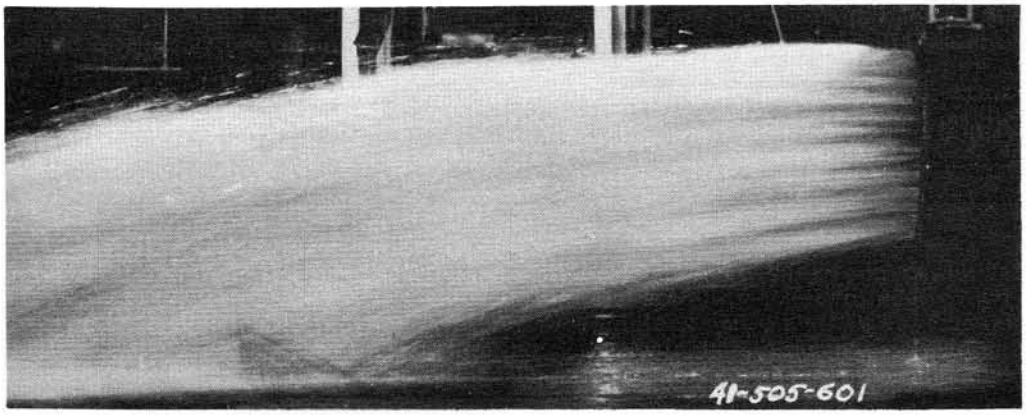


Fig. VIII-3 - Full Flow at Exit of 18-in. Steel Pipe.

DISCHARGE COEFFICIENTS

It was possible to obtain a good value for the weir coefficient but no valid determination of the entrance loss coefficient was obtainable from the data.

Weir Coefficient

The discharge coefficient C for use in Eq. I-1 is given in Fig. VIII-4. The plotted points are those obtained during the tests. The curve shown is suggested for design use, subject to the restrictions indicated below. This curve has the equation

$$C = 3.10 + 1.64\frac{H}{D_{rc}} \qquad \text{(VIII-1)}$$

which, when inserted in Eq. I-1. gives

$$Q = \left[3.10 + 1.64\frac{H}{D_{rc}}\right] L H^{3/2} \qquad \text{(VIII-2)}$$

As indicated in Fig. VIII-4 these equations are valid up to a head H of $0.95D_{rc}$.

The relative approach channel width W_c/L for this inlet is 1.89. Thus, it would be classed as "narrow" according to Reference I-11, and the coefficient should be multiplied by $1/0.97 = 1.03$ to correct for the effect of the narrow approach channel.

Entrance Loss Coefficient

In order to compute the entrance loss coefficient, it is necessary to know the bend loss and the friction loss. It was thought that the friction losses could be determined from the slope of the hydraulic grade line as measured by the piezometers and the elbow loss could be obtained from the drop in the grade line at the elbow. However, it was not possible to define the hydraulic grade line with sufficient precision. This is probably largely attributable to the relatively short lengths of pipe between the entrance and the elbow and between the elbow and the exit.

An attempt was made to compute the elbow and friction losses after estimating values of the coefficients. These computations resulted in negative entrance loss coefficients--a highly improbably condition. In order to have a zero entrance loss it would have been necessary, if the bend loss coefficient were 0.23, to have a Manning's n of 0.009. A coefficient this low is difficult to imagine for the surface shown in Fig. VIII-2.

Further attempts to evaluate the entrance loss coefficient were abandoned.

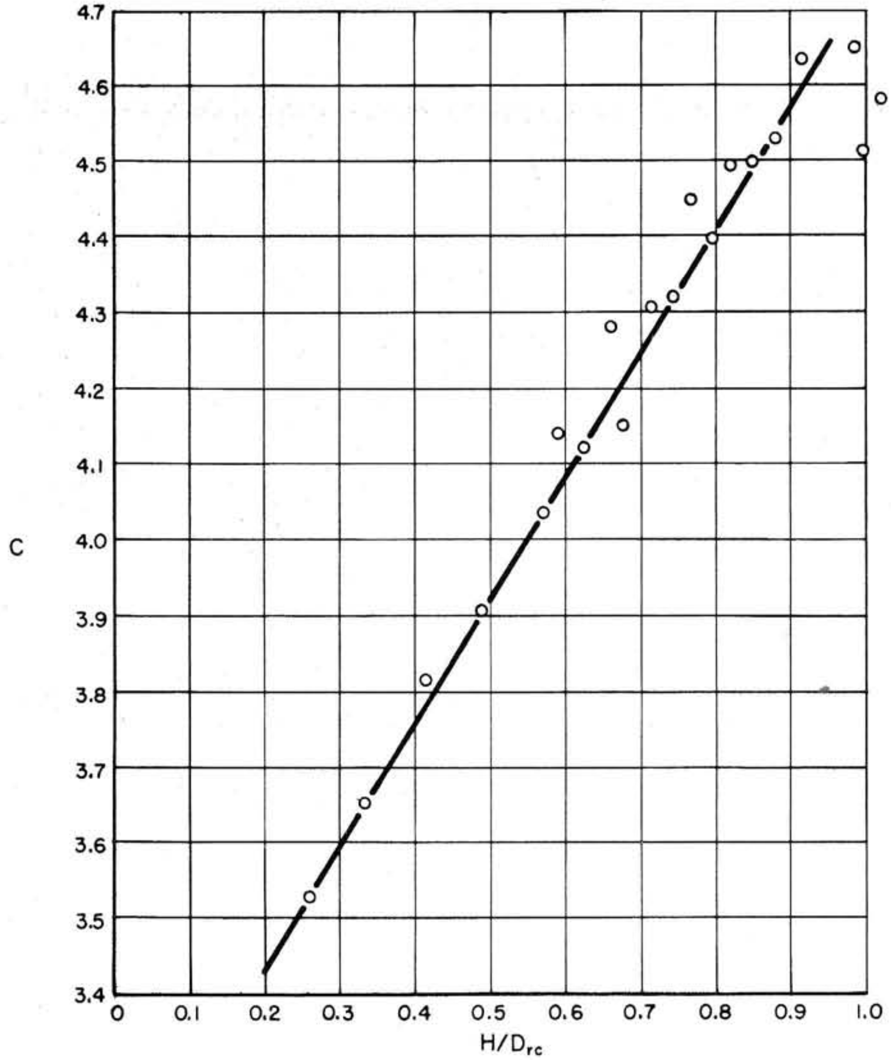


Fig. VIII-4 - Weir Flow Discharge Coefficient for 18-in. Steel Pipe.

PRESSURE COEFFICIENTS

The difficulties outlined above also prevented the determination of the pressure coefficients; good values of the bend and friction loss coefficients are necessary in order to compute the pressure coefficients, so no attempt was made to determine them.

CONCLUSIONS AND RECOMMENDATIONS

Drop inlets having the form shown in Fig. VIII-1 are satisfactory and geometrically similar structures are recommended for field installation. Vibrations sufficient to be felt were transmitted to the turbine room stairway when air was sucked through the spillway. Up-lift pressures can be anticipated in the vicinity of the inlet. Hydrostatic overturning forces on the anti-vortex wall should be anticipated and provided for by the designer.

The capacity of the drop inlet crest acting as a weir can be determined from Eq. I-1 and Fig. VIII-4 or from Eq. VIII-2.

A reliable entrance loss coefficient could not be determined from the available data.

Local pressure constants were not determined.

30-INCH SQUARE DROP INLET CULVERT

A drop inlet spillway located on the Soil Conservation Experiment Station at McCredie, Missouri, is used to determine the runoff from the tributary watershed in addition to its normal use as a gully control structure. It was therefore necessary to calibrate the crest of this spillway in order to obtain a reliable relationship between the head on the spillway crest and the discharge. This calibration was made at the St. Anthony Falls Hydraulic Laboratory on a full scale and on a one-fourth scale model.

The results of these tests are of some general interest since they show that the spillway may not flow full and because a comparison is made between the laboratory-developed rating and field determinations of the discharge.

DESCRIPTION OF SPILLWAY

The closed conduit spillway, which is shown in Fig. VIII-5, is 2 ft 6 in. square in cross section with 4-in. fillets in each corner.

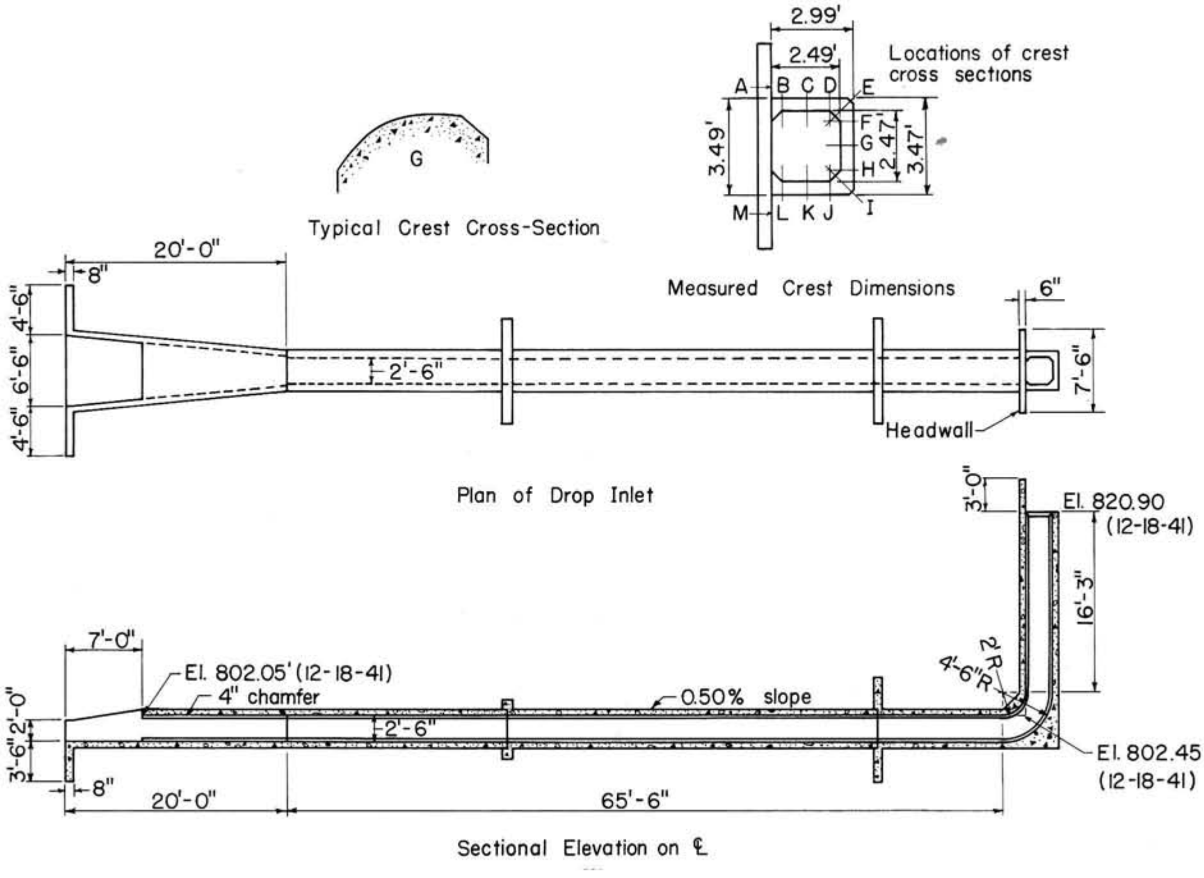


Fig. VIII-5 - 30-in. Square Drop Inlet Closed Conduit Spillway at McCredie, Missouri.

The drop inlet is 16 ft 3 in. high plus 4 ft 6 in. for the elbow, making a total height of 20 ft 9 in. The elbow at its base has a 2-ft inner radius and a 4-ft 6-in. outer radius. There is an anti-vortex wall 3 ft 0 in. high by 7 ft 6 in. long on the downstream side of the crest. The crest itself has a nominal radius of 4 in. The radius apparently was formed by troweling and is therefore somewhat irregular. The actual crest was cross sectioned at 13 points and templets cut to these cross sections were used to construct the model crests. Level readings showed the crest to be level within 0.04 ft.

The barrel is 65 ft 6 in. in length. A flare 20 ft long increases the exit width to 6 ft 6 in. Thirteen feet of this flare are covered. The barrel slope is 0.005.

The upstream slope of the dam is 1 on 3. It is covered with broken stone of about one man size to an elevation of one foot above the spillway crest. Comparative views of the riprap in the vicinity of the crest are shown in Fig. VIII-6 for the prototype and for the two models.

The riprap is depressed in the vicinity of the crest. Elevations of the riprap for 20 ft either side of the drop inlet were furnished in order to reproduce accurately the riprap elevations in the model.

The model crests were made of cement mortar carefully shaped to conform to the field measurements. Only 4 ft of drop inlet were reproduced for the full-scale model. This model was used to check the results obtained on the one-fourth scale model and to accurately determine the head-discharge relationship for the lower heads where the effect of surface tension might affect the capacity of a reduced size model. The crest of the one-fourth size model was of mortar, but the remainder of the spillway was of galvanized sheet metal.

APPARATUS AND PROCEDURE

A special test basin 20 ft square by 5 ft deep was built for these tests. The basin was located over a 10-in. square opening into the tunnel leading to the Laboratory weighing tanks. These tanks were used to determine the flow. Calibrations have shown them to be accurate to within 0.05 per cent.

The water was obtained from a 12-in. supply line leading from the Laboratory supply channel, passed through a control valve and discharged into the test basin. The water left the test basin through the opening into the tunnel, passed through the weighing tanks, and was returned to the river.

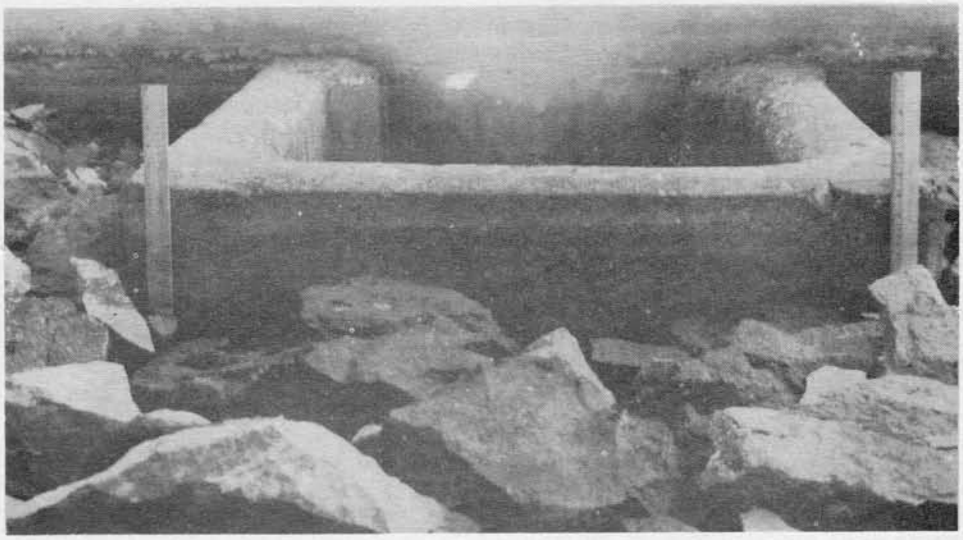
One-Fourth Scale Model

It is desirable, especially for purposes of calibration, to use large-sized models. The largest complete model that could readily be accommodated in the Laboratory was one fourth the size of the prototype. The barrel of this model was located in the waste tunnel, the drop inlet projecting into the test basin through the 10-in. square hole. Unfortunately, sufficient space was not available between this hole and the weighing tank diverter chamber for the installation of the barrel. In order to get the barrel into the tunnel, it was reversed in direction with respect to the drop inlet; that is, the barrel was directed upstream instead of downstream as in the prototype. It is felt that this dissimilarity had no adverse effect on the flow characteristics of the spillway.

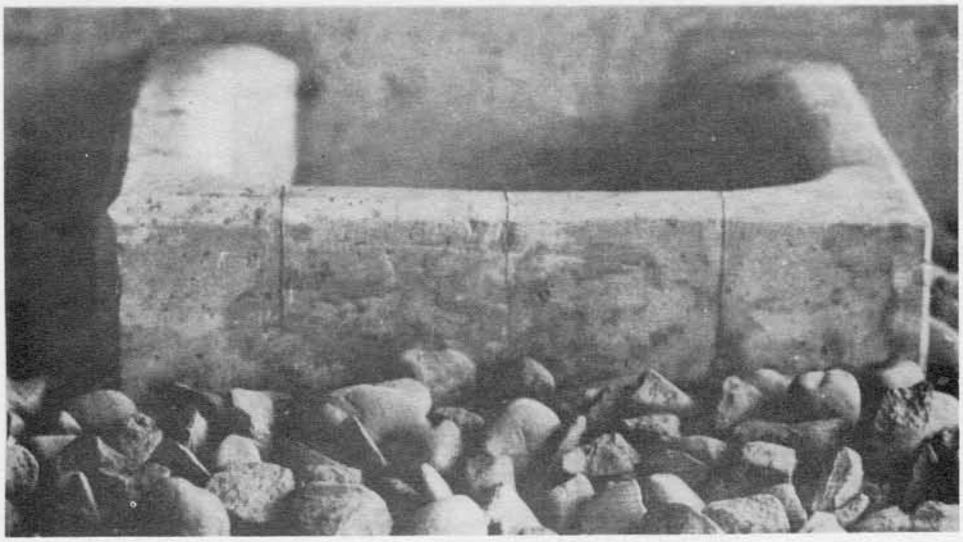
An attempt was made to hold the dimensions to within 0.01 in. of the true dimension.

Full-Scale Model

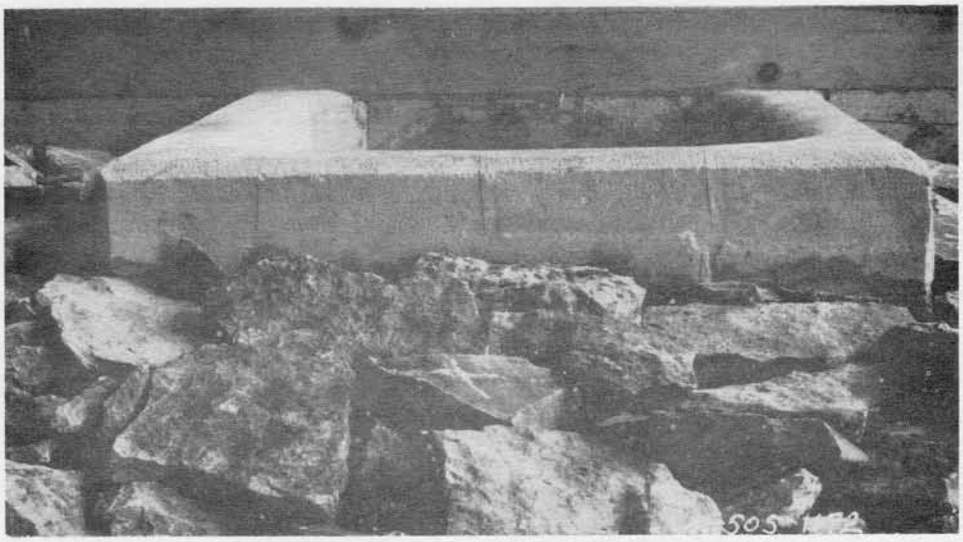
The full-scale model was built over the 10-in. square opening into the waste tunnel. It was realized that the size of this opening would limit the flow. However, the tests on the full-scale model were planned to define accurately the head-discharge relationship only at the lower heads where surface tension might distort the results obtained on the one-fourth scale model. This limitation on the flow that could be accomodated was therefore not serious; complete similarity between the full model and the prototype was obtained for the lower heads at which this model was operated.



(a) Prototype.



(b) Fourth-scale model.



(c) Full-scale model.

Fig. VIII-6 - Crest of Drop Inlet Spillway at McCredie, Missouri.

TEST PROCEDURE

The procedure for conducting a test was as follows: The control valve in the 12-in. supply line was adjusted to give the approximate desired flow, and this flow was allowed to run through the model until the rate of flow through the model and the weighing tanks had become constant. A head reading was then obtained, the rate of flow determined, notes made on the operation of the model, and the head reading checked. The control valve was then opened or closed to change the rate of flow, and the procedure was repeated. Photographs were taken of typical flow conditions.

The rate of flow was determined by the time required to fill the weighing tanks. The head over the crest was determined with the aid of a point gage which was read to 0.001 ft. This gage was located in a glass stilling well which was connected to the headpool some distance upstream from the crest and at one side of the test basin.

A water level recorder was in operation throughout the tests. This recorder was used as a control only and no readings were taken from it. Through its use it was possible to determine if the water level had reached a constant elevation or if the water surface was fluctuating.

DESCRIPTION OF FLOW

Three conditions of flow were observed for the one-fourth size model: weir flow, orifice flow at the drop inlet crest, and full conduit flow. The full-size model was designed to reproduce only the low heads over the crest encountered during weir flow. Head-discharge

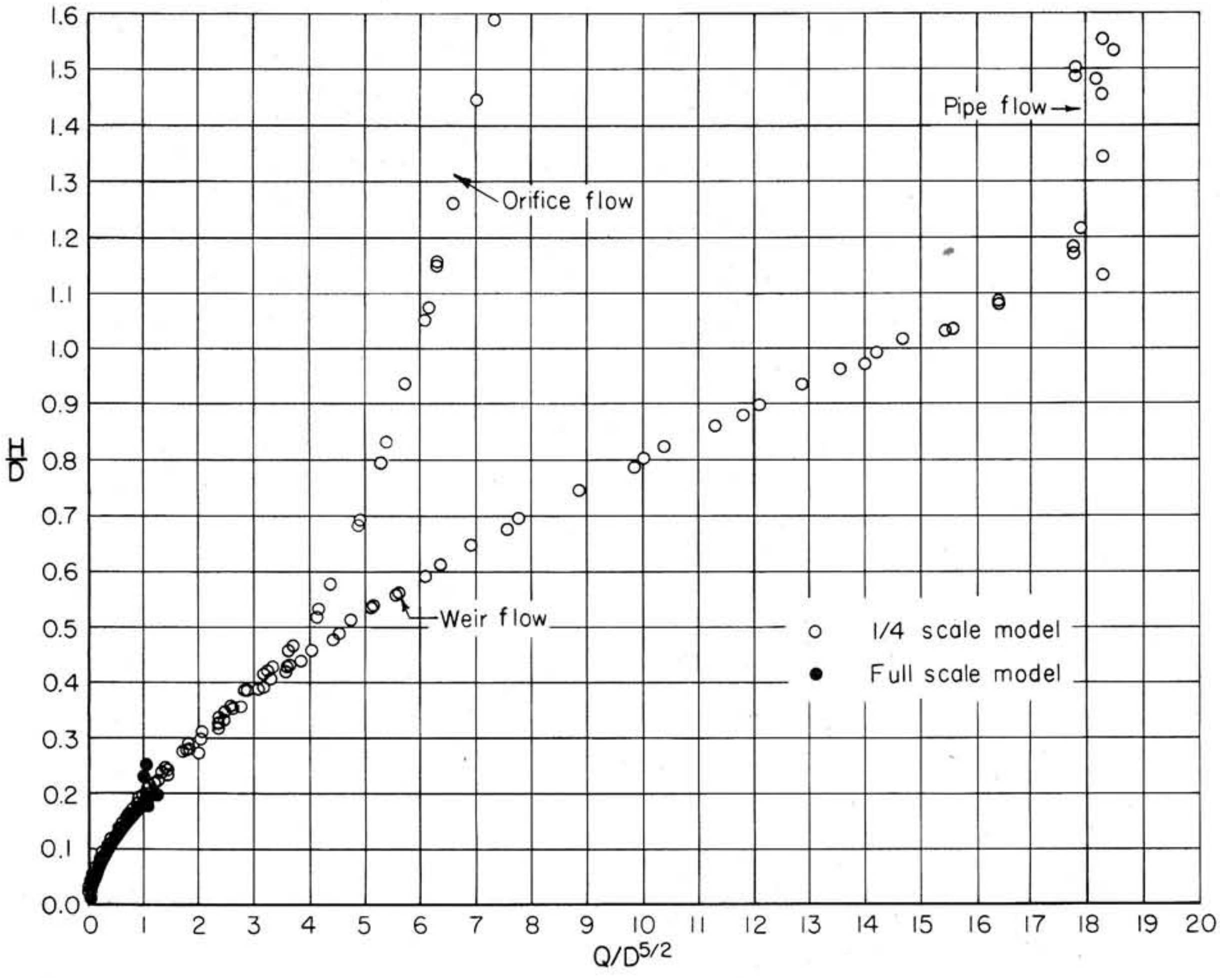


Fig. VIII-7 - Head-Discharge Curve for McCredie, Missouri, Closed Conduit Spillway.

data from 129 runs on the one-fourth size model and 83 runs on the full model are shown in Fig. VIII-7.

For the full-scale model the nappe clung to the crest and the sides of the drop inlet until a relative head H/D of 0.06 was reached. At higher heads the nappe began to spring free. By the time $H/D = 0.11$, the nappe was completely free. Corresponding values for the one-fourth size model are 0.12 and 0.15. Quantitative similarity was not anticipated in this respect because of surface tension effects, the full-scale model being built so the surface tension effects would be the same as in the prototype.

Tests on the one-fourth size model were made with both free and clinging nappes. Up to $H/D = 0.20$ no change in the flow was caused when the nappe changed from freely falling to clinging. Between $H/D = 0.20$ and 0.24 the discharge for the free nappe condition averages about 2 per cent less than for the clinging nappe. Above $H/D = 0.24$ the discharge for the free nappe was considerably less than for the clinging nappe. This can be seen in Fig. VIII-7. The inlet finally became submerged with the water falling freely down the drop inlet. This results in orifice control at the drop inlet crest. When the water clings to the sides of the drop inlet, the weir at the inlet crest controls the head-discharge relationship to much higher flows. It should be noted that the weir head-discharge curve can apply from the lowest flow up to the discharge where full conduit flow controls the head-discharge relationship.

It can be seen in Fig. VIII-7 that it is possible to have two widely different discharges at certain heads. Both free and clinging nappes were observed for the one-fourth size model, but only the free nappe was observed for the full-scale model when H/D was in excess of 0.1. The following quotation, taken from a letter written by Mr. D. D. Smith, formerly Project Supervisor at McCredie, Missouri, to Mr. Neal E. Minshall, Project Supervisor at Madison, Wisconsin, who was making the runoff studies there, is pertinent. "As I recall, the nappe did not cling to the inside of the riser for the higher heads, although it did on the lower." In view of the full-scale test results and Mr. Smith's statement, it appears likely that the prototype nappe is free at the higher heads and that the model findings are verified.

The important thing to remember in connection with this drop inlet closed conduit spillway is that one cannot be certain whether the weir and pipe controls will exist at the higher discharges or whether orifice control will exist. If the control is orifice--the more probable situation--the capacity of the structure is much less than if pipe flow conditions exist.

DISCHARGE COEFFICIENTS

Weir Coefficient

Coefficients of discharge C for weir flow conditions are plotted in Fig. VIII-8. These are for use in Eq. I-1. The crest length L is the length taken at the point of horizontal tangency of the crest rounding. It is not the inside length of the crest. Although all full-scale model data are plotted in Fig. VIII-8, the full-scale data obtained at heads H/L in excess of 0.035 are not considered typical or similar to the one-fourth size model, as is noted below.

No general equation for the coefficient of discharge was obtained. However, the equation for weir flow given in the project report* can be written

$$Q = 4.04\, L H^{1.625} \tag{VIII-3}$$

The solid curve in Fig. VIII-8 is the variable discharge coefficient obtained from Eq. VIII-3 for use in Eq. I-1; the coefficient is variable because the exponent of H is 1.625 instead of 1.5.

The ability of Eq. VIII-3 to predict the discharge was tested by computing the discharge using the equation and comparing it with the discharge observed during the tests. The average of the percentage differences between the computed and the observed discharges for the one-fourth scale model is 0.1 per cent with a maximum of +5.9 per cent and a minimum of -3.9 per cent. Only three differences out of 90 were greater than 4 per cent.

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*Fred W. Blaisdell, Model Calibration of Drop Inlet Culvert Located at McCredie, Missouri, Soil Conservation Service Project Report No. MN-R-3-25, St. Anthony Falls Hydraulic Laboratory, October 1944.

The average percentage difference between the equation and the observed discharges for the full-scale model is -2.7 per cent with a maximum of +12.1 per cent and a minimum of -21.7 per cent. Fifty-one runs were used in computing the average difference. The maximum head reading used was 0.11D (0.035L) since the turbulence in the test basin at higher flows was considered excessive and not typical. The maximum and minimum differences occurred at relative heads H/D of 0.05 and 0.005. The rating could, at some loss in simplicity, have been corrected for the large percentage difference at low heads. However, the maximum actual difference was only about 0.1 cfs and a correction of this magnitude would not be warranted from a practical point of view.

In April 1944 Mr. Neal E. Minshall prepared a field rating for the McCredie drop inlet closed conduit spillway. This rating is based on storage depletion after ten storms that occurred in 1941, 1942 and 1943. The rate of storage depletion was computed from water level recorder records for each 0.03 ft reduction in level and the area of the pond was determined by careful surveys. Mr. Minshall's rating curve had the equation

$$Q = 26.5\ H^{1.5} \qquad \text{(VIII-4)}$$

Mr. Minshall kindly loaned his data so his results could be compared with the rating equation developed from the Laboratory tests. The average difference between the discharges determined from the Laboratory equation and from the field rating was +2.8 per cent for 132 determinations of the discharge at relative heads between 0.02 and 0.2. The differences varied from a maximum of +55 per cent to a minimum of -37 per cent. There was a very considerable scatter to the data. The trend of the differences was, however, increasing with the head.

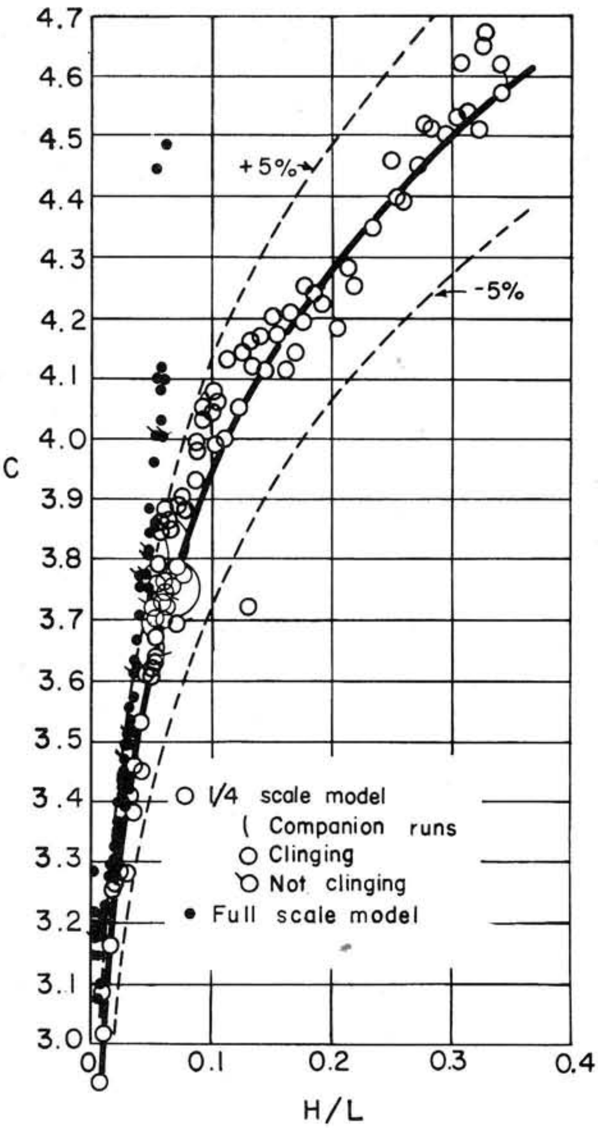


Fig. VIII-8 - Head-Weir Coefficient Curve for McCredie, Missouri, Closed Conduit Spillway.

It can be concluded that the rating equation obtained as a result of the Laboratory tests (Eq. VIII-3) gives a satisfactory prediction of the prototype flow for weir control.

Orifice Coefficient

The orifice flow data obtained from the one-fourth size model are plotted in Fig. VIII-9. The head H_o shown is head above the crest; $H_o = H$ in this case. The manner of plotting was adopted to determine if a head correction was necessary since from the shape of the crest it was not possible to determine the exact location of the zero head elevation. The plot indicates that no head correction is required when H/D is greater than 0.14 (H_o = 0.35). Apparently orifice control does not exist below H/D = 0.06 (H_o = 0.15), so that between H/D = 0.06 and 0.14 the data do not follow the curve drawn for the higher heads.

The discharge coefficient C_o for use in Eq. I-7 is

$$C_o = 6.15 = 0.767\sqrt{2g} \qquad \text{(VIII-5)}$$

The area A is the cross sectional area of the drop inlet.

Entrance Loss Coefficient

It is not possible to determine the entrance loss coefficient for full conduit flow without an exact determination of the friction factor for the conduit. This friction factor was not measured. However, the Manning n was estimated to be 0.009 and K_e for use in Eq. I-5 computed on that basis. According to Rouse [I-43, p. 211] f = 0.019 for galvanized iron, such as was used in the model, and n = 0.0094. The agreement is close. The mean value of K_e obtained from 9 runs was 0.15. If n = 0.010 had been used then K_e would have been 0.01. It appears that K_e = 0.15 is a satisfactory value for use with this drop inlet closed conduit spillway.

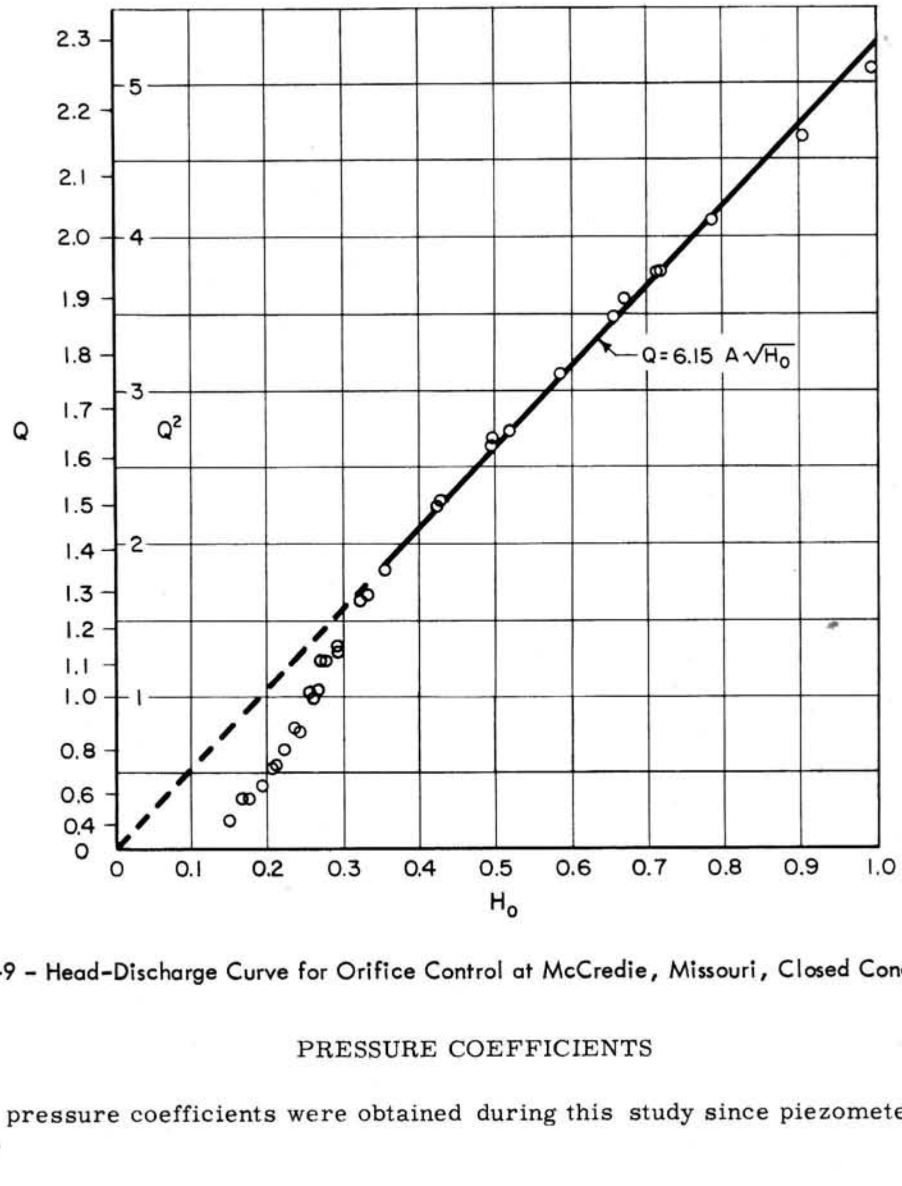


Fig. VIII-9 - Head-Discharge Curve for Orifice Control at McCredie, Missouri, Closed Conduit Spillway.

PRESSURE COEFFICIENTS

No pressure coefficients were obtained during this study since piezometers were not installed.

CONCLUSIONS AND RECOMMENDATIONS

Drop inlets of the same size as the barrel are not recommended because it is possible to obtain, at identical heads, orifice control at the inlet crest as well as weir or pipe control. Thus it is impossible to determine the actual flow through the spillway for relative heads in excess of H/D = 0.35.

The capacity of the drop inlet crest acting as a weir can be determined from Eq. VIII-3 or from Eq. I-1 if the coefficients of Fig. VIII-8 are used.

The capacity of the spillway, if the control is an orifice at the crest of the drop inlet, is given by Eq. I-7 using the coefficient $C_o = 6.15$.

The entrance loss coefficient K_e for full conduit flow is about 0.15.

3.0-FOOT SQUARE DROP INLET CULVERT

A study was made of a 3-ft square drop inlet culvert when it was desired to add a flaring drop inlet on to a highway culvert already in place. The fact that the slope of the culvert, which was 0.0667, was greater than the friction slope made it questionable whether the barrel would flow full.

Some of the information obtained from the model tests is of general interest since it was shown that a small change in the drop inlet dimensions would greatly improve the performance of the spillway. The tests were conducted in 1941 and therefore are among the earliest made with the culvert barrel on a steep slope.

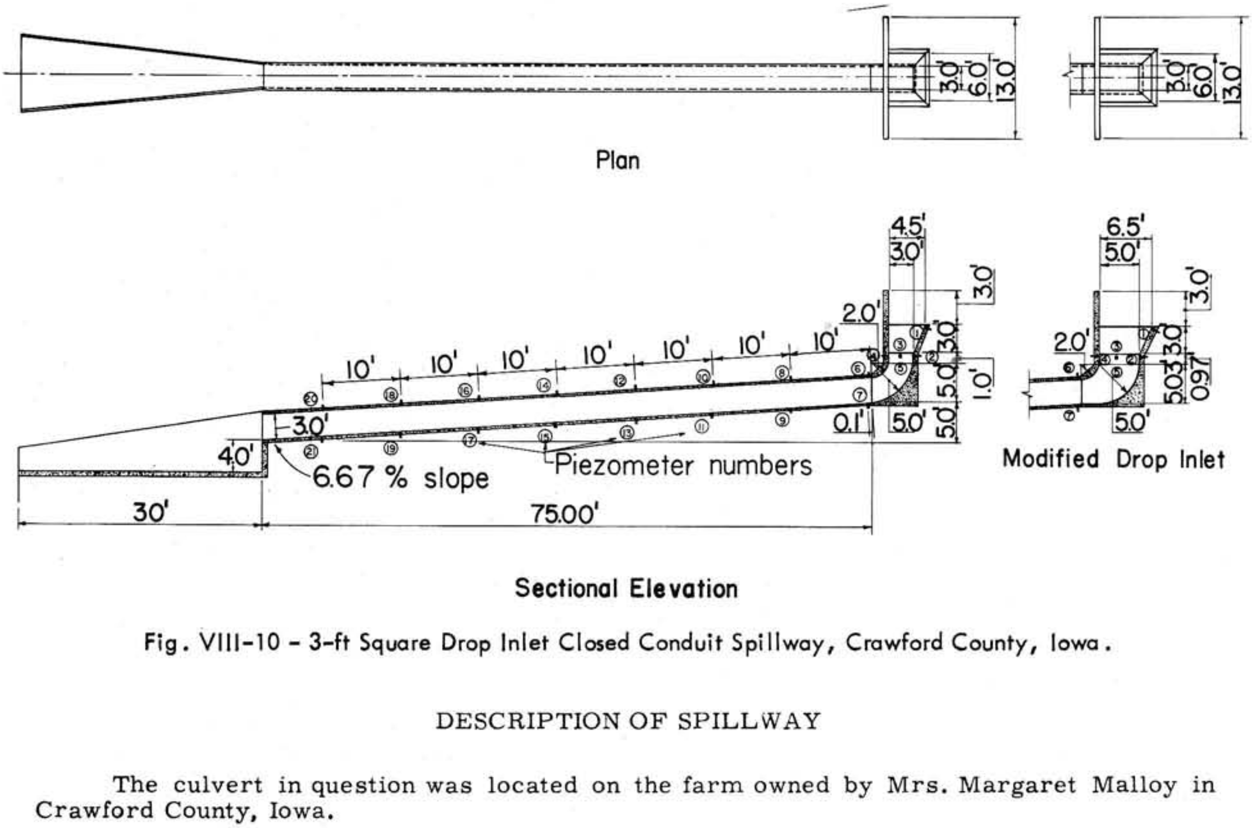


Fig. VIII-10 - 3-ft Square Drop Inlet Closed Conduit Spillway, Crawford County, Iowa.

DESCRIPTION OF SPILLWAY

The culvert in question was located on the farm owned by Mrs. Margaret Malloy in Crawford County, Iowa.

A plan and section of the original culvert and the proposed drop inlet and drop outlet are shown in Fig. VIII-10. The original concrete culvert is 3 ft square by 75 ft long. The proposed flared drop inlet shown is proportioned in accordance with the designs of Hamilton*. The modified drop inlet has the center of the invert radius moved two feet further upstream than in the original design resulting in a drop inlet having a larger area and a longer crest length.

The new spillway was designed for a flow of 198 cfs.

*C. L. Hamilton, Design and Construction of the Drop Inlet Soil Saving Dam, Soil Conservation Service SCS-EP-14, June 1937.

The approach to the drop inlet was modeled to reproduce the topography on Mrs. Malloy's farm. A map of the pertinent topography in the vicinity of the proposed drop inlet spillway is shown in Fig. VIII-11. There it can be seen that the upstream gully was dammed some 400 ft upstream from the highway culvert and that most of the flow approached the culvert at an angle of about 17 degrees with the culvert axis through the vegetated spillway around the end of the upstream dam. In the model all of the water entered the pond through this vegetated channel, the flows entering from the highway ditches being so small as to have a negligible effect on the flow pattern.

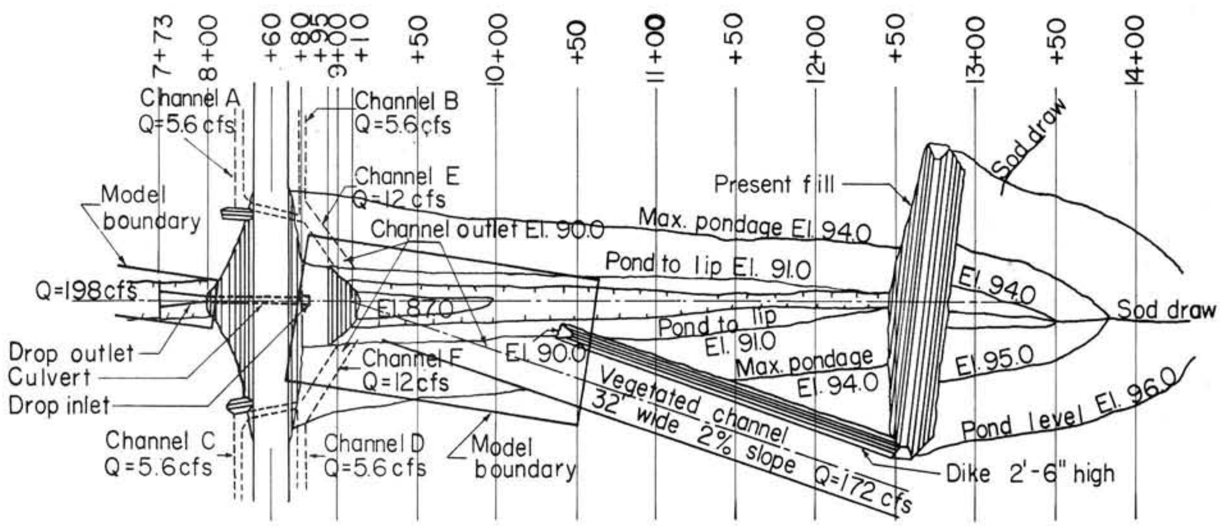


Fig. VIII-11 - Topography in Vicinity of Closed Conduit Spillway at Crawford County, Iowa.

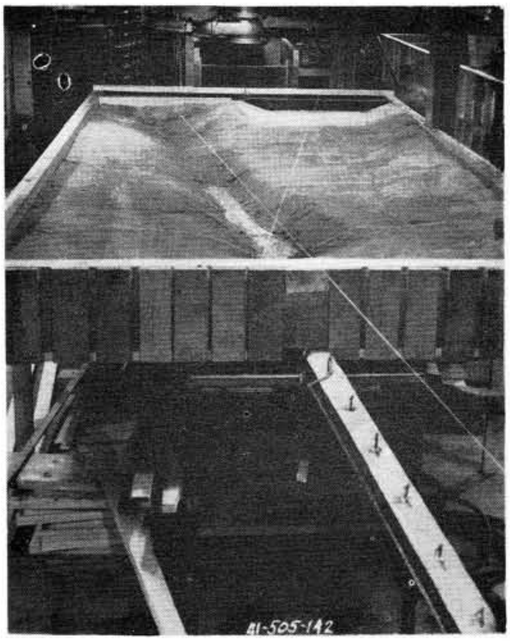


Fig. VIII-12 - Model of Closed Conduit Spillway, Crawford County, Iowa.

APPARATUS AND PROCEDURE

The area included in the model is shown in Fig. VIII-11. The upstream portion of the model represents an area 90 ft wide by 185 ft long to a scale of 1 in 10. It was necessary to skew the model in order to include it in the space available. This is a result of the flow approaching the structure through the vegetated spillway from the upstream structure.

The completed model is shown in Fig. VIII-12. Strings are located along the centerline of the original gully and the structure, along the centerline of the vegetated channel, and the ground surface to locate the cross sections at Stations 9+00, 9+50 and 10+00. The topography of both upstream and downstream channels was shaped in concrete sand. The water enters the model through a 4-in. pipe line.

The drop inlet and culvert were fabricated from No. 14 gage galvanized sheet metal. Piezometers were located in the drop inlet and along the barrel at the points indicated in Fig. VIII-10.

Elevations of the water surface and the pressure heads along the barrel were measured by means of point gages to 0.001 ft (0.01 ft in the prototype). The rate of flow was measured with a 1.0 ft type HS flume.

DESCRIPTION OF FLOW

Three flow conditions were observed for the original drop inlet design: weir flow, short tube flow, and pipe flow. Only weir and pipe flow were observed for the modified drop inlet. The head-discharge data are shown in Fig. VIII-13 and the type of control for each branch of the curve is labeled thereon. The two-section weir flow curve obtained for the original inlet needs no further explanation in view of the discussion given in Part I under the heading "Composite Head-Discharge Relationship" since the action is similar. A continuous weir flow curve was obtained for the modified inlet.

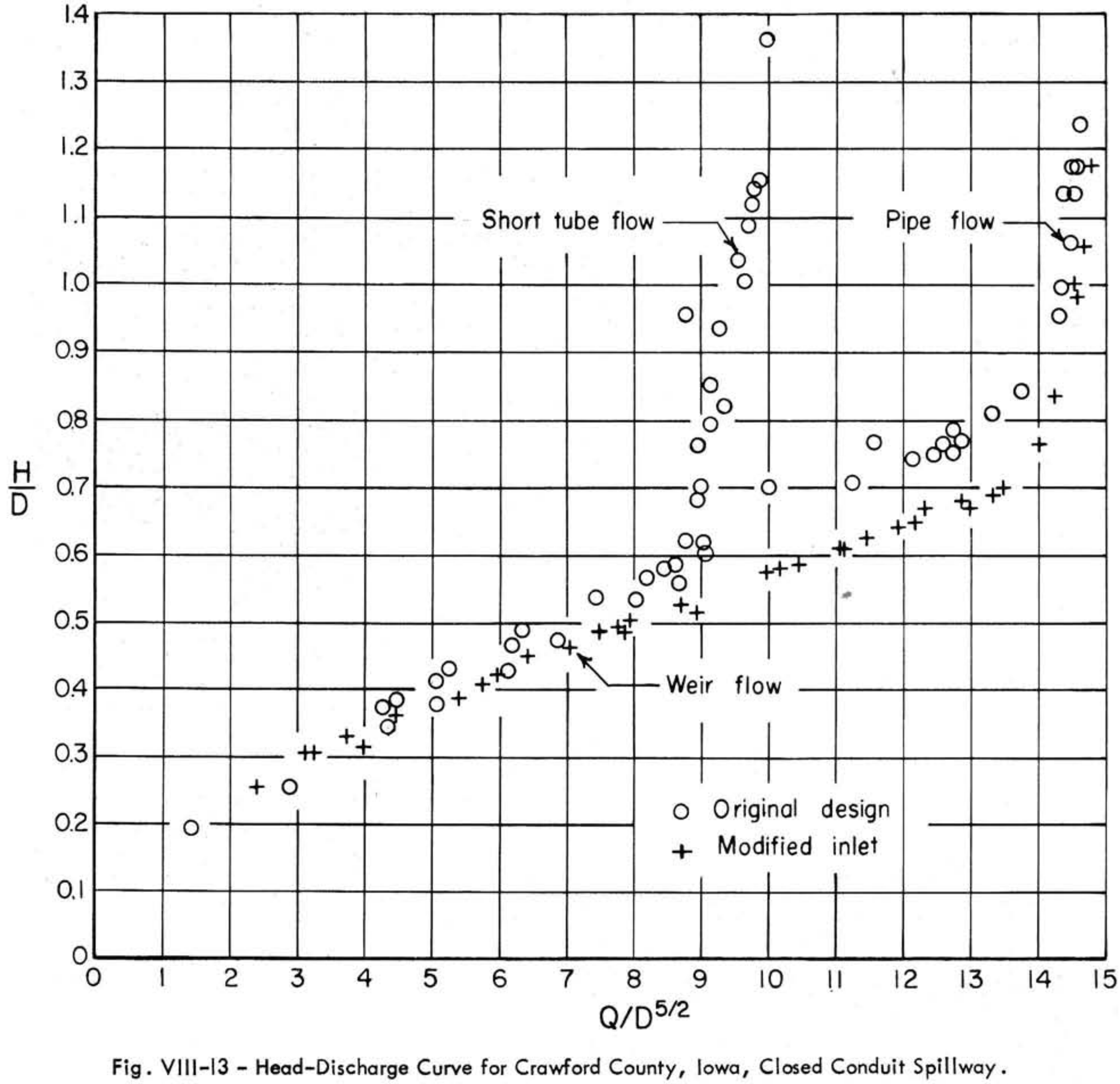


Fig. VIII-13 - Head-Discharge Curve for Crawford County, Iowa, Closed Conduit Spillway.

The original inlet acts like a short tube over a short range of discharges. Again the action has been described in Part I. The build-up of head along the lower weir curve and along the short tube curve, the sudden increase in flow occurring when the control jumps to the pipe curve, and the drawdown along the pipe and upper weir curves have all been described. It has been pointed out in Part I under the heading "The Composite Head-Discharge Relationship" that the type of head-discharge relationship obtained for short tube flow is undesirable. In the case of the model study discussed here, the highway would have been overtopped because the

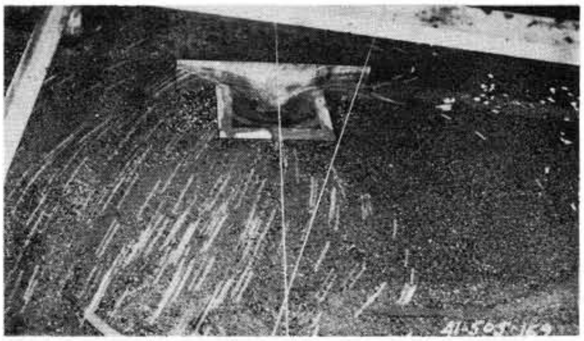


Fig. VIII-14 - Paper Punchings Trace Surface Currents Near Original Drop Inlet of Crawford County, Iowa, Closed Conduit Spillway. H/D = 0.83.

capacity of the original drop inlet culvert under short tube flow was less than the design capacity ($Q/D^{5/2} = 12.7$).

In contrast to the conditions observed in the original drop inlet, the capacity of the modified drop inlet was sufficient to prevent overtopping of the highway at the design flow. The reason for this is that the inlet modifications were such that short tube flow did not occur; only the desirable weir and pipe flow controls were obtained for the modified drop inlet.

The flow through the vegetated channel (see Figs. VIII-11 and VIII-12) into the pond above the drop inlet caused circulation in the pond and the water approached the drop inlet at an angle. This can be seen in Fig. VIII-14 where paper punchings show the velocity on the left to be high and that on the right to be negligible. All this aggravated the tendency to circulation around the anti-vortex wall. The large quantity of water circulating behind the anti-vortex wall from the left to the right and entering the drop inlet from the right is shown by dye in Fig. VIII-15. An earth dike behind the anti-vortex wall prevents the circulation in back of the anti-vortex wall in Fig. VIII-16. The much better distribution of the velocities approaching the drop inlet is readily apparent when Fig. VIII-16 is compared with Fig. VIII-14. A comparison of the scour around the inlet without and with the dike is shown in Fig. VIII-17, the much more symmetrical pattern for the inlet with the dike being readily apparent. The performance of the modified inlet was much better than that of the original inlet even though the dike was omitted.

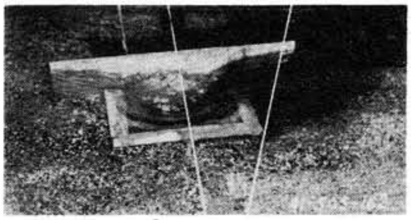


Fig. VIII-15 - Dye Injected Behind Anti-Vortex Wall Traces Path of Flow for Original Design of Crawford County, Iowa, Closed Conduit Spillway.

The dike from the headwall to the highway fill improved flow conditions and reduced the scour around the inlet but its effect on the spillway capacity could not be detected.

DISCHARGE COEFFICIENTS

Weir Coefficient

Coefficients of discharge C for weir flow conditions are plotted in Fig. VIII-18. These are for use in Eq. I-1. The crest length L is the inside length since there was no rounding of the crest. The curves were determined by first plotting $(Q/D^{5/2})^{2/3}$ against H/L and determining the head-discharge equations from the straight lines obtained from this plot. For the original inlet the equation is

$$Q = 3.90\, L\,(H - 0.002\, L)^{3/2} \qquad \text{(VIII-6)}$$

while for the modified inlet the equation is

$$Q = 4.23\, L\,(H - 0.009\, L)^{3/2} \qquad \text{(VIII-7)}$$

The equations for the discharge coefficient C in Eq. I-1 are, for the original inlet,

$$C = 3.90\left[1 - 0.002\,\frac{L}{H}\right]^{3/2} \qquad \text{(VIII-8)}$$

and, for the modified inlet,

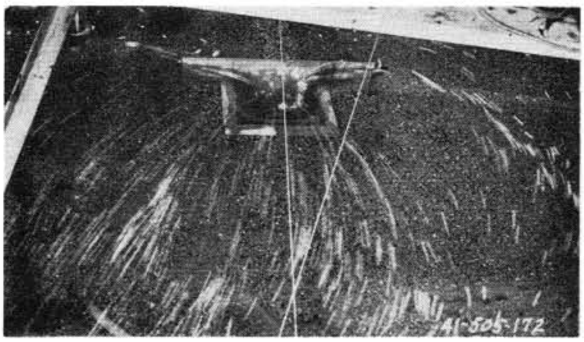


Fig. VIII-16 - Paper Punchings Show How Dike From Anti-Vortex Wall to Roadway Fill Prevents Flow in Back of Wall and Improves Flow Distribution Around Drop Inlet of Crawford County, Iowa, Closed Conduit Spillway.

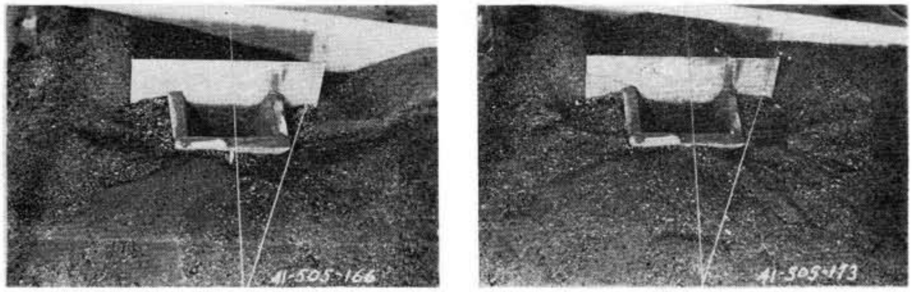


Fig. VIII-17 - Scour in Vicinity of Inlet of Crawford County, Iowa, Closed Conduit Spillway.

$$C = 4.23 \left[1 - 0.009 \frac{L}{H} \right]^{3/2} \qquad \text{(VIII-9)}$$

Equations VIII-8 and VIII-9 have been plotted as solid lines in Fig. VIII-18. The reason for the rather wide differences between the coefficients for the original and modified inlet is quite likely because the effect of extending the weir crest two feet upstream is to cause more symmetrical flow into the inlet, as can be seen by comparing Fig. VIII-19 with VIII-14.

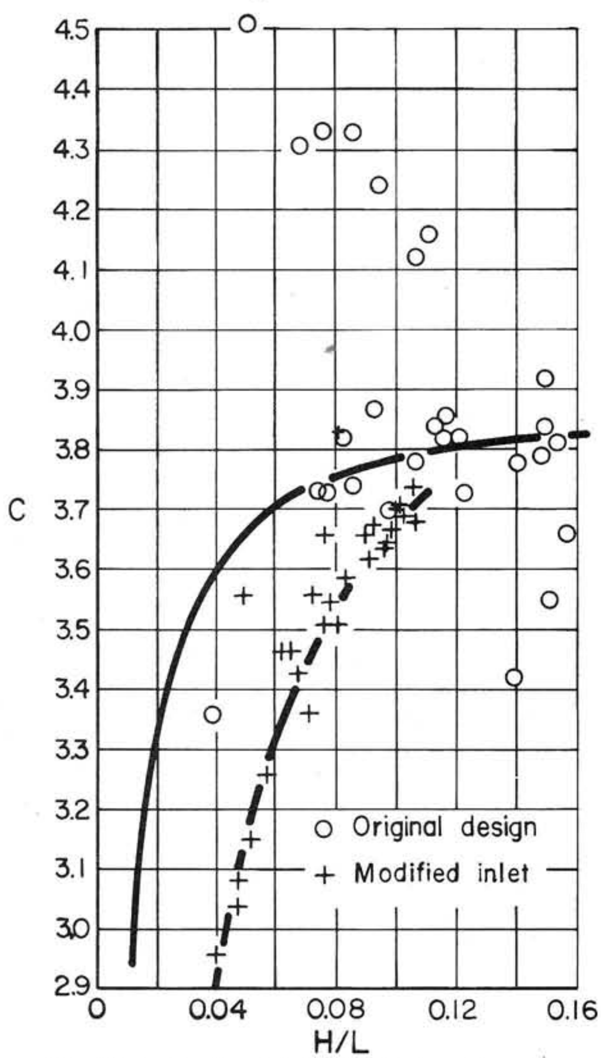


Fig. VIII-18 - Head-Weir Coefficient Curve for Crawford County, Iowa, Closed Conduit Spillway.

Short-Tube Coefficient

Short-tube flow was observed only for the original drop inlet. The equation for short-tube flow was determined by plotting $(Q/D^{5/2})^2$ against H/D as is shown in Fig. VIII-20. This method of plotting was used because the data points theoretically fall on a straight line and the projection of this line can be used to determine the effective head causing short-tube flow. In Eq. I-8 the discharge coefficient C_s was found to be 1.91. The area A is that of the vertical section of the drop inlet or 9 sq ft in the prototype, and the head H_s is $H + 1.50D$, where H is the head on the crest. The equation for short-tube flow is

$$Q = 1.9\,A\sqrt{H + 1.50\,D} \qquad \text{(VIII-10)}$$

The flared portion of the drop inlet is 1.0D deep and the vertical portion is 0.33D deep. Subtracting this sum from 1.50D gives 0.17D as the effective distance below the beginning of the elbow curve at which the water breaks away from the drop inlet wall. Whether or not this distance will be the same for other drop inlet heights cannot be definitely stated from the available data. However, it might be used to determine the effective head for short-tube flow for other drop inlet heights.

Entrance Loss Coefficient

The entrance loss coefficient K_e for

use in Eq. I-5 was found to be 0.06 for the original inlet design and 0.04 for the modified inlet. It is the average of eight observations for the original inlet and five observations for the modified inlet.

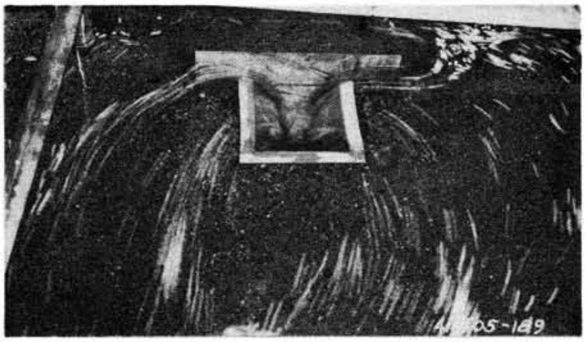


Fig. VIII-19 - Paper Punchings Trace Surface Currents Near Modified Drop Inlet of Crawford County, Iowa, Closed Conduit Spillway.

PRESSURE COEFFICIENTS

The pressure coefficients h_n/h_{vp} for use in Eq. I-14 are given in Fig. VIII-21. The points shown are based on two observations for the original inlet and five observations for the modified inlet.

It is interesting to note that the lowest values of h_n/h_{vp} were obtained just below the flare (Piezometers 2, 3, 4 and 5 in Fig. VIII-10) in the drop inlet for the original design. For the modified design the pressures were positive at this point. The pressure coefficient just inside the barrel at its top (Piezometer 6) is also lower for the original design than for the modified design, while the floor pressure (Piezometer 7) is less for the modified design than for the original design. These findings reflect the greater area and lower velocities as well as the less abrupt change in the flow direction for the modified inlet.

The values of h_n/h_{vp} downstream of Piezometers 6 and 7 should be zero. The values are reasonably close to zero for the modified inlet. The reason why the values of h_n/h_{vp} for the original inlet fall below zero in the barrel is not known.

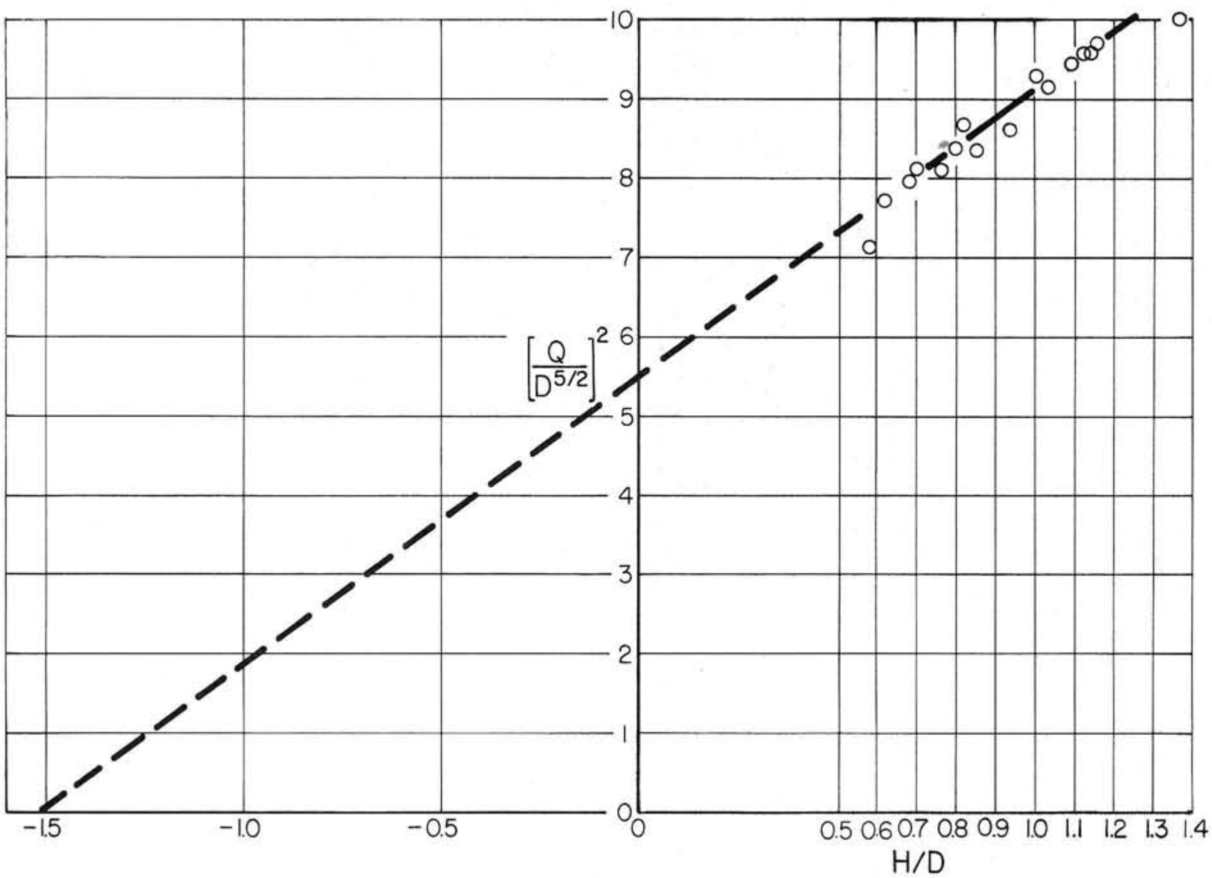


Fig. VIII-20 - Head-Discharge Curve for Short Tube Flow in Crawford County, Iowa, Closed Conduit Spillway.

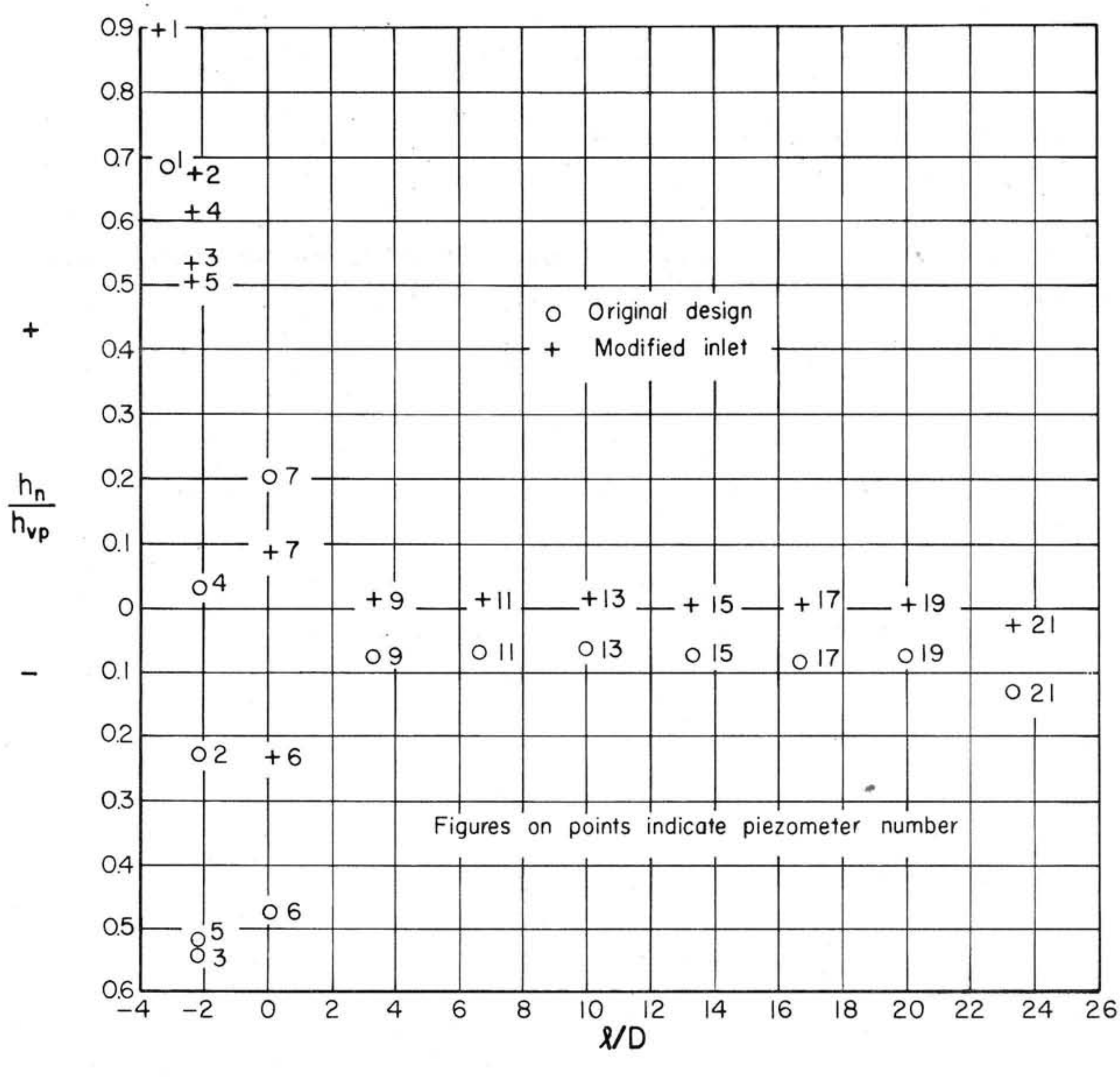


Fig. VIII-21 - Local Pressures for Crawford County, Iowa, Closed Conduit Spillway.

CONCLUSIONS AND RECOMMENDATIONS

It is recommended that the original flared drop inlet design not be used. The presence of short-tube flow makes it impossible to determine positively the rate of outflow or the water surface elevation at a given rate of inflow to the pond.

The modified flared drop inlet shown in Fig. VIII-10 operated satisfactorily and its use is recommended. Values of the weir coefficient for insertion in Eq. I-1 may be taken from Eq. VIII-9 or from Fig. VIII-18. The entrance loss coefficient is 0.04. The local pressure coefficients for insertion in Eq. I-14 are given in Fig. VIII-21.

Part IX

Field Tests

INTRODUCTION

A number of tests have been made by others on closed conduit spillways. Since the form of these spillways is different from any of those previously discussed in this paper, the information obtained is of interest and value. Information on five structures is available: (1) Tests on four shapes of an inlet to a 24-in. concrete pipe and two drop inlets were conducted at Stillwater, Oklahoma, under the direction of W.O. Ree. (2) Tests on a 2-ft square drop inlet closed conduit spillway located near Nelson, Wisconsin, were carried out by the Soil Conservation Service and the University of Wisconsin. (3) Tests on an 8-in. diameter and (4) tests on a 14-in. diameter closed conduit spillway, both located near Edwardsville, Illinois, were reported by Richard P. Weeber. (5) Tests on a 12-in. diameter closed conduit spillway located near Bethany, Missouri, were conducted by A. W. Zingg. Such pertinent information as can be gleaned from these tests will be presented.

24-INCH DIAMETER CLOSED CONDUIT SPILLWAY AT STILLWATER, OKLAHOMA

The following information is taken from the unpublished report by W. O. Ree entitled "Hydraulic Tests of a Pipe Outlet Spillway" dated February 1954 [I-42] which describes tests made at the Agricultural Research Service's Stillwater Outdoor Hydraulic Laboratory, Stillwater, Oklahoma, in 1951. This is not a complete summary of Mr. Ree's report; important

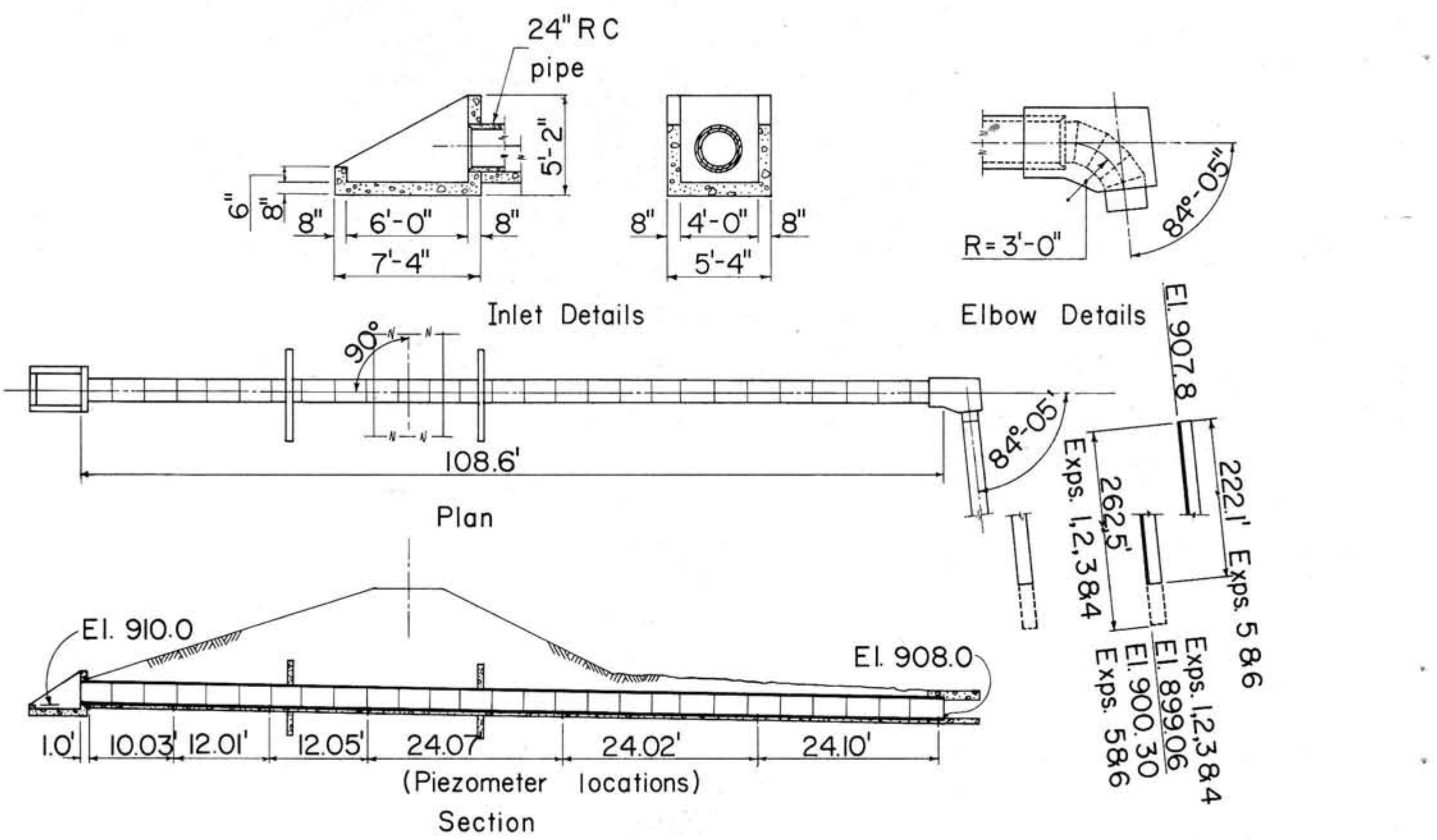


Fig. IX-1 - Closed Conduit Spillway at Stillwater, Oklahoma.

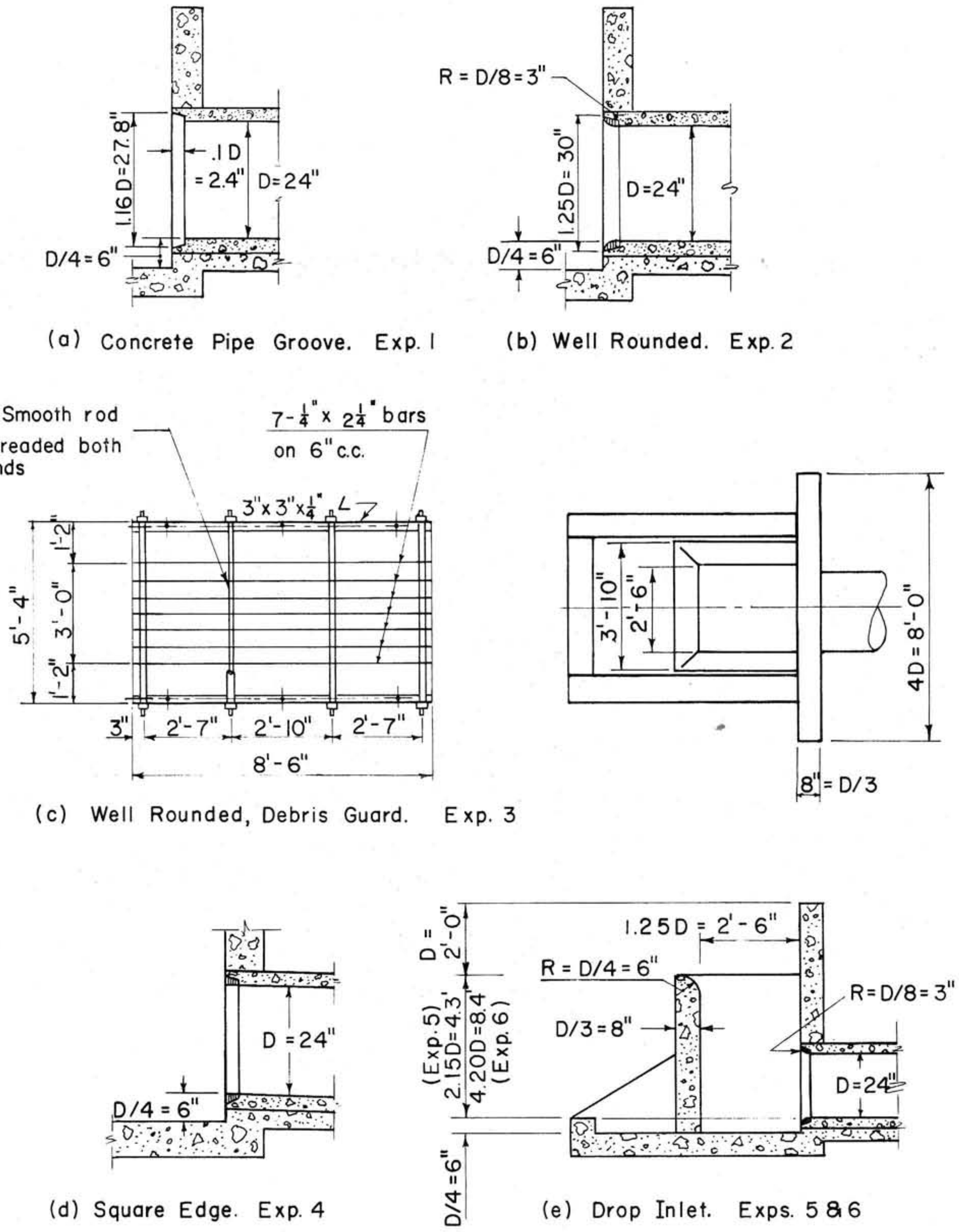


Fig. IX-2 - Entrances for Stillwater, Oklahoma, Closed Conduit Spillway.

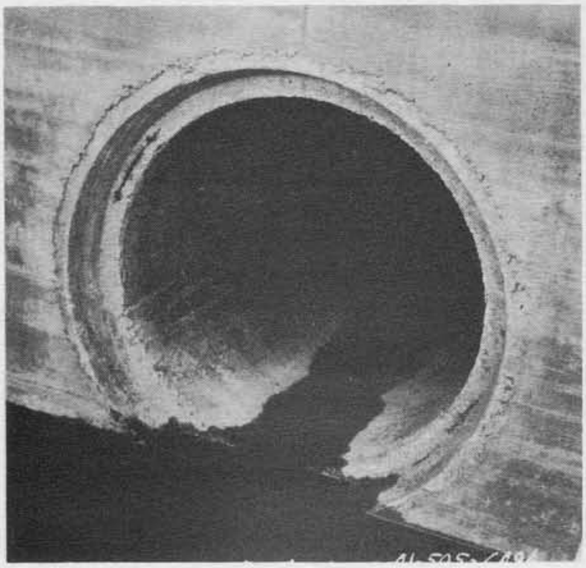


Fig. IX-3 - Concrete Pipe Groove Entrances, Experiment 1, Stillwater, Oklahoma.

Fig. IX-4 - Rounded Pipe Entrance, Experiments 2, 3, 5 and 6, Stillwater, Oklahoma.

Fig. IX-5 - Debris Guard, Experiment 3, Stillwater, Oklahoma.

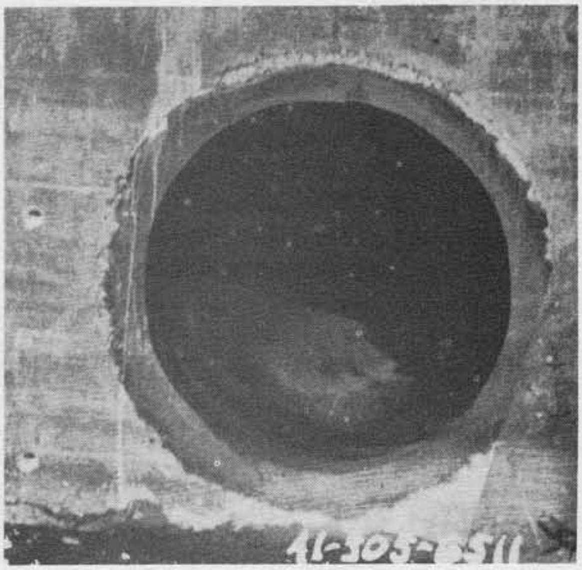


Fig. IX-6 - Square-Edged Pipe Entrance, Experiment 4, Stillwater, Oklahoma.

subjects omitted here include elbow losses, friction factors, leakage of the pipe, and scour of the stream bed at the outlet.

DESCRIPTION OF SPILLWAY

The spillway consists of an entrance structure, 108.3 ft of 24-in. concrete tongue-and-groove pipe, a 24-in. corrugated metal, bituminous-coated pipe elbow having an 84-degree deflection, and 222.1 to 262.5 ft of 24-in. corrugated metal, bituminous-coated, paved invert pipe. Heads on the inlet invert up to 14 ft can be obtained. The total fall of the pipe is 10.94 ft for the longer and 9.70 ft for the shorter corrugated pipes. The slope of the concrete pipe is 0.0185 and the slope of the corrugated pipe is 0.0334. The spillway is shown in Fig. IX-1.

Six different entrances were used. The inlet structure shown in Fig. IX-1 was in place for all entrances but was hydraulically pertinent to only the first four entrances tested.

The concrete pipe groove entrance shown in Fig. IX-3 and detailed in Fig. IX-2a was used for Experiment 1. The groove was filled with mortar and finished to a 3-in. radius for Experiment 2, as shown in Fig. IX-4 and detailed in Fig. IX-2b. The same rounded pipe entrance was used for Experiment 3, but a debris guard was placed over the entrance structure, as shown in Fig. IX-5 and detailed in Fig. IX-2c. The debris guard was removed for Experiment 4 and the pipe entrance given a square edge, as shown in Fig. IX-6 and detailed in Fig. IX-2d. The rounded pipe inlet shown in Figs. IX-4 and IX-2b was restored for Experiments 5 and 6, but was used together with the drop inlet shown in Fig. IX-7 and detailed in Fig. IX-2e.

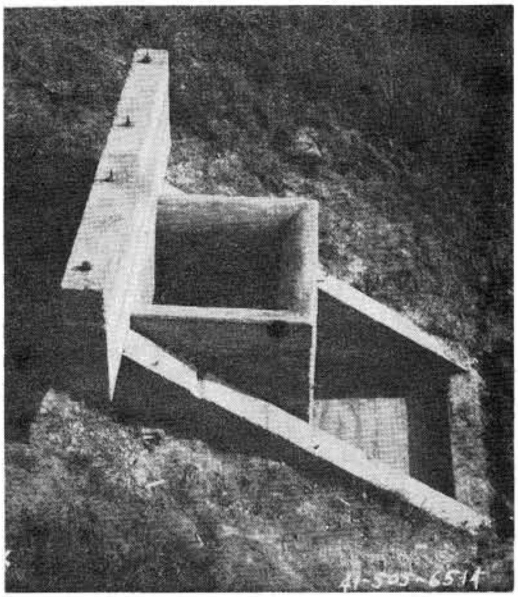


Fig. IX-7 - Drop Inlet Entrance, Experiment 5, Stillwater, Oklahoma.

The concrete pipe used for these experiments was culvert pipe in 4-ft lengths. Mortared joints were used--admittedly a poor practice where water-tightness is important--and some of the mortar dripped into the pipe, which added to its roughness. The texture of the pipe and the appearance of a joint are shown in Fig. IX-8. The paved invert, corrugated metal pipe had been stored outdoors and was about 4 years old. The bituminous coating had shrunk, cracked, and checked. Some of the cracks were wide enough to admit the fingers. This cracking undoubtedly contributed to an increase in the friction coefficient. The appearance of this coating is shown in Fig. IX-9. The 24-in. corrugated pipe was 262.5 ft long for the first four experiments. The last 40.4 ft of corrugated pipe were removed for Experiments 5 and 6 because this length of pipe was unpaved, had a different roughness than the paved pipe, and the experimental arrangement did not permit a determination of the different friction coefficients.

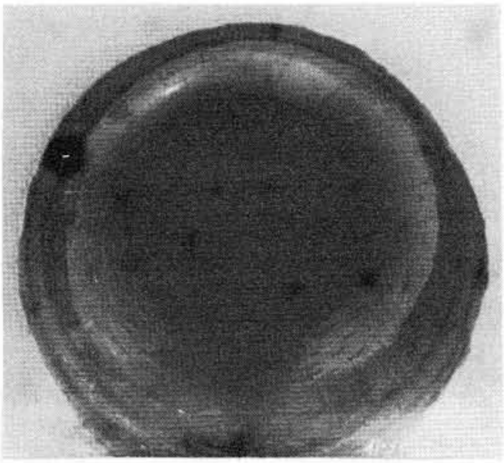


Fig. IX-8 - Texture of Concrete Culvert Pipe and Joint, Stillwater, Oklahoma.

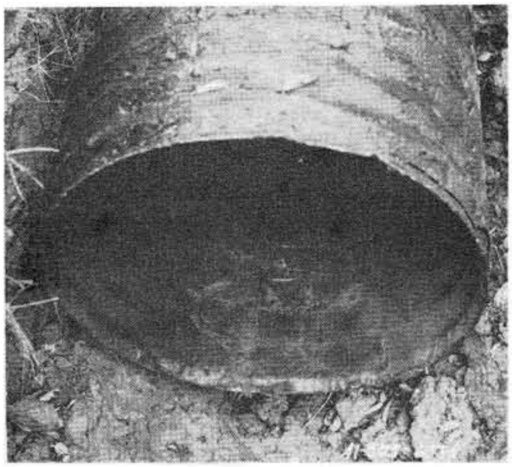


Fig. IX-9 - Cracking of Bituminous Coating of 24-in. Corrugated Metal Pipe, Stillwater, Oklahoma. n = 0.0218.

The elbow is described by Mr. Ree [I-42, p. 49]: "The [corrugated] elbow at the junction of the concrete and the corrugated pipes is a five-piece miter-cut elbow. Its construction is relatively rough with pipe edges projecting into the inside of the bend. The invert of the elbow is paved."

The discharge from the corrugated pipe was free and unaffected by downstream water levels. No outlet was used.

APPARATUS AND PROCEDURE

The closed conduit spillway was built on the grounds of the Stillwater Outdoor Hydraulic Laboratory. A forebay or reservoir was formed by excavating the sides of a hill and using the excavated material to build an earth embankment. The surface area of this reservoir was about 7,000 sq ft at Elevation 911 (the invert of the pipe at the inlet was at Elevation 910) and 30,000 sq ft at Elevation 924. Water for the experiments was obtained from Lake Carl Blackwell, carried through the dam in siphons, transported in canals to the site of the reservoir, and entered the reservoir through a modified 4-ft Parshall measuring flume. The arrangement is shown in Fig. IX-10.

Fig. IX-10 - Test Site at Stillwater, Oklahoma. Dam on Lake Carl Blackwell, Siphons, and Valve House in Background; Parshall Flume and Gate in Upper Center Discharging 20 cfs; Headwall for Experiment 5 in Foreground; Staff Gage and Stilling Well at Right.

Flow through the closed conduit spillway was measured in two different ways. A calibrated, modified Parshall flume was used to determine the rate of flow for part-full pipe flow and for weir flow discharges, and the headpool level was steady. It took so long to achieve steady pool levels when the spillway was flowing full that the full flow discharges were determined "on the run" for most tests. This was accomplished as follows: A flow greatly exceeding the capacity of the spillway was turned into the reservoir until the reservoir was full. The inflow was then completely stopped and the reservoir allowed to drain through the spillway. Water levels were measured and the time noted at frequent intervals. This permitted computations of the rate of change of reservoir level. Entering a curve giving the reservoir area at different elevations, the rate of change of stage was converted to rate of discharge.

Pool levels were recorded by an automatic water level recorder mounted on the stilling well shown in Fig. IX-10. However, the levels actually used in the analysis were determined by an electric "point" gage to 0.001 ft. This instrument consisted of a plumb bob attached to a steel tape, a vernier to permit accurate readings, and a glow light to indicate when the plumb bob touched the water surface.

Pressures within the pipe line were measured by piezometers located as shown in Fig. IX-1 and open water manometer columns. The manometers for the concrete pipe, one of the corrugated pipe manometers, and a headpool manometer were grouped on a single manometer board. Dye was added to the columns to make them more readily visible and the readings

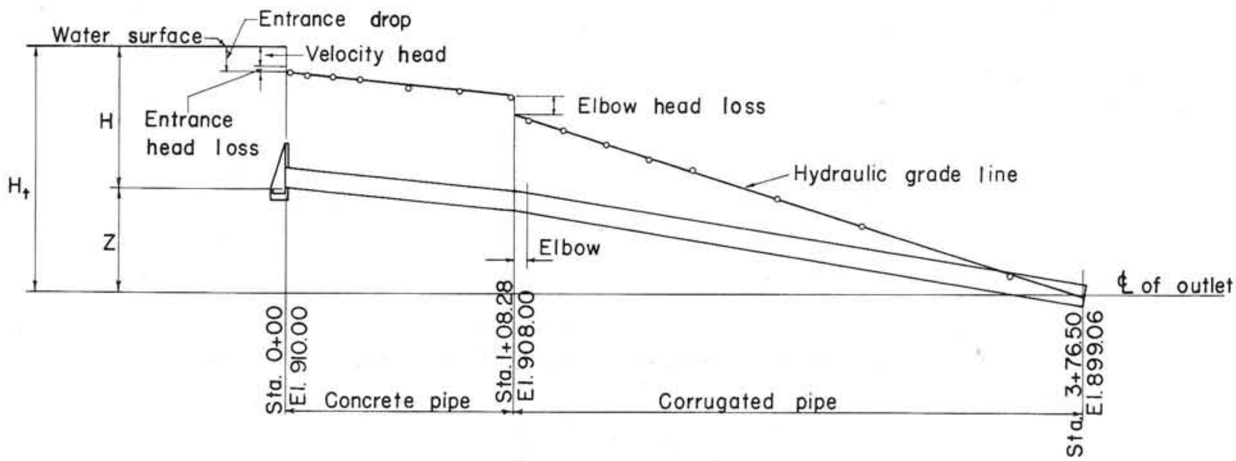


Fig. IX-11 - Typical Hydraulic Grade Line and Head Losses at Entrance and Elbow, Stillwater, Oklahoma.

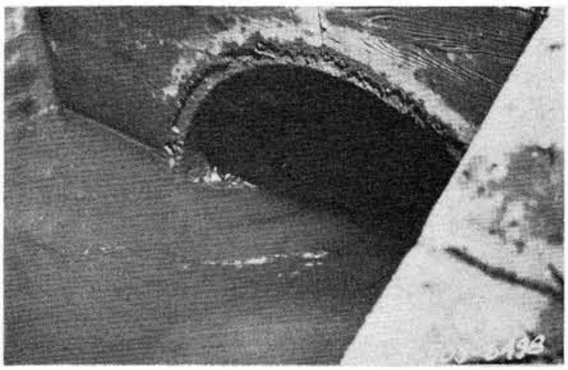


Fig. IX-12 - Flow Condition at Concrete Pipe Groove Entrance, Stillwater, Oklahoma.

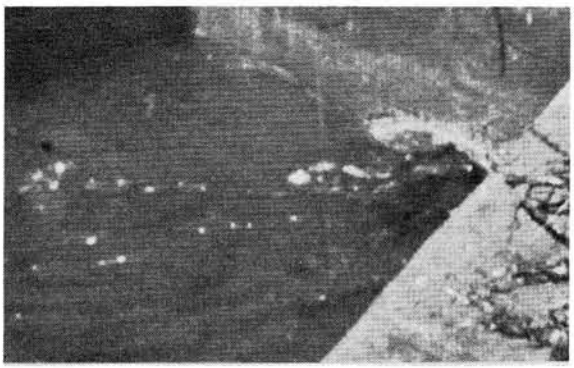


Fig. IX-13 - Flow Condition at Rounded Pipe Entrance, Stillwater, Oklahoma.

were recorded photographically. The manometers for the corrugated pipe were located on individual boards adjacent to the piezometer connection and were read manually.

Standard procedures were used in the conduct of the tests and in the analysis of the data. Hydraulic grade lines were plotted for each test. From them, the entrance loss, the elbow loss, and the friction loss in both concrete and corrugated metal pipe were determined. A typical hydraulic grade line plotted by Mr. Ree is shown in Fig. IX-11.

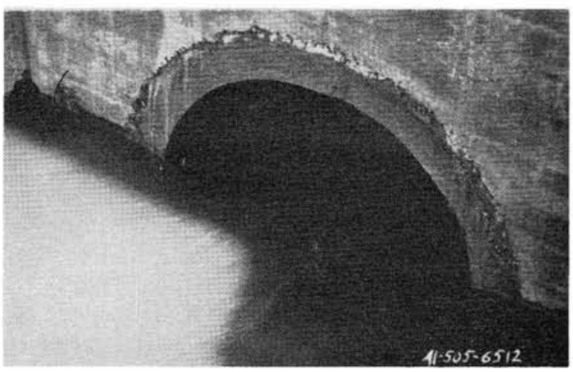
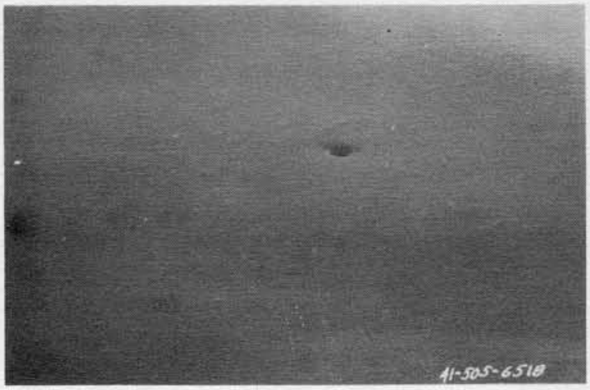


Fig. IX-14 - Flow Condition at Square-Edged Pipe Entrance, Stillwater, Oklahoma.

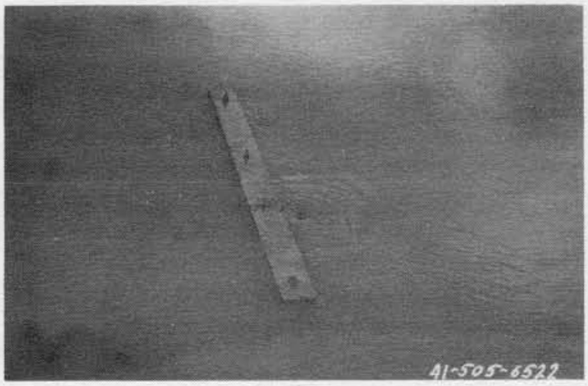
DESCRIPTION OF FLOW

Fig. IX-11 shows the hydraulic grade line to be above the pipe and the pressures within the pipe to be positive (greater than atmospheric except for a short distance near the outlet. This was the usual condition when the pipe was full. The minimum slope of the hydraulic grade line in the corrugated pipe, which occurred only for the lowest flows with the pipe full, was 0.0372. This compares with a pipe slope of 0.0334. Since zero pressure is taken at the centerline of the pipe at its exit, the top of the corrugated pipe was under negative pressure (less than atmospheric) for its entire length. For this particular test, the slope of the hydraulic grade line in the concrete pipe was 0.0111, which is less than the pipe slope of 0.0185. How-

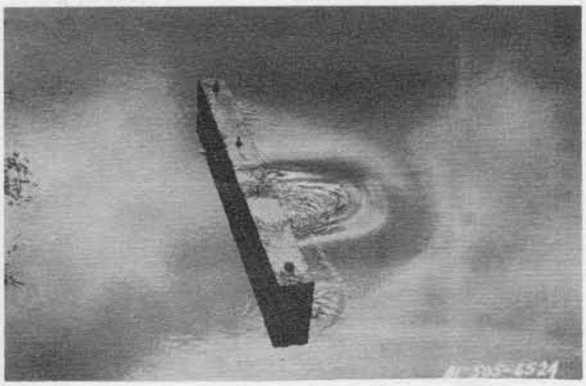


(a) Head on crest is 5.70 ft.
Pipe flow control.

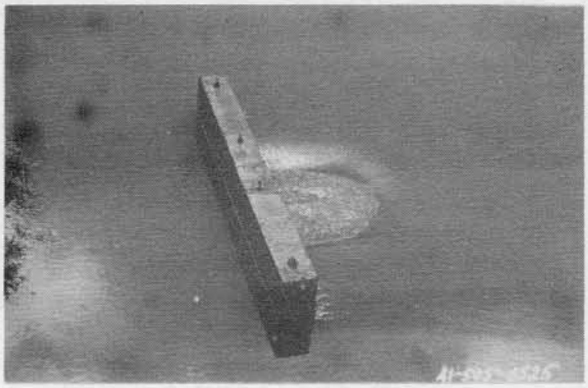
(b) Head on crest is 2.25 ft.
Pipe flow control.



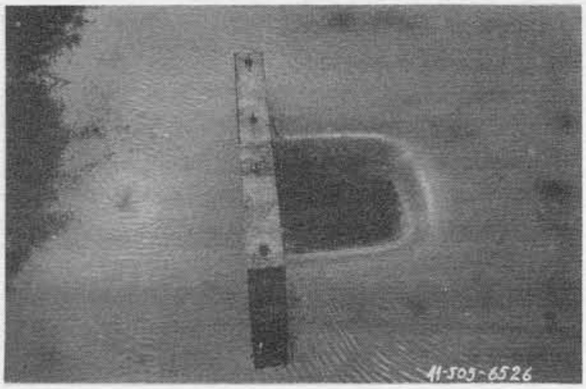
(c) Head on crest is 1.95 ft.
Pipe flow control.



(d) Head on crest is 1.25 ft. Pipe flow control or beginning of transition from pipe to weir flow control.

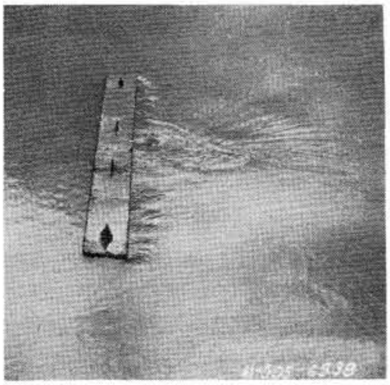


(e) Head on crest is 1.00 ft. Weir flow control or near end of transition from pipe to weir control.

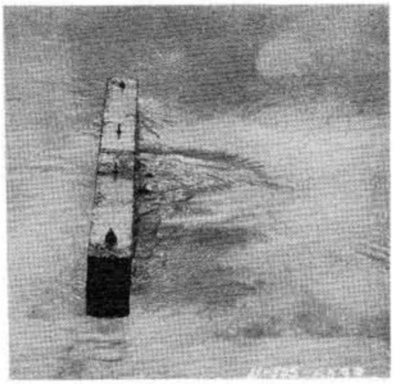


(f) Head on crest is 0.75 ft.
Weir flow control.

Fig. IX-15 - Flow at Entrance to Drop Inlet 4.3 Ft Deep, Stillwater, Oklahoma.

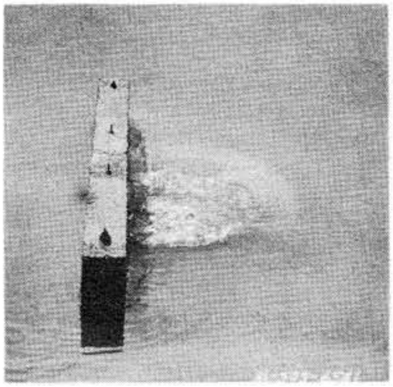


(a) Pipe flow control with very slight depression over drop inlet. Head over crest is 1.91 ft.

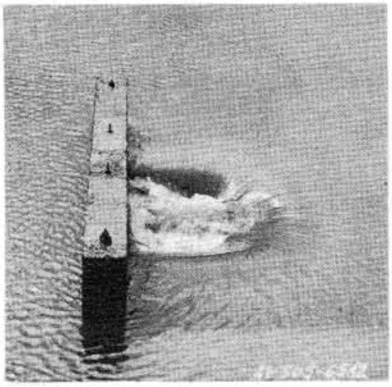


(b) Pipe flow control with small depression over drop inlet. Head over crest is 1.41 ft.

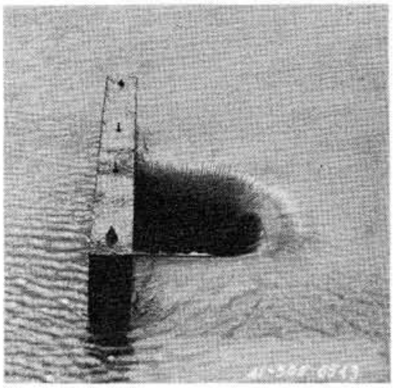
(c) End of pipe flow control or beginning of transition to weir flow control. Head over crest is 1.21 ft.



(d) Probably near end of transition from pipe flow control to weir flow control. Head over crest is 1.11 ft.



(e) Weir flow control with water almost filling drop inlet. Head over crest is 0.96 ft.



(f) Weir flow control. Water just shows in drop inlet. Head over crest is 0.91 ft. Discharge for (f) is only 0.1 cfs less than for (e).

Fig. IX-16 - Transition from Pipe to Weir Control at Entrance to Drop Inlet 8.4 ft Deep, Stillwater, Oklahoma.

ever, the elbow loss raised the hydraulic grade line so that negligible negative pressures (-0.08 ft) existed in the top of the concrete pipe for about 11 ft at the entrance. Because the pipe was under only slight negative pressures for this low flow and because the friction slope and the pressure increase with the flow, it is evident from this discussion that the pipe was under positive pressure for most of the full flow tests.

It was observed during the tests that the pipe filled as soon as sufficient flow was admitted to the pond; there was no evidence of orifice or short-tube flow at any time, the control passing directly and smoothly from weir control at low flows to pipe control as the flow increased. The transition from pipe control to weir control for decreasing flows was also direct and positive for all inlets.

Flow conditions at the entrance with the pipe flowing partly full are shown in Figs. IX-12, IX-13, IX-5, and IX-14. Mr. Ree shows that the inside diameter governs the head-discharge relationship, as will be shown later. Close inspection of Fig. IX-12 shows a contraction at the entrance of the groove and an additional contraction at the base of the groove, which confirms Mr. Ree's observation in this particular instance.

Flow conditions at the drop inlet 4.3 ft deep are shown in Fig. IX-15 as a series of pictures taken as the pond level decreased. A similar series for the 8.4 ft deep drop inlet is shown in Fig. IX-16 for decreasing heads in the vicinity of the change from pipe flow control to weir flow control. Through comparisons with this series of pictures, one can determine the probable type of flow in other drop inlets by visual observation.

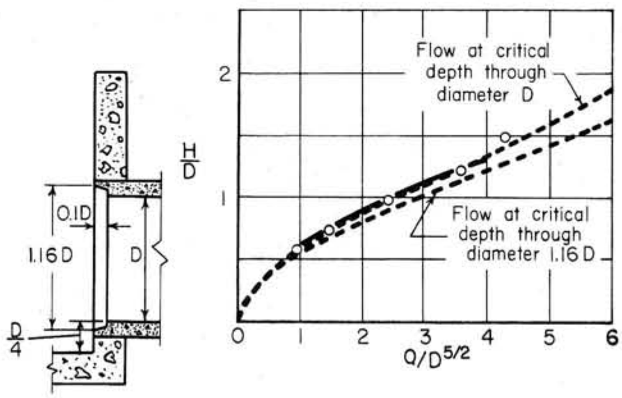


Fig. IX-17 - Head-Discharge Curve for Part Full Flow with Concrete Pipe Groove Entrance, Experiment 1, Stillwater, Oklahoma.

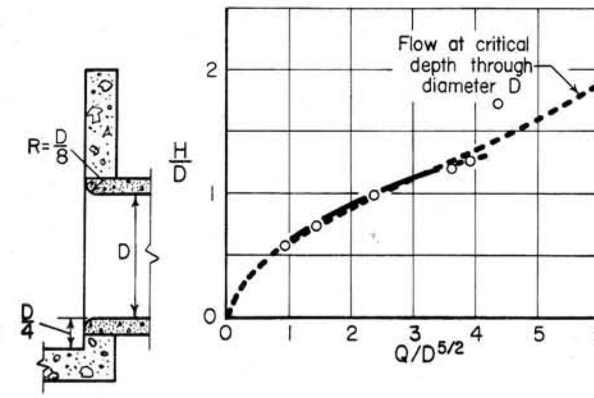


Fig. IX-18 - Head-Discharge Curve for Part Full Flow with Rounded Pipe Entrance, Experiments 2 and 3 (with Trash Rack), Stillwater, Oklahoma.

Vortices are apparent in Figs. IX-15 and IX-16. Vortices were also observed for each of the other inlets tested. At times air was sucked into the spillway with a noise that was audible for a considerable distance away even though the depth of the water over the crown of the pipe was as much as 11 ft. The vortices were not sufficiently strong to cause a large reduction in the spillway capacity. However, Mr. Ree ingeniously shows that the vortices do have some effect on the spillway capacity and that this effect increases as the observed vortex intensity increases.

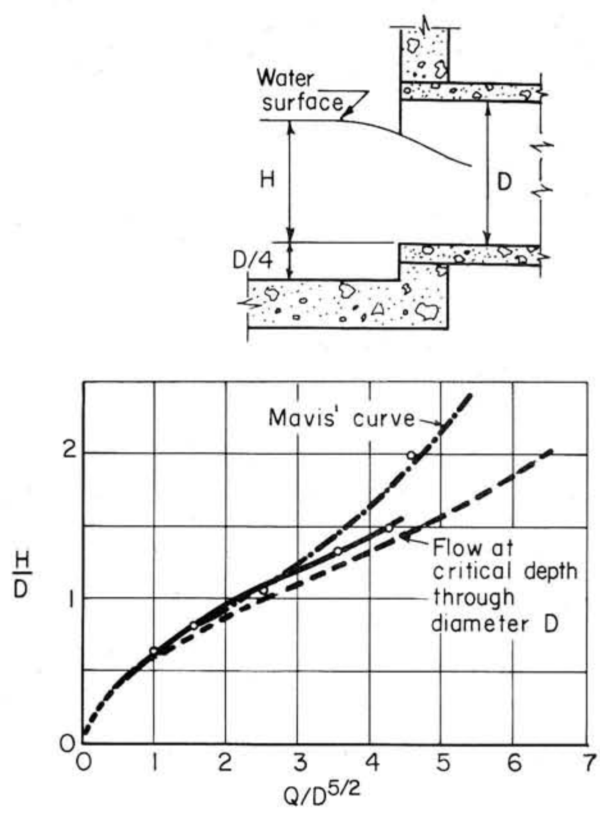


Fig. IX-19 - Head-Discharge Curve for Part Full Flow with Square-Edged Pipe Entrance, Experiment 4, Stillwater, Oklahoma.

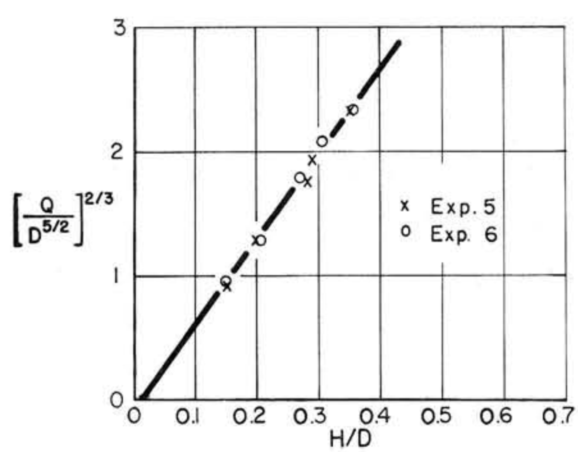


Fig. IX-20 - Head-Coefficient Curve for Weir Flow over Drop Inlet Crests, Experiments 5 and 6, Stillwater, Oklahoma.

DISCHARGE COEFFICIENTS

The control passed smoothly and directly from weir control to pipe control so only these controls will be discussed.

Weir Coefficient

As mentioned in Part I, the theoretical equation for part full flow at the entrance to a pipe is so complicated that the presentation of coefficients becomes impractical from the standpoint of application. Therefore, the results of the tests are presented in Figs. IX-17, IX-18, and IX-19 in the form of semi-dimensionless curves copied from Ree's report for the concrete pipe groove entrance, the rounded pipe entrance, and the square-edged pipe entrance respectively. A caution should be observed in using these curves: **The curves should not be extrapolated beyond the limits shown** because the head increases rapidly with further increase in discharge. Also shown in these figures are head-discharge curves computed by Ree on the assumption that critical depth occurs at the entrance. These curves are based on the inside diameter of the pipe, and show that the pipe diameter rather than the maximum entrance diameter governs the head-discharge relationship. The curves which represent the data may be used to predict the discharge of pipes having similar entrances if the dimensions of the entrance in terms of the pipe diameter and H/D are multiplied by the pipe diameter measured in feet and $H/D^{5/2}$ is multiplied by the five-halves power of the pipe diameter.

TABLE IX-1

ENTRANCE LOSS COEFFICIENTS

Stillwater, Oklahoma, Experiments

Inlet	K_e Maximum	K_e Minimum	K_e Average
Concrete pipe groove	0.41	0.23	0.33
Rounded	0.34	0.18	0.27
Rounded, with trash rack	0.25	0.21	0.23
Square edged	0.74	0.64	0.70
Drop, 4.3 ft deep	0.72	0.57	0.64
Drop, 8.4 ft deep	0.60	0.41	0.52

The drop inlet crest has the form shown in Fig. I-3b so the crest length was taken as $L = 2B' + W' = 9.50$ ft. Coefficients for use in Eq. I-1 may be computed from Fig. IX-20. The curve drawn in Fig. IX-20 is based on the equation presented by Ree

$$\frac{Q}{D^{5/2}} = 3.87 \frac{L}{D}\left[\frac{H}{D} - 0.016\right]^{3/2} \qquad \text{(IX-1)}$$

TABLE IX-2

RECOMMENDED VALUES OF THE LOCAL PRESSURE DEVIATION AT THE TOP OF THE PIPE 0.5 D DOWNSTREAM FROM THE ENTRANCE

Stillwater, Oklahoma, Experiments

Entrance	h_n/h_{vp}
Concrete pipe groove	0.0
Rounded	0.0
Rounded, with trash rack	0.0
Square edged	-1.0
Drop inlet 4.3 ft deep	-0.8
Drop inlet 8.4 ft deep	-0.8

Entrance Loss Coefficient

Entrance loss coefficients were computed by Ree for all tests in which the pipe flowed completely full. Since Ree determined the hydraulic grade line for each test, no assumptions of the friction factor or the position of the hydraulic grade line at the conduit exit are necessary. It may be assumed therefore that Mr. Ree's values of K_e are accurate. The maximum, minimum, and average observed values of K_e are given in Table IX-1. It is recommended that the average value of K_e be used in Eqs. I-4 and I-5 for purposes of design.

The lower average value of K_e obtained with the trash rack in place compared with the trash rack absent is shown to be plausible by Mr. Ree whose explanation, based on Dr. Keulegan's analysis [I-30], shows that the turbulence created by the trash rack can result in a smaller entrance loss. The turbulence caused by the trash rack can be seen in Fig. IX-5.

PRESSURE COEFFICIENTS

Average values of the local pressure deviations h_n/h_{vp} computed for full flow in a hypothetical, horizontal, frictionless pipe are given in Table IX-2. These values are for use in Eq. I-14.

Values of h_n/h_{vp} along the conduit that are unaffected by local disturbances should be zero. The fact that they average above zero is quite likely because the datum plane was assumed to pass through the center of the conduit exit whereas it should have been taken above the center of the exit, as can be seen by referring to Ree's data plotted in Fig. X-32.* It is interesting to note that the elbow--a local disturbance--did not cause significant variations in the measured values of h_n/h_{vp}.

For purposes of design, it is suggested that the value of h_n/h_{vp} be taken as zero at all locations along the conduit except for the top of the pipe 0.5D downstream from its entrance. Suggested values of h_n/h_{vp} for this location are given in Table IX-2.

CONCLUSIONS AND RECOMMENDATIONS

Closed conduit spillways similar to that shown in Fig. IX-1 having entrances geometrically similar to those shown in Fig. IX-2 are recommended for use. The control changed smoothly from weir control to pipe control as the flow increased and from pipe control to weir control as the flow decreased. No undesirable flow characteristics were observed. However, results of tests by others indicate some of these entrances may exhibit orifice control under certain conditions, and it is suggested that the use of the square-edged entrance, at least, be avoided.

Curves for determining the part full flow capacity of the concrete pipe groove, rounded, and square-edged entrances are given in Figs. IX-17, IX-18, and IX-19 respectively. The capacity of the drop inlet acting as a weir is given by Eq. IX-1. Values of the full flow entrance loss coefficients for use in Eqs. I-4 and I-5 are given in Table IX-1 for each entrance tested.

Values of the local pressure deviation h_n/h_{vp} for use in Eq. I-14 should be taken as zero except near the entrance to the pipe. At that location, values of h_n/h_{vp} may be taken from Table IX-2.

2-FOOT SQUARE DROP INLET CLOSED CONDUIT SPILLWAY NEAR NELSON, WISCONSIN

> In 1936 CCC Camp SCS-WIS-15 completed plans to construct a twin 6 x 6 drop inlet soil-saving dam on the Martha Johnson farm near Nelson, Wisconsin. Because of peculiarly favorable conditions it appeared feasible to construct a 2 x 2 drop inlet under the same earth dam with the twix [sic] 6 x 6 to be used to conduct a full scale field model test.
>
> The twin 6 x 6, the 2 x 2 model, and earth fill were completed during the summer of 1936 and on April 8, 1937, after the pond had filled the first test run was made....

This quotation is from the anonymous mimeographed report which is reproduced in the section "Original Reports." An unsuccessful attempt was made to locate the original data. Mr. R. E. Reinke, Soil Conservationist, Soil Conservation Service, was one of those contacted. He wrote on March 2, 1953: "...I have learned that the old CCC Camp records have been destroyed. From what I can gather, the files destroyed included...even technical data. With reference to technical data, I believe we can sincerely regret that this type of file was destroyed...." Mr. Edwin Freyburger, Regional Engineer for the Soil Conservation Service, furnished on October 23, 1953, the Regional Office engineering file which contained the mimeographed report referred to above, the original computations for that report, a typewritten

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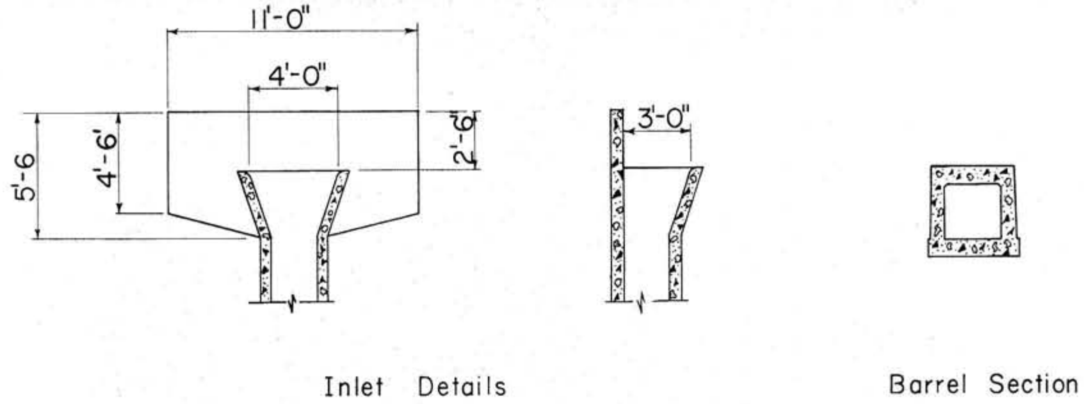
*Fred W. Blaisdell and Charles A. Donnelly, Hydraulics of Closed Conduit Spillways--Part X. The Hood Inlet, St. Anthony Falls Hydraulic Laboratory Technical Paper No. 20-B, April 1958.

description of the spillway performance by an unknown writer, a copy of a letter from E. F. Gahnz to Professor L. H. Kessler giving measured elevations and dimensions, as well as other correspondence and plans. On March 8, 1955, Mr. H. F. Smith, Soil Conservation Service Area Conservationist at Eau Claire, Wisconsin, who apparently was responsible for installing the spillway, sent his file containing blueprints of the dam site, the drainage area, gully cross sections and longitudinal sections, the 6 by 6 twin drop inlet spillway plans, plotted cross sections recording areas at 1-ft vertical intervals, and a tracing of the experimental installation. Moving pictures of the tests were obtained from Mr. Reinke and from Mr. Neal E. Minshall, Project Supervisor for the Agricultural Research Service at Madison, Wisconsin. It is regretted that the original data could not be obtained. However, because nothing better was available, the data used in the preparation of the mimeographed report and data points taken from Professor E. R. Dodge's Fig. 2 [I-16] were employed in this analysis.



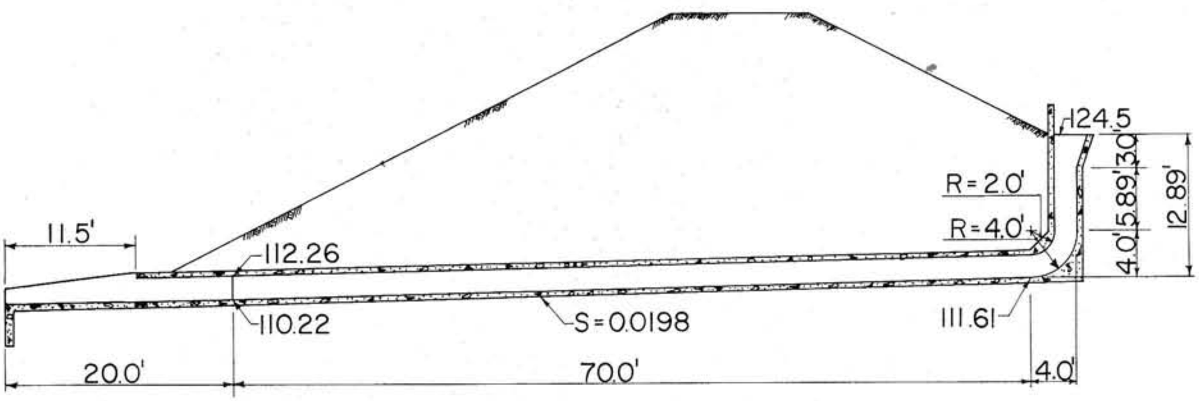


Fig. IX-21 - Martha Johnson Closed Conduit Spillway.

DESCRIPTION OF SPILLWAY

The closed conduit spillway is composed of a flared inlet, a riser, an elbow, a barrel, and a flared outlet. Fig. IX-21 has been prepared from the tracing furnished by Mr. Smith and Professor Dodge's Fig. 1 [I-16]. Fig. IX-22 shows the spillway on May 23, 1944.

The crest of the drop inlet is rounded, as in Fig. I-3b, on about a 4-in. radius. This radius was scaled from the original tracing. The crest dimensions are therefore taken to be 3 ft 4 in. by 4 ft 8 in. giving a crest length of 11 ft 4 in. The elevation of the crest is 124.5.

(a) Experimental 2 ft by 2 ft spillway has been capped and only headwall shows beyond twin 6 ft by 6 ft spillway.

(b) Outlet has silted in.

Fig. IX-22 - Martha Johnson Closed Conduit Spillways in May 1944.

The flare of the inlet was at a rate of 1 ft horizontally in 3 ft vertically, the height of the flared section being 3 ft 0 in.

The computed height of the 2 ft square riser is 5.89 ft.

The elbow has an inner radius of 2 ft 0 in. and an outer radius of 4 ft 0 in.

The barrel is 70 ft 0 in. long and is considered to terminate at the beginning of the flared outlet section. The barrel invert elevation at the end of the elbow is 111.61 and at the beginning of the flare is 110.22 according to Gahnz. The drop of 1.39 ft results in a barrel slope of 0.0198, which checks the figure given by Gahnz. The barrel crown elevation at the beginning of the flare is 112.26 resulting in a mean centerline elevation of 111.24. The drop Z through the spillway is 13.26 ft. The cross sectional area at the beginning of the flare is 4.11 sq ft, the minimum cross sectional area is 4.00 sq ft, and the average area is 4.054 sq ft. The cross section is not a square, as can be seen in Fig. IX-21. The fillets at the top shown on the original tracing were scaled to be 2 in. on a side.

The flared outlet section is 20 ft long.

APPARATUS AND PROCEDURE

The crest of the 2 by 2 spillway was 3.5 ft lower than the crest of the twin 6 by 6 spillway so "a heavy metal cover, sealed with felt and loaded down with sandbags was set over the 2 x 2 inlet...." By April 8, 1937, the pond had filled so the cover was removed and the spillway was tested on that date. Instructions for preparations prior to the test were transmitted to Mr. Smith by E. J. Peterson, Supervising Engineer, on September 11, 1936, as follows:

1. After you have completed computing the pond area from your cross sections I would like to pick it up and have another complete and independent computation made by one of the engineers at Independence with the idea of getting a check. All we will need is the total pond volume and the pond volume at each foot interval between elevation 128.00 and 124.5.

2. All obstruction should be cleared from the inside of the 2 x 2 barrel. By that I mean that all small chunks of concrete that might have slopped over in pouring should be removed, all protruding wires should be cut off, etc.

3. The area of the barrel should be determined at 5-foot intervals along the barrel by careful measurement. These 5-foot intervals should extend from the beginning of the flare to the beginning of the elbow. The riser should also be measured at elevations 122.5, 120.0, 117.5, and 115.08. Measure the profile of the elbow section. Cross sections of the flared section at the discharge end can also be taken at the 5-foot interval.

4. When they had completed the floor section, I asked Gahnz to take a profile of the slab so he should have that. Recheck all elevations of the 2 x 2. Take readings around the lip of the 2 x 2 at several points to see that the whale [sic] inlet is level. Recheck all piezometer tube distances.

Heads were apparently measured by a float gage located in a stilling well and read against a brass scale locally graduated to 0.005 ft. The location of the gage can be seen in Fig. IX-23.

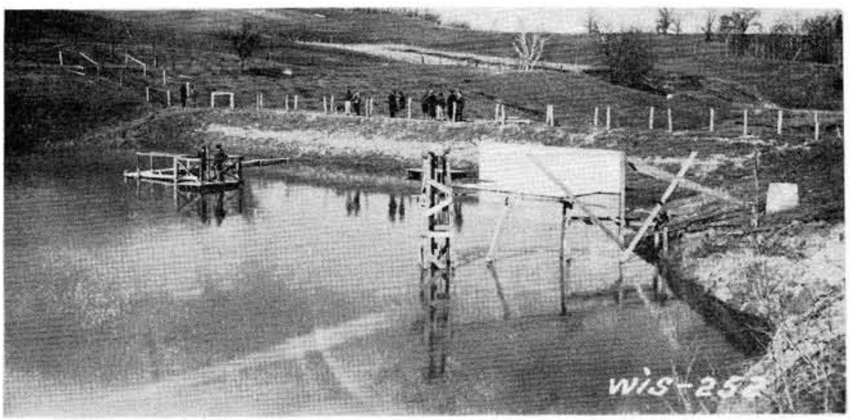


Fig. IX-23 - Martha Johnson Pond on April 8, 1937.

Dodge [I-16] says:

The test was started by pulling the cover from the inlet with a truck and block-and-tackle. Pond elevation was determined at 1-min intervals with a float gage reading to 0.01 ft. Average head and discharge over successive 2-min intervals were computed and plotted in Fig. 2 as circles. Flow calculations were based upon pond storage as previously determined by two independent methods: (1) planimetering a topographic map, and (2) allowing the pond to discharge through two 8-in. sharp-edged orifices while the pond elevation was read at measured time intervals.

The original "pond volume calibration chart" was made a part of the mimeographed report and is reproduced in the section "Original Reports." Notes on a photograph indicate that the pond volume was based on cross sections taken at 10-ft intervals.

DESCRIPTION OF FLOW

The most apt description is given by one who has actually observed the phenomena. The unknown author of the typewritten report has written:

A...cover...was removed by means of a cable system hooked to a tractor. A sudden, slight depression over the inlet and whoosh,--a solid stream of water shot out at the discharge. It was the first time any of us had ever seen a drop inlet actually flowing full.

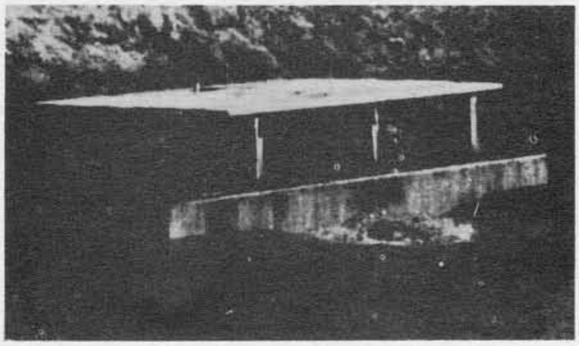
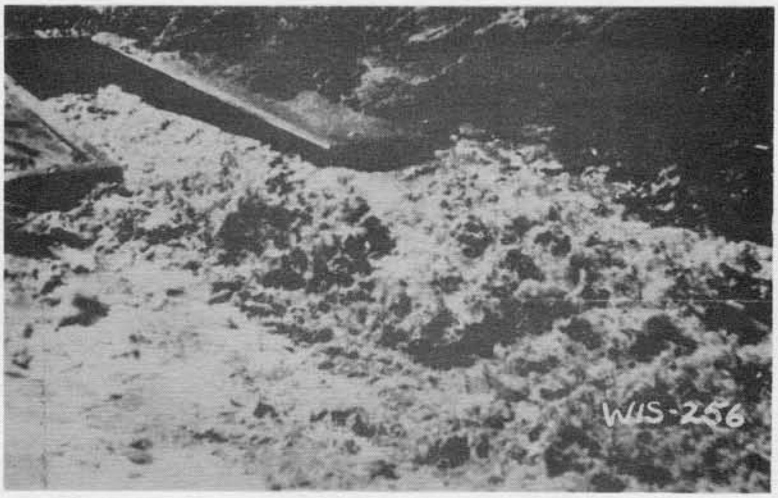
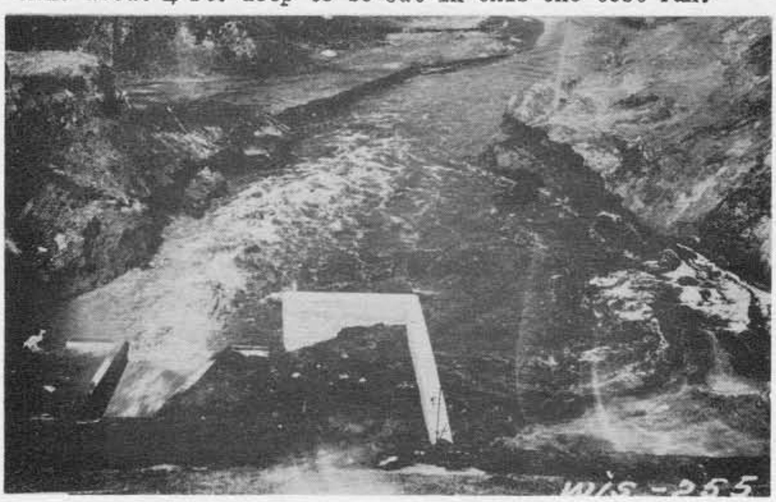


Fig. IX-24 - Martha Johnson 2-Ft Square Drop Inlet Spillway on April 8, 1937. Head Over Crest is About 2 Ft. Discharge About 100 cfs.



(a) Note on picture reads, "This view shows outlet condition with structure flowing full or approximate discharge of 100 c.f.s. Note the turbulence beyond the end of apron which caused a hole about 4 ft. deep to be cut in this one test run."



(b) Note on picture reads, "This view shows outlet condition when the structure is carrying about 10 cu. ft. per second. Note that the hydraulic jump is forming at about the end of the concrete apron."

Fig. IX-25 - Martha Johnson 2-Ft Square Drop Inlet Spillway on April 8, 1937.

The pond over the inlet appeared to be undisturbed until the critical head was reached. At this point a small drawdown appeared with some turbulence. Later, more or less gentle surging took place until the vacuum broke. At this point, of course, we are below the design head and weir flow takes place.

At the maximum discharge it appeared that the barrel did not flow full, since the surface of the discharging water flowed about 6 inches below the top of the end of the barrel. From this point the water followed, very closely, the slope of the apron walls and about 6 inches from the top of the walls.

A review of Mr. Minshall's moving pictures shows this to be a good description of what took place. The rapid scouring of the hole and the side whirls are especially noticeable. Mr. Minshall's movies correspond to Figs. IX-24 and IX-25. Mr. Reinke's movies correspond to Fig. IX-22 which shows the piles along the side of the channel, but otherwise the two movies show the same phenomena. This is an indication that two tests were made on the experimental 2 by 2 closed conduit spillway, but no mention of, or data on, the second test has been discovered. Mr. Reinke's movie shows an observer taking measurements of the piezometric pressure, indicating that data was taken during the second test.

DISCHARGE COEFFICIENTS

The head-discharge data used in the preparation of the mimeographed report and the data read from Dodge's Fig. 2 [I-16] were used to determine the coefficients for the weir and the entrance. The mimeographed report data were also recomputed by the author using the pond volume calibration determined by cross sections. (The original computations were apparently made using the average of the volumes determined by the orifice and by cross sections.) The author's incremental volumes were also smoothed to give a uniform rate of pond

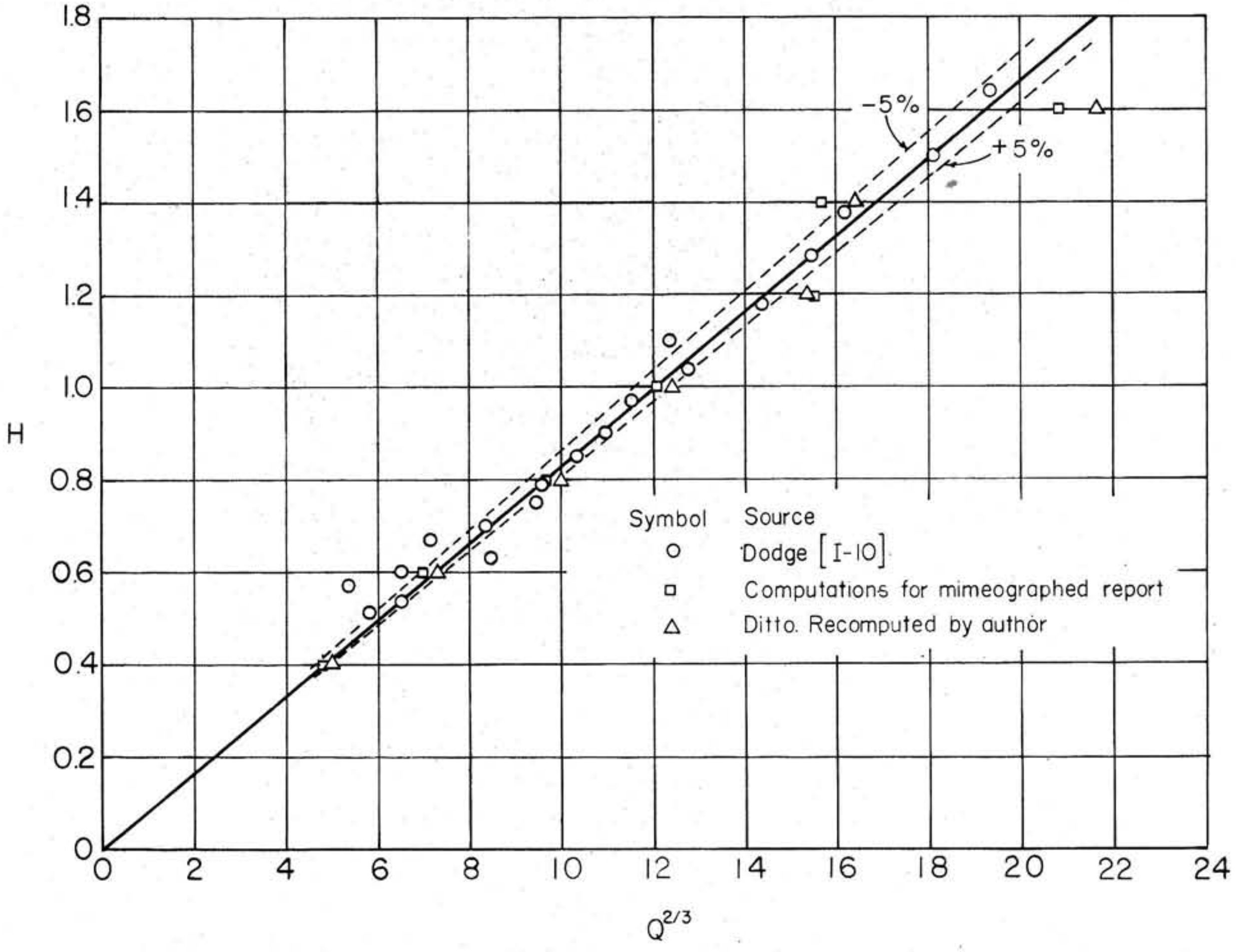


Fig. IX-26 - Weir Flow Head-Discharge Curve for Martha Johnson Closed Conduit Spillway.

volume change with change in elevation. This resulted in greater discharges than were computed originally, the increase being particularly noticeable for full pipe flow.

Weir Coefficient

The weir flow data are plotted in Fig. IX-26. The curve has the equation

$$Q = 3.67\,L\,H^{3/2} \tag{IX-2}$$

giving a coefficient of 3.67 in Eq. I-1. Curves have been drawn 5 per cent above and below the head-discharge curve to indicate the precision of the data. Most of the data as originally computed fall within the 5 per cent limits. However, if the reader places more reliance in the pond volumes as determined by cross sectioning, the curve is about 5 per cent low. For these conditions a better coefficient is 3.85. Ree [I-42] gives $C = 0.486\sqrt{2g} = 3.87$ for his drop inlets.

Entrance Loss Coefficient

It was necessary to make assumptions in order to compute the entrance loss coefficient because the pressure data, which would have permitted a determination of the hydraulic grade line and the frictional resistance, could not be located.

The data were plotted as in Fig. IX-27. The equation of the curve is

$$Q = A\sqrt{\frac{2g\,H_t}{1.75}} \tag{IX-3}$$

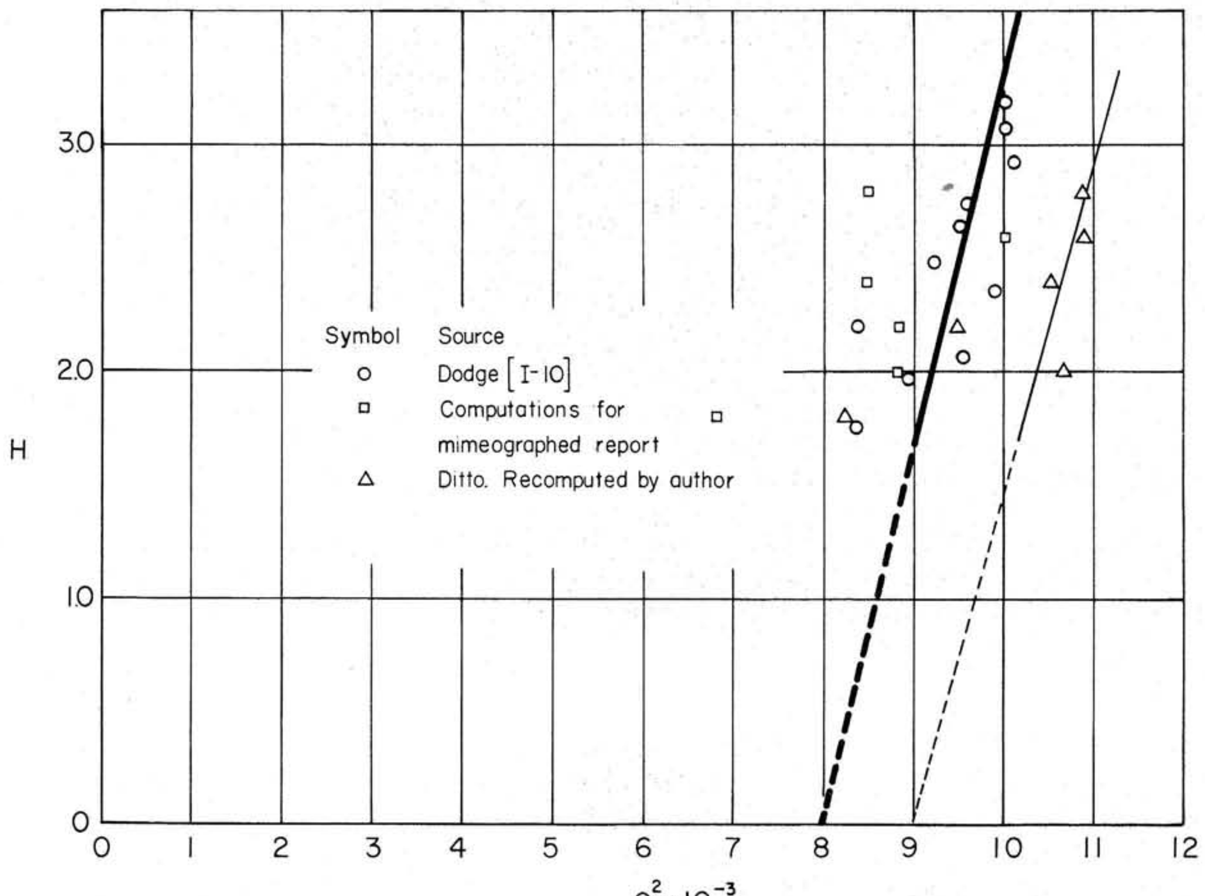


Fig. IX-27 - Pipe Flow Head-Discharge Curve for Martha Johnson Closed Conduit Spillway.

By comparison with Eq. I-5, the denominator under the radical is seen to be made up of the entrance and elbow, exit, and friction losses or

$$K_e + K_o + f\frac{\ell}{D} = 1.75 \tag{IX-4}$$

K_o is usually assumed to be 1.0 and K_e must be positive. If f is greater than 0.0187 (n greater than 0.0114), K_e becomes negative. Ree [I-42] has found that n = 0.0117 for 2-ft diameter concrete pipe so the 0.0114 value obtained when K_e = 0 appears to be on the right order of magnitude. If n is less than 0.0114, then K_e becomes greater than zero. For example, if n = 0.0105, f = 0.0162 and K_e = 0.10. The condition of the concrete at the time of the test is unknown. However, an entrance loss coefficient of 0.10 is not unreasonable, and its use is recommended.

If the data recomputed by the author is used, n = 0.0098 if K_e = 0. For K_e to be 0.10, n must be 0.0089.

PRESSURE COEFFICIENTS

No data from which pressure coefficients might be computed is available.

CONCLUSIONS AND RECOMMENDATIONS

Drop inlets similar in form to that shown in Fig. IX-21 have only the desirable weir and pipe controls. They are recommended for field use.

The weir coefficient for use in Eq. I-1 is 3.67 or, if the user has more confidence in the pond volume determination by cross sections, the coefficient may be increased to 3.85.

An entrance loss coefficient of 0.10 is suggested for use in Eq. I-5.

No data was available with which to compute the local pressure constants.

ORIGINAL REPORTS

Two of the unpublished reports covering the Martha Johnson closed conduit spillway tests are reproduced here to place them on record. The authors of both reports are unknown.

Mimeographed Report

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Project Office - Independence, Wis.

Drop Inlet Culvert Field Model Located at Nelson, Wisconsin

In June 1933, erosion control work was made a part of the Emergency Conservation Work. During the preceding five years, Professor O. R. Zeasman (Soils and Agricultural Engineering, College of Agriculture) had carried on considerable extension work, constructing gully control structures using large sewer pipe and corrugated culvert pipe for drop inlet culverts through earth fill dams. When Civilian Conservation Corps Camps were assigned to erosion control work it appeared that reinforced concrete would be the most practicable and satisfactory material to use for gully control work since plenty of labor was thus made available but comparatively little money was provided for materials.

Little or no research data was available covering the types of structures contemplated and during the early weeks of the ECW activities Prof. L. H. Kessler (Hydraulics and Sanitary Engineering, College of Engineering, University of Wisconsin) assisted by eight engineers and designers hurriedly completed hydraulic tests on small scale models and prepared tentative designs.

During the winter of 1933-34, a group of engineers, under the direction of Prof. Kessler and Mr. Neal E. Minshall, carried on a more detailed extension of the studies begun by Prof. Kessler. The results of these tests are covered in Prof. Kessler's Bulletin of the University of Wisconsin, Engineering Experiment Station Series No. 80, entitled "Experimental Investigation of Drop Inlets and Spillways for Erosion Control Structures". Standard designs of reinforced concrete drop inlet culverts were also prepared by this group of engineers largely guided by the field experiences of the summer of 1933. These designs have since been revised and included in a handbook compiled by the Technical Unit of the Engineering Section of the Soil Conservation Service, United States Department of Agriculture, entitled "Design and Construction of the Drop Inlet Soil-Saving Dam", (SCS-EP-14, June 1937).

In 1936 CCC Camp SCS-Wis-15 completed plans to construct a twin 6 x 6 drop inlet soil-saving dam on the Martha Johnson farm near Nelson, Wisconsin. Because of peculiarly favorable conditions it appeared feasible to construct a 2 x 2 drop inlet under the same earth dam with the twix [sic] 6 x 6 to be used to conduct a full scale field model test.

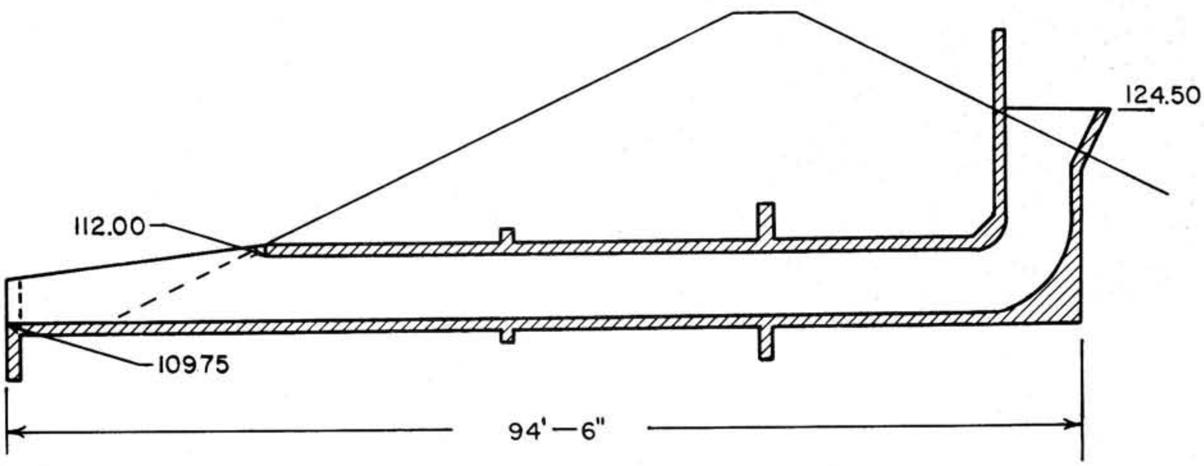
The twin 6 x 6, the 2 x 2 model, and earth fill were completed during the summer of 1936 and on April 8, 1937, after the pond had filled the first test run was made. This test showed that the field model carried almost exactly the quantity of water computed from the small scale laboratory tests.

The discharge through the full scale field model was measures [sic] by computing the pond volumes at various elevations from a detail topographic map and then timing the drop in elevation of the pond during the test run. The pond volume was checked later by measuring the discharge through two 8 inch orifices placed in the riser of the twin 6 x 6.

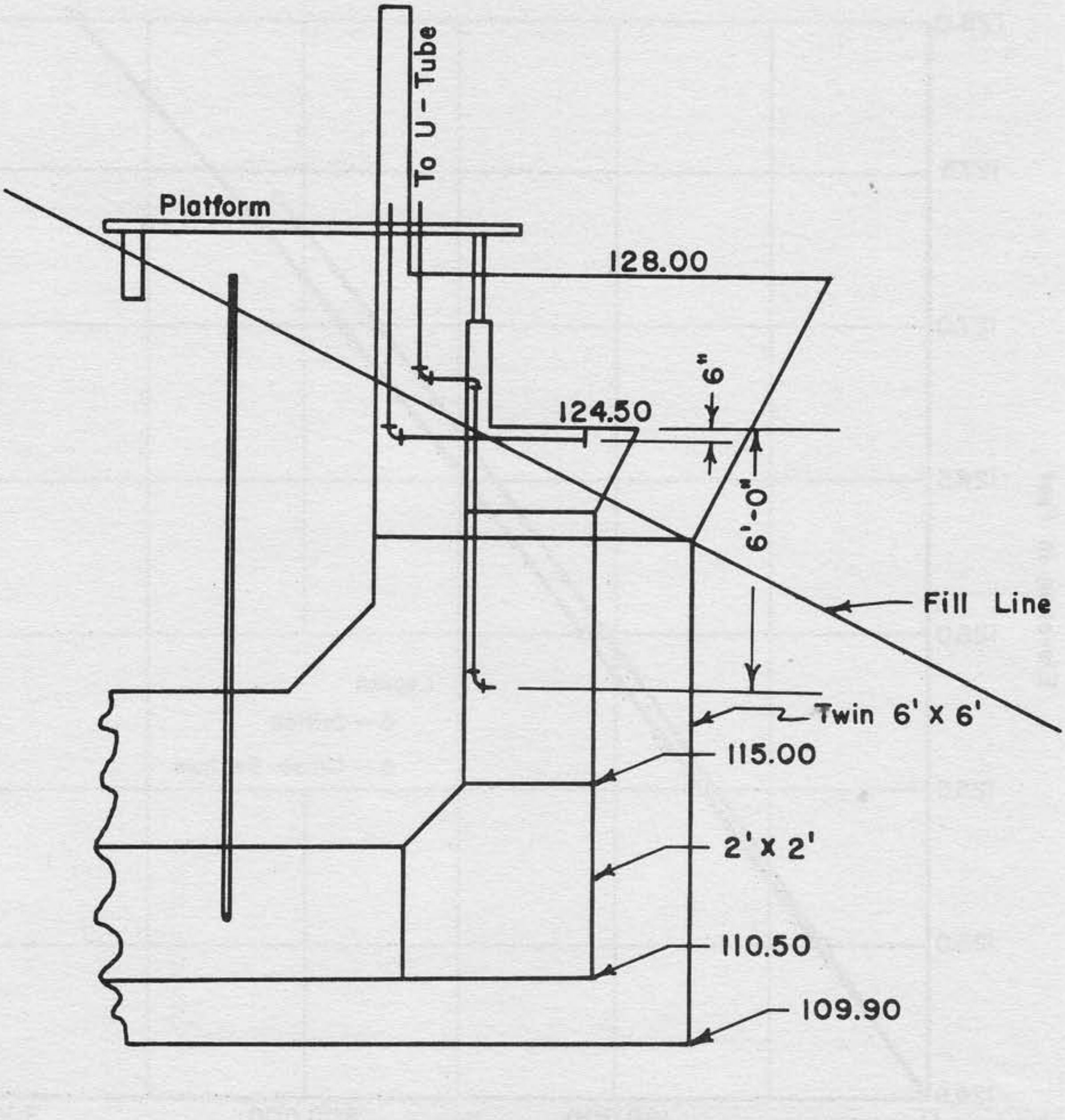
The attached "Pond Volume Calibration Chart" indicates the pond volumes at various elevations as computed from the topog map and the orifice test. The "Diagrammatic Sketch" shows the standard layouts for all drop inlet culverts and gives the elevations of inlet and discharge of the field model. The "Sketch Showing Relative Positions of Twin 6 x 6 and 2 x 2 drop Inlets" also shows the position of piezometer connections. The two orifices are located in the front face of the twix [sic] 6 x 6 riser.

The "Discharge Curve" attached hereto was plotted from data obtained in the first test run made on April 8, 1937 and indicates the discharge characteristics of this type of structure. Weir flow appears to hold up to a head over lip of from 1.25 to 1.5 feet for this particular total head. Beyond a head of 1.75 feet the structure appears to flow as a draft tube and very little capacity is gained by increasing the head beyond this point. This is what is termed the "design head".

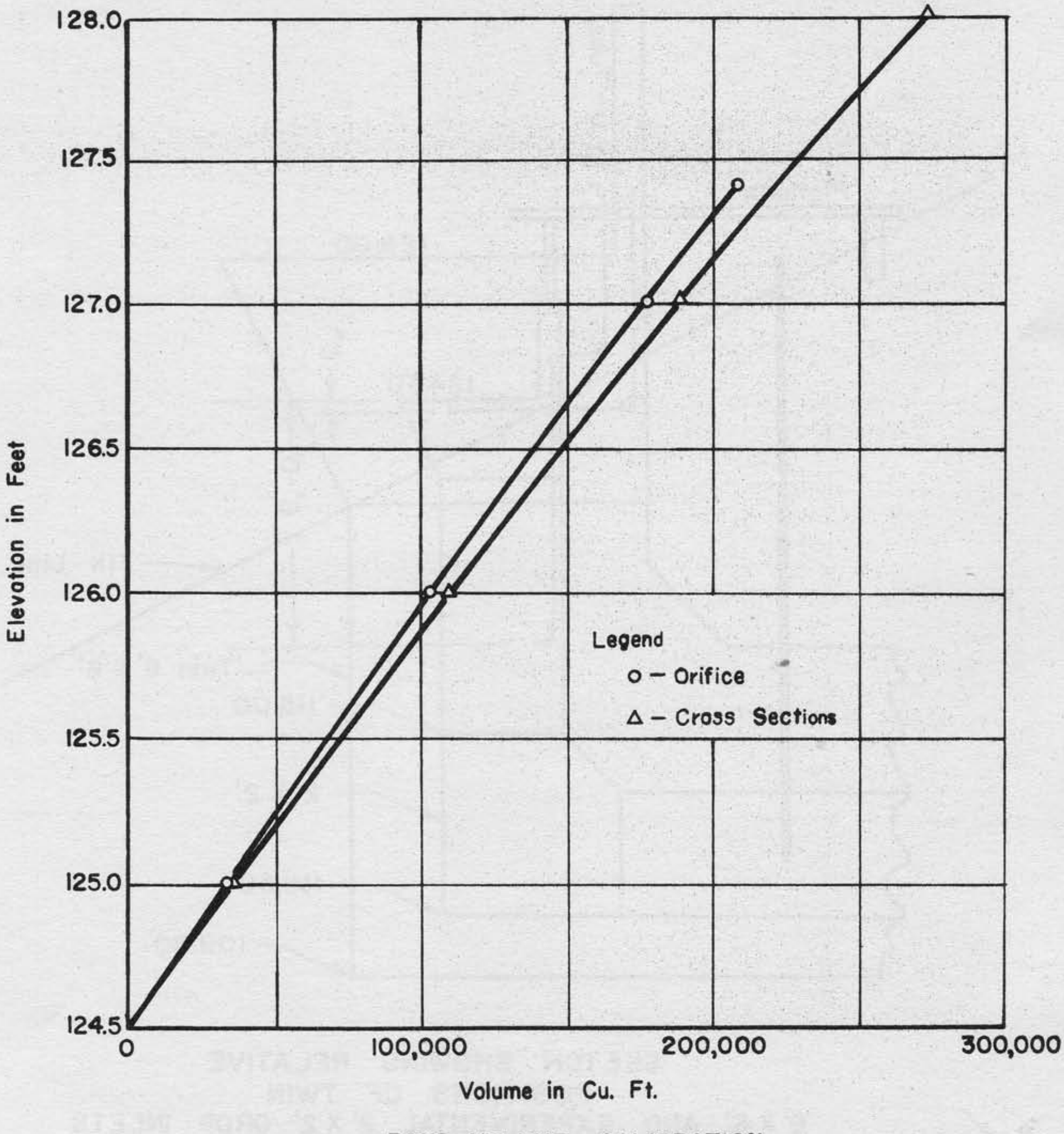
This field model test had served as a check on the small scale laboratory tests and has caused engineers in this Service to have full confidence in the designs now in use.



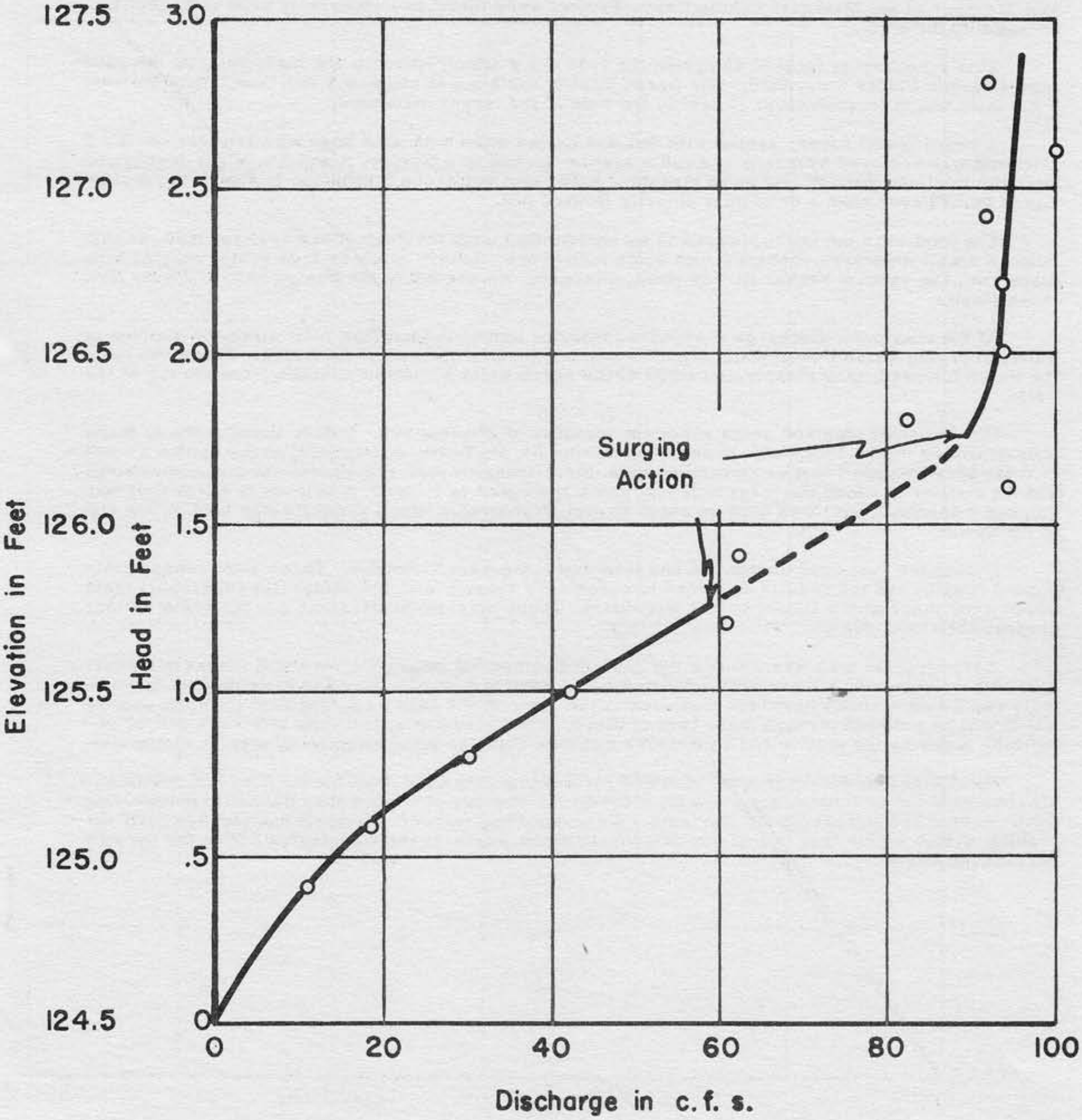
DIAGRAMMATIC SKETCH
EXPERIMENTAL 2' x 2' DROP INLET
M. JOHNSON FARM, NELSON, WISCONSIN



SKETCH SHOWING RELATIVE
POSITIONS OF TWIN
6' X 6' AND EXPERIMENTAL 2' X 2' DROP INLETS
M. JOHNSON FARM, NELSON, WISCONSIN



POND VOLUME CALIBRATION
EXPERIMENTAL 2' x 2' DROP INLET



DISCHARGE CURVE
EXPERIMENTAL 2' X 2' DROP INLET
1st RUN 4/8/37

Typewritten Report

Experimental 2 x 2 Test

On April 8 the Experimental 2 x 2 at Nelson was opened and a test run made. Mr. Wood, Assistant Regional Engineer; Mr. Slavin, Head of the Regional Nursery Section, and Mr. Kennard, Project Manager of the Missouri Dubois Creek Project were there, and apparently were favorably impressed by the sight.

This structure is located alongside the twin 6 x 6 which controls the main gully of the 2600 acre Cascade Valley watershed. The flared inlet of the 2 x 2 is about 3.5 feet lower than the twin 6 x 6 inlet and is located about 15 feet to the side of the larger structure.

A heavy metal cover, sealed with felt and loaded down with sand bags was set over the 2 x 2 inlet and was removed by means of a cable system hooked to a tractor. A sudden, slight depression over the inlet and whoosh, --a solid stream of water shot out at the discharge. It was the first time any of us had ever seen a drop inlet actually flowing full.

The pond over the inlet appeared to be undisturbed until the critical head was reached. At this point, a small drawdown appeared with some turbulence. Later, more or less gentle surging took place until the vacuum broke. At this point, of course, we are below the design head and weir flow takes place.

At the maximum discharge it appeared that the barrel did not flow full, since the surface of the discharging water flowed about 6 inches below the top of the end of the barrel. From this point the water followed, very closely, the slope of the apron walls and about 6 inches from the top of the walls.

Mr. Minshall obtained some excellent pictures of the test run. In fact there were so many photographers on the loose, expert and other optimists, we had to county [sic] noses before we left to make sure we hadn't lost a couple in the mud. Mr. Minshall's pictures showed the initial discharge and the manner in which the large hole just below the apron is formed. A hole about 4 feet deep was cut, but a sandbark [sic] was built up about 80 feet downstream which ponded water back to the end of the apron.

During the run, pond elevations and time intervals were recorded. These were immediately plotted roughly and the results appeared to check very closely with Mr. Minshalls 1934 model tests which were made at the University of Wisconsin. These original model tests are the basis for our present drop inlet design.

A topographic map was used as the basis for computing pond volumes and it seems quite evident that we will need a more accurate pond measurement in order to refine the results. Accordingly two 8 inch orifices have been installed in the riser of the twin 6 x 6. The next time the pond is full it will be emptied through these twin orifices. Pond elevations, and time intervals will be recorded. Knowing the orifice coefficients we can then compute pond volumes at various elevations.

Mr. Peterson, deserves a lot of credit for pushing this field model test. Also Mr. Smith and his cohorts at Camp Nelson, deserve a lot of credit for the excellent work they have done in installing the structure and preparing for the tests. We understand that Mr. Minshall has already used the results of this initial test to sell our Wisconsin Drop Inlets to the Washington Office for the new Service Handbook.

8-INCH DIAMETER CLOSED CONDUIT SPILLWAY NEAR EDWARDSVILLE, ILLINOIS

The following information is taken from an unpublished report by Richard P. Weeber entitled "Results of Field Tests on Inclined Pipes Used in Earth Dam Construction." The report was written at Soil Conservation Service Project Illinois-2, Edwardsville, Illinois, and is dated March 30, 1940.

DESCRIPTION OF SPILLWAY

The closed conduit spillway consists of a 3 ft by 3 ft concrete box inlet about 1.67 ft deep from which an 8-in. vitrified clay tile pipe 120 ft long carries the flow to an outlet structure. The total fall in the structure is about 13.27 ft. The pipe slope is 0.10. A cross section through the spillway is shown in Fig. IX-28. Since this structure was located on the farm of Henry Love, it will be discussed as the Love spillway.

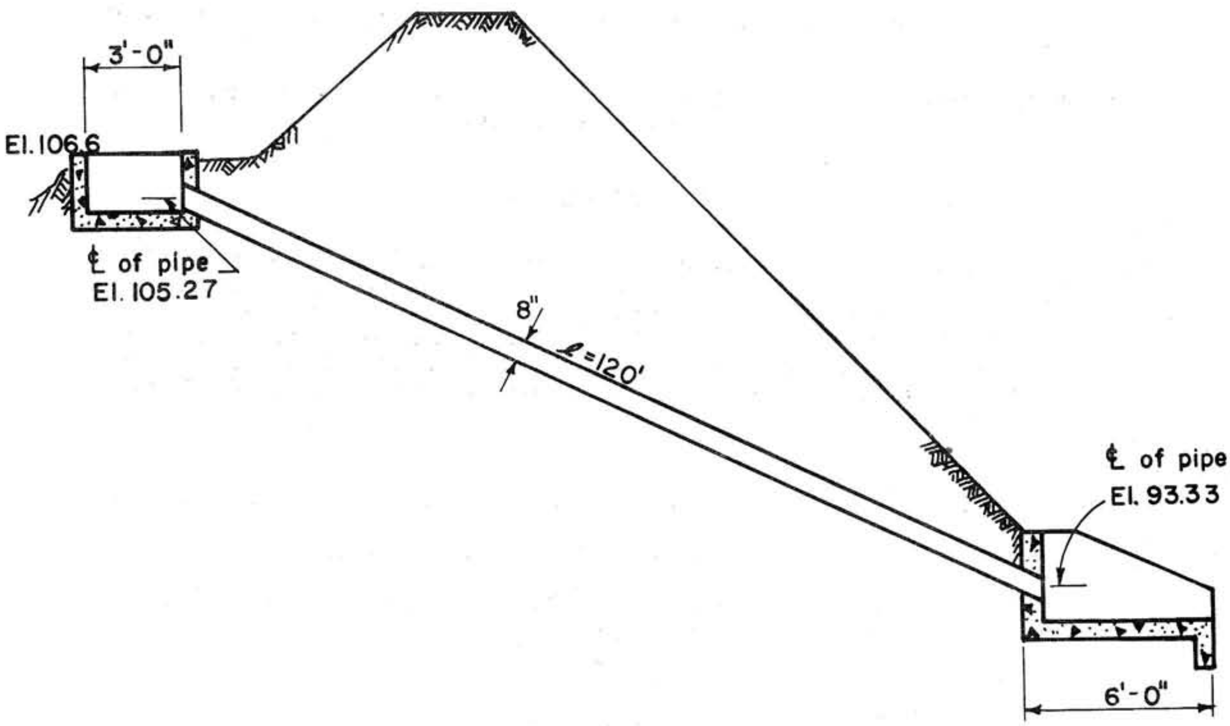


Fig. IX-28 - Love Closed Conduit Spillway.

APPARATUS AND PROCEDURE

In preparation for the tests, a metallic staff gage was installed about 75 ft from the inlet and, quoting Mr. Weeber, "a careful survey of both ponds [Love and Young ponds] was made to ascertain their surface areas at normal pond levels as well as at higher stages...."

The pipe entrance was blocked by a manhole cover on April 25, 1939, to permit natural runoff to fill the pond. The pond had filled sufficiently by August 14, 1939, to permit a test to be made. The test was started by removing the manhole cover from the inlet. Readings of the staff gage were taken at two-minute intervals, timed by a stop watch.

The method of analysis of the data is described by Mr. Weeber as follows:

> The basis on which data on the carrying capacity of these pipes could be arrived at from the field tests, is the fact that by knowing the volume of water carried through the pipes during a certain time and gageheight interval, the discharge may be determined.

The surface area of the pond surface at each gage height had been determined by a careful survey of the shore line. From the staffgage and time observations (or the hydrograph of the water-stage recorder), we know the time interval (in seconds) it took to lower the pond surface a certain gageheight interval (infeet) [sic]. The mean surface area based on the upper and lower limit of the gageheight interval multiplied by the gage height interval gives the volume in cubic feet. Dividing this volume in cubic feet by the time in seconds it took to lower the pond this certain gageheight interval, we obtain the discharge in cubic feet for [sic] second (cfs) corresponding to this gage height.

An example probably serves best to illustrate the method of analysis: On the Love pond at 3:54 P.M. August 18, the outside staffgage reading gave a G.D.E. of 9.425, which corresponds to a pond surface area of 41750 sq ft as obtained from survey and prepared stage-area table or curve. At 3:56 P.M. the G.D.E. was 9.410, equal to 41511 sq ft. Mean surface area is 41633 sq ft; with a stage interval of (9.425 - 9.410) = 0.015 ft. We obtain as "volume for stage interval" (41633 x 0.015) = 624.5 cft. The elapsed time was 2 minutes or 120 seconds. Hence: 624.5 + 120 = 5.20 cfs is the discharge at G.D.E. 9.417.

Computations, in tabular form were made of the observed data of each test run and the results plotted on graph paper with the G.D.E. as ordinate and the discharge in cfs as abscissa. Sufficient points were plotted and a mean curve drawn through these points, resulting in the "stage discharge curves from field test....

A second test was made on August 18, 1939, readings being taken until darkness made it difficult to read the staff gage.

Attempts were made to locate the original survey and test data in order to make a complete reanalysis. All efforts in this direction were unsuccessful.

DESCRIPTION OF FLOW

The cover was removed from the pipe entrance of the Love spillway on August 14, 1939, at 11:15 a.m. The head over the crest of the box inlet was 1.94 ft and the total depth above the pipe invert at the entrance was about 3.60 ft or 5.4D. The pond had a surface area of 0.783 acres at this elevation. The pipe began to run full almost instantaneously. The pipe ran full until the head on the inlet was 0.49 ft, when air pockets were first observed and the pipe ran 0.8 to 1.0 full. The following times, heads and flow conditions were reported by Mr. Weeber.

Time August 14, 1939	Head on crest ft	Flow condition
11:15 A.M.	1.940	Begin test. Pipe full.
1:35 P.M.	0.490	Pipe 0.8 to 1.0 full; air pockets.
1:42 P.M.	0.410	Pipe 3/4 full.
1:44 P.M.	0.390	Pipe 0.5 to 0.75, occasionally back to full again.
1:55 P.M.	0.295	Pipe 0.5 full.
2:00 P.M.	0.260	Pipe 1/4 to 1/2 full.
2:02 P.M.	0.250	Pipe 1/4 full.
2:06 P.M.	0.230	Pipe 1/4 full.
2:11 P.M.	0.205	Pipe 1/4 full.
2:18 P.M.	0.180	Pipe 0.10.
2:27 P.M.	0.158	Pipe 0.10.
2:38 P.M.	0.14	End of staff gage readings at two minute intervals.
3:00 P.M.	0.100	
4:00 P.M.	0.080	
5:00 P.M.	0.060	
6:00 P.M.	0.050	
7:00 P.M.	0.045	

A rain from 7:35 p.m. on August 17, 1939, to 7:30 a.m. on August 18, 1939, raised the pond level to a head over the box inlet of 3.34 ft. The pond surface area was 1.289 acres. The depth over the pipe entrance at the invert was about 5.0 ft or 7.5D. The second test was begun at 1:12 p.m. on August 18 and continued until 6 p.m. when darkness called a halt to the readings. No observational data are given for this test.

The time-discharge curve for the test of August 14, 1939, as drawn by Mr. Weeber is shown in Fig. IX-29. This curve exhibits some peculiarities that cannot be explained. Until 1:40 p.m. there is a steady decrease in discharge while the pipe flows full under decreasing

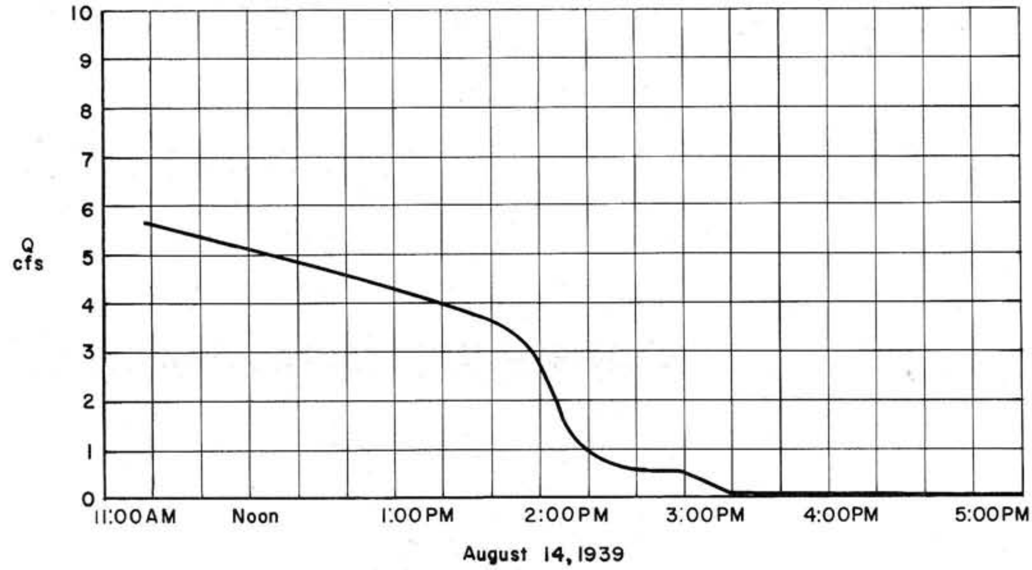


Fig. IX-29 - Time-Discharge Curve for **Love Closed** Conduit Spillway. **Test** of August 14, 1939.

head as the pond level drops. This is to be expected. Also to be expected is the transition from pipe to weir control from about 1:40 p.m. to about 2:00 p.m. The drop in level after 2:00 p.m. at a decreasing rate is also expected as the head over the crest decreases under weir control. However, the sudden breaks at 3:00 p.m. and 3:20 p.m. are illogical on the basis of the information available.

DISCHARGE COEFFICIENTS

The head-discharge curve contained in Weeber's report is presented in Fig. IX-30. This curve was used in determining the discharge coefficients.

Weir Coefficient

When data extracted from the curve of Fig. IX-30 is plotted on logarithmic paper, a straight line is obtained for weir control but the line has a slope (exponent of H in Eq. I-1) of about 2.9. This is nearly the exponent for a semi-cubic weir and almost twice the exponent expected for a rectangular weir, such as actually exists. However, it will be noted that the head-discharge curve shown in Fig. IX-30 shows a head reading of 0.08 ft when there is no flow. If H is plotted against $Q^{2/3}$ a straight line is obtained having the equation

$$Q = 2.76\,L\,(H - 0.070)^{3/2} \qquad \text{(IX-5)}$$

indicating that the zero head reading is in error by 0.070 ft. It may have changed between the time the staff gage was set and the time the observations were made.

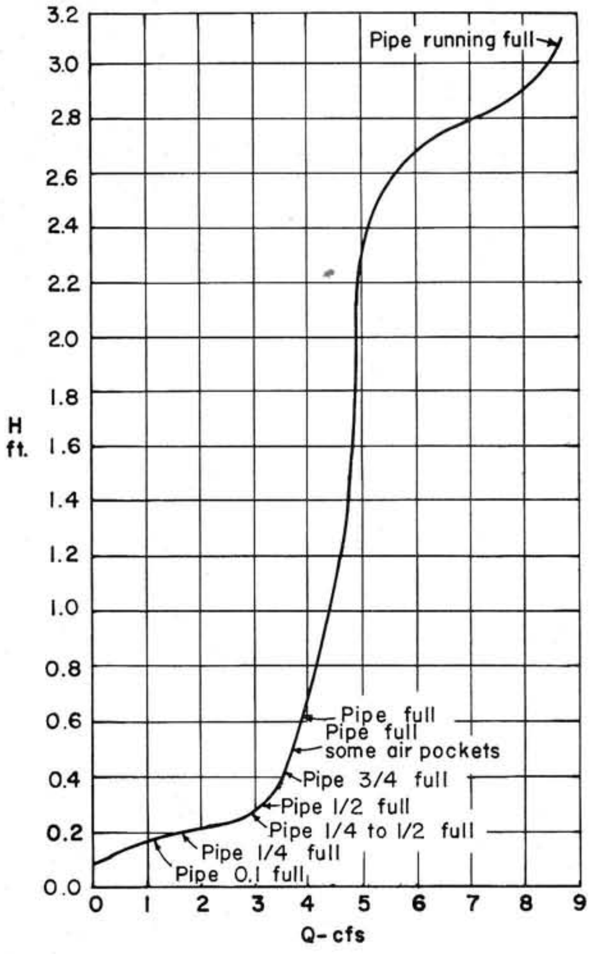


Fig. IX-30 - Head-Discharge Curve for Love Closed Conduit Spillway. Tests of August 14 & 18, 1939.

The weir discharge coefficient is very low. Because of the low precision of the basic data, it is suggested that this value be not used for design purposes.

Entrance Loss Coefficient

Although the entrance loss coefficient was computed, the values obtained will not be presented here. The value of K_e requires an accurate evaluation of the friction loss. The condition of the pipe is not known so it is not possible to evaluate the friction loss and K_e with sufficient precision to permit the recommendation of a design value of K_e.

The peculiar S-shape of the head-discharge curve (Fig. IX-30) at the highest heads should be noted. The time-discharge curve (Fig. IX-29) does not exhibit this peculiarily. Data for checking the head-discharge curve are not available and no reason is advanced for the S-shape.

PRESSURE COEFFICIENTS

No data are available for the computation of the pressure coefficients for this spillway.

CONCLUSIONS AND RECOMMENDATIONS

The fact that a closed conduit spillway on a steep slope will flow full is confirmed by these experiments. The least head over the invert of the pipe when the spillway began to flow full was 5.5D. This is close to the minimum depth found in Part V* that will insure full pipe flow, so full flow should be expected.

The weir coefficient C, 2.76, for use in Eq. I-1 is low and probably unreliable. Its use is not recommended.

No reliable values of the entrance loss coefficients or pressure coefficients could be determined.

The discharge curves presented here exhibit some unexplained peculiarities. However, the spillway seems to have performed well. If this type of spillway is used, it is recommended that design values be taken from similar structures reported in Parts IV and V.*

It is recommended that spillways similar to the Love spillway not be constructed because, although the pipe did fill at the high initial heads, previous tests indicate that the drop inlet should be 5D deep in order to insure full flow at minimum heads over the spillway crest. The Love drop inlet was only 2.5D deep.

14-INCH DIAMETER CLOSED CONDUIT SPILLWAY NEAR EDWARDSVILLE, ILLINOIS

The information on this structure, located on the C. G. Young farm near Edwardsville, Illinois, is taken from the report by Richard P. Weeber mentioned in the previous section.

DESCRIPTION OF SPILLWAY

The spillway consists of a concrete box inlet 4 ft by 4 ft in plan and about 1.71 ft deep. There is a headwall about 6 in. high on the box. A 14-in. metal pipe (old boiler tube) 103 ft long carries the flow from the box inlet to an outlet structure containing two cross sills. A photograph taken in 1954 shows that the pipe entrance is square edged. The total fall through the structure, including the outlet, is 21.7 ft. The slope of the barrel is 0.177. The surface area of the pond at the elevation of the inlet was 3.67 acres. A cross section along the centerline of the spillway is shown in Fig. IX-31.

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*Fred W. Blaisdell, Hydraulics of Closed Conduit Spillways--Parts II through VII. Results of Tests on Several Forms of the Spillway, St. Anthony Falls Hydraulic Laboratory Technical Paper No. 18-B, March 1958.

APPARATUS AND PROCEDURE

The methods employed to conduct the tests and make the analysis were similar to those for the Love spillway described previously and reference is made to the previous subsection for the details. In addition to a staff gage located about 48 ft from the inlet, a Friez type FW-1 water level recorder was installed about 60 ft from the inlet. A time scale of 1 in. equals 50 min and a stage scale of 5 in. equals 1 ft was used. A stilling well was installed for the recorder float and a graduated float tape with an index pointer was used to check the recorder pen markings. A photostat of the recorder chart is appended to Weeber's report.

The pipe entrance was blocked by a manhole cover on May 5, 1939.

As for the Love spillway, all attempts to locate the original data ended in failure.

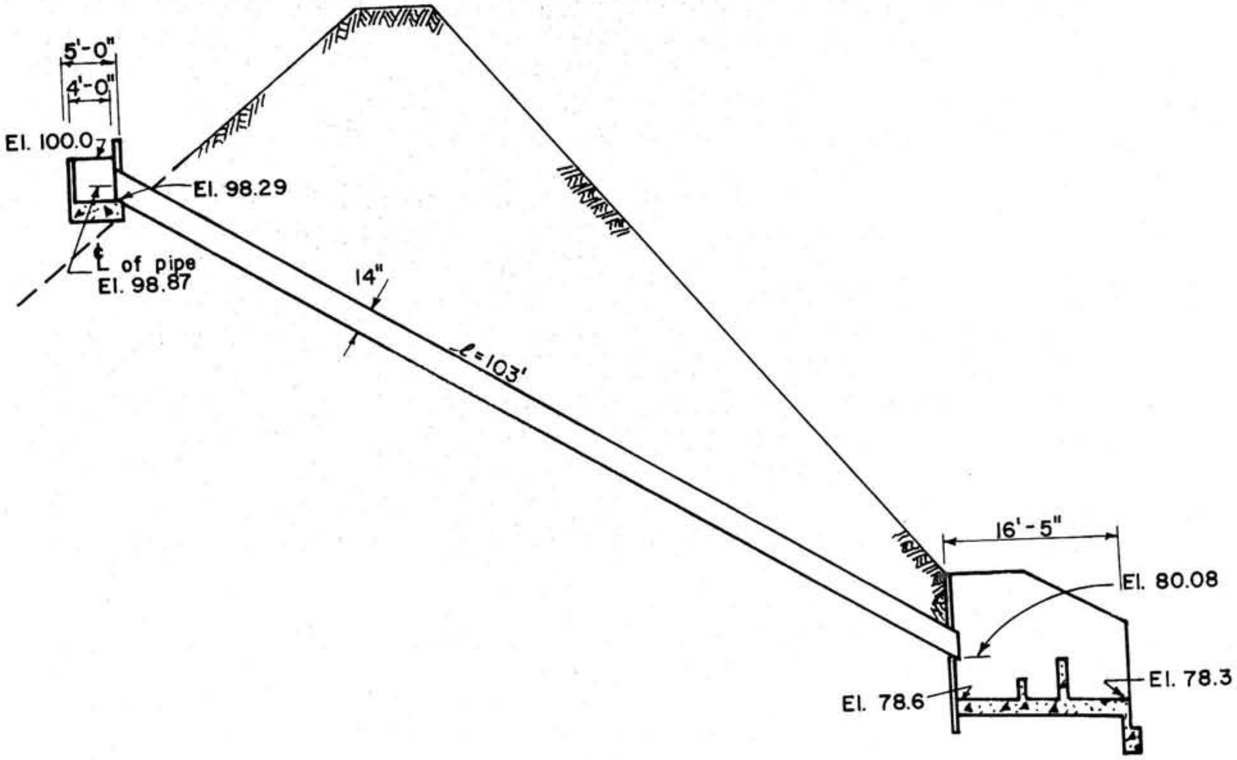

Fig. IX-31 - Young Closed Conduit Spillway.

DESCRIPTION OF FLOW

Inflow into the pond had built up a head over the box inlet crest of 2.32 ft by August 15, 1939. The pond surface area was 5.305 acres.

The cover was removed from the pipe entrance and the pond started to drain at 10:54 a.m. Mr. Weeber reports (Elevations reported by Mr. Weeber have been changed to head on the crest of the inlet):

> At the beginning of the test run of the Young pond, the pipe was flowing about 0.8 full. However, by 11:18 A.M. it started to flow full. The changing slope of the hydrograph as traced on the chart by the recording pen clearly indicates this increased pipe carrying capacity. The first reduction in the full pipe flow was observed at 3:40 P.M. when occasional air pockets could be observed at the outlet end of the pipe. The pond had lowered by this time from [a head of 2.32 ft to 0.90 ft].
>
> At 3:50 P.M. the pipe was 0.9 full. From 4:08 P.M. to 4:20 P.M. the pipe was flowing one-half full, with rather irregular flow, running at times to as much as 0.8 full. At 4:30 P.M. the flow was 0.4 full. The last visual observation was made at 8:11 P.M. The pipe was flowing about 0.10 full with a [head of 0.3 ft]. The attached print of the waterstage recorder chart shows the complete hydrograph with regard to gageheight and time. The chart was removed on August 17, 1939, at watch time 8:33 A.M. Outside staffgage reading 10.025; chartline 10.032; index point 10.031.

At 11:18 a.m. when the pipe filled, the head on the crest was about 2.26 ft and the depth over the pipe invert at the entrance was 3.97 ft or 3.4D. This is significantly less than the minimum depth found necessary in Part V* to insure full flow. Therefore, positive assurance that the pipe would flow full could not be expected. The fact that the pipe filled only after running for 24 min is further evidence that drop inlets should have the minimum depth indicated in Part V.

According to Mr. Weeber's descriptions, it is possible that orifice control existed from 10:54 a.m. until 11:18 a.m., pipe control from 11:18 a.m. to 3:40 p.m., transition from pipe to weir from 3:40 p.m. to sometime preceding 4:20 p.m., and a definite weir control after 4:30 p.m.

A curve of time vs. discharge for the Young spillway is shown in Fig. IX-32. The very erratic nature of the curve for the first hour probably reflects the difficulty of obtaining accurate discharges from computations of pond volume increments and time increments. It is also probable that the smooth curve is drawn through points which exhibit considerable scatter. It is for this reason that attempts were made to secure the original data and recompute them. The irregularities in the curve at about 7:00 p.m. and midnight are probably not representative of actual conditions. Similar irregularities are not apparent on the water level recorder chart. All this indicates that the data as presented exhibit a very low degree of precision.

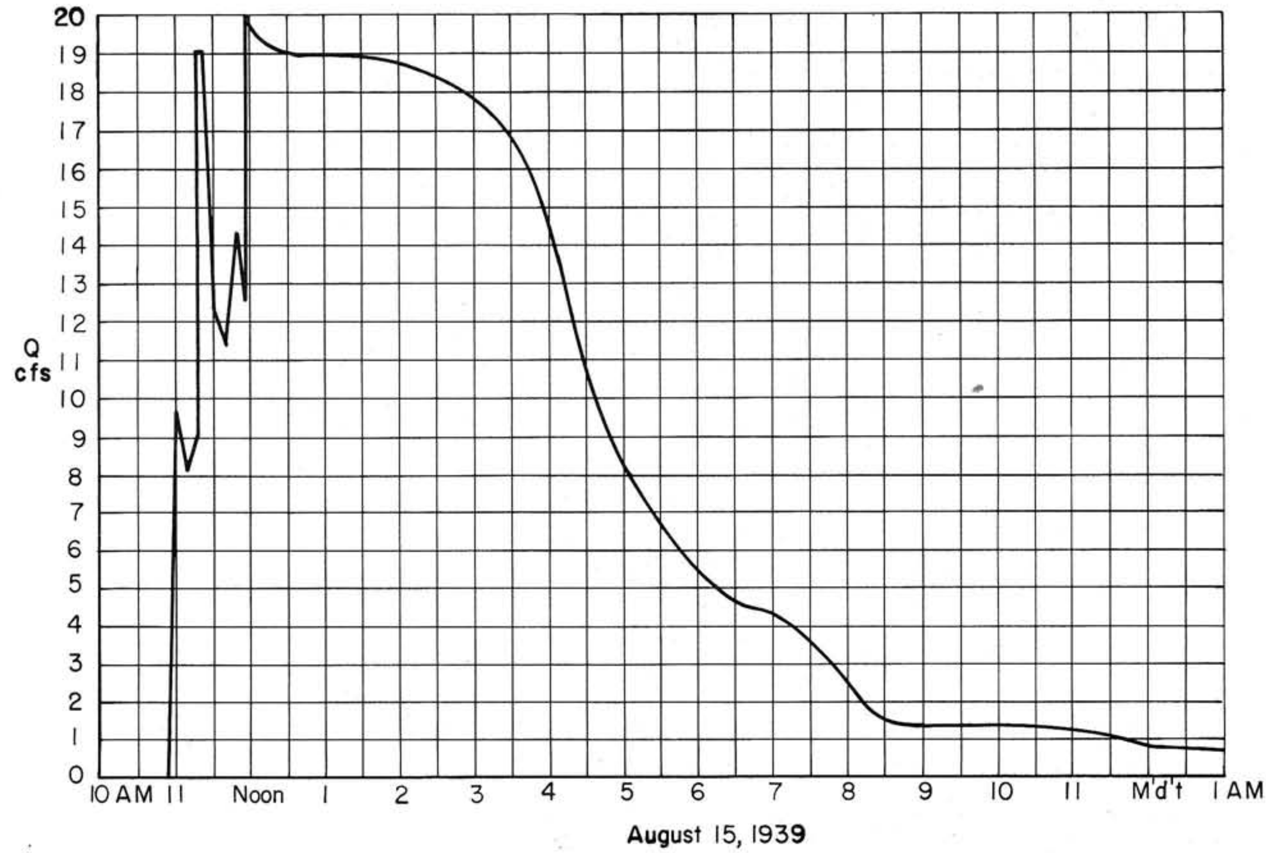

Fig. IX-32 - Time-Discharge Curve for Young Closed Conduit Spillway.

DISCHARGE COEFFICIENTS

The head-discharge curve plotted by Weeber is presented in Fig. IX-33. This is not a complete curve, since the data for orifice control during the first 24 min are not presented.

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*Fred W. Blaisdell, Hydraulics of Closed Conduit Spillways--Parts II through VII. Results of Tests on Several Forms of the Spillway, St. Anthony Falls Hydraulic Laboratory Technical Paper No. 18-B, March 1958.

This curve and the curve of Fig. IX-32 were used as sources of data for the subsequent analyses.

Weir Coefficient

Weir flow existed during the period after perhaps 4:00 p.m. However, water must have flowed over the low headwall until the head dropped to 0.5 ft at about 6:00 p.m. Reference to Fig. IX-32 shows that the time-discharge curve takes a peculiar shape after 6:00 p.m. Because of this, the head-discharge data are assumed to be unreliable and no attempt is made to present weir coefficients.

Orifice Coefficient

Orifice flow existed from 10:54 a.m. to 11:18 a.m. The orifice head at the beginning of the period was 3.45 ft and at the end of the period was 3.39 ft. From Fig. IX-32 it can be seen that the discharge during the period averaged about 9 cfs. Using these figures, the value of C_o in Eq. I-7 averages 4.55 with extreme values of 4.0 and 4.8. These values of C_o are admittedly of low precision.

Entrance Loss Coefficient

The entrance loss coefficient was not computed. The questionable head-discharge data and the heavy dependence of K_e on an accurate evaluation of the pipe friction factor made computations of K_e inadvisable.

PRESSURE COEFFICIENTS

No data are available for the computation of the pressure coefficients for this spillway.

CONCLUSIONS AND RECOMMENDATIONS

The use of spillways similar to the Young spillway is not recommended. The small depth of the drop inlet is not sufficient to assure full flow beyond reasonable doubt.

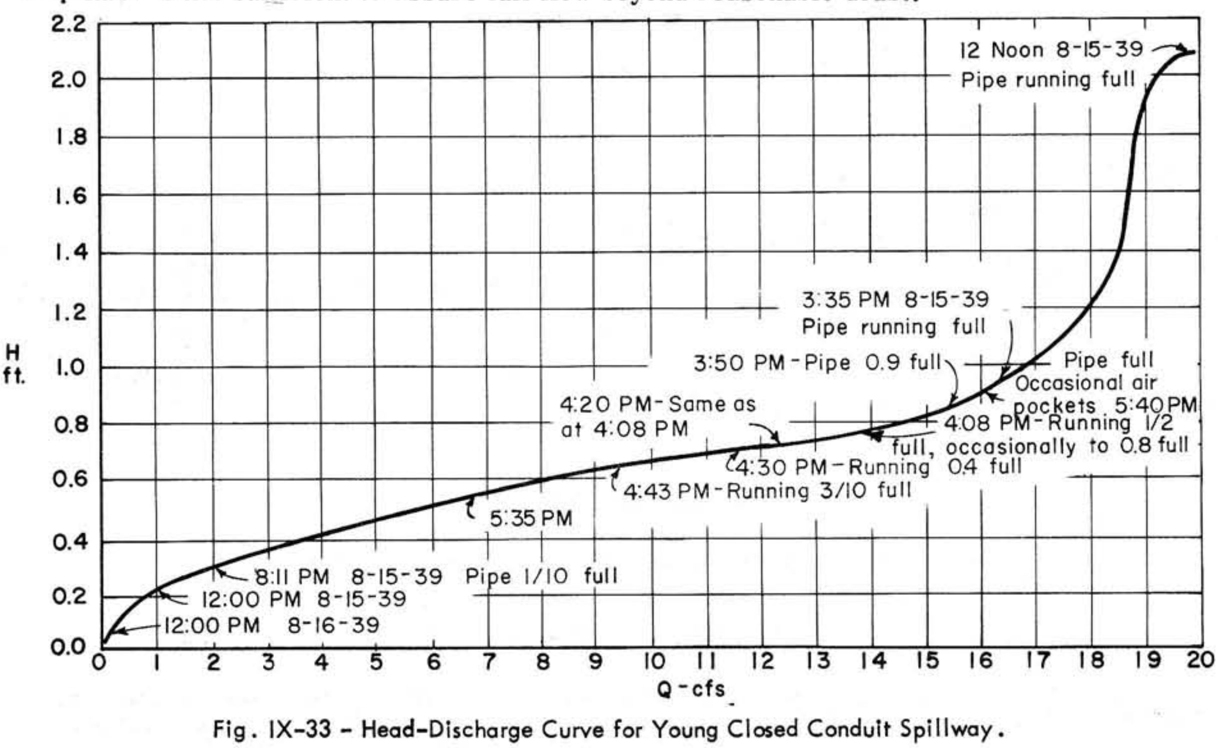
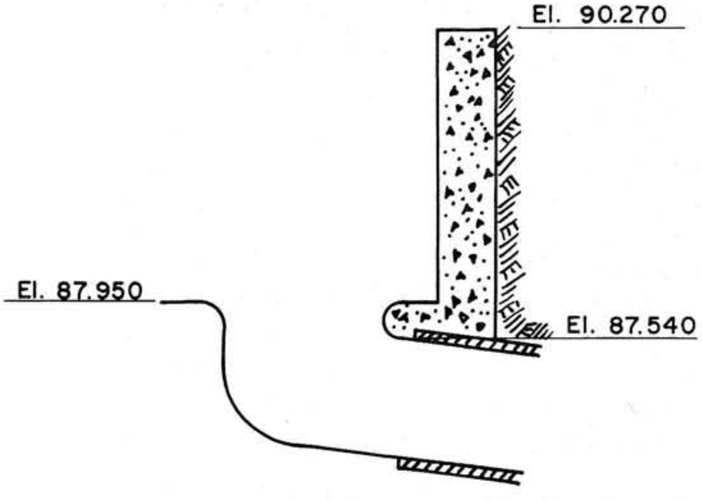


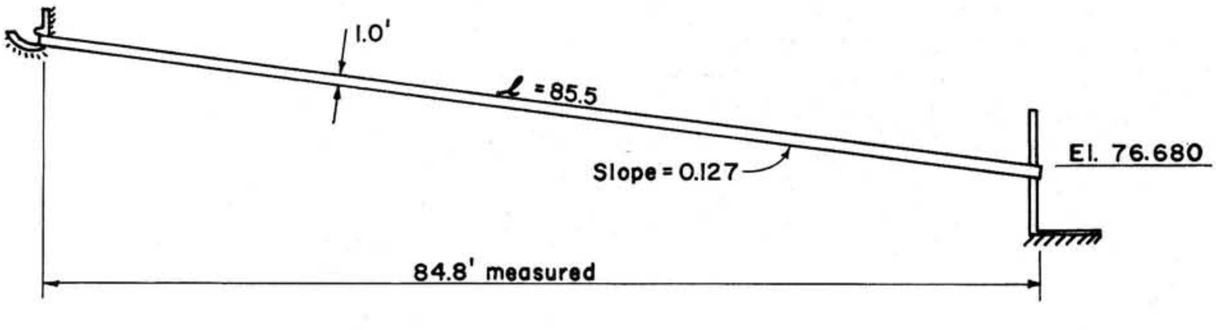
Fig. IX-33 - Head-Discharge Curve for Young Closed Conduit Spillway.

12-INCH CLOSED CONDUIT SPILLWAY AT THE SOIL CONSERVATION EXPERIMENT STATION, BETHANY, MISSOURI

The information presented here on the closed conduit spillway at the retention reservoir dam located on the Soil Conservation Experiment Station at Bethany, Missouri, was obtained from D. D. Smith, formerly Project Supervisor, Agricultural Research Service. The experiments were performed under the direction of A. W. Zingg, now Chief, Watershed Technology Research Branch, Soil and Water Conservation Research Division, Agricultural Research Service. Mr. Smith furnished the original pond survey notes used in determining the pond area; the original water level recorder chart; Mr. Zingg's original plots of pond stage-area, stage-volume, head-time, and discharge-time; the scale drawings prepared by Mr. Zingg; and photographs. Mr. Zingg's test was carried out in 1939.



Inlet Details



Sectional Elevation on ℄

Fig. IX-34 - Bethany, Missouri, Closed Conduit Spillway.

DESCRIPTION OF SPILLWAY

The closed conduit spillway consists of a concrete elbow entrance with rounded crest and headwall, about 85.5 ft of 12-in. diameter vitrified clay tile pipe, and a masonry outlet structure. It was built in 1936.* An auxiliary spillway with a crest 4 ft above the closed conduit spillway crest was provided for flows which exceeded the capacity of the closed conduit spillway. The pond area was 1.6 acres at the emergency spillway crest elevation and 0.4 acres at the inlet crest elevation.

A section through the inlet is shown in Fig. IX-34. It is noted that the vertical distance from the inlet crest to the crown of the pipe scales 0.31 ft, while the measured distance, obtained from the original survey notes, is 0.41 ft. The length of crest is computed from the diameter scaled from Fig. IX-34. Its length is 4.24 ft, computed using the formula of Fig. I-3d but substituting for D_r the D_{rc} as shown in Fig. I-3c. No allowance was made for the fact that the headwall is set back about 4 in. from the crest. A picture of the inlet is shown in Fig. IX-35.

The slope of the conduit is 0.127 and the total drop from the inlet crest to the center of the pipe at the outlet is 11.77 ft. A section through the spillway is shown in Fig. IX-34.

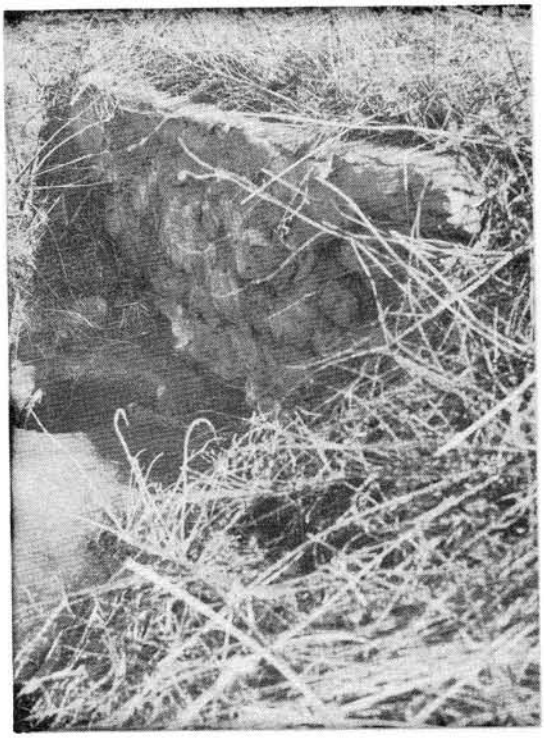


Fig. IX-35 - Entrance to Bethany, Missouri, Closed Conduit Spillway.

APPARATUS AND PROCEDURE

No description of the flow conditions during the tests is available. Notes on the water level recorder chart indicate that the test was started at 9:38 a.m. and ended at 4:40 p.m. Apparently the same methods were used as for the Young spillway; that is, the inlet was apparently plugged and the plug removed suddenly to start the test. The pond was full to the emergency spillway crest when the test was begun.

In preparation for the analysis, the pond survey data was plotted, 1-ft contours from Elevation 88 to Elevation 92 inclusive were drawn, and the area of each contour was planimetered. The pond elevation-area curve thus obtained checked within 1.5 per cent of that computed by Zingg.

In analyzing the data, the stage was read from the water level recorder chart at 5-min intervals, increments of time and stage between readings were computed, and the rate of change of pond surface elevation determined. The mean head on the inlet crest for each interval was computed and the pond area for this head was read from the elevation-area curve previously plotted. The pond area multiplied by the rate of change of pond surface elevation gives the rate of discharge through the spillway. The method of analysis is thus similar to that used for many of the Stillwater, Oklahoma, tests described by W. O. Ree [I-42].

DESCRIPTION OF FLOW

No observations of flow conditions during the test are available. Reference to Fig. IX-36 shows however that both pipe and weir flow controls were obtained. Apparently the change from pipe control to weir control was abrupt and definite.

DISCHARGE COEFFICIENTS

The head-discharge data are plotted in Fig. IX-36. It will be noted that the data points

*"A 20-Year Appraisal of Engineering Practices in Soil and Water Conservation," by Dwight D. Smith, Agricultural Engineering, September 1952, p. 553.

fluctuate considerably but that the curves shown represent the average quite well and follow the trend of the data.

Weir Coefficient

Two curves of weir flow are shown in Fig. IX-36. The actual equations of these curves were derived from the plot shown in Fig. IX-37. It can be seen from Fig. IX-37 that the upper curve applies between heads of 0.9 and 1.4 ft while the lower curve applies from heads of 0.0

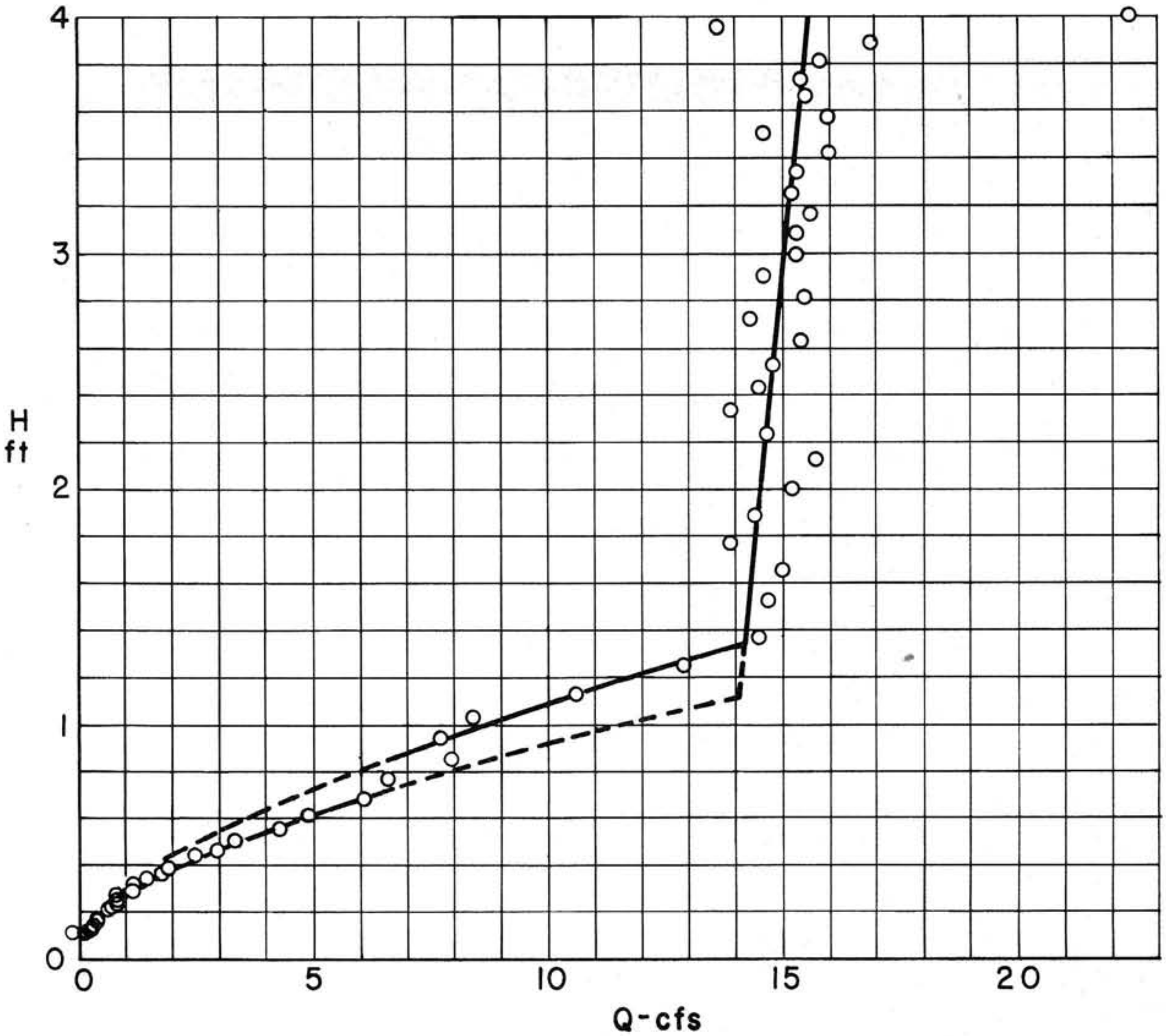


Fig. IX-36 - Head-Discharge Curve for Bethany, Missouri, Closed Conduit Spillway.

ft to 0.7 ft. The reason for the change can only be a matter of conjecture in the absence of supporting data. Both curves seem well defined. It should be noted that neither curve passes through the origin, there being an indicated head of 0.10 ft at zero discharge. The equation for the lower curve is

$$Q = 3.25\,L\,(H - 0.10)^{3/2} \quad \text{for} \quad 0 < H < 0.7 \tag{IX-6a}$$

and for the upper curve is

$$Q = 2.40\,L\,(H - 0.10)^{3/2} \quad \text{for} \quad 0.9 < H < 1.4 \tag{IX-6b}$$

For $0.7 < H < 0.9$, the discharge should be somewhere between the values given by Eqs. IX-6a and IX-6b.

Entrance Loss Coefficient

The pipe flow portion of the head-discharge curve shown in Fig. IX-36 has the equation

$$Q = 5.01\,A\sqrt{H + 11.77} = 5.01\,A\sqrt{H_t} \qquad \text{(IX-7)}$$

This is the correct theoretical form of the pipe flow equation; it indicates that the trend of the data is theoretically correct.

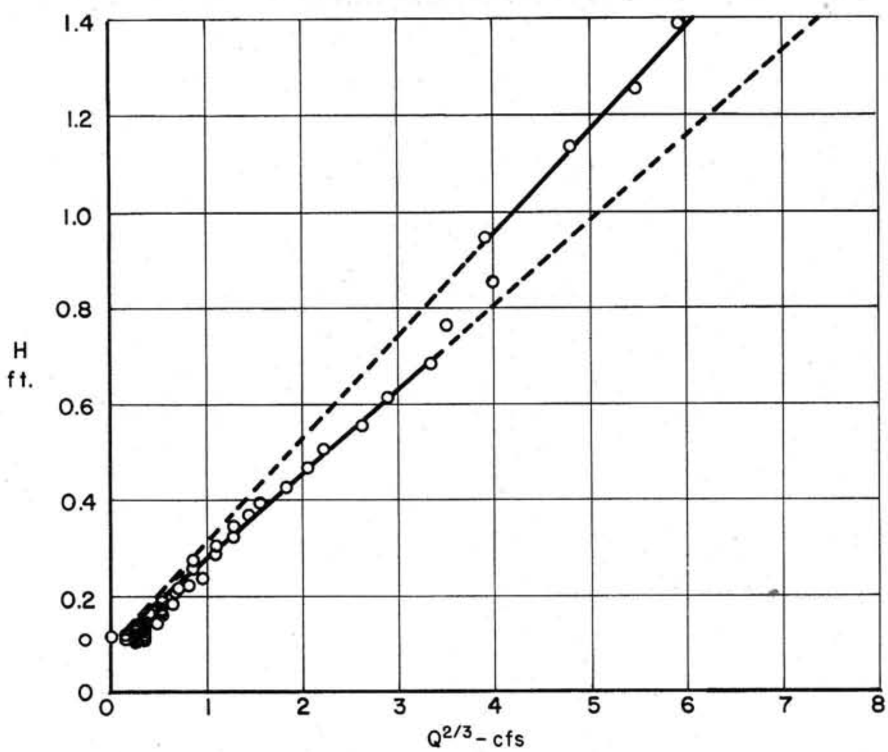


Fig. IX-37 - Weir Flow Head-Discharge Curve for Bethany, Missouri, Closed Conduit Spillway.

If Eq. IX-7 is put into the form of Eq. I-5, it becomes

$$Q = A\sqrt{\frac{2g\,H_t}{2.57}} \qquad \text{(IX-8)}$$

Since $K_o = 1.0$, $K_e + f\frac{\ell}{D} = 1.57$. The value of ℓ/D is 85.5. A reasonable value of Manning's n might be 0.008, from $K_e = 0.54$. If $n = 0.009$, $K_e = 0.29$, and if $n = 0.010$, $K_e = 0.00$. Therefore, it is apparent that the friction in the conduit must be known with good accuracy if K_e is to be reliably determined. The values of K_e are given here only to show their possible range, and they are given without recommendation.

PRESSURE COEFFICIENT

No data are available for the computation of the pressure coefficients for this spillway.

CONCLUSIONS AND RECOMMENDATIONS

Insufficient information is available to permit definite conclusions regarding the performance of the Bethany closed conduit spillway. Based on such information as is available, it appears that the spillway has a form that permits only the desirable weir and pipe flow controls to exist in spite of the fact that the height of the entrance is only 1.4D. However, the inlet is well shaped and this undoubtedly contributes to the apparently good performance.

No recommendation is made regarding the use of this form of spillway, either for or against, since information on which a definite recommendation can be based is not available.