DETERMINATION OF THE HEAD-DISCHARGE RELATIONSHIP FOR AN EXISTING HIGHWAY CULVERT TO ENABLE ITS USE FOR MEASURING RUNOFF

By

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DETERMINATION OF THE HEAD-DISCHARGE

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TO ENABLE ITS USE FOR MEASURING RUNOFF

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PREFACE

Surface runoff and its control are important factors of the economy in many parts of the country. Many cities and towns in the Southwest depend entirely on storage reserviors for their domestic water supply. Definite information on total watershed yield is therefore essential for design of these structures. An accurate estimation of runoff must also be made for design of irrigation reserviors, and knowledge of peak rates of runoff is important for design of terrace outlet channels, diversion terraces, and for detention reservior spillways.

In all but a few areas, definite information on runoff from watersheds of 500 to 5,000 acres is not available, and design of conservation structures is based purely on experience or the rational runoff formula--Q=CIA--or Talbot's formula or a combination of these.

The U. S. Geological Survey maintains gaging stations on most large rivers and tributaries. However, most of the drainage areas of these streams are expressed in hundreds or even thousands of square miles--much larger than those for which conservation structures and culverts must be designed. Data concerning runoff from watersheds up to a few hundred acres are readily available for many sections of the country, as this can be readily obtained by installing simple measuring devices such as weirs or flumes.

Information on runoff from drainage areas between these small watersheds and those for which the U. S. G. S. records is limited to a few localities. Work is in progress at the three Soil Conservation Service hydrologic research stations at Cochocton, Ohio, Hastings, Nebraska, and Waco, Texas. Watersheds at these stations range in size from 10 to 5,000 acres, with only a very few larger than 1,000 acres. Data from the larger watersheds have been recorded for several years and are proving useful for design purposes in those regions. However, local variations of such physical conditions as soil, cover, and topography of the watershed and such climatic factors as intensities and amounts of rainfall limit the applicability of the data when used in a physiographic area different from that of the experimental site.

In 1951, the Oklahoma Agricultural Experiment Station and the U. S. Department of Agriculture, Soil Conservation Service-Research, initiated a joint hydrologic research project to determine rates and amounts of runoff from agricultural areas in North Central Oklahoma. Suitable watersheds were found along State Highway 40 approximately fifteen miles north of Stillwater. These watersheds were easily accessible from the highway and had as outlets culverts with free outfall, which are being used as measuring devices. A complete description of the watersheds and a discussion of the hydrologic techniques employed in the study are embodied in a Master's Thesis by Ted L. Willrich.[15]

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INTRODUCTION

This thesis presents the results of an investigation to determine the suitability of an existing highway culvert for measuring runoff from its watershed. The watershed is located along State Highway 40, twenty miles north of Stillwater, Oklahoma, and is similar in both vegetation and topography to others being studied in the area. The investigation was undertaken as a part of the hydrologic research work being conducted by the Agricultural Engineering Department of Oklahoma A. & M. College in cooperation with the Soil Conservation Service-Research, U. S. Department of Agriculture.

Previous studies of the use of culverts for runoff measurement were not entirely applicable in this case because flow is governed by downstream channel control. However, because of the watershed's size--3,000 acres--and its similarity to the others being studied, runoff data were greatly desirable.

The culvert, a five barrel rectangular concrete structure, was built in 1936; and when the highway was paved in 1944, the channel was widened and straightened for 300 feet both upstream and downstream. Its present condition is shown by figures 1 and 2 . Also in 1944 the floor of the culvert was cleaned. Since then silt has been deposited on the floor to an average depth of two feet. The reason for the present deposition is probably either that the culvert floor was originally built lower than the natural bed of the stream or the increase in channel width made in 1944 resulted in a decreased velocity of the water flowing.

Acknowledgement

The research work was carried on by the author under the general supervision of Professor E. W. Schroeder, Head, Agricultural Engineering Department, and Frank R. Crow, Thesis Adviser. Immediate direction was given by Mr. W. O. Ree, Project Supervisor, Stillwater Outdoor Hydraulic Laboratory of the Soil Conservation Service.

The author is greatly indebted to Mr. Ree for his patient instruction and counsel given throughout the experimentation and analysis of results and for his making available the facilities of the laboratory.

Appreciation is also expressed to the personnel of the Soil Conservation Service who constructed the laboratory set-up and the models.

THE PROBLEM

The problem of calibrating the culvert was accentuated by adverse channel conditions and by silt and debris in the culvert barrels.

The channel exercises considerable downstream control on the flow. Generally, to be of value as a flow rate measuring device, the culvert must exercise control over the flow. This can be either entrance control, exit control, or control by internal wall friction. However, the condition of flow through the culvert studied is part full with outfall partially submerged (depth greater than critical depth) with some increase in headwater. Only partial control is maintained by the entrance.

The channel is nearly level for 600 feet upstream and 300 feet downstream from the culvert. The approach channel and exit channel are straight and uniform for 300 feet in each direction. At Sta. 3+25 E (downstream), control is exercised by a partial fill across the channel. This is at the downstream limit of the 1944 channel improvement operations. The elevation of this constriction is 91.4,* and it backs water up to the culvert entrance. Another obstruction occurs 1,180 feet downstream where a small tributary enters the main stream. An accumulation of silt and small rocks which was washed down the small channel at times of no flow in the larger one has created a block of the main channel to an elevation estimated at 90.3. This causes slack water as far back as St. 3+25 E.

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*Assumed datum, 100.00 feet--top of northwest wingwall.

The culvert consists of five rectangular concrete barrels, each 8 x 10 x 38 feet, separated by concrete walls one foot thick. Wingwalls extend 11 feet at a 30 degree angle with the axis of the culvert. Figures 3 and 4 show general views of the entrance (west) and exit ends. Other views are shown on succeeding pages. The invert has no slope and at this writing is covered with an accumulation of silt to a depth ranging from 1.6 to 2.4 feet. The deposition is believed to be fairly stable. A considerable amount of logs, brush, and trash has collected at the entrance and is shown by figure 1 .

Under ordinary circumstances such a culvert would be rejected immediately as a flow rate measuring device because of the downstream channel control and the silt and debris in the barrels. However, because of the size of the watershed and its similarity to others being studied nearby, data were highly desirable. Although the accuracy of the flow measurements will be relatively low, data on peak flow will still be valuable. Soil Conservation Service officials have stated that even an estimate indicating the order of magnitude of flow rates would be of value.

Figure 1. Culvert entrance.

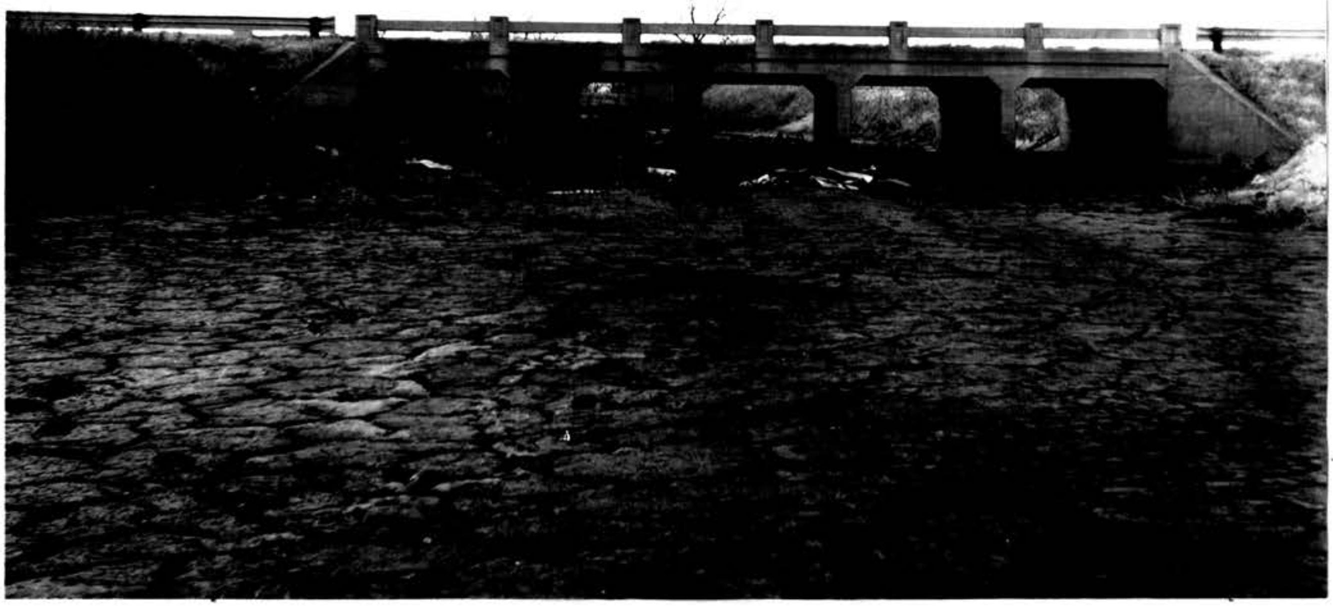


Figure 2. Culvert exit.

Figure 3 . Culvert exit viewed from 325 feet downstream.

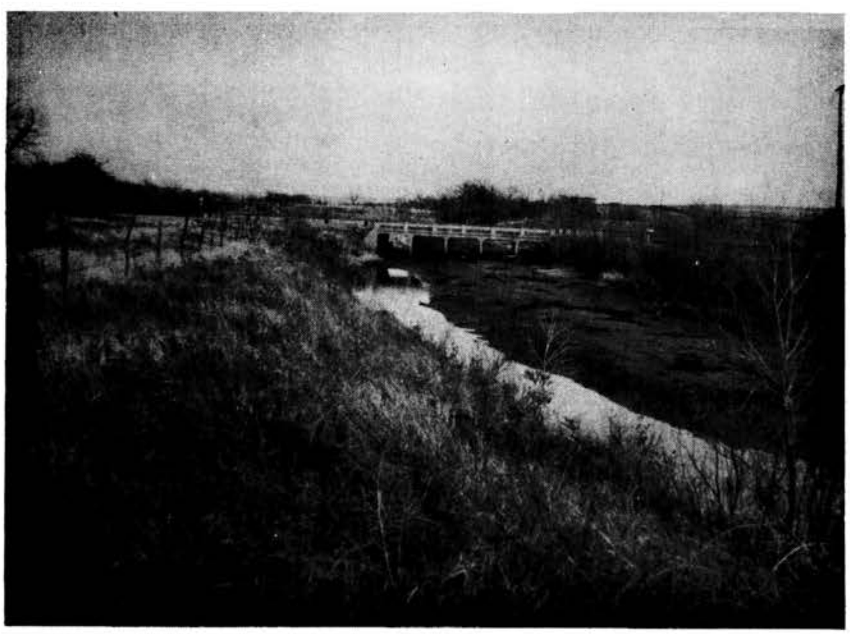


Figure 4 . Culvert entrance viewed from 300 feet upstream.

Figure 5 . Channel immediately below culvert.

Figure 6 . Channel immediately above culvert

THE OBJECTIVES

The general objective of this investigation was to determine the head-discharge relationship for an existing highway culvert with downstream channel control.

Specifically:

1. The practicability of using this culvert as a flow rate measuring device.

2. The most expedient way of determining the flow passing through the culvert.

3. The head-discharge relationship (rating table) for the culvert.

4. The probable accuracy of the rating table developed.

REVIEW OF LITERATURE AND DEVELOPMENT OF THEORY

The elementary analysis of the flow of water through culverts was outlined by Woodward in 1920. It was not until 1925, however, that the exhaustive report of culvert tests conducted at the University of Iowa was published by Yarnell, Nagler, and Woodward.[16] Although the latter investigation included tests of some of the culverts flowing part full, the formulas derived on the basis of the Iowa tests pertained to culverts flowing full. Among the significant contributions of those tests were the evaluation under actual operating conditions of roughness factors for concrete, corrugated metal, and vitrified clay pipe culverts and concrete box culverts and a preliminary evaluation of the effect of entrance conditions on the flow of water through culverts of the various types.

Ree and Crow[9] have conducted experiments on the use of rectangular culverts for runoff measurement. The effect of culvert shape (depth to width ratio) on the head-discharge relationship was determined. Also shown were the head-discharge relationship for various culverts equipped with weir sills and the methods of determining the head-discharge rating for any culvert under similar conditions. The work was limited, however, to culverts flowing part full with free outfall and is not applicable to the culvert described herein. They based their work on studies by Mavis.

Mavis[8] showed that for a conduit flowing part full with free outfall, the relation between pond elevation and discharge is well defined and stable for any conduit whose slope exceeds the "neutral slope" which is required to overcome friction losses induced by the roughness of conduit walls. It is dependent on diameter or geometrical proportions.

If the rate of discharge remains constant, and if the tailwater level is raised, the headwater remains unchanged until the tailwater level approaches the level of the headwater pond or reaches the top of the conduit at the outlet. This is represented by the line 1-2 in Figure 7 . As the tailwater level is further raised to submerge the outlet and fill the conduit, the flow is represented by the line 3-4. The rate of discharge is then a function of h which is the difference between headwater and tailwater elevations.

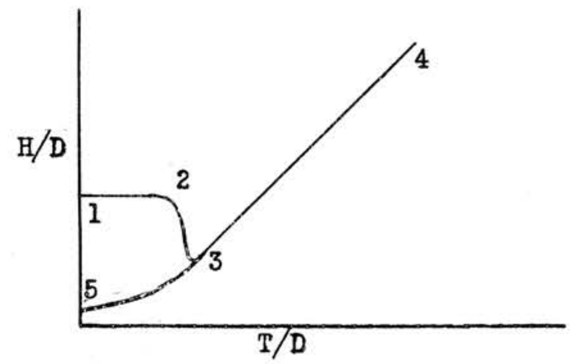


Figure 7

In this range the total head loss, h, is the sum of the entrance loss, the velocity head loss, and the friction head loss through the culvert. Within the range 3-4 any change in the elevation of tailwater level is promptly reflected in an equal change of headwater level, assuming the rate of discharge to remain constant.

The transition 2-3 occurs when the tailwater level is at or near the crown of the culvert at outlet or as tailwater elevation approaches headwater elevation. It is a critical zone where flow is passing from free to submerged discharge and represents the flow conditions investigated in this thesis.

If the culvert has been flowing full and if the tailwater level is lowered below the crown of the culvert at the outlet, the headwater pool level is also lowered, but at a rate which decreases proportionately as the tailwater is lowered. This is represented by the range 3-5 in Figure 7 and it represents a zone of operation which may be quite unstable.

Mavis[8] set up five conditions of flow through a culvert and Schiller[12] has added a sixth, type III, below.

Type I. Part-full with free outfall.

Type II. Part-full with outfall partially submerged.

Type III. Part-full with outfall completely submerged.

Type IV. Full with outfall completely submerged.

Type V. Full with outfall partially submerged.

Type VI. Full with free outfall.

Of these conditions, types I, II, and IV are stable and of primary practical importance.

The culvert investigated in this thesis problem does not correspond exactly to any of these six types. Flow exists between part-full with partially submerged outfall and part-full with completely submerged outfall.

It was therefore deemed necessary to seek another approach. Some of the assumptions made were:

1. Friction loss in the culvert is negligible because it is short.

2. Tailwater is above critical depth but not excessively so, i. e. not in the range 3-4 in Figure

3. Flow exists in the transition range between free outfall and full downstream channel control.

With these conditions set it can be assumed that the culvert acts as a partially submerged hydraulic structure. Previous work with flow over or through such structures has been confined to weirs, but one investigation that seemed to offer some promise was that of Villemonte.[14]

Experimenting with submerged weir plates, Villemonte developed a general discharge equation for submerged weirs. He assumed that if the net flow over a weir is the free-flow discharge Q_1 due to head h_1 minus the free-flow discharge Q_2 due to head h_2 (Figure 8), then the net flow, $Q = Q_1 - Q_2$.

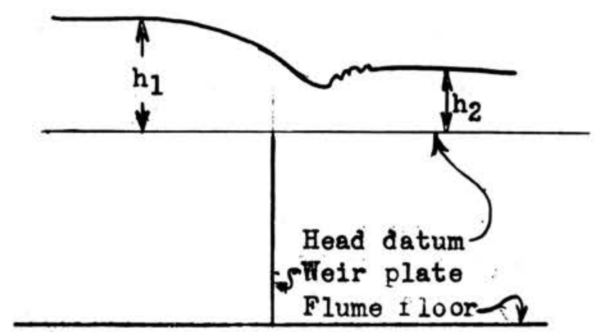


Figure 8

The foregoing is an application of the counterflow theory of flow and implies that the head, h_2, does not directly affect the flow of water due to h_1 and likewise that head, h_1, does not prohibit counterflow due to h_2. Its use is thus equivalent to an application of the principle of superposition which is frequently used in evaluating the combined effect of several independent conditions. Experimental tests by Villemonte showed that in the equation

$$\frac{Q}{Q_1} = 1 - \frac{Q_2}{Q_1},$$

$\frac{Q}{Q_1}$ is related functionally to $1 - \frac{Q_2}{Q_1}$ but not linearly. Results showed that this relationship may be expressed in the form

$$\frac{Q}{Q_1} = f\left(1 - \frac{Q_2}{Q_1}\right) = k\left(1 - \frac{Q_2}{Q_1}\right)^m.$$

Since the general discharge formula for a weir is $Q = Ch^n$

$$\frac{Q}{Q_1} = k\left(1 - \frac{C_2 h_2^{n_2}}{C_1 h_1^{n_1}}\right)^m.$$

For any given type of weir the coefficients, C_1 and C_2, and the exponents, n_1 and n_2, should be equal. The resulting equation is then

$$\frac{Q}{Q_1} = k(1 - s^n)^m,$$

where s = submergence = $\frac{h_2}{h_1}$.

Villemonte found after testing seven types of weirs that k was equal to 1.00 and m equalled 0.385 for a practical submergence range of 0.00 to 0.90. For application to the partially submerged culvert described herein, k and m were to be evaluated by an analysis of the experimental data.

Hydraulic Similitude

To be similar hydraulically to its prototype, a hydraulic model must satisfy four laws of similitude governing the various forces acting on it. These laws - developed by the men for whom they are named - are defined by the Froude number which relates gravitational forces, the Reynolds number which relates viscosity, the Weber number which relates surface tension and the Cauchy number relating elasticity. To be hydraulically similar both model and prototype must have the same Froude number, the same Reynolds number, etc.

For general model work with orifices, certain channels, and flow over weirs, the force of gravity can be assumed to be the only one acting, the others being negligible. Such was the case for this study. Viscosity forces were neglected because the medium used in the model was the same as that in the prototype. Surface tension was deemed similar due to the fact that the models were relatively large, and elasticity was eliminated because water is normally considered inelastic.

The development of gravitational similitude is as follows, where F is the Froude number and the subscripts m and p refer respectively to model and prototype:

$$F = \frac{V}{\sqrt{gL}}$$

$$F_m = F_p$$

$$\frac{V_m}{\sqrt{gL_m}} = \frac{V_p}{\sqrt{gL_p}}$$

$$\frac{V_p}{V_m} = \frac{\sqrt{gL_p}}{\sqrt{gL_m}} = \sqrt{\frac{L_p}{L_m}}$$

Since in dimensional analysis $A = L^2$, in hydraulic similitude the area ratio is equal to the length ratio squared,

$$\frac{A_p}{A_m} = \left(\frac{L_p}{L_m}\right)^2;$$

and since

$$\frac{Q_p}{Q_m} = \frac{A_pV_p}{A_mV_m},$$

$$\frac{Q_p}{Q_m} = \left(\frac{L_p}{L_m}\right)^2\left(\frac{L_p}{L_m}\right)^{\frac{1}{2}} = \left(\frac{L_p}{L_m}\right)^{\frac{5}{2}}.$$

Thus for a 1:48 scale ratio the ratio of prototype discharge to model discharge is

$$\frac{Q_p}{Q_m} = \left(\frac{L_p}{L_m}\right)^{\frac{5}{2}} = \left(\frac{48}{1}\right)^{\frac{5}{2}} = 16{,}000.$$

For a 1:12 scale ratio the discharge ratio is

$$\frac{Q_p}{Q_m} = \left(\frac{L_p}{L_m}\right)^{\frac{5}{2}} = \left(\frac{12}{1}\right)^{\frac{5}{2}} = 500.$$

PROCEDURE

General

The various phases of the investigation were undertaken in the following order:

1. Estimate of probable peak rate of runoff from the watershed.

2. Estimate of tailwater elevation for various flow rates through the prototype.

3. Construction and testing of pilot model for the purpose of determining the practicability of continuing the investigation.

4. Construction of a larger model for a more detailed study if the pilot model proved successful.

Initial Survey

A topographic survey of the area was made early in December, 1952, by Mr. W. O. Ree, other Soil Conservation Service personnel, and the author. A map, Figure 9 , was prepared of the stream channel from 650 feet west or upstream to 900 feet downstream from the culvert entrance. From this map the cross-sectional area, wetted perimeter, and hydraulic radius of the channel were computed for each one foot increment in depth at one hundred foot stations along the centerline of the channel.

An estimation of the water surface profile or backwater curve was next undertaken. A preliminary step, however, was the estimation of the peak rate of runoff.

Figure 9. Topographic map of channel.

Estimate of Peak Flow

Before the maximum expected runoff rate could be estimated, two quantities had to be determined--the watershed area and the rainfall intensity frequency that was to be used.

The watershed is outlined on the aerial photograph, Figure 10, and the area was planimetered to be 2,997 acres. For ease of computations this was rounded off to 3,000 acres. Several farm ponds have been added since the photograph was taken in 1937, but these have been disregarded in estimating peak flow.

Because many conservation structures are designed for rainfall intensity based on a 25 year frequency, that period was chosen for this work.

The peak flow rate was estimated by averaging flow rates determined by studies made at several experiment stations in the Southwest and Central Great Plains.

From the results of studies in the High Plains region of Eastern Colorado and New Mexico,[11] peak flow rate expectancies are 3,040, 3,200, 3,280, and 2,810 cfs, depending on soil conditions.

Use of a quantity called the meander factor was employed at the Central Great Plains Experimental Watershed near Hastings, Nebraska. This factor is a dimensionless figure expressed as the ratio of the length of the meandering channel divided by the length of the floodplain. When greater than 1.00 it tends to reduce the peak rates of runoff because of the additional channel storage and the increased time of concentration. Assuming a meander factor of 1.74 (which from the published data was common for the larger watersheds), the peak flow rate was 680

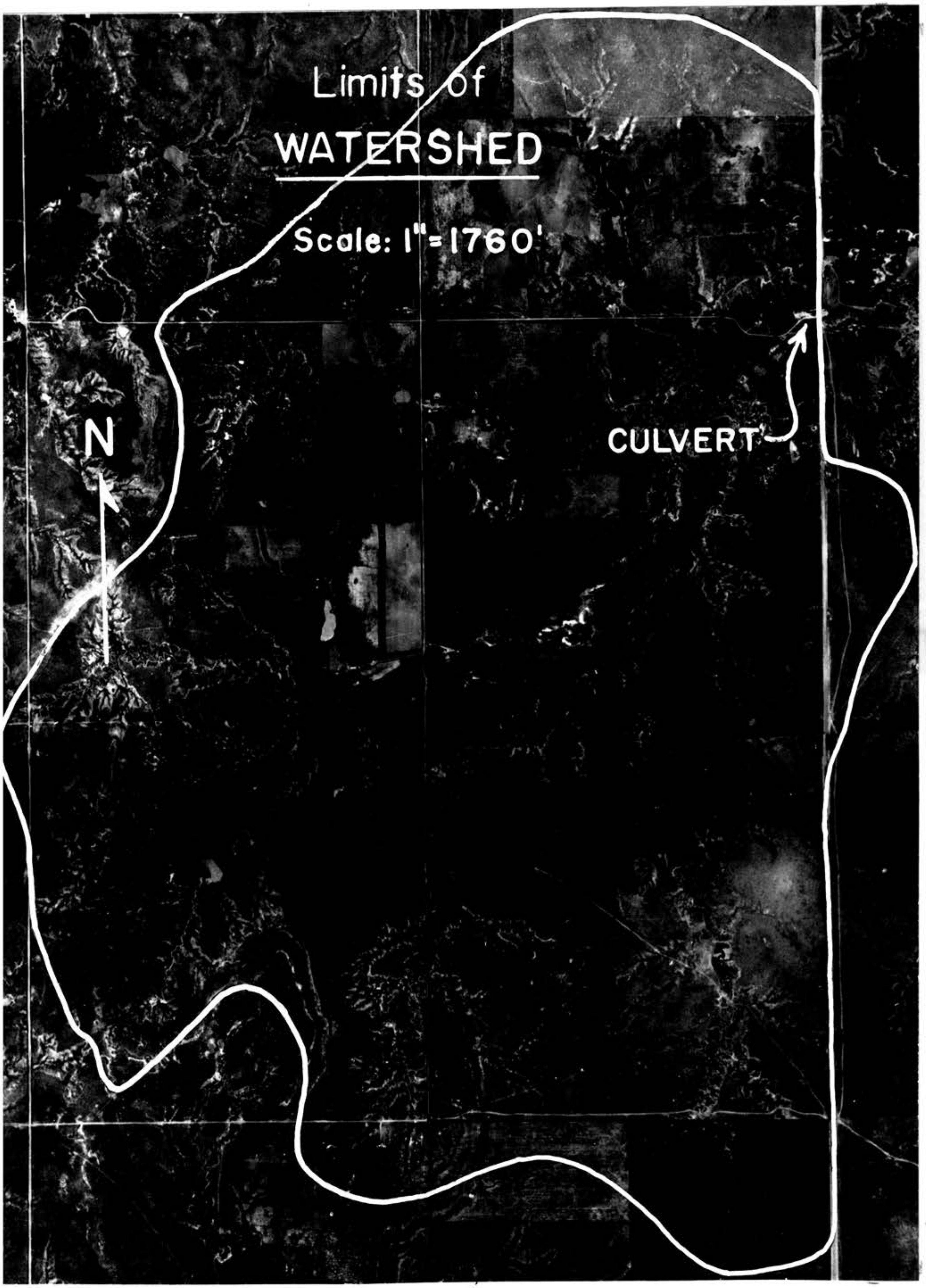


Figure 10.

cfs. However, with a meander factor of 1.00 the curve showed a peak rate of 2000 cfs. Because of uncertainty concerning the meander factor, these data were not weighed heavily.

From data obtained at the Blacklands Experimental Watershed near Waco, Texas, peak rate was estimated at 2944 cfs. This estimate was deemed very close to that which could be expected from the watershed described in this thesis, because of very close similarity of soil texture, rainfall intensities, vegetation and topography.

All the estimates except those from the Central Great Plains Experimental Watershed showed close proximity to 3000 cfs, so that value was assumed as the peak flow rate to be expected at the culvert.

Estimate of Tailwater Elevation

Two independent methods were used for calculating the backwater curve: the Standard Step method and Leach's method, both outlined in King's Handbook of Hydraulics[5].

Both methods involved the use of Manning's roughness coefficient, n. Values of this coefficient were obtained by first estimating Kutter's n by comparing the channel with photographs of other channels of known n values in a U. S. D. A. bulletin by Ramser[10], and then converting to Manning's n with the aid of special curves.

The reach of channel and the photograph or plate number used from the bulletin for the different stages of flow are shown below: n_m is Manning's n used in calculations.

Reach	n_m	Photograph	Stage
8+17 - 7+00 E	0.031	Pl. 4 B Pl. 2 B Pl. 22 B	Medium Low High
7+00 - 6+00 E	0.035	About the same as above except low stage slightly smoother.	
6+00 - 3+25.5 E	0.035	Same as 7+00 - 6+00.	
3+25.5 - 0+38 E	0.028	Pl. 13 A	Low
0+00 - 3+00 W	0.028	Pl. 3 B	
3+00 - 6+00 W	0.035	Same as 7+00 to 6+00.	

Standard Step Method

Backwater curve is the term applied to the profile of the water surface above a dam or other obstruction. In the channel below the culvert the "obstruction" consisted of the constriction at Sta. 3+25 and the fact that the stream bed has very little grade.

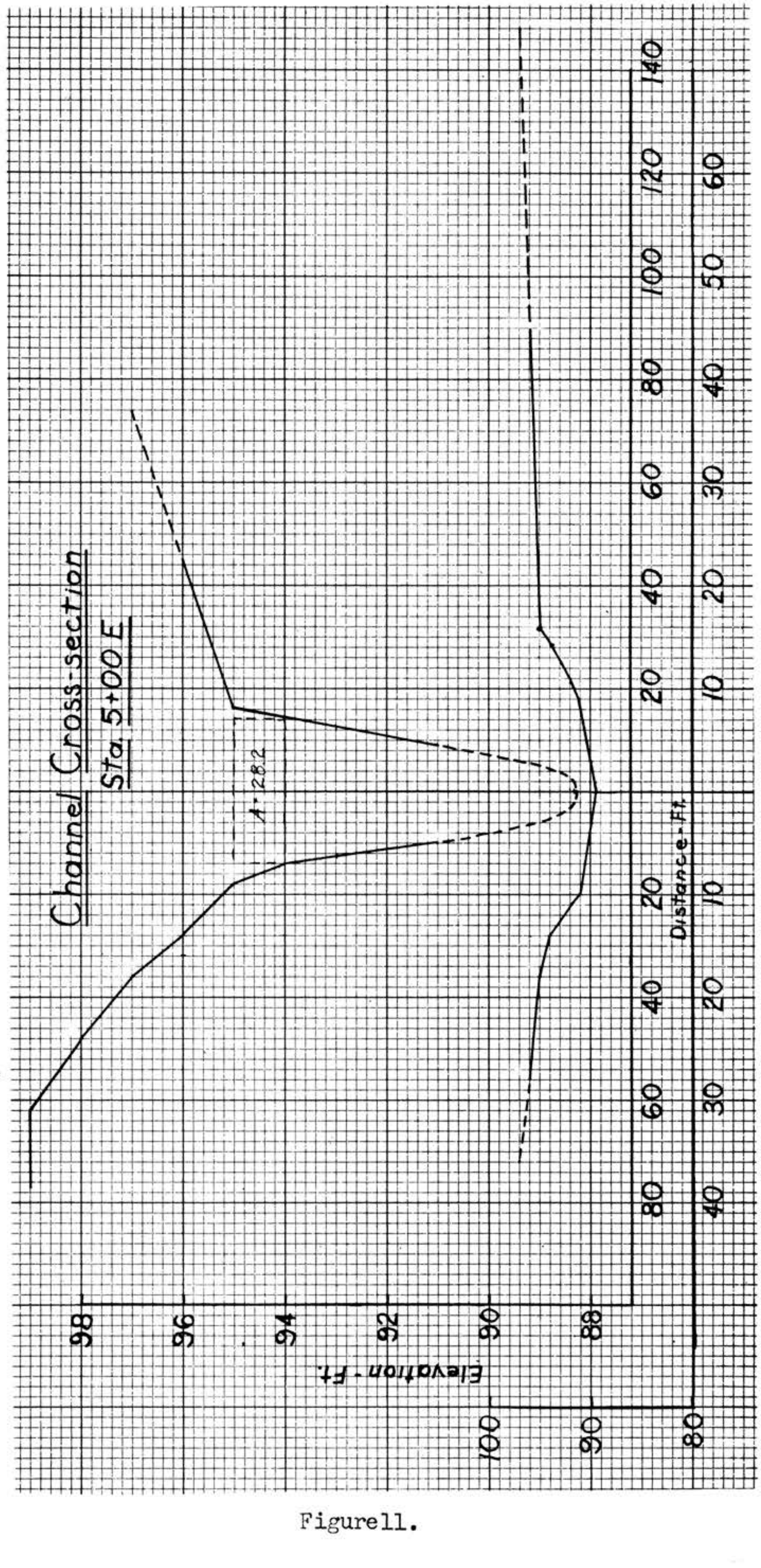
Summarily the backwater problem entails starting with a given discharge and an assumed elevation of water surface and computing the slope of successive reaches upstream. From these slopes, elevations of water surface at the heads of the reaches were determined.

Representative channel cross-sections of the three general types of channel encountered in this study are shown in figures 11, 12, and 13.

Data necessary for calculation of the backwater curve by the Standard Step method is as follows:

1. The discharge for which the profile is desired.

2. A water surface elevation at the downstream end of the desired profile if the depth of flow is greater than critical, or at the upstream end if depth is less than critical.


Channel Cross-section
Sta. 5+00 E
A 28.2
Elevation - Ft.
Distance - Ft.

Figurell.

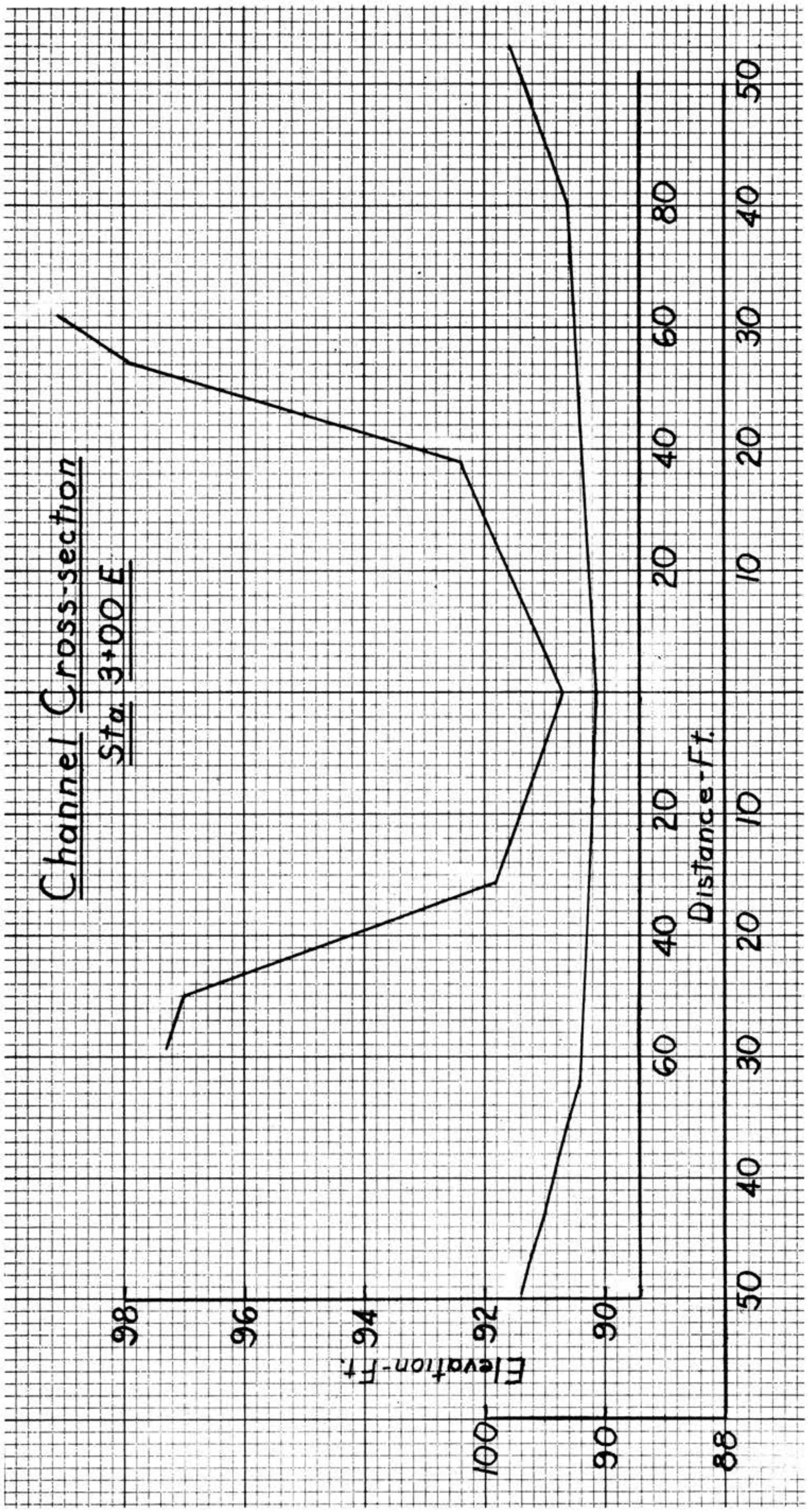

Channel Cross-section
Sta. 3+00 E
Distance - Ft.
Elevation - Ft.
98
96
94
92
90
100
90
88
50
40
30
20
10
10
20
30
40
50
60
40
20
20
40
60
80

Figure 12.

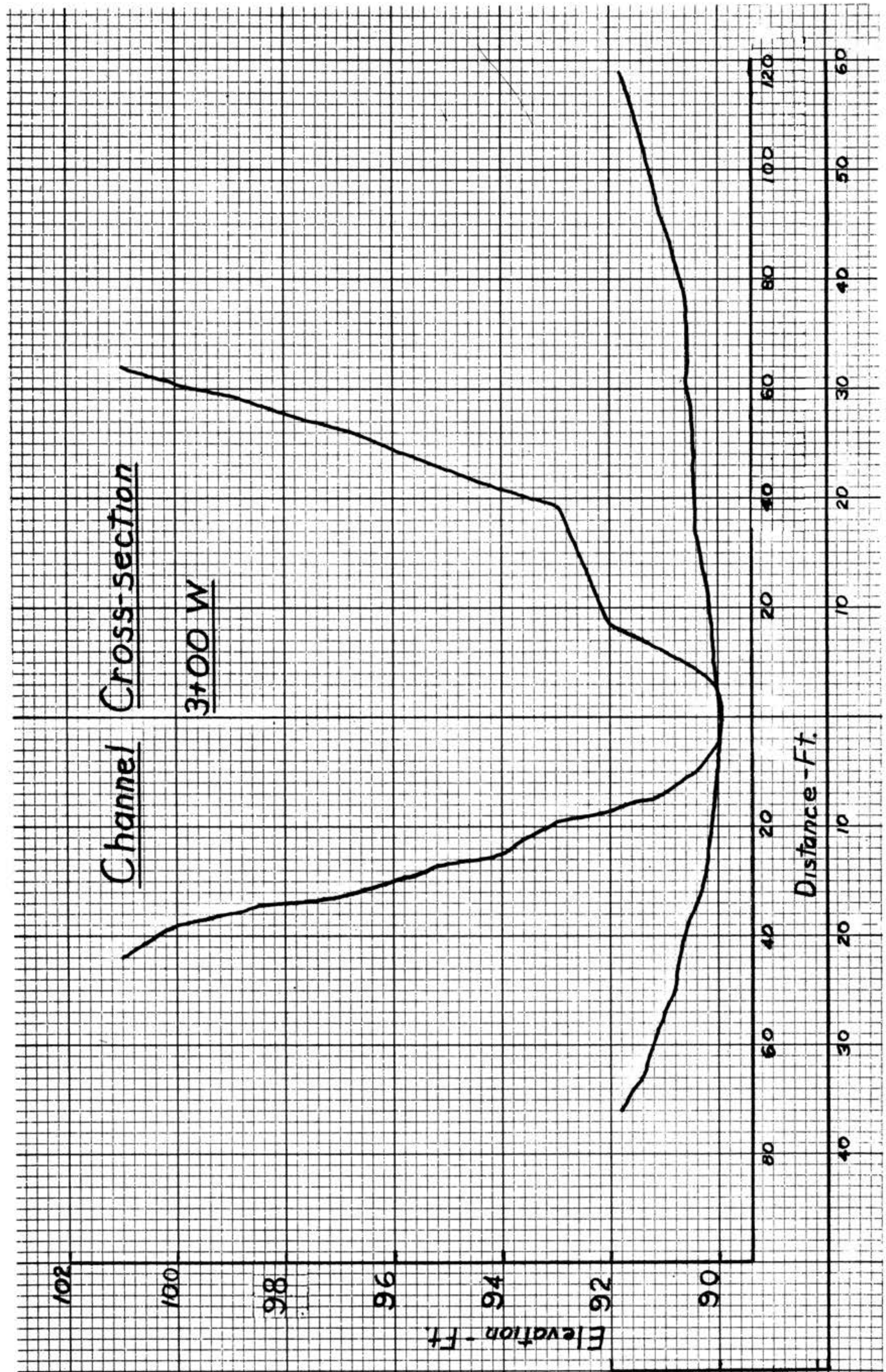

Channel Cross-Section
3+00 W
Distance - Ft.
Elevation - Ft.
102
100
98
96
94
92
90
80
20
40
60
80
100
120
10
20
30
40
50
60

Figure 13.

3. The cross-sectional area and hydraulic radius at increments along the channel, for all depths of flow within the range expected.

4. The hydraulic roughness of the various sections of channel.

The use of the Standard Step Method involved a trial and error procedure. In the chapter on development of theory the depth of flow in the channel was assumed to be greater than critical depth, so the water surface elevation was estimated at the downstream end of the desired profile.

A flow of 1,000 cfs was chosen for the first computation, and the water surface at Sta. 8+00 E was assumed to be at elevation 94.0 A sample calculation is described below. Complete computations are given in the appendix on page 64.

To check the accuracy of the assumed elevation, the elevation of the energy line was determined by two independent methods. If both methods did not yield the same elevation a new water surface elevation was assumed and the energy lines recalculated. Both energy line calculations were required to result in the same elevation before the assumed water surface elevation could be taken as correct.

The velocity head at Sta. 8+00 E was calculated to be 1.30 which when added to the assumed water surface elevation of 94.0 gave 95.3 as an estimate of energy line elevation at that point. The next step was application of the Manning formula to compute the slope of the water surface or friction slope at Sta. 8+00 E. This resulted in $s_f = 0.00710$. As there was no previous reach with which to average the water surface slope calculations proceeded to the next station.

At Sta. 7+00 E the water surface elevation was estimated at 94.71 by assuming that the same friction slope existed throughout the reach, 8 to 7. Addition of the velocity head of 0.52 yielded the first energy line elevation estimate of 95.23. The energy line elevation was then obtained as for Sta. 7+00 by means of the Manning formula and average friction slope for the reach. By averaging the slope at 8+00 and that at 7+00, 0.00487 was obtained as the average friction slope for the reach. The total friction loss for the reach was (100)(0.00487) = 0.49 which when added to the preceding initial energy line estimate of 95.30 gave 95.79. In this case the 95.79 did not correspond to the 95.23 obtained by use of the velocity head. Therefore the water surface elevation was not correct and was re-estimated as 95.50. Completion of the above computation resulted in a smaller difference between energy line elevations but still not equality. One more trial was necessary before energy line elevations reached equality at 95.80, indicating the assumed water surface elevation of 95.47 to be correct at Sta. 7+00 E.

The Standard Step Method is quite cumbersome and was abandoned in favor of Leach's method after completion of computations for one flow.

Leach's Method

The method of backwater curve determination as originally set forth by Leach is outlined by King in Handbook of Hydraulics.[5] The method provides for the difference in flow characteristics between channel flow and flood plain flow. As high flows below the culvert are divided between these two types of flow, Leach's method was believed to be more accurate as it provides for separate computations for the different parts of the channel. Also it uses average cross-sectional area and hydraulic radius.

In the investigation described herein Leach's analysis was carried through for three flow rates: 500 cfs, 1000 cfs, and 1500 cfs. The latter, however, could not be calculated farther upstream than Sta. 5+00 E because of the high elevation the water surface was reaching. From a study of the backwater curves of figure 14 it was believed that the water surface would be at or very near the top of the culvert for that flow. Attempts to route flows approaching the estimated peak rate of runoff were therefore abandoned.

Leach's backwater curve method is based on the Manning formula and involves the use of a factor, K_d. The discharge of an open channel by the Manning formula is

$$Q = \frac{1.486}{n} AR^{\frac{2}{3}} s^{\frac{1}{2}}$$

or

$$Q = K_d \sqrt{s}$$

in which

$$K_d = \frac{1.486}{n} AR^{\frac{2}{3}}$$

For a constant n, K_d varies only with the stage. The friction slope,

$$s = \left(\frac{Q}{K_d}\right)^2.$$

At stages when the flow was partly over a wide, shallow flood plain, and partly in the main channel separate computations were required for each part of the flow. Determination of values of K_d is given in the appendix on pages 68 - 70. These values were plotted against elevation to enable K_d to be estimated for any elevation.

As in the Standard Step Method, the water surface elevation was assumed and then checked by use of the friction loss in the reach. The method is shown by the computation sheet in the appendix on pages 65 - 67.

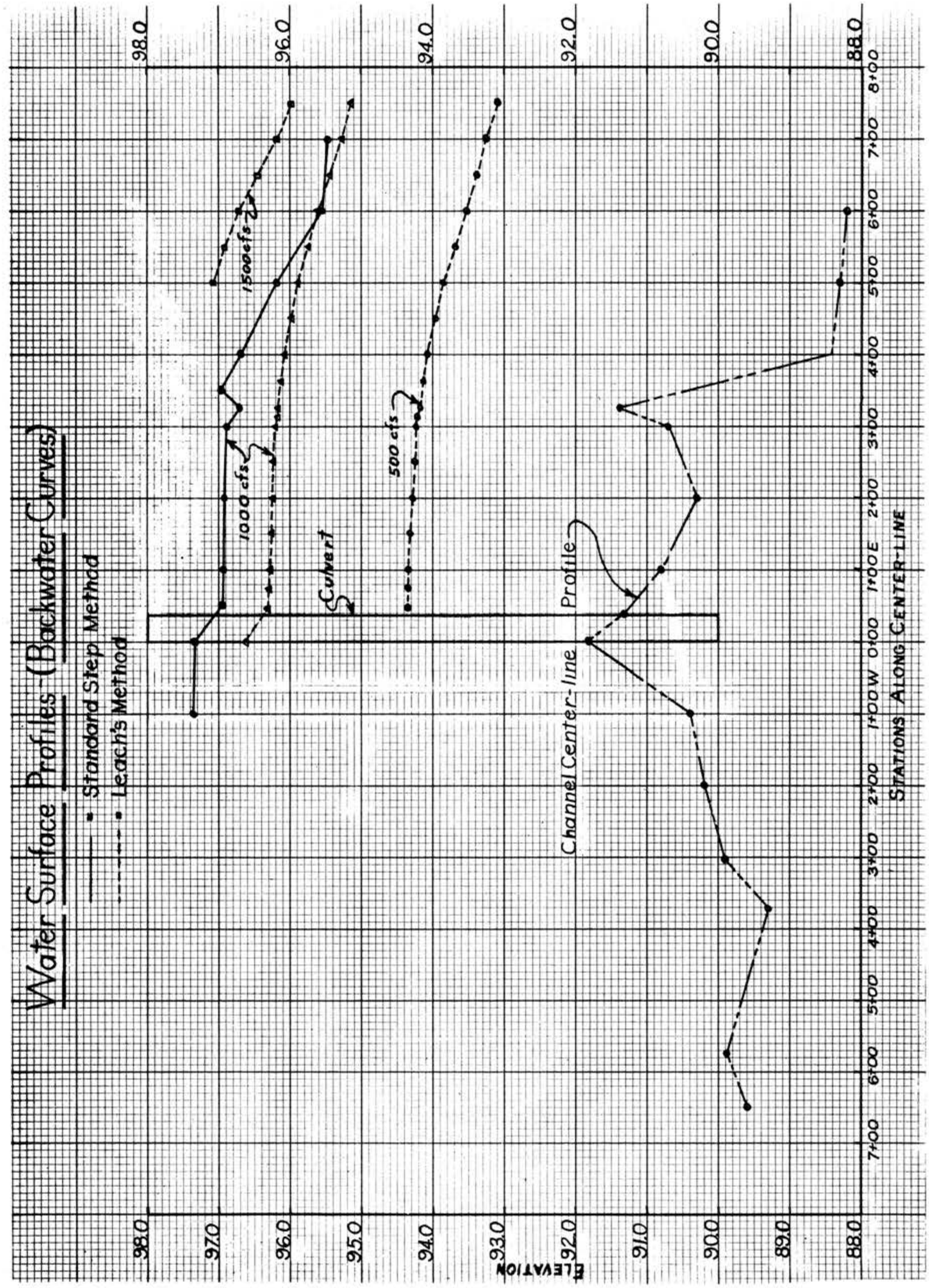

Water Surface Profiles-(Backwater Curves)
Standard Step Method
Leach's Method
1500 cfs
1000 cfs
500 cfs
Culvert
Profile
Channel Center-line
Stations Along Center-line
Elevation
98.0
97.0
96.0
95.0
94.0
93.0
92.0
91.0
90.0
89.0
88.0
7+00
6+00
5+00
4+00
3+00
2+00
1+00 W
0+00
1+00 E
2+00
3+00
4+00
5+00
6+00
7+00
8+00

Figure 14.

The Pilot Model

The plan for model study of the culvert called for construction of two models. The first, the pilot model, built at a scale ratio of 1:48 was for the purpose of determining the practicability of using this culvert as a flow rate measuring device. The second, at a 1:12 scale ratio, was to be used for closely calibrating the prototype.

The pilot model was built of $\frac{1}{4}$ inch plywood; and because the culvert was symetrical, only one-half of it (divided longitudinally) was modeled. The stream-bed was constructed level, and the only attempt to model the stream-channel was to slope the north bank at the prototype angle for a distance of twenty inches upstream. Material for the stream-channel and bank consisted of a sand-cement mortar with the surface roughened with a shortened, stiff "whisk broom".

The culvert outlet was freefall, but a vertically sliding tailgate could be raised or lowered to produce all desired degrees of submergence. The water supply came from a constant head tank through a 0.4 foot H. S. flume and a stilling pool and a rock baffle. The general set-up was inside the laboratory building and is shown by figures 15 and 21 .

Water surface elevation was measured by point gages at two locations - the headwater at a point 3-3/4 inches upstream from the entrance and the tailwater 20 inches downstream from the entrance, ($10\frac{1}{2}$ inches below the outlet).

Results of tests with the pilot model showed that Villemonte's method could probably be applied to develope a rating curve, so plans were made for construction of the full 5-barrel culvert at a 1:12 scale ratio.

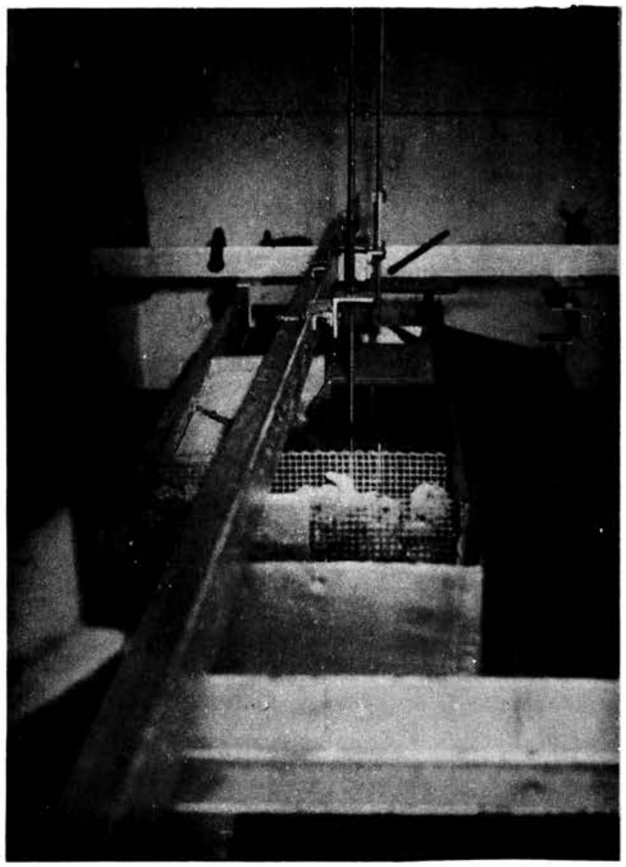


Figure 15. General view of 1:48 scale ratio model of culvert.

The Large Model

The location of the 1:12 scale ratio model was outdoors at about the center of the laboratory area where a sufficient flow of water could be readily controlled without interfering with other laboratory operations.

The culvert was constructed of a sand-cement mortar and was built on a four-inch reinforced slab of concrete 10 feet by 32 feet. The floor of the culvert was one inch thick, and the exit had free outfall. A shelter was built over the entire model area for protection from sun and hail. General model layout is shown by figures 16 and 22.

The inflow was measured by means of a 2 foot H flume. Both the flume and a rock baffle 11 feet in front of the culvert entrance can be seen in figure 19. Tailwater variations were made by means of a hinged gate $9\frac{1}{2}$ feet below the exit and shown in figure 20. A steady flow of approximately six cubic feet per second was turned into the forebay immediately above the H flume. For tests requiring less than six cfs, excess water was diverted down a waste channel controlled by a flap-gate hinged from the channel floor.

Water surface elevations were measured at several locations with the hope of finding a relationship between discharge and water surface elevation at one or more of these locations.

Point gages in tubular glass wells connected to piezometer openings in the channel floor were used to measure water surface elevation at two headwater and two tailwater locations. Six piezometer openings on the centerline of the culvert showed the profile inside the center barrel. These piezometer openings were flush with the culvert floor and were connected to a manometer board which, for easier reading, was sloped at a

Figure 16. General view of 1:12 scale ratio model of culvert.

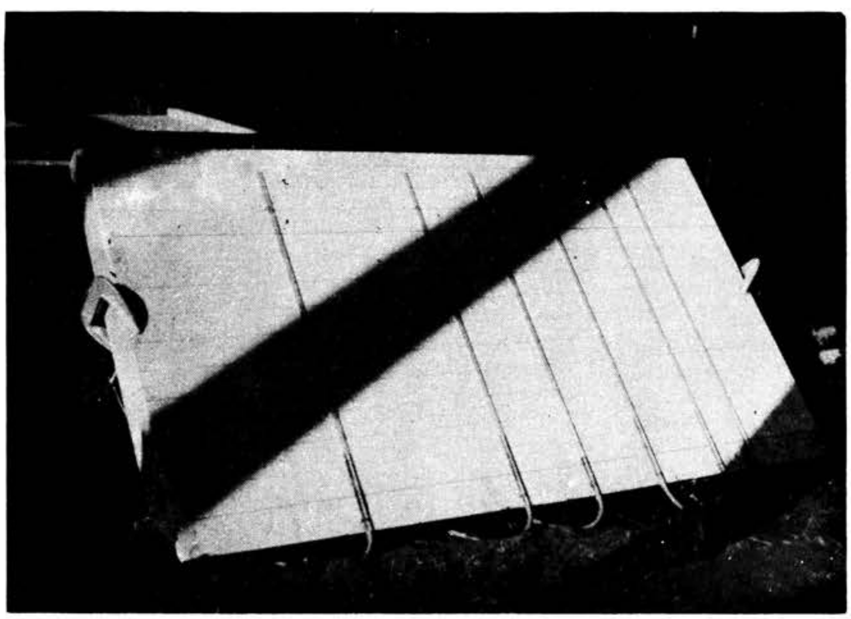


Figure 17. Manometer board for piezometer openings in center barrel of 1:12 scale ratio culvert model.

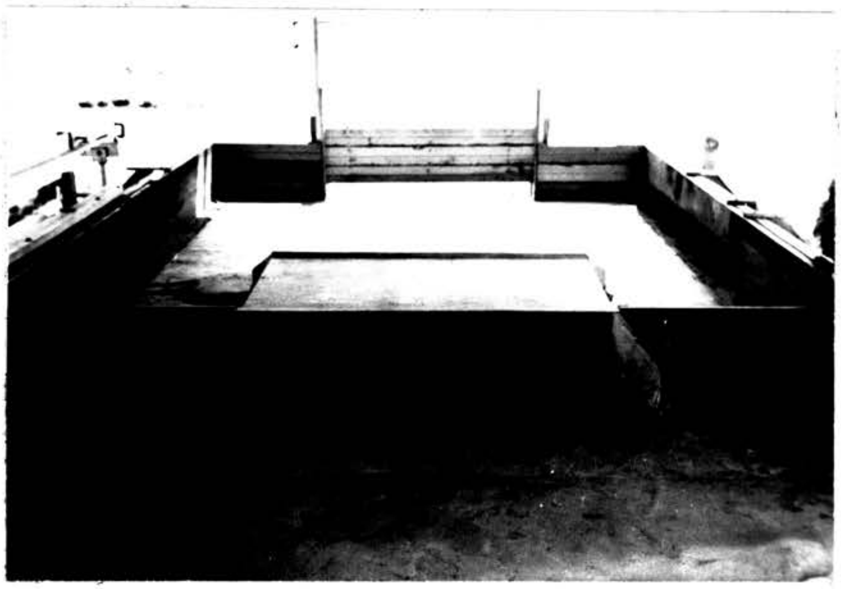


Figure 18. Entrance of 1:12 scale ratio model. Note the highway shoulder modeling.

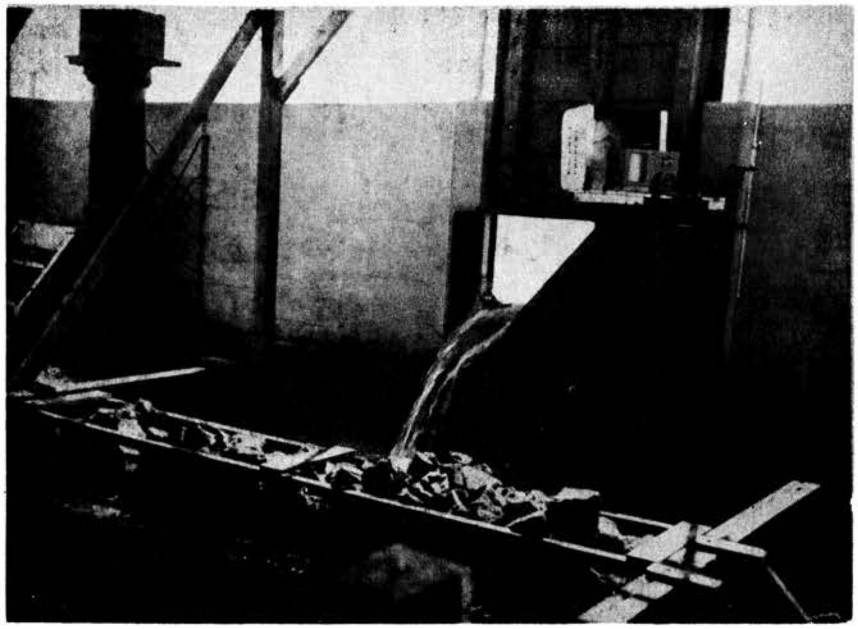


Figure 19. Rock baffle and 2 foot H flume located at upper end of 1:12 scale ratio model basin.

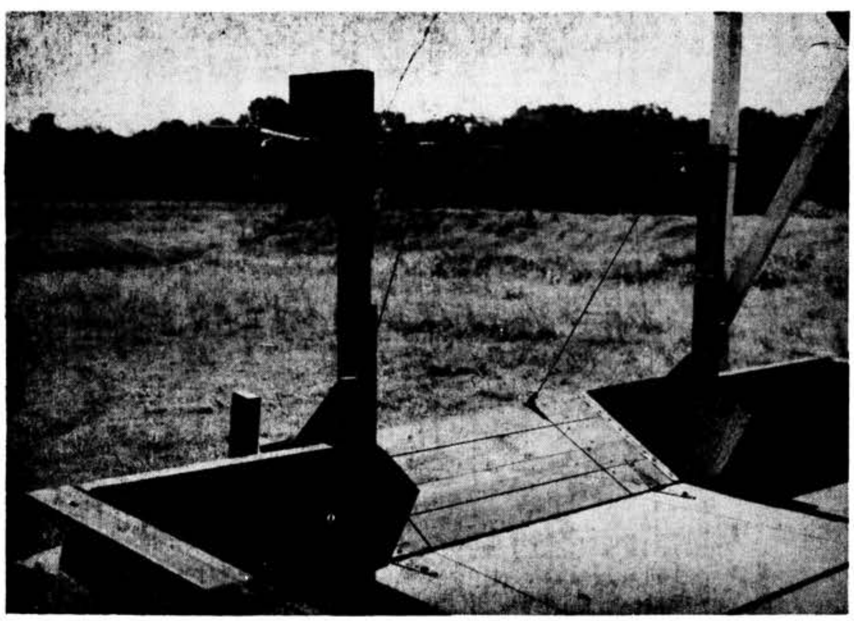


Figure 20 Tailgate located at foot of 1:12 scale ratio model basin.

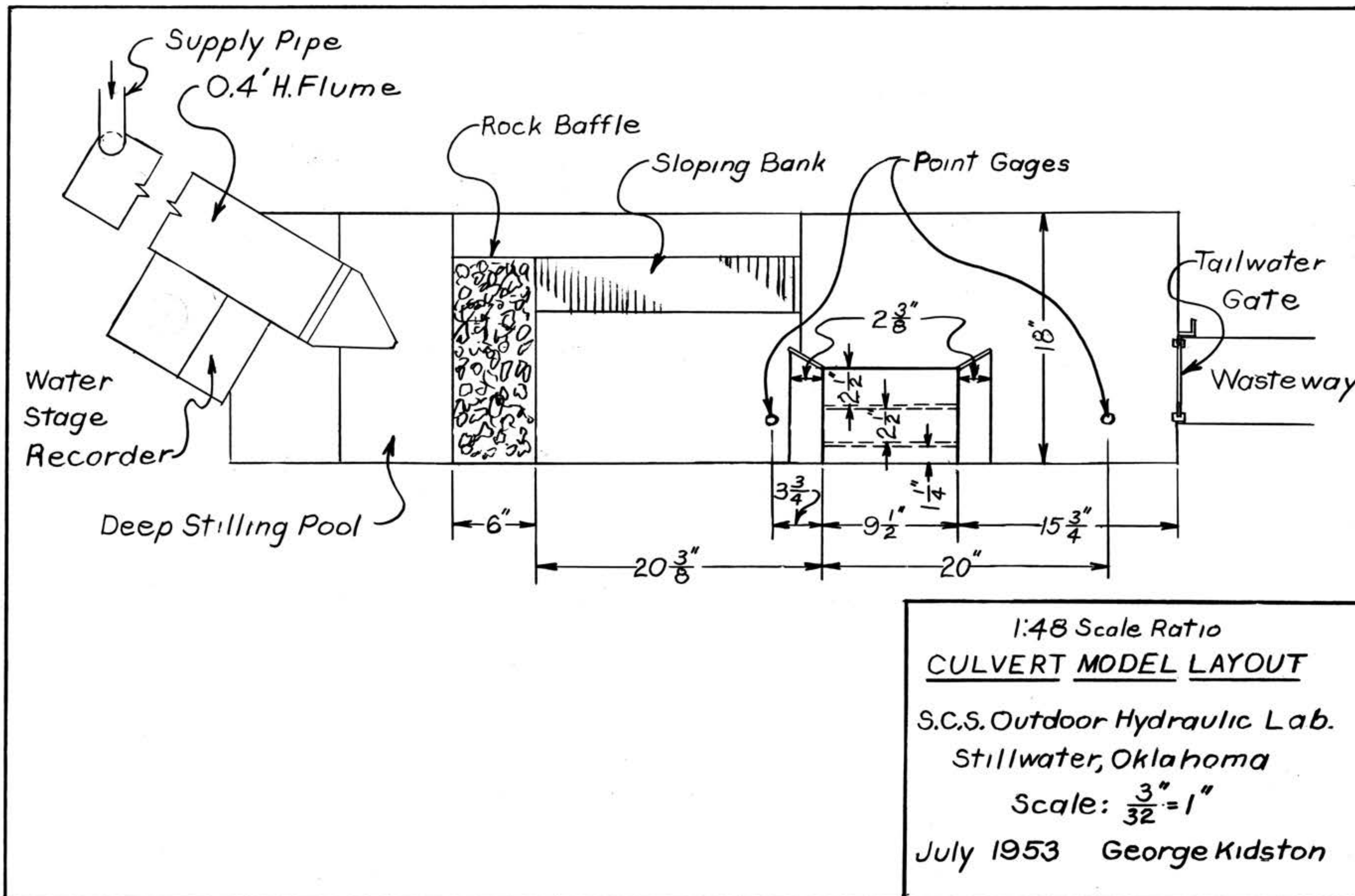
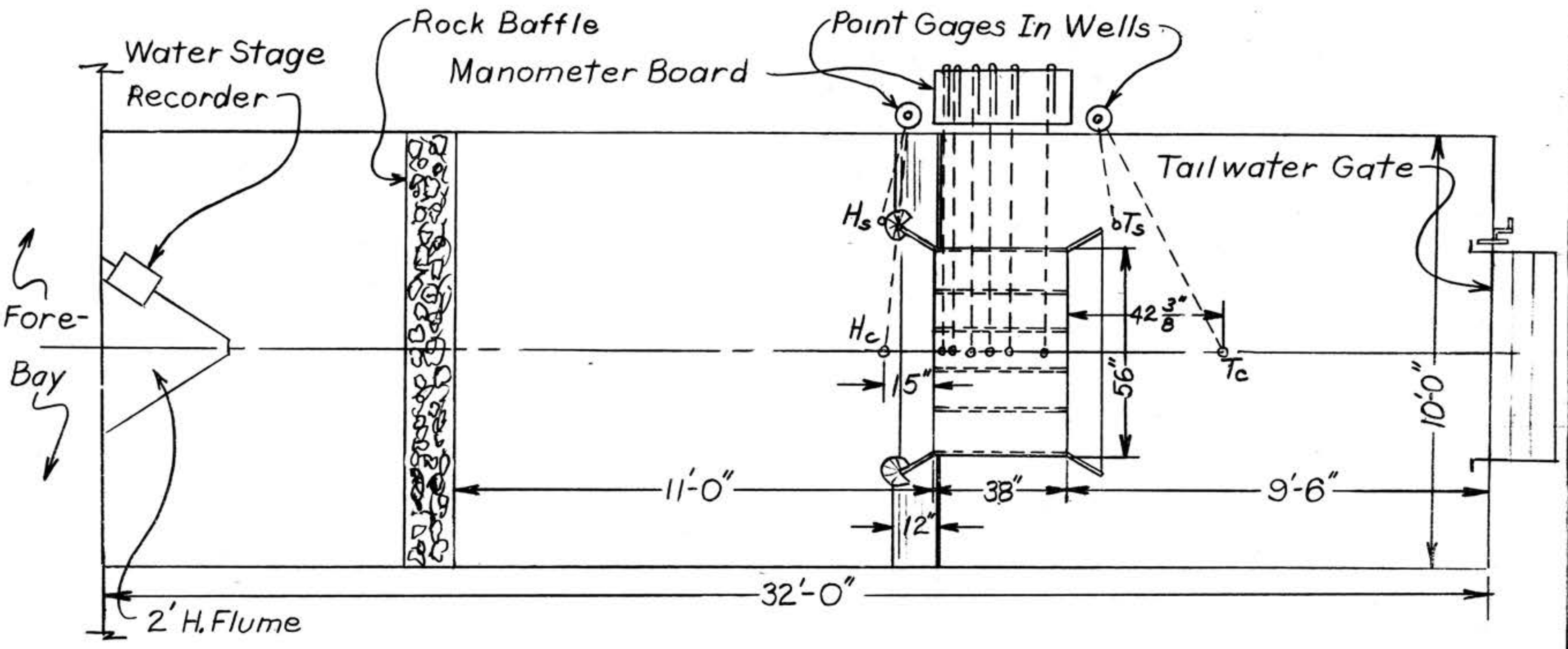

Supply Pipe
0.4' H. Flume
Rock Baffle
Sloping Bank
Point Gages
Tailwater Gate
Wasteway
Water Stage Recorder
Deep Stilling Pool
6"
20 3/8"
3 3/4
9 1/2"
15 3/4
20"
2 3/8"
1 1/2"
2 1/2"
1 1/4"
18"
1:48 Scale Ratio
CULVERT MODEL LAYOUT
S.C.S. Outdoor Hydraulic Lab.
Stillwater, Oklahoma
Scale: 3/32" = 1"
July 1953 George Kidston

Figure 21

Figure 22



Key

H_s = Headwater at Side

H_c = Headwater on Centerline

T_s = Tailwater at side

T_c = Tailwater on Centerline

1:12 Scale Ratio

CULVERT MODEL LAYOUT

S.C.S. Outdoor Hydraulic Lab.
Stillwater, Oklahoma

Scale: 1" = 12"

July 1953 George Kidston

45 degree angle. It is shown in figure 17.

The headwater point gage could be used to measure headwater either 15 inches in front of the entrance on the centerline or 3-5/8 inches off the end of the north-west wingwall, depending on which piezometer opening was connected to the point gage well. The tailwater point gage was used to measure tailwater either 42-3/8 inches below the exit on the centerline or 4-1/8 inches off the end of the north-east wingwall. These headwater and tailwater piezometer openings were approximately one inch above the floor of the approach and exit channels, thus making them on a level with the culvert floor. Mortar was used to slope the floor up gently to these openings.

The relative location of the ten piezometer openings is shown by figure 22.

Tests

The primary objective of the pilot model tests was to determine whether or not it would be advisable to proceed with the plan for using the culvert as a flow measuring device. Consequently Villemonte's method of analysis was to be applied to data from the pilot model tests. To enable this analysis, a curve showing the relationship of head-on-entrance to free outfall discharge was necessary.

The first series of tests, therefore, was run with the culvert flowing with free outfall. Seven discharges representing prototype flows varying from 187 to 2410 cubic feet per second were run.

Subsequent tests were made by maintaining a constant discharge and varying the tailwater. Values of submergence ranged from 0.338 to 0.994. Headwater-tailwater curves for the various discharges are shown by figure 23.

Tests with the large model were essentially the same as for the pilot model. Thirteen free outfall tests were run with discharges representing prototype flows of from 170 to 3400 cfs. The head-discharge relationship is shown in figure 24. Flow was then left constant and tailwater varied for six flows with prototype discharge varying from 400 to 3160 cfs. Submergence ranged from 0.304 to 0.992.

HEADWATER-TAILWATER CURVES

C-6-A EXP. I

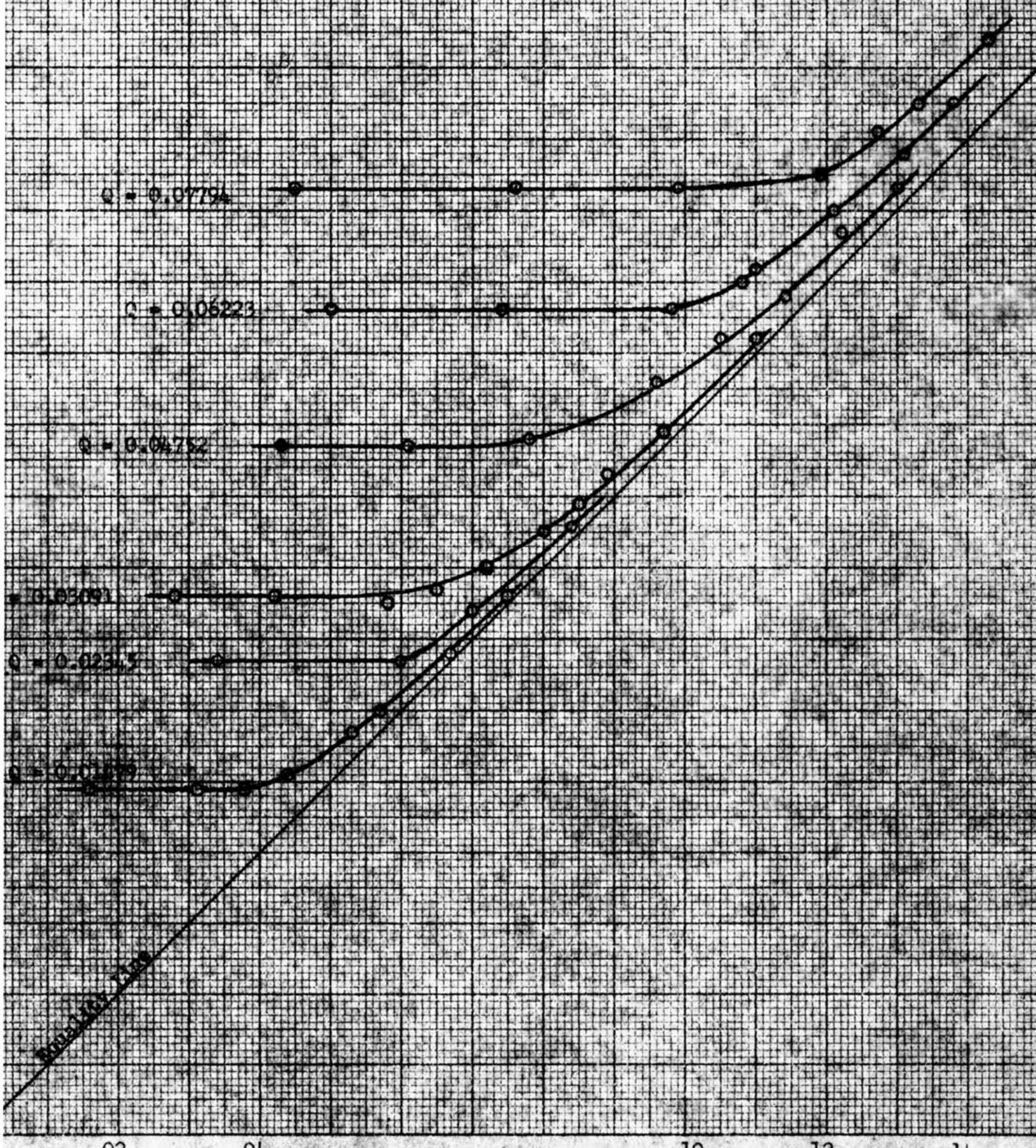


Figure 23

RESULTS AND ANALYSIS

A comparison of pilot model results with backwater curves showed the possibility of using the culvert as a measuring device.

Headwater vs tailwater curves were drawn for the pilot model and are shown by figure 23. The lower, horizontal part of each curve represents a flow condition with tailwater depth below critical depth, i. e. free outfall. The curved portion of each line represents a transition zone where an increase in tailwater results in a similar but somewhat smaller increase in headwater. The upper parts of the curves are very nearly parallel with the 45 degree "equality line", indicating that an increase in tailwater will result in an equal increase in headwater.

To determine on which part of the curve actual prototype conditions would exist, a flow of 1000 cfs was chosen for analysis, and from Leach's backwater curve tailwater depth was found to be approximately 4.5 feet relative to the elevation of the silt at the culvert entrance. The equivalent head on the model was then 4.5 divided by the model scale ratio of 48 or 0.0938 feet. Equivalent discharge was 1000 divided by 32,000 or 0.03125. Entering the pilot model headwater-tailwater curves at a tailwater of 0.0938 and rising to the estimated intersection with the Q = 0.03125 curve show that that particular flow condition is at the upper extremity of the transition zone and may or may not be useful.

Similar calculations for a flow of 500 cfs resulted in a point fully within the transition zone, so on the basis of this analysis the investigation was continued. If prototype flow had been found to exist in the upper zone where difference between headwater and tailwater elevation are no indication of discharge rate, the investigation would have been discontinued.

The head-discharge relationship of a culvert for the condition of free outfall plots as a straight line on log-log paper as shown in figure 24. The equation of any straight line on log-log paper is of the form $y = C \cdot x^n$ where n is the slope of the line, C is a constant numerically equal to y when x = 1, and x is the independent variable.

For the free outfall head-discharge curve, h is the independent variable and Q the dependent variable. n was found to be 1.5, and C for the large model was 13.6, i. e. the value of Q from the curve when h = 1.00. For application to the protoype the equation for the model was multiplied by the scale ratio of 12, resulting in

$$Q_1 = 163.2 \cdot h^{1.5} \qquad (1)$$

as the free outfall equation for the culvert.

As originally stated in the chapter on theory, Villemonte's method of analysis provides the functional relationship between the two ratios - submerged discharge to free outfall discharge and tailwater head to headwater head:

$$\frac{Q}{Q_1} = k(1-s^n)^m \qquad (2)$$

The determination of values of k, n, and m will now be undertaken.

The plotting of the functions $\frac{Q}{Q_1}$ and $(1-s^n)$ was drawn for the centerline headwater and tailwater piezometer openings as well as for the headwater and tailwater openings just off the ends of the wingwalls. The curves are shown by figures 25 and 26. The scatter of points was less

Head-Discharge Relationship

C-6-B
Free Outfall

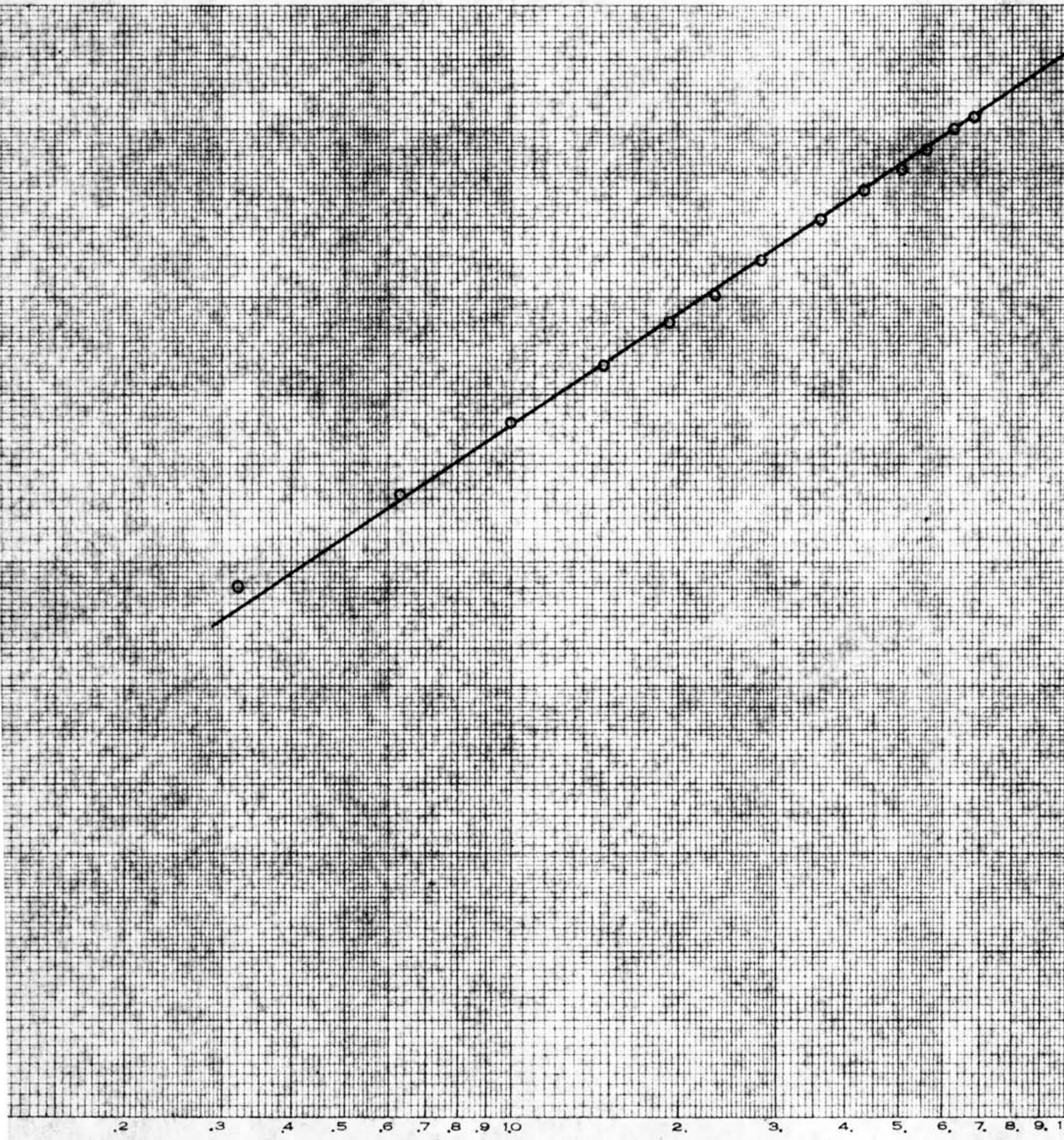


Figure 24

Effective Submergence

On Culvert

C-6-B

Center piezometer openings

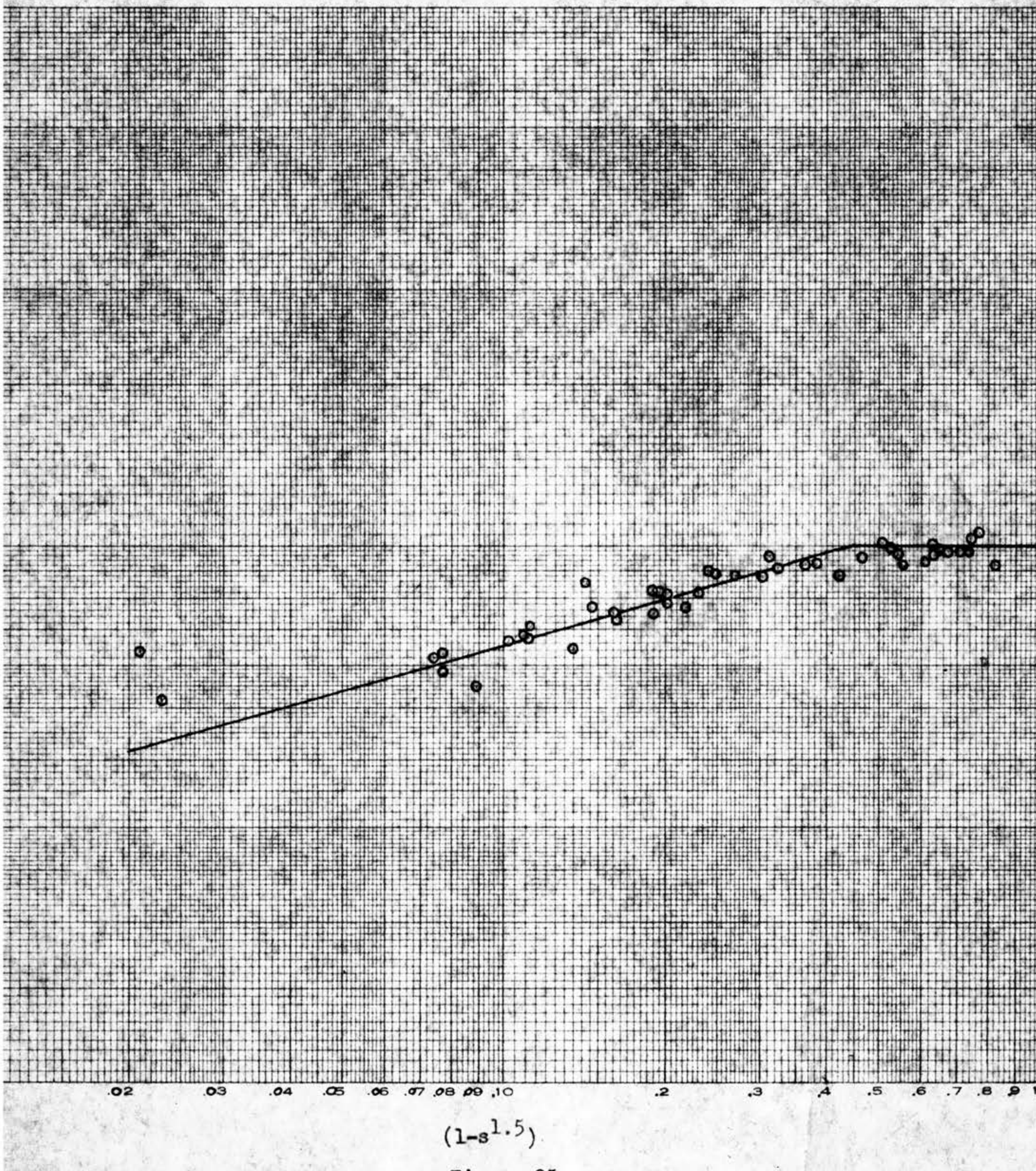
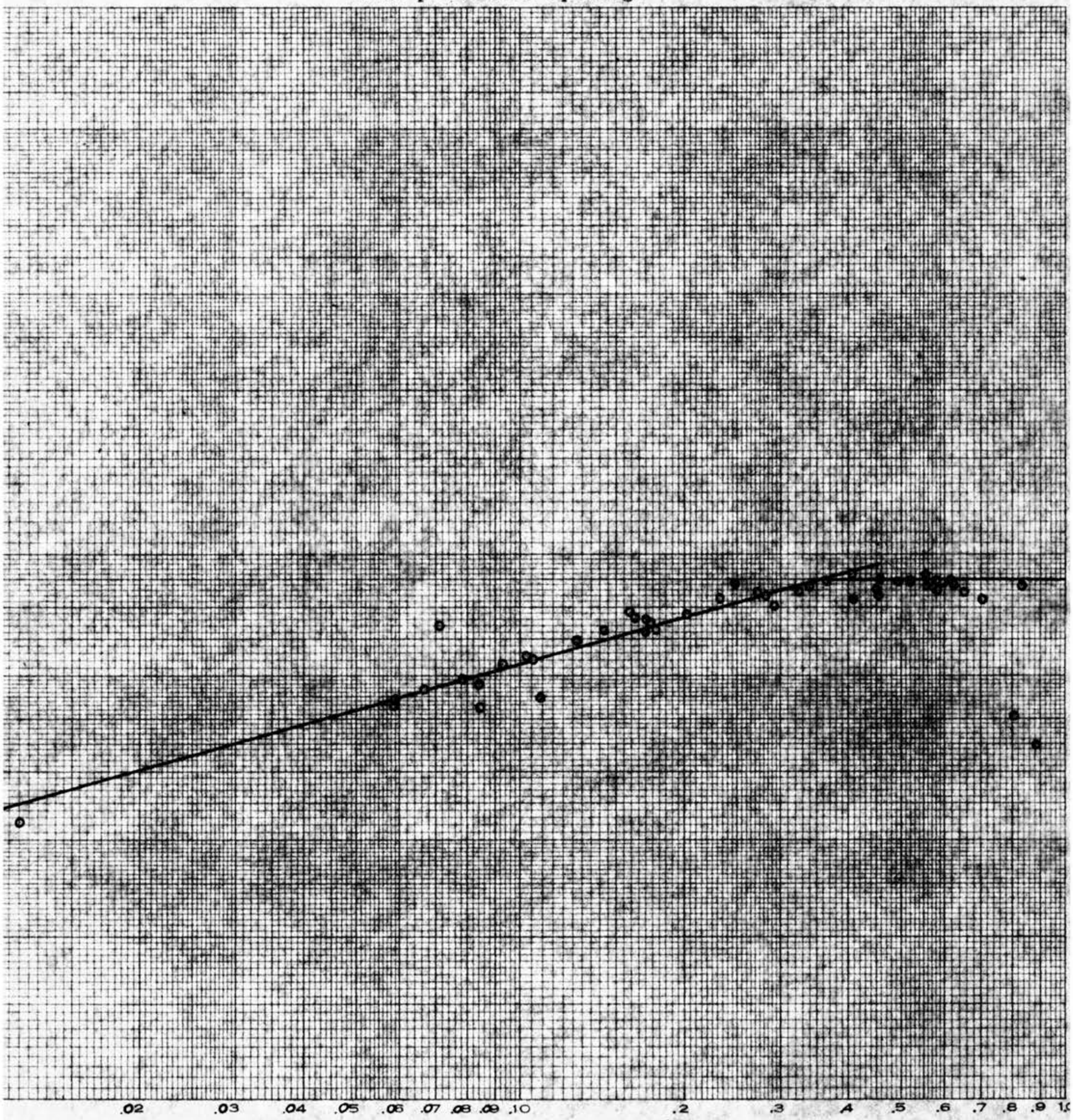


Figure 25

Effective Submergence On Culvert

C-6-B

Side piezometer openings



$(1-s^{1.5})$

Figure 26

pronounced for the side piezometer openings, although the whole curve appeared to be slightly lower than the one plotted with data from the centerline openings. For this reason and because the water stage recorders were to be placed at the side, the more detailed analysis was applied to the side openings.

Two straight lines were drawn through the points of both curves. On the centerline opening plotting, the sloped line intersected the $\frac{Q}{Q_1} = 1.00$ at $(1-s^n) = 0.45$. From that point on, a horizontal line seemed to fit the points. Apparently, the explanation of the horizontal line is that tailwater depth is equal to or less than critical depth and therefore has no submergence effect.

Explanation of the break in the line can be made by referring to the centerline piezometer opening curve of figure 25. Here the break was assumed to occur at $\frac{Q}{Q_1} = 1.00$. The value of $(1-s^{1.5})$ at that point was then calculated from the critical depth relationship shown below to be 0.455, where d_c = critical depth.

$$h_t = d_c$$

$$h_t = 2/3 h_h$$

$$\text{or} \qquad \frac{h_t}{h_h} = 2/3 = s$$

$$\text{Then} \qquad s^n = (2/3)^n = 0.545$$

$$\text{and} \qquad (1-s^n) = 1 - 0.545 = 0.455.$$

The existence of a horizontal portion of the curve also occurred with the side piezometer opening plotting but at a value of $\frac{Q}{Q_1}$ e qual to 0.90, indicating the affect, possibly, of direction change or turbulence of the flow around the wingwalls. Never-the-less the curve was well defined, so derivation of an equation for the curve was undertaken.

The exponent m in Villemonte's equation (2) was found to be 0.276; and k was the value of $\frac{Q}{Q_1}$ when $(1-s^n) = 1$, or 1.20. In the chapter on theory n was shown to be equal to the free outfall exponent which in this case was 1.50.

The general equality is then

$$\frac{Q}{Q_1} = 1.20 \ (1-s^{1.5})^{0.276} \qquad (3)$$

for all values of s greater than 0.667. Then by substituting equation (1) for Q_1 and $\frac{h_t}{h_h}$ for s, the discharge equation for the culvert is obtained as

$$Q = 195.8 \ h_h^{1.5} \left[1-(\frac{h_t}{h_h})^{1.5}\right]^{0.276} \qquad (4)$$

For values of s less than 0.667 when tailwater has no submergence, the relationship between discharge and head-on-entrance is direct and given by equation (1).

Solution of the discharge equation could be accomplished by computing a rating table or by obtaining values directly from the curve with the aid of the free outfall curve or by developing a nomograph. Considering both accuracy and ease of setting up, the latter method would seem most suitable.

Accuracy

For the purpose of giving proper evaluation to the flow rate determinations made with this thesis, an estimate of probable accuracy is necessary.

Two sources of error were considered: (1) inability of the water stage recorders to record water surface elevation more accurately than 0.02 foot, and (2) the possibility of a portion of the silt deposit being washed out without being noticed.

Water stage recorder inaccuracy was determined by selecting an example. For this purpose values of headwater equal to 4.84 and tailwater equal to 4.50 were assumed. It was also assumed that the entrance datum remained stable. Solving the general discharge equation (4) with these values of head gave Q = 1113 cfs as the actual flow rate through the culvert. The inaccuracies of the recorders were then considered to be acting in opposite direction, thus producing the maximum difference in flow. The heads were then h_h= 4.86 and h_t= 4.48. Solving equation (4) showed that the indicated discharge was 1155 cfs or 3.77% greater than the actual flow.

For computation of the inaccuracy which would be introduced if some of the silt were washed out of the barrels, the water level recorders were presumed to be completely accurate, and an entrance datum change of 0.3 foot was assumed.

If the headwater and tailwater recorder readings are assumed to be the same as before, i. e. h_h = 4.84 and h_t = 4.50, actual heads would be 0.3 foot greater than these or h_h = 5.14 and h_t = 4.80. While the indicated flow would remain 1113 cfs, actual flow would become 1198 cfs, or an error of 7.09%.

Therefore, the approximate maximum error in flow rate determinations for the above condition is 3.77 plus 7.09 or 10,86%.

CONCLUSIONS

1. The culvert is adaptable as a flow rate measuring device in spite of the following adverse conditions: (1) The culvert acts as a partially submerged hydraulic structure, (2) It has considerable downstream channel control, and (3) There is an accumulation of trash and silt in the barrels.

2. A counterflow theory of flow developed by Villemonte for flow over submerged and partially submerged weirs is applicable to the culvert and results in a functional relationship between the ratio of tailwater head to headwater head and the ratio of actual discharge to free outfall discharge for the same head. The resulting general discharge equation is

$$Q = 195.8\, h_h^{1.5} \left[1 - \left(\frac{h_t}{h_h}\right)^{1.5}\right]^{0.276} .$$

This method for determining flow rate requires the measurement of headwater and tailwater elevations. A suitable location for the water level recorders is four feet off the ends of the wingwalls.
The datum for water surface elevation must be the average elevation of the silt deposit at the culvert entrance. Determination of the datum should be made after every moderate to heavy flow rate or other incident which might change the silt pattern at the entrance.

3. The curve of figure 26 may be used in conjunction with the free outfall curve of figure 24 to obtain the discharge rate through the culvert. There are also other ways this can be done. They are: (1) The general discharge equation can be solved directly. (2) A rating table could

be developed, and (3) A nomograph could be constructed for direct solution of the discharge equation.

4. For a flow rate of approximately 1000 cubic feet per second the estimated maximum error in flow rate determination is 10.86%. This includes a 3.77% error attributed to the water stage recorders and 7.09% to a possible shifting of the head reference datum.

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APPENDIX

C-6-A, Exp't. 1

st o.	Time	Discharge: .4' H.S. Gage rdg.	Discharge: Zero	Discharge: Head	Discharge: Q	Head Water Gage rdg.	Ent. Zero	Head on Ent.	Water temp.	Tail Water Gage rdg.	Tail Water Head
1	12:59	1.506	1.310	0.204	.01873	2.615	2.562	0.053	13°C		
2	1:26	1.474		0.164	.01169	2.605		0.043			
3	1:37	1.428		0.118	.005871	2.592		0.030			
4	1:57	1.548		0.238	.02629	2.630		0.068			
5	2:05	1.629		0.319	.05071	2.664		0.102			
6	2:14	1.602		0.292	.04151	2.652		0.090			
7	2:29	1.662		0.352	.06346	2.680		0.118			
8	2:38	1.689		0.379	.07518	2.692		0.130			
9	3:30	1.566	1.310	0.256	.03091	2.638	2.562	0.076	15°C	2.590	0.028
0	3:42					2.638		0.076		2.604	0.042
1	3:45					2.639		0.077		2.627	0.065
2	3:48					2.643		0.081		2.634	0.072
3	3:53					2.651		0.089		2.647	0.085
4	3:56					2.661		0.099		2.659	0.097
5	4:00					2.674		0.112		2.672	0.110
6	4:03					2.689		0.127		2.687	0.125
7	4:07					2.697		0.135		2.696	0.134
8	4:11					2.708		0.146		2.707	0.145
9	4:15					2.718		0.156		2.717	0.155
0*	4:19					2.729		0.167		2.728	0.166
~~1~~	~~10:22~~	~~1.494~~	~~1.310~~	~~0.185~~	~~.01515~~	~~2.616~~	~~2.569~~		~~10°C~~	~~2.608~~	
2	10:27	1.493	1.310	0.183	.01479	2.624	2.568	0.056		2.621	0.053
3	10:30	1.493				2.617		0.049		2.599	0.031
4	10:35					2.628		0.060		2.625	0.057
5	10:40					2.619		0.051	12°C	2.612	0.044
6	10:48					2.617		0.049		2.606	0.038
7	10:53					2.636		0.068		2.635	0.067
8	10:57					2.644		0.076		2.643	0.075
9	11:00					2.652		0.084		2.651	0.083
0	??	1.567		0.257	.03118	2.661		0.093		2.657	0.089
1	11:17	1.566		0.256	.03091	2.653		0.085		2.648	0.080
2	11:22					2.648		0.080		2.640	0.072
3	11:27					2.643		0.075		2.626	0.058
4	11:30					2.661		0.093		2.657	0.089
5	11:48	1.620		0.310	.04752	2.666		0.098		2.646	0.078
6	11:52					2.674		0.106		2.664	0.096
7	11:57					2.680		0.112		2.673	0.105

*Water surface touches inside top of west end of culvert.

C-6-A, Exp't. 1

[illegible]	Time	Discharge: .4' H.S. Gage rdg.	Discharge: Zero	Discharge: Head	Discharge: Q	Head Water Gage rdg.	Ent. Zero	Head on Ent.	Water temp.	Tail Water Gage rdg.	Tail Water Head
[illegible]	12:02	1.620	1.310	0.310	.04752	2.686	2.568	0.118	12°C	2.682	0.114
[illegible]	12:06					2.695		0.127		2.690	0.122
[illegible]	12:10					2.701		0.133		2.698	0.130
[illegible]	1:15					2.665		0.097		2.629	0.061
[illegible]	1:18					2.665		0.097		2.611	0.043
[illegible]	1:45	1.493		0.183	.01479	2.617				2.584	0.016
[illegible]	2:00	1.659	1.310	0.349	.06223	2.684	2.568	0.116		2.618	0.050
[illegible]	2:03					2.684		0.116		2.642	0.074
[illegible]	2:05					2.684		0.116		2.666	0.098
[illegible]	2:08					2.688		0.120		2.676	0.108
[illegible]	2:11					2.690		0.122		2.678	0.110
[illegible]	2:14					2.698		0.130		2.689	0.121
[illegible]	2:17					2.706		0.138		2.699	0.131
[illegible]	2:20					2.713		0.145		2.706	0.138
[illegible]	2:22					2.719		0.151		2.713	0.145
[illegible]	2:25					2.729		0.161		2.724	0.155
[illegible]	2:29					2.734		0.166		2.729	0.160
[illegible]	3:06	1.695	1.310	0.385	.07794	2.701		0.133	19°C	2.613	0.045
[illegible]	3:08					2.701		0.133		2.644	0.076
[illegible]	3:11					2.701		0.133		2.657	0.089
[illegible]	3:14					2.701		0.133		2.667	0.099
[illegible]						2.703		0.135		2.687	0.119
[illegible]						2.709		0.141		2.695	0.127
[illegible]*	3:45	1.695	1.310	0.385	.07794	2.713		0.145		2.701	0.133
[illegible]	4:08					2.722		0.154		2.711	0.143
[illegible]	4:11					2.726		0.158		2.717	0.149
[illegible]	4:16					2.734		0.166		2.726	0.158
[illegible]**	4:28					2.741		0.173		2.731	0.163
[illegible]	4:49	1.536	1.310	0.226	.02345	2.635		0.067	21°C	2.602	0.034
[illegible]	4:51					2.632		0.064		2.613	0.045
[illegible]	4:54					2.635		0.067		2.628	0.060
[illegible]	4:57					2.642		0.074		2.638	0.070
[illegible]	5:00					2.654		0.086		2.652	0.084
[illegible]	5:05					2.664		0.096		2.662	0.094
[illegible]	5:08					2.675		0.107		2.673	0.105
[illegible]	5:11					2.688		0.120		2.687	0.119
[illegible]	5:14					2.699		0.131		2.698	0.130

*Pump stopped--flow reset.
**W. S. touching culvert top.

C-6 CHANNEL

lev.	Increment of Depth Δd	Width	Increment of Area ΔA	Accumulated Area A	Wetted Perimeter w.p.	Hydraulic Radius R
			Sta. 8+00 E			
3.2						
			35.33*			
1.0		18.5		35.33	19.4	1.821
	1		20.5			
2.0		22.5		55.83	23.8	2.346
	1		24.75			
3.0		27.0		80.58	28.6	2.817
	1		28.5			
4.0		30.0		109.08	32.0	3.409
	1		44.0			
5.0		58.0		153.08	60.6	2.526
	1		80.5			
3.0**		103.0		233.58	105.6	2.212
			Sta. 7+00 E			
3.2						
			55.25*			
1.0		26.5		55.25	27.6	2.002
	1		27.75			
2.0		29.0		83.00	30.6	2.712
	1		30.0			
3.0		31.0		113.00	33.3	3.393
	1		32.5			
4.0		34.0		145.50	36.9	3.943
	1		43.5			
5.0		53.0		189.00	56.4	3.351
	1		77.5			
3.0		102.0		266.50	105.4	2.528
	1		127.0			
7.0**		152.0		393.5	155.4	2.532

*Determined by planimeter.
**Survey not carried that high.

C-6 CHANNEL

Elev.	Δd	Width	ΔA	A	w.p.	R
			Sta. 6+00 E			
88.2						
			37.27*			
91.0		18.0		37.27	18.8	1.982
	1		19.0			
92.0		20.0		56.27	21.8	2.581
	1		22.0			
93.0		24.0		78.27	26.0	3.010
	1		25.5			
94.0		27.0		103.77	29.7	3.494
	1		31.0			
95.0		35.0		134.77	37.8	3.565
	1		56.5			
96.0		78.0		191.27	81.0	2.361
	1		110.0			
97.0**		142.0		301.27	145.0	2.078
			Sta. 5+00 E			
88.3						
			34.70*			
91.0		19.0		34.70	19.3	1.798
	1		20.5			
92.0		22.0		55.20	23.0	2.400
	1		23.5			
93.0		25.0		78.70	26.6	2.959
	1		26.5			
94.0		28.0		105.20	30.4	3.461
	1		31.0			
95.0		34.0		136.20	36.2	3.762
	1		53.5			
96.0		73.0		189.70	75.4	2.516
	1		91.5			
97.0**		110.0		281.20	112.5	2.500

*Determined by planimeter.
**Survey not carried to 97 on that bank. See cross-section.

C-6 CHANNEL

Elev.	Δd	Width	ΔA	A	w.p.	R
			Sta. 4+00 E			
88.3						
			47.55*			
91.0		25.0		47.55	25.2	1.887
	1		26.5			
92.0		28.0		74.05	29.0	2.553
	1		30.0			
93.0		32.0		104.05	33.8	3.078
	1		34.75			
94.0		37.5		138.80	39.4	3.523
	1		44.25			
95.0**		51.0		183.05	53.5	3.42
		67.0		183.05	69.5	2.63
	1		86.75			
96.0		106.5		269.80	109.4	2.47
	1		127.25			
97.0***		148.0		397.05	150.9	2.63
			Sta. 3+25.5 E			
91.4		0				
	.6		4.2			
92.0		14.0		4.2	14.2	0.296
	.2		3.3			
92.2		19.0		7.5		
	.8		21.6			
93.0		33.0		29.1	33.2	0.876
	1		41.75			
94.0		50.5		70.85	50.8	1.395
	1		58.25			
95.0		66.0		129.10	66.0	1.956
	1		72.5			
96.0		79.0		201.60	79.4	2.539
	1		85.5			
97.0		92.0		287.10	92.5	3.104

*Determined by planimeter.
**Disregarding depression.
***Estimated. Survey not carried that high.

C-6 CHANNEL

Elev.	Δd	Width	ΔA	A	w.p.	R
			Sta. 3+00 E			
90.7		0				
	.3		2.25			
91.0		15.0		2.25	15.0	0.150
	.8		28.0			
91.8		55.0		30.25		
	.2		11.60			
92.0		61.0		41.85	61.1	0.685
	.4		26.5			
92.4		71.5		68.35		
	.6		43.96			
93.0		75.0		112.31	75.4	1.489
	1		78.5			
94.0		82.0		190.81	82.5	2.313
	1		85.0			
95.0		88.0		275.81	89.4	3.086
	1		91.25			
96.0		94.5		367.06	96.0	3.824
	1		98.0			
97.0		101.5		465.06	103.0	4.515
			Sta. 2+00 E			
90.3		0				
	.2		3.4			
90.5		34.0		3.4		
	.5		23.0			
91.0		58.0		26.4	58.0	0.455
	1		61.5			
92.0		65.0		87.9	65.2	1.348
	1		68.5			
93.0		72.0		156.4	72.6	2.154
	1		74.25			
94.0		76.5		230.65	77.8	2.965
	1		79.0			
95.0		81.5		309.65	83.0	3.731
	1		84.0			
96.0*		86.5		393.65	88.2	4.46
		120.5		393.65	123.2	3.20
			130.0			
97.0		139.5		523.65	141.5	3.70

*Disregarding the depression.

C-6 CHANNEL

Elev.	Δd	Width	ΔA	A	w.p.	R
			Sta. 1+00 E			
90.8		0				
	.2		2.4			
91.0		24.0		2.4	24.0	0.10
	.2		7.15			
91.2		47.5		9.55		
	.1		5.1			
91.3		54.5		14.65		
	.7		39.9			
92.0		59.5		54.55	59.6	0.915
	.4		24.4			
92.4		62.5		78.95		
	.6		38.25			
93.0		65.0		117.20	65.4	1.792
	1		67.25			
94.0		69.5		184.45	70.2	2.627
	1		71.25			
95.0		73.0		255.70	74.6	3.428
	1		80.5			
96.0		88.0		336.20	90.5	3.715
	1		102.25			
97.0*		116.5		438.45	118.5	3.70

*Survey not carried that far.

C-6 CHANNEL

Elev.	Δd	Width	ΔA	A	w.p.	R
			Sta. 1+00 W			
90.4						
			6.42*			
91.0		18.0		6.42	18.0	0.357
	1		27.25			
92.0		36.5		33.67	36.6	0.920
	.2		7.7			
92.2**		40.5		41.37	40.5	1.021
		63.5		41.37	63.5	0.651
	.8		53.0			
93.0		69.0		94.37	69.8	1.35
	.5		35.75			
93.5		74.0		130.12		
	.5		37.50			
94.0		76.5		167.62	77.6	2.16
	1		79.75			
95.0		83.0		247.37	84.4	2.93
	1		86.0			
96.0		89.0		333.37	90.6	3.68
	1		92.0			
97.0		95.0		425.37	96.8	4.40
	1		97.75			
98.0***		100.5		523.12	103.3	5.06

*Determined by planimeter.
**Disregarding the depression.
***Survey not carried that high.

C-6 CHANNEL

Elev.	Δd	Width	ΔA	A	w.p.	R
			Sta. 2+00 W			
90.2						
			6.42*			
91.0		13.5		6.42	13.6	0.472
	1		19.75			
92.0		26.0		26.17	26.3	0.995
	.4		11.40			
92.4		31.0		37.57		
	.4		19.20			
92.8		65.0		56.77		
	.2		13.05			
93.0		65.5		69.82	65.9	1.059
	1		67.5			
94.0		70.0		137.32	70.8	1.925
	1		72.5			
95.0		75.0		209.82	76.8	2.732
	1		77.5			
96.0		80.0		287.32	81.4	3.530
	1		82.5			
97.0		84.5		369.82	86.8	4.261
	.1		8.48			
97.1		85.0		378.30		
	.9		87.5			
98.0		90.0		465.80	92.4	5.041

*Determined by planimeter.

C-6 CHANNEL

Elev.	Δd	Width	ΔA	A	w.p.	R
			Sta. 4+00 W			
89.4						
			16.70*			
91.0		15.0		16.70	15.0	1.113
	1		16.25			
92.0		17.5		32.95	18.4	1.791
	1		19.75			
93.0		22.0		52.70	23.2	2.271
	1		24.0			
94.0		26.0		76.70	28.0	2.740
	1		28.0			
95.0		30.0		104.70	32.3	3.241
	1		35.0			
96.0		40.0		139.70	42.4	3.295
	1		50.0			
97.0		60.0		189.70	62.8	3.020
			Sta. 3+00 W			
89.9						
			19.28*			
91.0		26.0		19.28	26.1	0.7385
	1		30.0			
92.0		34.0		49.26	34.3	1.435
	1		46.0			
93.0		58.0		95.26	58.4	1.631
	1		62.5			
94.0		67.0		157.76	67.6	2.333
	1		69.5			
95.0		72.0		227.26	73.0	3.110
	1		75.5			
96.0		79.0		302.76	80.3	3.770
	1		82.5			
97.0		86.0		385.26	87.8	4.390
	1		88.0			
98.0		90.0		473.26	92.0	5.145
	1		92.5			
99.0		95.0		565.76	97.3	5.814

*Determined by planimeter.

C-6 CHANNEL

Elev.	Δd	Width	ΔA	A	w.p.	R
			Sta. 5+50 W			
89.8						
			14.13*			
91.0		16.0		14.13	16.2	0.872
	1		17.5			
92.0		19.0		31.63	18.2	1.738
	1		21.5			
93.0		24.0		53.13	25.2	2.107
	1		26.0			
94.0		28.0		79.13	29.6	2.672
	1		30.5			
95.0		33.0		109.63	34.8	3.150
	1		38.0			
96.0		43.0		147.63	45.0	3.280
	1		46.5			
97.0		50.0		194.13	52.2	3.719
	1		55.5			
98.0		61.0		249.63	63.6	3.924
			Sta. 5+00 W			
89.6						
			17.99*			
91.0		18.0		17.99	18.2	0.988
	1		19.25			
92.0		20.5		37.24	21.4	1.740
	1		21.5			
93.0		22.5		58.74	24.0	2.448
	1		25.25			
94.0		28.0		83.99	30.0	2.800
	1		31.5			
95.0		35.0		115.49	37.2	3.104
	1		38.5			
96.0		42.0		153.99	44.6	3.452
	1		47.0			
97.0		52.0		200.99	54.4	3.694
	1		58.5			
98.0		65.0		259.49	67.8	3.828

*Determined by planimeter.

WATER SURFACE PROFILE

Standard Step Method – Q=1000 c.f.s.

Sta.	W. S. Elev.	A	v	$\frac{v^2}{2g}$	Elev. E. L.	R	n_m	S_f	Ave. S_f	L	Friction loss	Elev. E. L.
8+00 E	94.0	109.0	9.17	1.30	95.30	3.41	0.031	0.00710	0.00487	100.0	0.49	
~~7+00 E~~	~~94.71~~	~~173.0~~	~~5.78~~	~~0.52~~	~~95.23~~	~~3.61~~	~~0.031~~	~~0.00264~~				~~95.79~~
~~7+00 E~~	~~95.50~~	~~218.0~~	~~4.59~~	~~0.33~~	~~95.83~~	~~2.93~~	~~0.035~~	~~0.00281~~	~~0.00496~~	~~100.0~~	~~0.50~~	~~95.80~~
7+00 E	95.47	217.0	4.61	0.33	95.80	2.96	0.035	0.00280	0.00495	100.0	0.50	95.80
~~6+00 E~~	~~95.80~~	~~176.0~~	~~5.68~~	~~0.50~~	~~96.30~~	~~3.10~~	~~0.035~~	~~0.00396~~	~~0.00338~~	~~100.0~~	~~0.34~~	~~96.14~~
~~6+00 E~~	~~95.70~~	~~168.0~~	~~5.95~~	~~0.55~~	~~96.25~~	~~3.26~~	~~0.035~~	~~0.00408~~	~~0.00344~~	~~100.0~~	~~0.34~~	~~96.14~~
6+00 E	95.55	160.0	6.25	0.60	96.15	3.43	0.035	0.00420	0.00350	100.0	0.35	96.15
~~5+00 E~~	~~95.62~~	~~165.0~~	~~6.06~~	~~0.57~~	~~96.19~~	~~3.40~~	~~0.035~~	~~0.00400~~	~~0.00410~~	~~100.0~~	~~0.41~~	~~96.56~~
~~5+00 E~~	~~95.80~~	~~175.0~~	~~5.71~~	~~0.51~~	~~96.31~~	~~3.14~~	~~0.035~~	~~0.00396~~	~~0.00408~~	~~100.0~~	~~0.41~~	~~96.56~~
~~5+00 E~~	~~96.00~~	~~190.0~~	~~5.26~~	~~0.43~~	~~96.43~~	~~2.51~~	~~0.035~~	~~0.00452~~	~~0.00436~~	~~100.0~~	~~0.44~~	~~96.59~~
~~5+00 E~~	~~96.15~~	~~204.0~~	~~4.90~~	~~0.37~~	~~96.52~~	~~2.47~~	~~0.035~~	~~0.00400~~	~~0.00410~~	~~100.0~~	~~0.41~~	~~96.56~~
5+00 E	96.19	206.0	4.85	0.365	96.56	2.47	0.035	0.00391	0.00406	100.0	0.41	96.56
~~4+00 E~~	~~96.50~~	~~330.0~~	~~3.03~~	~~0.14~~	~~96.64~~	~~2.55~~	~~0.035~~	~~0.00148~~	~~0.00270~~	~~100.0~~	~~0.27~~	~~96.83~~
4+00 E	96.70	357.0	2.80	0.12	96.82	2.58	0.035	0.00124	0.00258	100.0	0.26	96.82
3+50 E	96.99							0.00117	0.00117	50.0	0.06	96.88
~~3+25.5E~~	~~96.99~~	~~286.0~~	~~3.50~~	~~0.19~~	~~97.18~~	~~3.01~~	~~0.040~~	~~0.00205~~	~~0.00161~~	~~24.5~~	~~0.04~~	~~96.91~~
3+25.5E	96.70	260.0	3.85	0.23	96.93	2.88	0.040	0.00264	0.00190	24.5	0.05	96.92
~~3+00 E~~	~~96.80~~	~~445.0~~	~~2.25~~	~~0.08~~	~~96.88~~	~~4.38~~	~~0.028~~	~~0.00025~~	~~0.00145~~	~~25.5~~	~~0.04~~	~~96.96~~
3+00 E	96.88	454.0	2.20	0.08	96.96	4.44	0.028	0.00024	0.00144	25.5	0.04	96.96
2+00 E	96.92	510.0	1.96	0.06	96.98	3.66	0.028	0.000245	0.000242	100.0	0.02	96.98
~~1+00 E~~	~~96.96~~	~~432.0~~	~~2.31~~	~~0.08~~	~~97.04~~	~~3.70~~	~~0.028~~	~~0.000332~~	~~0.000288~~	~~100.0~~	~~0.03~~	~~97.01~~
1+00 E	96.94	430.0	2.33	0.08	97.02	3.70	0.028	0.000342	0.000294	100.0	0.03	97.01
0+40 E	96.95	264.0	3.79	0.22	97.17					60.0		
0+00	97.35	276.5	3.62	0.20	97.55							
1+00 W	97.37	461.0	2.17	0.07	97.44	4.67	0.028	0.000218				

WATER SURFACE PROFILE

Leach's method (K_d) Q=500 cfs.

Sta.	Sec.	Elev.	K_d	$\frac{Q}{K_d}$	$s=\left(\frac{Q}{K_d}\right)^2_s$	L	Friction loss	W. S. Elev.
00 E		92.93						
	~~7+50 E~~	~~93.00~~	~~8,400.0~~	~~.0595~~	~~.00354~~	~~50.0~~	~~.177~~	~~93.11~~
	7+50 E	93.10	8,800.0	.0568	.00323	50.0	.162	98.09
00 E								93.25
	~~6+50 E~~	~~93.40~~	~~9,800.0~~	~~.0510~~	~~.00260~~	~~50.0~~	~~.130~~	~~93.38~~
	6+50 E	93.38	9,700.0	.0515	.00265	50.0	.133	93.39
00 E								93.52
	5+50 E	93.70	8,600.0	.0581	.00338	50.0	.169	93.69
00 E								93.86
	4+50 E	93.95	10,800.0	.0463	.00214	50.0	.107	93.96
00 E								94.07
	3+62.75 E	94.12	13,100.0	.0382	.00146	37.2	.0543	94.13
25.5 E								94.18
	~~3+12.25 E~~	~~94.19~~	~~9,900.0~~	~~.0505~~	~~.00255~~	~~12.25~~	~~.0312~~	~~94.21~~
	3+12.25	94.21	10,000.0	.0500	.00250	12.25	.0306	94.21
00 E								94.24
	~~2+50 E~~	~~94.30~~	~~24,100.0~~	~~.0207~~	~~.000428~~	~~50.0~~	~~.0214~~	~~94.26~~
	2+50 E	94.26	23,700.0	.0211	.000445	50.0	.0222	94.26
00 E								94.29
	1+50 E	94.32	25,100.0	.0199	.000396	50.0	.0198	94.31
00 E								94.33
	0+73.5 E	94.35	21,800.0	.0229	.000524	26.5	.0139	94.34
47 E								94.35

WATER SURFACE PROFILE

Leach's Method (K_d) Q=1000 cfs.

Sta.	Sec.	Elev.	K_d	$\frac{Q}{K_d}$	$s=\left(\frac{Q}{K_d}\right)^2$	L	Friction loss	W. S. Elev.
+00 E		95.00						
	~~7+50 E~~	~~95.30~~	~~19,500.0~~	~~.0513~~	~~.00263~~	~~50.0~~	~~.132~~	~~95.13~~
	7+50 E	95.15	18,500.0	.0540	.00292	50.0	.146	95.15
+00 E								95.29
	~~6+50 E~~	~~95.23~~	~~17,600.0~~	~~.0568~~	~~.00322~~	~~50.0~~	~~.161~~	~~95.45~~
	6+50 E	95.45	17,900.0	.0559	.00313	50.0	.156	95.45
+00 E								95.60
	~~5+50 E~~	~~95.72~~	~~18,000.0~~	~~.0555~~	~~.00308~~	~~50.0~~	~~.154~~	~~95.75~~
	5+50 E	95.75	18,100.0	.0552	.00305	50.0	.153	95.75
+00 E								95.90
	4+50 E	96.00	23,300.0	.0429	.00184	50.0	.092	95.99
+00 E								96.08
	~~3+62.8 E~~	~~96.18~~	~~28,000.0~~	~~.0357~~	~~.00128~~	~~37.2~~	~~.048~~	~~96.13~~
	3+62.8 E	96.13	27,600.0	.0362	.00131	37.2	.049	96.13
-25.5 E								96.18
	3+12.2 E	96.19	29,000.0	.0345	.00120	12.2	.0146	96.19
+00 E								96.21
	~~2+50 E~~	~~96.25~~	~~52,200.0~~	~~.0192~~	~~.000369~~	~~50.0~~	~~.0184~~	~~96.23~~
	2+50 E	96.23	51,800.0	.0193	.000373	50.0	.0186	96.23
-00 E								96.25
	1+50 E	96.27	49,000.0	.0204	.000416	50.0	.0208	96.27
-00 E								96.29
	0+73.5 E	96.30	43,000.0	.0233	.000543	26.5	.0144	96.30
-47 E								96.31
-00	Assume loss thru culvert $= \frac{v^2}{2g}$							96.62
-00 W								

WATER SURFACE PROFILE

Leach's Method (K_d) Q=1500 cfs.

Sta.	Sec.	Elev.	K_d	$\frac{Q}{K_d}$	$s=\left(\frac{Q}{K_d}\right)^2$	L	Friction loss	W. S. Elev.
+00 E		95.80						
	7+50 E	96.00	24,100.0	.0622	.00387	50.0	.194	95.99
+00 E								96.19
	~~6+50 E~~	~~96.37~~	~~20,000.0~~	~~.0750~~	~~.00563~~	~~50.0~~	~~.282~~	~~96.47~~
	6+50 E	96.46	20,500.0*	.0732	.00536	50.0	.268	96.46
+00 E								96.72
	5+50 E	96.90	24,800.0	.0605	.00366	50.0	.183	96.91
+00 E								97.09

*Estimate. Curve doesn't go that high.

DETERMINATION OF K_d

$$K_d = \frac{1.486\ A\ R^{\frac{2}{3}}}{n_m}$$

Reach	Elev.	A_D	A_U	$A_{ave.}$	R_D	R_U	$R_{ave.}$	$R^{\frac{2}{3}}_{ave.}$	n_k	K_d channel	K_d floodplain	K_d total	$W.P._D$	$W.P._U$	$W.P._{ave.}$	n_m
2+00 E	91.0	26.4	2.4	14.4	0.455	0.10	0.278	0.426	.028	243.1						.0375
to	92.0	87.9	54.55	71.22	1.348	0.915	1.132	1.086	.028	3,591.7						.032
1+00 E	93.0	156.4	117.20	136.80	2.154	1.792	1.973	1.573	.028	11,420.2						.028
	94.0	230.65	184.45	207.55	2.965	2.627	2.796	1.985	.028	21,974.9						.028
	95.0	309.65	255.70	282.68	3.731	3.428	3.580	2.340	.031	31,707.9						.031
	96.0	393.65	336.20	364.92	4.46	3.715	4.088	2.557	.031	44,728.6						.031
	97.0	480.15	424.20	452.18	-	-	5.061	2.947	.031	63,877.6			88.2	90.5	89.35	.031
	97.0	50.38	14.25	32.32	-	-	0.795	0.858	.075	-	421.8	64,299.4	53.3	28.0	40.65	.0978
1+00 E	91.0			2.4			0.10	0.215	.027	18.3						.042
to	92.0	Assume area		54.55			0.915	0.942	.028	2,388.8						.032
0+47 E	93.0	at 0+47 E is		117.20			1.792	1.475	.028	8,620.9						.030
	94.0	same as at		184.45			2.627	1.904	.028	18,506.1						.0282
	95.0	1+00 E.		255.70			3.428	2.273	.031	27,860.4						.031
	96.0			336.20			3.715	2.399	.031	38,662.1						.031
	97.0			424.20			4.687	2.801	.031	56,956.2					90.5	.031
	97.0			14.25			0.509	0.637	.075	-	124.9	57,081.1			28.0	.108

DETERMINATION OF K_d

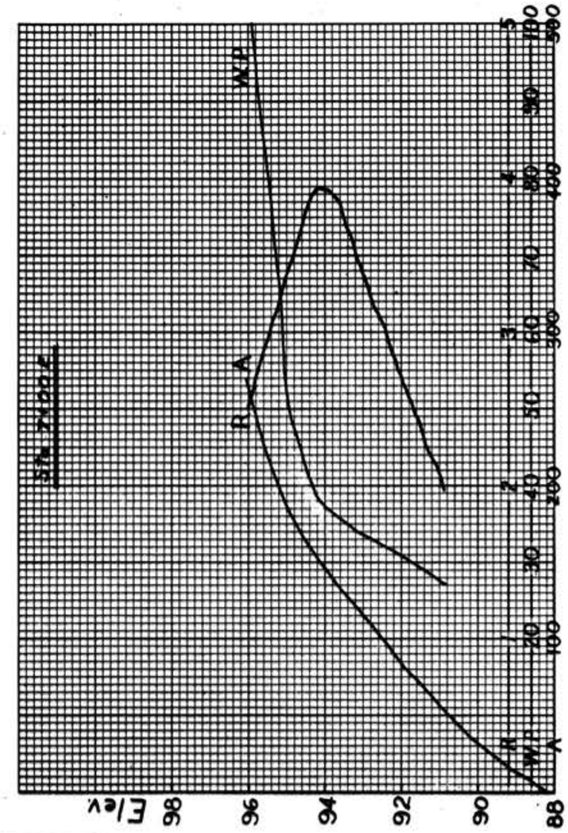
$$K_d = \frac{1.486\ A\ R^{\frac{2}{3}}}{n_m}$$

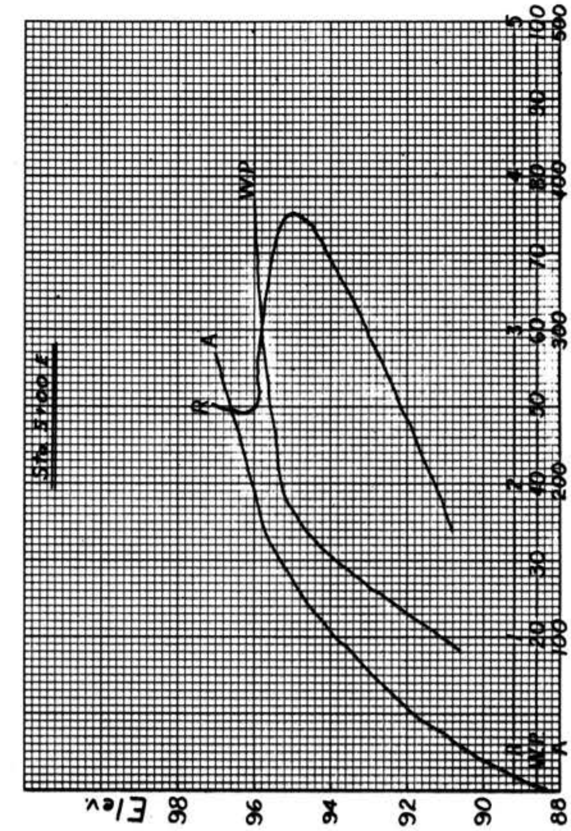
Reach	Elev.	A_D	A_U	$A_{ave.}$	R_D	R_U	$R_{ave.}$	$R^{\frac{2}{3}}_{ave.}$	n_k	K_d channel	K_d floodplain	K_d total	$W.P._D$	$W.P._U$	$W.P._{ave.}$	n_m
4+00 E	91.0			47.55			1.887	1.527	.036	2,877.2	-		-	-	-	.0375
to	92.0			74.05			2.553	1.868	.036	5,555.4	-		-	-	-	.0370
3+25.5 E	93.0			104.05			3.078	2.116	.036	8,914.8	-		-	-	-	.0367
	94.0			138.80			3.523	2.315	.038	12,565.4	-		-	-	-	.0380
	95.0			176.30			4.475	2.716	.038	18,724.8					39.4	.0380
	95.0			6.75			0.479	0.612	.075	-	55.7	18,780.5			14.1	.1102
	96.0			213.80			5.426	3.088	.038	25,817.9					39.4	.0380
	96.0			20.75			2.964	2.064	.075	-	656.1	26,474.0			70.0	.0970
	97.0			251.30			6.378	3.439	.038	34,064.5					39.4	.0377
	97.0			110.50			0.992	0.994	.075	-	1,745.6	35,810.1			111.4	.0935
3+25.5 E	92.0	4.20	41.85	23.02	0.296	0.685	0.490	0.622	-	625.9						.034
to	93.0	29.10	112.31	70.70	0.876	1.489	1.182	1.118	-	3,454.6						.034
3+00 E	94.0	70.85	190.81	130.82	1.395	2.313	1.854	1.509	-	8,627.9						.034
	95.0	129.10	275.81	202.46	1.956	3.086	2.521	1.852	-	16,387.8						.034
	96.0	201.60	367.06	284.33	2.539	3.824	3.182	2.164	-	26,891.8						.034
	97.0	287.10	465.06	376.08	3.104	4.515	3,810	2.439	-	40,089.6						.034
3+00 E	91.0	2.25	26.40	18.32	0.150	0.455	0.302	0.450	.028	327.4						.0375
to	92.0	41.85	87.90	64.88	0.685	1.348	1.016	1.010	.028	3,043.0						.032
2+00 E	93.0	112.31	156.40	134.36	1.489	2.154	1.822	1.492	.028	9,929.7						.030
	94.0	190.81	230.65	210.73	2.313	2.965	2.639	1.910	.028	20,986.2						.0285
	95.0	275.81	309.65	292.73	3.086	3.731	3.408	2.264	.031	31,768.8						.031
	96.0	367.06	393.65	380.36	3.824	4.46	4.142	2.580	.031	47.040.5						.031
	97.0	465.06	480.15	472.60	-	-	5.131	2.975	.031	67,396.6			96.0	88.2	92.10	.031
	97.0	0	50.38	25.19	-	-	0.835	0.886	.075	-	339.5	67,736.1	7.0	53.3	30.15	.0977

DETERMINATION OF K_d

$$K_d = \frac{1.486\ A\ R^{2/3}}{n_m}$$

Reach	Elev.	A_D	A_U	$A_{ave.}$	R_D	R_U	$R_{ave.}$	$R_{ave.}^{2/3}$	n_k	K_d Channel	K_d Floodplain	K_d Total	$W.P._D$	$W.P._U$	$W.P._{ave.}$	n
8+00 E	91.0	35.33	55.25	45.29	1.821	2.002	1.9115	1.541	.036	4,560.3	-		-	-	-	.037
to	92.0	55.83	83.00	69.42	2.346	2.712	2.529	1.857	.036	5,177.4	-		-	-	-	.037
7+00 E	93.0	80.58	113.00	96.79	2.817	3.393	3.105	2.128	.036	8,339.8	-		-	-	-	.036
	94.0	109.08	145.50	127.29	3.409	3.943	3.676	2.382	.038	11,856.9	-		-	-	-	.038
	95.0	139.08	179.50	159.29	-	-	4.631	2.778	.038	17,304.4	-		32.0	36.9	34.4	.03
	95.0	14.00	9.50	11.75	-	-	0.480	0.613	.075	-	97.1	17,401.5	28.6	19.5	24.5	.11
	96.0	169.08	213.50	186.29	-	-	5.415	3.084	.038	22,466.7	-		32.0	36.9	34.4	.03
	96.0	64.50	53.00	58.75	-	-	0.827	0.881	.075	-	1,692.9	24,159.6	73.6	68.5	71.0	.09
7+00 E	91.0	55.25	37.27	46.26	2.002	1.982	1.992	1.583	.036	2,901.8	-		-	-	-	.037
to	92.0	83.00	56.27	69.64	2.712	2.581	2.646	1.913	.036	5,350.5	-		-	-	-	.037
6+00 E	93.0	113.00	78.27	95.63	3.393	3.010	3.201	2.172	.036	8,573.7	-		-	-	-	.03
	94.0	145.50	103.77	124.64	3.943	3.494	3.718	2.400	.038	11,697.8	-		-	-	-	.038
	95.0	179.50	130.77	155.14	-	-	4.659	2.790	.038	16,926.3	-		36.9	29.7	33.3	.03
	95.0	9.50	4.00	6.75	-	-	0.489	0.621	.075	-	56.6	16,982.9	19.5	8.1	13.8	.11
	96.0	213.50	157.77	185.64	-	-	5.575	3.144	.038	17,700.1	-		36.9	29.7	33.3	.049
	96.0	53.00	33.50	43.25	-	-	0.722	0.805	.075	-	517.4	18,217.4	68.5	51.3	59.9	.100
	97.0	247.50	184.77	216.14	-	-	6.491	3.479	.038	22,869.0	-		36.9	29.7	33.3	.04
	97.0	147.00	116.50	131.75	-	-	1.127	1.083	.075	-	2,140.0	25,009.0	118.5	115.3	116.9	.10
6+00 E	91.0	37.27	34.70	35.98	1.982	1.798	1.890	1.529	.036	2,180.0	-		-	-	-	.03
to	92.0	56.27	55.20	55.74	2.581	2.400	2.491	1.838	.036	4,114.6	-		-	-	-	.03
5+00 E	93.0	78.27	78.70	78.48	3.010	2.959	2.984	2.073	.036	6,623.4	-		-	-	-	.036
	94.0	103.77	105.20	104.48	3.494	3.461	3.478	2.295	.038	9,376.7	-		-	-	-	.038
	95.0	130.77	133.40	132.08	-	-	4.395	2.683	.038	14,931.1	-		29.7	30.4	30.05	.03
	95.0	4.00	2.80	3.40	-	-	0.489	0.621	.075	-	28.5	14,959.6	8.1	5.8	6.95	.11
	96.0	157.77	161.60	159.68	-	-	5.314	3.045	.038	19,114.6	-		29.7	30.4	30.05	.03
	96.0	33.5	28.10	30.80	-	-	0.640	0.743	.075	-	326.4	19,441.0	51.3	45.0	48.15	.10
	97.0	184.77	189.80	187.28	-	-	6.232	3.387	.038	25,069.0	-		29.7	30.4	30.05	.03
	97.0	116.50	91.40	103.95	-	-	1.054	1.036	.075	-	172.1	25,241.1	115.3	82.0	98.65	.09
5+00 E	91.0	34.70	47.55	41.12	1.798	1.887	1.842	1.503	.036	2,449.1	-		-	-	-	.03
to	92.0	55.20	74.05	64.62	2.400	2.553	2.476	1.830	.036	4,749.4	-		-	-	-	.03
4+00 E	93.0	78.70	104.05	91.38	2.959	3.078	3.018	2.088	.036	7,768.0	-		-	-	-	.03
	94.0	105.20	138.80	122.00	3.461	3.523	3.492	2.302	.038	10,982.5	-		-	-	-	.03
	95.0	133.40	176.30	154.85	-	-	4.437	2.701	.038	16,442.3	-		30.4	39.4	34.9	.03
	95.0	2.80	6.75	4.78	-	-	0.480	0.613	.075	-	39.5	16,481.8	5.8	14.1	9.95	.11


Elev.
W.P
R
A


Elev.
W.P
R
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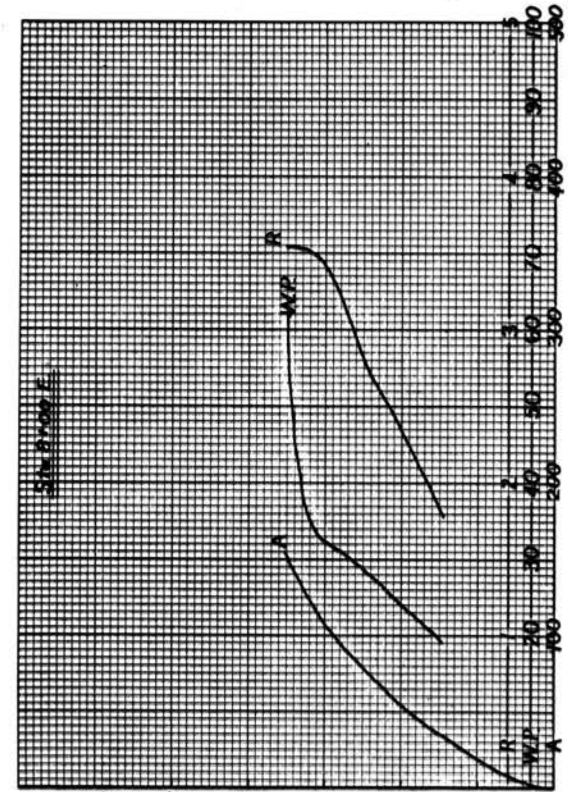
Elev.
W.P
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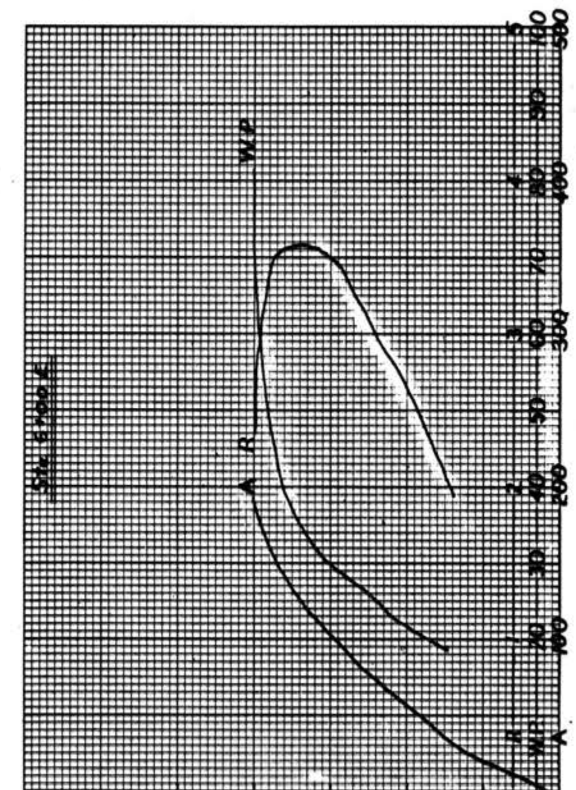
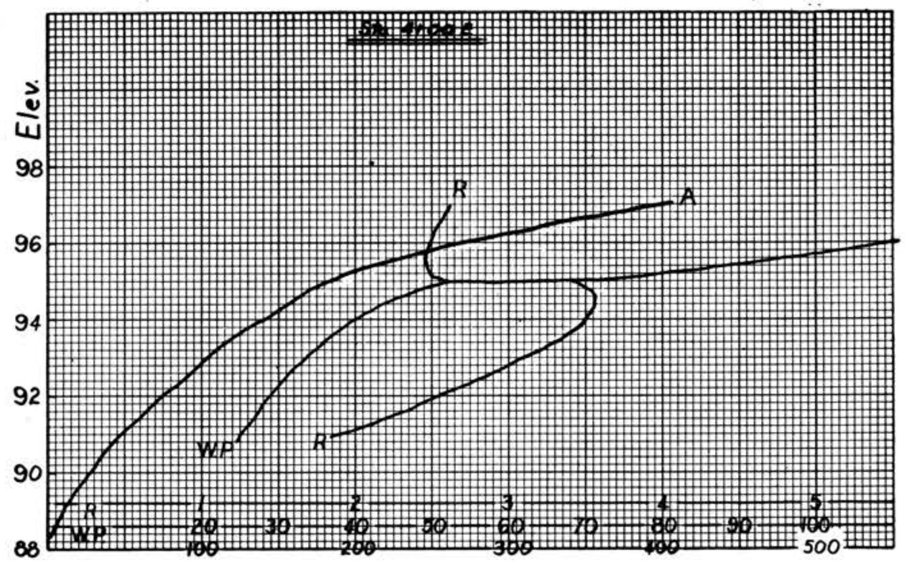
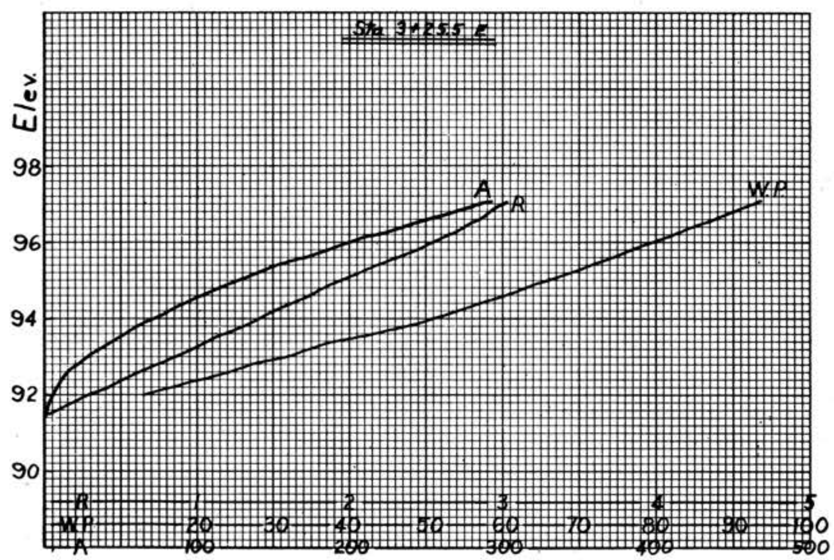
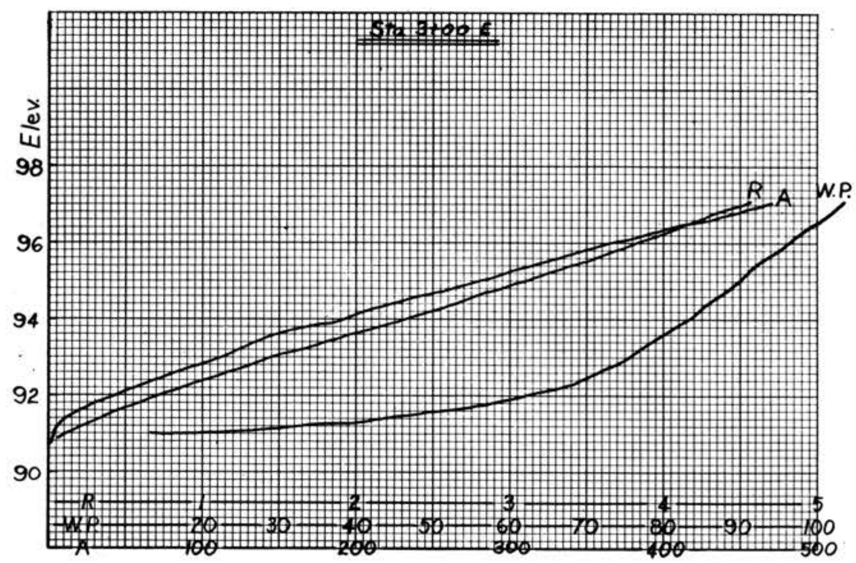

Elev.
W.P
R
A



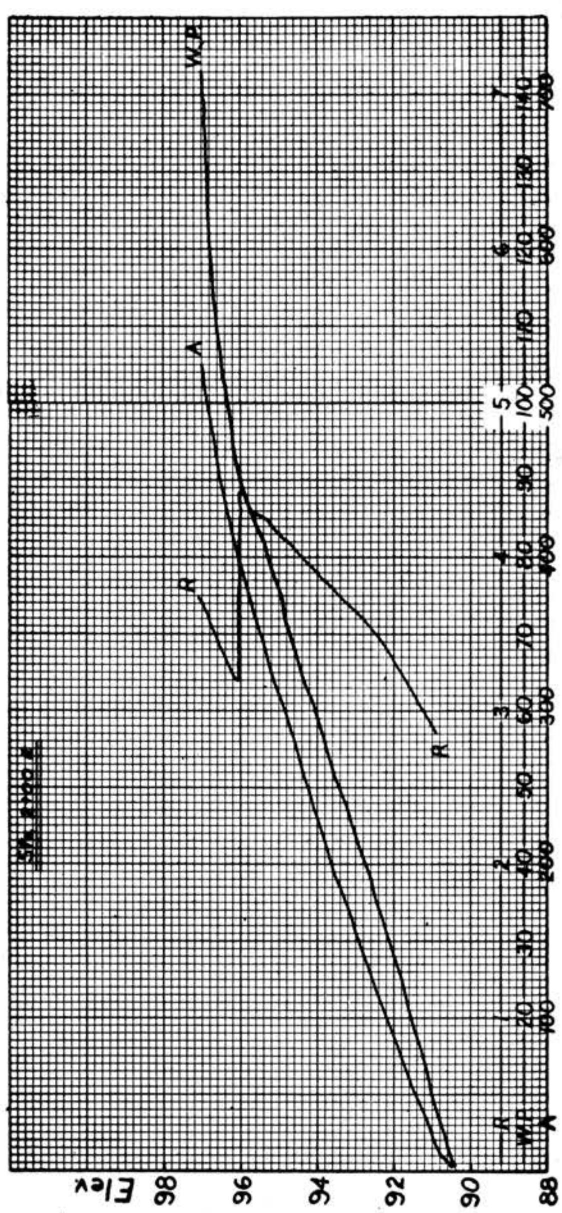
Sta. 4+00 E
Elev.
98
96
94
92
90
88
R
A
W.P.
R
1 2 3 4 5
W.P. 20 30 40 50 60 70 80 90 100
100 200 300 400 500



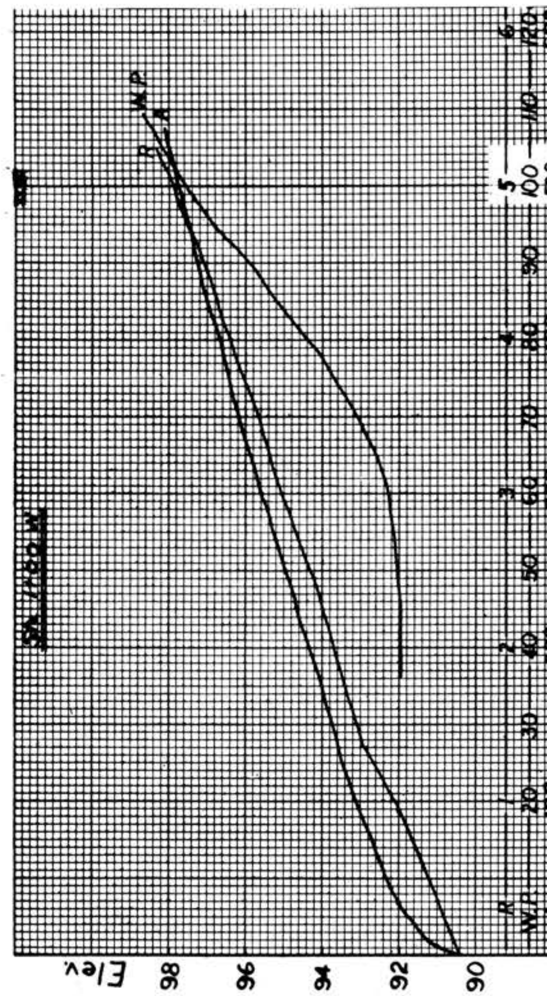
Sta. 3+25.5 E
Elev.
98
96
94
92
90
A
R
W.P.
R 1 2 3 4 5
W.P. 20 30 40 50 60 70 80 90 100
A 100 200 300 400 500



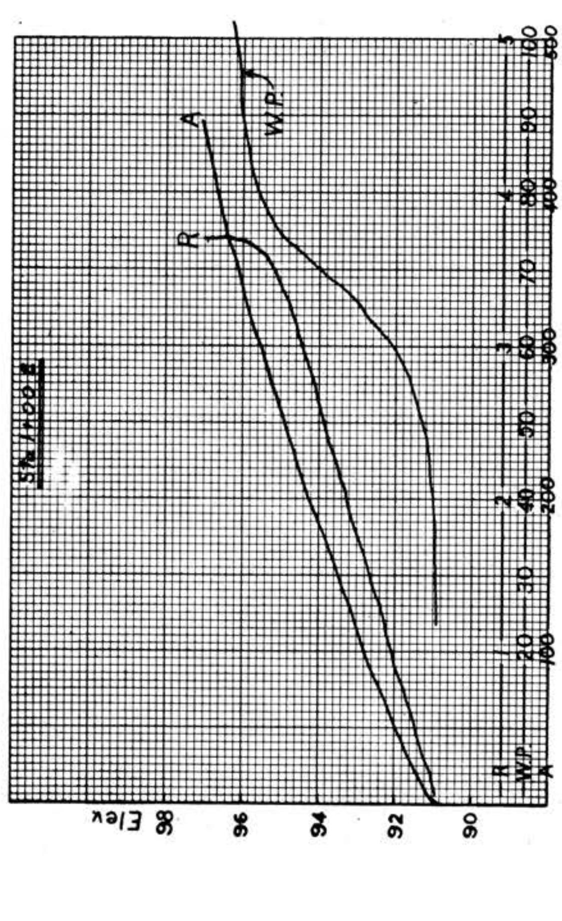
Sta. 3+00 E
Elev.
98
96
94
92
90
R
A
W.P.
R 1 2 3 4 5
W.P. 20 30 40 50 60 70 80 90 100
A 100 200 300 400 500



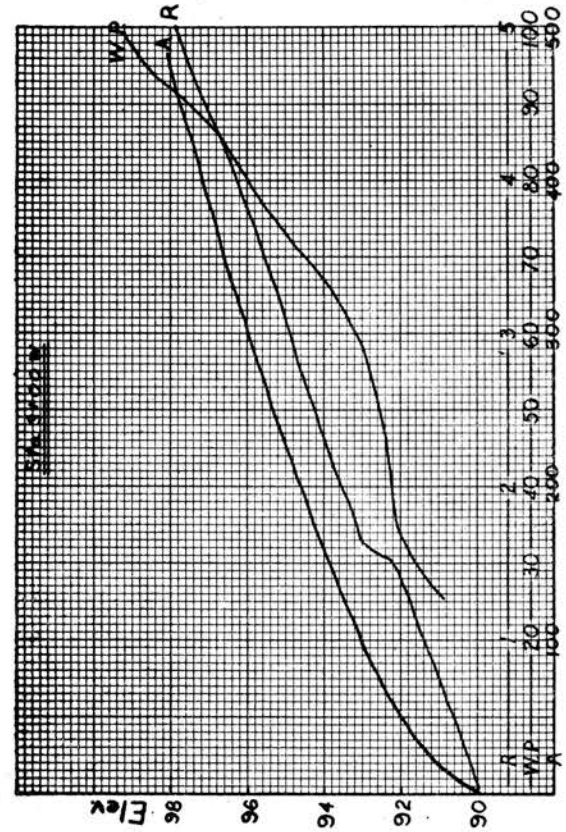
Elev
R
A
W.P.



Elev
R
A
W.P.



Elev
R
A
W.P.


Elev
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92
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W.P
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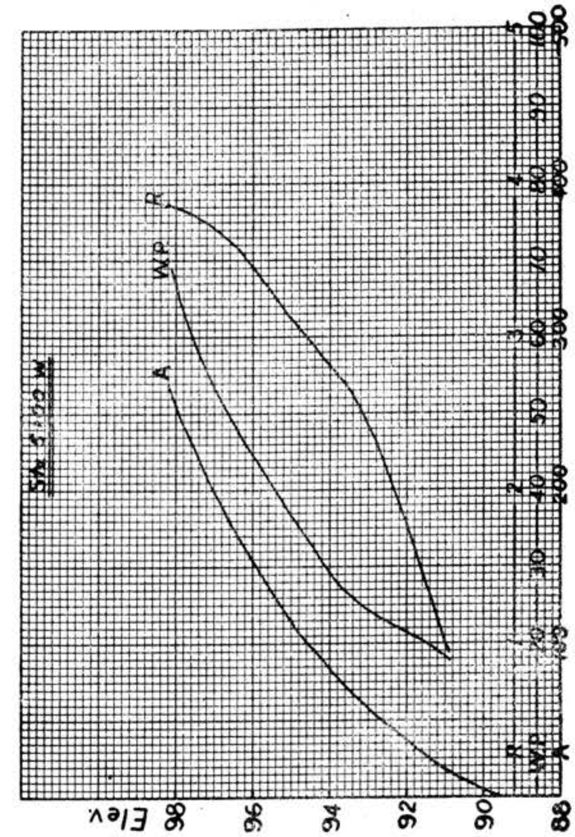
Elev
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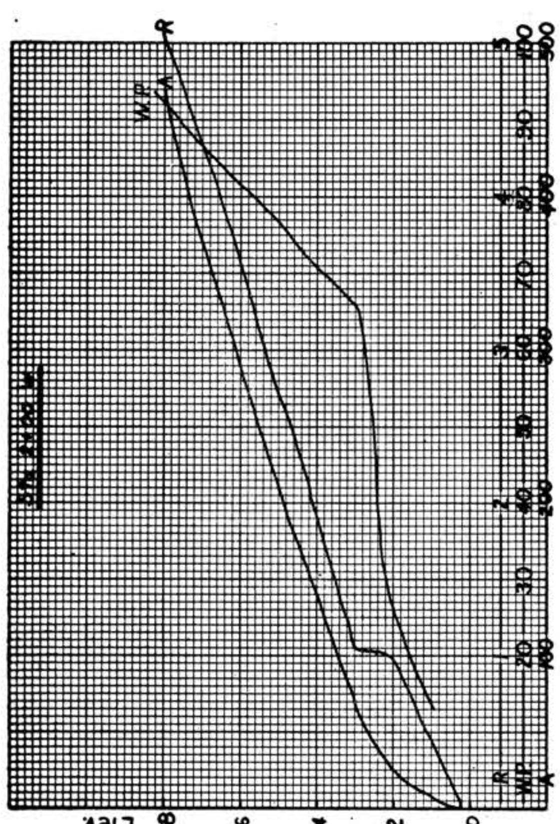
Elev
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W.P
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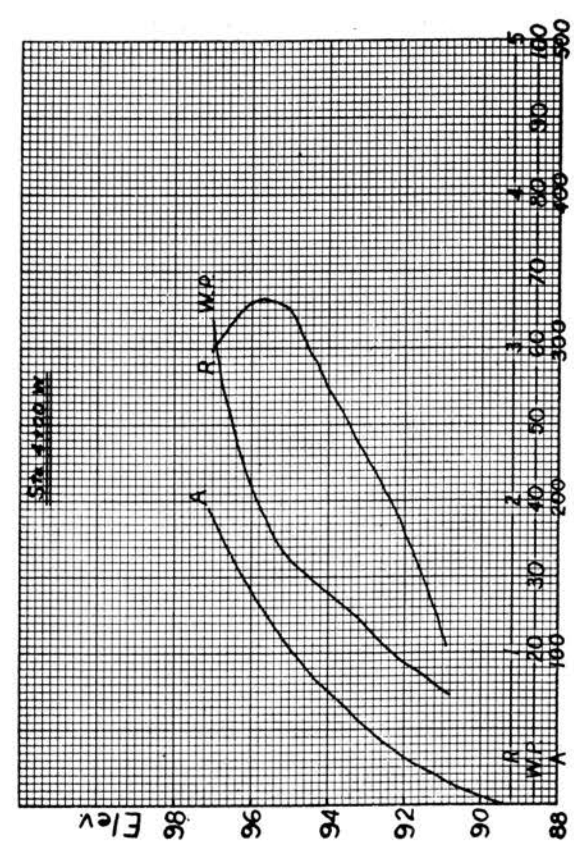
Elev
98
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92
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88
A
R
W.P
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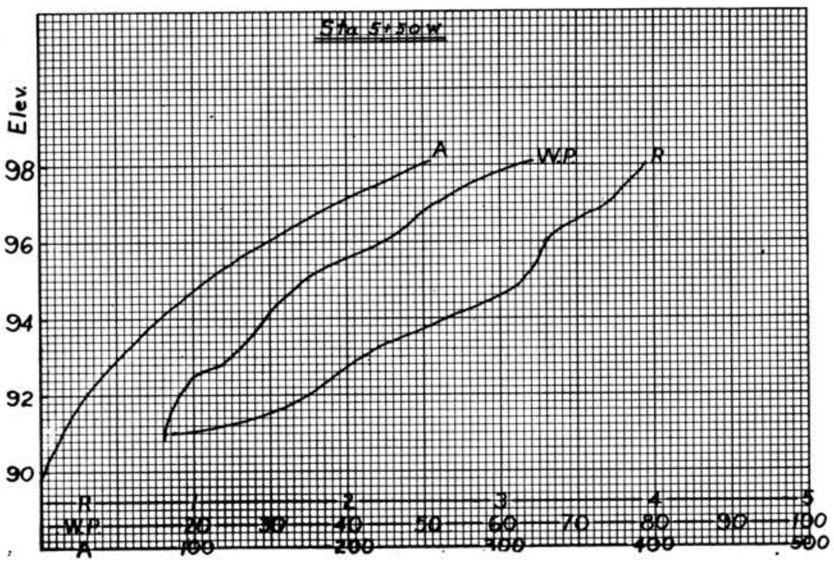
Sta. 5+50 W
Elev.
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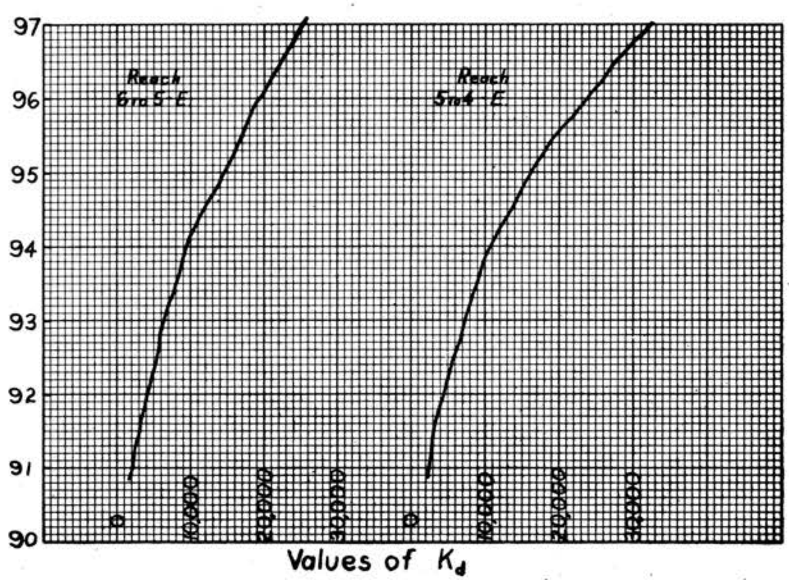
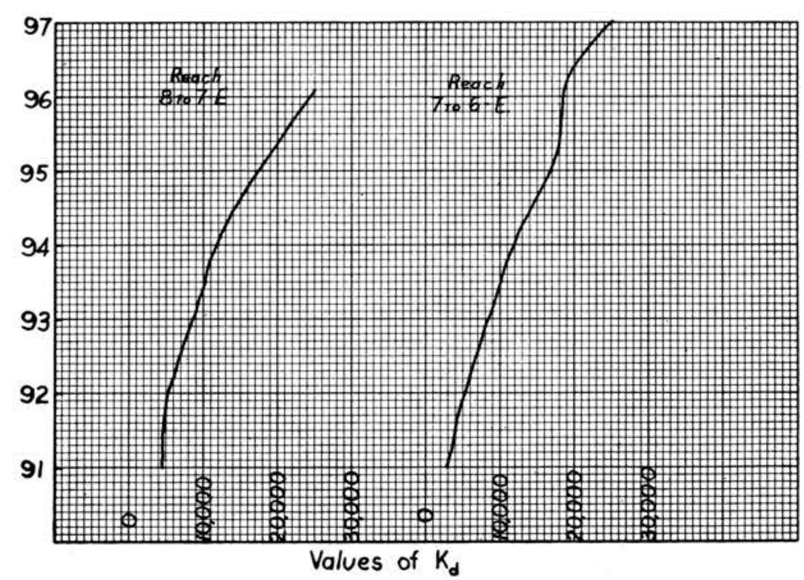
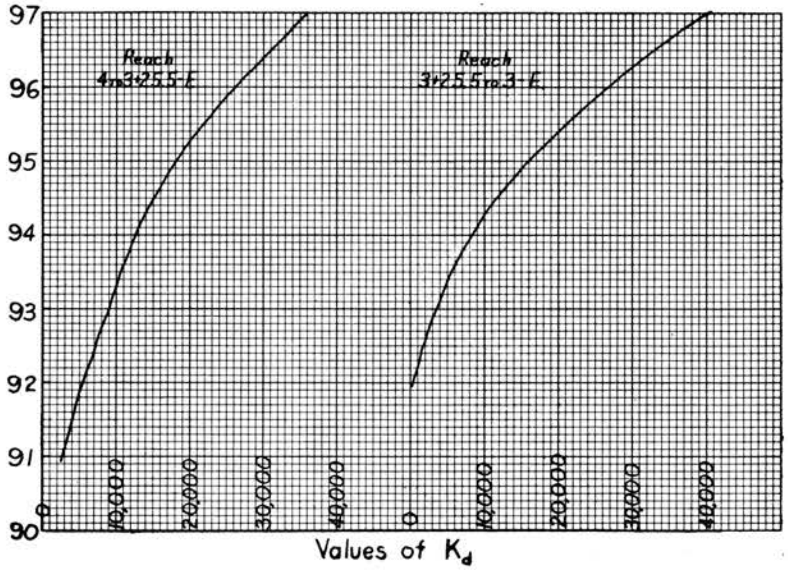
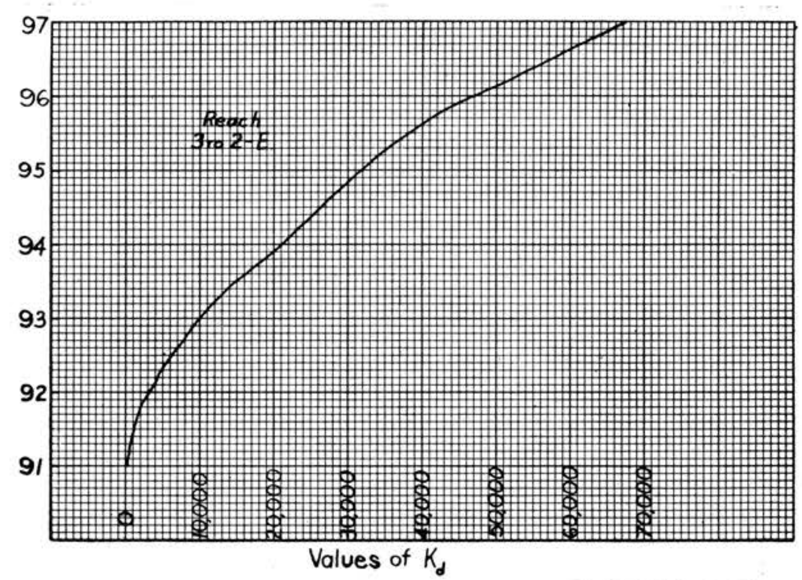
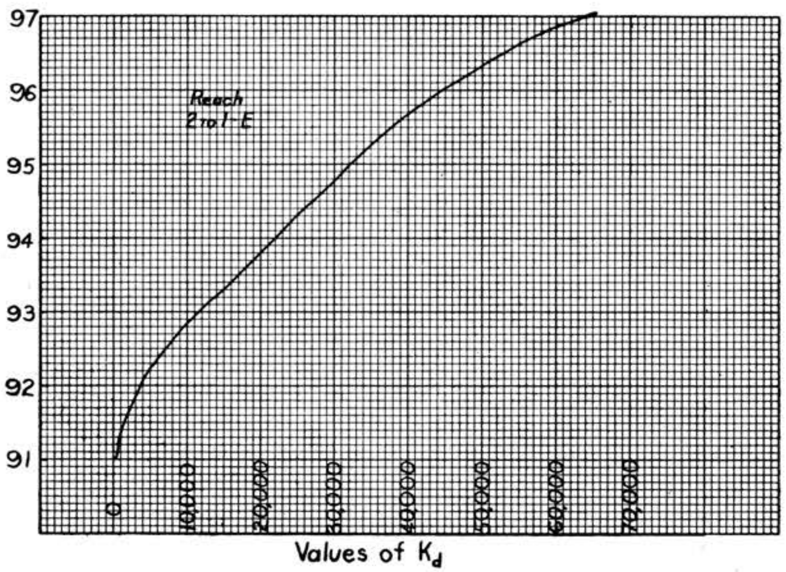
96
94
92
90
A
W.P.
R
R 1 2 3 4 5
W.P. 20 30 40 50 60 70 80 90 100
A 100 200 300 400 500

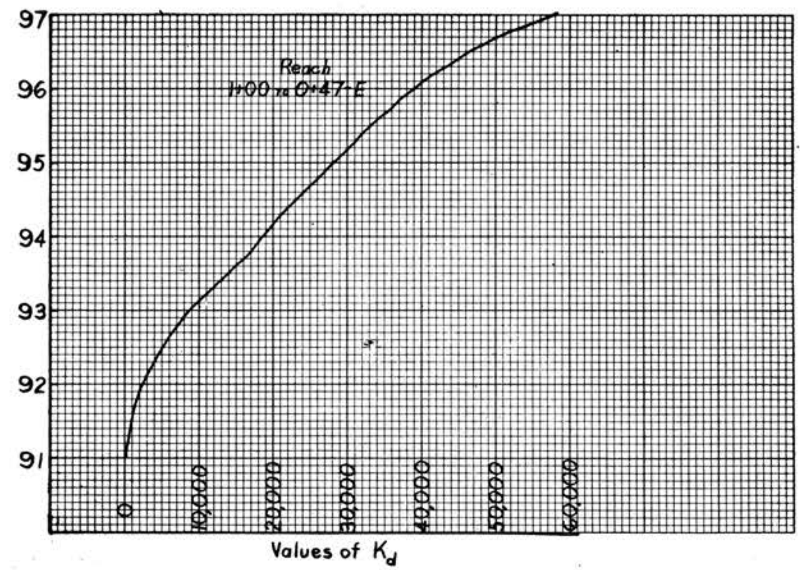

97
96
95
94
93
92
91
90
Reach 6 to 5-E.
Reach 5 to 4-E.
0 10000 20000 30000
0 10000 20000 30000
Values of K_d


97
96
95
94
93
92
91
Reach 8 to 7-E.
Reach 7 to 6-E.
0 10000 20000 30000
0 10000 20000 30000
Values of K_d


Reach 4 to 3+25.5-E
Reach 3+25.5 to 3-E
97
96
95
94
93
92
91
90
0
10,000
20,000
30,000
40,000
0
10,000
20,000
30,000
40,000
Values of K_d


Reach 3 to 2-E
97
96
95
94
93
92
91
0
10,000
20,000
30,000
40,000
50,000
60,000
70,000
Values of K_d


Reach 2 to 1-E
97
96
95
94
93
92
91
0
10,000
20,000
30,000
40,000
50,000
60,000
70,000
Values of K_d


Reach 1+00 to 0+47-E
97
96
95
94
93
92
91
0
10,000
20,000
30,000
40,000
50,000
60,000
Values of K_d

Exp. C-6-B

Stillwater Outdoor Hydraulic Laboratory

DATA-C-6 MODEL TESTS

Physical Description of Model:

Discharge - 2 ft. H-flume
Last Zero Check:

Test No.	Time	Pt. gage rdg.	Zero	Head	App. Q	Leak-age	Net Q	Proto. Q	Water Temp.

Water Surface Inside Center Barrel-head.

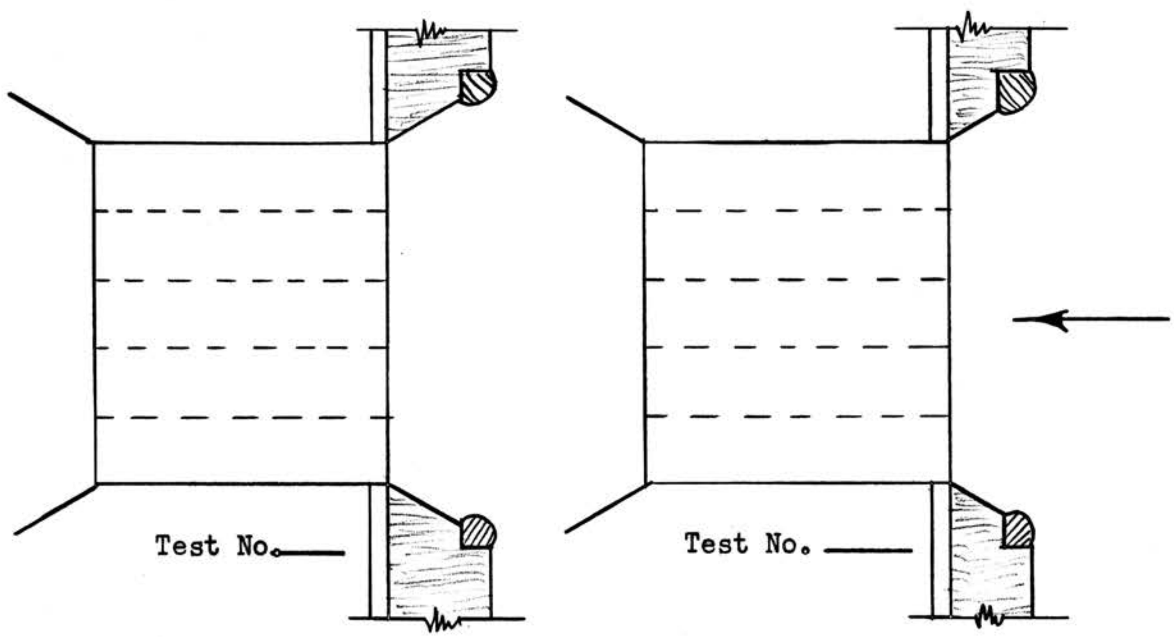
Test No.	Time	C2	C5	C10	C15	C20	C30

Headwater & Tailwater Elevations:

Test No.	Time	Open-ing	H.W. gage rdg.	Time	Open-ing	T.W. gage rdg.

Photos Taken: ______________________________

Flow Characteristics-Other Remarks: ______________________________



SUBMERGENCE ANALYSIS

C-6-B for T_s and H_s

by Villemonte Method

st o.	Q	H_s	Q_1	$\frac{Q}{Q_1}$	T_s	$s = \frac{T_s}{H_s}$	$s^{1.5}$	$(1-s^{1.5})$
9	.795	.169	.96	.828	.076	.449	.300	.700
0	.795	.169	.96	.828	.119	.705	.592	.408
1	.798	.204	1.26	.631	.191	.937	.907	.093
2	.801	.174	1.00	.801	.138	.794	.707	.293
3	.798	.187	1.11	.730	.164	.878	.823	.177
4	.801	.214	1.35	.591	.201	.940	.911	.089
5	.807	.228	1.48	.542	.219	.960	.941	.059
6	.795	.318	2.45	.324	.316	.992	.988	.012
7	1.10	.206	1.28	.860	.101	.491	.344	.656
8	1.10	.207	1.28	.860	.115	.558	.416	.584
9	1.12	.208	1.30	.862	.140	.673	.552	.448
0	1.12	.212	1.33	.841	.170	.802	.718	.282
1	1.13	.232	1.52	.742	.221	.952	.929	.071
2	1.12	.251	1.72	.651	.233	.930	.897	.103
3	1.12	.271	1.94	.578	.256	.943	.916	.084
4	1.13	.290	2.15	.527	.278	.960	.941	.059
5	1.94	.295	2.21	.878	.153	.519	.374	.626
6	1.94	.294	2.20	.881	.156	.532	.388	.612
7	1.94	.295	2.21	.878	.172	.582	.444	.556
8	1.94	.295	2.21	.878	.090	.304	.168	.832
9	1.94	.300	2.26	.860	.232	.772	.678	.322
0	1.94	.463	4.31	.450	.111	.240	.118	.882
1	1.95	.327	2.57	.760	.289	.883	.830	.170
2	1.94	.343	2.78	.699	.313	.913	.872	.128
3	1.95	.364	3.03	.643	.337	.928	.894	.106
4	1.97	.400	3.49	.566	.382	.955	.933	.067
5	2.60	.353	2.90	.897	.185	.524	.379	.621

SUBMERGENCE ANALYSIS

C-6-B for T_s and H_s

by Villemonte Method

st o.	Q	H_s	Q_1	$\frac{Q}{Q_1}$	T_s	$s=\frac{T_s}{H_s}$	$s^{1.5}$	$1-s^{1.5}$
6	2.59	.353	2.90	.891	.204	.578	.440	.560
7	2.59	.352	2.89	.894	.224	.635	.506	.494
8	2.60	.357	2.93	.889	.263	.738	.634	.366
9	2.60	.366	3.06	.850	.296	.810	.729	.271
0	2.61	.525	5.19	.504	.179	.339	.198	.802
1	2.60	.395	3.40	.767	.350	.888	.837	.163
2	2.61	-	-	-	-	-	-	-
3	4.10	.479	4.51	.909	.266	.557	.416	.584
4	4.12	.482	4.60	.897	.271	.562	.421	.579
5	4.14	.478	4.50	.920	.279	.584	.446	.554
6	4.14	.478	4.50	.920	.340	.710	.598	.402
7	4.14	.492	4.71	.880	.407	.828	.753	.247
8	4.15	.508	4.92	.843	.438	.667	.545	.455
9	4.16	.535	5.32	.781	.477	.891	.841	.159
0	4.12	.560	5.70	.722	.507	.903	.858	.142
1	4.14	.589	6.17	.672	.545	.926	.891	.109
2	4.14	.612	6.52	.634	.572	.934	.903	.097
3	4.17	.485	4.62	.901	.324	.669	.547	.453
4	6.30	.643	7.10	.888	.347	.539	.396	.604
5	6.30	.643	7.10	.888	.353	.549	.407	.593
6	6.31	.640	7.00	.900	.369	.577	.438	.562
7	6.31	.641	7.01	.900	.407	.634	.505	.495
8	6.31	.643	7.10	.889	.447	.698	.583	.417
9	6.30	.650	7.19	.877	.495	.761	.664	.336
0	6.30	.664	7.41	.849	.535	.805	.722	.278
1	6.32	.678	7.60	.831	.569	.840	.770	.230
2	6.31	.704	8.10	.780	.605	.860	.798	.202
3	6.31	.722	8.40	.750	.637	.881	.826	.174
4	6.34	.744	8.80	.720	.658	.883	.830	.170

SUBMERGENCE ANALYSIS

C-6-B for T_c and H_c

by Villemonte Method

st o.	Q	H_c	Q_1	$\frac{Q}{Q_1}$	T_c	$s = \frac{T_c}{H_c}$	$s^{1.5}$	$1-s^{1.5}$
9	.795	.157	.855	.929	.049	.311	.174	.826
0	.795	.158	.865	.919	.092	.580	.442	.558
1	.798	.193	1.17	.681	.179	.929	.891	.109
2	.801	.163	.91	.881	.113	.692	.576	.424
3	.798	.177	1.03	.772	.150	.849	.782	.218
4	.801	.205	1.27	.630	.194	.948	.923	.077
5	.807	.225	1.47	.548	.212	.940	.911	.089
6	.795	.317	2.43	.639	.313	.986	.979	.021
7	1.10	.177	1.03	1.066	.066	.373	.227	.773
8	1.10	.188	1.12	.982	.080	.425	.266	.734
9	1.12	.192	1.15	.973	.094	.488	.341	.659
0	1.12	.198	1.21	.927	.142	.718	.609	.391
1	1.13	.217	1.37	.826	.189	.870	.811	.189
2	1.12	.244	1.74	.643	.221	.908	.865	.135
3	1.12	.260	1.80	.621	.247	.950	.926	.074
4	1.13	.283	2.18	.518	.279	.985	.977	.023
5	1.94	.267	1.88	1.03	.108	.406	.258	.742
6	1.94	.276	1.99	.978	.114	.413	.266	.734
7	1.94	.275	1.99	.978	.135	.491	.344	.656
8	1.94	.278	2.02	.962	.142	.510	.364	.636
9	1.94	.280	2.04	.951	.184	.658	.534	.466
0	1.94	.295	2.21	.879	.232	.785	.696	.304
1	1.95	.312	2.41	.810	.268	.860	.798	.202
2	1.94	.333	2.66	.730	.296	.889	.838	.162
3	1.95	.352	2.89	.676	.325	.924	.888	.112
4	1.97	.392	3.40	.580	.372	.948	.923	.077
5	2.60	.332	2.65	.981	.142	.428	.280	.720

SUBMERGENCE ANALYSIS

C-6-B for T_c and H_c

by Villemonte Method

st o.	Q	H_c	Q_1	$\frac{Q}{Q_1}$	T_c	$s = \frac{T_c}{H_c}$	$s^{1.5}$	$1-s^{1.5}$
6	2.59	.333	2.66	.976	.148	.445	.296	.704
7	2.59	.333	2.66	.976	.164	.493	.346	.654
8	2.60	.337	2.70	.963	.200	.594	.458	.542
9	2.60	.345	2.80	.929	.255	.738	.634	.366
0	2.61	.359	2.95	.888	.296	.827	.752	.248
1	2.60	.377	3.18	.820	.327	.866	.806	.194
2	2.61	.400	3.49	.749	.356	.890	.840	.160
3	4.10	.451	4.15	.989	.226	.500	.354	.646
4	4.12	.451	4.15	.991	.234	.519	.374	.626
5	4.14	.449	4.10	1.007	.228	.510	.364	.636
6	4.14	.452	4.16	.992	.274	.608	.474	.526
7	4.14	.467	4.35	.950	.363	.778	.686	.314
8	4.15	.486	4.62	.899	.405	.832	.759	.241
9	4.16	.508	4.92	.847	.450	.888	.837	.163
0	4.12	.540	5.40	.763	.485	.900	.854	.146
1	4.14	.568	5.82	.710	.523	.923	.887	.113
2	4.14	.594	6.23	.662	.554	.931	.898	.102
3	4.17	.449	4.10	1.018	.239	.532	.388	.612
4	6.30	.606	6.41	.981	.304	.502	.356	.644
5	6.30	.607	6.42	.980	.297	.489	.342	.658
6	6.31	.609	6.48	.978	.286	.470	.322	.678
7	6.31	-	-	-	-	-	-	-
8	6.31	.610	6.49	.977	.370	.605	.470	.530
9	6.30	.623	6.77	.931	.434	.535	.392	.608
0	6.30	.636	6.90	.912	.488	.770	.676	.324
1	6.32	.651	7.20	.880	.526	.809	.728	.272
2	6.31	.682	7.73	.816	.572	.839	.768	.232
3	6.31	.703	8.09	.781	.603	.860	.798	.202
4	6.34	.726	8.45	.750	.629	.865	.809	.191

VITA

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Master of Science

Thesis: DETERMINATION OF THE HEAD-DISCHARGE RELATIONSHIP FOR AN EXISTING HIGHWAY CULVERT TO ENABLE ITS USE FOR MEASURING RUNOFF

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Date of Final Examination: August, 1953

THESIS TITLE: DETERMINATION OF THE HEAD-DISCHARGE RELATIONSHIP FOR AN EXISTING HIGHWAY CULVERT TO ENABLE ITS USE FOR MEASURING RUNOFF

AUTHOR: George Kidston

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