ROBERT HUMPHREYS
COMMISSIONER OF HIGHWAYS

COMMONWEALTH OF KENTUCKY
DEPARTMENT OF HIGHWAYS
FRANKFORT

March 20, 1956

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

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MEMO TO: D. V. Terrell
Director of Research

Frequently we have requests from other divisions of the Department to investigate certain conditions having a bearing on some project, or to assist them in the solution of unusual problems, the outcome of which requires no report in written form. However, in some cases the type of information assembled and the time and effort expended in working up the data more than warrant developing the material for permanent record. This is particularly true when problems of a similar nature might reasonably arise in the future.

The attached memorandum report by E. M. West dealing with an analysis of hydraulic features involved in a proposed channel change in the vicinity of the Markland Dam near Warsaw is of this nature. Both the effect on the U. S. 42 bridge over Stephens Creek and the anticipated flow characteristics at a bend in the channel are treated.

Only those having some responsibility for the drainage problems involved in the project will be directly concerned with the report, but undoubtedly many others will have more than passing interest in the methods that were used and their general application to problems of this nature. Undoubtedly there will be observations of actual performance made later, if the project is carried through in accordance with plans upon which this analysis was based.

Respectfully submitted,

L. E. Gregg

L. E. Gregg
Assistant Director of Research

LEG:hv
Copies to: Research Committee Members
J. C. Cobb (3)

Attach.

March 1, 1956

MEMO TO: L. E. Gregg
Assistant Director of Research

SUBJECT: A Hydraulic Analysis of the Channel Relocation of Stephens Creek, Proposed by the Corps of Engineers, Near Ohio River Lock and Dam No. 39, Gallatin County, Kentucky,

Recently Mr. J. O. Cornell requested the assistance of the Drainage Section of the Research Laboratory in investigating the channel that the U. S. Corps of Engineers has designed in conjunction with the raising of Dam No. 39 and the changing of the locking system. The principal reason for this investigation was to determine the effect, if any, that this project will have upon the five-span highway bridge on U. S. 42, located approximately two thousand feet upstream from the confluence of Stephens Creek and the Ohio River and at the upstream end of the new channel.

In order to evaluate the performance of the channel and consequently its effect on the bridge, water surface profiles were plotted for two assumed conditions: (1) for Stephens Creek at flood stage, with the Ohio River at a stage just sufficient to submerge the channel cross section at the confluence; and (2) for the Ohio River at normal pool stage with Stephens Creek at flood. The range of these conditions is expected to cover any normal flood conditions anticipated.

I have not considered conditions of the Ohio River similar to those of the 1937 flood, however; since floods of similar magnitudes would completely inundate the bridge and the entire surrounding area.

The backwater curves were in this case computed graphically; since the channel is uniform in cross section and slope. The method utilizes a modern type of formula for evaluating friction loss and takes into consideration the effect of the shape of the cross section. In cases where the stream does not have a uniform cross section or slope, however, it would be advisable to use other methods, such as Bresse's or the Standard Step Method.

An approximation of the difference in elevation of the two banks around the bend was made to estimate the effect of the sharp curve in the channel. Since the channel is uniform in cross section and construction, and the slope is constant, these approximations are considered reliable.

The entire investigation is based on an estimated discharge of 2,000 cfs. in Stephens Creek. Backwater curves for other values of discharge can be computed by changing columns 7-14 in the table of computations. However, from the results of the curves plotted, it appears highly unlikely that a sizeable increase in the discharge would make any appreciable difference in the backwater effect on the bridge.

It is suggested that these calculations and curves be sent to those connected with drainage, to be used as a guide in the solution of similar problems of backwater analysis which may arise in the future.

Eugene M. West

Eugene M. West
Research Engineer

CHANNEL DIVERSION FOR STEPHENS CREEK
AT OHIO RIVER LOCK-AND-DAM NO. 39

INTRODUCTION

The channel proposed by the Corps of Engineers will be 14 feet wide at the bottom, with side slopes of 3 to 1 to a height of 25 feet and a bottom slope of 0.4 percent. The side slopes are to be rip-rapped with large stones dumped in place. Starting just below the bridge on US 42 the new channel will extend some 2,000 feet downstream to the Ohio River. (see Figs. 1 & 2.)

Normal pool level of the Ohio River at the confluence of Stephens Creek is at an elevation of 420 feet. The bridge elevation is 470 feet. (In 1937, the Ohio River flood reached an elevation of about 470 feet.)

An estimated discharge was arrived at by using Dickens' Formula and comparing the figure found with the capacity of the original channel. In this analysis the discharge calculations are not included, the methods having been covered elsewhere. A discharge of 2,000 cfs was used, this value being rounded off to make calculations easier. Actually, as will be observed from the results of the backwater calculations, a sizeable increase in the discharge would have little effect on the backwater curves but would obviously raise the normal depth of flow in the channel.

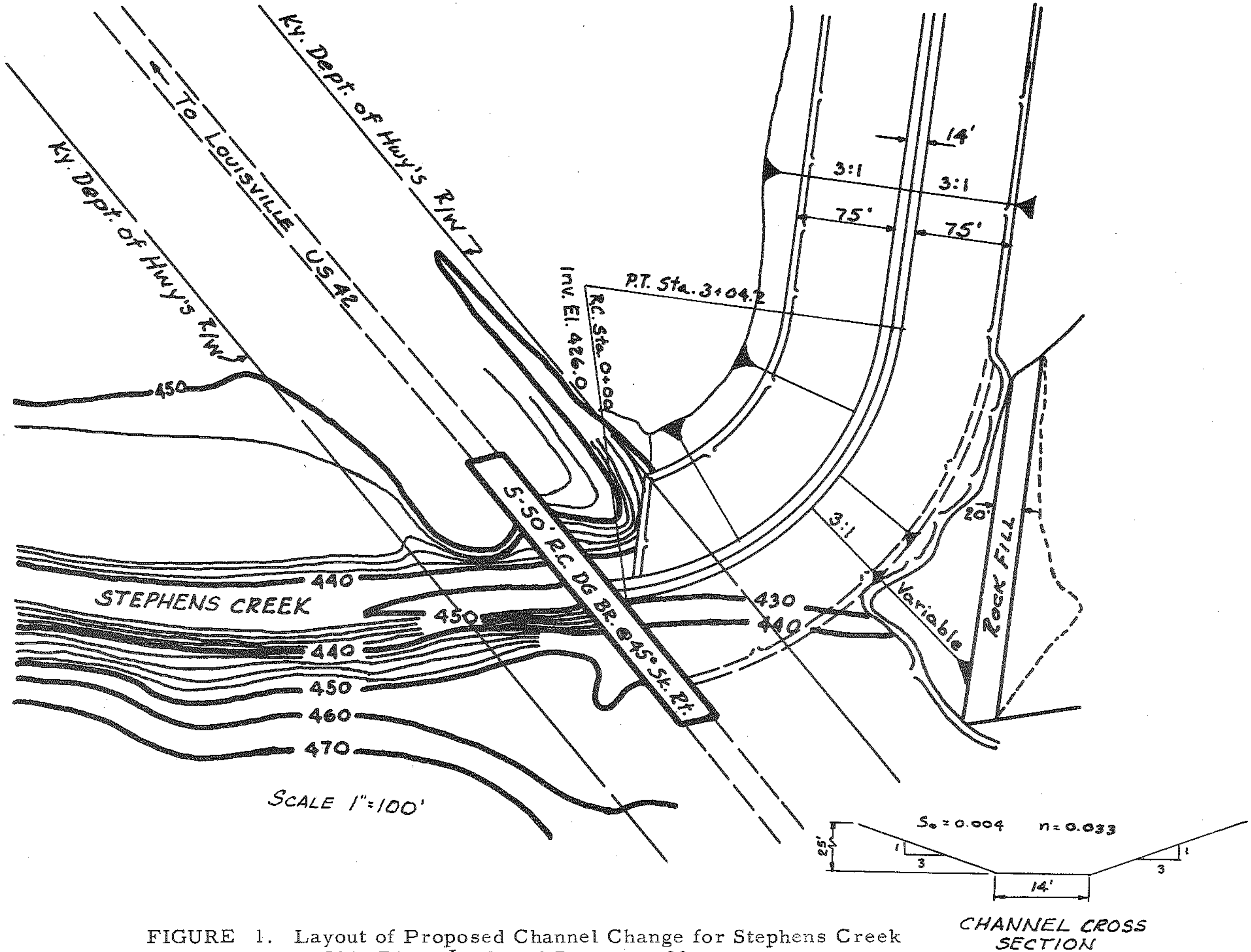


FIGURE 1. Layout of Proposed Channel Change for Stephens Creek at Ohio River Lock and Dam No. 39.

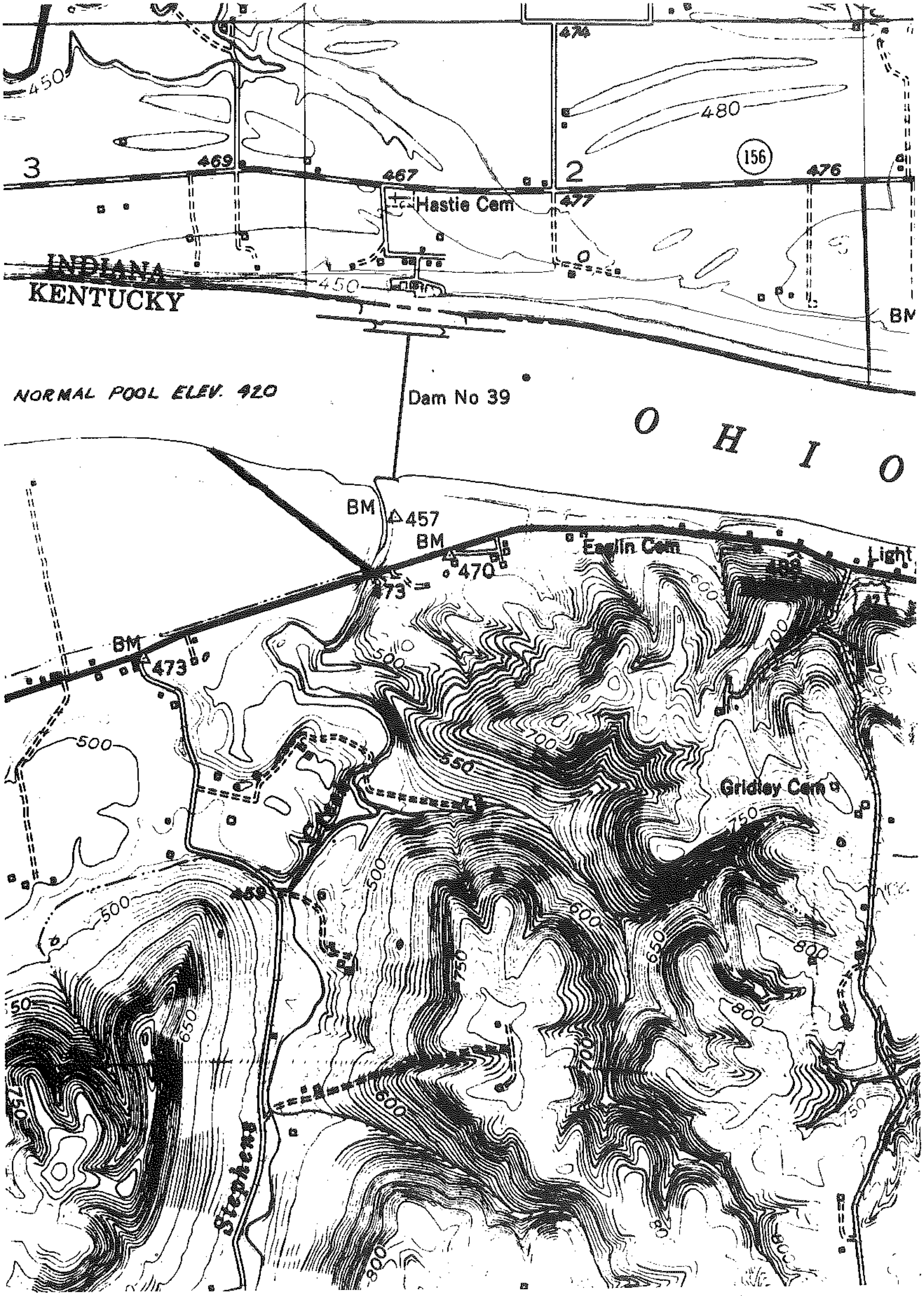


FIGURE 2. Portion of U. S. Geological Survey Map. Broken Lines Indicate Location of Proposed Channel Change.

Since the investigation was performed by making two separate studies, of (1) the backwater curves, and (2) the analysis of the flow around the bend in the new channel; these two aspects will be treated separately and independently in this report.

I BACKWATER CURVES

In general the term "backwater" suggests water backed up by a dam, or water held back in a tributary stream by a flood in the main stream into which it is discharging. The presence of this backwater brings about a condition which should be considered; the surface profile of the water in the transition area between the level pool and the unretarded approaching stream. There are other conditions of backwater and changes in the surface profile which are not evident in the term as we generally use it, such as those caused by a sudden change in the channel cross section, abrupt bends, changes of roughness, etc. There are also conditions of transition that are analyzed by backwater methods not apparent in the term backwater. An instance is the case of a stream with a sudden dropoff downstream, or of a channel emptying into another at a different level - as where there is a flood in a tributary and none in the main stream. Such conditions create a transition curve in the water surface between the stream and the main channel. The methods of computing the profiles for them are similar and are usually referred to as backwater calculations, although, properly speaking, they do not involve "backwater" situations in the usual sense of the term.

There are several methods for computing backwater curves. The selection of a method to use should be based on particular conditions, since each method has its own merits, dependent upon these conditions. In this case a graphical method was used. This was chosen for two reasons. The channel is a uniform trapezoidal shape and laid on a constant slope;

therefore, channel shape and roughness can easily be considered graphically. This representation eliminates a large portion of the computations necessary in other methods. Had the stream been a natural channel with changes in slope, one of the other methods would have been preferred. These others, such as Bresse's or the Standard Step Method, are outlined in "Steady Flow in Open Channels" by Woodward and Posey.

First to be considered in computing the backwater curves are their end limits. In the case of the Ohio River at flood stage - an elevation of 443.4, just enough to submerge the channel of Stephens Creek - and with Stephens Creek discharging 2,000 cfs, the lower end limit of the curve becomes tangent to the Ohio River at an elevation of 443.4 ft. The upper end limit becomes tangent to the surface of Stephens Creek at the creek's normal depth. From these limits, the backwater curve when calculated can be drawn. From this curve the point of tangency can be estimated; for any distance along the profile, the depth can be read.

Solving for the Normal Depth

The normal depth of flow in the new channel for a discharge of 2,000 cfs is computed as follows:

$$K = \frac{Qn}{b^{8/3}\ S^{1/2}} = \frac{2,000 \times 0.033}{(14)^{8/3} \times (0.004)^{1/2}} = .915$$

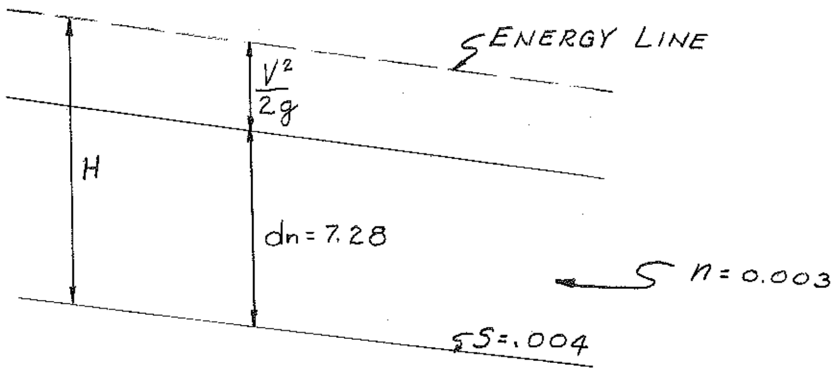
d_n/b = .52 (From King; Handbook of Hydraulics, Table 113)

d_n = .52b = .52(14) = 7.28 feet

where Q = 2,000 cfs

n = 0.033 (roughness)

s	=	0.004 (channel slope)
Z	=	3:1 (side slopes)
b	=	14 ft. (bottom width)
d_n	=	Normal depth



The total Energy (H) then becomes $d_n + V^2/2g$ (the velocity head)

For the velocity head:

$$V = \frac{Q}{A} = \frac{2,000}{261.1} = \underline{7.66} \text{ ft. per sec.}$$

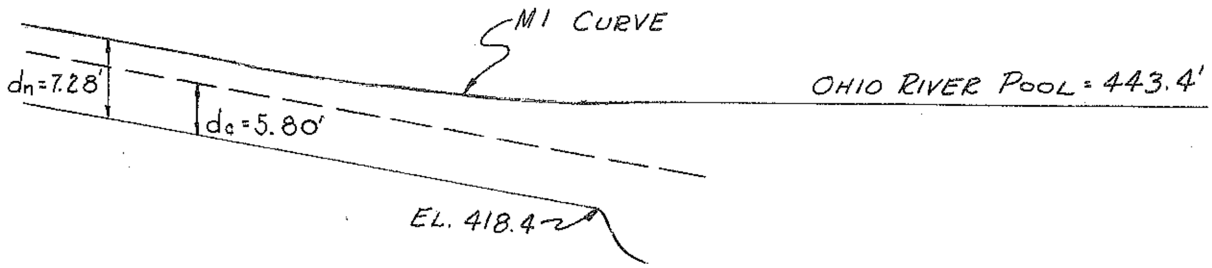
$$\frac{V^2}{2g} = \frac{7.66^2}{64.4} = .91 \text{ ft.}$$

$$H = 7.28 + .91 = 8.19$$

$\frac{V^2}{2g}$ is less than 1/2 d_n and flow is sub-critical. For flow less than critical the backwater curve will be an M1 type. (See Woodward & Posey; <u>Flow in Open Channels</u>, pp. 64-65.)

M1 Type Backwater Curve

The M1 type curve is concave upward, similar to the following sketch:



When the depth of submergence (pool level) is greater than critical depth, the M1 curve begins at the water surface.

The critical depth (d_c) is 5.80 ft.

$K_c \quad = \quad \dfrac{Q}{b^{5/2}} \quad = \quad \dfrac{2,000}{(14)^{5/2}} \quad = \quad 2.727$ (King; Handbook of Hydraulics, Table 125)

$Q \quad = \quad K_c' b \qquad \dfrac{d_c}{b} \quad = \quad .415$

Then: $d_c = .414 \times 14 = 5.80$ ft.

The two factors that define the profile of a backwater curve are the effect of friction and the change in kinetic energy of the water. The curvature, the change in slope of the line of the water surface, is caused by the rate of change in the kinetic energy brought about by the loss due to friction.

The friction slope, or rate of loss of head through friction, is given by Manning's formula where:

$$S_f = \frac{n^2 Q^2}{2.21 A^2 R^{4/3}}$$

The rate of change in velocity head is $-\dfrac{Q^2 T}{g A^3}$ times a function of an increment of the variable depth of flow corresponding to an increment of distance, written as dy/dx.

The slope of the channel less the slope of the rate of the change in the depth is the same as the velocity head change plus the friction slope. Thus:

(Solving for the increment of distance with respect to the variable increment of depth, or dx with respect to dyx:)

$$dx = \frac{1 - \frac{Q^2 T}{g A^2}}{S_o - S_f} dy$$

In this equation a value of x, the distance along the channel, is equated to a function of the depth y times an increment of the depth:

$$dx = f(y)\, dy$$

since: $dx = \dfrac{1 - \frac{Q^2 T}{g A^3}}{S_o - S_f} dy$ is a function of the depth.

To simplify the calculation and to eliminate the integral calculus involved, a method of graphical integration is used.

Values of f (y) are computed corresponding to values of y (the depth) covering the range of depths of the backwater curve. This can be done in tabular form (see Table 1). Values of the function of y, f(y), computed in the table are plotted on rectangular coordinate paper against the values of depth y used in column 1 of the table and through these points a smooth curve is drawn. The area under the curve between any two depths

TABLE 1. TABULAR SOLUTIONS FOR BACKWATER CURVES

1	2	3	4	5	6	7	8	9	10	11	12	13	14
DEPTH Y	SURFACE WIDTH T	AREA A	A^3	HYDRAULIC RADIUS $R = \frac{A}{P}$	$R^{4/3}$	$\frac{Q^2 T}{g A^3}$	$1 - \frac{Q^2 T}{g A^3}$	$\frac{D}{b}$	$\left(\frac{Q}{b^{8/3} S_f}\right)^{1/2}$	$S_f^{1/2}$	S_f	$S_o - S_f$	$\frac{\text{Col. 8}}{\text{Col. 13}}$
							M 1 TYPE CURVE						
7'	56'	245	14,706,125	4.20	6.776	.47304	.52696	0.500	0.833	.069502	.00483	.00083	634.9
8'	62'	304	28,094,464	4.71	7.894	.27414	.72586	0.571	1.11	.052158	.00272	.00128	567.0
9'	68'	369	50,243,409	5.20	9.009	.16813	.83187	0.643	1.44	.040205	.00162	.00238	349.5
10'	74'	440	85,184,000	5.70	10.18	.10791	.89209	0.714	1.81	.031986	.00102	.00298	299.4
12'	86'	600	216,000,000	6.67	12.56	.04946	.95054	0.857	2.78	.020826	.00043	.00357	266.3
14'	98'	784	481,890,304	7.65	15.07	.02526	.97474	1.000	3.97	.014583	.00021	.00379	257.2
16'	110'	992	976,191,488	8.61	17.65	.01400	.98600	1.143	5.41	.010701	.00012	.00389	253.5
18'	122'	1,224	1,833,767,424	9.57	20.32	.00826	.99174	1.286	7.21	.008030	.00007	.00394	251.7
20'	134'	1,480	3,241,792,000	10.53	23.08	.00513	.99487	1.429	9.29	.006232	.00004	.00396	251.2
22'	146'	1,760	5,451,776,000	11.49	25.93	.00333	.99667	1.571	11.72	.004940	.00002	.00398	250.7
25'	164'	2,225	11,015,140,625	12.93	30.35	.00185	.99815	1.786	15.99	.003621	.00001	.00399	250.4
							M 2 TYPE CURVE						
5.8	48.8	182	6,028,568	3.59	5.497	1.00359	-.00359	.414	.5566	.104015	.01082	-.00682	.526
6.0	50.0	192	7,077,888	3.70	5.723	.87755	.12245	.429	.6010	.096331	.00928	-.00528	-23.191
6.2	51.2	202	8,242,408	3.80	5.930	.77165	.22835	.443	.6433	.089997	.00810	-.00410	-55.709
6.4	52.4	212	9,528,128	3.90	6.139	.68317	.31683	.457	.6867	.084309	.00711	-.00311	-101.940
6.6	53.6	223	11,089,567	4.00	6.350	.60042	.39958	.471	.7324	.079048	.00625	-.00225	-177.670
6.8	54.8	234	12,812,904	4.10	6.562	.53130	.46870	.486	.7834	.073902	.00546	-.00146	-320.588
7.0	56.0	245	14,706,125	4.20	6.776	.47304	.52696	.500	.8330	.069502	.00483	-.00083	-634.128
7.2	57.2	257	16,974,593	4.30	6.992	.41860	.58140	.514	.8724	.066363	.00440	-.00040	-1439.109
7.28	57.7	261	17,779,581	4.35	7.101	.40314	.59686	.520	.9060	.063902	.00408	-.00008	-7191.084

from column 1 is the distance along the stream between the two depths.

Column 1 of the table gives depths along the curve. The limits of the range in depths y to use are the depth at the control - in this case the 25 ft. depth at the Ohio River - and the normal depth of flow in the channel (7.28 ft). The curve then is tangent to the water surface at the points of these depths. To define the curve between these extremes successive values of depth within this range were tried.

Columns 2-6 give properties and functions of the properties of the shape of the channel cross section for the various depths from Column 1.

Columns 7 and 8 give functions of the depth as previously described.

Columns 9-11 give functions used in finding the friction slope. These values are used with King's Table 113 to find the friction slope.

Column 12 gives the friction slope.

Column 13 gives the difference between the channel slope and the friction slope.

Column 14 gives the function of y obtained by dividing the values in column 8 by those in 13.

Values of y in column 1 are plotted against values of f(y) in column 14 (see Figure 3).

The graphical integration of the curve for the function of y (Fig. 3) shown in Fig. 4, is accomplished by a summation of the areas under the curve made in Table 2 and by plotting them against the depth (see Fig. 4). The curve drawn through these points is the backwater curve.

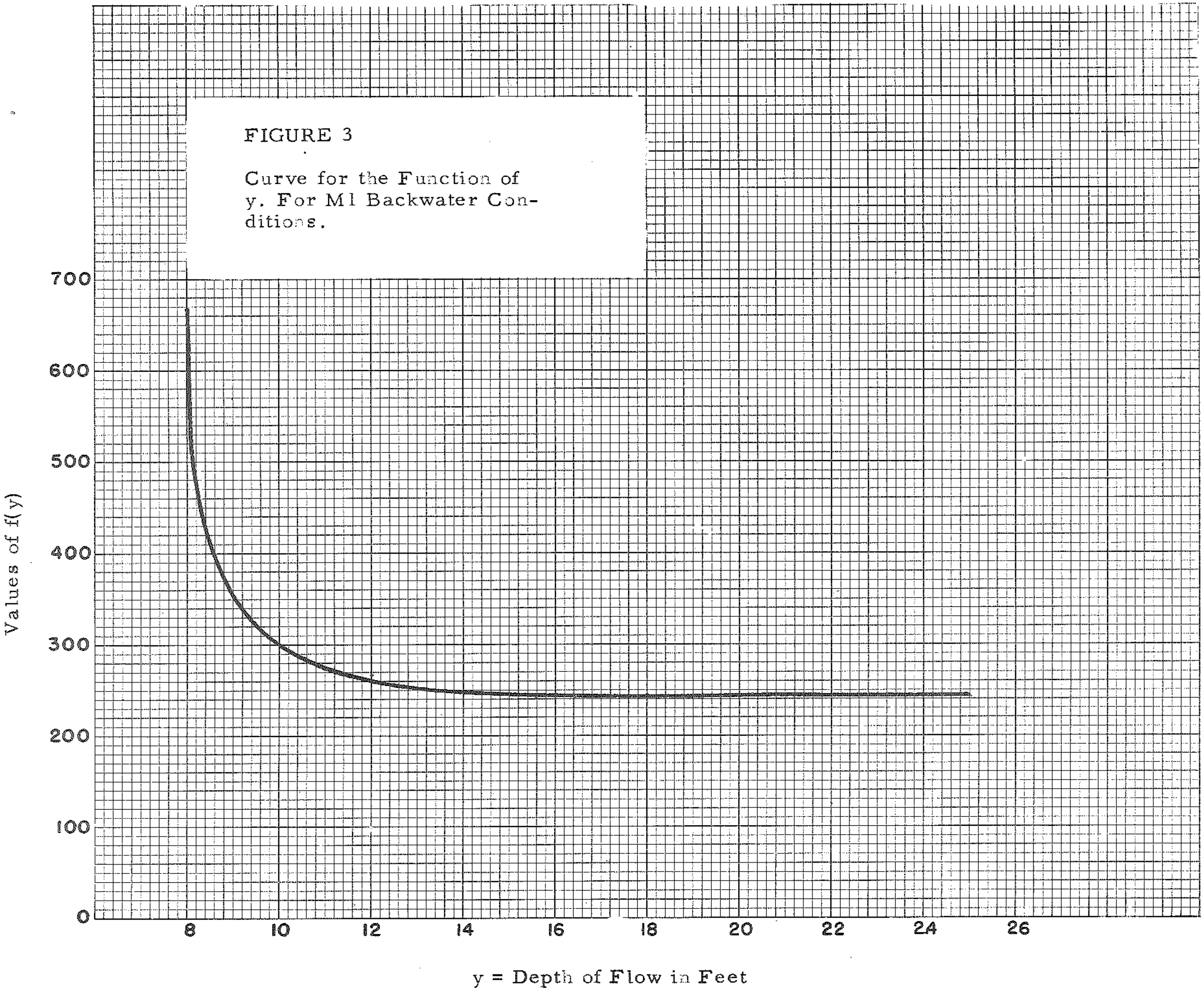


FIGURE 3

Curve for the Function of y. For M1 Backwater Conditions.

Y			INCREMENT AREA	TOTAL AREA
25			0.0	0.0
22	$\frac{250.4 + 250.7}{2} \times 3$	=	751.8	751.8
20	$\frac{251.2 + 250.7}{2} \times 2$	=	501.9	1253.7
18	$\frac{251.7 + 251.2}{2} \times 2$	=	502.9	1756.6
16	$\frac{254.5 + 251.7}{2} \times 2$	=	506.2	2262.8
14	$\frac{257.2 + 254.5}{2} \times 2$	=	511.7	2774.5
12	$\frac{266.3 + 257.2}{2} \times 2$	=	523.5	3298.0
10	$\frac{299.4 + 266.3}{2} \times 2$	=	565.7	3863.7
9	$\frac{349.8 + 299.4}{2} \times 1$	=	324.6	4188.3
8	$\frac{567.0 + 349.8}{2} \times 1$	=	458.4	4646.7

TABLE 2

Integration of f(y)

(from y = 8 to y = 25)

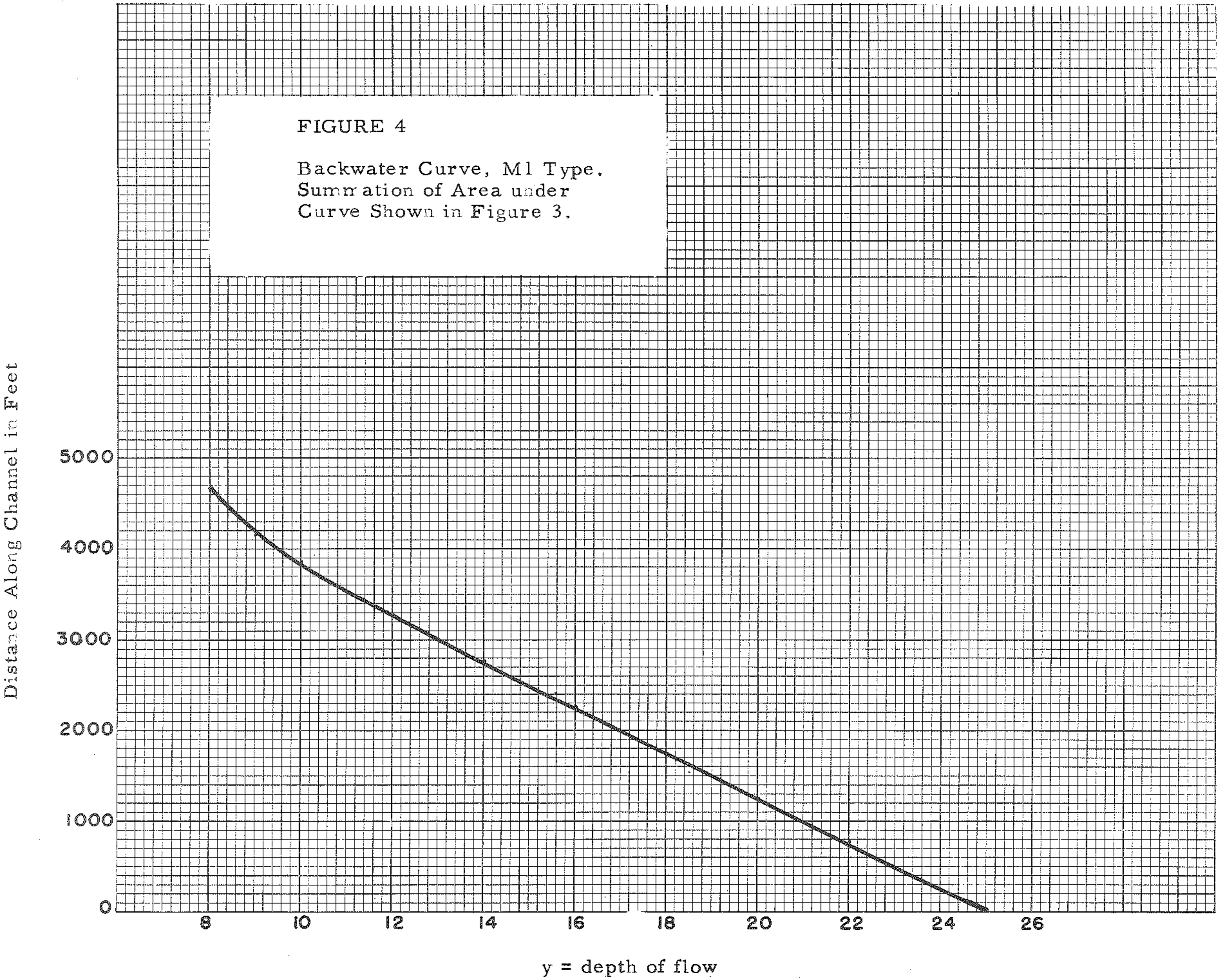


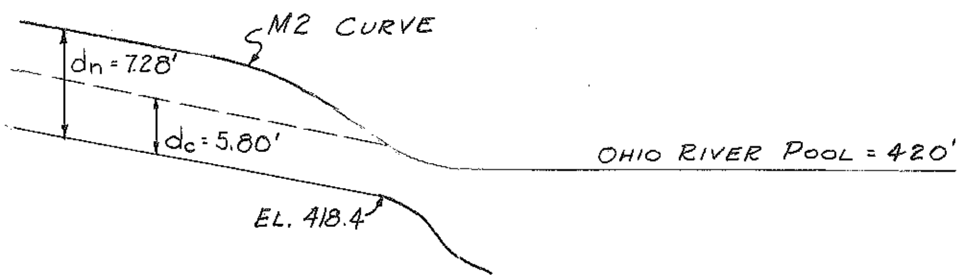
FIGURE 4

Backwater Curve, M1 Type.
Summation of Area under
Curve Shown in Figure 3.

The distance to any desired depth or the depth at any desired distance along the channel can be read directly from Fig. 4. It can be noted that the curve becomes asymptotic to the water surface in the channel at normal depth of flow.

M2 Type Backwater Curve

The other condition investigated was that of the Ohio River at normal pool level and Stephens Creek at flood stage, discharging 2,000 cfs, illustrated below:



Since the amount of submergence in the channel due to the pool level of the Ohio River is only 1.6 feet - less than the critical depth for Stephens Creek, which is 5.81 feet - the curve will be an M2 type. Theoretically the lower end of the backwater curve should terminate a abruptly tangent to a vertical line, and at a height equal to critical depth; however, because of vertical components of velocity it will merge into a local phenomenon known as dropoff. This would tend to give a smoother transition to pool level in the Ohio River.

The limits then, for this curve are to critical depth at the outlet or pool end and then normal depth of flow in the un-accelerated portion of the channel upstream.

The M2 curve calculations are similar to those of the M1, the limiting conditions being the difference in the two curve types.

The difference in the calculations is in the selection of the trial values of the depth y (See Table 1). In this case values of y should begin at or near the critical depth and be carried back by intervals to a value approaching the normal depth. As the value of y approaches the normal depth the f(y) approaches infinity. Values of less than critical depth will give positive values for f(y). In this case all values of f(y) are negative, indicating that the depth cannot become that low.

In this computation the f(y) did come out with a positive sign for critical depth. However, this value is small (.5) and within the margin of error for this type of calculation.

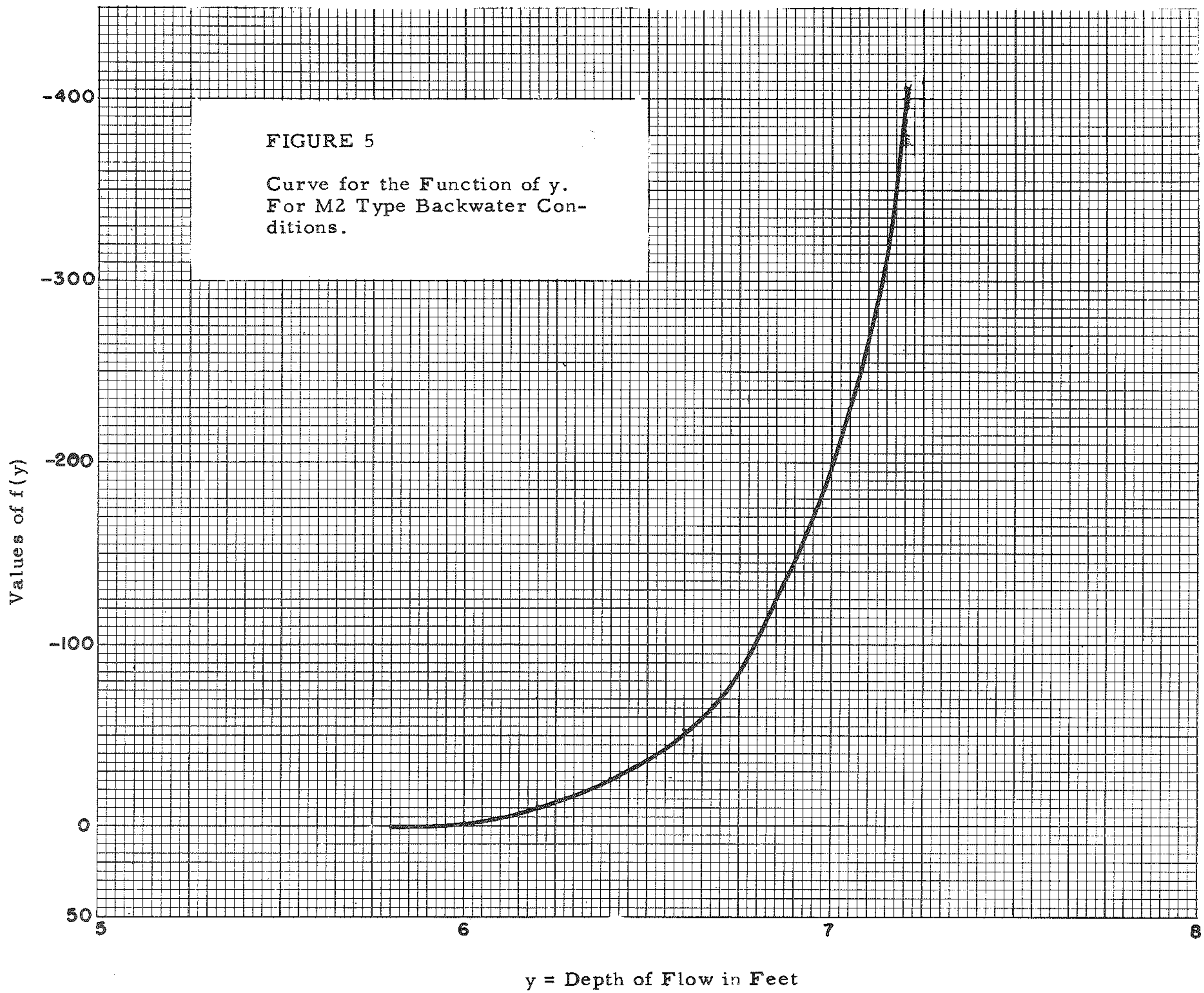


FIGURE 5
Curve for the Function of y. For M2 Type Backwater Conditions.
-400
-300
-200
-100
0
50
5
6
7
8
Values of f(y)
y = Depth of Flow in Feet

y		INCREMENT AREA	TOTAL AREA
5.8	$\frac{.815 + 0}{2} \times 2 =$	0.1	0.1
6.0	$\frac{0 - 23.191}{2} \times 2 =$	- 2.3	- 2.2
6.2	$\frac{-23.191 - 55.709}{2} \times 2 =$	- 7.9	- 10.1
6.4	$\frac{-55.709 - 101.940}{2} \times 2 =$	- 15.8	- 25.9
6.6	$\frac{-101.940 - 177.670}{2} \times 2 =$	- 28.0	- 53.9
6.8	$\frac{-177.670 - 320.588}{2} \times 2 =$	- 49.8	- 103.7
7.0	$\frac{-320.588 - 634.128}{2} \times 2 =$	- 95.5	- 199.2
7.2	$\frac{-634.128 - 1439.109}{2} \times 2 =$	- 207.3	- 406.5

TABLE 3

Integration of f(y)

(from y = 5.8 to y = 7.2 ft.)

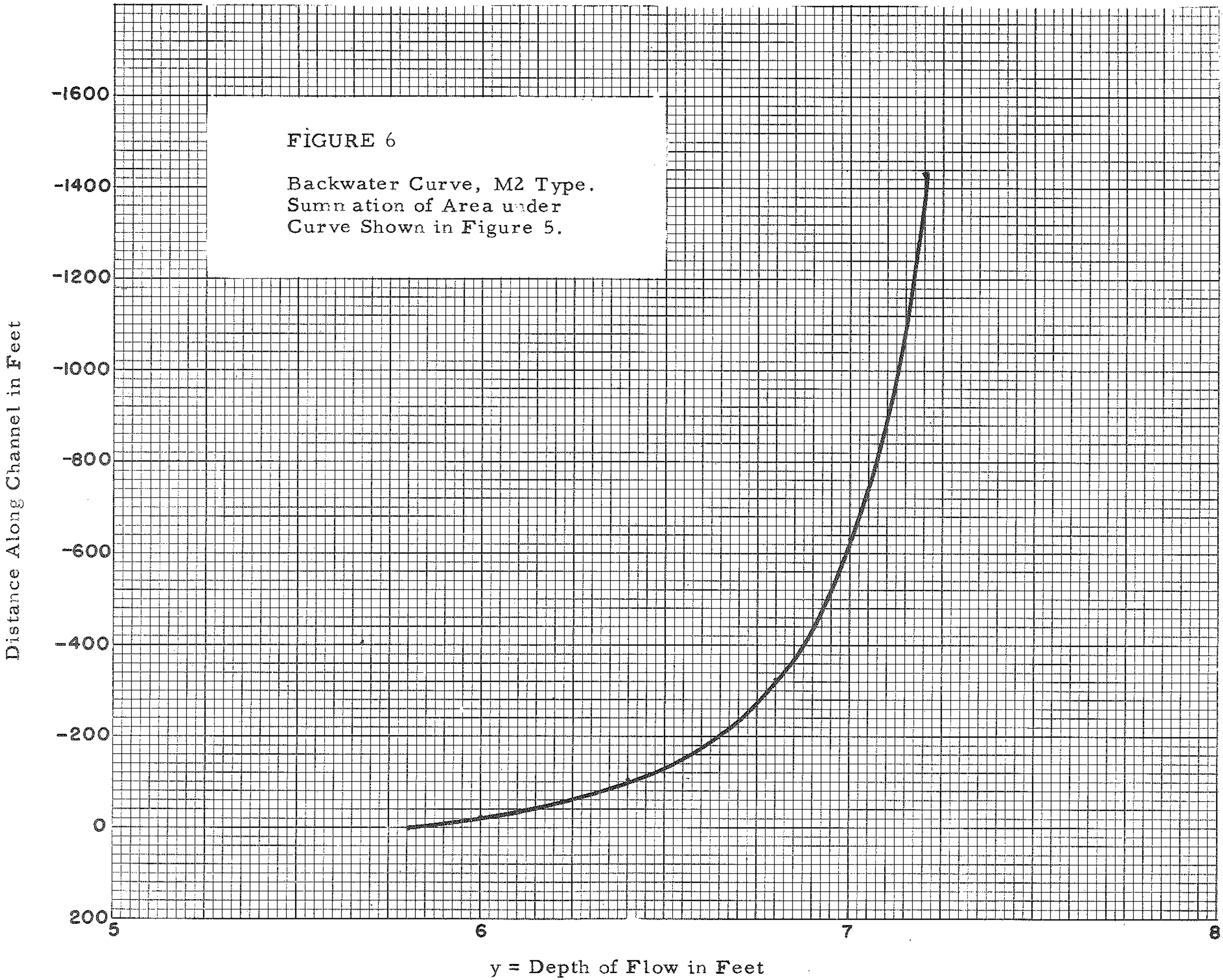


FIGURE 6

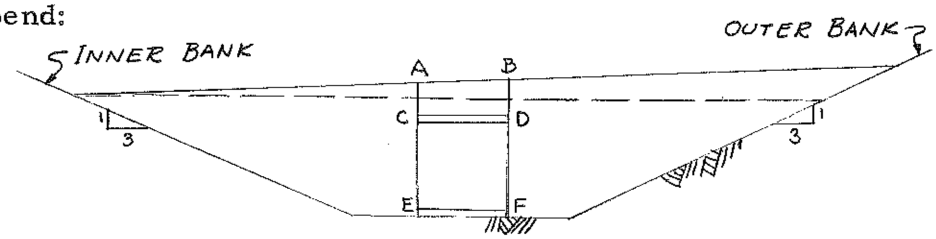
Backwater Curve, M2 Type. Summation of Area under Curve Shown in Figure 5.

II ANALYSIS OF THE FLOW AROUND THE BEND

Another question to arise concern the effect of the sharp bend in the new channel just downstream from the bridge. Some of the basic theories involved are included below, mainly for the purpose of pointing our the degree of confidence that can be placed in the results.

In a straight, uniform channel it is assumed that the transverse profile of the water surface is horizontal, or level. Some observers claim that a higher elevation of the water surface - convexity - exists in the middle of the stream; however, there has been no satisfactory proof to substantuate this theory.

But when conditions in an open channel are such that straight, uniform flow no longer exists, the transverse profile can no longer be horizontal. When there is a bend in the stream and the water moves in a curve, there must be an unbalanced force acting against the water in the direction of the center of the curvature. This is apparent in Newton's first law of motion, which states that matter in motion will move in straight lines unless deflected by the action of some unbalanced force. Below is a presentation of a typcial transverse section of the channel in the bend:



Consider the portion of the stream enclosed in the area designated ABFE. As this portion moves around the bend it is deflected toward the inside or toward the center of curvature of the moving water. This deflection is caused by the increased pressure on face BF over face AE. This pressure differential can exist only when the water surface at B is higher than at A, and can be calculated by the formula for centrifugal force, if the velocity of the moving water and the curvature of its path are known.

Formulas for the difference in water surface elevation between the inner and outer banks of a stream flowing around a curve at velocities less than critical can be derived as follows: assume that all parts of the element ABFE, as shown, are moving at the same velocity and in the same circular path around the center of curvature.

Let: b = breadth of the stream
r = radius of curvature of flow at the element ABFE,
r_1 = radius of curvature of the inner bank
r_2 = radius of curvature of the outer bank
d_r = distance AB
y = depth at element AD
dy = height B above A
V = velocity of water
w = weight of unit volume of water
R = radius of curvature at center of stream

The centrifugal force acting on the element AFBE is equal to the pressure difference of face BF over face AE, due to the height of the water at B above the surface at A. Considering the length of the section ABFE as unity, up and down stream, its volume is ydr, its weight is wydr, and its mass is (w/g) ydr. The excess pressure on the face BF is wydy.

$$F = w V^2 / gr \text{ (The formula for centrifugal force)}$$

then: $$\frac{wy \; V^2 \; dr}{gr} = wydy$$

and: $$dy = \frac{V^2 \; dr}{gr}$$

To integrate this equation it is necessary to state the values for the velocity at points all across the channel in terms of r, and formulas may be written under a variety of assumptions. A fairly good approximation of the difference in water surface elevation can be obtained by assuming that the velocity is constant at average, assuming r to be constant at the value for the center of the stream. From these assumptions then:

$$\text{Difference in elevation of the two banks} = \frac{V^2 b}{gR}$$

This approximation, however, will always give too small a value because the effect of the filaments with higher velocities more than offsets the effect of the slower filaments, since in the equation the velocity value is squared.

Another assumption, one that will give results which are somewhat closer to the actual conditions, is that the velocity is zero at each bank and maximum in the center of the stream; the variation between plotting out as a parabolic curve. In this case then:

Difference in water surface elevation =

$$\frac{V_m^2}{g}\left[\frac{20}{3}\,\frac{R}{b} - 16\,\frac{R^3}{b^3} + \left(\frac{4R^2}{b^2} - 1\right)^2 \log_e \frac{2R + b}{2R - b}\right]$$

Thus we have two assumptions which are intended to approximate

Thus, the results of both methods are virtually the same. The effect of the variable velocity distribution throughout a cross section would, probably, increase slightly the radius of curature actually followed by the moving water, especially for a short curve.

From the two solutions it is apparent that the bend has an effect upon the transverse profile of the new channel which gives a difference in elevation between the inner and outer banks of .5 feet. Since the normal depth of flow in the channel is only 7.28 ft and the channel provides for a depth of 25 ft, it may be concluded that the effect of the bend is negligible.

In other situations of a similar nature but with the velocity above critical, and/or in cases of more limited channel depth, the effect of the bend could bec ome highly significant. In any case, an analysis of flow around bends should be made. It is suggested that a textbook, such as Woodward and Posey; Flow in Open Channels, be consulted for flow problems of this nature.