US Army Corps of Engineers
Hydrologic Engineering Center

Computer Determination of Flow Through Bridges

July 1970

TP-20

REPORT DOCUMENTATION PAGE

Form Approved OMB No. 0704-0188

1. REPORT DATE *(DD-MM-YYYY)*
July 1970

2. REPORT TYPE
Technical Paper

3. DATES COVERED *(From - To)*

4. TITLE AND SUBTITLE
Computer Determination of Flow Through Bridges

5a. CONTRACT NUMBER

5b. GRANT NUMBER

5c. PROGRAM ELEMENT NUMBER

6. AUTHOR(S)
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5d. PROJECT NUMBER

5e. TASK NUMBER

5F. WORK UNIT NUMBER

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)
US Army Corps of Engineers
Institute for Water Resources
Hydrologic Engineering Center (HEC)
609 Second Street
Davis, CA 95616-4687

8. PERFORMING ORGANIZATION REPORT NUMBER
TP-20

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)

10. SPONSOR/ MONITOR'S ACRONYM(S)

11. SPONSOR/ MONITOR'S REPORT NUMBER(S)

12. DISTRIBUTION / AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

13. SUPPLEMENTARY NOTES
Published in the Journal of the Hydraulics Division, ASCE, Vol. 96, No. HY7, July 1970, pp. 1455-1468.

14. ABSTRACT
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15. SUBJECT TERMS
backwater, bridge piers, hydraulics, computers, models

16. SECURITY CLASSIFICATION OF:			17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON
a. REPORT	b. ABSTRACT	c. THIS PAGE			
U	U	U	UU	36	19b. TELEPHONE NUMBER

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COMPUTER DETERMINATION OF FLOW THROUGH BRIDGES

SUMMARY

Techniques used for the computer determination of water surface profiles through bridges are presented. Application of momentum principles to determine low flow profiles and methods for determining the controlling type(s) of flow (low flow, pressure flow, weir flow, etc) are described.

COMPUTER DETERMINATION OF FLOW THROUGH BRIDGES

ABSTRACT

KEY WORDS: backwater; bridge piers; hydraulics; computers, models

Techniques used for the computer determination of water surface profiles through bridge structures of various shapes and sizes, including culverts under high fills, are presented. The techniques are incorporated in the Hydrologic Engineering Center Program on Water Surface Profiles. Theoretical aspects of procedures for low flow through bridges using momentum principles are described in detail for both supercritical and subcritical flow. Methods for determining the controlling type of flow (low flow, pressure flow, weir flow, or various combinations of these) and corresponding changes in water surface elevation are described and illustrated. Stress is placed on the need for additional research and field data to evaluate the accuracy of various techniques that are suited to computer determination of water surface profiles through bridges. Limitations of the procedures are described.

COMPUTER DETERMINATION OF FLOW THROUGH BRIDGES[1]

By Bill S. Eichert,[2] M. ASCE, and John Peters,[3] A.M. ASCE

INTRODUCTION

Many thousands of miles of water surface profiles are computed each year by engineers throughout the world in order to evaluate flood damage potential and to design channels that will reduce future flooding. Many of these studies require the determination of profiles through various types and shapes of bridges. Because of the repetitious nature of these studies and the large amount of engineering labor required, it is desirable that digital computer programs for determining water surface profiles include comprehensive bridge routines that will handle the variety of bridge flow conditions that can be encountered in a typical study. A brief comparison of bridge loss routines from several sophisticated computer programs is contained in reference 1. This paper describes comprehensive bridge flow routines that are incorporated in The Hydrologic Engineering Center Program on Water Surface Profiles (reference 2), which is now in use in many offices in the United States. Theoretical aspects of procedures used in The Hydrologic Engineering Center's (HEC) computer program for low flow through bridges (where the water surface is not in contact with the low chord of the bridge) are presented. Theory of weir

[1]Published in the Journal of the Hydraulics Division, ASCE, vol. 96, no. HY7, July 1970, pp. 1455-1468.

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flow and orifice (pressure) flow are omitted because these are found in many standard texts. The methods used to determine the controlling type of flow or combination of flows are also described and illustrated. Shortcomings of the bridge routines are emphasized in the hope that further development of practical methods for predicting flow through bridges will be stimulated.

METHODS AVAILABLE

Several procedures are available for computing energy losses through bridges including the criteria set forth by the US Army Engineer Waterways Experiment Station (reference 3), the Bureau of Public Roads (reference 4), and the US Geological Survey (reference 5). All of these procedures are applicable primarily to conditions of low flow control. When the free surface of the water is obstructed by the bridge, pressure flow or a combination of pressure flow and weir flow can exist. Little information is available for computing losses under these conditions, although procedures for pressure flow and weir flow through flood control conduits and spillways are well established. Unfortunately, the various methods available do not give comparable answers for a fixed set of conditions. Technical procedures are not available for accurately determining flow conditions that would cause a bridge to fail. Current procedures do not take into account the local scour that occurs at high flows, although this factor should be evaluated when possible. Provision for debris and trash obstructions in the bridge opening are often ignored in the computations, although these can produce substantial increases in the upstream water surface elevation.

LOW FLOW THROUGH BRIDGES

As previously stated, low flow through bridges refers to flow conditions that exist when water is not in contact with the low chord of the bridge. In the HEC computer program approach the losses due to expansion and contraction of the flow area on the upstream and downstream sides of the structure are computed separately from the loss through the structure itself. The contraction and expansion losses at the bridge are evaluated in the same way as expansion and contraction losses where bridges are not present; that is, by multiplying a loss coefficient times the absolute difference between the velocity heads within and outside of the bridge constriction. For bridges without piers, skin friction along the sides of the bridge are accounted for with normal backwater computations using standard step procedures (reference 6). When piers are present, the pier losses can be evaluated by application of momentum principles as proposed by Koch and Carstanjen (reference 7). Application of momentum principles for rectangular-shaped channels is given in reference 3. The general theory, applicable to channels of any shape, is given below. The HEC program is presently programmed for trapezoidal-shaped constrictions in natural channels.

Consider the plan and profile views of flow past bridge piers shown in Figure 1. Section 1 and Section A are located immediately upstream and downstream from the upstream ends of the piers, respectively. It is assumed that the pressure distribution in these sections is hydrostatic.

Momentum theory applied between the two sections results in

$$\beta_1 Q\rho v_1 + \gamma A_1\bar{y}_1 - \beta_A Q\rho v_A - \gamma A_A\bar{y}_A = F \qquad (1)$$

where:

Q = discharge - ft^3/sec

ρ = fluid density - lb sec^2/ft^4

v_1, v_A = mean velocities at Sections 1 and A, respectively - ft/sec

β_1, β_A = momentum coefficients - dimensionless

A_1, A_A = flow areas at Sections 1 and A, respectively - ft^2

γ = unit weight of fluid - lb/ft^3

$\bar{y}_1, \bar{y}_A$ = vertical distances from water surface to centroids of Sections 1 and A, respectively - ft

F = force exerted by piers on flow - lb

Division of Equation (1) by γ and substitution of $Q \div A$ for v results in

$$\frac{Q^2}{gA_1} + A_1\bar{y}_1 - \frac{Q^2}{gA_A} - A_A\bar{y}_A = \frac{F}{\gamma} \qquad (2)$$

It is assumed in the above equation and subsequent equations that the momentum coefficients due to a non-uniform distribution of velocity are equal to 1. It is also assumed that boundary friction forces are negligible compared with the force exerted on the flow by the piers. Koch and Carstanjen proposed that the static and dynamic forces exerted by the upstream ends of square piers be given as

$$\gamma A_{P1} y_{P1} \text{ and } \gamma\frac{A_{P1}}{A_1}\,\frac{Q^2}{gA_1}, \text{ respectively}$$

where:

A_{P1} = projected area of the piers (or obstruction, if trash is included) normal to the direction of flow corresponding to the flow depth y_1 at Section 1.

$\bar{y}_{P1}$ = vertical distance from water surface to centroid of A_{P1}.

Koch and Carstanjen suggested that the dynamic force be reduced by one-third for piers with semi-circular ends. In general terms the dynamic force can be expressed as

$$\frac{C_D}{2} \gamma \frac{A_{P1}}{A_1} \frac{Q^2}{gA_1}$$

where C_D is a drag coefficient equal to 2 for square pier ends and 1.33 for piers with semicircular ends. Substituting the above expressions for static and dynamic forces in equation (2) and solving for the momentum flux (i.e., sum of $\frac{Q^2}{gA} + A\bar{y}$) at Section A,

$$M_A = \frac{Q^2}{gA_A} + A_A\bar{y}_A = A_1\bar{y}_1 - A_{P1}\bar{y}_{P1} + \frac{Q^2}{gA_1^2}\left[A_1 - \frac{C_D}{2}A_{P1}\right] \qquad (3)$$

In a similar fashion, momentum theory may be applied between Sections B and 3, which are located immediately upstream and downstream from the downstream end of the piers, respectively. In this case, the force exerted by the piers is in the downstream direction and has only a static component, which is equal to $\gamma A_{P3}\bar{y}_{P3}$, where

A_{P3} = projected area of piers normal to the direction of flow corresponsing to the flow depth y_3 at Section 3.

$\bar{y}_{P3}$ = vertical distance from water surface to centroid of A_{P3}.

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The momentum flux at Section B can therefore be given as:

$$M_B = \frac{Q^2}{gA_B} + A_B\bar{y}_B = A_3\bar{y}_3 - A_{P3}\bar{y}_{P3} + \frac{Q^2}{gA_3} \qquad (4)$$

Assuming that any force exerted by bridge piers is negligible between Sections A and B, the momentum flux will be constant at these and all intermediate sections. That is

$$M_A = M_2 = M_B \qquad (5)$$

where M_2 = momentum flux at any intermediate section between Sections A and B, $= \frac{Q^2}{gA_2} + A_2\bar{y}_2$

Equations (3), (4) and (5) can be combined to give the following relation:

$$A_1\bar{y}_1 - A_{P1}\bar{y}_{P1} + \frac{Q^2}{gA_1^2}\left[A_1 - \frac{C_D}{2}A_{P1}\right] = \frac{Q^2}{gA_2} + A_2\bar{y}_2 =$$

$$A_3\bar{y}_3 - A_{P3}\bar{y}_{P3} + \frac{Q^2}{gA_3} \qquad (6)$$

Equation (6) contains three expressions for the momentum flux in the constriction. The expressions are illustrated by the three curves in Figures 2a, b and c, which relate the momentum flux <u>in the constriction</u> to depths upstream from, within, and downstream from the constriction, respectively. These curves are functions solely of the discharge and the geometry of the cross sections and piers.

Six low-flow conditions can occur in the constriction, as illustrated in Figure 3. In Figures 3a, b and c, the constriction is in a reach where flow would be sub-critical if the presence of the bridge piers were ignored.

A water surface profile can be calculated using the HEC program for such a reach from a downstream control to the downstream end of the bridge. However, the depth thus calculated for Section 3 can only exist if the momentum flux in the constriction calculated on the basis of the downstream depth exceeds the critical momentum flux in the constriction. Figure 2c can be used to determine the momentum flux that would exist in the constriction for a depth at Section 3 from water surface profile computations. The critical (minimum) momentum flux for the constriction is represented by M_{CRIT} in Figure 2b. If the momentum flux in the constriction calculated on the basis of the downstream depth exceeds the critical momentum flux, flow as in Figure 3a will occur. If the two momentum fluxes are equal, flow as in Figure 3b will occur. If the critical momentum flux is the greater of the two fluxes, a hydraulic jump will be formed as shown in Figure 3c. For flow as in Figures 3b and c, the momentum flux that exists in the constriction is the critical momentum flux, and unknown upstream and downstream depths can be determined from Figures 2a and 2c, respectively, using the critical momentum flux, M_{CRIT}. For flow as in Figure 3a, the momentum flux that exists in the constriction can be determined from Figure 2c on the basis of a previously calculated depth at Section 3. The unknown depths upstream from and within the constriction can be determined for this momentum flux from Figures 2a and b, respectively.

Similar reasoning can be applied to ascertain the flow conditions shown in Figures 3d, e and f for a reach where flow would be super-critical if the presence of the bridge piers were ignored. However, for a supercritical reach the "starting point" is a previously determined depth at Section 1 rather than Section 3. If the momentum flux in the constriction determined from Figure 2a on the basis of the depth at Section 1 is greater than M_{CRIT}, flow as in Figure 3d will occur. If the two momentum fluxes are equal, flow as in Figure 3e will occur. If M_{CRIT} is the greater of the two momentum fluxes, a hydraulic jump will occur upstream from the bridge piers as shown in Figure 3f.

Subroutine BLFLO in HEC programs 22-J2-L212 and 22-J2-L232 determines low-flow profiles using the above procedures (see flow chart, Figure 4) according to the following steps:

(1) The momentum flux for the constriction is determined on the basis of a previously calculated upstream or downstream depth (Equations on Figures 2a and 2c);

(2) The critical momentum flux (minimum possible), M_{CRIT}, is determined for the constriction;

(3) The type of flow (i.e., one of the types shown in Figure 3) is determined by comparing the momentum fluxes determined in steps (1) and (2);

(4) Unknown flow depths are determined for the particular flow type by using the appropriate portions of equation (6), with the exception that if the flow type is that shown in Figure 3a, Yarnell's equation, which is given below, is used.

Data collected and analyzed by the Los Angeles District, Corps of Engineers (reference 8) indicate that application of the semi-empirical

Yarnell equation yields more satisfactory results for the type of flow shown in Figure 3a. The Yarnell equation is

$$\Delta y = 2K \left(K + \frac{10\, v_3^2}{2gy_3} - 0.6\right) \left[\frac{A_{P3}}{A_3} + 15 \left(\frac{A_{P3}}{A_3}\right)^4 \right] \frac{v_3^2}{2g} \qquad (7)$$

where

$\Delta y = y_1 - y_3$ = difference between upstream and downstream water surface elevations

K = experimental pier-shape coefficient

Values for K are given in Table 1.

TABLE 1

TYPE OF PIER	K FOR USE IN YARNELL'S EQUATION
Semicircular nose and tail	0.90
Twin cylinder piers with connecting diaphragm	0.95
Twin cylinder piers without diaphragm	1.05
90° triangular nose and tail	1.05
Square nose and tail	1.25

PRESSURE FLOW

The method used in the computer program to determine the existence of pressure flow is to compare the energy grade line elevations required to pass a given discharge based on low-flow control and pressure-flow control. The higher energy elevation represents the controlling type of flow, as shown on Figure 5. The delineation between pressure flow and the combination of pressure and weir flow is determined by comparing the energy grade line elevation for pressure flow alone to the elevation of the top of roadway. If this pressure flow energy grade line is above the top of the roadway elevation, then a combination of pressure flow and weir flow is assumed. The basic formula for pressure flow is:

$$Q = A\sqrt{\frac{2gH}{k}} \qquad (8)$$

where:

Q = The discharge for pressure flow

A = The cross sectional area of the submerged opening

H = The head under the bridge, normally measured from the upstream energy grade line to the downstream tailwater

k = The total loss coefficient, including the velocity head conversion from static energy to kinetic energy, friction loss, and other minor losses

WEIR FLOW

Flow over the top of roadway is computed by subdividing the roadway cross section into segments and computing the flow over each by the weir formula. Corrections for submergence caused by flow over the weir and through the bridge are presently based on relations developed for ogee overflow crests (reference 3) and are shown in Table 2. It is planned to incorporate submergence relations determined recently by W. A. Thomas for broad-crested weirs (reference 8). The basic weir flow equation used in the program is

$$Q = CL\,H^{3/2}$$

where:

Q = The discharge for weir flow

L = The effective length of flow

H = The head over the roadway which is measured to the upstream energy grade line

C = Coefficient of discharge

The various values of C required to reconstruct profiles through bridges are much lower than the theoretical coefficient (3.1 for English system, and 1.72 for metric system) for critical depth in a rectangular channel since the weir is broad-crested. A typical value for C is 2.5.

TABLE 2

PERCENT REDUCTION OF WEIR FLOW COEFFICIENT	RATIO OF TAILWATER VELOCITY HEAD (h_a) TO DIFFERENCE IN ELEVATION BETWEEN UPSTREAM ENERGY GRADE LINE AND WEIR CREST (H_e)
100	0
60	.052
40	.086
10	.23
6	.34
3	.495
1	.705
0	.850

COMBINATION FLOWS

A combination flow (pressure and weir, or low flow and weir) may be determined by an iterative process of assuming energy grade line elevations and computing corresponding discharges through the bridge (pressure or low flow) and overbanks (weir flow) until the total discharge corresponds to the given discharge. The determination of the existence of the various types of flows is illustrated in Figures 6 and 7. The combination of low-flow under the bridge and weir flow in the overbanks (condition c) exists when the energy grade line elevation required to pass the discharge under consideration is above the minimum roadway elevation and below the low chord of the bridge. Figure 8 is a flow diagram which depicts the methodology of computing the flow profile for the combination flows using the HEC computer program.

BACKWATER WITH CORRECTION FOR BRIDGE DECK

A simple method of computing water-surface profiles through bridges (called the normal bridge method) involves performing normal backwater computations, making appropriate corrections for the area and wetted perimeter of the bridge deck. This procedure for non-trapezoidal sections where low-flow controls is superior to the method discussed in previous paragraphs since a trapezoidal channel was assumed. When the discharge is supercritical and the outlet is submerged, the normal method is not applicable, and the method described in the preceding paragraphs should be used, since critical depth should not be crossed in normal backwater computations. However, if the downstream tailwater is also over the roadway, then critical depth is not crossed and the normal bridge method is appropriate.

The losses computed by the normal bridge routine from cross sections on either end of the bridge do not reflect the shock losses experienced at the entrance and exit of the bridge. These losses may be taken into consideration by using a full river and a constricted cross section immediately upstream and downstream from the bridge and performing normal backwater computations through these cross sections.

LIMITATIONS OF THE HEC BRIDGE ROUTINE

Little experimental data is available for determining the dynamic force for piers of various shapes in equation 3. While the procedure for determining pier losses for low-flow control has been verified to a certain extent by Koch and Carstanjen (reference 7) and the Los Angeles District Corps of Engineers (reference 9), additional testing is warranted.

The coefficient of freeflow discharge used in the weir equation must be determined largely by judgment. The correction for submergence is based on experimental data for ogee spillways and becomes unreliable for high values of submergence.

Some accuracy is probably lost by the assumption that the water surface profile perpendicular to the direction of flow is horizontal. Since the velocity of flow increases from the overbanks to the center of the channel, the water surface elevation drops towards the center of the channel. This condition can cause weir flow in the overbanks and low flow in the channel for a level top of bridge.

CONCLUSIONS

This paper has presented a procedure for determining water surface profiles through bridge structures of various shapes and sizes, including culverts under high fills. The procedure allows a continuous water surface profile to be computed through bridge structures, over dams, and through culverts for both subcritical and supercritical flows. The procedure is a practical technique that has been developed during the last three years while working with over 60 different offices (approximately 30 Corps of Engineers offices) throughout the United States and Canada.

While reasonable answers have been produced for many unusual types and shapes of bridges, few occasions have arisen where accurate high-water marks and measured discharges were available to test the accuracy of the procedures.

It is hoped that this paper will stimulate an interest in this subject leading to research projects which will provide the necessary field data to evaluate techniques that are suited to computer solution of water surface profiles through bridges.

ACKNOWLEDGEMENT

The cooperation of many District offices of the Corps of Engineers, several consulting engineering firms and State offices has enabled The Hydrologic Engineering Center to develop and test this procedure. The Galveston, Wilmington, Los Angeles and St. Paul District offices deserve special recognition for their contributions.

APPENDIX I. - REFERENCES

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APPENDIX II. - NOTATION

The following symbols are used in this paper:

A = cross sectional flow area;

C = discharge coefficient for weir flow equation;

C_D = drag coefficient;

F = force exerted by bridge piers on fluid;

g = acceleration due to gravity;

H = head in orifice and weir flow equations;

k = total loss coefficient for orifice flow equation;

K = pier-shape coefficient in Yarnell equation;

L = effective flow length in weir flow equation;

M = momentum flux;

M_{CRIT} = momentum flux for flow at critical depth;

Q = discharge;

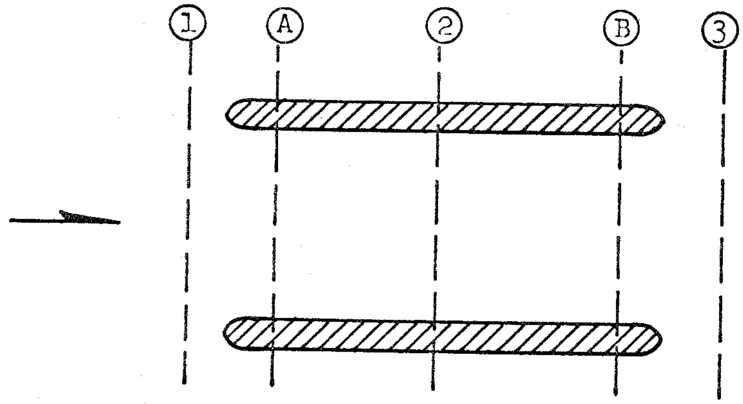
v = mean flow velocity;

$\bar{y}$ = vertical distance from water surface to centroid of flow area;

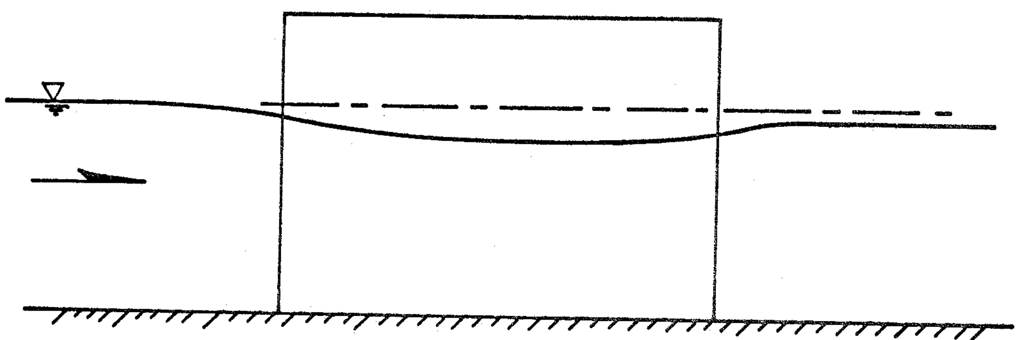
β = momentum coefficient;

γ = specific weight of water;

ρ = density of water.



PLAN



PROFILE

FIGURE 1

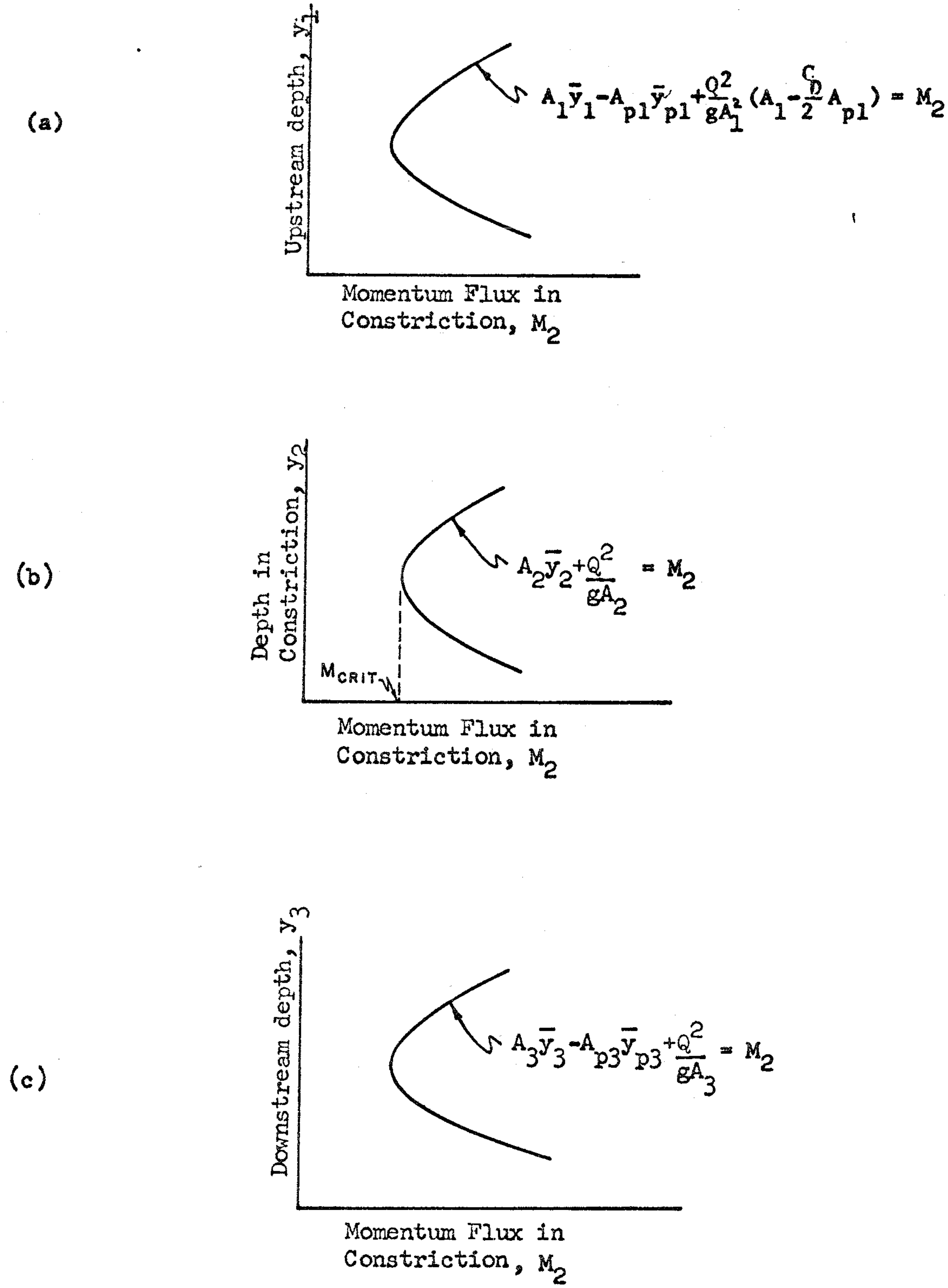
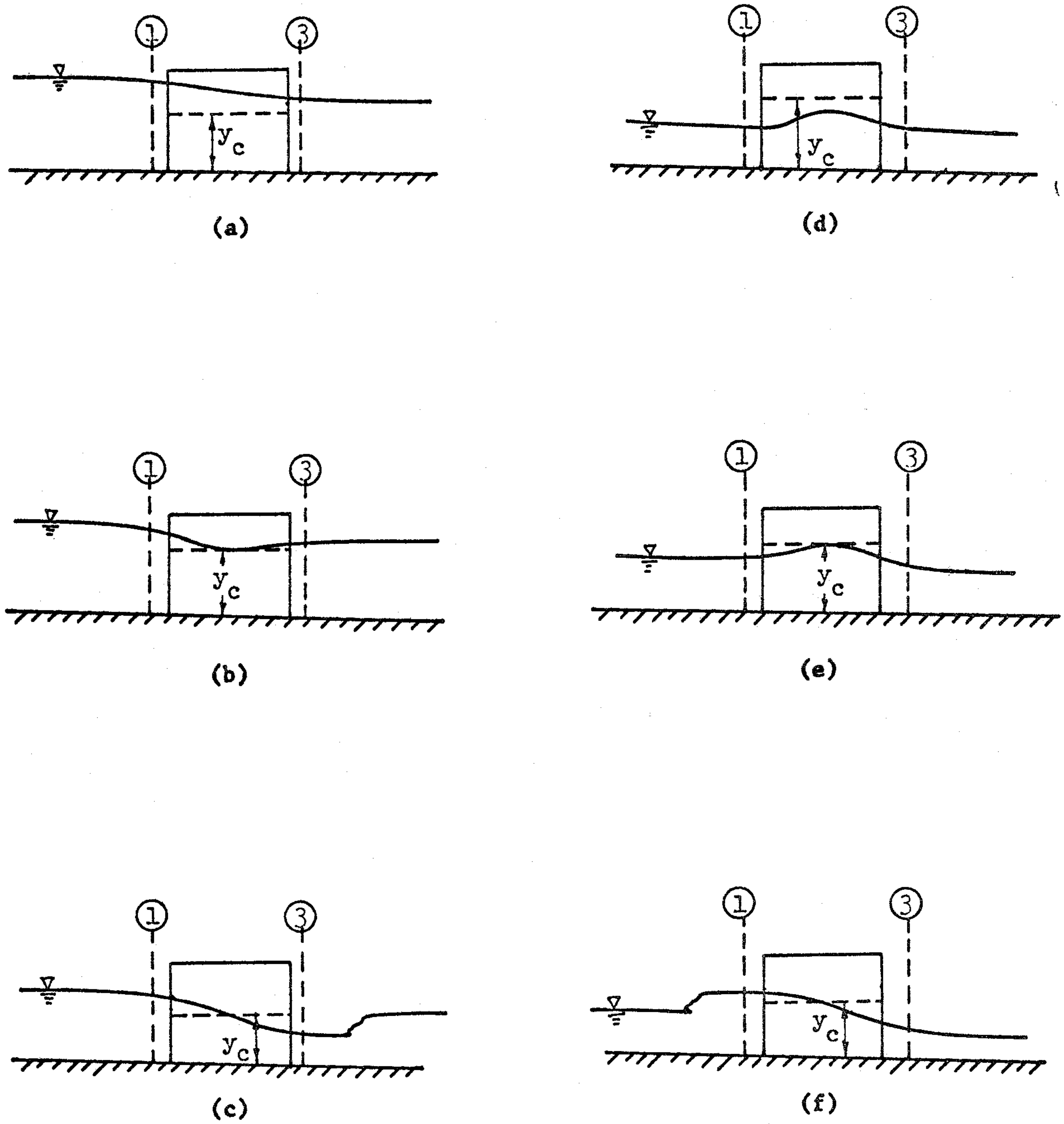

(a)
Upstream depth, y_1
$A_1\bar{y}_1 - A_{p1}\bar{y}_{p1} + \frac{Q^2}{gA_1^2}(A_1 - \frac{C_D}{2}A_{p1}) = M_2$
Momentum Flux in Constriction, M_2
(b)
Depth in Constriction, y_2
$A_2\bar{y}_2 + \frac{Q^2}{gA_2} = M_2$
M_{CRIT}
Momentum Flux in Constriction, M_2
(c)
Downstream depth, y_3
$A_3\bar{y}_3 - A_{p3}\bar{y}_{p3} + \frac{Q^2}{gA_3} = M_2$
Momentum Flux in Constriction, M_2

FIGURE 2



Mild bed slopes

Steep bed slopes

FIGURE 3

GENERAL PROGRAM LOGIC
HEC SUBROUTINE BLFLO
SPECIAL BRIDGE ROUTINE
LOW FLOW CONTROL
(TRAPEZOIDAL CHANNEL)

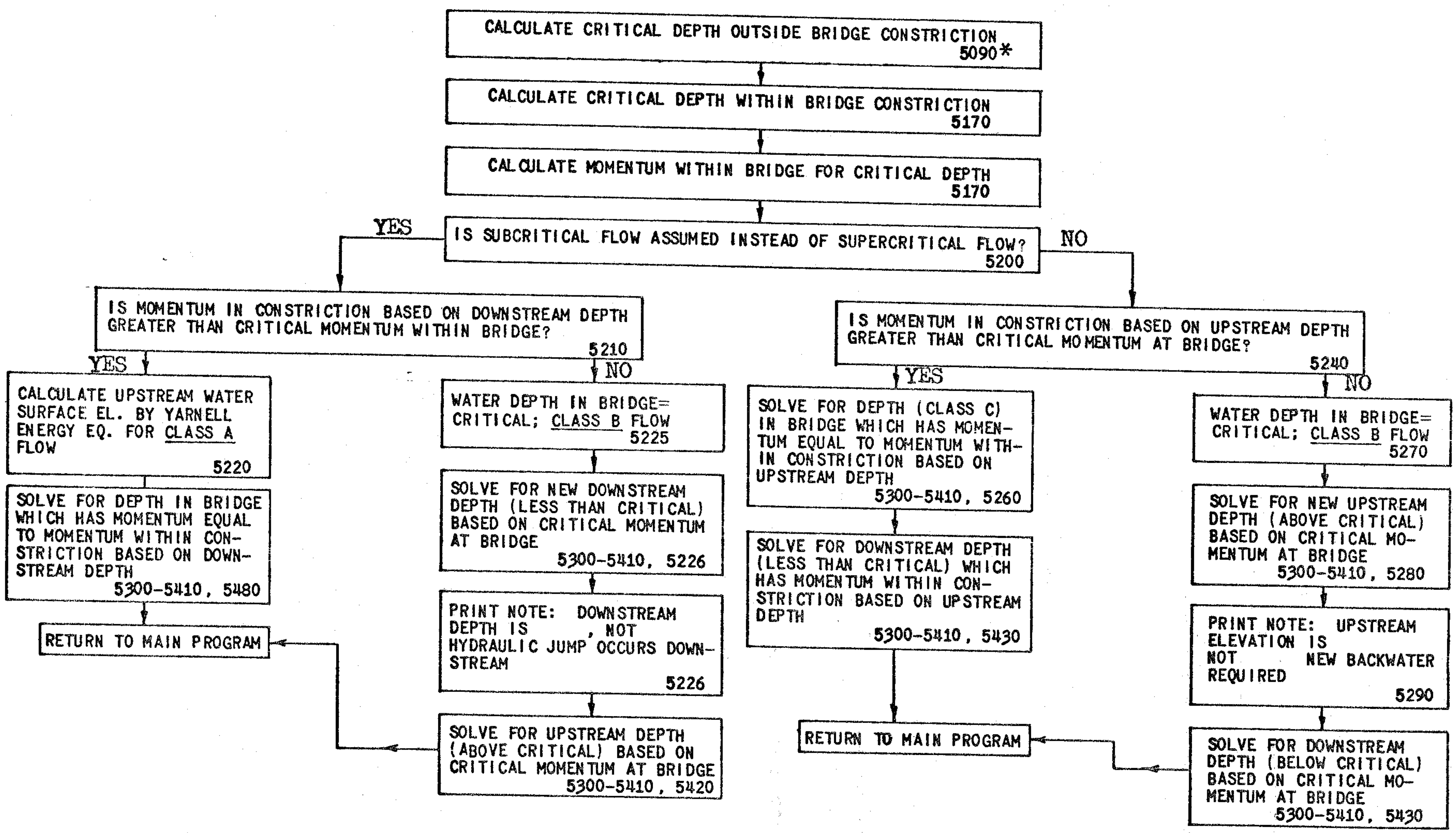
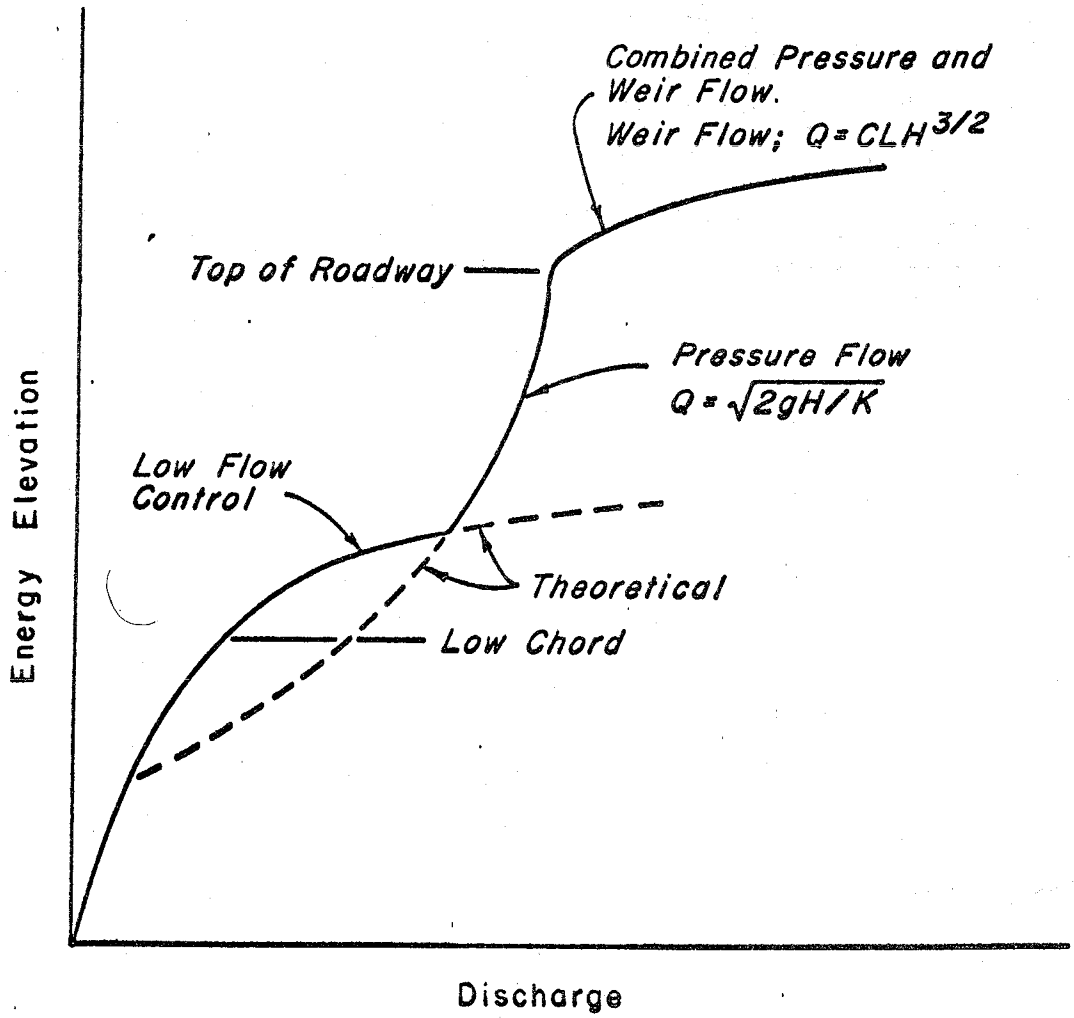


FIGURE 4

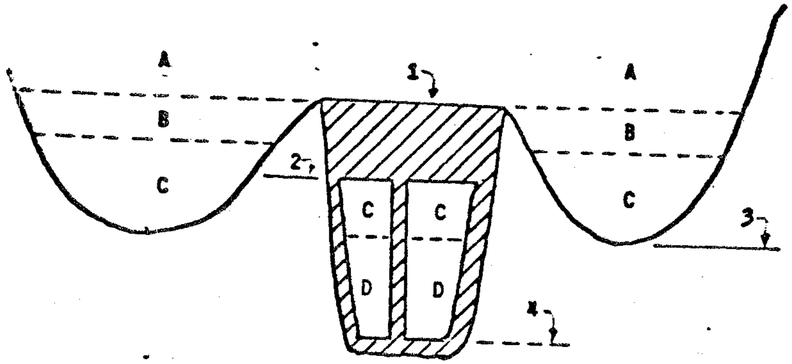
*Numbers refer to statement numbers in source deck of computer program.

FIGURE 4

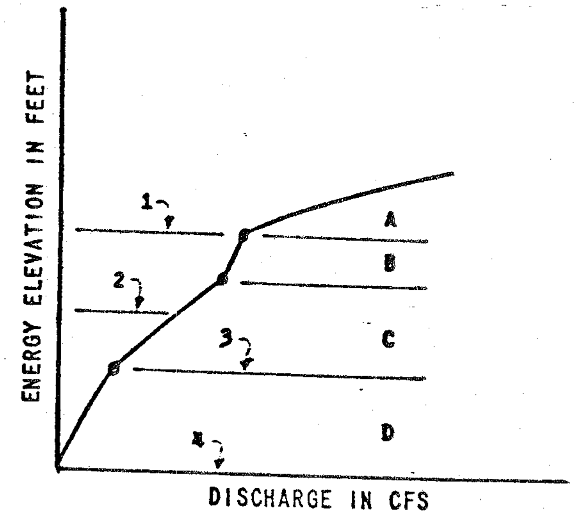


TYPICAL DISCHARGE RATING CURVE
FOR BRIDGE CULVERT

FIGURE 5



BRIDGE CROSS SECTION



BRIDGE RATING CURVE

ELEVATIONS

1 Top of Roadway

2 Low Chord

3 Roadway Approach

4 Channel Invert

FLOW CONDITION

A - Combined pressure under bridge, weir in over banks, weir over bridge

B - Combined pressure under bridge, weir in overbanks

C - Combined low flow under bridge, weir in overbanks

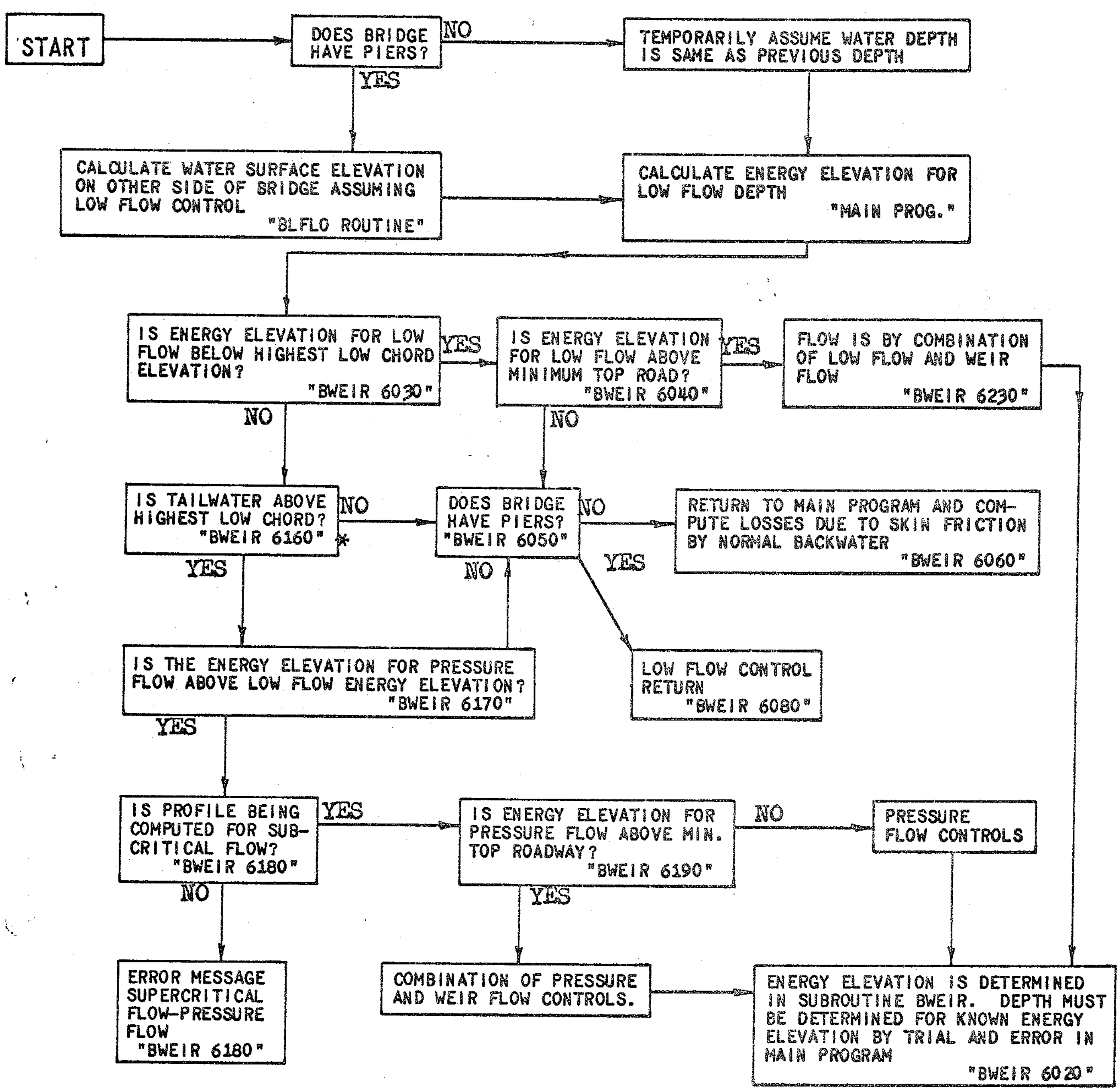
D - Low flow under bridge

FIGURE 6
ILLUSTRATION OF FLOW TYPES
BRIDGE WITH LOW APPROACHES

FIGURE 6

9

GENERAL FLOW DIAGRAM
HEC SPECIAL BRIDGE ROUTINE



*Refers to statement 6030 of computer program 22-J2-L232 subroutine BWEIR.

FIGURE 7

FLOW DIAGRAM FOR COMPUTING PROFILES
FOR
COMBINATION OF WEIR FLOW AND ORIFICE (OR LOW) FLOW
HEC COMPUTER PROGRAM 22-J2-L232
FEBRUARY 1969

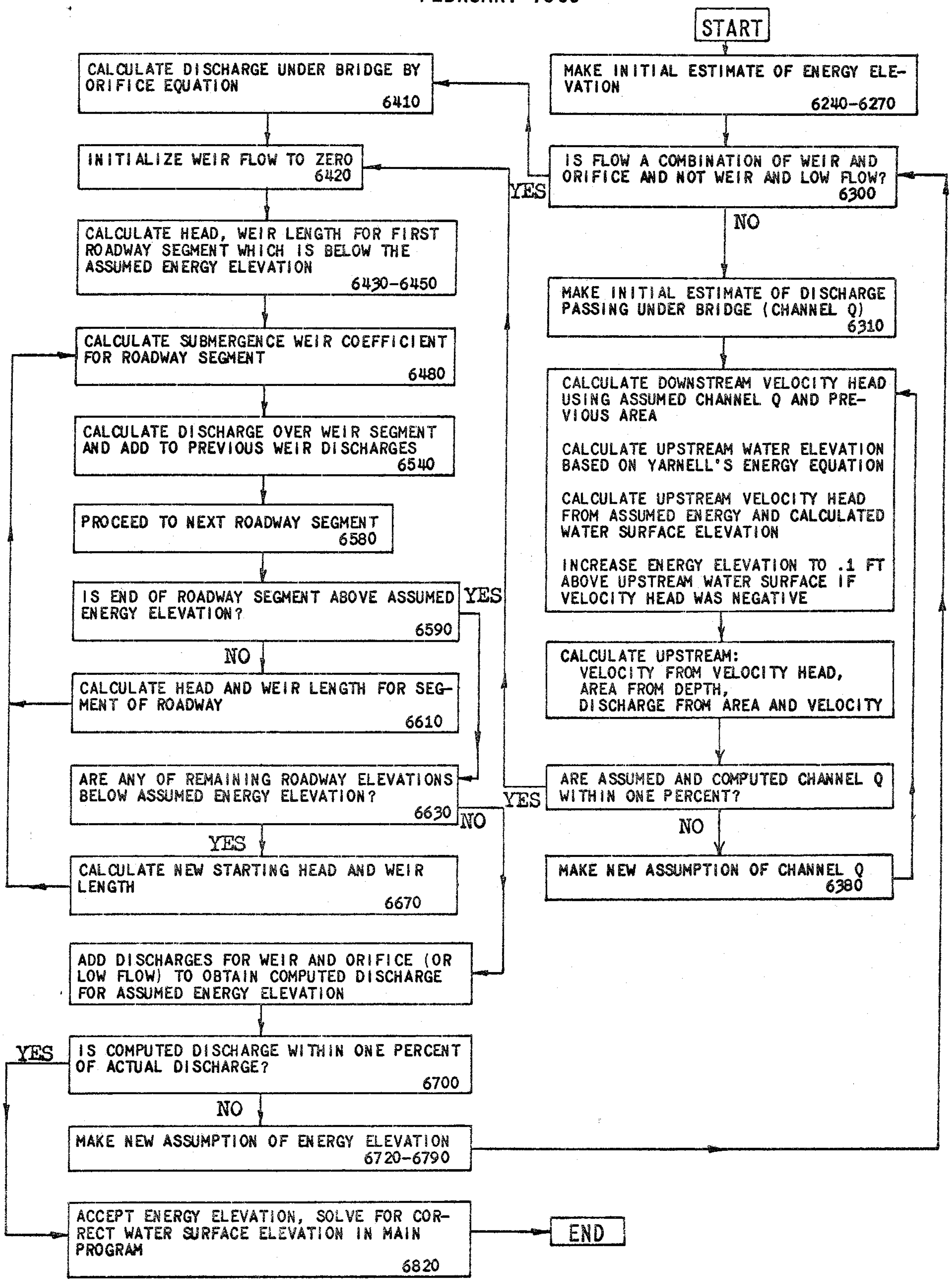


FIGURE 8

Technical Paper Series

TP-1 Use of Interrelated Records to Simulate Streamflow
TP-2 Optimization Techniques for Hydrologic Engineering
TP-3 Methods of Determination of Safe Yield and Compensation Water from Storage Reservoirs
TP-4 Functional Evaluation of a Water Resources System
TP-5 Streamflow Synthesis for Ungaged Rivers
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