BACKWATER AND DISCHARGE AT HIGHWAY CROSSINGS WITH MULTIPLE BRIDGES IN LOUISIANA AND MISSISSIPPI

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ILLUSTRATIONS

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Symbol	Definition	Units
K_c	Controlling conveyance used for computing flow through the constriction	ft^3/s
K_d	Conveyance of the spur dike cross section	ft^3/s
K_q	Portion of the approach conveyance, K_1, corresponding to the bridge width, b	ft^3/s
$L_{(i-j)}$	Distance between cross sections i and j	ft
L_{av}	Average streamline length in the approach reach	ft
L	Length of bridge abutment in direction of flow	ft
L_d	Length of spur dikes	ft
L_L	Length of interior embankment left of stagnation point	
L_w	Distance from approach section to upsteam side of constriction or the toe of the spur dikes when spur dikes are included on the bridge	ft
m	Channel-constriction ratio, $1 - K_q/K_1$	
m'	Geometric channel constriction ratio, $1 - b/B$	
n	Subscript denoting flow under natural conditions	
n	Manning's roughness coefficient	$ft^{1/6}$
Q	Total discharge	ft^3/s
q_i	Discharge through the i^{th} opening in a multi-opening crossing	ft^3/s
q^*_i	Channel resistance ratio for opening i	
V_i	Mean velocity at cross section i during constricted flow conditions, Q/A_i	ft/s
x	Horizontal distance from the intersection of abutment and embankment slopes to the location on upstream embankment having the same elevation as the water surface at section 1	ft
α_i, α_{in}	Energy coefficient at cross section i, under constricted and natural flow conditions, respectively	
β_i	Momentum coefficient at cross-section i	
Φ	Angle of skew; acute angle between the plane of the constriction and a line normal to the thread of the stream	
Θ	Acute angle between a wing wall and the plane of constriction	

SYMBOLS AND UNITS

Symbol	Definition	Units
a_i	Area of subsection i of a specified cross section	ft^2
A_i	Area of flow at cross section i	ft^2
A_{in}	Area of flow below the natrual water surface elevation	ft^2
A_j	Submerged cross-sectional area of piers or piles	ft^2
b	Width of bridge opening	ft
b_t	Width of bridge opening at the water surface	ft
C	Coefficient of discharge	
C_b	Backwater ratio, $h_1^*/\Delta h$	
C_I	Coefficient of discharge for equivalent base type opening (method I)	
d	Subscript denoting a variable measured at the cross section across the upstream toe of the spur dikes	
e	Eccentricity ratio based on conveyance distribution in approach reach	
E	Slope of the embankments, horizontal distance per unit vertical distance	
g	Acceleration of gravity	ft/s^2
h_i	Water-surface elevation at cross section i during constricted flow conditions	ft
h_{in}	Water-surface elevation at cross section i during natural flow conditions	
h_e	Head loss due to flow expansion between sections 3 and 4	ft
$h_{f(i-j)}$	Head loss due to friction between upstream and downstream cross sections i and j during constricted flow conditions	ft
$h_{f(i-j)n}$	Head loss due to friction between upstream and downstream cross sections i and j during natural flow conditions	ft
h_i^*	Total backwater or rise above the natural water surface caused by the constriction at a cross section	ft
h_s	Water-surface elevation at the stagnation points on the interior embankment	ft
Δh	Fall between section 1 and 3, h_1-h_3	
i	Subscript denoting cross-section number	
(i-j)	Subscript denoting a variable measured between an upstream (subscript i) and downstream (subscript j), cross section	
k_i	Conveyance of subsection i of a specified cross section	ft^3/s
k_{in}	Conveyance of subsection i during natural flow conditions	ft^3/s
K_i	Conveyance of cross section i	ft^3/s

FACTORS FOR CONVERTING INCH-POUND UNITS TO

INTERNATIONAL SYSTEM (SI) UNITS

Multiply	By	To Obtain
inch (in)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
foot per second (ft/s)	0.3048	meter per second (m/s)
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
mile (mi)	1.609	kilometer (km)

BACKWATER AND DISCHARGE AT HIGHWAY CROSSINGS WITH

MULTIPLE BRIDGES IN LOUISIANA AND MISSISSIPPI

by

B. E. Colson and Verne R. Schneider

ABSTRACT

Data were collected for nine floods in Mississippi and Louisiana at eight stream crossings having two to six separate bridge openings. Discharge through each bridge, water-surface profiles, valley cross sections, and bridge geometry were measured. The multiple openings were divided into equivalent single-opening cases by apportioning interior embankments in direct proportion to the area of openings on either side. Using existing procedures for computing discharge, the bias in computed discharge was 2 percent with a root mean square error of 18 percent.

Backwater was computed by two current U.S. Geological Survey methods that use the average flow path in the friction loss term for the approach. One method gave a root mean square error of 0.34 feet with a bias of -0.25 feet, suggesting that the method underestimates backwater. The other method gave a root mean square error of 0.39 feet with a bias of -0.03 feet. The results indicate that the method developed for single-opening highway crossings can be applied to the multiple bridge crossings.

INTRODUCTION

The U.S. Geological Survey, in cooperation with the Highway Departments of Mississippi, Alabama, and Louisiana, completed in 1976 a study of backwater computations at bridges in the wide, densely vegetated flood plains that are common in the Gulf Coastal Plain areas of the three States. The results of studies for the single-opening bridge case (only one bridge in a highway embankment crossing a stream) have been published by Schneider and others (1976). At the conclusion of that study, it was apparent that additional field data were needed to extend the results to the multiple-bridge system (more than one bridge in a highway embankment crossing a stream). In 1977, the Survey in cooperation with the Mississippi State Highway Department began a study to collect the necessary field data to test the method at multiple-opening bridges.

Knowledge of the backwater caused by the constriction formed by multiple bridges of highway stream-crossings is essential in the design of bridge openings. Peak discharge of floods is computed from elevation change across the embankment and the geometry of the channels and bridges.

PURPOSE AND SCOPE

The principle objective of this project was to measure the backwater and discharge distribution for multiple bridges. These data have been used to determine if the methods developed by Schneider and others (1976) for single-opening highway crossings could be applied to multiple bridges. In addition, the method developed by Tracy and Carter (1955) and Cragwall (1958) was modified to use the procedure proposed by Schneider and others (1976) to calculate friction losses in the approach reach. This modified procedure was then also tested.

METHODS FOR COMPUTING DISCHARGE

Kindsvater and Carter (1955) conducted the analytical and experimental work leading to the development of the Survey method of computing discharge through width constrictions. The discharge relationship is derived from the energy and continuity balance between an approach section and the most contracted section designated sections 1 and 3, respectively, in figure 1,

$$Q = CA_3\sqrt{2g\left(\Delta h_1 + \alpha_1\frac{V_1^2}{2g} - h_{f(1-3)}\right)} \qquad (1)$$

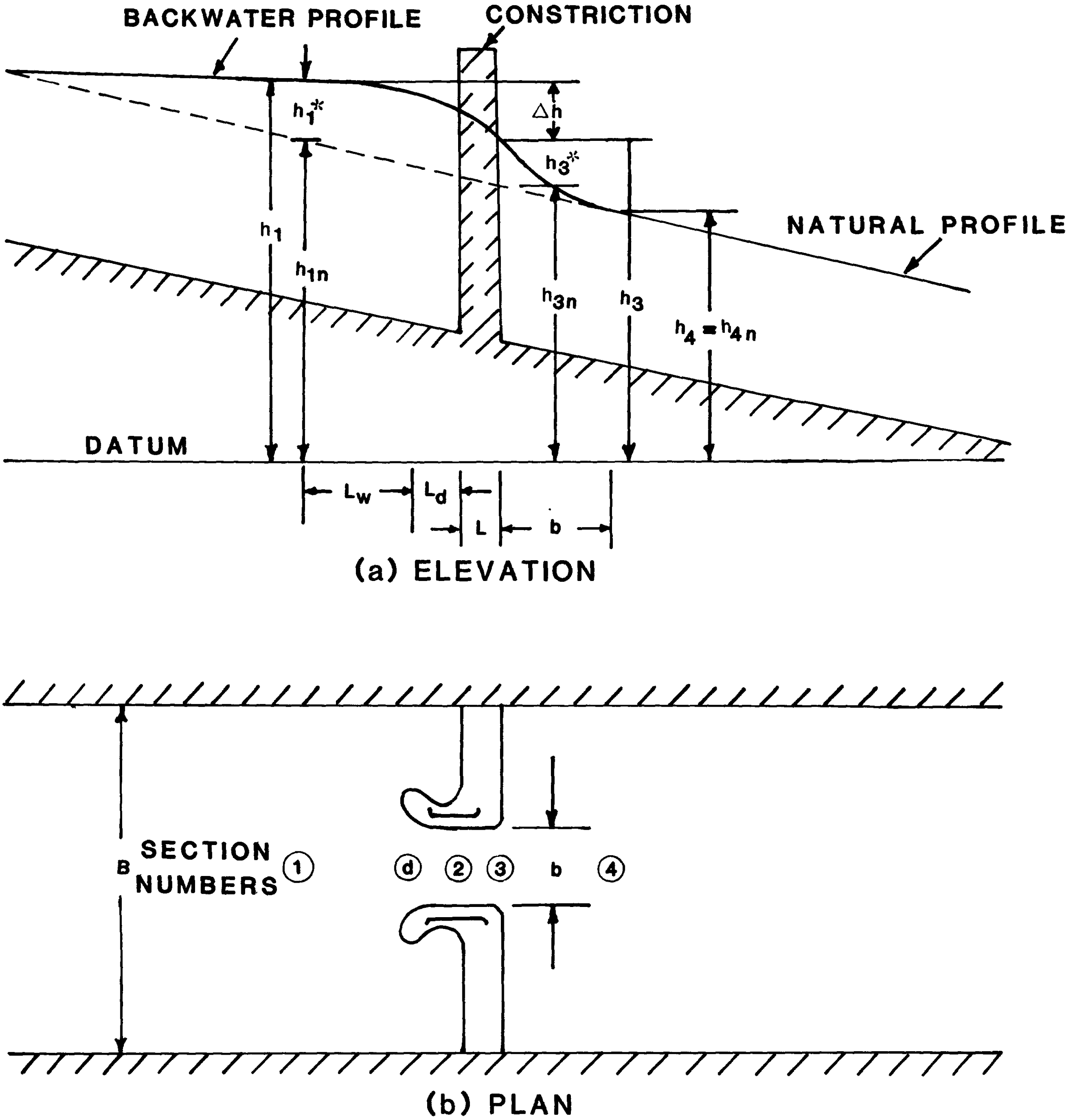

Figure 1.-- Definition sketch of the variables used in computing backwater and discharge by the proposed method.

where

- Q is the total discharge in cubic feet per second.
- A is the flow area at section 3 below the measured water-surface elevation in square feet.
- Δh is the difference in water-surface elevation between sections 1 and 3 in feet.
- α is the energy coefficient at cross section 1.
- h_f is the head loss due to friction in feet.
- C is the discharge coefficient.
- V is the mean velocity at cross section 1 in feet per second.

Laboratory investigations were conducted to define the discharge coefficients for four typical abutment geometries. The coefficient C represents a combination of (1) a coefficient of contraction, (2) a coefficient which takes into account the eddy losses, and (3) the velocity head coefficient, α_3, for the contracted section (Kindsvater and others, 1953). The procedures for selecting the discharge coefficients are discussed by Matthai (1967).

The energy loss (Matthai, 1967) due to friction is the product of the geometric mean of the energy slopes at the end cross sections of the reach times the distance between the sections. The energy loss due to friction is obtained from the equation

$$h_{f(1-3)} = \frac{L_w Q^2}{K_1 K_3} + \frac{L Q^2}{K_3{}^2} \qquad (2)$$

where

- L_w is the length of the approach reach.
- L is the length of the bridge opening.
- K_1 is the total conveyances of section 1.
- K_3 is the total conveyances of section 3.

When the approach reach has dense brush and the reach under the bridge is relatively clear, the weighted conveyance computed from the conveyances of sections 1 and 3 will be too high. A more accurate approximation of the friction loss may be obtained if $L_w(Q^2/K_1K_q)$ is substituted for the first term in equation 2. K_q is that part of the approach section conveyance corresponding to the projected bridge opening b.

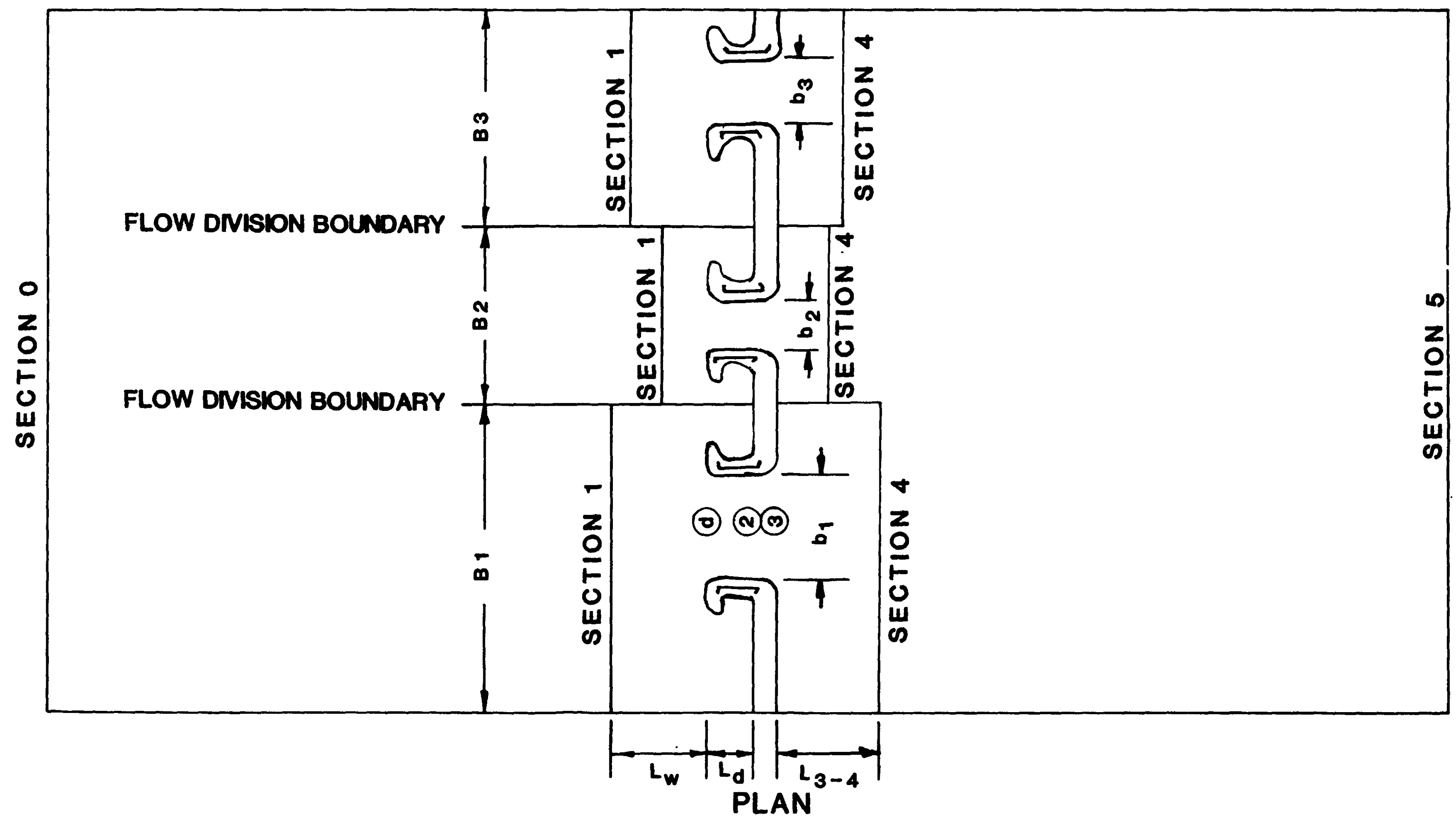

Figure 2.-- Definition sketch showing variables used in computing backwater and discharge at multiple-opening constriction.

embankments) are located in direct proportion to the gross flow area of the opening on either side, the larger length of embankment being assigned to the larger opening. From the flow division points, lines are projected parallel to the flow from the embankment upstream to the approach section to form flow boundaries for each opening. This procedure defines a separate approach for each individual bridge. A channel resistance ratio (q*) based on the approach geometry is computed by iterative procedures from the equation:

$$q_i^* = 1 + 0.46 \log \frac{k_i/a_i}{K_1 A_1} \qquad (8)$$

The discharge q through each bridge is computed by the formula:

$$q_i = q_i^* \frac{C_i A_{3i}}{\Sigma (CA_3)_i} Q \qquad (9)$$

where i = i...n.
where

q_i	is the discharge for the individual bridge.
q_i*	is the channel resistance ratio for the individual bridge opening.
k_i	is the approach channel conveyance for the individual bridge opening.
A_{3i}	is the area for the individual bridge opening.
a_i	is the approach area for the individual bridge opening.
K_1	is the total approach conveyance.
A_1	is the total approach area.
C_i	is the coefficient of discharge for the individual bridge opening.
$\Sigma(CA_3)_i$	is the sum of the live flow areas for a particular site.

Each bridge opening with the associated flow boundaries can be treated as an equivalent single-opening crossing for computation of backwater.

Laboratory investigations by Davidian and others (1962) have shown that discharge coefficients developed for bridges at single-opening constrictions are valid for multiple-bridge constrictions. These coefficients have been well defined for the four most common types of bridge openings.

Lee (1976) tested the method of Davidian and others (1962) with data from three sites in Louisiana. Two of the sites had two bridges and one site had three bridges. Peak discharge was measured at all bridges so that the actual flow distribution was known. At the three-bridge site and at one two-bridge site, the discharge distributed using equations 8 and 9 agreed within 6 percent of the measured discharge. The main channel bridge at one two-bridge site was in error by 20 percent which induced an error of 100 percent at the small relief bridge.

The most recent method developed by the Survey for computing backwater at single-opening constriction is described by Schneider and

When spur dikes are included on the bridge, the energy loss due to friction is

$$h_{f(1-3)} = \frac{L_w Q^2}{K_1 K_d} + \frac{L_d Q^2}{K_d K_3} + \frac{L Q^2}{K_3^{\,2}} \qquad (3)$$

where

L_d is the length of the spur dike in the direction of flow in feet.

K_d is the conveyance of the cross section at the toe of the spur dikes.

Schneider and others (1976) modified the friction loss term for $\frac{L_w Q^2}{K_1 K_3}$ (equation 2) to more accurately estimate the friction losses in the approach reach. In the modified term $\frac{L_{av} Q^2}{K_1 K_c}$, L_{av} is the average flow length in the approach reach and K_c, the controlling conveyance at the downstream end of the approach reach, is the smaller of the conveyances K_3 and K_q.

Matthai (1967) recommends that the conveyance that best represents the conditions at the end of the reach be used to compute friction losses. An examination of the data reported by Schneider and others (1976) indicates that K_c was the representative conveyance. Studies indicate that care in selecting the representative conveyance is important because it significantly affects the magnitude of the friction loss and therefore the computation of discharge and backwater.

METHODS FOR COMPUTING BACKWATER

Tracy and Carter (1955) defined backwater, h_1^*, as one component of the fall, Δh, between sections 1 and 3 as illustrated in figure 1. The fall was resolved into three components.

$$\Delta h = h_1^* - h_3^* + h_{f(1-3)n} \qquad (4)$$

where

h_1^* is the increase in the water-surface at section 1 in feet.

h_3^* is the backwater at section 3 in feet.

h_f is the head loss due to friction in feet.

n is a subscript denoting flow under natural conditions.

Tracy and Carter (1955) defined h_3^* as positive when the contricted water-surface elevation at section 3 was below the natural water-surface elevation. After finding that the constricted water-surface elevation at section 3 could be above the natural elevation, Schneider and others (1976) adopted the convention of positive ($h_3^* = h_3 - h_{3n}$) when the constricted water-surface elevation is above natural. This convention is followed in this report.

Equation 4 is divided by Δh,

$$\frac{h_1^*}{\Delta h} = 1 + \frac{h_3^*}{\Delta h} - \frac{h_{f(1-3)n}}{\Delta h} \qquad (5)$$

The ratio, $\frac{h_1^*}{\Delta h}$, is defined as the backwater ratio, C_b. Equation 1 is solved for Δh and $V_3 = Q/A_3$ is substituted from the continuity equation,

$$h_1^* = C_b\left(\frac{V_3^2}{2gC^2} + h_{f(1-3)} - \alpha_1\frac{V_1^2}{2g}\right) \qquad (6)$$

The backwater ratio is a function of the channel-constriction ratio, Manning's n value, and the constriction geometry. Equation 6 is substituted into equation 4 and is solved for h_3^*.

$$h_3^* = -(1-C_b)\left(\frac{V_3^2}{2gC^2} + h_{f(1-3)} - \alpha_1\frac{V_1^2}{2g}\right) + h_{f(1-3)n} \qquad (7)$$

The procedures for computing backwater by this method are reported by Cragwall (1958).

A laboratory study on backwater at multiple bridge systems was made by Davidian and others (1962). The objective was to develop methods for the computation of discharge through multiple-opening constrictions, prediction of maximum backwater, and prediction of division of flow through the several openings.

Current one-dimensional methods of computing backwater at multiple-opening highway crossings all rely on reducing the bridge openings to equivalent single openings. This is accomplished by establishing pseudo-fixed boundaries between separate openings.

The method of apportioning flow through multiple bridges, as given by Davidian and others (1962), requires three items of data: (1) stage-discharge relation at the site, (2) a valley cross-section, and, (3) locations and geometry of all bridges. A definition sketch for flow through a typical multiple-opening constriction is shown in figure 2. The flow division points (along the interior others (1976). The natural profile is computed using a standard step-backwater procedure (Chow, 1959), where friction losses are

$$h_{f(i-j)n} = \frac{L_{(i-j)}\ Q^2}{K_{in}K_{jn}} \tag{10}$$

The constricted profile is also computed using a standard step-backwater procedure. The approach section is located one bridge-opening length upstream (fig. 2). The friction losses are computed using the average flow length in the approach. Section 4 is located one bridge-opening length downstream from the highway crossing. The water-surface elevation at section 4 is assumed to be at the natural elevation. An expansion loss term is applied between sections 3 and 4. The constricted water-surface profile is computed by iteration, until successive estimates agree within a preselected tolerance. An example of this method is given by Schneider and others (1976).

For computation of friction losses, Schneider and others(1976) divided the approach reach into as many as three separate subreaches. The constriction subreach is considered to be the length, $L_{(2-3)}$ of the abutment in the direction of flow. The friction loss is computed as:

$$h_{f(2-3)} = L_{(2-3)}\left(\frac{Q}{K_3}\right)^2 \tag{11}$$

For a bridge without spur dikes,

$$h_{f(1-2)} = L_{av}\frac{Q^2}{K_1K_c} \tag{12}$$

In this case (no spur dikes)

$$h_{f(1-3)} = h_{f(1-2)} + h_{f(2-3)} \tag{13}$$

When spur dikes are present, the approach reach is further divided into two subreaches.

$$h_{f(1-d)} = \frac{L_{av}Q^2}{K_1K_c} \tag{14}$$

and

$$h_{f(d-2)} = \frac{L_{(d-2)}Q^2}{K_dK_3} \tag{15}$$

So that, when spur dikes are present

$$h_{f(1-3)} = h_{f(1-d)} + h_{f(d-2)} + h_{f(2-3)} \tag{16}$$

In the flow expansion reach, the flow is assumed to be at natural elevation one-bridge-width downstream from section 3. Therefore, the area and conveyance of section 4 are computed at the natural elevation.

The friction losses are estimated from equation 17 using the straight-line distance between sections,

$$h_{f(3-4)} = \frac{bQ^2}{K_c K_{4n}} \tag{17}$$

where the controlling conveyance, K_c, is the smallest of the convenyances, K_q, K_3, or K_d.

Schneider and others (1976), following a suggestion by Henderson (1966, page 277, problems 7.1 and 7.2), present an approximate solution of the momentum, energy, and continuity equation for expansion losses of an ideal abrupt expansion in open-channel flow:

$$h_e = \frac{Q^2}{2gA_4{}^2}\left[\left(2\beta_4 - \alpha_4\right) - 2\beta_3\frac{A_4}{A_3} + \alpha_3\left(\frac{A_4}{A_3}\right)^2\right] \tag{18}$$

Where

α_i is the energy coefficient

β_i is the momentum coefficient

It can be shown that alpha and beta at section 3 are related to the bridge geometry and can be estimated from the bridge coefficient

$$\alpha_3 = \frac{1}{C^2} \tag{19}$$

$$\beta_3 = \frac{1}{C} \tag{20}$$

Alpha and beta at section 4 can be computed from the cross section properties as

$$\alpha 4 = \frac{\Sigma\ (k_i{}^3/a_i{}^2)}{K_4^3/A_4^2} \tag{21}$$

and

$$\beta 4 = \frac{\Sigma\ k_i{}^2/a_i}{K_4^2/A_4} \tag{22}$$

where

k_i is the subsection conveyance.

a_i is the subsection area.

K_4 is the cross section conveyance.

A_4 is the cross section area of section 4.

Schneider and others (1976) tested the method developed by Bradley (1970). They found that for the wide flood plains backwater was undercomputed significantly. The friction losses appear to be underestimated by the method. Hence this procedure was not tested in this report.

Several other research attempts have been made to develop flow models. The latter efforts have been primarily in the application of two-dimensional finite-element flow models, Lee (1980) Lee and Bennett, (1981), and Lee and others (1982). These are promising in that they allow much more flexible application of hydraulic theory. Present two-dimensional models require relatively large amounts of manpower and computer time. However, the probable successful efforts to automate the data handling process, will make the 2D model more accessible.

DATA COLLECTION

Field data were collected using the procedures outlined by Benson and Dalrymple (1967) and Matthai (1967). In general, the data were collected and reported in the same way as outlined by Schneider and others (1976). Data include peak discharge, valley cross sections, water-surface elevations, bridge geometry, and Manning's roughness coefficient, n. High-water-mark elevations, valley cross-section ground elevations, highway profile, and bridge geometry were surveyed using standard leveling techniques. Highwater-mark elevations were measured with a resolution of 0.01 ft and ground-surface elevations and highway profile to 0.1 ft. The site location and flood date are contained in table 1. A summary of site data is in table 2.

The total data set as reduced and assembled generally includes the following:

1. Summary
    - A. Location of site
    - B. Description of site
    - C. Description of flood
    - D. Description of discharge measurement
    - E. Field survey
    - F. Computations
    - G. Results of computations
    - H. Datum
2. Topographic map
3. Aerial photographs
4. Highway plans
5. Flood-frequency curve
6. Stage-discharge relation
7. Discharge measurement notes
8. Velocity distribution and measuring section diagram
9. Plan of roadway crossing and location of high-water marks
10. Bridge geometry
11. List of high-water marks
12. Water-surface profile along highway embankments
13. Valley cross sections
14. Flood profiles
15. Field notes
16. Computer printouts
17. Stereoscopic slides documenting flood-plain roughness

Flood No.	Station name and location	Date of flood peak
1	Thompson Creek at Strengthford, Miss., lat 31°36'59", long 88°52'53" in sec. 34, T. 8 N., R. 9 W., St. Stephens meridian, on county highway 0.3 mile east of Strengthford, Wayne County, Miss.	03-03-71
2	Sipsey Creek near Forest, Miss., lat 32°32'33", long 89°21'32" in sec. 15, T. 8 N., R. 9 E., Choctaw meridian, on State Highway 21, 15 miles northeast of Forest, Scott County, Miss.	10-17-75
3	Big Black River near Winona, Miss., lat 33°22'58", long 89°36'52", in sec. 36, T. 18 N., R. 6 E., Choctaw meridian, on State Highway 407, 9 miles southeast of Winona, Montgomery County, Miss.	03-04-77
4	Big Black River near Canton, Miss., lat 32°42'26", long 90°05'39", in sec. 16, T. 10 N., R. 5 E., Choctaw meridian, on State Highway 16, 6.8 miles northwest of Canton, Madison County, Miss.	03-08-77
5	Big Black River near Canton, Miss., lat 32°42'26", long 90°05'39", in sec. 16, T. 10 N., R. 5 E., Choctaw meridian, on State Highway 16, 6.8 miles northwest of Canton, Madison County, Miss.	04-14-79
6	East Fork Amite River near Peoria, Miss., lat 31°05'54", long 90°43'00", in sec. 32, T. 2 N., R. 5 E., Washington meridian, on State Highway 584, 4 miles southwest of Peoria, Amite County, Miss.	04-22-77
7	Castor Creek near Grayson, La., lat 32°04'55", long 92°12'24", in sec. 30, T. 13 N., R. 3 E., Louisiana meridian, on Louisiana Highway 126, 6.5 miles west of Grayson, Caldwell Parish, La.	12-07-71
8	Bayou de Loutre near Farmerville, La., lat 32°52'25", long 92°22'40", in sec. 20, T. 22 N., R. 1 E., Louisiana meridian on Louisiana Highway 549, 7 miles north of Farmerville, Union Parish, La.	03-15-73
9	Sixmile Creek near Sugartown, La., lat 30°48'52", long 92°55'34", in sec. 12, T. 3 S., R. 6 W., Louisiana meridian on Louisiana Highway 112, 6.5 miles east of Sugartown, Allen Parish, La.	03-25-73

a Elliptical

and flood date

Number of bridge openings	Total peak discharge (ft^3/s) (rounded)	Recurrence interval (years)	Average flood plain width (ft)	Channel slope (ft/mi)	Dike type	Manning's roughness coefficient
2	2170	2	2000	5.5	--	0.20
2	7510	7	2500	4.2	--	0.08-0.16
6	23300	5	9000	2.9	--	0.06-0.15
4	30300	7	10000	1.5	a	0.06-0.15
4	85800	100	10000	1.5	a	0.06-0.15
3	27000	100	5000	6.1	a	0.08-0.15
2	5850	2	2100	1.8	--	0.14
3	4900	2	2300	4.5	--	0.11
2	12900	12	3300	5.0	--	0.13-0.18

Table 2.--Summary

Site	Bridge	Abut. type	E	m	m'	b ft	b_t ft
1	MC	4	4:1	0.77	0.57	192	192
	RO-1	4	4:1	.94	.96	56	56
2	MC	4	2:1	.70	.92	96	96
	RO-1	4	2:1	.96	.97	39	39
3	MC	4	2:1	.50	.82	374	374
	RO-1	4	2:1	.66	.86	149	149
	RO-2	4	2:1	.68	.77	185	185
	RO-3	4	2:1	.66	.68	379	379
	RO-4	4	2:1	.68	.72	303	303
	RO-5	4	2:1	.80	.89	114	114
4	MC	3	2:1	.21	.72	649	653
	RO-1	3	2:1	.83	.79	188	191
	RO-2	3	2:1	.84	.91	220	222
	RO-3	3	2:1	.82	.88	344	368
5	MC	3	2:1	.35	.70	651	669
	RO-1	3	2:1	.84	.83	206	225
	RO-2	3	2:1	.86	.89	223	235
	RO-3	3	2:1	.85	.89	377	396
6	MC	3	2:1	.58	.86	343	355
	RO-1	3	2:1	.83	.81	169	178
	RO-2	3	2:1	.88	.88	176	178
7	MC	4	2:1	.32	.60	474	474
	RO-1	4	2:1	.83	.85	132	132
8	MC	4	2:1	.29	.72	95	95
	RO-1	4	2:1	.84	.87	113	113
	RO-2	4	2:1	.64	.93	76	76
9	MC	3	3:1	.25	.64	606	606
	RO-1	3	3:1	.79	.90	162	162

MC Main channel
RO Relief opening
a Elliptical

of site data

L_w ft	L_{av} ft	Dike type	L_d ft	L ft	X ft	e	Θ	C	C_I
192	213	-	-	37	-	.41	53°	0.74	0.64
56	145	-	-	37	-	.32	53°	.85	.70
96	192	-	-	28	-	.64	45°	.73	.67
39	115	-	-	29	-	.10	45°	.79	.73
374	527	-	-	27	-	.55	35°	.69	.67
149	231	-	-	22	-	.90	35°	.68	.65
185	241	-	-	22	-	1.00	35°	.68	.65
379	447	-	-	22	-	.35	35°	.65	.65
303	376	-	-	22	-	.57	35°	.65	.64
114	194	-	-	22	-	.75	35°	.68	.64
653	805	a	150	40	18	0	-	.89	.73
191	252	a	100	40	13	.03	-	.90	.67
222	411	a	120	40	11	.63	-	.87	.65
368	568	a	120	40	12	.74	-	.83	.64
669	788	a	150	40	8	0	-	.81	.71
225	297	a	100	40	3	.15	-	.89	.68
235	379	a	120	40	1	.54	-	.88	.67
396	641	a	120	40	2	.79	-	.83	.67
355	532	a	175	27	2	.67	-	.88	.66
178	233	a	170	27	2	.76	-	.90	.66
178	290	a	150	27	2	.58	-	.91	.66
474	531	-	-	31	-	.64	45°	.79	.70
132	198	-	-	31	-	.42	45°	.72	.63
95	118	-	-	27	-	.04	45°	.85	.79
113	181	-	-	27	-	.48	45°	.73	.64
76	161	-	-	27	-	.30	45°	.78	.69
606	697	-	-	34	11	.66	-	.79	.74
162	284	-	-	34	11	1.00	-	.70	.66

Peak Discharge Measurement

Peak discharge was measured by current meter at the flood peak or was obtained from stage-discharge relations. The stage-discharge relations were extrapolated several feet at some sites. Available data on the volume of runoff and the duration of the peak indicated that steady flow existed throughout the reach during the peak at most sites. When necessary, flow over the highway embankment was computed using the procedure described by Hulsing (1967). At these sites the amount of flow over the highway embankment was small compared to the total discharge.

Valley Cross Sections

At least four valley cross sections were selected. Each cross section was approximately one valley width apart. At each site at least two valley cross sections were located upstream and two valley cross sections down-stream of the highway embankment. In addition, an approach cross section was surveyed approximately one-bridgeopening width upstream from the constriction. Additional cross sections were surveyed as required to define road fills, pipeline crossings, and other features affecting the flood profile.

Locations for the valley cross sections were selected using a plot of the flood profiles obtained along each edge of the flood plain and by inspection of topographic maps. The cross sections were drawn on the map at approximately valley-width intervals and were alined perpendicular to the assumed direction of flow. Identifiable landmarks were used to locate the cross sections in the field, where they were oriented to the correct azimuth by compass. The survey datum was established at the bridge. A base line was surveyed from the highway to establish horizontal and vertical control for the cross section.

Water-Surface Elevation

Water-surface elevations were determined by high-water marks recovered along the cross sections and base lines. Water surfaces also were marked along the upstream and downstream sides of the embankment during the peak discharge measurement. Additional high-water marks were selected at random locations upstream and downstream of the bridge to describe the lines of constant watersurface elevation in the approach and flow-expansion reaches.

Bridge Geometry

Bridge geometry data, collected according to the procedures discussed by Matthai (1967), included abutment slope, bridge cross section, and pier and spur dike geometry and location.

Manning's Roughness Coefficient

An attempt was made to field-select Manning's roughness coefficient, n. Selection is usually based on experience obtained by computing water-surface profiles in channels where peak discharge and water-surface elevations are known (n-verification studies) and by studying stereoscopic slides that document features affecting the magnitude of n. Although n was selected by experienced personnel and, at most sites, by the same individual for consistency, neither published n-verification studies nor stereoscopic slides were available for comparative purposes. Therefore, the field-selected n's were adjusted using the measured discharge and the measured water-surface profile downstream of the bridge. The n-values were adjusted so that the water-surface profile computed using a step-backwater procedure (Shearman, 1976) agreed with the measured profile downstream of the bridge. Cross sections were subdivided for major changes in geometry and roughness which persisted throughout the reach and n selected for each subdivision. When the reach included an open field which extended approximately one-half the distance upstream and downstream to the next cross sections, the reach was subdivided and n selected for the open-field condition. Composite n-values were used where frequent roughness changes occurred that did not affect the entire reach.

ANALYSIS OF DATA

Backwater is defined as the difference between the natural and the constricted water-surface elevation. The natural (unconstricted) profile prior to construction of the highway was not available for any of the sites, and was therefore, computed using standard step-backwater techniques (Chow, 1959).

The constricted water-surface elevations were obtained at the approach for each bridge opening by interpolation of the profiles defined by high water marks surveyed along each edge of the valley. These elevations were compared with the stagnation elevations observed at each edge of the valley and on the interior embankments.

Backwater was computed by two methods named method I and method II. Method I was the technique developed by Tracy and Carter (1955) and reported by Cragwall (1958). The method was modified in this study so that the average flow distance; L_{av}, is used in equation 2 and 3 in place of L_w. Because method I, as developed, applies only to sites without spur dikes, it was not applied to sites 4, 5, and 6.

Method II is the procedure developed by Schneider and others (1976). In both methods, the approach is divided into equivalent single openings using the methods developed by Davidian and others (1962). Data for computing backwater and discharge are summarized in table 3. The results for discharge and backwater computation are summarized in table 4.

Computation of Natural Profile

In the step-backwater procedure, peak discharge, cross-section geometry, and n-values were used to compute the natural profile. The water surface profile was defined by highwater marks surveyed along each edge of the valley sufficiently far in each direction to

Table 3.--Summary of data for computing backwater and discharge

Site	Bridge	Q	h_{in}	A_{1n}	K_{1n}	α_{1n}	h_{3n}	A_{3n}	K_{3n}
		ft/s	ft	ft^2	ft^3/s		ft	ft^2	ft^3/s
1	MC	1440	215.83	2410	36300	1.00	215.63	860	29500
	RO-1	727	215.60	2510	30500	1.00	215.49	246	7910
2	MC	5870	371.63	6060	232000	1.00	371.55	1270	55900
	RO-1	1640	371.58	4360	89000	1.00	371.55	295	9620
3	MC	9700	302.05	9840	429000	1.00	301.90	3050	209000
	RO-1	2610	301.76	5200	148000	1.00	301.70	1120	66900
	RO-2	2120	301.58	3360	85900	1.00	301.50	763	36000
	RO-3	4350	301.36	6260	185000	1.00	301.20	2080	185000
	RO-4	3340	301.12	4460	112000	1.00	301.00	1770	97300
	RO-5	1190	300.91	1070	11000	1.00	300.86	567	29300
4	MC	16000	189.48	13000	854000	4.90	189.25	5960	559000
	RO-1	2130	188.87	1980	33200	1.00	188.80	494	20800
	RO-2	6130	188.96	9400	231000	1.00	188.80	1390	943000
	RO-3	6050	188.52	7670	159000	1.00	188.40	1480	80000
5	MC	35200	193.16	20800	1453000	4.15	192.61	8170	844000
	RO-1	10500	193.02	7580	255000	1.00	192.94	1380	91400
	RO-2	18100	193.04	17000	678000	1.00	192.96	2290	199000
	RO-3	22000	193.12	22600	796000	1.00	192.98	3240	253000
6	MC	14200	267.56	13200	551000	2.64	267.30	3120	188000
	RO-1	7470	266.93	5570	184000	1.00	266.80	1300	59300
	RO-2	5340	266.70	5760	150000	1.00	266.56	1020	42500
7	MC	4890	13.06	5800	183000	1.00	12.68	2570	103000
	RO-1	964	13.17	2700	61000	1.00	12.80	1280	65900
8	MC	2420	44.76	1440	50900	1.00	44.61	815	62300
	RO-1	1600	44.80	3910	141000	1.00	44.60	659	43200
	RO-2	878	44.82	2500	58600	1.00	44.62	325	18700
9	MC	11100	16.34	8580	475000	4.03	15.88	3990	282000
	RO-1	1850	16.04	7890	183000	1.00	15.88	724	44100

MC Main channel
RO Relief opening

Table 3.--Summary of data for computing backwater and discharge--Continued

Site	Bridge	h_1 ft	A_1 ft^2	K_1 ft^3/s	α_1	K_q	h_s ft	A_d ft^2	K_d ft^3/s
1	MC	216.67	3120	55200	1.00	12900	n.a	-	-
	RO-1	216.68	4400	68300	1.00	3650	n.a.	-	-
2	MC	373.45	8160	362000	1.97	109000	373.16	-	-
	RO-1	373.45	6870	187000	1.00	7890	373.18	-	-
3	MC	302.75	11200	504000	4.76	256000	302.65	-	-
	RO-1	302.50	5980	187000	1.00	71000	302.45	-	-
	RO-2	302.30	3930	112000	1.00	37200	302.25	-	-
	RO-3	302.12	7170	231000	1.00	81500	302.00	-	-
	RO-4	301.85	5260	147000	1.00	47300	301.76	-	-
	RO-5	301.65	1850	26400	1.00	5450	301.58	-	-
4	MC	190.10	14400	941000	4.90	757000	190.09	5240	548000
	RO-1	190.05	3050	67700	1.00	11300	190.04	508	19300
	RO-2	189.80	11400	319000	1.00	53200	189.84	3110	301000
	RO-3	189.50	10400	237000	1.00	45900	189.56	2330	172000
5	MC	194.65	24100	1746000	3.84	1164000	194.32	8140	884000
	RO-1	194.65	9540	373000	1.00	60000	194.46	2070	173000
	RO-2	194.64	20400	914000	1.00	132000	194.54	4970	614000
	RO-3	194.64	27800	1110000	1.00	168000	194.56	4550	494000
6	MC	269.30	17400	792000	2.39	335000	268.96	3430	292000
	RO-1	269.30	7700	315000	1.00	54900	268.48	2120	151000
	RO-2	269.30	9350	325000	1.00	39200	268.04	1680	110000
7	MC	13.13	5880	187000	1.00	138000	13.10	-	-
	RO-1	13.17	2700	61000	1.00	10500	13.18	-	-
8	MC	45.12	1560	58200	1.00	48800	45.08	-	-
	RO-1	45.16	4230	161000	1.00	25200	45.12	-	-
	RO-2	45.18	2890	74900	1.00	18800	45.14	-	-
9	MC	16.65	9100	511000	4.00	382000	n.a.	-	-
	RO-1	16.50	8590	214000	1.00	46600	n.a.	-	-

MC Main channel
RO Relief opening
n.a. not available

Table 3.--Summary of data for computing backwater and discharge--Continued

Site	Bridge	h_3 ft	A_3 ft^2	K_3 ft^3/s	A_j ft^2	h_4 ft	A_4 ft^2	K_4 ft^3/s	α_4	β_4
1	MC	215.80	892	31100	35.5	213.87	2110	25700	1.00	1.00
	RO-1	215.66	255	8360	11.8	213.88	3230	39800	1.00	1.00
2	MC	371.80	1300	56700	115	371.21	4390	129000	2.62	1.40
	RO-1	371.77	303	9950	26.9	371.28	2860	55200	1.00	1.00
3	MC	302.00	3090	212000	118	301.47	9620	358000	3.64	1.56
	RO-1	301.95	1150	69900	57.9	301.39	3890	96100	1.00	1.00
	RO-2	301.60	781	37200	38.4	301.18	3790	100000	1.00	1.00
	RO-3	301.34	2130	114000	108	300.96	5410	153000	1.00	1.00
	RO-4	301.09	1800	99400	87.0	300.84	7100	229000	1.00	1.00
	RO-5	300.89	570	29600	24.4	300.72	4270	120000	1.00	1.00
4	MC	189.48	6110	578000	325	188.97	11900	740000	5.35	2.11
	RO-1	188.90	513	21900	12.8	188.70	4720	130000	1.00	1.00
	RO-2	188.89	1410	96300	34.0	188.57	11600	332000	1.00	1.00
	RO-3	188.47	1510	82000	37.7	188.33	22500	803000	1.00	1.00
5	MC	192.87	8350	868000	413	192.65	23500	1367000	5.80	2.03
	RO-1	192.66	1310	85400	31.6	192.67	11000	451000	1.00	1.00
	RO-2	192.67	2220	189000	52.9	192.69	19300	830000	1.00	1.00
	RO-3	192.75	3150	242000	76.2	192.75	37600	1861000	1.00	1.00
6	MC	267.34	3130	189000	77.8	267.04	17600	666000	2.22	1.28
	RO-1	266.88	1310	60200	61.8	266.71	5810	204000	1.00	1.00
	RO-2	266.76	1050	44600	49.1	266.33	5130	125000	1.00	1.00
7	MC	12.72	2590	104000	129	12.60	5390	164000	1.00	1.00
	RO-1	12.80	1280	69000	58.8	12.60	1720	30100	1.00	1.00
8	MC	44.75	828	63600	38.5	44.42	1660	43500	1.00	1.00
	RO-1	44.69	669	44200	31.4	44.42	3360	79600	1.00	1.00
	RO-2	44.60	323	18500	13.9	44.42	1980	33500	1.00	1.00
9	MC	15.85	3980	280000	227	15.44	8820	226000	1.00	1.00
	RO-1	15.85	677	42800	43.0	15.74	3920	58800	1.00	1.00

MC Main channel
RO Relief opening

Table 4.--Summary of measured and computed backwater and discharge

		MEASURED BACKWATER			DISCHARGE		
Site	Bridge	Δh	h_1*	h_3*	$Q_{meas.}$	$Q_{comp.}$	Diff.
		ft	ft	ft	ft^3/s	ft^3/s	percent
1	MC	0.87	0.84	0.17	1440	1530	6.3
	RO-1	1.02	1.08	.17	727	841	15.7
2	MC	1.65	1.82	.25	5870	6880	17.2
	RO-1	1.68	1.87	.22	1640	1620	- 1.2
3	MC	.75	.70	.10	9700	9470	- 2.4
	RO-1	.55	.74	.25	2610	3430	31.4
	RO-2	.70	.72	.10	2120	2340	10.4
	RO-3	.78	.76	.14	4350	4840	11.3
	RO-4	.76	.73	.09	3340	3370	.9
	RO-5	.76	.74	.03	1190	719	-39.6
4	MC	.62	.62	.23	16000	16400	2.5
	RO-1	1.15	1.18	.10	2130	1400	-34.3
	RO-2	.91	1.12	.09	6130	6300	2.8
	RO-3	1.03	.98	.07	6050	5200	-14.0
5	MC	1.78	1.49	.26	35200	42300	20.2
	RO-1	1.99	1.63	-.28	10500	8410	-19.9
	RO-2	1.97	1.60	-.29	18100	16900	- 6.6
	RO-3	1.89	1.52	-.23	22000	19200	-12.7
6	MC	1.96	1.74	.04	14200	15500	9.2
	RO-1	2.42	2.37	.08	7470	7590	1.6
	RO-2	2.54	2.60	.20	5340	6060	13.5
7	MC	.41	.07	.04	4890	3500	-28.4
	RO-1	.37	.00	.00	964	1060	10.0
8	MC	.37	.36	.14	2420	2250	- 7.0
	RO-1	.47	.36	.09	1600	1960	22.5
	RO-2	.58	.36	-.02	878	1150	31.0
9	MC	.80	.31	-.02	11100	11400	2.7
	RO-1	.65	.61	-.03	1850	2340	26.5

MC Main channel
RO Relief opening

Table 4.--Summary of measured and computed backwater and discharge--Continued

		METHOD I COMPUTED BACKWATER					
Site	Bridge	Δh ft	Diff. from meas. ft	h_1* ft	Diff. from meas. ft	h_3* ft	Diff. from meas. ft
1	MC	0.53	-0.34	0.41	-0.43	0.08	-0.09
	RO-1	.69	- .33	.63	- .45	- .05	- .22
2	MC	1.34	- .31	.98	- .84	.28	.03
	RO-1	1.83	.15	1.73	- .14	.07	- .15
3	MC	.84	.09	.53	- .17	.16	.06
	RO-1	.36	- .19	.26	- .48	.04	- .21
	RO-2	.65	- .05	.48	- .24	.08	- .02
	RO-3	.55	- .23	.42	- .34	- .03	- .17
	RO-4	.48	- .28	.37	- .36	- .01	- .10
	RO-5	.62	- .14	.50	- .24	- .07	- .10
4	MC	-	-	-	-	-	-
	RO-1	-	-	-	-	-	-
	RO-2	-	-	-	-	-	-
	RO-3	-	-	-	-	-	-
5	MC	-	-	-	-	-	-
	RO-1	-	-	-	-	-	-
	RO-2	-	-	-	-	-	-
	RO-3	-	-	-	-	-	-
6	MC	-	-	-	-	-	-
	RO-1	-	-	-	-	-	-
	RO-2	-	-	-	-	-	-
7	MC	.72	.31	.28	.21	.06	.02
	RO-1	.06	- .31	.05	.05	- .36	- .36
8	MC	.40	.03	.15	- .21	.10	- .04
	RO-1	.26	- .21	.22	- .14	- .16	- .25
	RO-2	.33	- .25	.22	- .14	- .09	- .07
9	MC	.75	- .05	.31	.00	- .02	.00
	RO-1	.36	- .29	.30	- .31	- .09	- .06

MC Main channel
RO Relief opening

Table 4.--Summary of measured and computed backwater and discharge--Continued

		METHOD II COMPUTED BACKWATER					
Site	Bridge	Δh	Diff. from meas.	h_1*	Diff. from meas.	h_3*	Diff. from meas.
		ft	ft	ft	ft	ft	ft
1	MC	1.07	0.20	0.50	-0.34	-0.38	-0.55
	RO-1	1.96	.94	.48	- .60	-1.37	-1.54
2	MC	1.30	- .35	1.15	- .67	- .06	- .31
	RO-1	1.91	.23	1.74	- .13	- .14	- .36
3	MC	.81	.06	.62	- .08	- .05	- .15
	RO-1	.38	- .17	.12	- .62	- .20	- .45
	RO-2	.68	- .02	.46	- .26	- .13	- .23
	RO-3	.62	- .16	.75	- .01	.28	.14
	RO-4	.74	- .02	.73	.00	.12	.03
	RO-5	1.20	.44	1.17	.43	.03	.00
4	MC	.62	.00	.53	- .09	.14	- .09
	RO-1	1.23	.08	1.50	.32	.33	.23
	RO-2	.83	- .08	.83	- .29	.17	.08
	RO-3	1.21	.18	1.29	.31	.20	.13
5	MC	1.25	- .53	1.36	- .13	.65	.39
	RO-1	2.34	.35	2.48	.85	.22	.50
	RO-2	1.95	- .02	2.00	.40	.14	.43
	RO-3	2.11	.22	2.13	.61	.16	.39
6	MC	1.62	- .34	1.54	- .20	.18	.14
	RO-1	1.93	- .49	2.37	.00	.57	.49
	RO-2	1.60	- .94	2.23	- .37	.77	.57
7	MC	.59	.18	.70	.63	.49	.45
	RO-1	.28	- .09	.08	.08	.16	.16
8	MC	.45	.08	.28	- .08	- .02	- .16
	RO-1	.35	- .12	.09	- .27	- .06	- .15
	RO-2	.39	- .19	.06	- .30	- .13	- .11
9	MC	.62	- .18	.70	.39	.54	.56
	RO-1	.40	- .25	.27	- .34	.03	.06

MC Main channel
RO Relief opening

extend beyond the effects of the highway construction. The water surface at the fartherest downstream section (section 5) was used as the starting elevation. Cross sections usually were divided into three subsections with the main channel separating the flood plain. Cross sections were divided into only two subsections in those sections where the main channel was at the edge of the valley. The field-selected n-values were adjusted where necessary until the computed water-surface elevation matched the observed watersurface elevation at the most upstream section. The computed profile was examined to ensure that it reflected the known physical features of the flood plain.

Measurement of h_1^*

Backwater h_1^* at the approach section was measured one bridge-opening width upstream (fig. 1). The observed water surface at the approach section (section 1) was determined from the water-surface elevations surveyed along each edge of the valley. Where a sloping water surface extended across the approach section, the water-surface elevation was determined by interpolation at the boundary between the equivalent single channels as described by Davidian and others (1962) for each bridge-opening. The average of the elevations of the boundaries appropriate for each opening was used for the observed elevation at section 1. The computed natural water-surface elevation was subtracted from the observed water-surface elevation at the approach and is shown at "h_1^* measured" in table 4.

Measurement of h_3^*

The difference between the contracted water-surface elevation and the natural profile at the downstream side of the contraction is defined as h_3^*. The contracted water surface was measured as the average of the level determined at the downstream end of the abutments of each opening. The natural profile was determined from the profile computed through step-backwater procedures. Negative values of h_3^* represent contracted water-surface elevations that are below the natural profile and positive values of h_3^* represent those that are above the natural profile (table 4).

Stagnation Points

On the upstream side of the constriction embankments, flow stagnation occurs in the corners formed by the embankment and each edge of the valley. Flow stagnation occurs also at each of the interior embankments between bridge-openings. The location of the point of stagnation is a function of the location and geometry of the constriction and of the hydraulic characteristics of the approach channel. The flow divides at the stagnation point and passes through the openings on each side. This point was readily observable in the field and its elevation is reported as h_s in table 3.

An attempt was made to find the analogous stagnation point on the downstream side of the embankment. This point may be visualized as the point where the flow from adjacent bridges converges and turns downstream.

When the stage was just above the low-water channel, very little if any flow appeared to be parallel to the downstream embankment. Parallel flow was observable along the downstream embankment as the stage rose with increasing discharge. However, the point downstream analogous to the upstream stagnation point could not be located.

Computation of Discharge

For computation of discharge and backwater, lines are projected parallel to the flow from the flow division points to the approach and exit sections (sections 1 and 4). These lines are treated as fixed boundaries of an equivalent single opening constriction.

Discharge for each bridge was computed using the recovered highwater marks. The cross section properties were calculated. Total fall, Δh was calculated as the difference between the measured values of h_1 and h_3. Discharge was then calculated from equation 1 with the energy losses computed from equation 13 for sites without spur dikes and from equation 16 for sites with spur dikes. Since water-surface elevation cannot usually be directly measured at the upstream end of the spur dikes, this elevation was estimated to be $1/2\ (h_1 + h_3)$. The dike area, A_d and conveyance K_d, in table 3 were calculated for this elevation. The contracted water-surface elevation at section 3 is obtained by extrapolating the measured water-surface profile along the downstream side of the embankment to the intersection of the abutment and embankment for each side and averaging the values obtained. The computed and measured discharges are compared in table 4.

Computation of Backwater

Backwater is the difference between the water-surface profiles for the natural and constricted conditions. The natural profile is computed using a standard step-backwater procedure (Chow, 1959), where the friction losses are computed from equation 10. The constricted profile is also computed using a standard step-backwater procedure where the friction losses are computed from equation 13 and 16. Both profiles use section 4 as a common starting point. The average flow path needed in equation 13 and 16 is obtained from Schneider and others (1976).

The constricted water-surface profile is computed by iteration because the controlling conveyances are not known. The controlling conveyance, K_c, is computed at the natural water-surface elevation and used as the first estimate. Revised estimates of the controlling conveyances are determined at the computed constricted elevations and compared to the previous estimates. Successive estimates of the constricted profile are continued until the controlling conveyances agree within a preselected tolerance. With a tolerance criterion of

$$K_c^{k-1} \cong 0.95 K_c^{k} \qquad (23)$$

convergance can be achieved in two or three iterations. The superscript is the iteration number.

Errors in Computed Backwater and Discharge

Error is calculated as the difference between the computed and measured quantity. Two measures of error are used to evaluate each computation method (table 5). The bias, defined as the algebraic mean error, indicates whether or not the error magnitudes tend to be evenly distributed above and below zero. The root mean square (rms) error is defined as the square root of the mean of the sum of the squares of the errors. The rms error expresses the magnitude of error likely to occur in any computation using the method in question.

DISCUSSION OF RESULTS

Computations of backwater or discharge at multiple bridge sites depend on the distribution of flow through the several openings and the division of flow boundaries in the vicinity of the constriction. The distribution of flow in direct proportion to the gross area (Davidian and others, 1962) of the bridge opening gave consistent results. The ratio of the interior embankment length of the stagnation point to the total interior embankment length between each pair of openings was computed and these ratios are compared with the observed results in figure 3. There is considerable scatter about the line of equal value but on the average the answer may be a reasonable estimate of the stagnation points.

The description of flow through multiple openings in a highway is complex. Flow through each opening is affected by the hydraulic characteristics of the approach channels as well as the configuration and geometry of adjacent bridges. The methods described in this report to calculate discharge (figure 4) give results that are within $\pm$ 15 percent of the measured discharge in 17 of 28 cases, and are relatively simple to apply. Overall the bias was +2 percent with a rms error $\pm$18 percent. Schneider and others (1976) obtained about the same results for the bias (3 percent) but less scatter in the error (rms error about 9 percent). The need to divide the flood plain into a single equivalent channel for each bridge introduces additional error and also affects the backwater results.

The bias and rms error for the total fall, Δh, the backwater (h_1^*) at section 1 and the backwater (h_3^*) at section 3, computed by method I and method II are summarized in table 5. The errors are expressed as the difference in feet between the computed and measured value.

Comparison of the computed total fall with the measured total fall are shown in figures 5 and 6. Figure 5 shows that method I underestimates the total fall for 13 of the 17 bridges to which it

Table 5.--Comparison of measured and computed backwater based on 17 bridge openings for method I and 28 bridge openings for method II.

	Total Fall, Δh		Backwater at Section 1, h_1^*		Backwater at Section 3, h_3^*	
	method I	method II	method I	method II	method I	method II
RMSE (ft)	0.23	0.35	0.34	0.39	0.15	0.43
Bias (ft)	-.14	-.03	-.25	-.03	-.10	.02

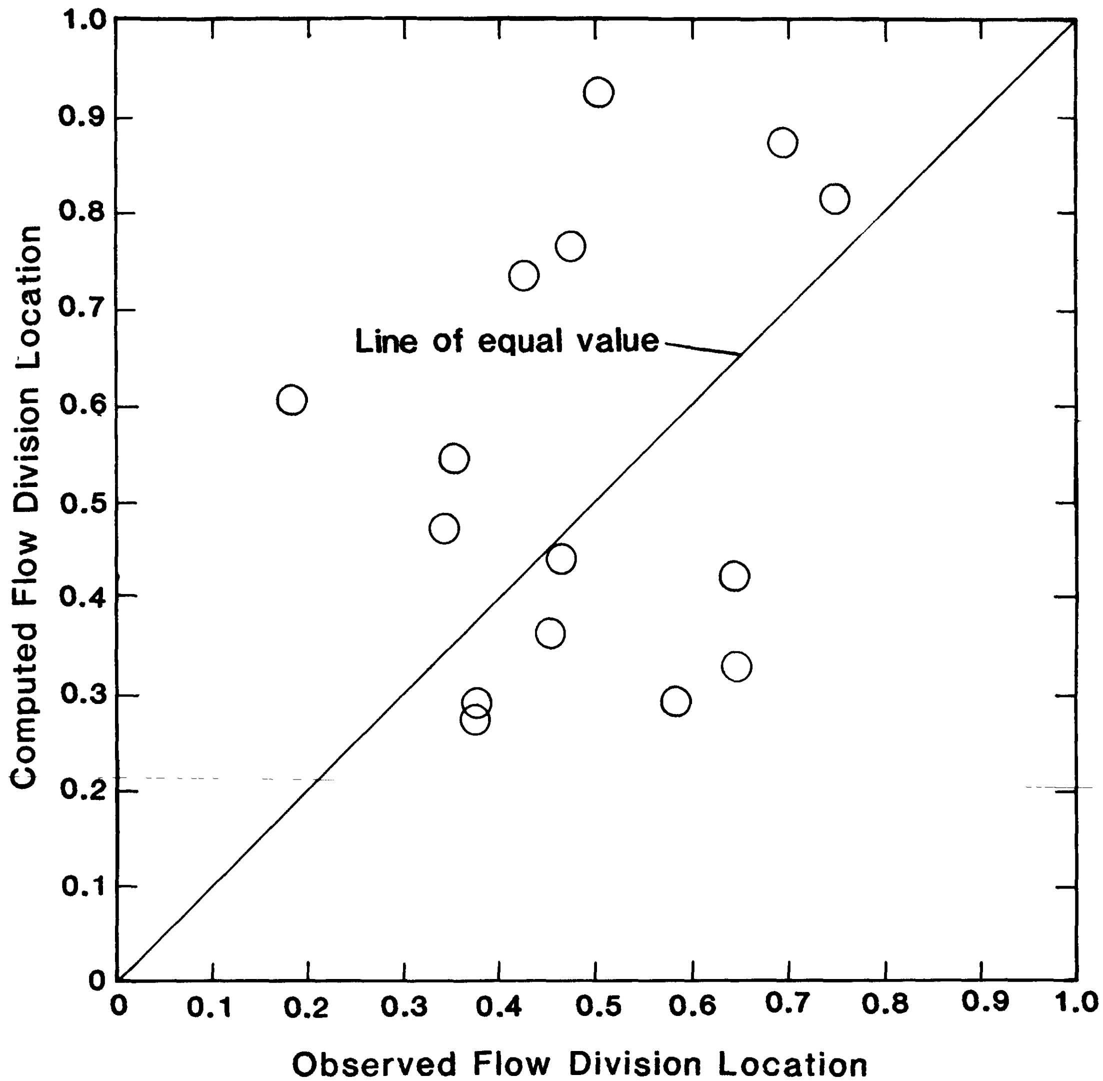

Figure 3.-- Comparison of measured and computed flow division points (stagnation points on the upstream side of the embankment).

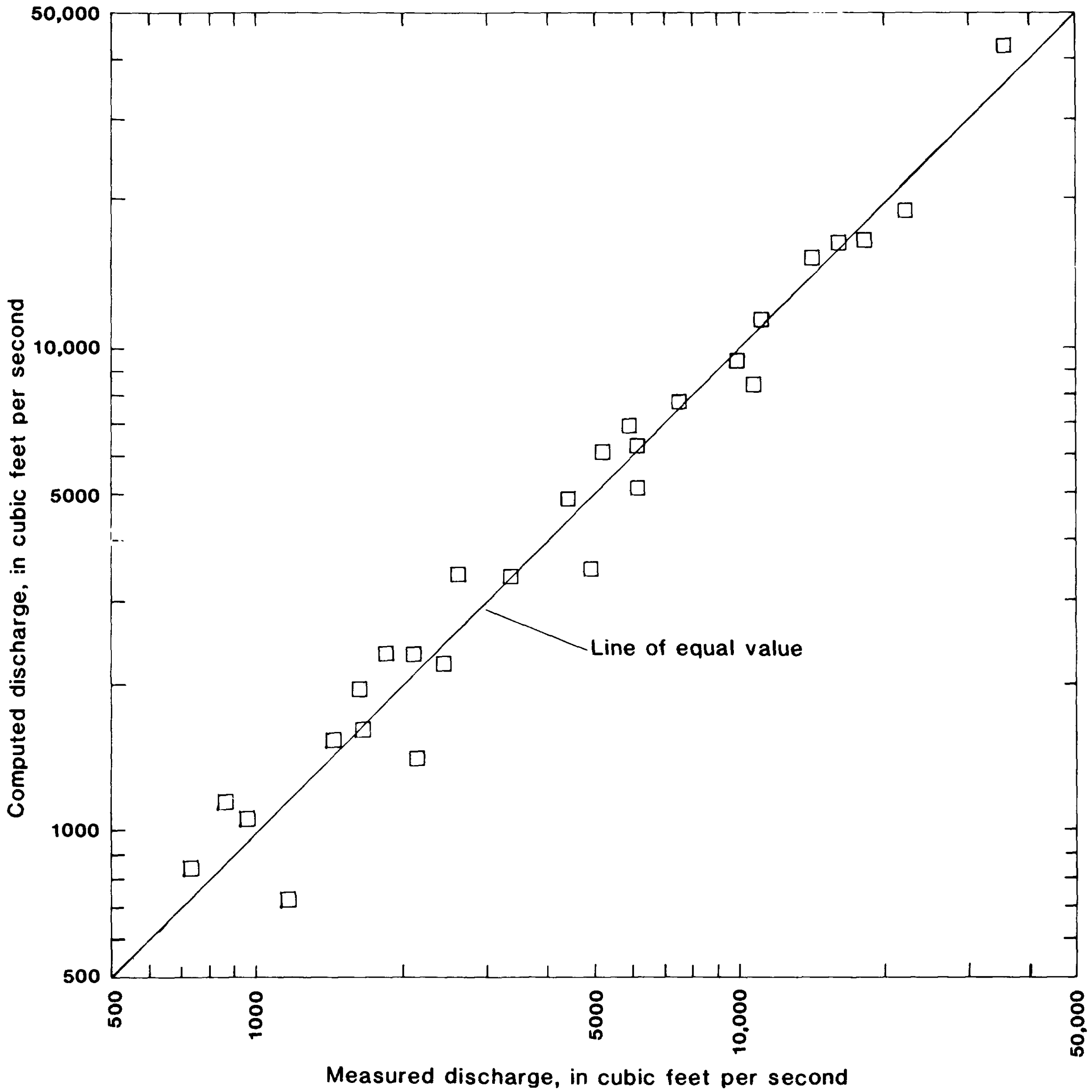

Figure 4.-- Comparison of the measured and computed discharge.

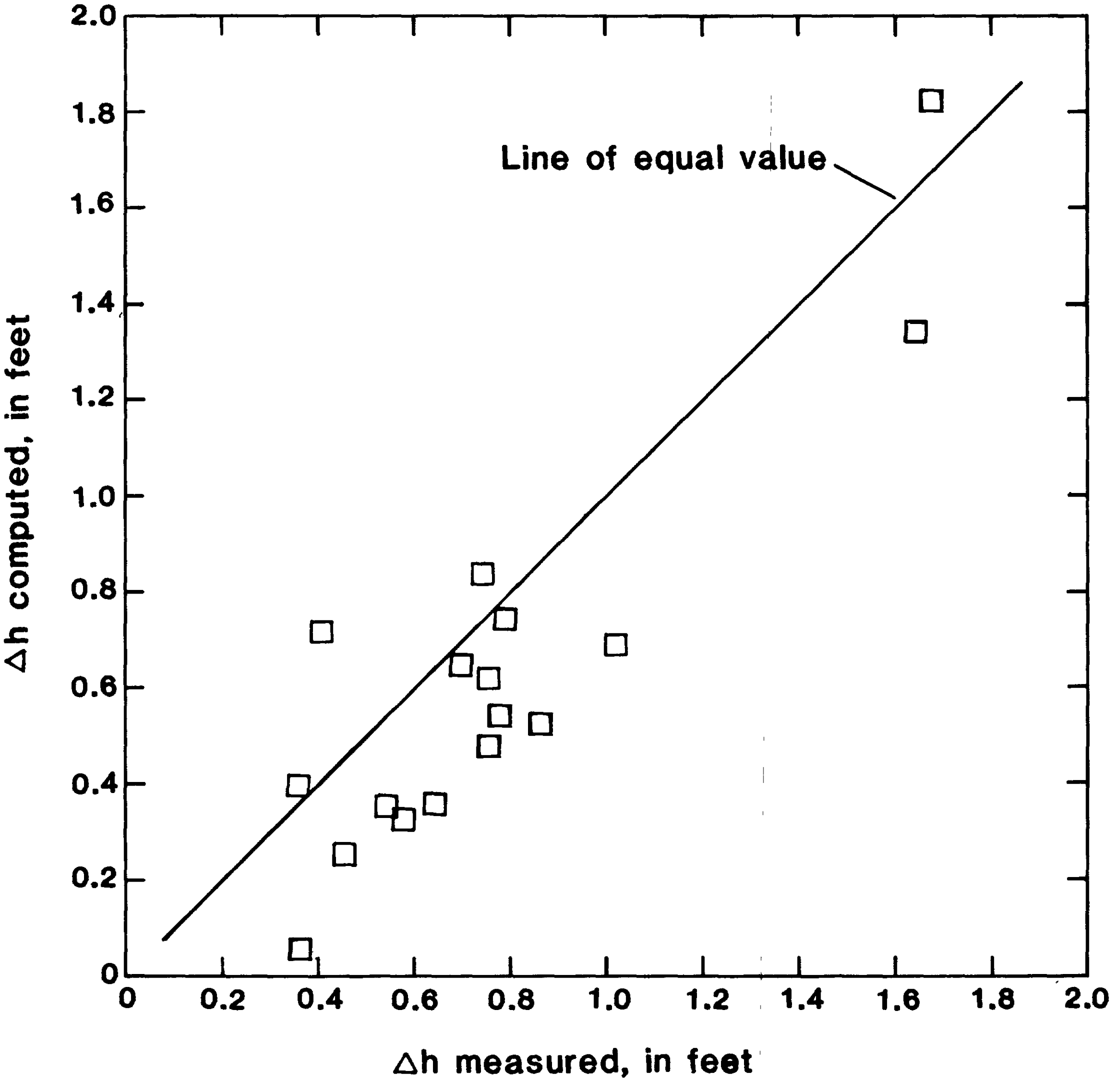

Figure 5.-- Comparison of the computed and measured Δh for method I.

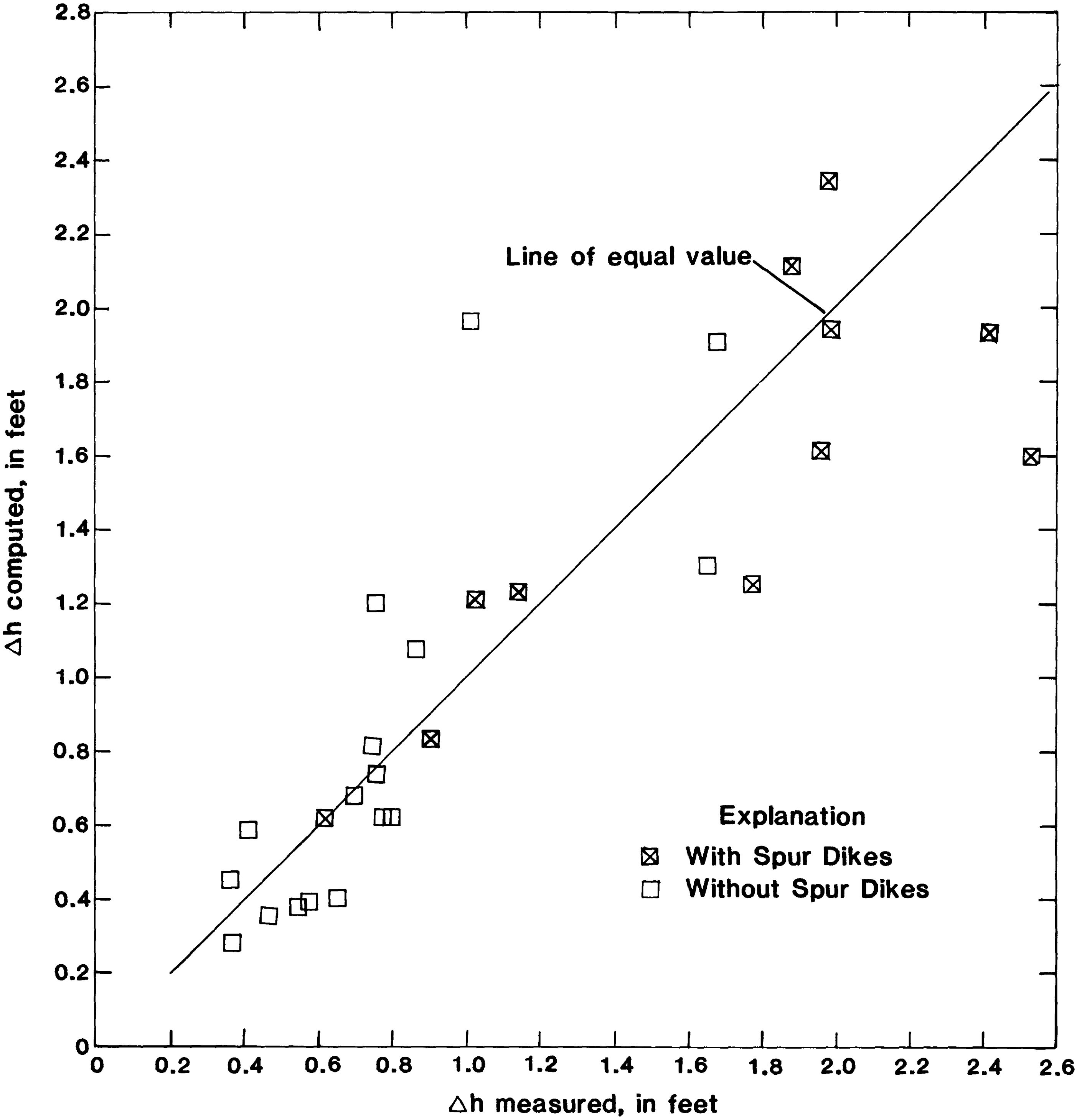

Figure 6.--Comparison of the computed and measured Δh for method II.

applies. Figure 6 shows that for method II the results for 28 bridges are equally distributed about the line of equal value.

Comparison of the computed backwater with the measured backwater at section 1 are shown in figures 7 and 8. Figure 7 shows that method I underestimates the backwater, h_1^*, for 13 of the 17 bridges. Figure 8 shows that for method II the results are equally divided about the line of equal value.

Schneider and others (1976) calculated backwater by the method developed by Tracy and Carter (1955) and found it underestimated backwater. Method I is an improvement of the method to the extent that L_w was replaced by L_{av} for computing friction loss. Hence, backwater calculated by method I is more accurate than the method developed by Tracy and Carter (1955) but the bias of -0.25 ft for h_1^* indicates it still underestimates backwater. The bias of -0.03 ft for the backwater, h_1^*, calculated by method II is considered negligible. The slight increase in rms error from 0.34 ft for method I to 0.39 ft for method II probably is not significant. Schneider and others (1976) reported for method II bias of -0.04 ft with a rms error of 0.24 ft for single opening systems. The results from computation of discharge (table 4) indicate similar error in evaluating losses between section 1 and 3 for these sites. Therefore, these results indicate that the method developed for single-opening highway crossings can be applied to multiple bridges with comparable results.

The backwater, h_3^*, at section 3, (fig. 9) computed by method I was less than that observed for 13 of the 17 bridge openings (table 4) and averaged 0.10 ft less than measured. Backwater, h_3^* (fig. 10) computed by method II was equal or greater than measured at 17 of the 28 bridge openings and averaged 0.02 ft more than the measured value. Backwater at section 3 is difficult to measure accurately because of the turbulent flow condition and the large spatial changes in water surface elevation. In general h_3^* is a relatively small value being less than 0.3 ft at all sites.

SUMMARY AND CONCLUSION

Data were collected for nine flood events to supplement laboratory studies of backwater at multiple bridge systems. These data consisted of measured discharge and water surface profiles through 28 bridge openings.

The multiple openings were divided into equivalent single opening cases by apportioning the interior embankments in direct proportion to the area of the openings on either side (Davidian and others, 1962). The discharge was computed using procedures described by Matthai (1967) and by Schneider and others (1976). The best results were obtained by using the average flow path (Schneider and others, 1976) for approach friction losses in the method given by Mattahi (1967). This gave computed discharges within 15 percent of the measured values for 17 of the 28 openings. The mean error for

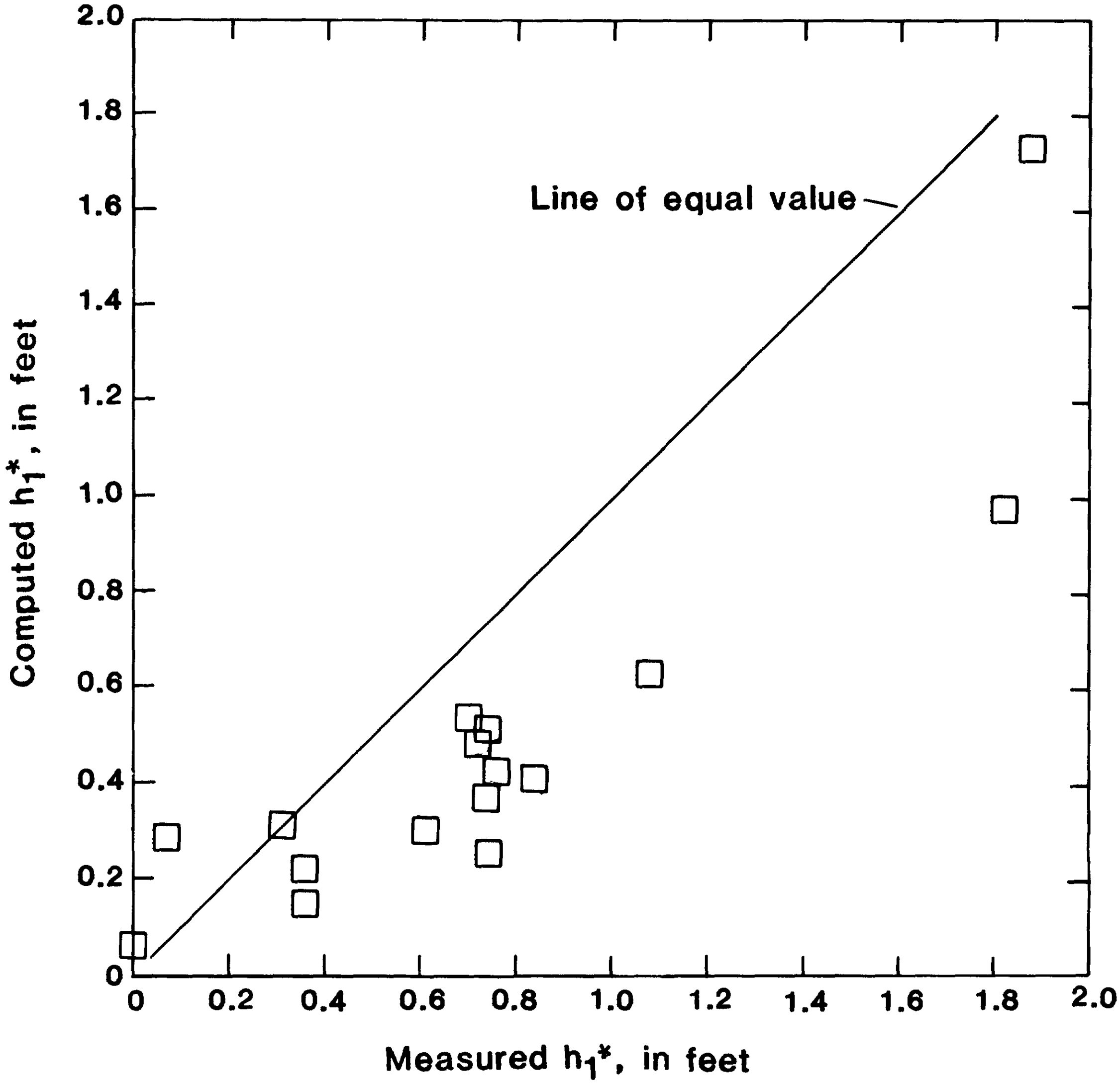

Figure 7.-- Comparison of computed and measured backwater, h_1^*, for method I.

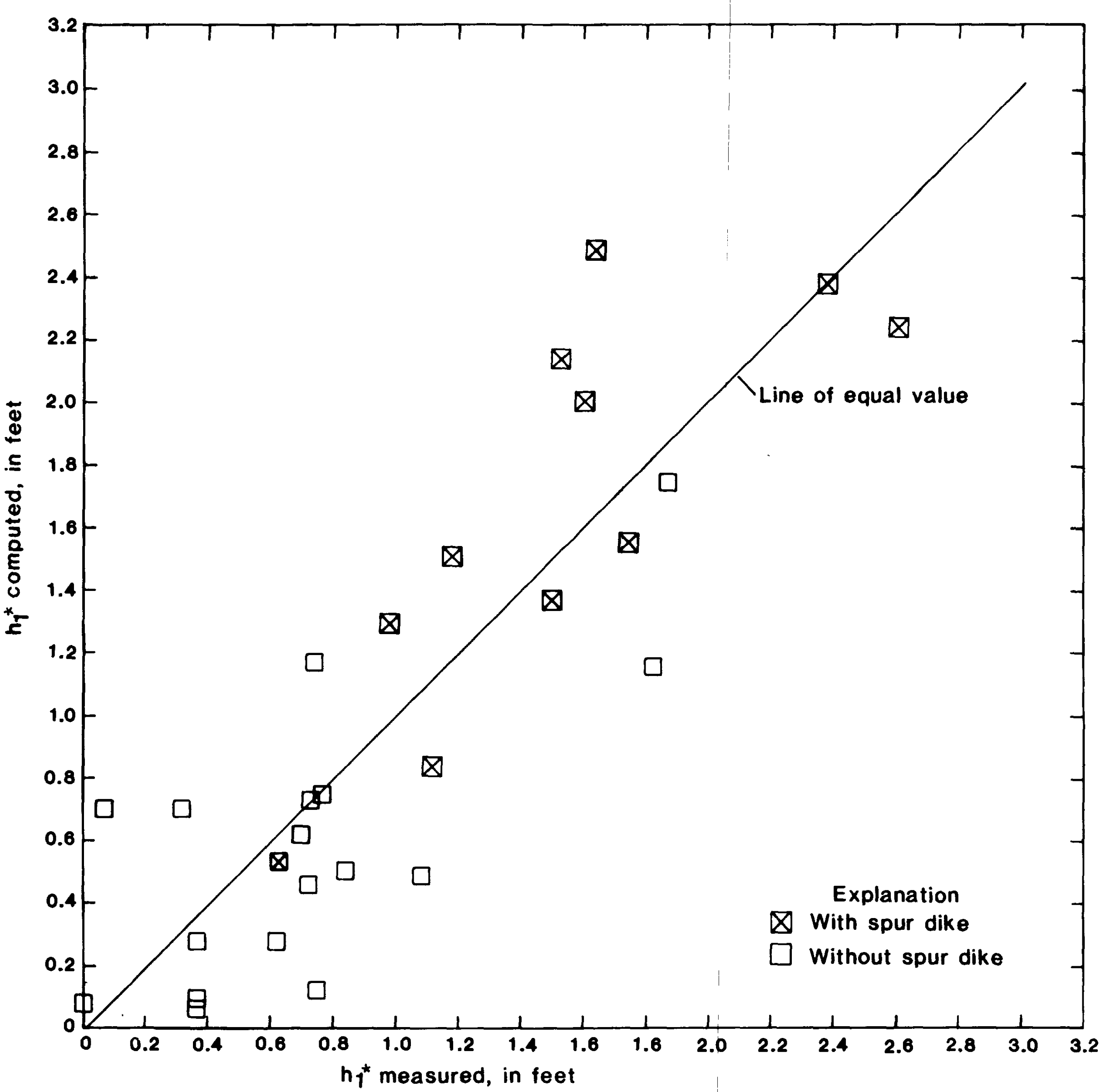

Figure 8.— Computed and measured backwater, h_1^*, for method II.

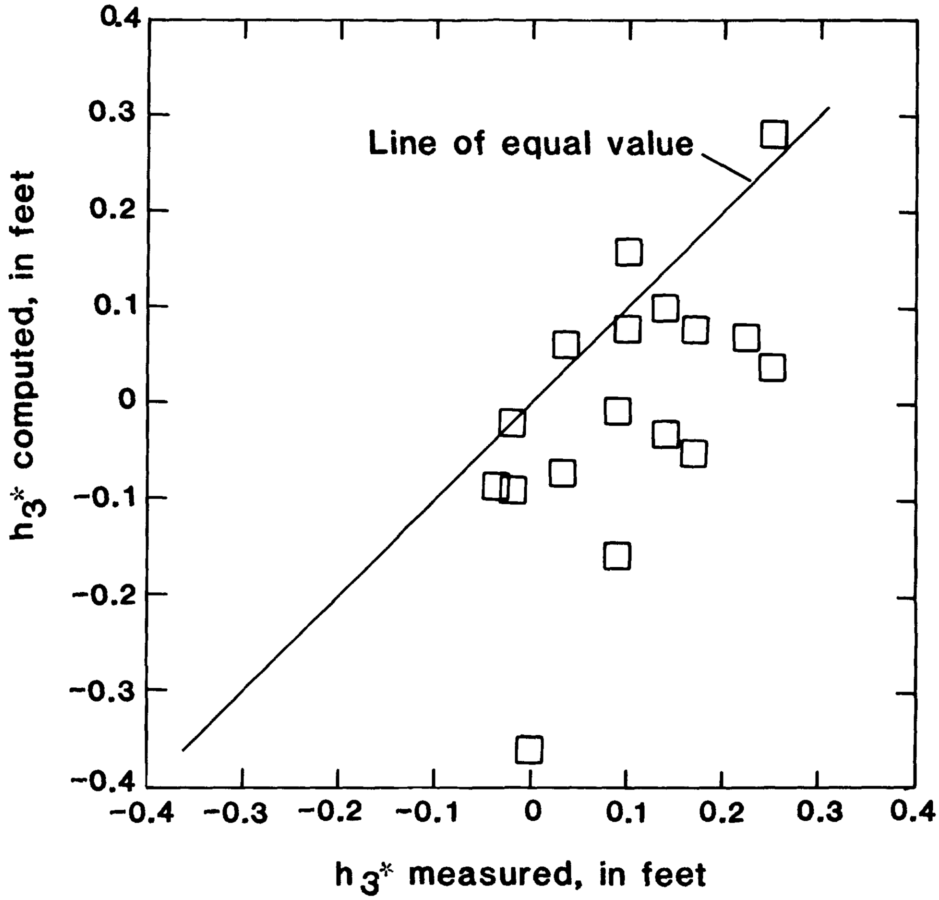

Figure 9.—Comparison of the computed and measured backwater, h_3^*, for method I.

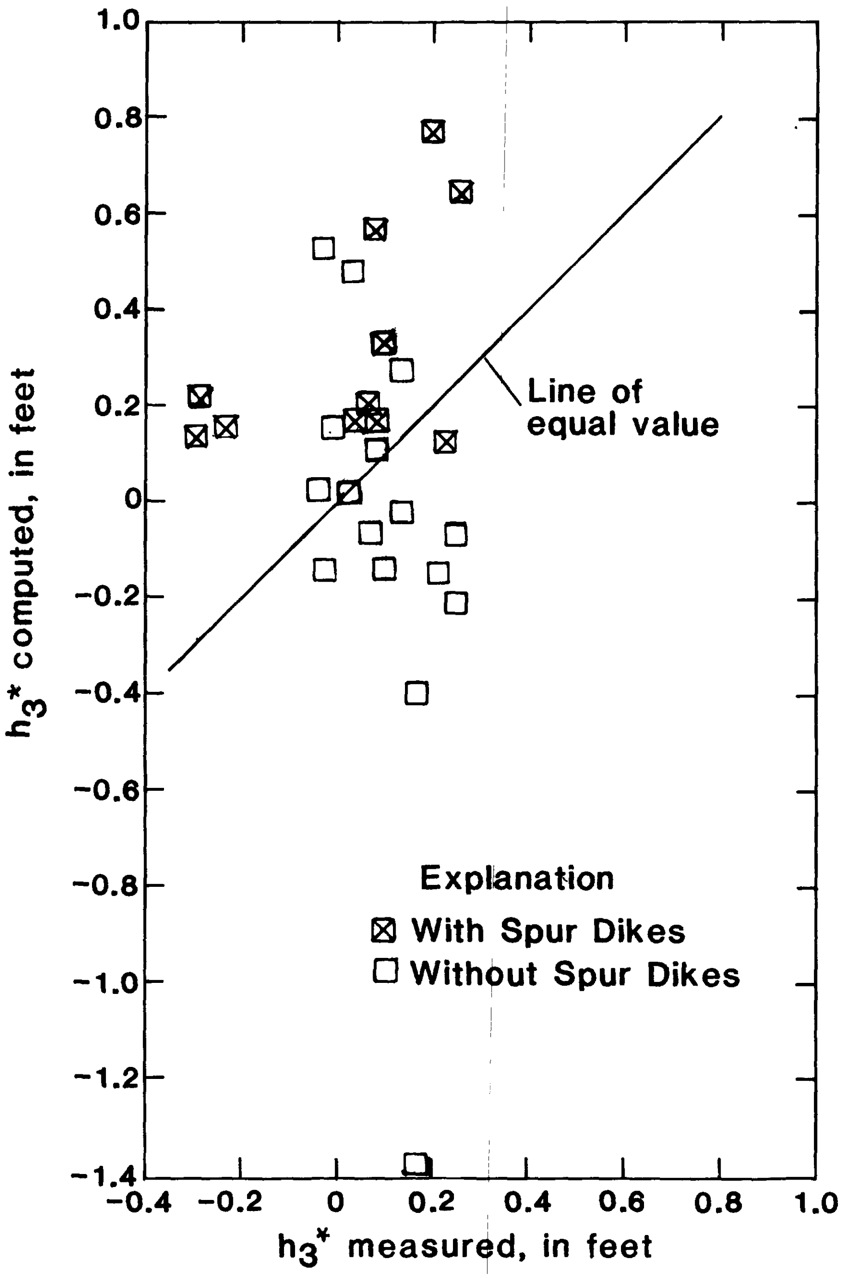

Figure 10.-- Comparison of the computed and measured backwater, h_3^*, for method II.

all openings was 2 percent with a root mean square error of 18 percent.

Backwater was measured by comparing the computed natural profile with the water-surface elevations obtained from high water marks. Backwater was computed by method I (Tracy and Carter, 1955) in which the friction loss term in the approach reach was modified to account for the average flow path, and by method II (Schneider and others, 1976). The bias and rms error for backwater at section 1 computed by method I are -0.25 ft and ±0.34 ft, respectively. Method I underestimates backwater for 14 of 17 sites. The bias and rms error for backwater at section 1 computed by method II are -0.03 ft and ±0.39 ft, respectively, with results about evenly divided by the line of equal value. These results indicate that the method developed for single-opening highway crossings can be applied to the multiple-bridge opening.

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