THESIS

KNOWN DISCHARGE UNCOUPLED SEDIMENT ROUTING

Submitted by

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In partial fulfillment of the requirements

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WE HEREBY RECOMMEND THAT THE THESIS PREPARED UNDER OUR SUPERVISION

BY Glen Owen Brown

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ABSTRACT OF THESIS

KNOWN DISCHARGE UNCOUPLED SEDIMENT ROUTING

A known discharge, uncoupled, sediment routing model, KUWASER has been developed. The model sequentially solves the steady flow and sediment continuity equations. This procedure allows for efficient solution of sediment routing problems on large river systems. The model can perform backwater calculations and sediment routing in mainstem and multiple tributaries including divided flow reaches. The user can determine river response to river management practices such as channel improvement, realignment, dredging or tributary modifications.

The model was tested against two other models, a stage-discharge relationship and a fixed bed model by comparing the frequency of model errors in stage prediction. A sensitivity analysis was performed to determine the sensitivity of the models results to variations in six input parameters. The Yazoo River Basin in Mississippi was used as a case study to demonstrate the model capabilities. The model can be an effective tool in the prediction of river response.

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Of greatest importance to me has been the love, encouragement and indulgence of my wife Barbara McColl Brown.

DEDICATION

To my parents, William E. and Arlyne M. Brown. My first and best teachers.

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A	cross sectional area of water flow
A'	cross sectional area of total flow
A_b	cross section bed area
a	reference depth
a_i	incremental flow area
a_n	coefficient of Manning's n-value relationship
a_t	coefficient of tributary sediment rating curve
a_x	coefficient of stage-discharge relationship
$a_1 - a_{10}$	coefficients of hydraulic property relationships
b_n	coefficient of Manning's n-value relationship
b_t	coefficient of tributary sediment rating curve
b_x	coefficient of stage-discharge relationship
$b_1 - b_{10}$	coefficients of hydraulic property relationships
C	sediment concentration
c	sediment transport coefficient
C_e	coefficient of eddy loss
C_s	concentration of sediment in flow
D	vertical depth of flow, thalweg depth
D_a	average depth for incremental area
D_e	effective depth
D_{lob}	left over bank station
D_{rob}	right over bank station
D'	depth of coordinate point

d	depth of flow normal to the bed
f	Darcy-Weisbach f
g	acceleration of gravity
H	total hydraulic head
H_{ℓ}	friction head loss
$H_{\ell v}$	eddy head loss
I_1, I_2	weir coefficients
K	flow conveyance
k_i	incremental conveyance
N	number of cross section area increments
n	Manning's n-value
n_{a_i}	average Manning's n-value
n_o	initial Manning's n-value
p_i	incremental wetted perimeter
Q	water discharge
Q'	total discharge
q	lateral discharge per unit length of channel
q'	total lateral discharge
Q_{ℓ}	tributary water discharge
Q_{nps}	non-point source discharge
Q_s	sediment transport
$Q_{s\ell}$	tributary sediment discharge
$q_{s\ell}$	lateral sediment discharge per unit length of channel
R	hydraulic radius of flow
R_c	coefficient of correlation
r	sediment transport function exponent
r_i	incremental hydraulic radius

S_f	friction slope
S_h	head loss slope
$S_{\ell v}$	eddy loss slope
S_o	bed slope
S_s	sediment specific weight
s	sediment transport function exponent
t	time
U_*	shear velocity
V	mean flow velocity
v	local temporal mean velocity of main flow
W	channel width
w	particle fall velocity
W_e	effective width
X	particle settling length
X_b	width of incremental area
x	distance along channel bottom
y	depth to channel bottom
Z	channel bottom elevation
Z_b	change in depth for incremental area
Z_{ob}	over bank elevation
α	velocity head correction factor
ΔA_b	change in bed cross-sectional area
ΔV	change in sediment volume between cross sections
ΔX	horizontal distance between cross sections
γ	specific weight of fluid
κ	von Karman constant
ρ	sediment deposit porosity

ρ'	sediment deposit density
θ	bed angle to horizontal
ψ	arbitrary variable
ν	water's kinematic viscosity

Chapter 1

INTRODUCTION

Man's association with alluvial rivers extends back past the start of recorded history. The civilizations in ancient China, Mesopotamia and Egypt were all tied to the rivers they resided next to. Unfortunately the relationship between man and rivers has not been peaceful. It is the rule not the exception in alluvial river systems that blanks will erode, channels will silt in and floodplains will be flooded. Rivers follow the laws of hydraulics, not man's, thus many times to the unintiated they seem to act in a capricious fashion. A river may take years to make a small change in alignment, while a large bendway can be cut off and abandoned overnight. This study is a very modest effort toward improving our understanding of, and thus our association with the rivers we depend on so much to transport us, provide power, supply water, remove our effluent and to carry the rain out of our backyards.

The method by which this thesis tries to improve the understanding of rivers is in the prediction of river response. A river's response can be determined by either physical or mathematical physical process models. Physical models are limited by size and cannot model large systems or long periods of time. There are several process models presently in use such as RIVER (Chen, 1975) and HEC-6 (U.S. Army Corps of Engineers, 1976). While mathematical models do not have a size limitation they are usually limited to simulation periods of a year or two, because of computer costs. With complex problems involved in large basin analysis, it was necessary to develop a new model, KUWASER, in the Sedimentation Study of the Yazoo River Basin for the United States Army Corps of Engineers, Vicksburg District (Simons, Li, Brown, Chen, Ward, Doung, and Ponce, 1978). The model KUWASER is the subject of this thesis.

In determining long-term river system response, sediment movement through a river system is of primary concern and detailed information on flood wave movement is of secondary importance. Except when lateral inflow occurs, it is reasonable to assume the discharge is constant along a river reach during an individual computational time-interval. In this case the complete flow routing equations of St. Venant reduce to the equation of spatially varied steady flow. This equation can be solved along traditional lines provided significant changes in bed elevations are not occurring during the time interval. This approach, involving sequential (uncoupled) application of steady flow and sediment equations, is the basis of the present model, KUWASER. Programs of this type are called known discharge, uncoupled, sediment routing models. The known discharge, uncoupled, sediment routing formulation has allowed development of an extremely fast and efficient model, that makes the long-term simulation of sediment movement feasible.

Known-discharge formulation requires the user to identify for the mathematical model water discharge throughout the system for the entire modeling period. Water discharge is determined by either assuming steady flow or using a separate unsteady flow routing model. The model computes the water surface profile by assuming gradually varied steady flow. Since it does not linearize the momentum equation, the model accurately simulates unsteady flow profiles when coupled with an acceptable unsteady flow water routing model. Most routing programs presently used utilize a trial and error standard step method to calculate the backwater curve. However, KUWASER differs from these models in that it utilizes channel geometry relationships in an analytical, first order Newton's approximation to solve for the backwater profile. This method is more efficient than the trial and error algorithm.

The program can perform backwater calculations and sediment routing in mainstem and multiple tributaries including divided flow reaches. With this feature it is possible to determine the response of tributaries to changes on the mainstem, such as channel improvement, realignment, or dredging and to determine response of the mainstem river to tributary modifications. Therefore, the most efficient tributary management procedures can be determined to minimize the mainstem sedimentation problems.

The model uses a method that relates the change in bed elevation at a point to the flow conveyance above the point. This method predicts more accurately the distribution of aggradation or degradation in the cross section and estimates channel response more adequately than a conventional one-dimensional model.

Chapter II presents a review of the literature and the governing equations of sediment routing. Chapter III contains the model formulation, while Chapter IV presents model applications and a model sensitivity analysis comprises Chapter V. Conclusions are contained in Chapter VI. The appendices present a user's manual for the model KUWASER.

Chapter 2

GOVERNING EQUATIONS AND LITERATURE

Governing Equations

Many authors have presented the equations governing open channel flow. The best general purpose presentations included Chow (1959), Henderson (1966), and Liggett (1975). Chen (1973) developed the one-dimensional differential equations of gradually varied, unsteady flow applicable to non-uniform alluvial channels in their least restricted forms. Chen's equations are presented here. In the equations the following assumptions are made.

1. The flow is one-dimensional
2. The vertical pressure distribution is hydrostatic.
3. The flow can be described by values that are averaged over the cross section.
4. The momentum coefficient is equal to one.
5. The water surface slope is small.
6. The water sediment mixture is incompressible and of constant density.

Figure 2.1 shows an alluvial channel where:

A' = the channel cross sectional area of total flow (water and sediment)

A_b = the cross section bed area

D = depth of flow

Q' = the stream total discharge (water and sediment)

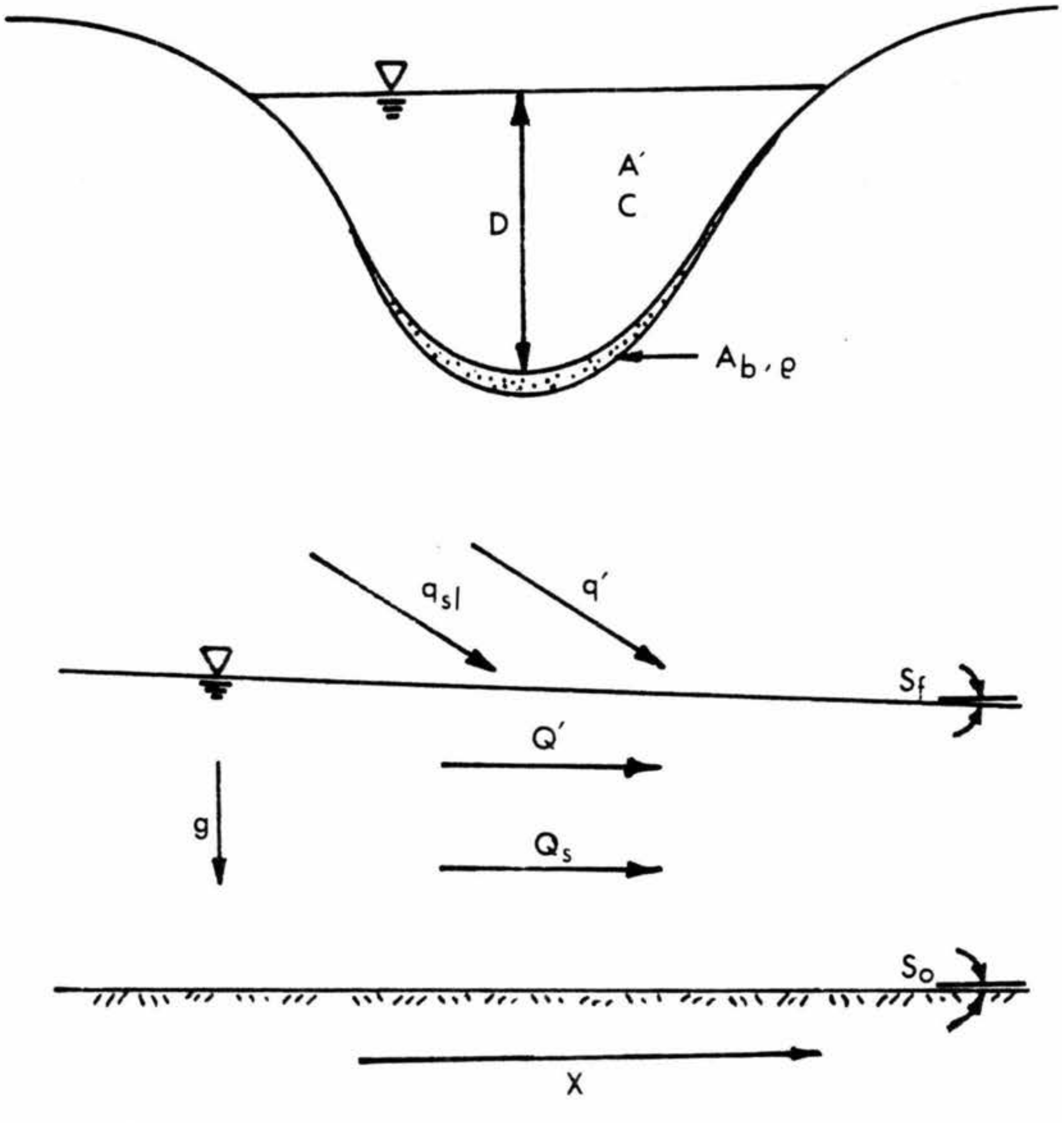

Figure 2.1. Open channel alluvial flow.

q' = the lateral total discharge (water and sediment)

Q_s = the stream sediment discharge

$q_{s\ell}$ = the lateral sediment discharge

g = acceleration of gravity

C = concentration of sediment in flow

S_o = bed slope

S_f = friction slope

t = time

x = distance along channel

ρ' = deposited sediment density

Water Continuity

The equation of water continuity for sediment-laden water is:

$$\frac{\partial Q'}{\partial x} + \frac{\partial A'}{\partial t} + \frac{\partial A_b}{\partial t} - q' = 0 \tag{2.1}$$

The first term on the left is the change in discharge along the channel, or the "channel storage," the second is the change in flow area over time, or the "rate of rise," the third is the change in sediment eroded or deposited over time from the bed or the "rate of channel erosion," and the fourth is lateral inflow. This equation assumes an incompressible fluid but allows for a nonhomogeneous sediment content.

Sediment Continuity

The equation of sediment continuity is:

$$\frac{\partial Q_s}{\partial x} + \rho' \frac{\partial A_b}{\partial t} + \frac{\partial}{\partial t} (A'C) - q_{s\ell} = 0 \tag{2.2}$$

The first term on the left is the change in sediment discharge along the reach, or "the channel sediment storage", the second is the change in sediment eroded or deposited over time, or the "rate of erosion," the third is the change in sediment suspended over time or the "rate of suspension" and the fourth is the lateral sediment inflow.

Equation of Motion

The equation of motion for sediment laden water based on momentum principles is:

$$\frac{1}{gA'}\frac{\partial Q'}{\partial t} + \frac{1}{gA'}\frac{\partial}{\partial x}\left(\frac{Q'^2}{A'}\right) + \frac{\partial D}{\partial x} = S_o - S_f \tag{2.3}$$

The first term on the left is the force caused by the "local acceleration," the second is the force caused by the "convective acceleration," and the third is the pressure forces acting along the channel or the "pressure gradient." On the right side of the equation the first term represents gravitational forces or the "bottom slope" and the second shear forces, or the "friction slope."

Manning's Equation

Many equations have been developed to determine channel resistance. The Manning equation is the most well known and used, and is thus used here. The Manning equation is an empirical, uniform, steady flow equation. Errors are made when it is applied to non-prismatic natural channels with unsteady flow. It is believed that these errors are small when compared with those ordinarily occurring in the use of an uniform flow formula in natural channels (Chow, 1959). Likewise, it is believed there is no need to differentiate between total flow and water flow.

The Manning equation is:

$$Q = \frac{1.486}{n} A\, R^{2/3}\, S_f^{\,1/2} \text{ (English units)} \tag{2.4}$$

where Q = water discharge

A = water flow area

R = hydraulic radius

n = Manning channel roughness factor

The channel conveyance K, is defined by:

$$K = \frac{1.49}{n} A R^{2/3} \tag{2.5}$$

Combining Equations 2.4 and 2.5 yields:

$$Q = K S_f^{1/2} \tag{2.6}$$

Thus channel discharge is proportional to conveyance. Solving for the friction slope:

$$S_f = (\frac{Q}{K})^2 \tag{2.7}$$

Equation 2.7 can be directly solved only for steady, uniform flow. In all other cases it must be solved in an iterative process.

Sediment Transport

No theoretically exact equation for sediment transport has been developed. The reason for this lack is two-fold. First, the processes by which flowing water imparts energy to sediment particles is poorly quantified. Second, not all sediment particles are created equal. They vary in size, shape, and density. Some particles in the clay sizes are influenced strongly by ionic forces, some are not. In a given flow some particles travel entirely in suspension, some entirely by rolling along the bed, and others by a combination. Some particles can be supply limited at a given moment while at the same time other size particles are transport limited. Considering the complexity and variability of alluvial systems a complete, theoretical solution to the sediment transport equation may evade us for a long time. For the present, the engineer can select from numerous empirical equations. Simons and Sentürk (1977) present a review of the most widely known equations. Laursen (1956) showed that several bed load formulas can be reduced to the form:

$$Q_s = c\ V^r\ D^s \tag{2.8}$$

where V is the stream velocity, and c is an empirical coefficient which is a function of the characteristics of the channel and sediment. The exponent r is assumed greater than 0 and s less than 0. In the most widely used equations, $3 \leq r \leq 6$, and $-2 \leq s \leq -2/3$. Equations such as 2.8 are only valid for particle sizes which are transport limited.

Solution Methods

The solution of Equations 2.1, 2.2, 2.3, 2.4 and 2.8 will provide the characteristics of a gradually varied, unsteady flow in an alluvial channel. An exact solution requires the simultaneous solution of Equations 2.1, 2.2 and 2.3 with Equations 2.4 and 2.8 evaluated on a continuous temporal and spatial basis. Because of the complexities involved, no such complete solution has been accomplished. Application of this theory requires the development of simplified solutions. There are almost as many simplifying solution methods for these equations as there are researchers in the field. This section will review the different solution techniques that have been tried.

Solution methods can be classified by three criteria: (1) the number of terms that are solved in each of Equatons 2.1, 2.2. and 2.3; (2) the order in which Equations 2.1, 2.2 and 2.3 are solved; and (3) whether Manning's equation is linearized in the solution of the equation of motion.

Reducing Terms

Equation 2.1 can be rearranged to show an overview of the different approximations:

$$\frac{\partial A'_d}{\partial t} + \frac{\partial A'}{\partial t} + \frac{\partial Q'}{\partial x} - q = 0 \tag{2.10}$$

steady flow

unsteady flow
uncoupled sediment

unsteady flow
coupled sediment

With all terms the equation considers unsteady flow of water and sediment. By dropping the first term the sediment continuity is "uncoupled" from the water continuity. The variables A' and Q', total flow area and volume are reduced to A and Q water flow area and volume. The errors made in the uncoupled form are reduced by decreasing sediment concentration. Dropping the second term reduces the equation to the simple steady flow formulation. In the last case the partial derivative becomes a full derivative. The errors made in the steady flow form are reduced as $\partial A'/\partial t$ decreases.

Equation 2.2 rearranged to show an overview of the different approximations appears as follows:

$$\frac{\partial}{\partial t}(A'C) + \rho' \frac{\partial A_d}{\partial t} + \frac{\partial Q_s}{\partial x} - q_{s\ell} = 0 \tag{2.11}$$

reduced continuity

strict continuity

Dropping the first term which accounts for the change in volume of sediment in motion reduces Equation 2.11 from strict continuity to a reduced continuity form. The error made in the reduced continuity form decreases as the time increment in a finite difference solution increases.

Equation 2.3 can be rearranged to show an overview of the different approximations.

$$\frac{1}{gA'}\frac{\partial Q'}{\partial t} + \frac{1}{gA'}\frac{\partial}{\partial x}\left(\frac{Q'^2}{A'}\right) + \frac{\partial D}{\partial x} - S_o + S_f = 0 \tag{2.12}$$

steady uniform flow

steady nonuniform non-inertial flow

steady nonuniform flow

unsteady nonuniform flow

The momentum contribution of lateral inflow has been dropped for clarity, but it can be added to either of the first two forms. Again as in the water continuity equation, by uncoupling the water and sediment the variables Q' and A' are reduced to Q and A. Dropping the first term of the equation yields the steady nonuniform flow equation. The errors made using this approximation decrease with decreasing $\partial Q/\partial t$. Dropping the second term yields the steady nonuniform non-inertial form. This form assumes the inertia term is negligible when compared to the pressure gradient, friction and gravity terms. Dropping the third term yields the steady uniform flow form. This form assumes only the friction and gravity terms are important.

Order of Solution

The order by which Equations 2.1, 2.2 and 2.3 are solved directly affects the solution. The three equations are solved most accurately by simultaneous solution. Sequential or "uncoupled" solutions which solve the equations one or two at a time greatly reduce computation effort but reduce accuracy.

The sediment continuity equation is often uncoupled from the water continuity and motion equations. When this is done the solution of Equations 2.1 and 2.3 (in whatever form) is used to calculate the sediment transport and continuity for a given time period. Thus the stream channel is considered to have constant geometry during a time period.

Coupling of the various forms of the equations of motion and water continuity yield several approximations. Table 2.1 presents the different approximations. Garbrecht (1981) explains each of the unsteady methods. Two combinations are not named. While the combinations are possible no application of them is known.

When the equations of motion and water continuity are uncoupled the solution is classed as a "known discharge" method. The term known discharge refers to computation order where water continuity is solved before the equation of motion. In known discharge solutions the equation of motion can be coupled or uncoupled with the sediment continuity equations. Known discharge solutions can be unsteady if the local acceleration term is used in the equation of motion or steady if it is deleted.

Manning's Equation Solution

The equation of motion requires the solution of Manning's equation for the friction slope. In finite difference method the friction slope term is replaced by Equation 2.7. For exact solutions the friction slope must be evaluated at each time-space point. The friction slope is a quadratic function of Q and K and contributes a considerable amount of computations to any method. Exact solutions are sometimes dropped for computing savings. In the linear-implicit scheme (Chen, 1973), the friction slope is evaluated only on a specific time line and is estimated on the next. This is required to assure stability.

Table 2.1. Approximations Yielded by Various Combinations of the Equations of Motion and Water Continuity

Equation of Motion	Water Continuity	
	Unsteady Flow	Steady Flow
Unsteady Nonuniform Flow	Dynamic Wave	--
Steady Nonuniform Flow	Quasi-steady Dynamic Wave	Gradually Varied, Steady Flow
Steady Nonuniform Noninertial Flow	Diffusion Wave	--
Steady Uniform Flow	Kinematic Wave	Steady, Uniform Flow

Literature

As stated above there are almost as many solution methods for the three basic equations of alluvial flow as there are researchers in the field. This section will outline solutions that are known.

Chen (1973) developed an unsteady, uncoupled, sediment routing model which used a linear implicited, finite difference solution. Sediment transport, was calculated by Einstein's (1950) bed load function and Toffaleti's (1969) method. Erosion and deposition were distributed uniformly across a section. The effects of bed armoring were considered. The model was successfully applied to the Neuse, Mississippi and Missouri Rivers. The applications covered less than a year each. Chang and Richards (1971) presented a similar model using a method of characteristics solution.

Dass (1975) developed an unsteady, uncoupled, compound stream model. The model was applied to Pool 4 of the Mississippi River. Simons and Chen (1976) developed a unsteady, coupled model using a linear implicient solution also applied to Pool 4. Mahmood (1975) and Mahmood and Ponce (1976) used a unsteady, uncoupled, linear implicit method to model simple bed transients.

The U.S. Army Corps of Engineers (1976) developed a known discharge, steady flow, uncoupled, sediment routing model, HEC-6. Sediment transport is calculated by the Toffaleti method and erosion and deposition are distributed evenly across the wetted channel. The effects of bed armoring were considered. Amar and Thomas (1976) used the model on the Atchofaloya River. Approximately 150 miles of channel were modeled for ten years. The model gave good results but requires excessive computer time.

Chang and Hill (1976) developed a known discharge, steady flow, uncoupled, sediment routing model. The water surface profile was calculated by a variation of HEC-2 by the U.S. Army Corps of Engineers (1973). Erosion and deposition were distributed by a shear stress weighted function. The model was applied to a small drainage channel with success. Since the model uses the HEC-2 backwater calculations it requires excessive computer time.

Owens (1977) developed an unsteady, known discharge, coupled water and sediment model. His model included the rate of rise term in the equation of motion and coupled the sediment continuity and motion equations. The model allowed for compound stream geometry where main channel and overbank flows were differentiated. The model was applied to Pool 4 in the Upper Mississippi and Lower Chippewa Rivers, (approximately 10 miles). Sediment transport was calculated by an equation similar to Equation 2.8. Owens concluded that the known discharge, coupled, solution is nearly the same as Simons and Chen (1976) unsteady, coupled solution. Computation time was about the same. Two methods for distributing erosion and deposition were evaluated, one conveyance weighted and one velocity weighted. The conveyance weighted was generally better.

Garcia (1977) developed a known discharge, unsteady, coupled bed transient model. The model used an implicit solution of the sediment continuity and motion equation. The model was applied to a simple, prismatic channel for short time periods. While successful, no real world applications were proposed.

Selection of Model

The only model formulation which has been applied successfully to a case study near the size and duration required for this research is the known discharge, uncoupled, sediment routing model (the U.S. Army Corps of Engineers, 1976). The known discharge uncoupled formulation is not as elegant as other models but Owens (1977) has stated there is no evidence available to show one model superior to another. Because of its simplicity the known discharge, steady flow, uncoupled method is the easiest to apply, most economical to use, and possibility the most accurate, since it does not have the numerical solution problems of unsteady or coupled methods and it does not linearize the Manning's equation. Since water routing is of only secondary concern the unsteady known discharge formulation does not impose any severe limitations. However with uncoupling the equations of motion and sediment continuity care must be taken that the bed change during any single time period does not effect the water surface profile. In the necessary application to a large mild slope river, that limitation would not be a hinderance.

None of the previously mentioned models are very computationally efficient. HEC-6 has a very slow backwater method and it is felt that the sediment routing methods can be improved. It was therefore concluded that a new known discharge, steady flow, uncoupled, sediment routing model would be developed.

Chapter 3

MODEL FORMULATION

Water Surface Profile Computations

General

The water surface profile is computed assuming one-dimensional, spatially varied, steady flow. This implies the following assumptions: 1) the hydraulic characteristics of flow remain constant for the time interval under consideration; 2) the flow streamlines are practically parallel, i.e., a hydrostatic pressure distribution prevails over the channel section, and 3) the secondary flow (lateral or cross-stream) is negligible when compared to the longitudinal flow. A further assumption is that the frictional loss at a section is the same as for uniform flow with the same velocity and hydraulic radius. In addition to these basic assumptions, others will be made when appropriate, the most important being that the slope of the channel is so small that the depth of flow is assumed the same whether vertical or normal to the channel bottom.

Dynamic equations for spatially varied flow can be obtained for three different approaches. These approaches are: 1) momentum, 2) total head, and 3) energy. While the water surface profiles computed by each method are identical, the derived equations themselves are only identical for the special conditions of steady, uniform flow. Because of difficulties in using the momentum and energy equations on non-prismatic channels, the total head approach is used in the program.

The following is a brief derivation of the one dimensional dynamic equation for spatially varied flow from the total head equation. Yen and Wenzel (1970) and Li (1972) contain completed derivations of the spatially varied flow equations by all three methods and comparison of the results.

Continuity Equation

Figure 3.1 shows an incremental length dx of an open channel with a spatially varied flow profile. The continuity equation for the increment is:

$$\frac{dQ}{dx} = q \tag{3-1}$$

where Q is the channel discharge and q is the lateral inflow per unit channel length. Considering the incompressibility of water, $Q = VA$ and:

$$\frac{d(VA)}{dx} = q \tag{3-2}$$

Dynamic Equation of Spatially Varied Flow

The total hydraulic head above a selected datum at the upstream section is given by

$$H = Z + d \cos \theta + \alpha \frac{V^2}{2g} \tag{3-3}$$

where H is the total head above the horizontal datum, Z is the elevation of the channel bottom above the datum, d is the depth of flow, α is the velocity head correction factor, defined as $\frac{1}{Av^3} \int v^3 \, dA$, where v is the local mean temporal velocity of flow, θ is the bottom slope angle, and g is the acceleration due to gravity. By taking the bottom of the channel as the x axis and differentiating Equation 3-3 with respect to x, the following equation is obtained:

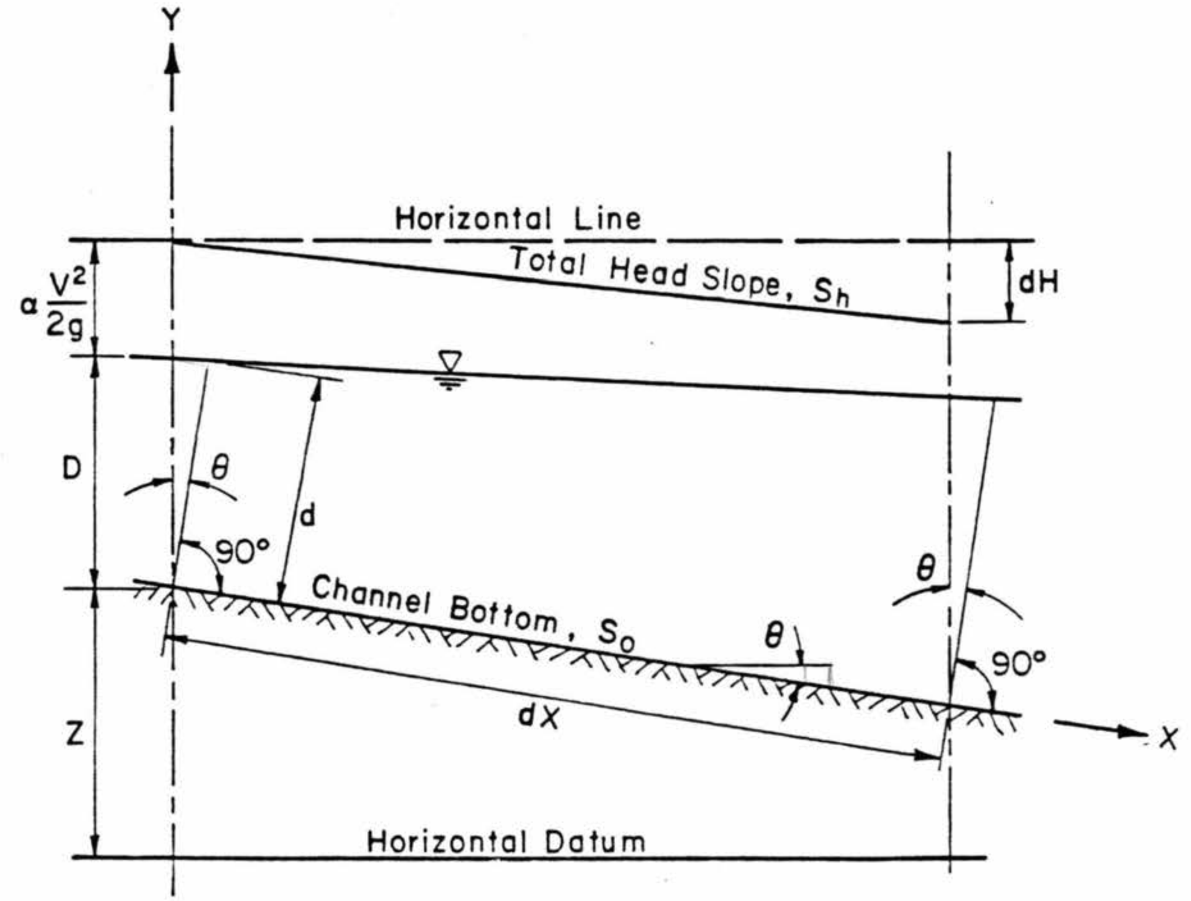

Figure 3.1 Incremental length of channel.

$$\frac{dH}{dx} = \frac{dZ}{dx} + \cos\theta \frac{dd}{dx} + \frac{d}{dx}\left(\alpha \frac{V^2}{2g}\right) \tag{3-4}$$

The total head slope S_h is then defined as:

$$S_h = - \frac{dH}{dx} \tag{3-5}$$

and the bed slope, S_o is expressed as:

$$S_o = - \frac{dZ}{dx} \tag{3-6}$$

Substituting Equations 3-5 and 3-6 into Equation 3-4 and solving for dd/dx yields:

$$\frac{dd}{dx} = \frac{1}{\cos\theta} \left[\left(S_o - S_h - \frac{d}{dx}\left(\alpha \frac{V^2}{2g}\right)\right]\right. \tag{3-7}$$

which is the general differential equation for spatially varied steady flow derived from the total head. It represents the slope of the water surface with respect to the channel bottom. In most cases the angle θ is so small that $\cos\theta \cong 1$, and $d \cong D$. Placing these approximations into Equation 3-7 results in:

$$\frac{dD}{dx} = S_o - S_h - \frac{d}{dx} \left(\alpha_2 \frac{V^2}{2g}\right) \tag{3-8}$$

where D is the depth of flow measured vertically from the bed. In irregular channels D is the thalweg depth. Chow (1959) and Henderson (1966) present additional spatially and gradually varied flow theories.

Manning's Equation

Total head slope S_h is computed by $S_h = S_f + S_{\ell v}$ where S_f is the friction slope, and $S_{\ell v}$ is the slope of head losses due to other factors.

The friction slope is evaluated using the empirical Manning's equation:

$$S_f = \left(\frac{Qn}{1.49A\ R^{2/3}}\right)^2 \qquad \text{(English units)} \tag{3-9}$$

where R is the hydraulic radius of flow and n is the Manning's coefficient of channel roughness.

Manning's equation was developed for uniform steady flow, therefore, using Equation 3-9 for spatially varied flow produces errors. However, the errors are believed to be small compared with those ordinarily incurred using the uniform-flow formula and in the selection of the roughness coefficient (Chow 1959).

Manning's n-value for alluvial streams is not constant but is a function of discharge and depth (Simons and Sentürk, 1977). While there are complicated procedures to determine channel roughness, which are fairly exact, in the mathematical model Manning's n-value is made a simple function of discharge, $n = n_o a_n Q^{b_n}$ where a_n and b_n are empirically determined coefficients, and n_o is the initial value of Manning's n. The channel conveyance is defined by

$$K = \frac{1.49}{n} A\ R^{2/3} \tag{3-10}$$

where K is the conveyance. Combining Equations 3-10 and 3-~~11~~9 yields:

$$S_f = \left(\frac{Q}{K}\right)^2 \tag{3-11}$$

Eddy Losses

The slope $S_{\ell v}$ is the slope of the head losses due to all factors except friction and is nominally referred to as the eddy loss slope.

No rational method is available to evaluate eddy losses. Eddy loss depends mainly on velocity head change and may be expressed as a part of it or:

$$S_{\ell v} = C_e \ [(\alpha_2 V_2^2 - \alpha_1 V_1^2)/2g] \ dx \tag{3-12}$$

where C_e is an empirical coefficient. For gradually converging and diverging reaches, C_e = 0 to 0.1 and 0.2 respectively.

Standard Step Method

The mathematical model uses a finite difference standard step method to solve Equation 3-8 for the water surface profile. The computations are carried out moving upstream cross section by cross section from a known water surface. Figure 3.2 shows a typical channel reach.

The total head and head loss between the two cross sections are equated:

$$Z_2 + D_2 + \alpha_2 \frac{V_2^2}{2g} = Z_1 + D_1 + \alpha_1 \frac{V_1^2}{2g} + H_\ell + H_{\ell v} \tag{3-13}$$

where H_ℓ is the friction loss. H_ℓ may be written as:

$$H_\ell = S_f \ \Delta X \tag{3-14}$$

where ΔX is the horizontal distance between cross sections, and $H_{\ell v}$ is the loss due to all other factors. $H_{\ell v}$ may be written as:

$$H_{\ell v} = S_{\ell v} \ \Delta X \tag{3-15}$$

The average of the friction slopes at the two cross sections is used to compute the friction loss. Thus combining Equations 3-11 and 3-14 results in:

$$H_\ell = \frac{\Delta X}{2} \ \frac{Q_1^2}{K_1} + \frac{Q_2^2}{K_2} \tag{3-16}$$

The continuity equation can be written

$$Q = VA \tag{3-17}$$

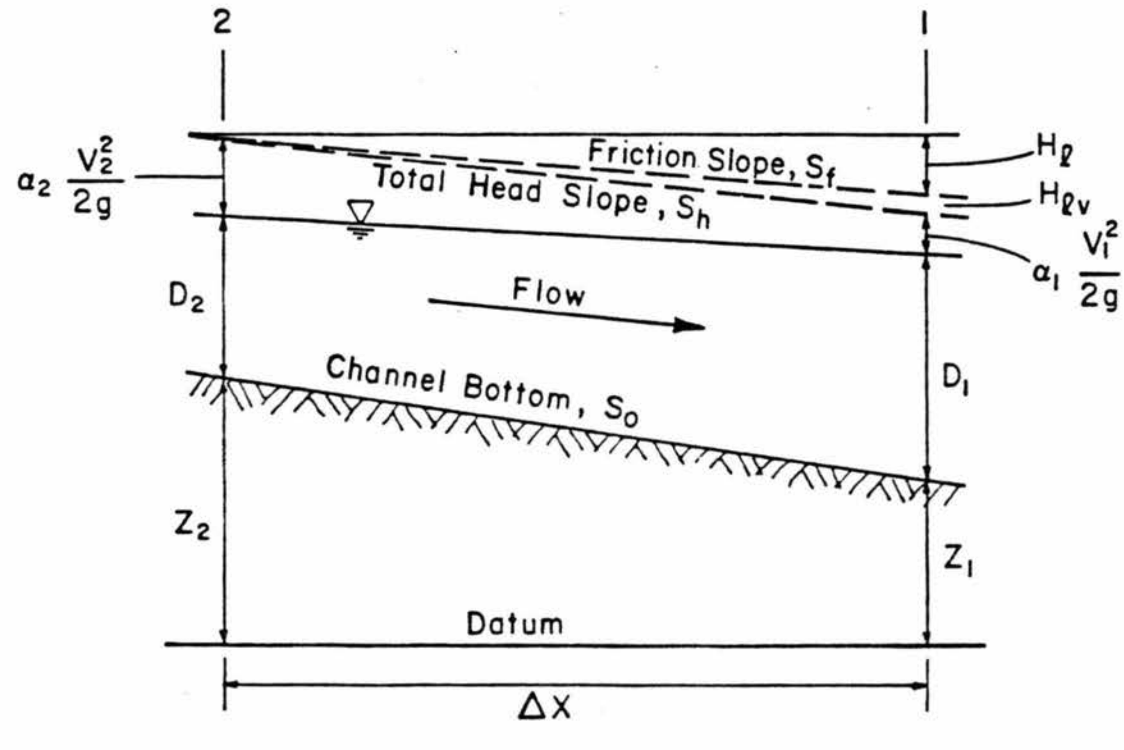

Figure 3.2 Channel reach for standard step method.

Total head at the downstream cross section, H_1, is known and is determined by:

$$H_1 = Z_1 + D_1 + \alpha_1 \frac{V_1{}^2}{2g} \tag{3-18}$$

Combining Equations 3-9, 3-12, 3-16, 3-17 and 3-18 yields:

$$\alpha_2 \left(\frac{Q_2}{A_2}\right)^2 \frac{1}{2g} + D_2 + Z_2 - \frac{\Delta X}{2}\left(\frac{Q_1^2}{K_1} + \frac{Q_2^2}{K_2}\right) - C_e\left(\frac{\alpha_2 V_2^2 - \alpha_1 V_1^2}{2g}\right) = H_1 \tag{3-19}$$

By starting at a known downstream water surface and proceeding upstream one cross section at a time, the water surface profile is computed.

Computation of Hydraulic Properties

The solution of Equation 3-19 requires determination of channel hydraulic properties, area, conveyance, and velocity coefficient at various depths. In backwater computations the hydraulic properties are computed with relationships developed using digitized channel geometry.

Coordinate Points. Channel cross sections are defined by (x,z) sets of coordinates. Figure 3.3 shows a typical cross section. To allow for different Manning's n-values across the section three subdivisions are made: Right Over Bank, Main Channel, and Left Over Bank. The subsections are divided by the two stations D_{lob} and D_{rob}. The Manning's n-value at each coordinate point is set according to its location.

Area. Area of flow for a given water surface elevation is computed by summing incremental areas between consecutive coordinates of the cross section. Figure 3.4 illustrates this technique. Total area of flow is the summation of the increment areas, a_i:

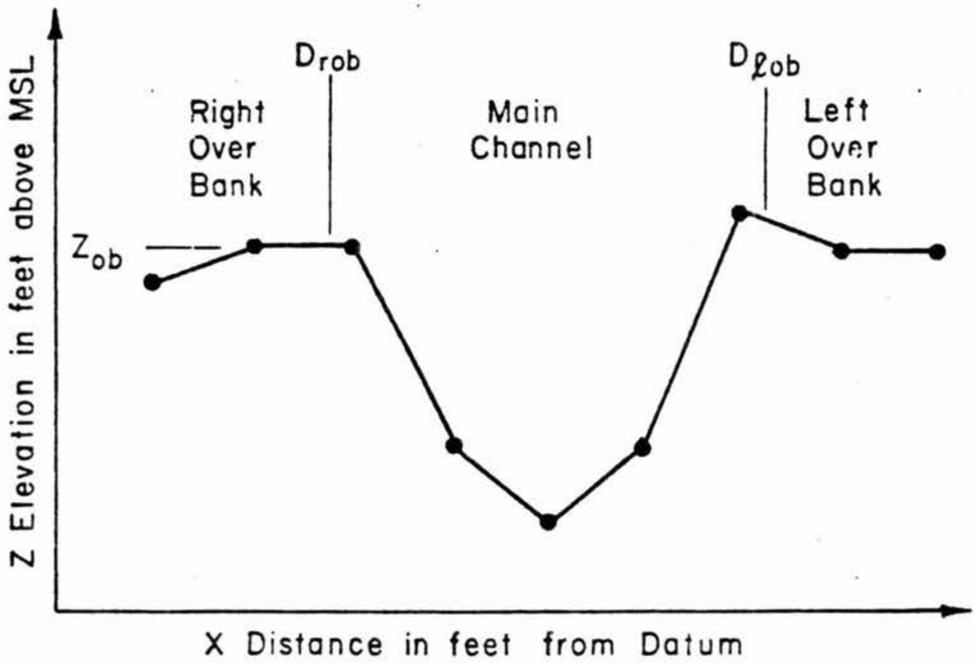

Figure 3.3 Typical channel cross section with subdivisions.

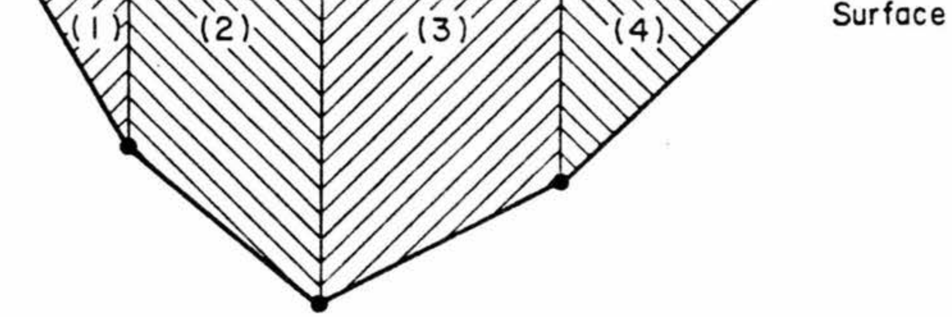

Figure 3.4 Incremental areas in a cross section.

$$A = \sum_{i=1}^{N} a_i \tag{3-20}$$

where N is the total number of cross section incremental areas. Incremental areas are computed by:

$$a_i = X_b \; D_a \tag{3-21}$$

where X_b is defined in Figure 3.5. D_a is defined as:

$$D_a = \frac{1}{2} \, (D'_1 + D'_2) \tag{3-22}$$

where D'_1 and D'_2 are defined in Figure 3.5. If the water surface intercepts the cross section between coordinate points as shown by increment 4 in Figure 3.4, straight line interpolation between the points is used to compute the triangular area. If the water surface is above a coordinate end point (first or last points), the area of flow is determined by extending a vertical line to the water surface from the end point as shown in Figure 3.6.

In many rivers, especially those with small gradients, man-made or natural levees reduce the area of flow until they are topped. Overbank flow area is not considered until the water surface exceeds the elevation Z_{ob} shown in Figure 3.3.

Wetted Perimeter. The wetted perimeter p_i is the length of the cross section below the water surface and is computed in increments by:

$$p_i = \sqrt{X_b^{\,2} + Z_b^{\,2}} \tag{3-23}$$

where Z_b is defined in Figure 3.5.

Hydraulic Radius. The incremental hydraulic radius, r_i, is calculated by:

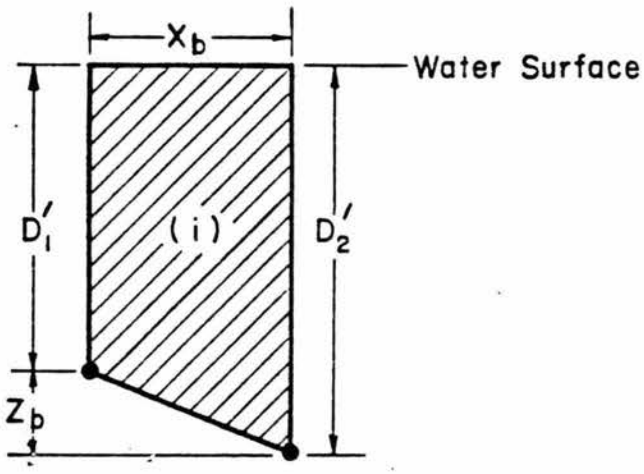

Figure 3.5 Incremental cross section area.

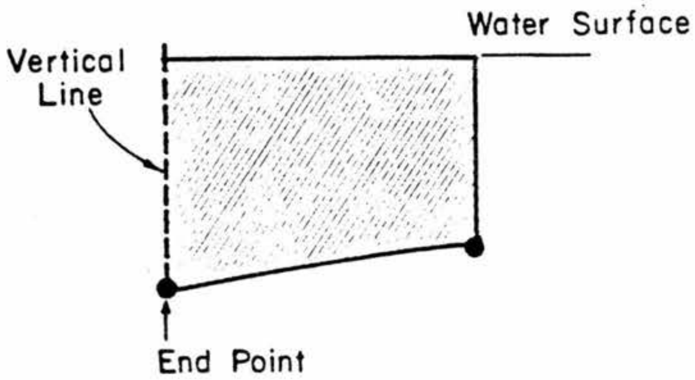

Figure 3.6 Flow area at end of cross section.

$$r_i = \frac{a_i}{p_i} \tag{3-24}$$

Conveyance. The total cross section conveyance is computed by summing the incremental conveyance:

$$K = \sum_{i=1}^{N} k_i \tag{3-25}$$

where k_i is the incremental conveyance and is computed by:

$$k_i = \frac{1.49}{n_{a_i}} a_i \, r_i^{2/3} \tag{3-26}$$

where n_{a_i} is the average Manning's n value at the two coordinate points which define the increment.

Alpha. Alpha, the velocity distribution factor, is used to account for distribution of flow across the cross section and not vertical shape of the velocity profile. Alpha is calculated by:

$$\alpha = \frac{\sum_{i=1}^{N} \left(\frac{k_i^3}{a_i^2}\right)}{K^3/A^2} \tag{3-27}$$

Effective Depth and Effective Width. Two conveyance weighed parameters, effective depth, D_e, and effective width, W_e, are used in the sediment transport calculations to represent the average hydraulic properties and are calculated as:

$$D_e = \frac{\sum_{i=1}^{N} \frac{D_{a_i}^{5/3} a_i}{n_{a_i}}}{\sum_{i=1}^{N} \frac{D_{a_i}^{2/3} a_i}{n_{a_i}}} \tag{3-28}$$

$$W_e = \frac{\sum_{i=1}^{N} D_{a_i}^{2/3} a_i}{D_e^{5/3}} \tag{3-29}$$

The effective depth and effective width values are not used in the water surface profile calculations.

Channel Hydraulic Property Relationship. Hydraulic property relationships are computed to relate area of flow, conveyance, alpha, effective depth, and effective width to the thalweg depth. Two separate sets are calculated: one set for main channel flow and a second for overbank flow. The hydraulic property relations are:

Main Channel Flow ($D \leq Z_{ob}$)

$$W_e = a_1 D^{b_1} \tag{3-30}$$

$$D_e = a_2 D^{b_2} \tag{3-31}$$

$$A = a_3 D^{b_3} \tag{3-32}$$

$$K = a_4 D^{b_4} \tag{3-33}$$

$$\alpha = a_5 D^{b_5} \tag{3-34}$$

Overbank Flow ($D > Z_{ob}$)

$$W_e = a_6 D^{b_6} \tag{3-35}$$

$$D_e = a_7 D^{b_7} \tag{3-36}$$

$$A = a_8 D^{b_8} \tag{3-37}$$

$$K = a_9 \, D^{b_9} \tag{3-38}$$

$$\alpha = a_{10} \, D^{b_{10}} \tag{3-39}$$

where D is the thalweg depth and a_1 to a_{10} and b_1 to b_{10} are computed coefficients. The overbank relationship includes both the overbank and main channel flow for depths greater than Z_{ob}.

Coefficients of the hydraulic properties relations are determined by 1) calculating the hydraulic properties of each cross section for ten evenly spaced incremental depths of flow in the main channel and for ten increments of depths above the overbank elevation, Z_{ob}. Then 2) coefficients of the relations are computed by a least squares regression. To maintain continuity in the backwater computation, overbank relations for area and conveyance are forced to have the same value as the main channel relations at the overbank elevation.

Newton-Raphson Solution for the Total Head Equation

The model solves the equation of spatially varied steady flow by using an analytical first order Newton-Raphson (N-R) method to give successive approximations in the standard step calculation. Combining Equations 3-~~17~~9, 3-32, 3-33 and 3-34 yields the spatially varied flow equation as a sole function of D_2:

$$(1 - C_e) \frac{a_5}{a_3^{\,2}} \frac{Q_2^2}{2g} D_2^{(b_5 - 2b_3)} + D_2 - \frac{4\Delta X \, Q_2^2}{(K_1^{\,2} + 2K_1 \, a_4 \, D_2^{\,b_4} + a_4^{\,2} \, D_2^{\,2b_4})}$$

$$- \frac{\Delta X}{2} \frac{Q_1^2}{K_1^2} + C_e \frac{\alpha_1 V_1^2}{2g} + Z_2 - H_1 = 0 \tag{3-40}$$

From the Taylor series expansion of an arbitrary function $F(\psi)$, the first order N-R approximation to $F(\psi)$ can be obtained by:

$$\psi^{*} = \psi_0 - \frac{F(\psi_0)}{F'(\psi_0)} \tag{3-41}$$

where ψ^{*} is the root of $F(\psi)$ and ψ_0 is the estimate of ψ^{*}. When applying Equation 3-41 to the solution of the backwater equation ψ is the depth of flow, D_2, at the upstream cross section, and $F(\psi)$ is the total head equation evaluated at that location as computed by Equation 3-40. For the N-R solution the first derivative of the total head equation must be obtained.

Differentiating Equation 3~~A~~-40 with respect to D_2 yields:

$$(1 - C_e)\,\frac{Q_2^2}{2g}\,\left(\frac{a_5}{a_3^2}\right)(b_5 - 2b_3)\,D_2^{(b_5 - 2b_3 - 1)} + 1 + \Delta X\, b_4\,\frac{Q_2^2}{a^2}\,D_2^{-(2b_4+1)} \tag{3-42}$$

Equation 3-42 is $F'(\psi)$ in the first order N-R. When Equation 3-40 is evaluated at depths other than its roots, the equation is not equal to zero, but is instead equal to the error in the total head. Figure 3.7 is a qualitative plot of Equation 3-40. The shape of the curve can be verified by consideration of Equation 3-19. As the depth approaches zero the area of flow goes to zero and the velocity goes to infinity. With a large velocity the velocity head term dominates the equation and causes a large positive error. Conversely, as the depth becomes large the depth itself dominates, and again the error becomes large and positive. At critical depth the specific head is at a minimum and the error is negative.

Care must be taken in using Equation 3-41. If the total head equation is not exactly convex, it takes many interactions to converge

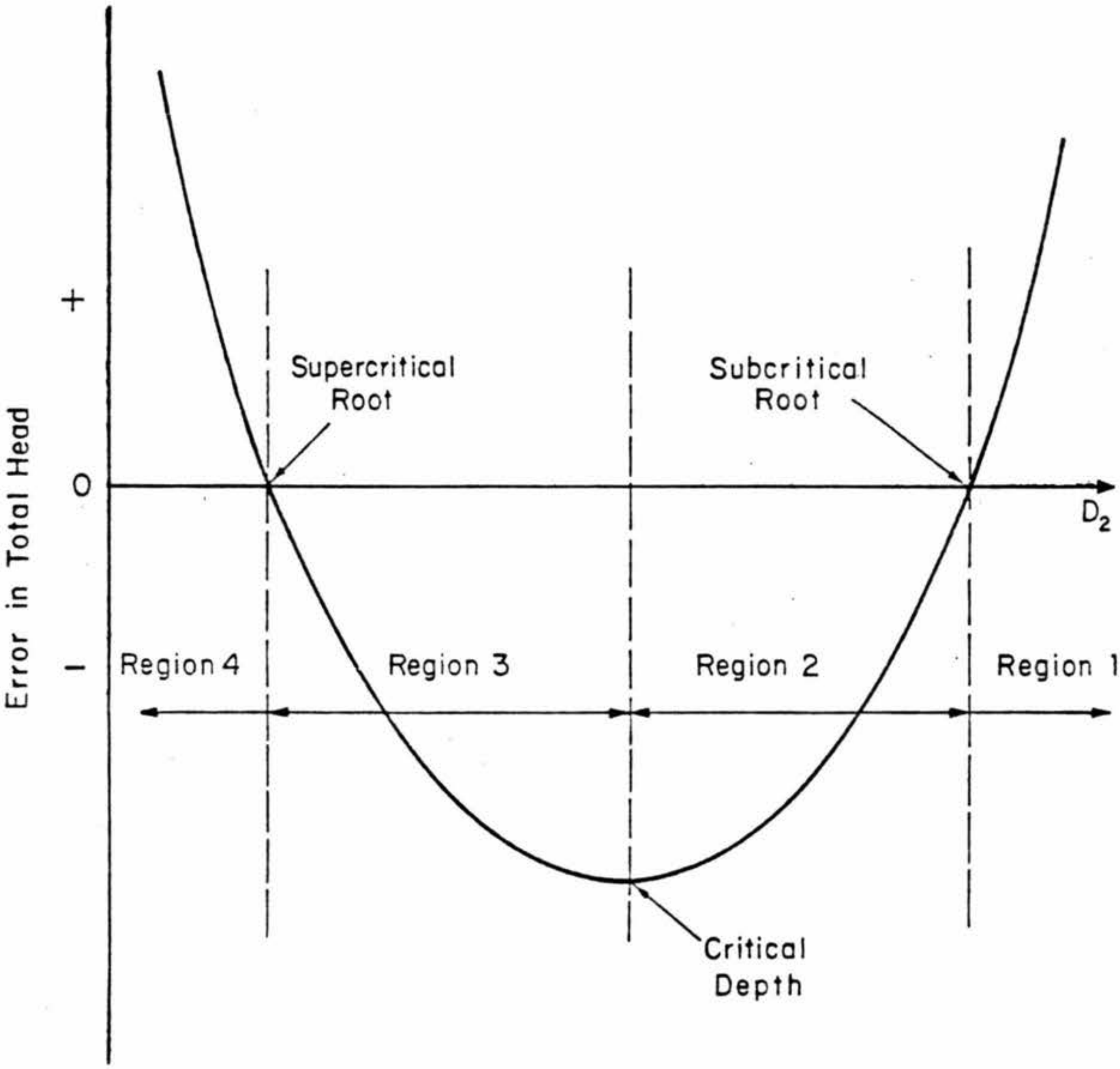

Figure 3.7 Error in total head equation as a function of depth of flow.

to a root. Also, if the initial guess is on the supercritical side of the curve, the N-R method drives the solution to the supercritical root, even if the flow is subcritical. By using simple logic the convergence of the N-R method has been greatly improved.

The curve in Figure 3.7 is broken into four different regions according to the signs of the error and first derivative. These regions are:

1. error positive, first derivative positive,
2. error negative, first derivative positive,
3. error negative, first derivative negative, and
4. error positive, first derivative negative.

The computer program's simple logic statements can detect in which region the estimate of D_2 is.

In most applications the subcritical root is desired, and logic in the computer program insures it will be found. If the estimate is in region 1 or 2 the N-R method finds the subcritical root, but if it is in regions 3 or 4, the N-R method is forced to the subcritical root. In region 3, simply taking the negative of the first derivative forces the solution to the subcritical root. In region 4, a new estimate is determined by computing critical depth and adding a constant to insure that the new estimate of D_2 is in region 1 or 2.

This approach differs from the Newton-Raphson outlined by Henderson (1966). The use of the hydraulic property relationships in this method improves the next estimate since it incorporates the change in the hydraulic properties with depth. Henderson's method uses only the conveyance and area at the estimated depth in computing the next guess.

Divided Flow

In cases where divided flow occurs, the program has the ability to determine the percentage of the total flow going down each of the divided flow reaches. Using an initial guess of the percenage of total flow in each reach the program calculates a water surface profile up each reach. If the upstream water surface elevations match within the defined backwater accuracy tolerance the calculations stop. If not the program uses a numerical second order curve fitting routine developed by Li (1972) to determine new estimates of the flow percentage. The program then repeats itself until the upstream stream water surface converges.

Weir Flow

Two flow conditions can occur at a single weir. If the weir height is small compared to the depth of flow the weir has no significant effect and the water surface profile is computed with the standard backwater curve. If the weir is not submerged the depth at the weir is computed by a broad crested weir formula

$$D = \left(\frac{Q}{W\, I_1}\right)^{I_2} \tag{3-43}$$

where I_1 and I_2 are constants depending on the shape and surface of the weir.

In the program the depth at the weir is computed by both methods and the greater of the two is used.

Sediment Routing

General

Once the backwater profile is determined for a given time period, sediment is routed through the system. Sediment routing is accomplished in three separate steps. The first step is calculation of the sediment transport at each cross section in the river, which requires knowledge of the velocity, depth, and width of flow obtained by the backwater calculations. The second step is routing of the sediment to determine change in cross section area due to sediment movement. The third step is distribution of the change in area through the cross section to obtain a new channel geometry.

Sediment Transport

In the mathematical simulation of stream bed aggradation-degradation, conventional algorithms for calculating sediment transport require large amounts of computer time. This excessive use of computer time makes these methods impractical. It is also difficult to calibrate conventional methods for observed data. Therefore, empirical relationships are often used in mathematical modeling. In the Sedimentation Study of the Yazoo River Basin (Simons, et al., 1978) a relationship of the following form was used

$$Q_s = 4.48 \times 10^{-6} V^{3.16} D_e^{0.94} W_e \tag{3-44}$$

where Q_s is the bed material sediment transport in cfs. Although this equation is not applicable to other rivers, it fits data for the Yazoo River Basin. The coefficients in Equation 3-44 were determined by: 1) taking suspended sediment measurements, 2) applying the Modified Einstein procedure to obtain the bed material load, and 3) using standard least squares regression to obtain the coefficients. Data does not always fit theory, as the exponent for D_e should be negative.

It is important to note that Equation 3-44 is only for the bed material load. In most cases the wash load is supply limited and all wash load entering a system will pass through it. An exception to this is when sedimentation behind a large dam is of interest. In this case, calculation of the wash load is required. It is also interesting to note in Equation 3-44 that the bed material size does not enter into sediment calculations. This is due to the limited range of data from which the equation was derived. In cases when there is insignificant data to develop relationships such as Equation 3-44, Colby's and Meyer-Peter, Müeller's methods have been used with success at minimal cost in computer time.

Sediment Routing

Channel aggradation-degradation is determined by solving the sediment continuity equation:

$$\frac{\partial Q_s}{\partial x} + (1 - \rho)\,\frac{\partial A_b}{\partial t} = q_{s\ell} \tag{3-45}$$

where A_b is the cross-sectional area of the bed, $q_{s\ell}$ is the lateral tributary sediment inflow and ρ is the porosity that is the volume of voids per unit volume of sediment in place. The first term in Equation 3-45 represents the change in sediment transport along the river, while the second term represents the change in bed area with time. A negative value of the second term signifies degradation while a positive value signifies aggradation.

Interior Sediment Routing. Figure 3.8 shows a typical interior sediment routing condition. Equation 3-45 is solved by a two step, finite difference algorithm. The first step is calculation of change in sediment volume between cross sections. The change in volume is computed by

$$\Delta V_i = (Q_{s_{i+1}} - Q_{s_i} + q_{s\ell_i})dt \tag{3-46}$$

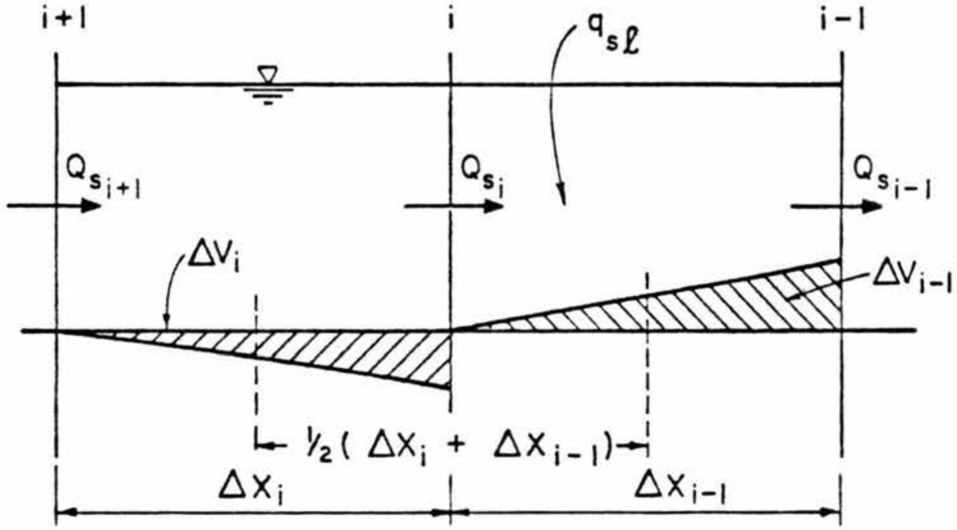

Figure 3.8 Finite difference sediment routing scheme.

where ΔV_i is the change in sediment volume between sections i and i+1. The second step in the sediment routing is determination of change in area at each cross section, that requires knowledge of the location of sediment erosion or deposition between cross sections. Modeling of sediment dispersion is required to compute exactly where in the reach between sections the sediment is eroded or deposited. Unfortunately, modeling of sediment dispersion requires a non-potential transport equation, excessive amounts of computer time, and considerable effort to calibrate. Therefore, an empirical distribution is used. A triangular distribution weighted downstream as shown in Figure 3.8, is used. One-quarter of the volume is deposited or eroded in the upstream half of the segment between sections, while three-quarters of the volume is deposited or eroded in the downstream half. This simply places more weight of sediment transport rate at the downstream section for determining the degradation and aggradation. With this assumption the change in bed area at a section is equal to

$$\Delta A_{b_i} = \frac{1}{(1 - \rho)} \frac{\frac{1}{4} \Delta V_{i-1} + \frac{3}{4} \Delta V_i}{\frac{1}{2} (\Delta X_i + \Delta X_{i-1})} \tag{3-47}$$

where ΔA_{b_i} is the change in bed area at the cross section. The physical significance of triangular distribution is seen if Equation 3-46 (neglecting lateral sediment inflow) is substituted into Equation 3-47:

$$\Delta A_{b_i} = \frac{1}{(1 - \rho)} \frac{\frac{1}{4}(Q_{s_i} - Q_{s_{i-1}}) + \frac{3}{4} (Q_{s_{i+1}} - Q_{s_i})}{\frac{1}{2} (\Delta X_{i-1} + \Delta X_i)} \, dt \tag{3-48}$$

$$\Delta A_{b_i} = \frac{1}{(1 - \rho)} \frac{(\frac{3}{2} Q_{s_{i+1}} - Q_{s_i} - \frac{1}{2} Q_{s_{i-1}})}{\Delta X_{i-1} + \Delta X_i} \, dt \tag{3-49}$$

As expected, the coefficient for sediment inflow, $Q_{s_{i+1}}$ is positive and the coefficient for the outflow $Q_{s_{i-1}}$ is negative. An important fact to note is that the multiplying factor for the sediment transport at the cross section is negative. This is physically logical. If the upstream and downstream transport is held constant, a reduction in the sediment transport at the section causes the section to aggrade while an increase in the transport causes it to degrade. If a triangular distribution weighted upstream were used the coefficient for sediment transport at the section would be positive. Thus as sediment transport at the section increased the section would aggrade. If a rectangular distribution were used the coefficient would be zero, and sediment transport at a section would have no effect on aggradation or deposition at the section.

Boundary Sediment Routing. At the upstream and downstream boundary cross sections Equation 3-47 cannot be used to compute the change in area at the section. This is usually not a problem since the upstream boundaries are always fixed, i.e. unchanged with time, and downstream boundaries are usually fixed to maintain numerical stability in the model. But in cases when a major tributary flows into the mainstem, the tributary's downstream cross section can be allowed to "float". Change in bed area for a downstream tributary section is computed by:

$$\Delta A_{bi} = \frac{1}{(1 - \rho)} \frac{\Delta V_i}{\frac{1}{2}\Delta X_i} \tag{3-50}$$

Distribution of Erosion and Deposition Across the Cross Section

Once the change in area at a cross section is computed the area must be distributed across the section to determine the new channel geometry. With a one dimensional model the exact location of scour or deposition can not be determined since the program does not compute the lateral flow effects. Therefore, empirical procedures are used to distribute the bed area change. A method that relates the change in bed elevation at a point to the hydraulic property of conveyance is used in the model. This method as shown in Figure 3.9 is considered appropriate because conveyance is directly related to velocity and thus sediment transport.

A qualitative analysis was performed to test the validity of the sediment distribution scheme based on conveyance (which is directly related to depth if Manning's roughness is the same across the whole cross section). Three cross sections were taken from the Greenwood Bendway of the Yazoo River. At several points in each cross section the percent of maximum depth of flow, and the percent of maximum change in bed elevation (from February 2 to February 18, 1977) were determined. The results are plotted on Figure 3.10. As one can see, change in bed elevation is roughly proportional to depth. The sediment distribution method based on the hydraulic properties at each point in the cross section more accurately represents the natural cross-sectional changes.

The model computes change in elevation for each cross section point by:

$$\Delta Z_j = \frac{k_\ell + k_{\ell+1}}{K_i} \quad \frac{\Delta A_{b_i}}{x_{j+1} - x_{j-1}} \tag{3-51}$$

where ΔZ_j is the change in elevation for point (j), k_ℓ and $k_{\ell+1}$ are the conveyance of the incremental areas to the right and left of the point, and x_{j+1} and x_{j-1} are the horizontal coordinates of the

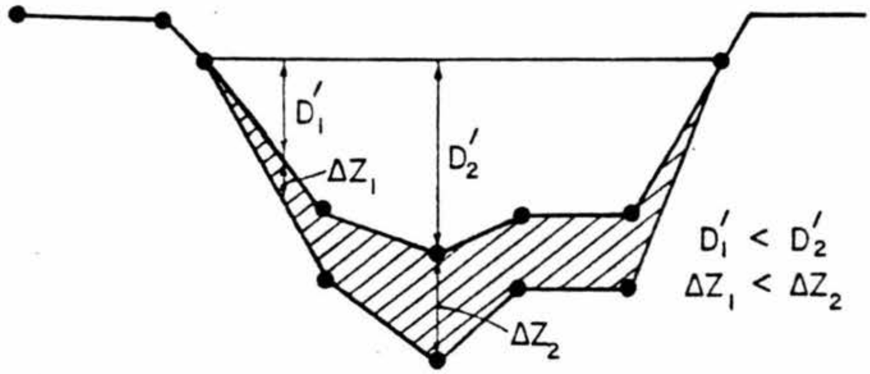

Figure 3.9 Sediment distribution based on depth of flow.

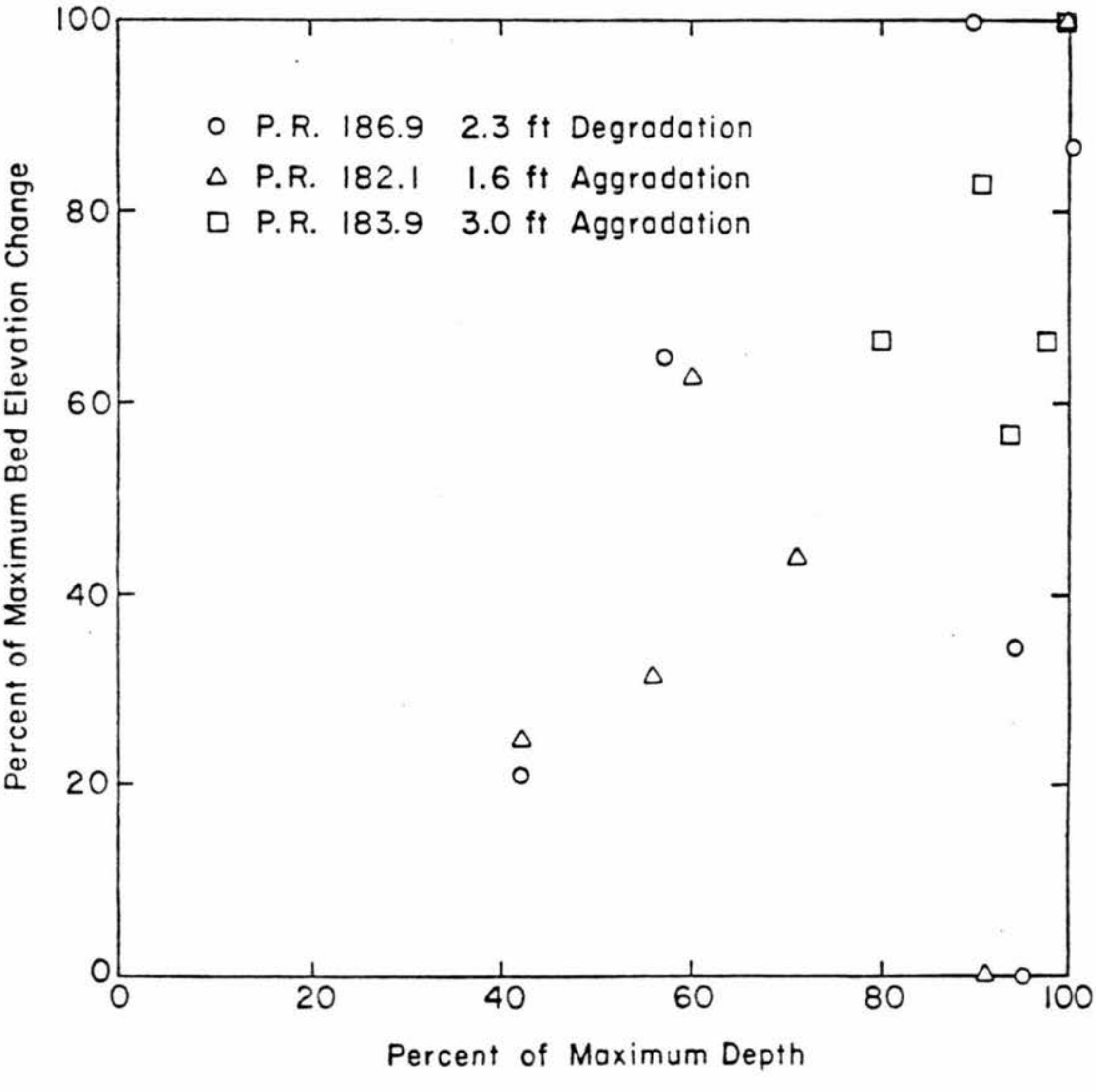

Figure 3.10 Percent bed elevation change compared to percent of maximum.

cross section points adjacent to (j) and K_i and ΔA_{b_i} are the total conveyance and bed area change at the ith cross section.

To save computer time sediment is not distributed at a cross section until a significant change in cross section area has occurred. This threshold can be determined according to the physical environment and the objective of study.

Weir Sediment

The sediment transport over a weir is assumed to be a percentage of the upstream sediment transport. The concept used in determining the percentage is shown in Figure 3-11. A suspended bed material curve at the cross section directly above the weir is shown. The material in the shaded portion of the curve is assumed to pass over the weir. The percentage is computed using the Lane-Kalinske's relationship (Simons and Sentürk, 1977) for sediment concentration:

$$C_y = C_a \exp^{\frac{6w}{\kappa U_*} \frac{y-a}{D}} \tag{3-52}$$

where C_y is the concentration at an arbitrary depth y, C_a is a reference concentration at a depth a, κ is the von Karman constant, w is the particle fall velocity, and U_* is the shear velocity.

By assuming a depth a which is close to the bed and much smaller than y, Equation 3-52 reduces to

$$C_y = C_a \exp^{\frac{-6w}{\kappa U_*} \frac{y}{D}} \tag{3-53}$$

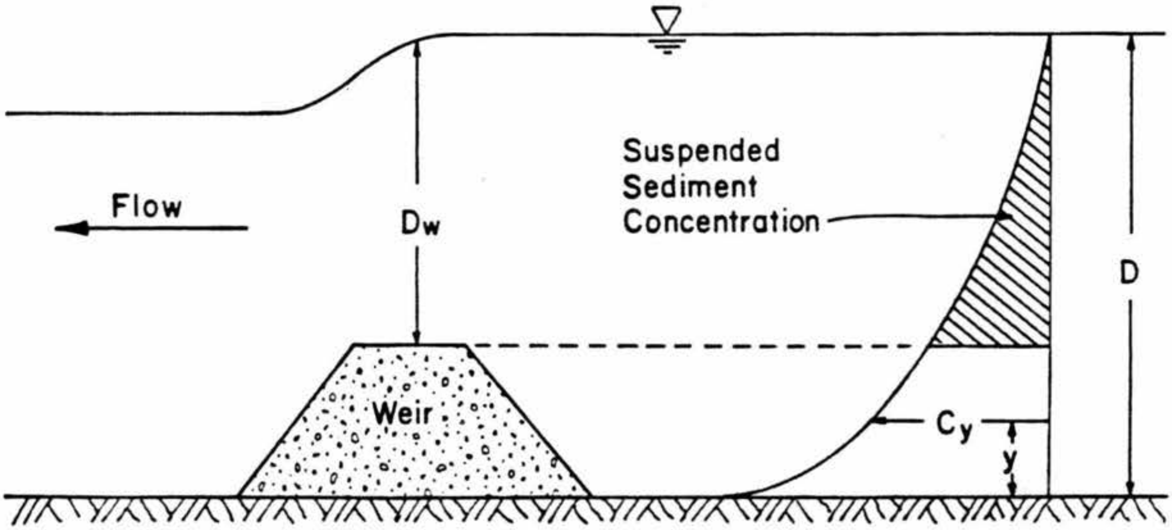

Figure 3-11 Sediment transport over weirs.

Integrating Equation 3-53 with respect to y yields

$$\int C_y d_y = C_a \frac{\exp \frac{-6w}{\kappa U_*} \frac{y}{D}}{\frac{-6w}{\kappa U_* D}} \qquad (3\text{-}54)$$

Evaluating Equation 3-54 for the shaded portion of the suspended sediment curve results in the percentage of upstream sediment transport P_c, which passes over the weir:

$$P_c = \frac{\int_{(D-D_w)}^{D} C_y d_y}{\int_{0}^{D} C_y d_y} \qquad (3\text{-}55)$$

where D_w is the depth of flow at the weir. The reference concentration C_a cancels from the equation.

The shear velocity is computed using the Darcy-Weisbach resistance formula:

$$U_* = \frac{V}{\sqrt{8/f}} \qquad (3\text{-}56)$$

The von Karman constant has a value much higher than in uniform flow, since the weir causes a large vertical turbulence. In the Sedimentation Study of the Yazoo River Basin, κ was set equal to 0.70.

Chapter 4

MODEL APPLICATIONS

General

This section presents applications of the model KUWASER. All of the applications described here were performed as part of the Sedimentation Study of the Yazoo River Basin (Simons et al., 1978). The following sections present examples of model calibration on two different river reaches, a comparison of model performance (as measured by stage prediction) between KUWASER, a stage discharge relation, and a fix bed model and the results of two of the actual Yazoo design alternative runs. The results are presented to demonstrate the wide range of information that can be determined through use of KUWASER.

Calibration

Description of Calibration Reaches

Two reaches of the Yazoo River were calibrated in the Sedimentation Study of the Yazoo River Basin. The reaches were the Ft. Pemberton cutoff at Greenwood Bendway and a reach on the Tallahatchie River from Swan Lake to upstream of Locopolis. These two reaches and the cross section used in the model are shown in Figure 4.1 and shown schematically in Figures 4.2 and 4.3.

These two reaches are quite different since the Swan Lake-Locopolis reach is stable and experiences little bed profile change. The Ft. Pemberton reach, however, is unstable and experienced a 20-foot degradation during 1973 and 1974.

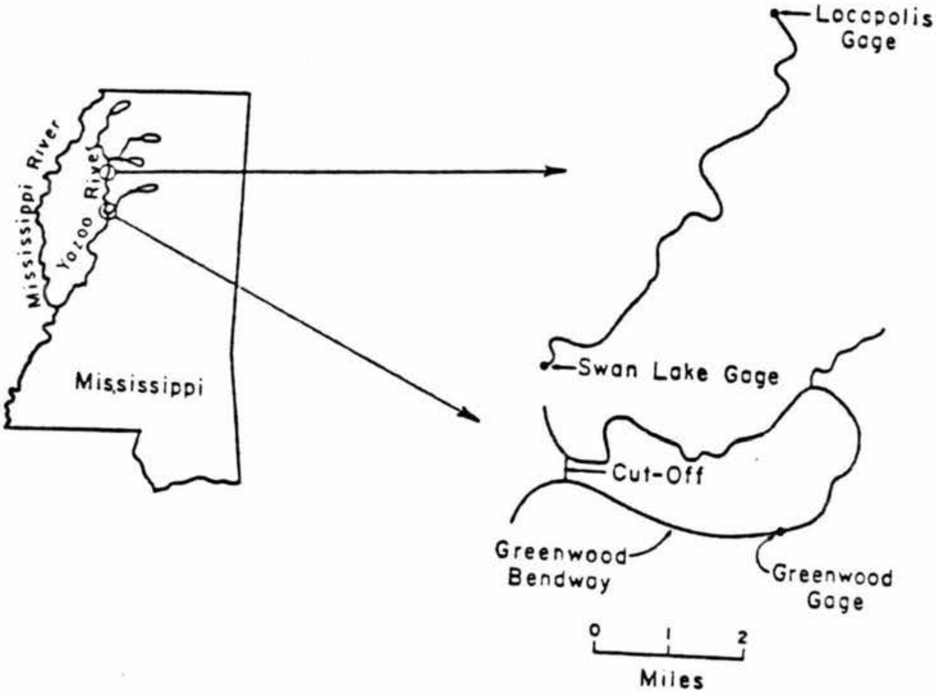

Figure 4.1. Location of calibration river reaches.

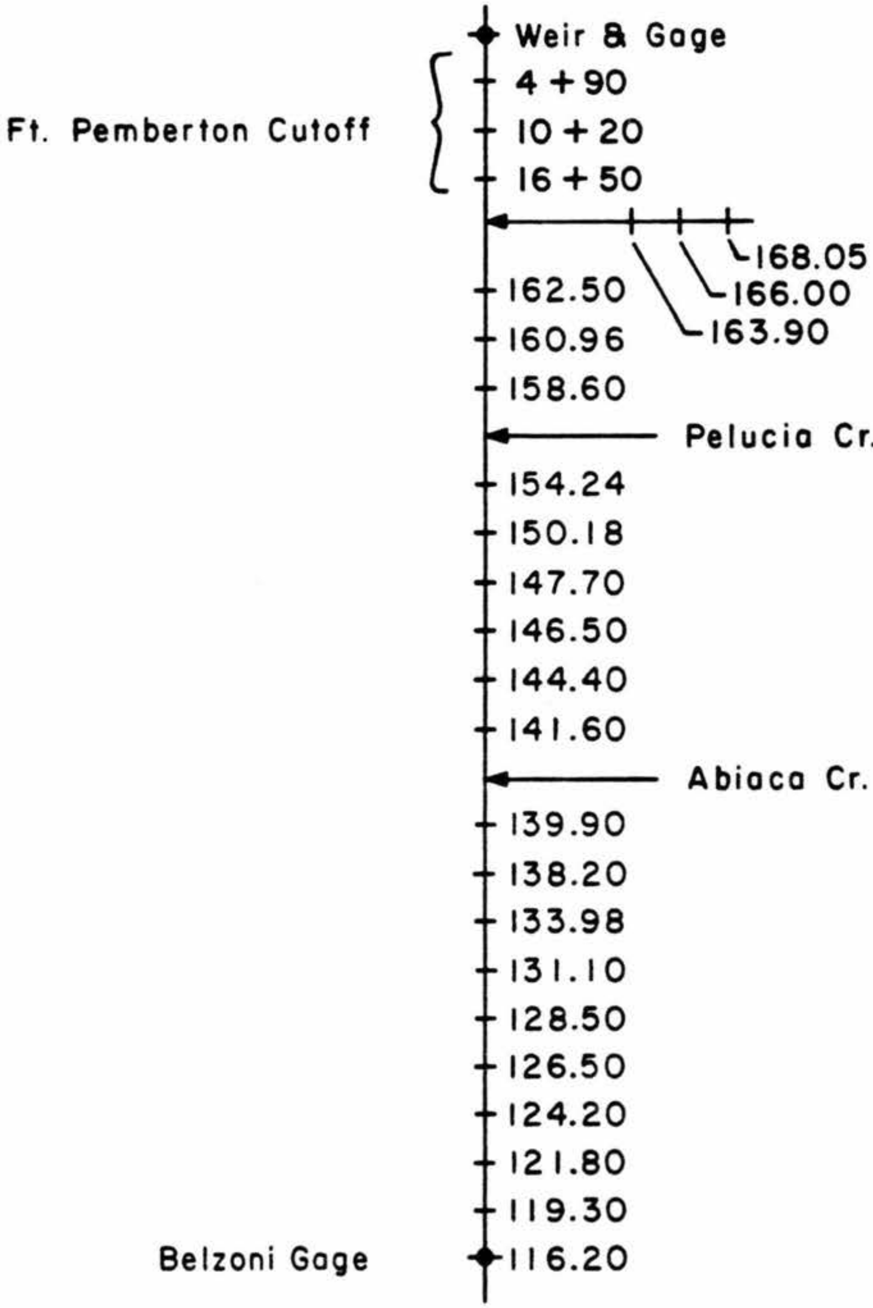

Figure 4.2. Ft. Pemberton calibration reach.

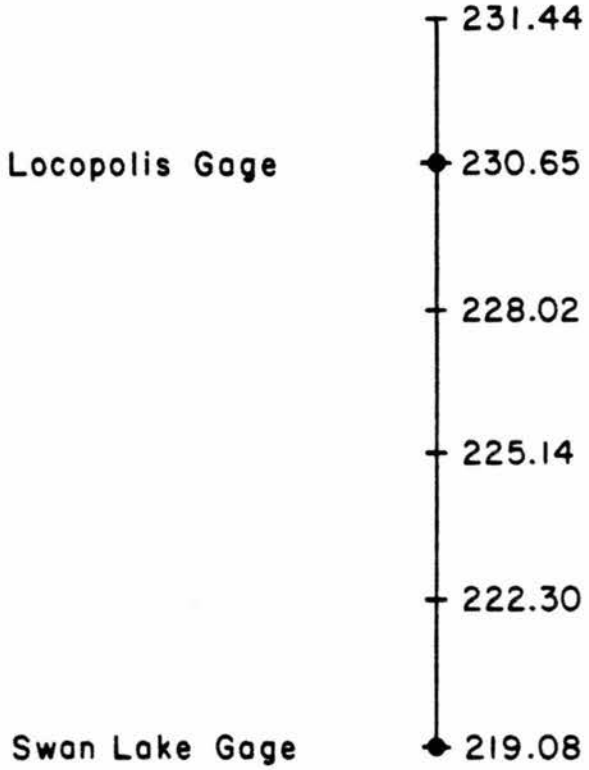

Figure 4.3. Swan Lake-Locopolis calibration reach.

For the Ft. Pemberton reach, backwater computations were carried upstream, starting with a known stage-discharge relationship at the Belzoni gage continuing to the Ft. Pemberton gage. It was also necessary to carry computations a short distance up the Greenwood Bendway to obtain the correct sediment input from the bendway. For the Swan Lake-Locopolis reach, backwater computations were carried upstream starting with a known stage-discharge relationship at the Swan Lake gage to upstream of Locopolis gage (river mile 231.44).

Procedure

Fort Pemberton. In the cutoff there were two sets of measurements. The model was calibrated to reproduce these values. These measurements were the stage at the Ft. Pemberton gage (Yazoo-Tallahatchie) and the bed elevation at four sections directly downstream of the weir in the cutoff.

Bed elevation was calibrated by adjusting the sediment transport over the weir at the upstream section, and the water surface elevation was calibrated by adjusting Manning's n. Since the two quantities are not independent, an iterative method was used where only one of the quantities was adjusted at a time.

First, Manning's n was calibrated by multiplying the estimated n value for the main channel and overbanks by a constant. Then, the model was run with the flows from April 12, 1973 to February 23, 1974 (318 days), and the error between the observed and computed water surfaces was minimized until no further reduction in error could be obtained. During these runs the sediment transport was set to zero, therefore, the bed was assumed fixed.

The model was then run for the same time period with the sediment transport in the river set to its normal value. With the initial bed profile of April 12, 1973, the sediment transport over the weir, which was computed with the same relationship as the river's, was adjusted by changing a constant until the computed bed at the end of the time period was matched as closely as possible with the observed value. Manning's n was then readjusted.

Results of the calibration suggested the Manning's n values of 0.030 for the main channel and 0.150 for the overbank flows. With these values of n, the average error between observed and computed stages at Ft. Pemberton for the calibration period was 0.87 foot with a maximum error of 3.72 feet. Figure 4.4 shows a plot of the observed and computed stage hydrographs for the calibration. Figure 4.5 shows a plot of observed and computed bed elevations at the four cross sections downstream of the weir. As indicated, the computed bed profile for February 23, 1974 closely matches the observed bed profile. Even through this was only calibration, the model was able to simulate the large degradation at Permanent Range (P.R.) 162.50, Station 16+50 and Station 4+90, and the hump that was formed at Station 10+20 by adjusting only one parameter describing sediment input to the cutoff. Sediment input to the cutoff should be modifid because of the presence of the weir. In addition, simulated changes in cross-sectional shapes are compared with measured changes (Figures 4.6 to 4.9). The model simulated the changes in the cross sections adequately.

Swan Lake-Locopolis. In the Swan Lake-Locopolis reach there was not enough cross-sectional data to calibrate sediment transport, therefore, only Manning's n was calibrated and sediment transport was

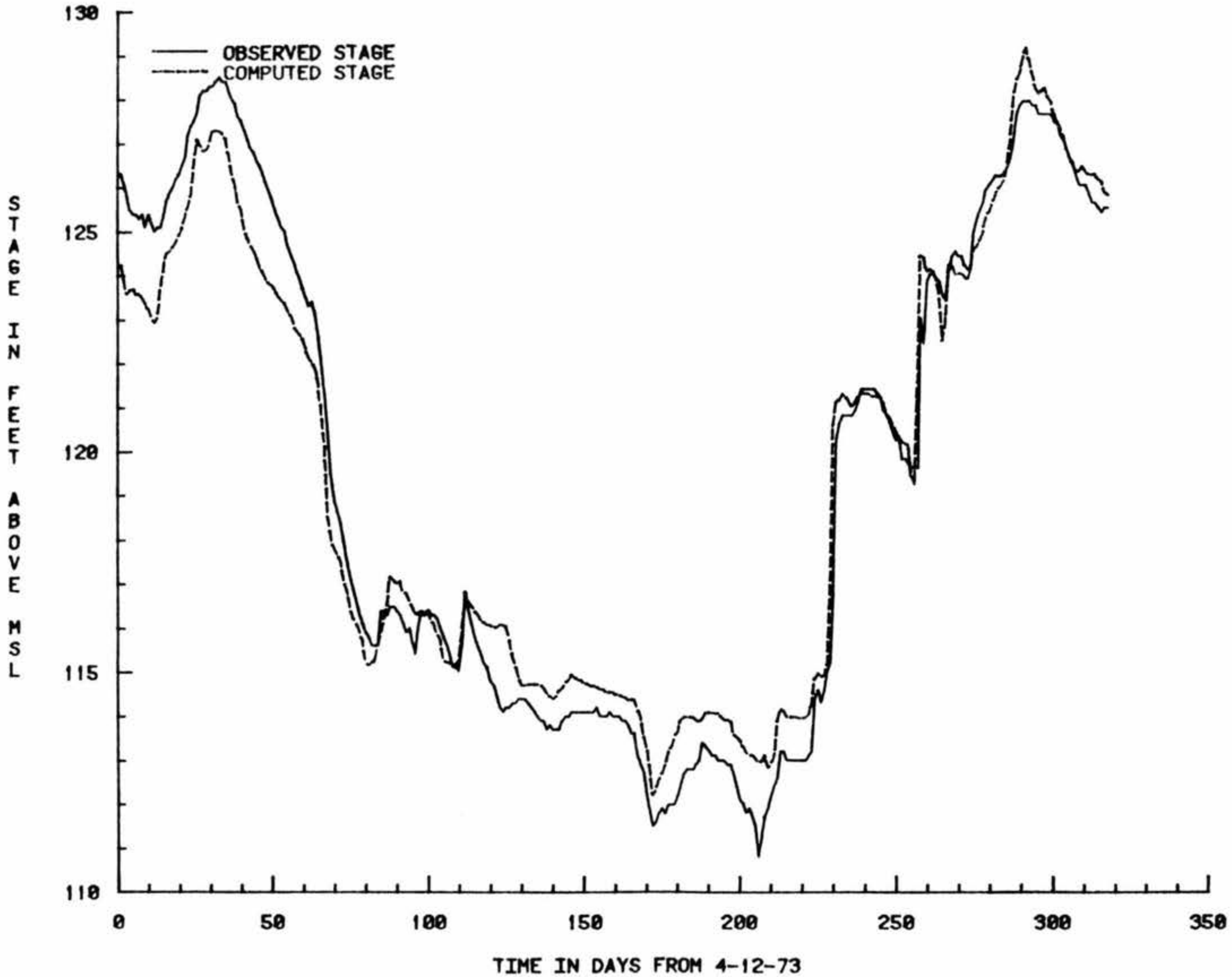

Figure 4.4. KUWASER calibration at Ft. Pemberton.

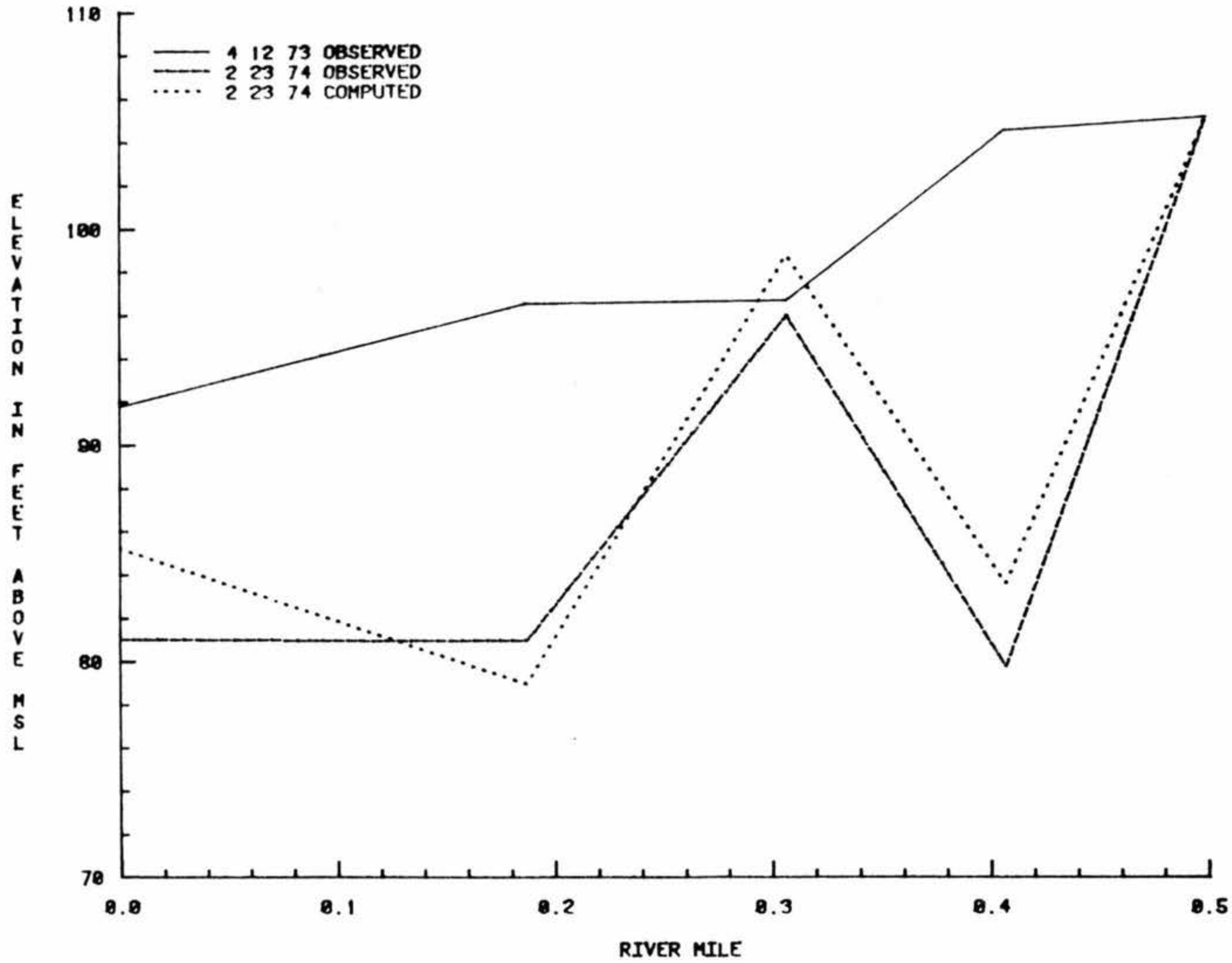

Figure 4.5. KUWASER calibration of bed elevation at Ft. Pemberton.

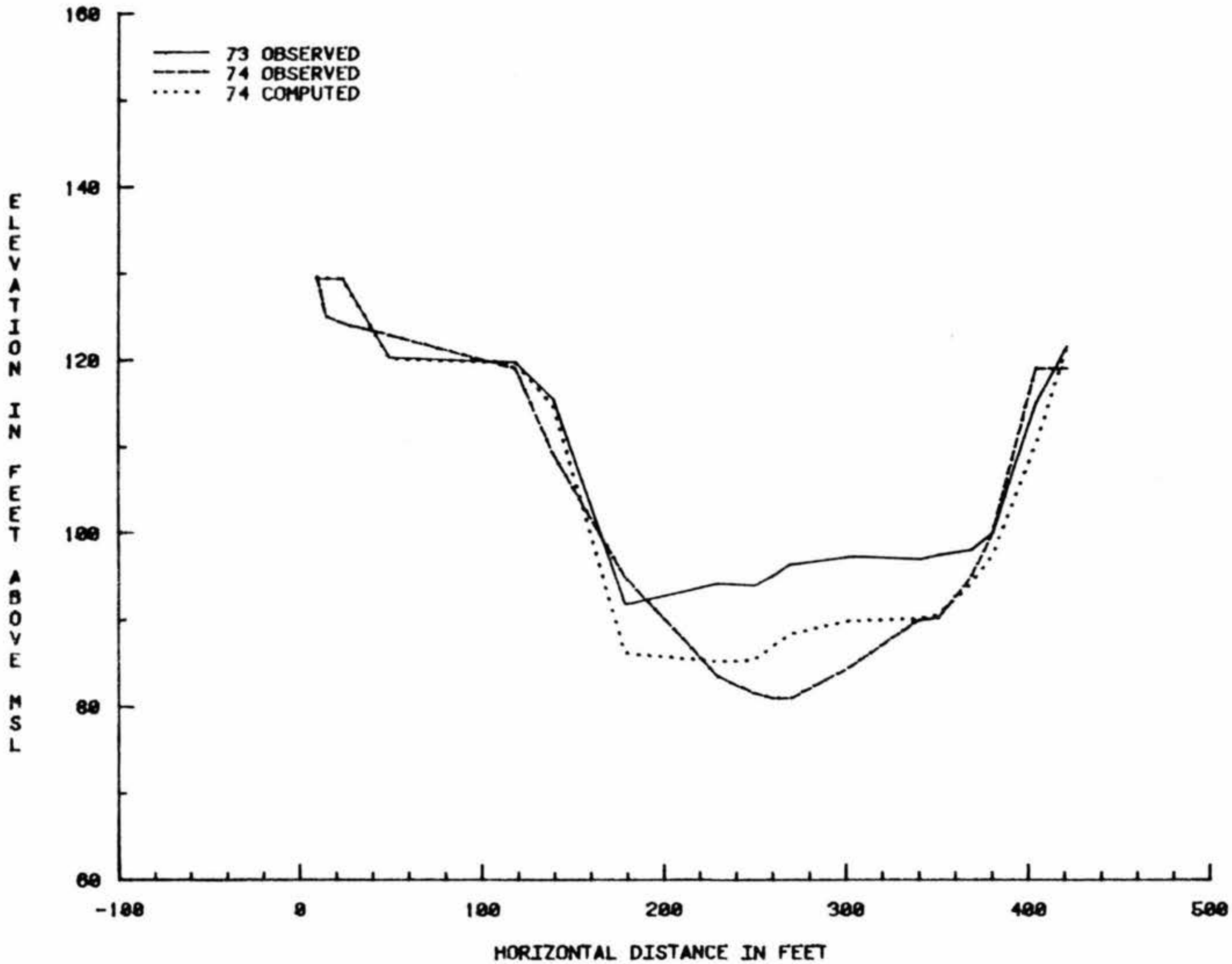

Figure 4.6. Comparison between computed and measured cross sections at Ft. Pemberton using KUWASER (P.R. 162.5).

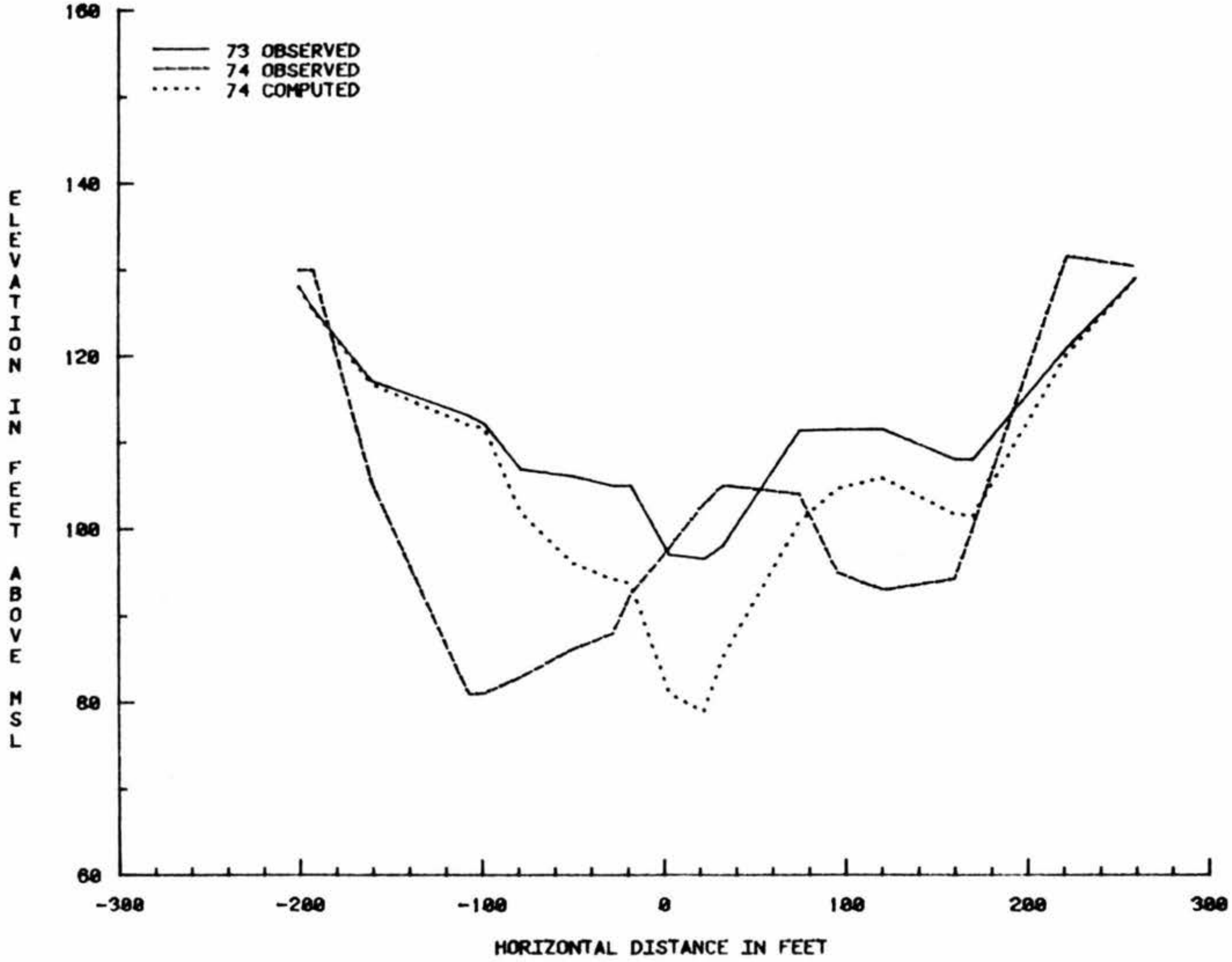

Figure 4.7. Comparison between computed and measured cross sections at Ft. Pemberton using KUWASER (Station 16+50).

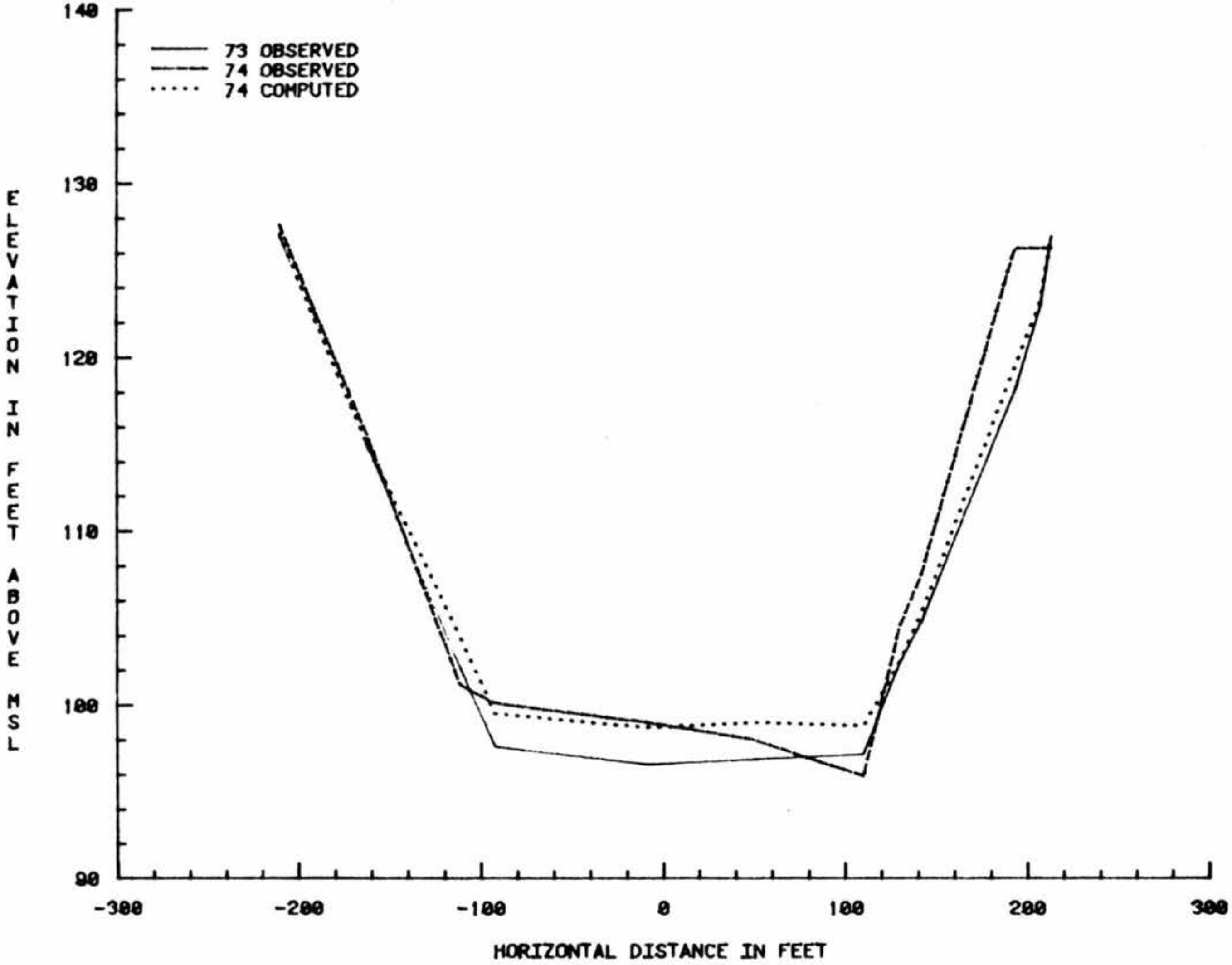

Figure 4.8. Comparison between computed and measured cross sections at Ft. Pemberton using KUWASER (Station 10+20).

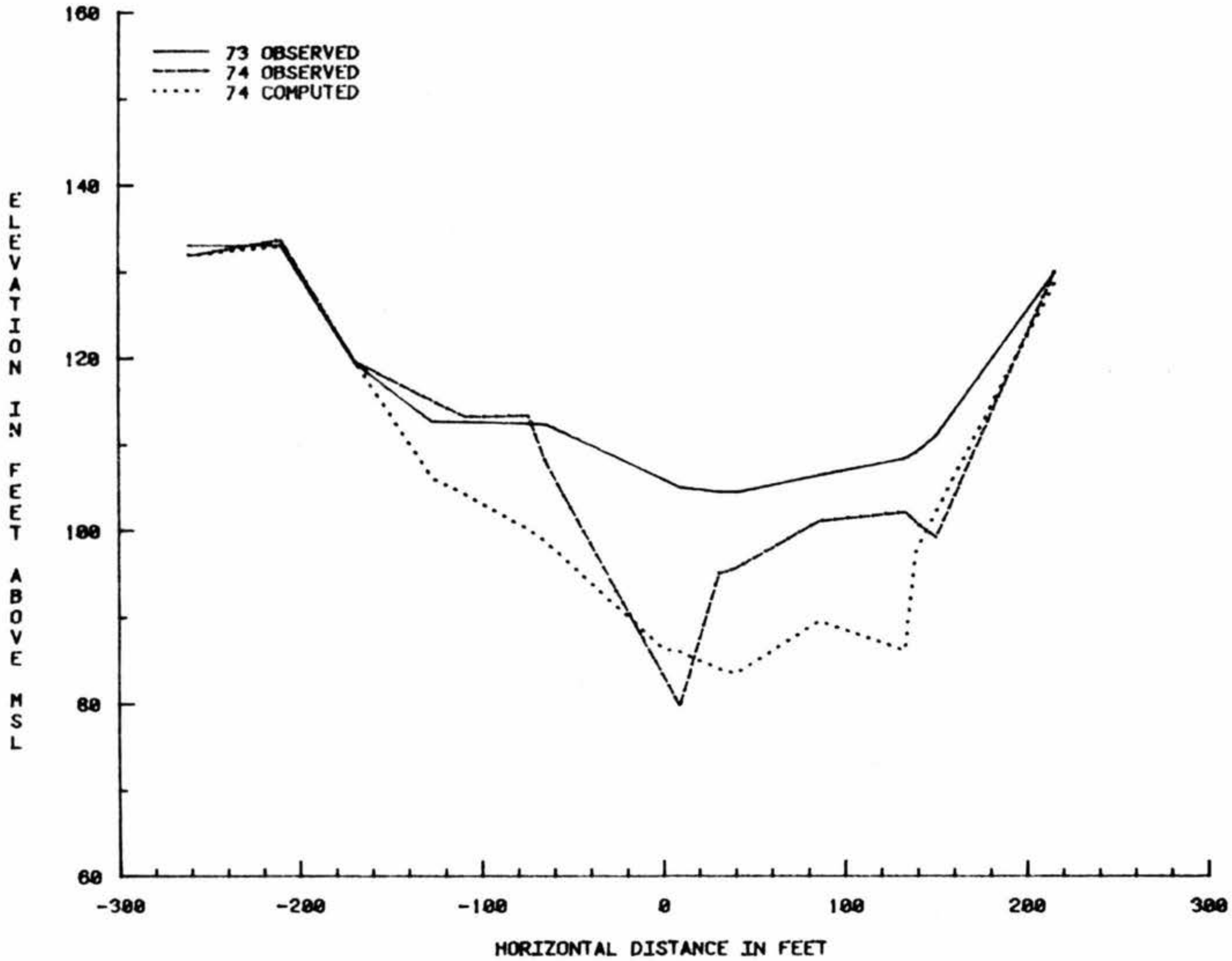

Figure 4.9. Comparison between computed and measured cross sections at Ft. Pemberton using KUWASER (Station 4+90).

assumed equal to its normal value. In this reach it was necessary to allow Manning's n to vary with discharge since there appears to be large differences in the headloss from Swan Lake to Locopolis for different flow levels. The period used for calibration was January 1, 1971 to December 31, 1972 (731 days). Results of the calibration are shown in Table 4.1. Manning's n varies as a power function of discharge. As indicated in the table, Manning's n is about 0.02 for the extreme high flow and approximately 0.042 for low flows. These values are very reasonable considering the hydraulics of the fluvial system. With these values of n, the average error between observed and computed stages at Locopolis for the calibration time period was 0.34 foot, with a maximum error of 2.12 feet. Figure 4.10 shows the observed and computed stage hydrographs for the calibration. As indicated, the error between the two is quite small.

Model Verification

To verify the applicability of the model, an additional run was made for each reach using the calibrated values of Manning's n and sediment transport equations. These runs were made for time periods immediately following calibration. At Ft. Pemberton the verification was from February 24, 1974 to December 31, 1974 (311 days). At Locopolis the period used was from January 1, 1973 to December 31, 1974 (730 days). At Ft. Pemberton the average error between the computed and measured stage for the verification period was 1.10 feet with a maximum error of 2.48 feet; and at Locopolis, the average was 0.74 foot with a maximum error of 2.21 feet. Figures 4.11 and 4.12 show plots of the measured and computed stages at Ft. Pemberton and Locopolis. In

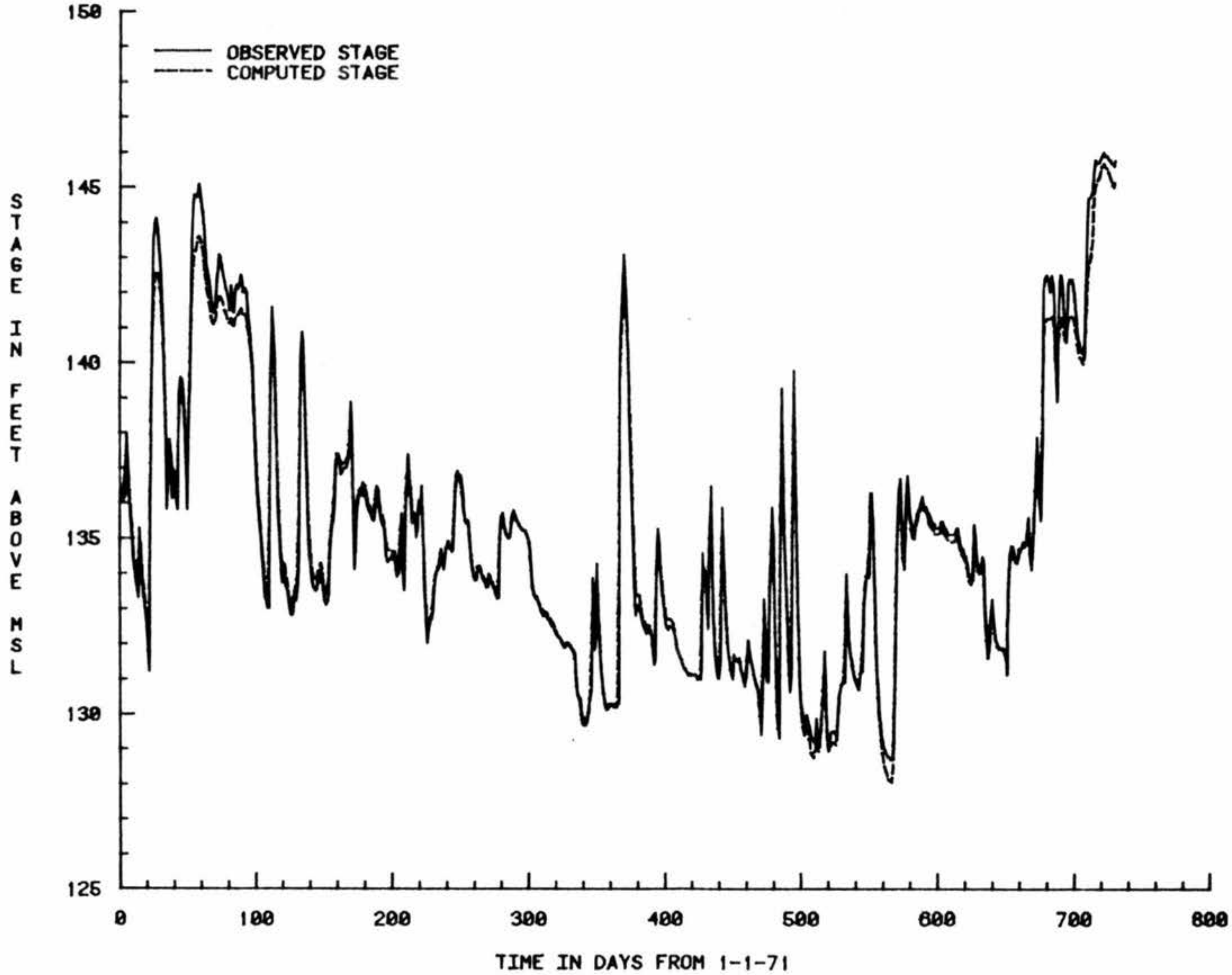

Figure 4.10. KUWASER calibration at Locopolis.

Table 4.1. Manning's n Calibration for Swan Lake

Discharge at Swan Lake	Manning's n	
	Main Channel	Overbank
2,000	.042	.210
10,000	.035	.175
35,000	.020	.100

addition, Figure 4.13 shows measured and computed bed elevations for June 1, 1974 between Swan Lake and Locopolis. As seen in Figure 4.14, the two bed profiles match adequately. The prediction is for the entire period January 1, 1971 to December 31, 1974. A summary of the calibration and verification results for KUWASER is given in Table 4.2.

Table 4.2. Calibration and Verification for KUWASER

Reach	Calibration Error (ft)		Verification Error (ft)	
	Mean	Max	Mean	Max
Fort Pemberton	0.88	3.60	0.45	1.78
Locopolis	0.34	2.12	0.74	2.21

Comparison with Other Models

General

In this section, frequency and magnitude of error associated with KUWASER and two other models are examined. The first comparison model is an empirical stage-discharge relationship or rating curve model. The second model is a steady, spatially varied flow rigid boundary model. The comparison of the three models provides more than a verification of the model developed here. The comparison also provides guidelines for application of each model

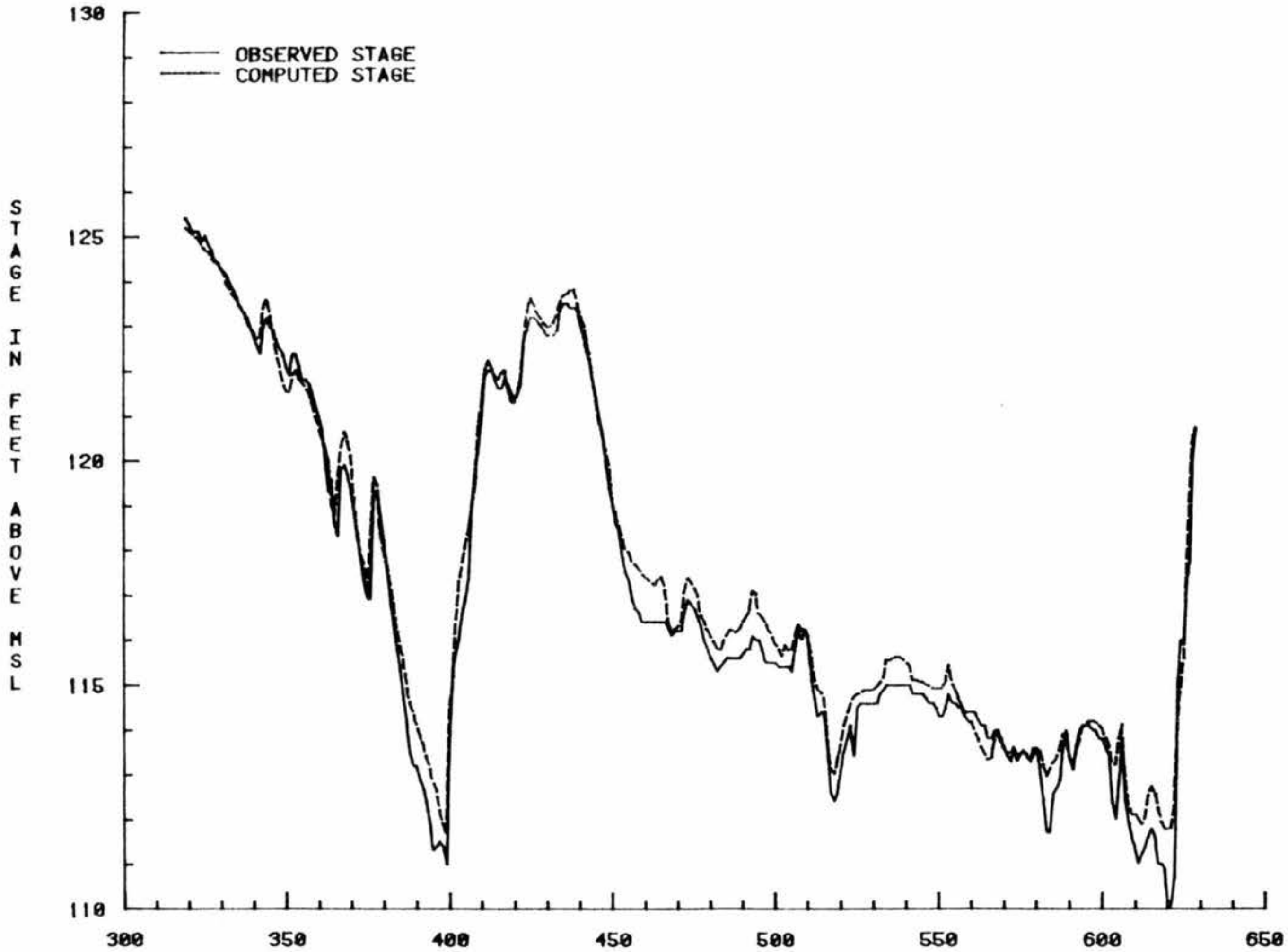

Figure 4.11. KUWASER verification at Ft. Pemberton.

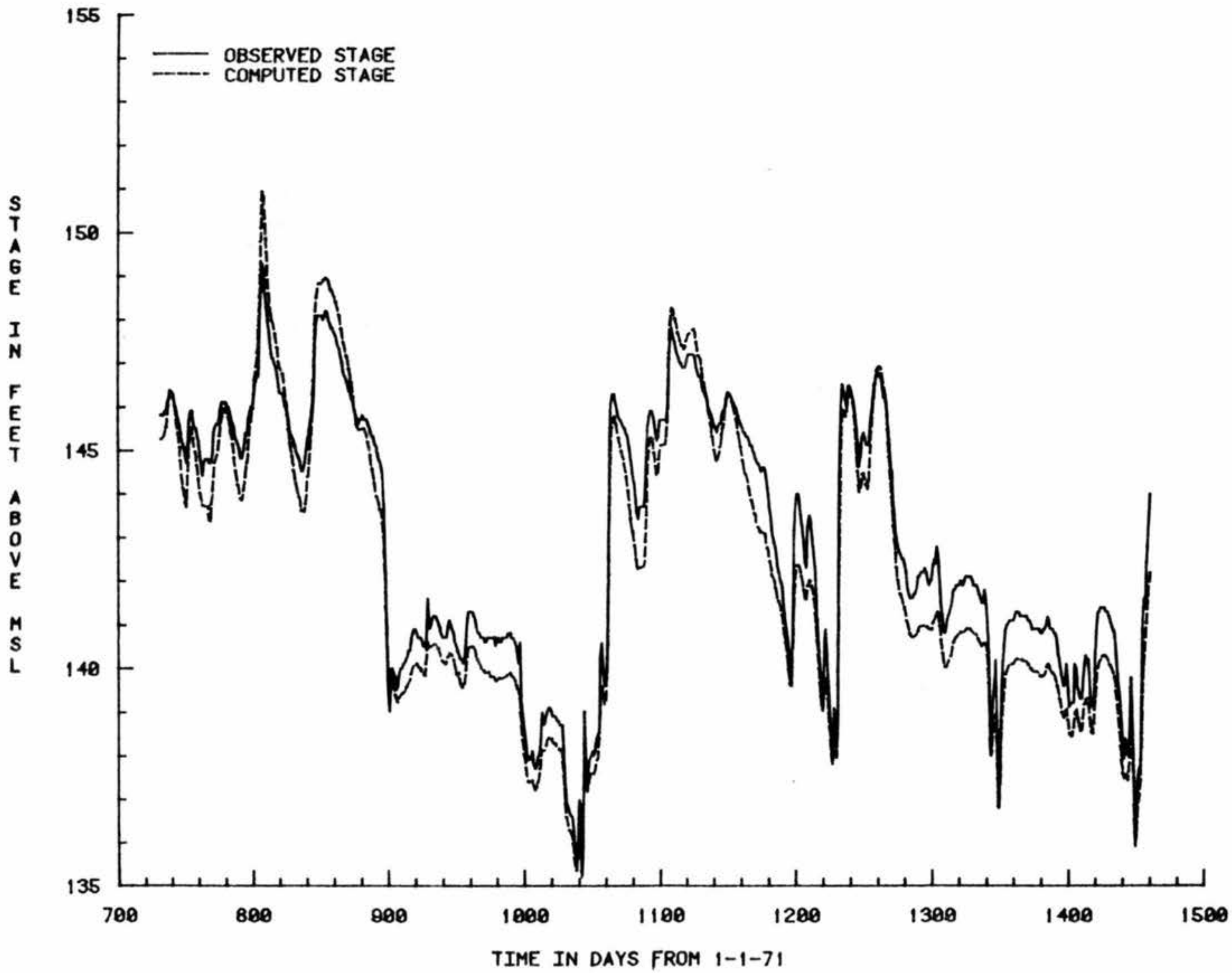

Figure 4.12. KUWASER verification at Locopolis.

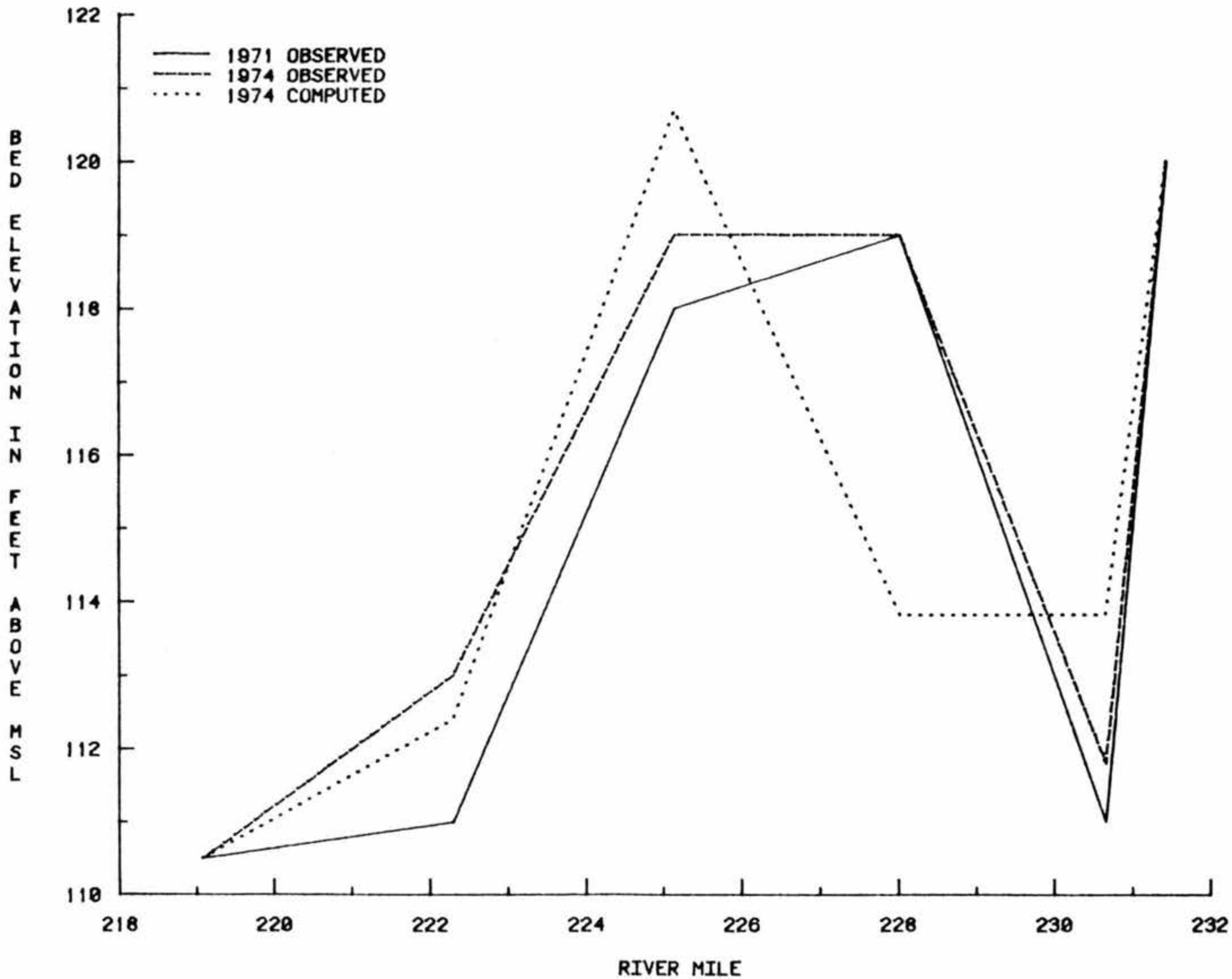

Figure 4.13. KUWASER verification of bed elevation in Swan Lake-Locopolis Reach.

Description of Comparison Models

Empirical Stage-Discharge Relationship Model. The relationship between stage and discharge is often called a rating curve. Generally, a rating curve is obtained by plotting the observed stage data against measured discharge. In reality, experience indicates that measurements of stage and discharge do not form a single valued relationship. Many rivers, especially those with a flatter gradient, display a hysteresis loop in the stage-discharge relationship due in part to dynamic effects and changing bed forms. Since these relationships can normally only be determined using water and sediment routing models, the following relation is commonly used to determine the stages at a given location along the river from the corresponding discharges:

$$Q = a_x(s-c)^{b_x} \tag{4.1}$$

where Q = discharge, s = stage, c = constant, usually gage zero, and a_x and b_x are time-variant coefficients. The unknown coefficients a_x and b_x are evaluated by least-squares regression techniques based on historical records.

Rigid Boundary Model. A known discharge, spatially varied flow model written by the author was used in the comparison. The model is identical in theory and operation to the model developed here except for the exclusion of sediment routing.

Calibration of Comparison Models

The comparison models were calibrated for the same locations and periods describe in the previous section. These are Ft. Pemberton from April 12, 1973 to February 23, 1974 (318 days) and Locopolis from January 1, 1971 to December 31, 1972 (731 days).

Empirical Stage-Discharge Relationship. Measured discharge and observed stage data from the two test reaches for the specified times were used to estimate the unknown parameters, a_x and b_x, that minimized the mean-square-error of the estimates. Regression constants and the coefficient of correlation, R_c, are as follows:

Ft. Pemberton: $a_x = 1783$, $b_x = 0.7538$ and $R_c = 0.901$

Locopolis: $a_x = 0.2872$, $b_x = 3.140$ and $R_c = 0.991$

The observed and computed stage hydrographs for the calibration periods are shown in Figures 4.14 and 4.15.

It is clear that the stage-discharge relationship is very easy to calibrate and use, but the figures show that even for the calibration periods the stage discharge has large errors. Thus, while the correlation coefficients are high, the model has a poor fit to the data.

Rigid Boundary Model. The river stage computed by this rigid boundary model was calibrated by adjusting the estimated channel roughness coefficient, Manning's n. Manning's n was calibrated by multiplying a constant with the estimated n value for the main channel and overbanks cross sections. The model was run with flow data from the calibration periods and error between the observed and computed water surface was determined. The process was repeated with different values of the constant until no reduction in error resulted.

Manning's n for the Fort Pemberton reach was determined to be 0.031. At the Locopolis reach it was necessary to allow Manning's n to vary with discharge, since there appeared to be large differences in headloss in the reach at different flow levels. The calibrated Manning's n varies as a power function of discharge, and is about 0.018 for the extreme high flow and approximately 0.040 for the low flow.

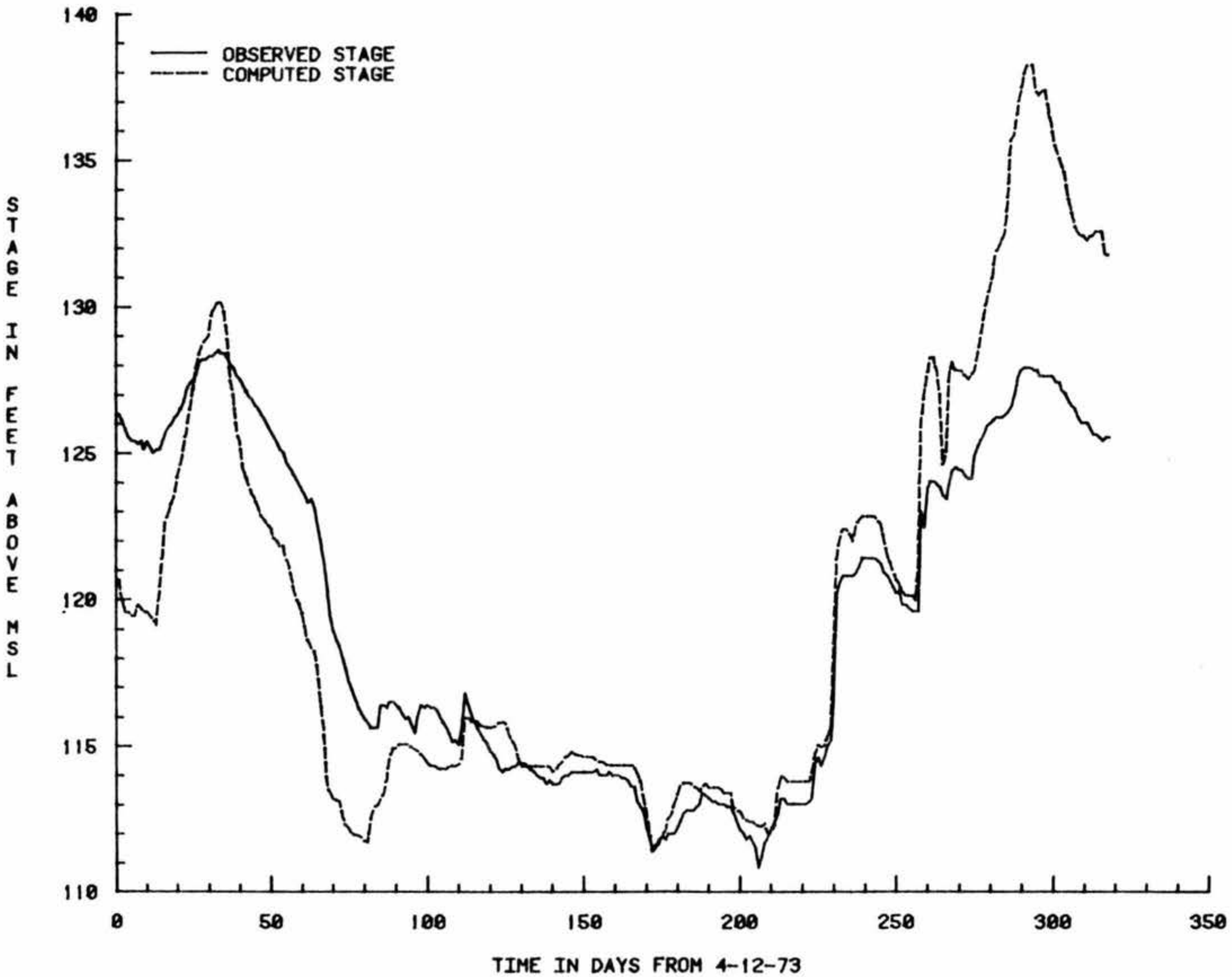

Figure 4.14. Stage-discharge model calibration at Ft. Pemberton.

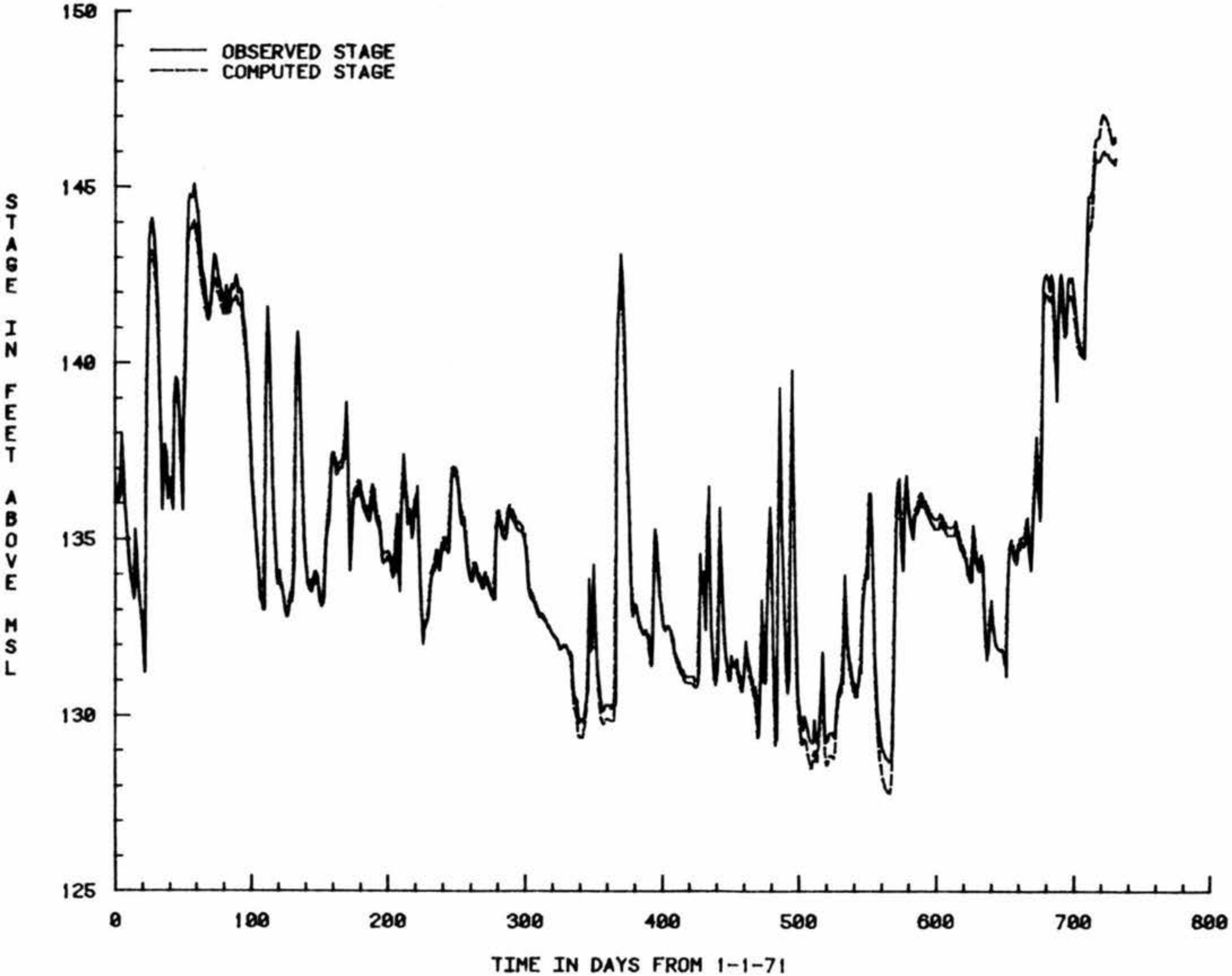

Figure 4.15. Stage-discharge model calibration at Locopolis.

These values are reasonable considering the frictional characteristics of the system. Figures 4.16 and 4.17 show the observed and computed stage hydrographs for the calibration period. As can be seen the rigid boundary model fit is much better than the stage-discharge and is similar to the sediment routing model.

Verification of Comparison on Models

To test the three models, an additional run utilizing the comparison models was made for each reach using the calibration results. These runs were made for the same periods described in Section 4.2. At Fort Pemberton, the verification period was from February 24, 1974 to December 31, 1974 (311 days). At Locopolis, the period was from January 1, 1973 to December 31, 1974 (730 days). Figures 4.18 through 4.21 show plots of the measured and computed stage at Fort Pemberton and Locopolis for the verification period.

Comparison of Methods

For comparison purposes, model error is defined as the difference between observed and predicted stage for each day. Figures 4.22 and 4.23 show the relative frequency distribution of error for the verification periods and Table 4.3 lists the statistics of the absolute error for each method.

As indicated in the figures and table, all three methods have approximately the same mean error for Locopolis, but the process models have much lower maximum errors than the stage-discharge relationships. Since there are few channel changes in this reach, the results of the two theoretical models, rigid boundary and movable bed, are essentially the same. At Fort Pemberton the sediment routing model is clearly

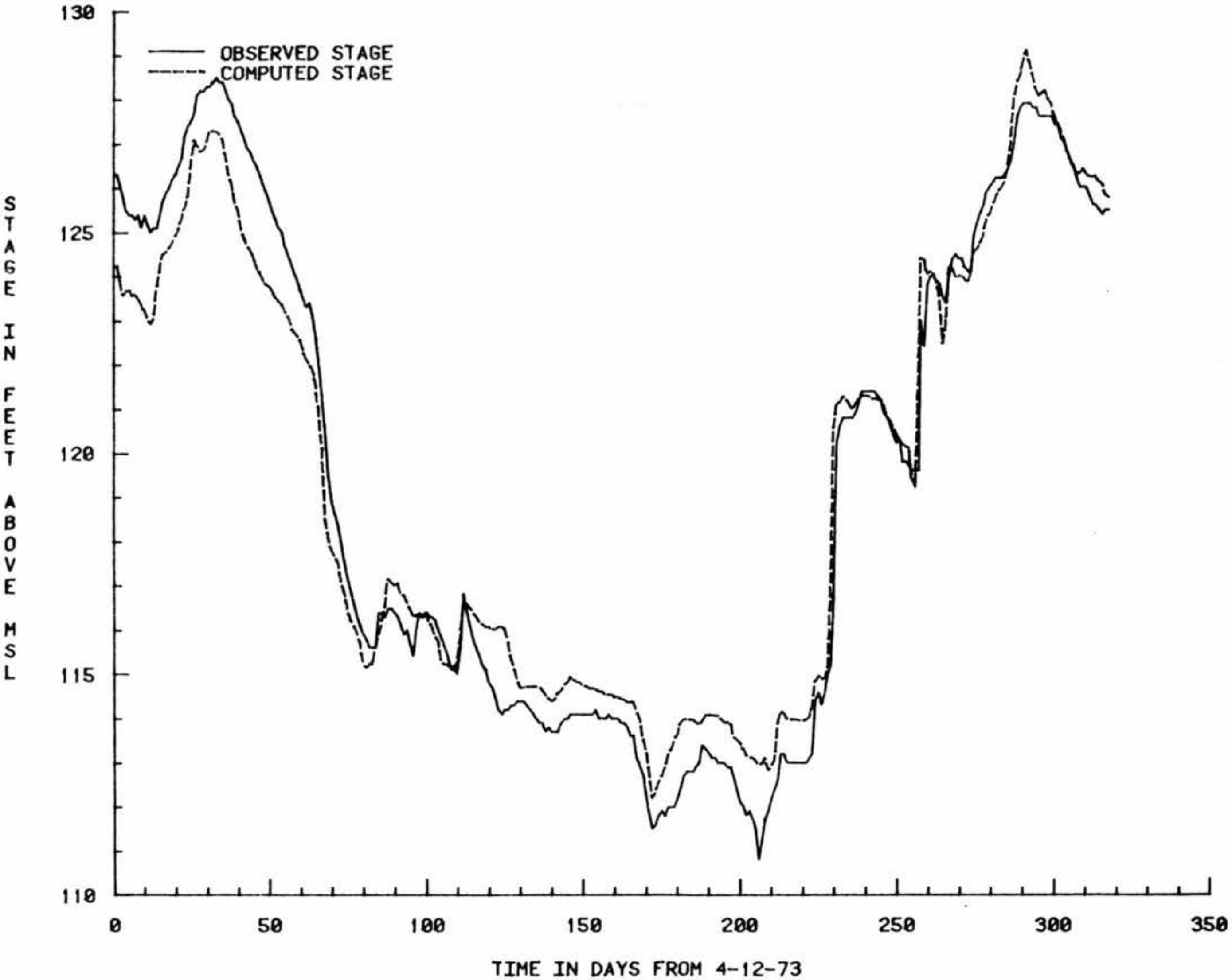

Figure 4.16. Rigid boundary calibration at Fort Pemberton.

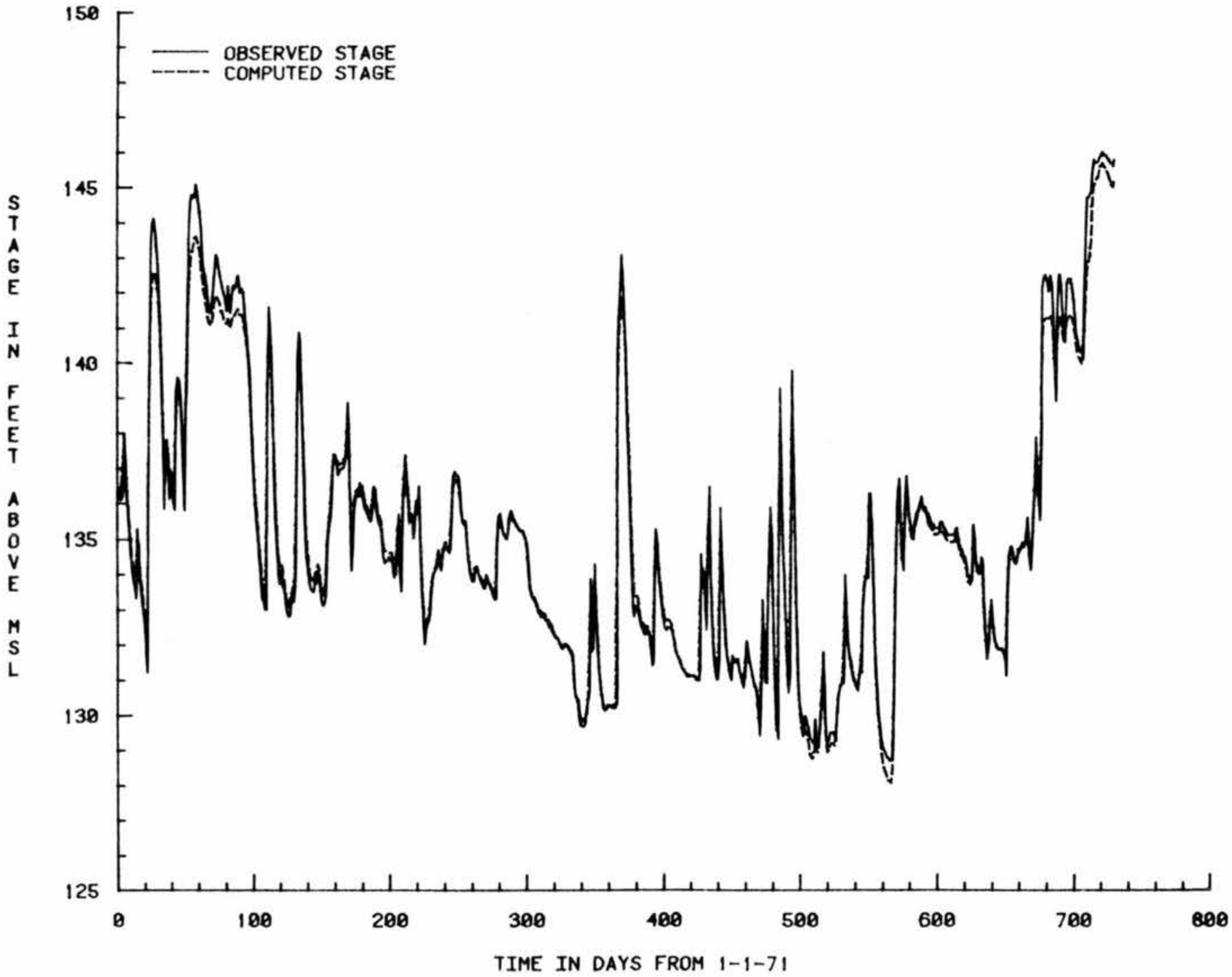

Figure 4.17. Rigid boundary calibration at Locopolis.

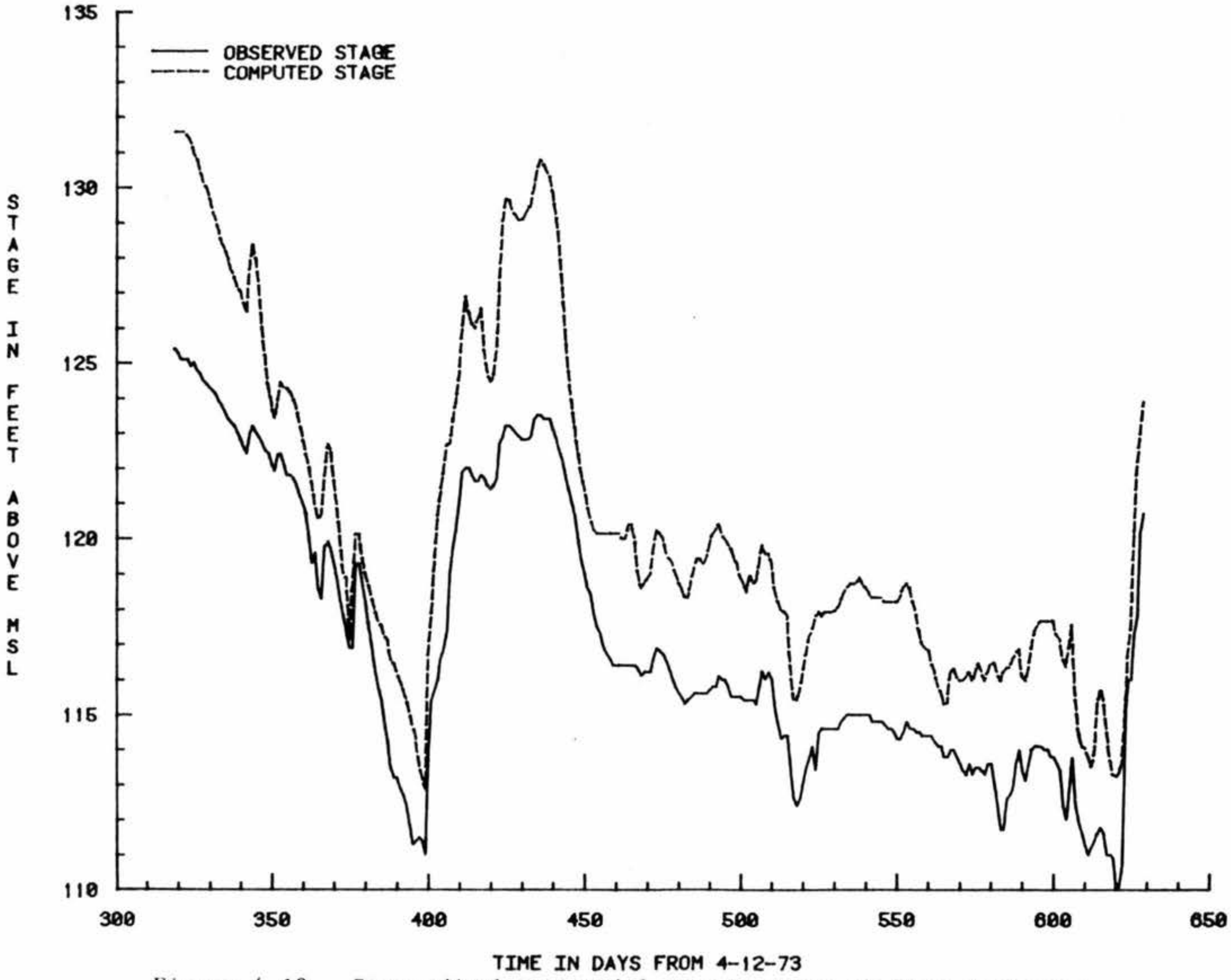

Figure 4.18. Stage-discharge model verification at Fort Pemberton (verification starts on Day 319, 2-24-74).

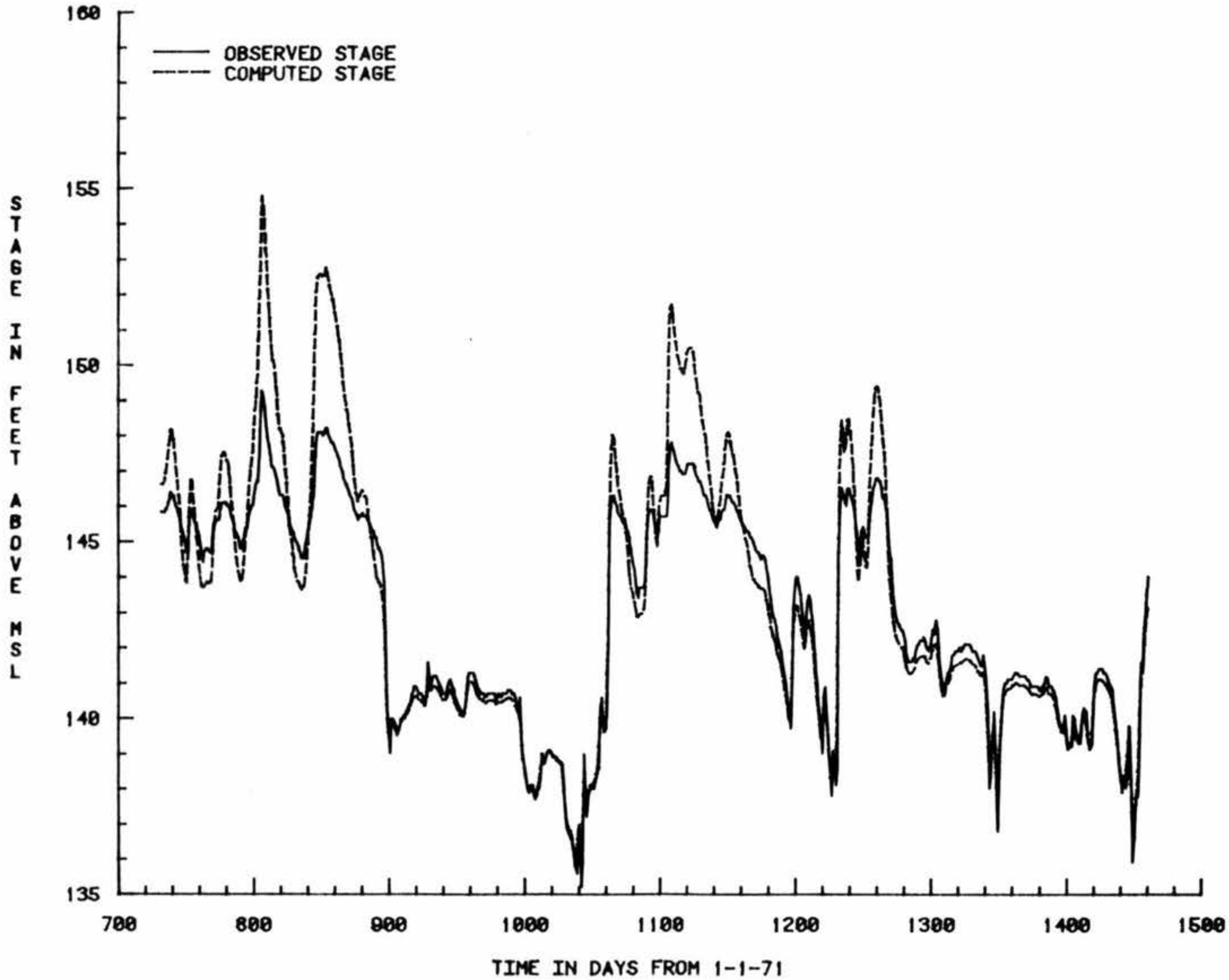

Figure 4.19. Stage-discharge model verification at Locopolis (verification starts on Day 732, 1-1-73).

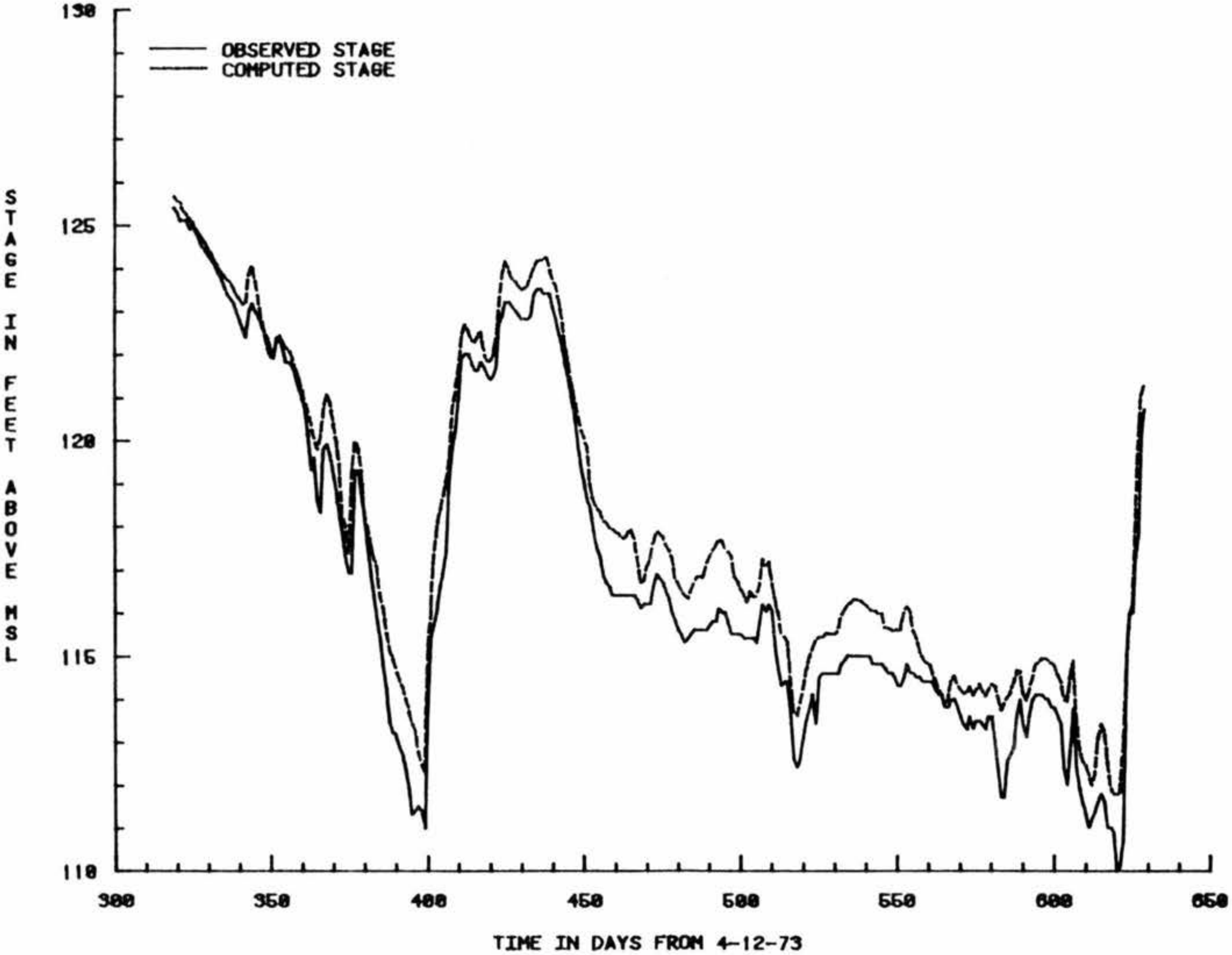

Figure 4.20. Rigid boundary model verification at Fort Pemberton (verification starts on Day 319, 2-24-74).

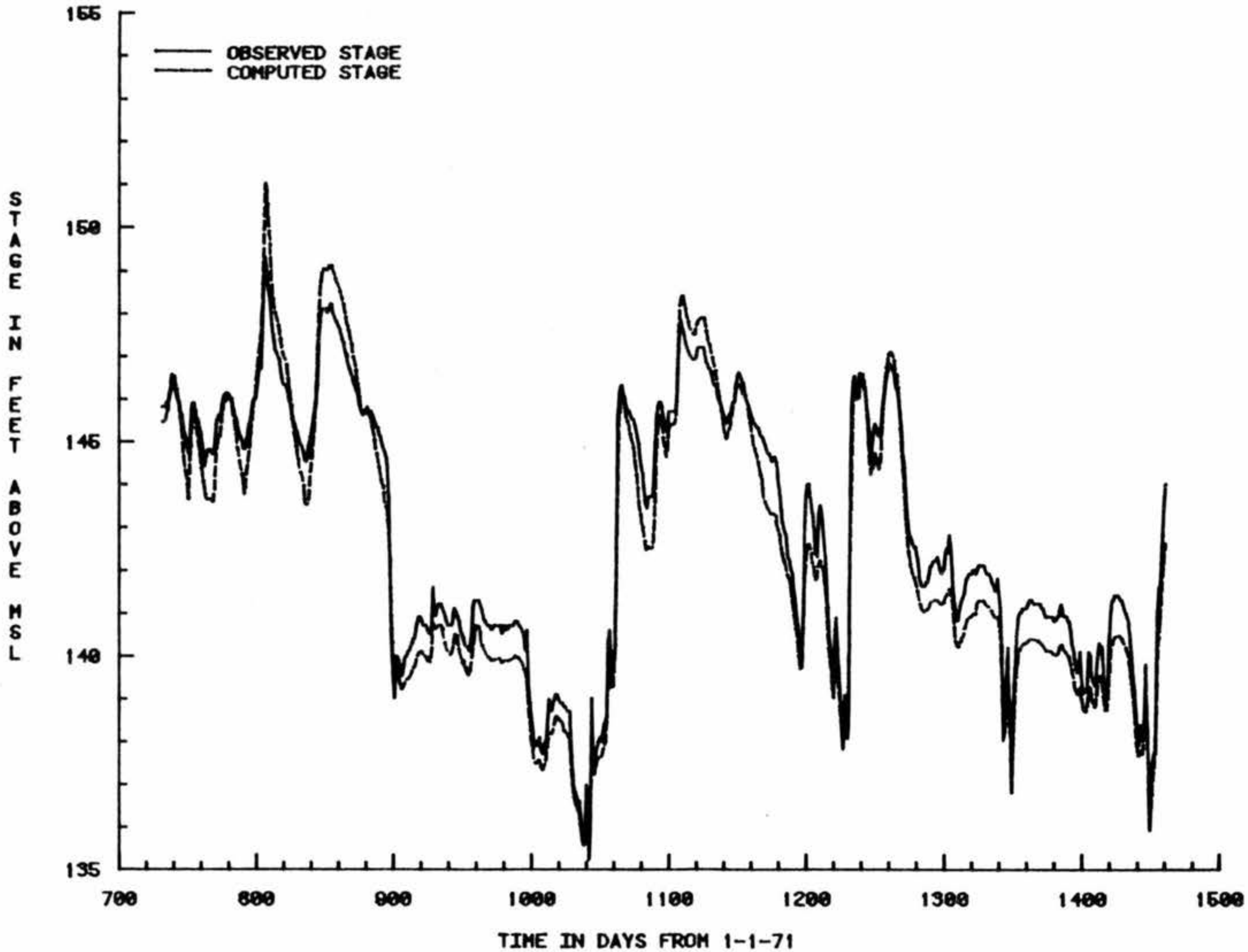

Figure 4.21. Rigid boundary model verification at Locopolis (verification starts on Day 732, 1-1-73).

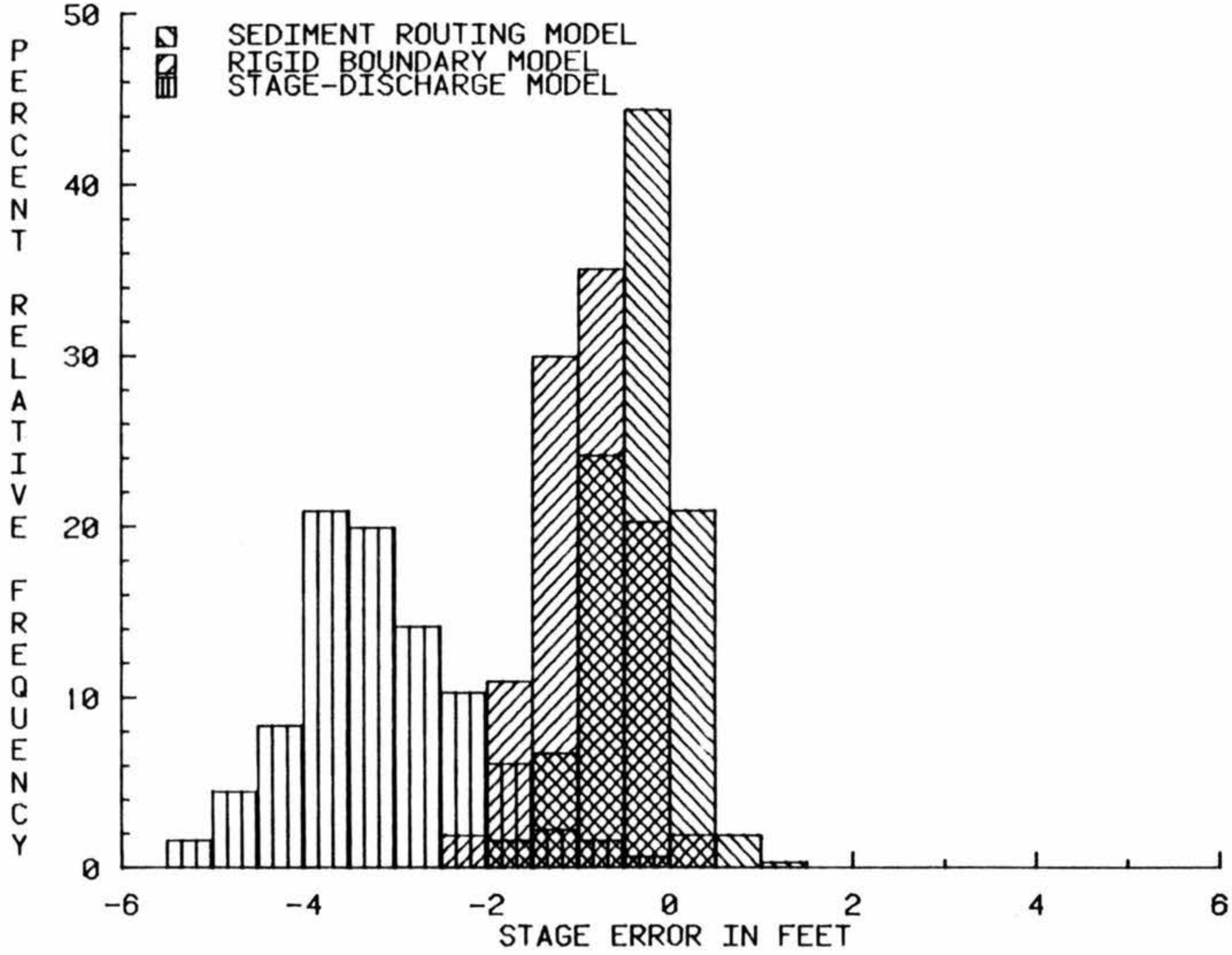

Figure 4.22. Model error frequency at Fort Pemberton.

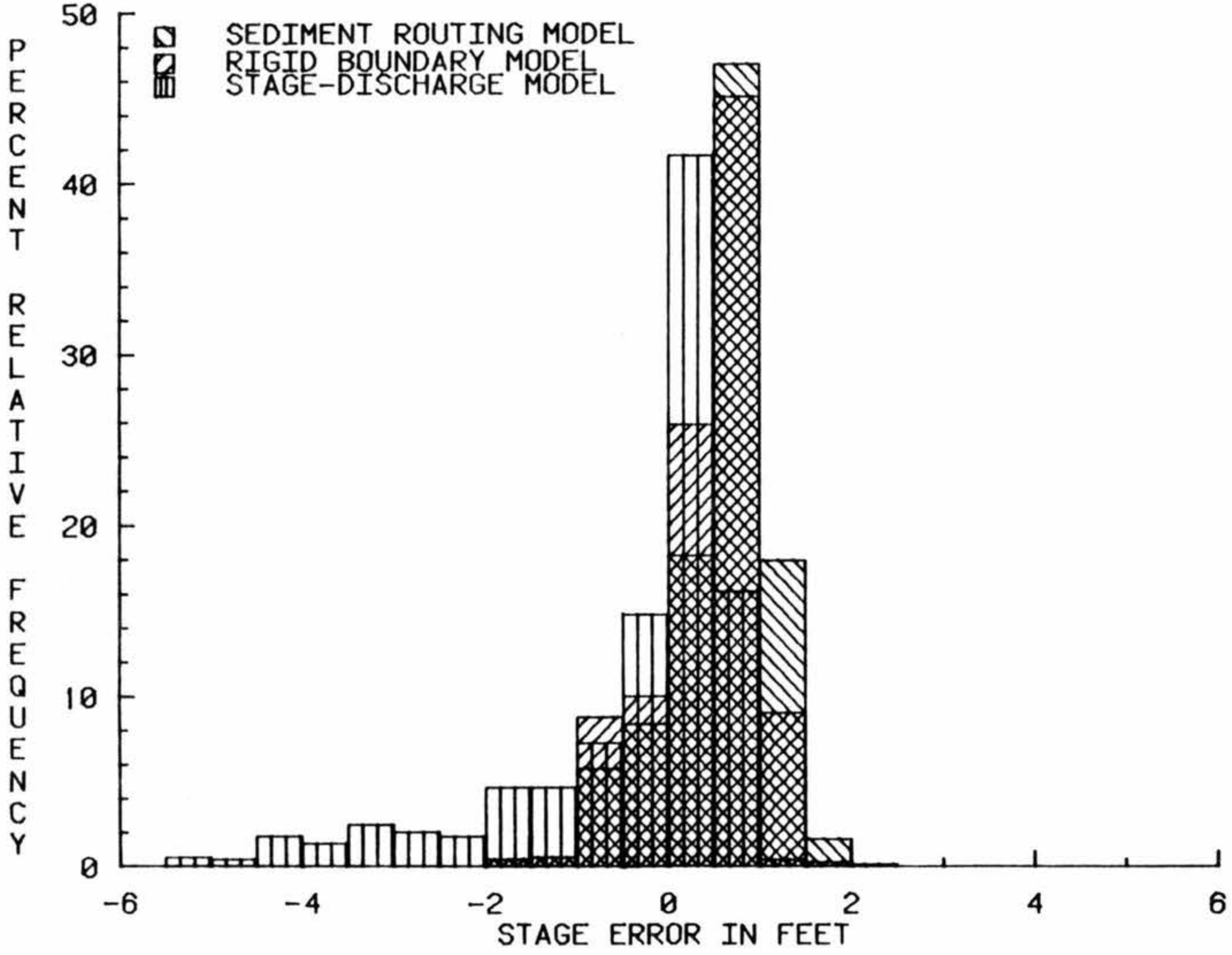

Figure 4.23. Model error frequency at Locopolis.

Table 4.3. Statistics of Absolute Model Errors

	Error in Feet			
	Calibration		Verification	
	Mean	Maximum	Mean	Maximum
Stage-Discharge Model				
Fort Pemberton	2.56	10.38	3.47	7.27
Locopolis	0.33	2.56	0.80	5.48
Rigid Boundary Model				
Fort Pemberton	0.30	3.97	0.92	2.39
Locopolis	0.29	2.27	0.62	1.91
Sediment Routing Model				
Fort Pemberton	0.88	3.60	0.45	1.78
Locopolis	0.34	2.12	0.74	2.21

better than the simple backwater model and superior to the empirical one. The reason for this is that the model can predict and adapt itself to changes in river conditions related to sediment movement and deposition. In contrast the empirical and rigid boundary models assume an invariant setting and hence they cannot detect and adapt themselves to a changing environment.

Application

General

This section presents the results of two Yazoo River Basin sedimentation study design alternative runs (Simons et al., 1978). These results are presented to demonstrate the models complete capabilities and to show the wide range of information that can be determined through use of the model. Figure 4.24 shows the Yazoo River Basin. The basin is divided into two regions. The first region is the delta. The delta extends from the loess bluffs to the east to the natural and artificial levies of the Mississippi River to the west. The delta is characterized by numerous old river channels, natural levies and mild slope streams which experience severe flooding and sedimentation. The

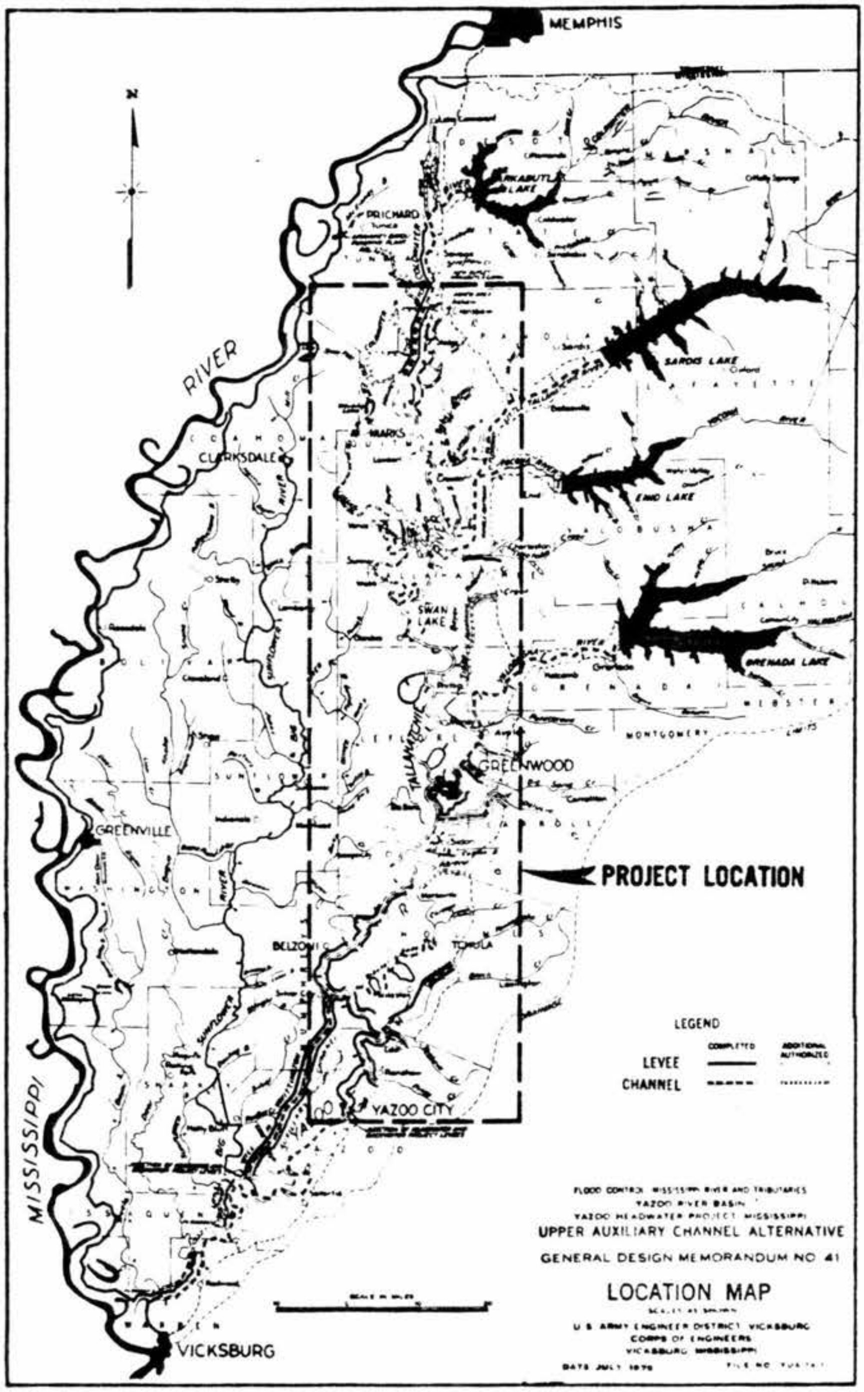

Figure 4.24. Project location map (after U.S. Corps of Engineers, 1975)

main stem of the Yazoo River lies completely in the delta region. The second region is the uplands. The uplands extend from the loess bluffs to the west to Pontotac Ridge in the northeast. The uplands vary in age and composition but are characterized by relatively steep streams which experience severe erosion and sedimentation problems. The majority of the Yazoo River tributaries originate in the uplands.

The Yazoo Basin Sedimentation Study involved an analysis of the main channel and its tributaries from which water and sediment is routed through the main channel. The purpose of the analysis was to determine the effectiveness of the proposed system considering flood control, navigation, and the location of aggradation and degradation problems in the main channel and its tributaries. Methods of minimizing operation and maintenance problems were also evaluated. The analysis provided a method for evaluating the Upper Yazoo Project system and the various design alternatives outlined by the U.S. Army Corps of Engineers (1975).

In the Phase I study the emphasis was to evaluate the river response to the various design alternatives on the main stem Yazoo-Tallahatchie-Coldwater River system and principal tributaries such as the Little Tallahatchie, Yocona, and Yalobusha Rivers. The Sunflower River Basin was excluded from the analysis. Utilizing the model, the effects of channel enlargement on flowline, sediment depositional rates, and other aspects of river response were evaluated.

<u>Alternative Runs</u>

Thirteen alternative study runs were made using the known discharge sediment routing model. These alternative study runs were conducted by routing sediment through the tributaries and the Yazoo

River main stem for a selected hydrograph utilizing the various alternative plans. Two of the alternative study runs are described here.

Run No. 1. Simulation run utilizing a 50-year synthetic hydrograph (11 years of recorded data and 39 years of generated data with natural (existing) river conditions. The Greenwood Cutoff was assumed closed for flows less than 25,000 cfs and open for flows greater than 25,000 cfs. It was further assumed that Abiaca Creek would only deliver about 20 percent of the sediment inflow at the hill line to the Yazoo River. Figure 4.25 shows the spatial design use in the run.

Run No. 2. Simulation run utilizing a 50-year synthetic hydrograph with Plane E conditions and Greenwood Cutoff operated as in Run No. 1. It was assumed that Abiaca Creek would be a leveed floodway that would deliver all of the sediment contributed at the hill line to the Yazoo River. Figure 4.26 shows the spatial design use in the alternative run.

Results

The spatial designs for various study runs differed. Nevertheless, it is worthwhile to identify the river segments utilizing the same designation system. Hence, referring back to Figure 4.25, River Segment No. 1 extends from the mouth of Big Sunflower to Belzoni. River Segment No. 2 extends from Belzoni to just below the Greenwood Bendway. River Segment No. 3 includes the Greenwood Bendway, and River Segment No. 4 extends from immediately upstream of the Greenwood Bendway to Arkabutla Dam. The Yalobusha River is identified as Rver Segment No. 5. River Segment No. 6 includes the Little Tallahatchie and the P-Q Floodway. The Yocona River is defined as River Segment

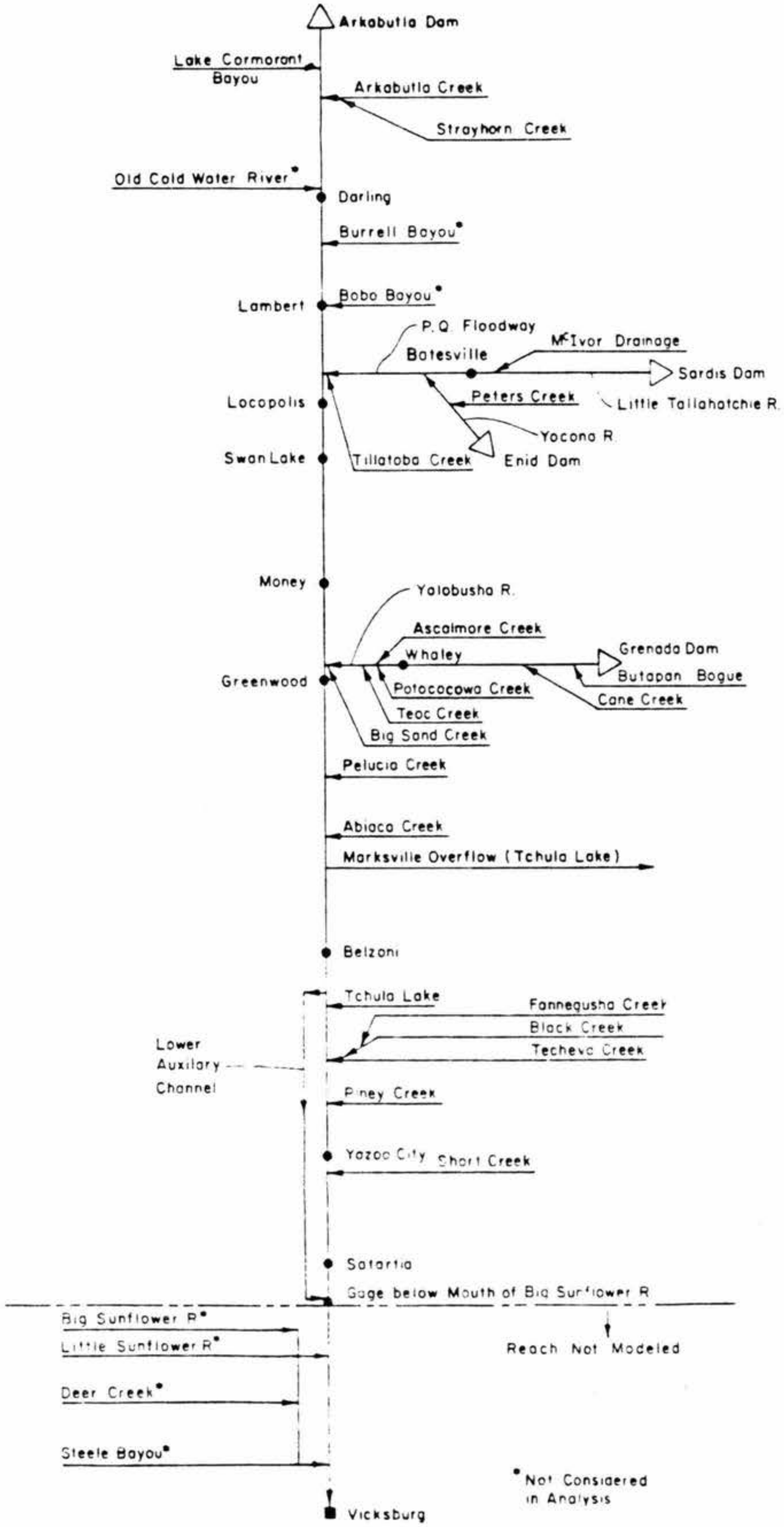

Figure 4.25. Spatial representation of the existing conditions (Run No. 1).

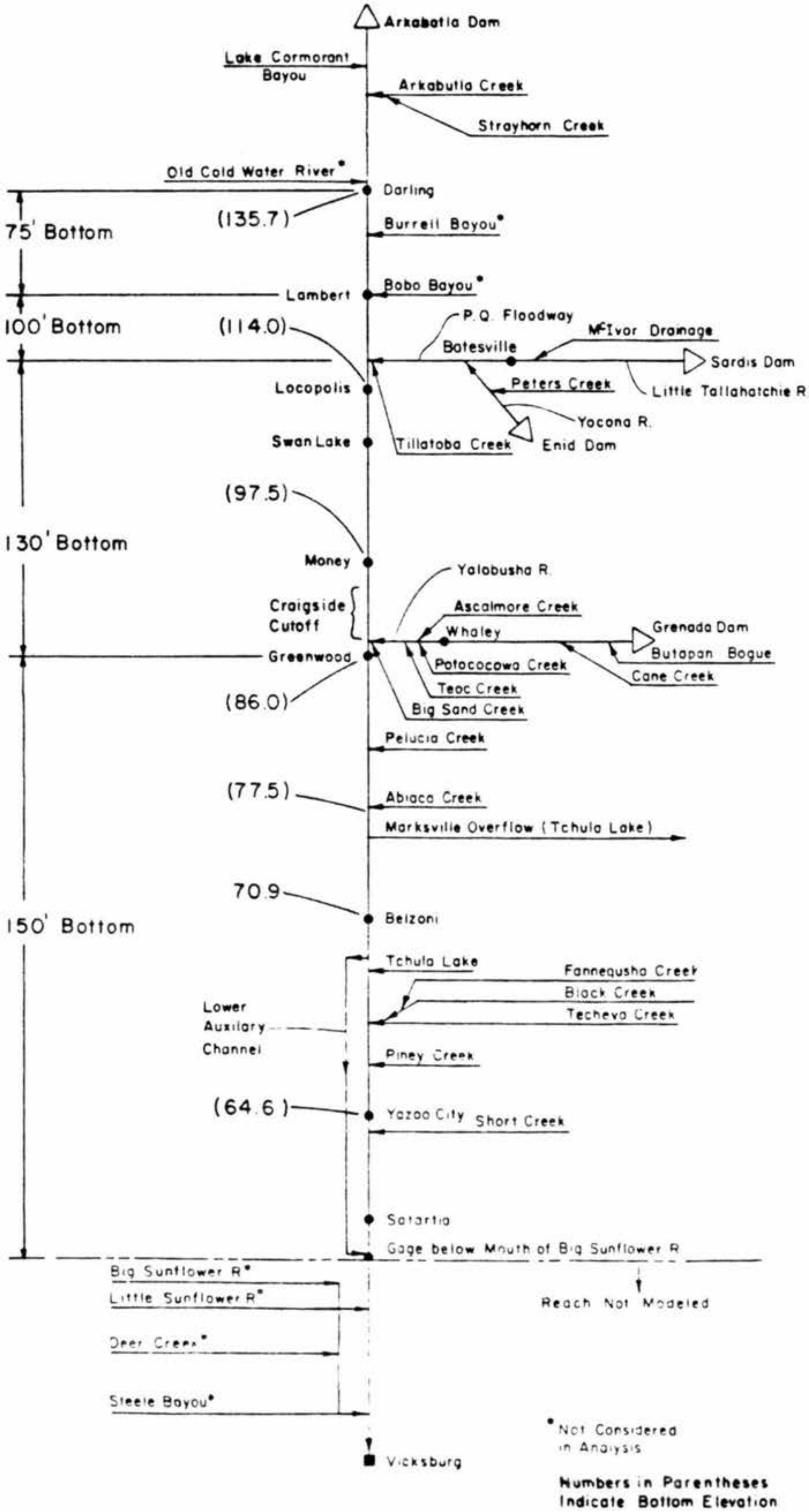

Figure 4.26. Spatial representation of Plan E conditions (Run No. 2).

No. 7 and the Greenwood Cutoff is identified as River Segment No. 8. Simulated results for different river conditions for a 50-year synthetic hydrograph are summarized in Table 4.4.

Table 4.4. Summary of Net Degradation and Aggradation for 50 years.

Alternative Runs No.	Net Degradation and Aggradation in 10^3 cubic yards							
	Reaches							
	1	2	3	4	5	6	7	8
1	--	1,941	445	8,097	1,242	-2,078	908	-48
2	--	22,075	2,846	16,909	4,181	-2,809	658	3

Run No. 1. For the main stem from Belzoni to Arkabutla Dam (River Segments No. 2, 3, 4, and 8), the estimated rate of net filling (net degradation and aggradation) is about 210,000 cubic yards per year with natural conditions. Figure 4.27 shows the beginning and final bed profiles after 50 years under natural conditions.

The maximum water surface elevations at each cross section are also plotted in Figure 4.27. These maximum water surface elevations are the maximum values at each cross section considering the 50-year simulation period. These values do not necessarily occur at the same time for all of the cross sections and may not take place during the period of maximum discharge. The downstream water surface elevations and the long-term sediment movement in the system will dictate local water surface elevations.

The cumulative net aggradation volumes in the main stem for the 50 years of simulation are shown in Figure 4.28. Generally, the main stem segments, except the Greenwood Cutoff, are depositing for the 50-year simulation cycle. River Segment No. 4 (from Greenwood to Arkabutla

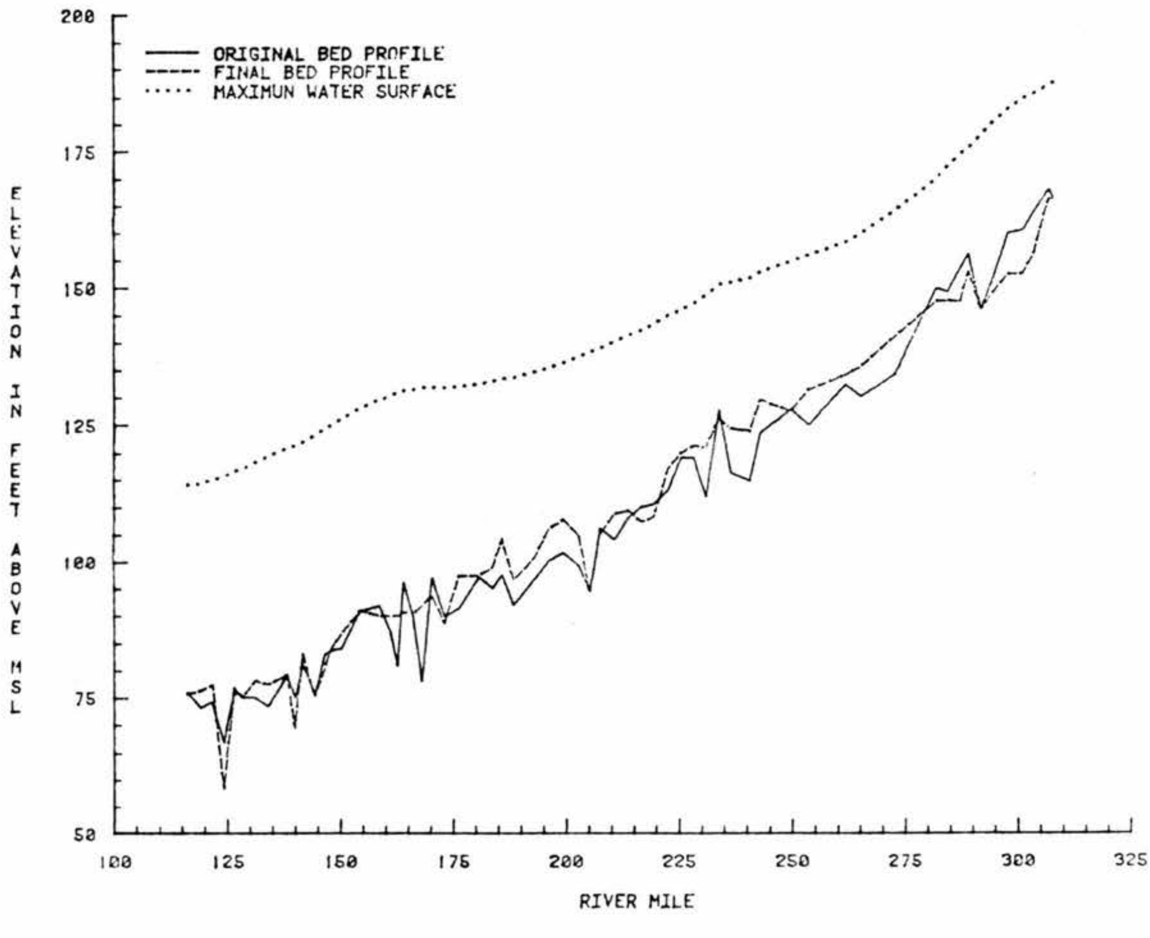

Figure 4.27. Beginning and final bed profiles and maximum water surface elevations for natural conditions (Run No. 1).

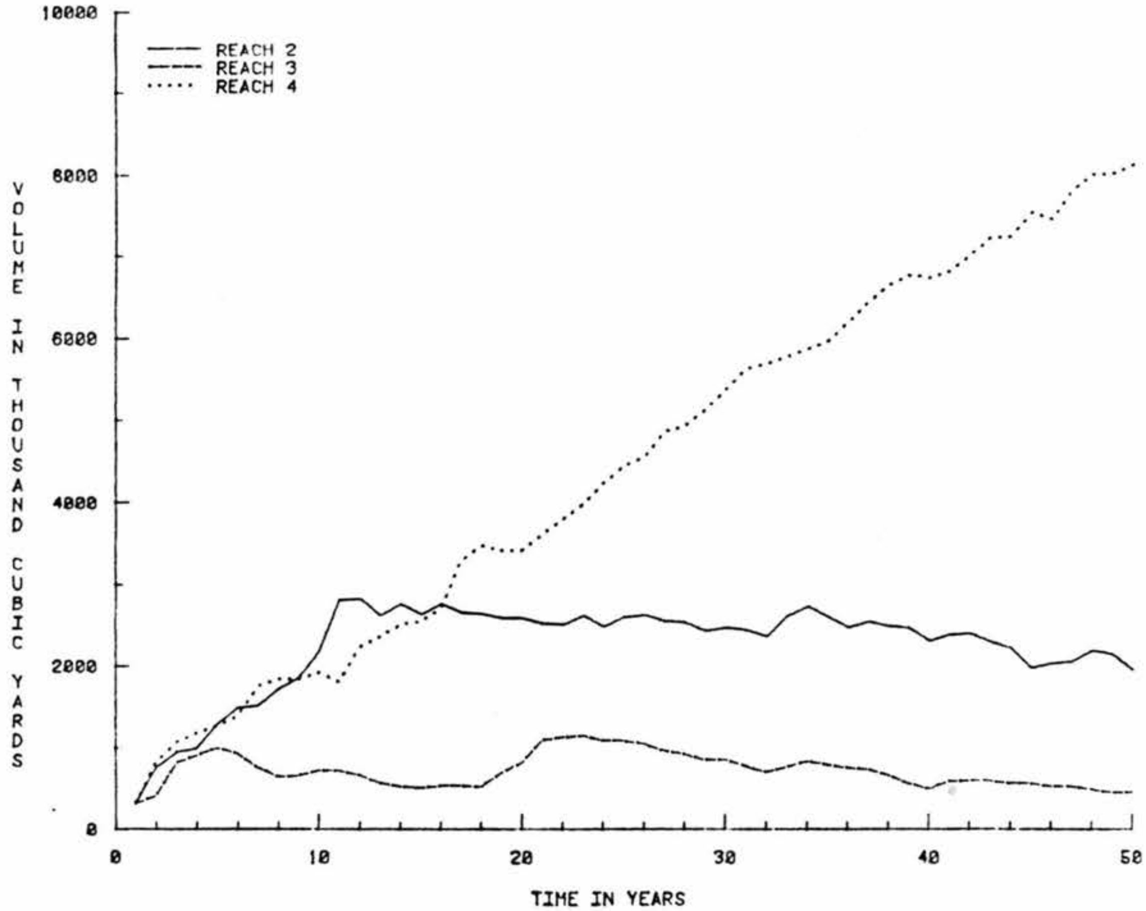

Figure 4.28. Cumulative aggradation (positive) volumes in the main stem for natural conditions (Run No. 1).

Dam) is filling, particularly below the P-Q Floodway. This is due to severe degradation in the P-Q Floodway.

The cumulative degradation and aggradation in the major tributaries (River Segments No. 5, 6, and 7) is shown in Figure 4.29. As indicated by model results, the Yalobusha River (River Segment No. 5) is aggrading which agrees with observations by the U.S. Army Corps of Engineers (Design Memorandum No. 41). This river has been filling with sediment since the enlargement of the channel cross section in 1954. River Segment No. 6 (P-Q Floodway and the Little Tallahatchie River) is degrading according to model calculations. This also agrees with observations by the U.S. Army Corps of Engineers. The river bed gradient of the Little Tallahatchie River is about four times that of the main stem river due to construction of the P-Q Floodway. Degradation in the P-Q Floodway was so severe that extensive emergency dredging was required to maintain the Tallahatchie River below the mouth of the P-Q Floodway. The Yocona River (River Segment No. 7) is sensitive to the water surface in the P-Q Floodway. The simulated results show that the Yocona River is almost in a state of equilibrium.

Examples of the stage-discharge relationship just below Greenwood Bendway (river mile 162.5) and at Swan Lake (river mile 219.08) during the first year, 10th year, 30th year, and 47th year are displayed in Figures 4.30 and 4.31. The 47th year was selected because it closely approximated the 1973 flood. These figures indicate the stage-discharge relationship at both stations are consistent with time for the natural condition.

Run No. 2. The net rate of sediment deposition in the main stem utilizing Plan E conditions increased approximately 400 percent. That

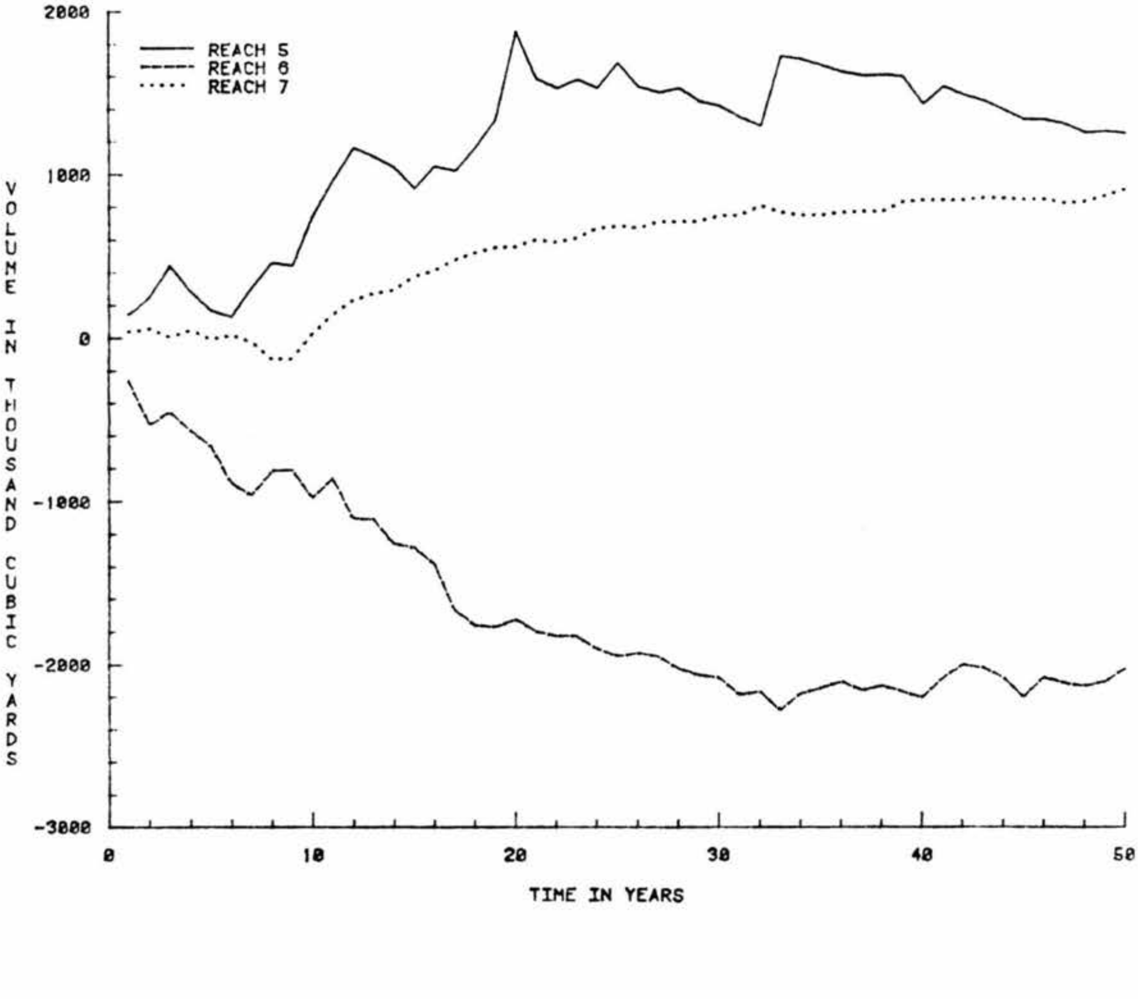

Figure 4.29. Cumulative degradation (negative) and aggradation (positive) volumes in the major tributaries for natural conditions (Run No. 1).

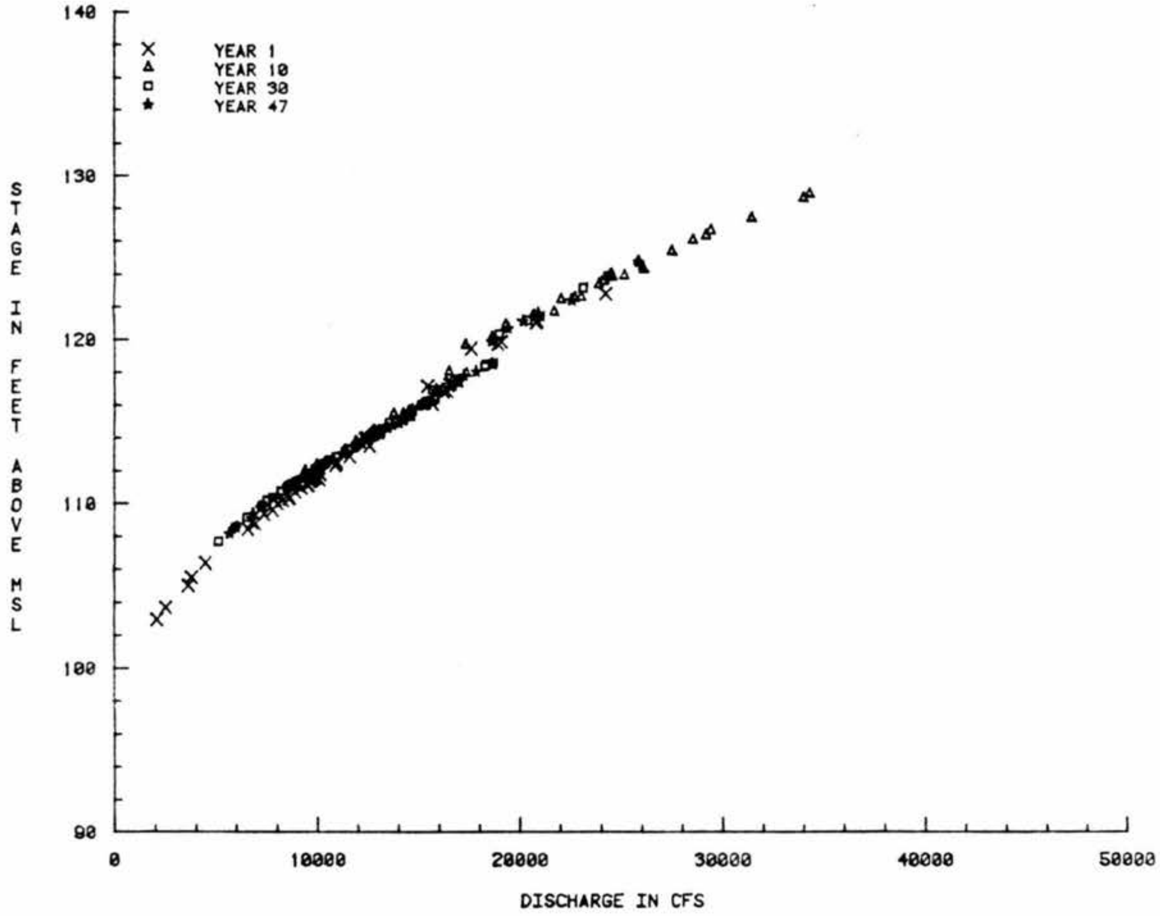

Figure 4.30. Stage discharge relationship downstream of the Greenwood Bendway (river mile 162.5) for natural conditions.

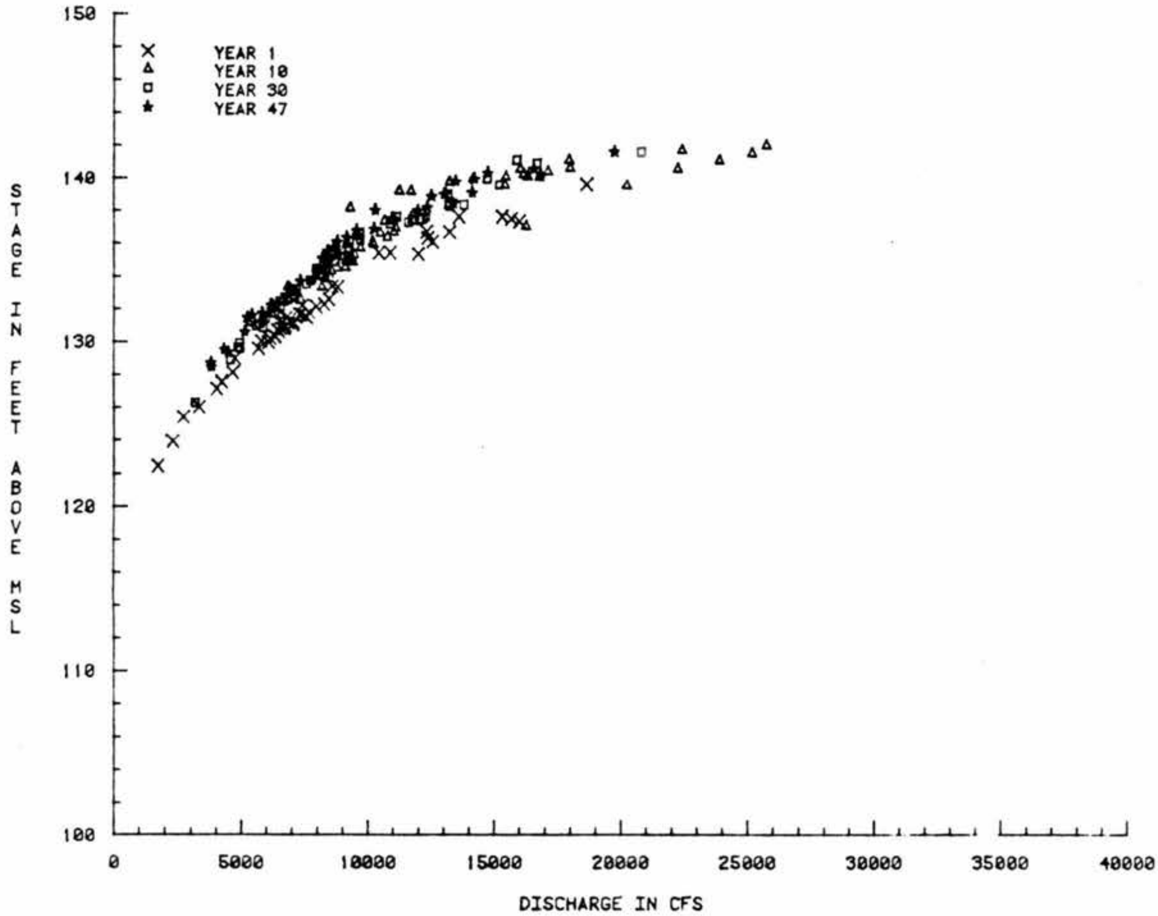

Figure 4.31. Stage discharge relationship at Swan Lake (river mile 219.08) for natural conditions.

is, the net depositional rate in the main stem is about 840,000 cubic yards per year. This is due to the enlargement of the channel, the increase in the sediment supply from Abiaca Creek (about 500 percent), and the increase in the degradation along the P-Q Floodway (about 40 percent). Lowering the water surface levels in the Tallahatchie enhances degradation in the P-Q Floodway.

Figure 4.32 shows the beginning and final bed profiles as well as the maximum water surface elevations along the main stem under Plan E conditions considering 50 years of simulation. This figure clearly indicates that the most important areas causing maintenance problems are the reaches below Abiaca Creek and below the mouth of the P-Q Floodway. The maximum water surface levels are generally higher than those under natural conditions for reaches downstream of Money. Cumulative aggradation volumes in the main stem for 50 years is shown in Figure 4.33. The study shows that deposition rates of Plan E are much larger than those for natural river conditions. The computed average rate of deposition for Plan E decreases with time, which is consistent with changes in hydraulic conditions.

For a detailed examination of Plan E, Figure 4.34 provides the maximum water surface elevations under both the natural and Plan E conditions for the 50-year simulation period. Generally, the maximum water surface elevations are higher for Plan E in the main stem except in the reach near and upstream of Swan Lake (river mile 219.908). The higher water surface elevations in the reach downstream of river mile 200.0 are due to accumulation of sediment for 50 years without maintenance, and the significant increase of sediment supply to the main stem from Abiaca and the P-Q Floodway. The reduction in stage

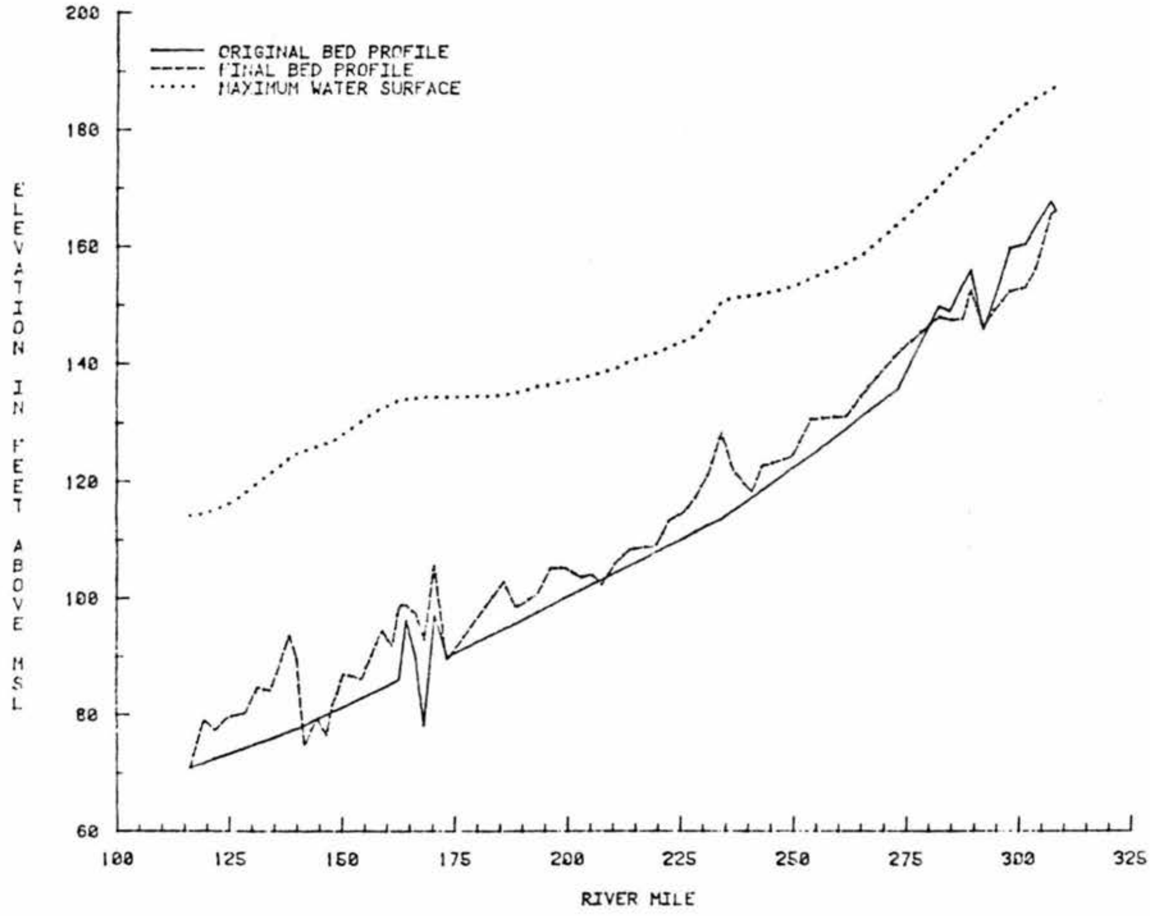

Figure 4.32. Beginning and final bed profiles and maximum water surface elevations for Plan E conditions (Run No. 2).

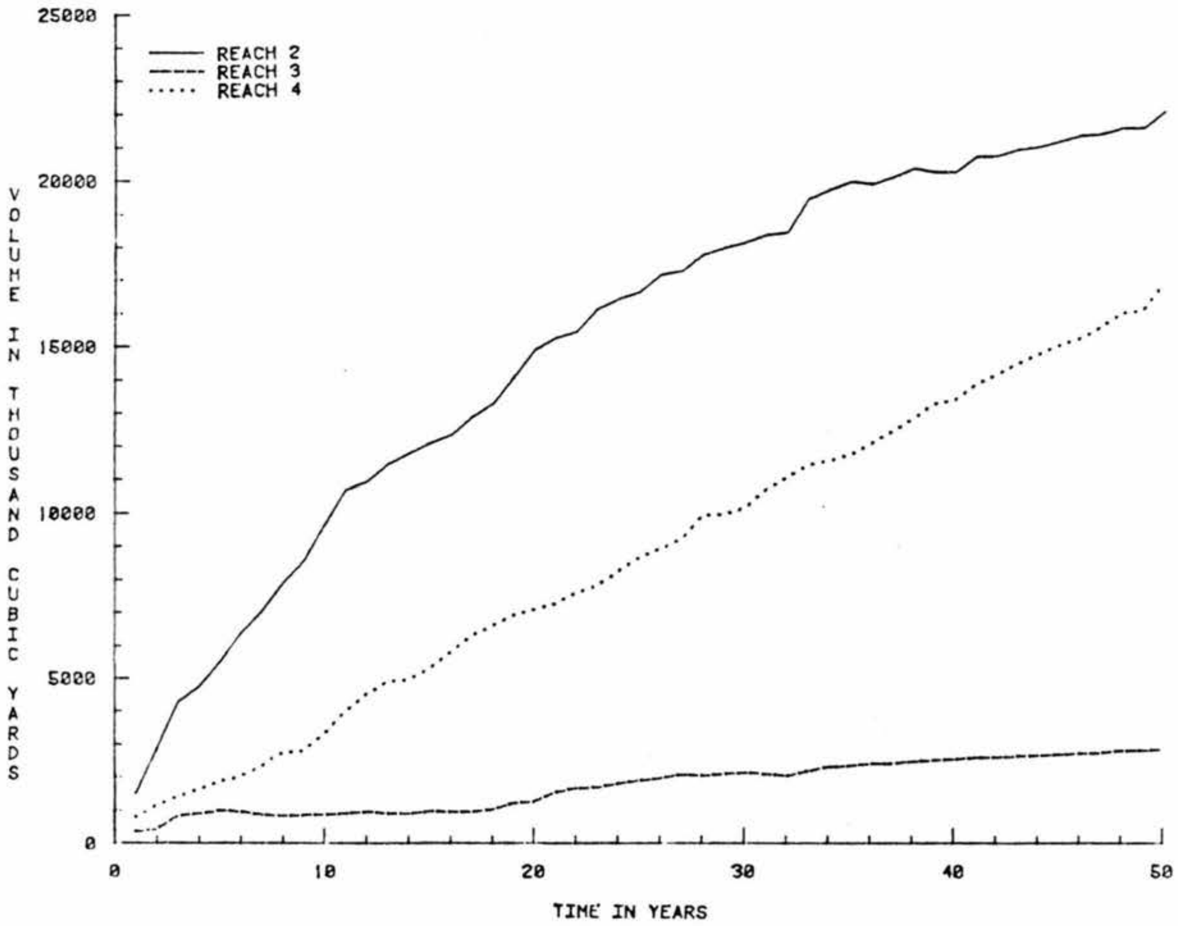

Figure 4.33. Cumulative aggradation (positive) volumes in the main stem for Plan E conditions (Run No. 2).

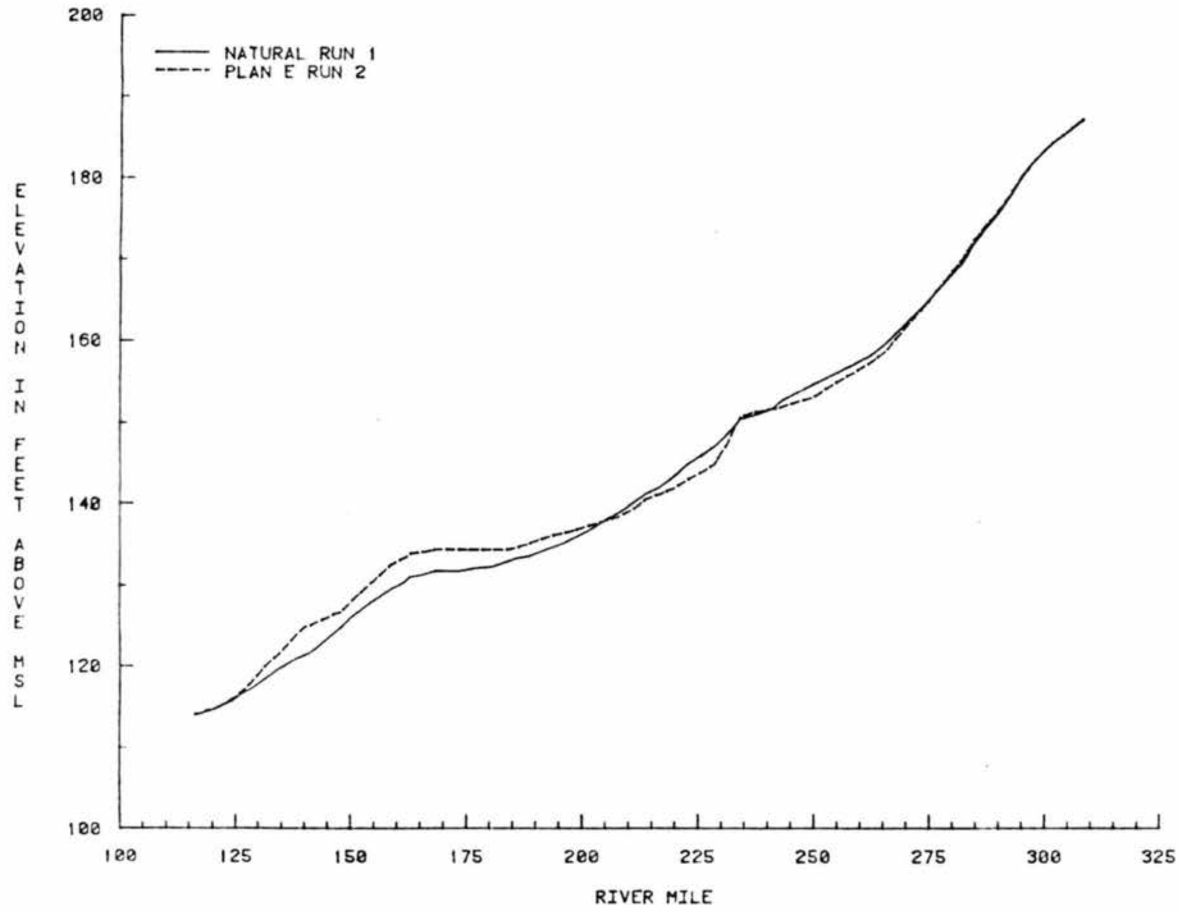

Figure 4.34. Comparison of maximum water surface elevations between natural and Plan E conditions (Runs No. 1 and 2).

near and upstream of Swan Lake is primarily the result of the Craigside Cutoff.

It is important to mention that if the Plan E channel is allowed to accumulate sediment for 50 years without maintenance, the efficiency of flood stage reduction of Plan E would be significantly decreased. This is further demonstrated in Figures 4.35 and 4.36. In these figures, the annual maximum water surface elevations just below the Greenwood Bendway (river mile 162.5) and at Swan Lake (river mile 219.08) for both natural and Plan E conditions are plotted. The river stage reduction by implementation of Plan E is only good in the first two years at river mile 162.5, but is always effective at Swan Lake. After the channel fills with sufficient sediment, river stage reduction is significantly decreased and finally terminated in some reaches. The stage frequency curves just below the Greenwood Bendway and at Swan Lake (river mile 219.08) for natural and Plan E conditions are shown in Figures 4.37 and 4.38, respectively. These curves indicate that the proposed Plan E can only be effective if maintenance measures such as dredging and/or control of sediment inflows from major sediment contributing tributaries are implemented. This is particularly true for the reach downstream of Money.

The stage-discharge relationship below Greenwood Bendway (river mile 162.5) and at Swan Lake (river mile 219.08) for different time periods are shown in Figures 4.39 and 4.40, respectively. Again, these figures indicate that the proposed Plan E can only be effective for flood control if maintenance dredging, control of sediment supply from tributaries, or other means of maintenance are implemented.

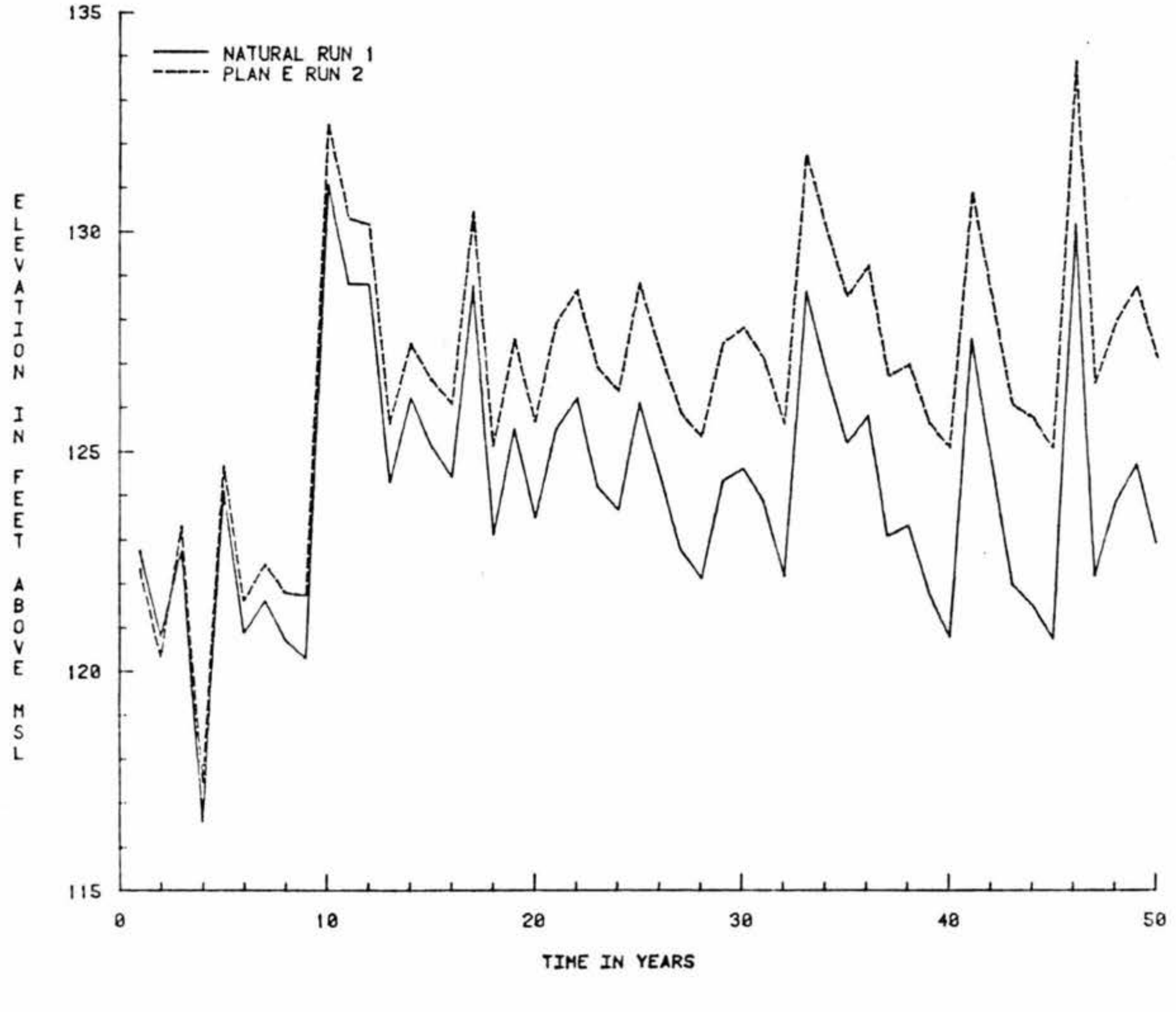

Figure 4.35. Annual maximum water surface elevations downstream of the Greenwood Bendway (river mile 162.5) for natural and Plan E conditions.

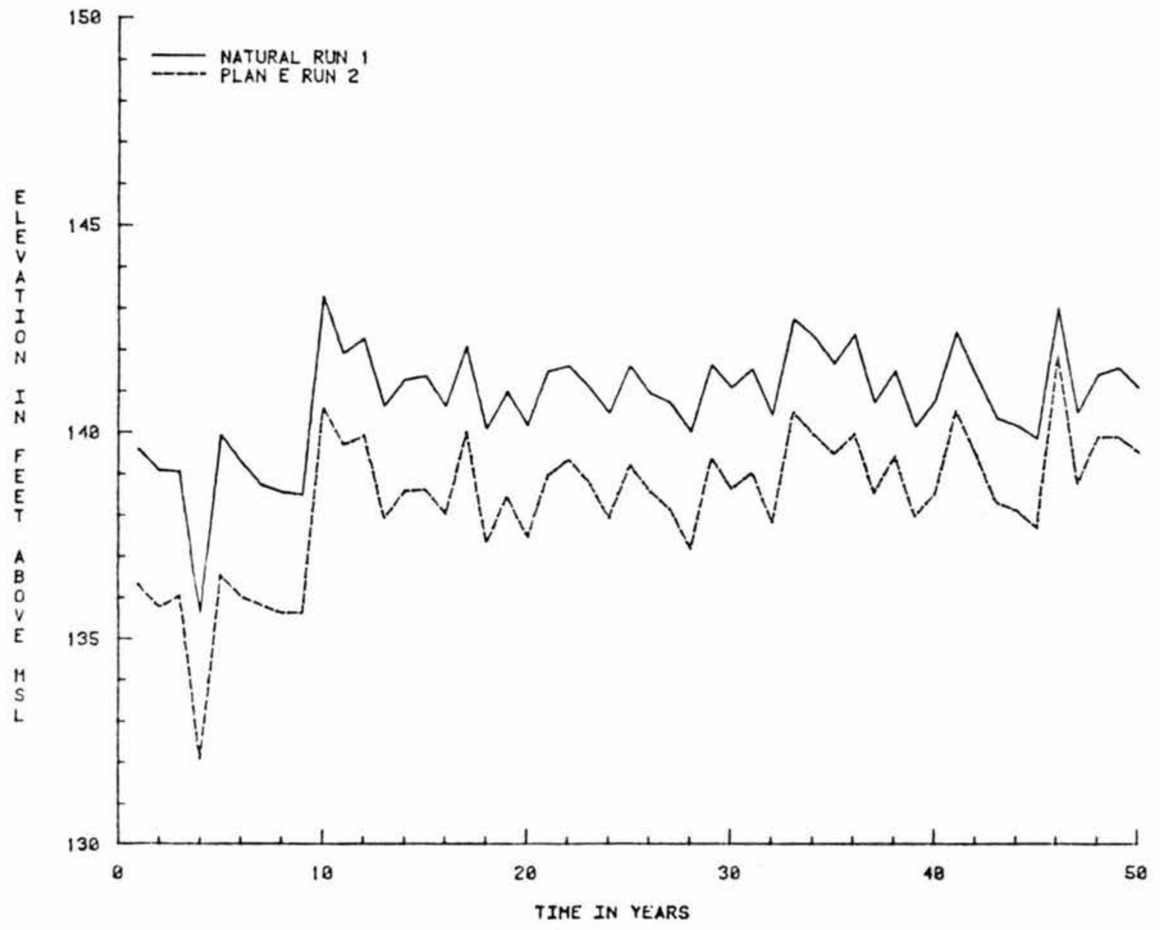

Figure 4.36. Annual maximum water surface elevations at Swan Lake (river mile 219.08) for natural and Plan E conditions.

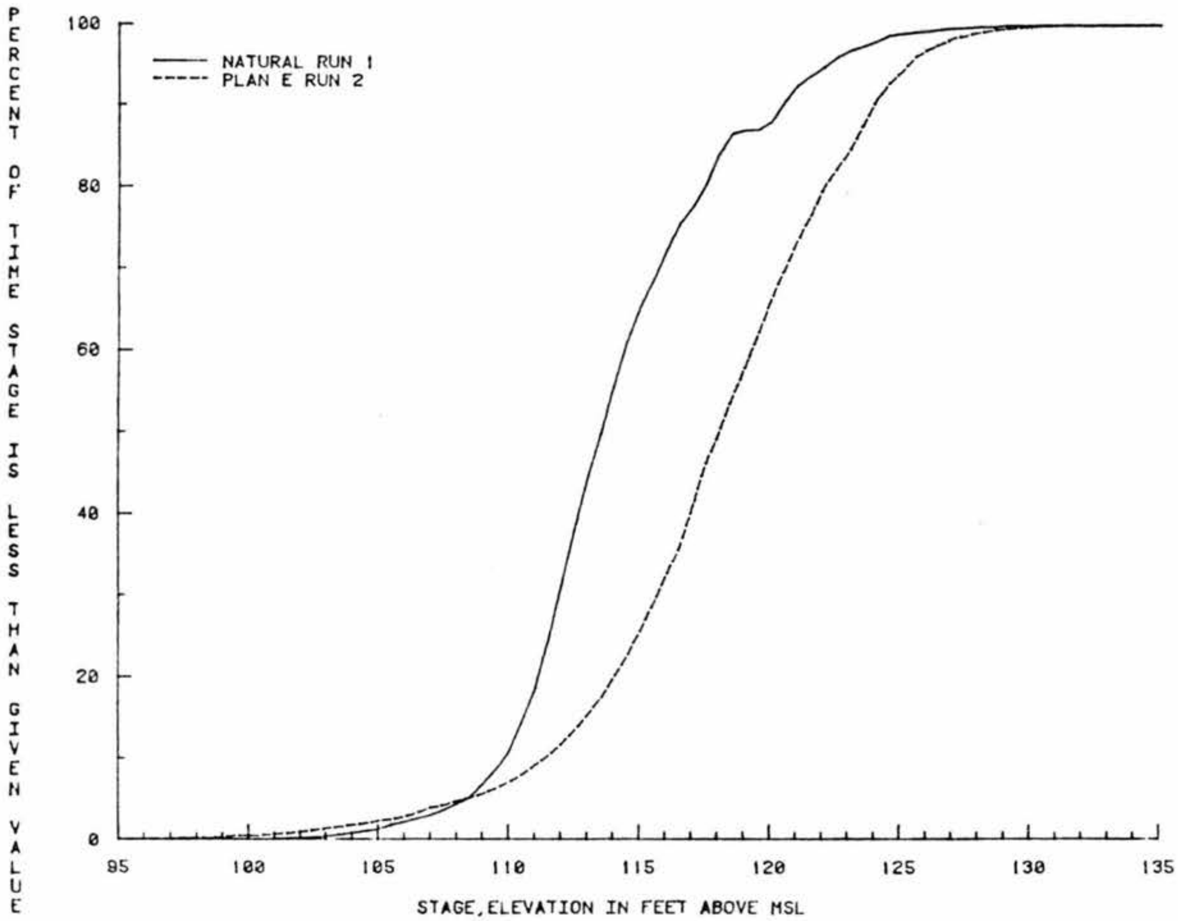

Figure 4.37. Stage frequency curve below the Greenwood Bendway (river mile 162.5) for natural and Plan E conditions.

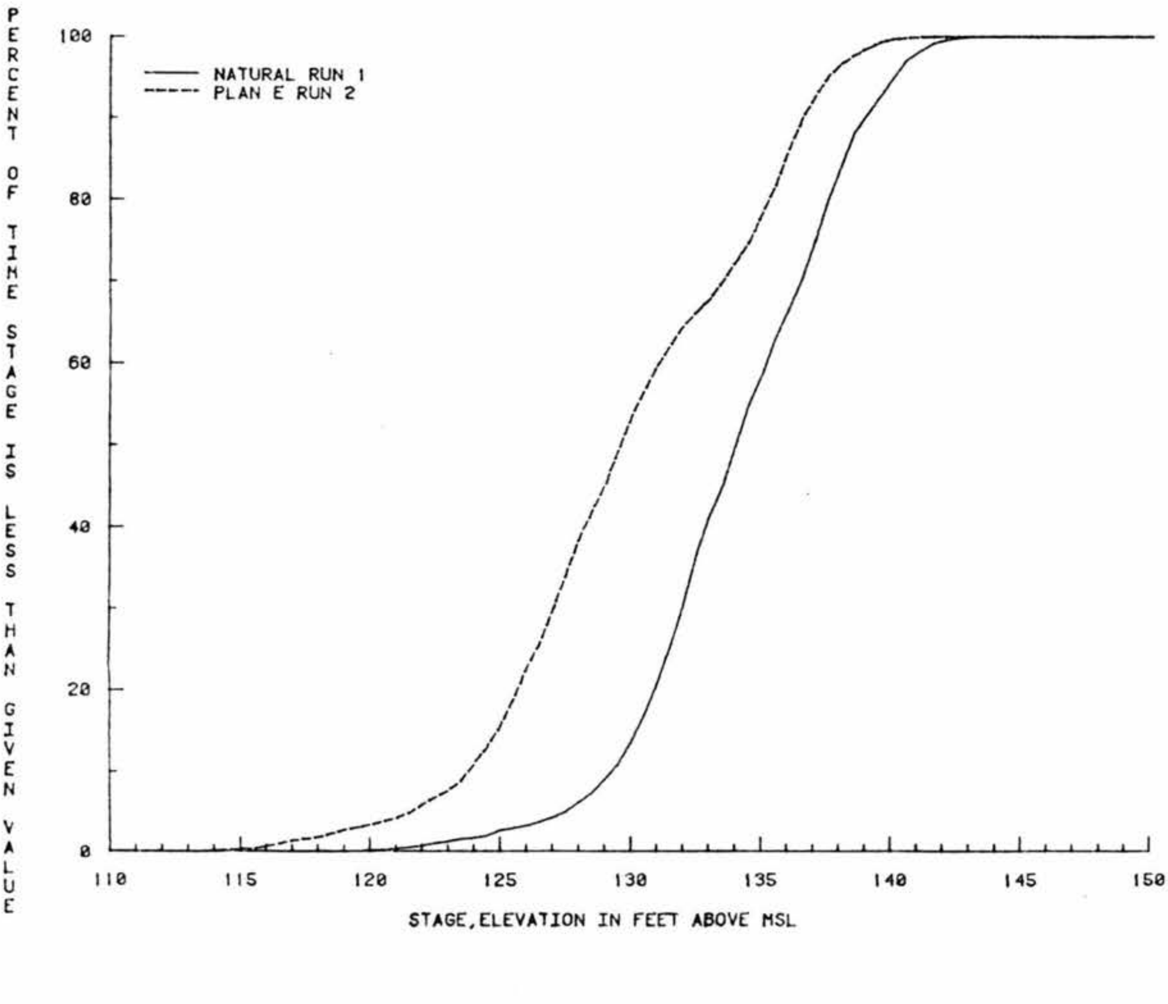

Figure 4.38. Stage frequency curve at Swan Lake (river mile 219.08) for natural and Plan E conditions.

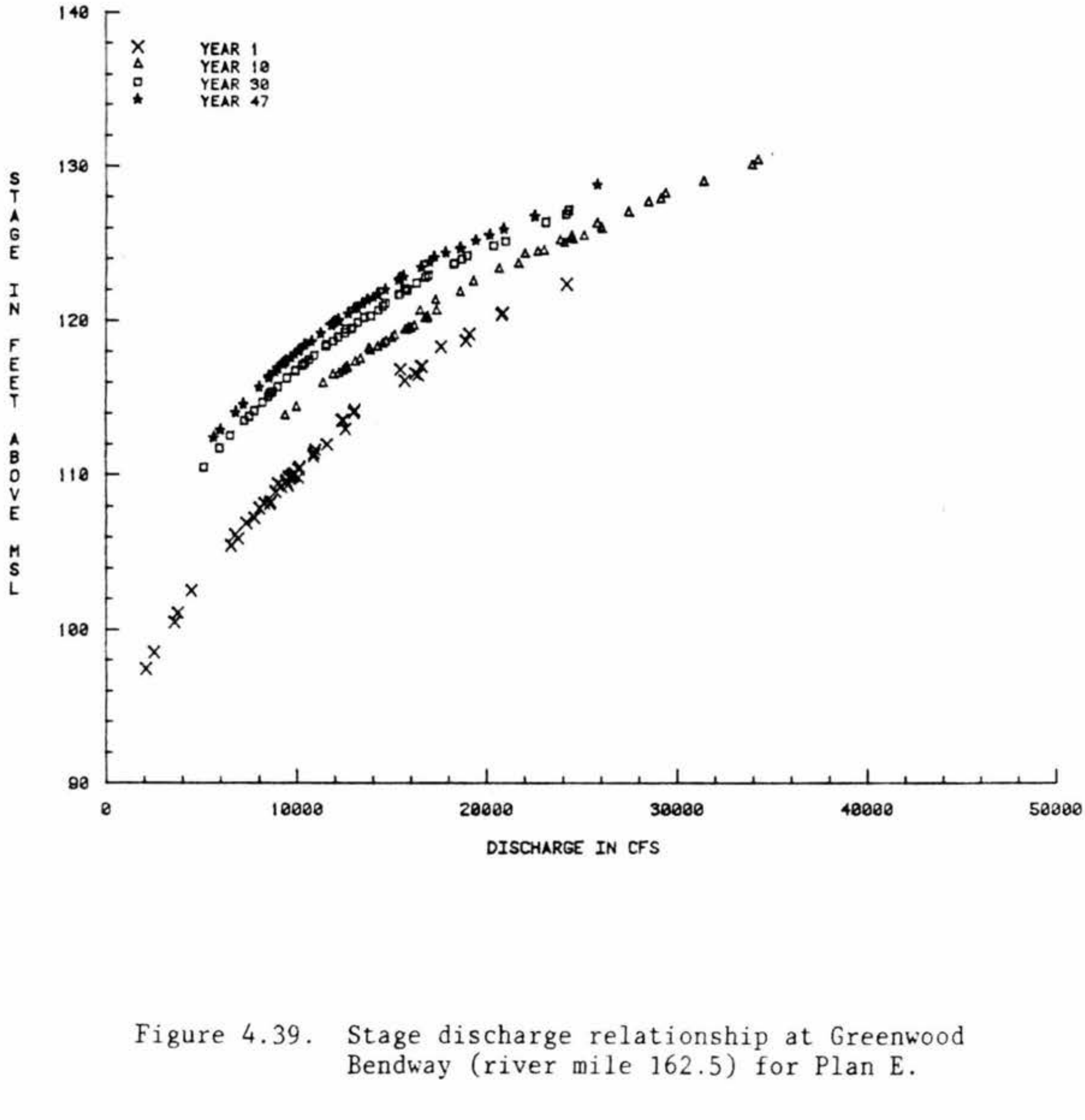

Figure 4.39. Stage discharge relationship at Greenwood Bendway (river mile 162.5) for Plan E.

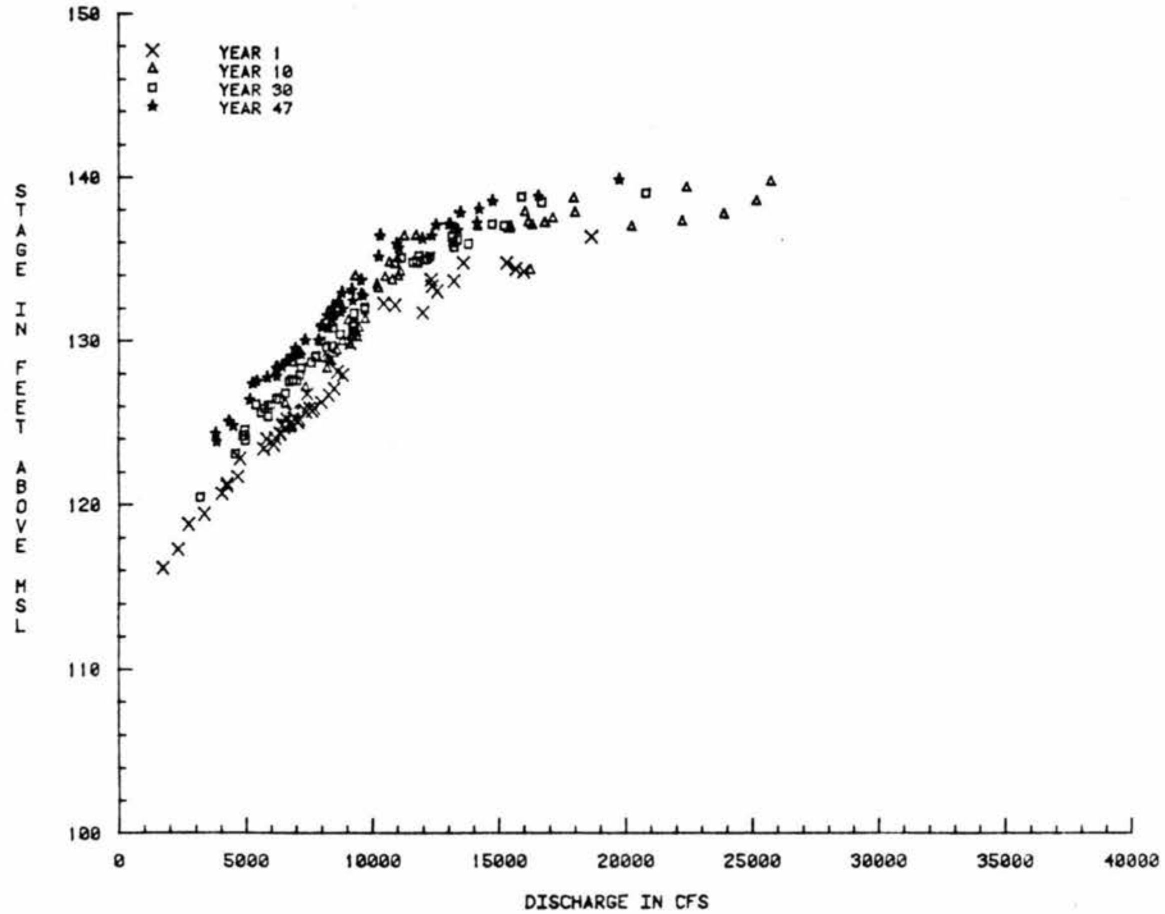

Figure 4.40. Stage discharge relationship at Swan Lake (river mile 219.08) for Plan E.

Conclusions from Calibration, Comparison and Application

From the calibration, comparison and application of the model KUWASER the following conclusions can be made. The known discharge, uncoupled sediment routing formulation is a viable method of modeling open channel flow in alluvial channels over great areas and long times. The application performed in the sedimentation study of the Yazoo River Basin (Simons et al., 1978) is the largest and longest application of a sediment routing model known.

Two features of the model may need additional work. First, the hydraulic property relationships for alpha, effective depth and effective width may be unnecessary. The value of alpha varied very little in the Yazoo cross sections. Since the hydraulic property relationship for alpha adds computations to the backwater calculations it may be better to use constant values. Two values of alpha would be needed at each section, one for main channel and one for overbank. The constant values could be reevaluated after distributing deposition. Effective depth and effective width are used as representative values of depth and width in the sediment transport calculations. In the review of the applications it became doubtful if these variables added any accuracy to the model. It is felt that top width and hydraulic depth (area divided by top width) could be used to calculate sediment transport which would achieve a reduction in computations. The second feature which may need additional work is the cross section distribution of erosion and deposition. While Figures 4.6 through 4.9 show the current method is adequate, it is felt that additional work is needed. One possible improvement would be a cross channel weighing function based on longitudinal curvature, channel shape and upstream conditions.

Chapter 5

SENSITIVITY ANALYSIS

General

A sensitivity analysis was performed on the model KUWASER. The analysis has two objectives. The first objective is to test the analytical sensitivity of the model results to changes in the input parameters. For example by evaluating channel sediment transport sensitivity it can be determined how accurate the sediment transport function must be to achieve any given level of accuracy in an output, such as volume of aggradation. The second objective is to provide users of this model or similar models with a design aid. For example, by comparing the effect on water surface profile and volume of aggradation by varying channel bottom width, side slope and thalwag slope the design engineer will be aided in determining which channel designs will meet required performance in flood control without causing excessive aggradation.

Procedure

Test Case

The sensitivity analysis was carried out by first selecting a "base" channel. The base channel is designed to be as simple as possible but to still retain similarity with channels that the model will be applied to. Figure 5.1 shows the base channel. The base channel is trapezoidal in shape with a 150 foot bottom, 3:1 side slopes, and 30 foot depth at overbank. The overbank area extends for

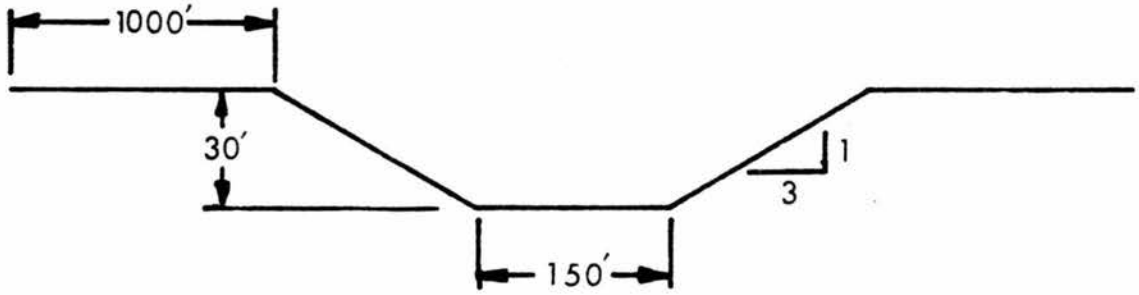

Figure 5.1. Sensitivity base channel.

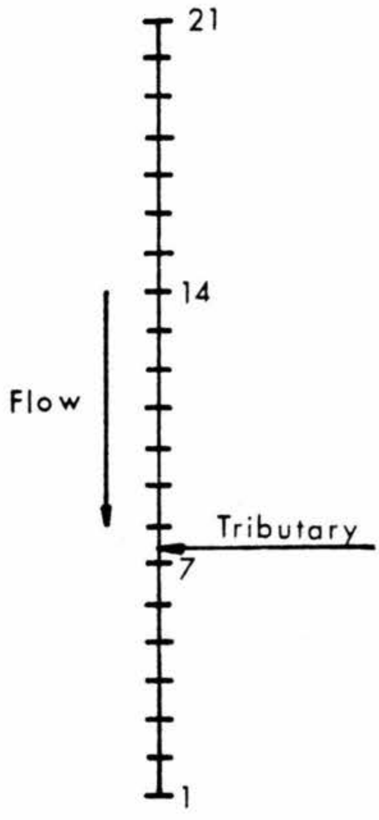

Figure 5.2. Sensitivity test reach.

1000 feet on either side. The Manning's n values are 0.030 for the main channel and 0.10 for the overbanks. The thalweg slope is 0.0075 percent (0.4 feet per mile). Figure 5.2 shows the test reach. The channel is 40 miles long with 21 cross sections spaced at two mile intervals. At river mile 13 between Sections 7 and 8 a point source tributary confluences. the initial downstream water surface is computed using a stage discharge relationship at Section 1 which maintains depth of flow greater than normal depth. This is similar to cases where a stream experiences a slight backwater effect from a downstream river or sea. The tributary has a very high sediment rating curve. The channel and tributary are not in balance. The tributary will cause severe aggradation in the channel under most circumstances. The channel is roughly similar to the Yazoo River between Belzoni and Greenwood, Mississippi, with the tributary being Abiaca Creek.

Discharge Sets

The model was run with three sets of discharges. These discharges are actual flows for the Yazoo River and Abiaca Creek. The flows are average weekly discharges computed from daily records. The first set is from 1964, a normal flow year. The second set is from 1973, a high flow year. The third set is the entire 10 year period 1964 through 1973. These three sets of discharges allow the evaluation of the model sensitivity over short term high and normal flows and a long term period.

Sensitivity Parameters

Six input parameters were selected for the sensitivity analysis. They are: (1) channel bottom width, (2) channel bottom slope, (3) channel side slope (4) Manning's n value, (5) tributary sediment input, and (6) channel sediment transport. The ranges given each parameter were chosen to reflect the normal variance of the parameter or the typical range a designer could utilize in this case. The values of each parameter were given are described below:

Bottom Width. Channel bottom width is the most important design parameter in river mechanics. The channel bottom width was given values of 100,125, 150 (base), 175, and 200 feet. This represents a range of -33 percent to +33 percent from the base.

Bottom Slope. The bottom (thalweg) slope was given values of 0.0050 percent, 0.0063 percent, 0.0075 percent (base), 0.0088 percent, and 0.0100 percent. This represents a range of -33 percent to +33 percent of the base value. The overbanks were kept constant and the main channel was rotated about river mile 20, Section 11. Thus, decreasing the slope raised the downstream end and lowered the upstream end, while increasing the slope lowered the downstream end and raised the upstream end. At Section 11 the bed elevation remained constant. The downstream stage-discharge relation was held constant also. This relatively complicated procedure was used since it more accurately models a man-induced river slope change such as cutoff construction or dredging.

Side Slope. The channel side slope was given values of 2:1, 3:1 (base), 4:1, and 5:1. This represents a range of -40 percent (5:1) to +50 percent (2:1) in the side slope.

Manning's n. Manning's n is both a design parameter which can be modified by channel snagging or flood plain management and an unknown variable which must be estimated. Manning's n value for the main channel was given values of 0.020, 0.025, 0.030 (base), 0.035 and

0.040. This represents a range of -33 percent to +33 percent of the base value. The overbank n values were not varied.

Tributary Sediment. Tributary sediment, like Manning's n is both a design parameter and an unknown variable. The tributary sediment was varied linearly by multiplying the base value for a given flow by a constant. The constant was given values of 0.0, 0.5, 1.0 (base), 1.5 and 2.0. This represents a range of -100 percent to +100 percent.

Channel Sediment. The channel sediment transport rate cannot be managed directly. It is the least well-known of all the input variables. The channel sediment transport rate was varied linearly by multiplying the normal value as computed by the transport function by a constant. The constant was given values of 0.5, 0.75, 1.00 (base), 1.25 and 1.50. This represents a range of -50 percent to +50 percent from the base.

Results

The effects of input parameter variance was evaluated for five values in the output. The five values are: (1) total volume of aggradation in the reach, (2) depth of flow at Section 7, (3) depth of flow at Section 14, (4) aggradation at Section 7, and (5) aggradation at Section 14. These five values were selected for their ease of evaluation and general importance. The total volume of aggradation is a overall measure of the effectiveness of sedimentation control alternatives. The depth of flow and aggradation at Section 7 are measures of the impact of the design parameters to the reach downstream of a tributary which generally flows at greater than normal depth. The depth of flow and aggradation at Section 14 are measures of the impact of the design parameters to an upstream reach which generally flows near normal depth. Before the quantitative results are presented it will be helpful to present a qualitative assessment based on the understanding of physical processor governing alluvial channel flow. The qualitative analysis will be helpful in explaining the quantitative results, as on first glance they may be confusing. Table 5.1 presents the qualitative sensitivity analysis for volume of aggradation. In the table two types of downstream control are listed, fixed stage-discharge relationship as used here and normal depth. For volume of aggradation, increasing bottom width increases aggradation for both control conditions, since the increased area will decrease sediment transport out of the reach. Increasing bottom slope will increase aggradation for the fixed stage-discharge case since increasing slope drops the bed at the downstream station while maintaining a constant stage. Thus it increases flow area, and reduces velocity and sediment transport out of the reach. For the normal depth control, increasing bottom slope will decrease aggradation since velocity, and thus sediment transport out of the reach will increase. Increasing channel side slope, making the banks steeper, will reduce flow area increase sediment transport out of the reach and reduce aggradation for both control cases. Increasing Manning's n for the fixed stage discharge case, will have no effect on sediment leaving the reach since it will not change the flow depth or velocity at the downstream section, but it will decrease sediment coming into the reach at the upstream reach because of the greater depth of flow and reduced velocity. Thus increasing Manning's n will decrease aggradation. For the normal depth case, increasing the n value will reduce sediment out of the reach and thus increase aggradation. Increasing tributary sediment will, of course, for both cases

Table 5.1. Qualitative Sensitivity Analysis

Downstream Control	Bottom Width	Bottom Slope	Side Slope	Manning's n	Tributary Sediment	Channel Sediment
Fixed Stage Discharge	+	+	-	-	+	+
Normal Depth	+	-	-	+	+	0

increase aggradation. Increasing channel sediment on the fixed stage discharge control will tend to increase aggradation. The downstream end is under backwater conditions with a low sediment transport while the upstream end is near normal depth with a high sediment transport. A straight percentage increase in transport will have the effect of causing a net increase of sediment input to the reach. For the normal depth case changes in channel sediment transport will have little effect on aggradation since both upstream and downstream sections are near normal depth and will have the equal transport rates. By similar analysis the sensitivity of other outputs can be determined. The following sections describe the results of the sensitivity analysis. A sensitivity analysis will vary from river to river and year to year. Thus, care must be taken in applying these results to other river environments.

Total Volume of Aggradation

Figures 5.3, 5.4 and 5.5 present the sensitivity of total volume of aggradation for short-term normal and high flows and long-term, respectively. For short-term, normal flow all parameters except Manning's n have positive slope. That is, an increase in the parameter increases aggradation. The tributary sediment transport rate is the

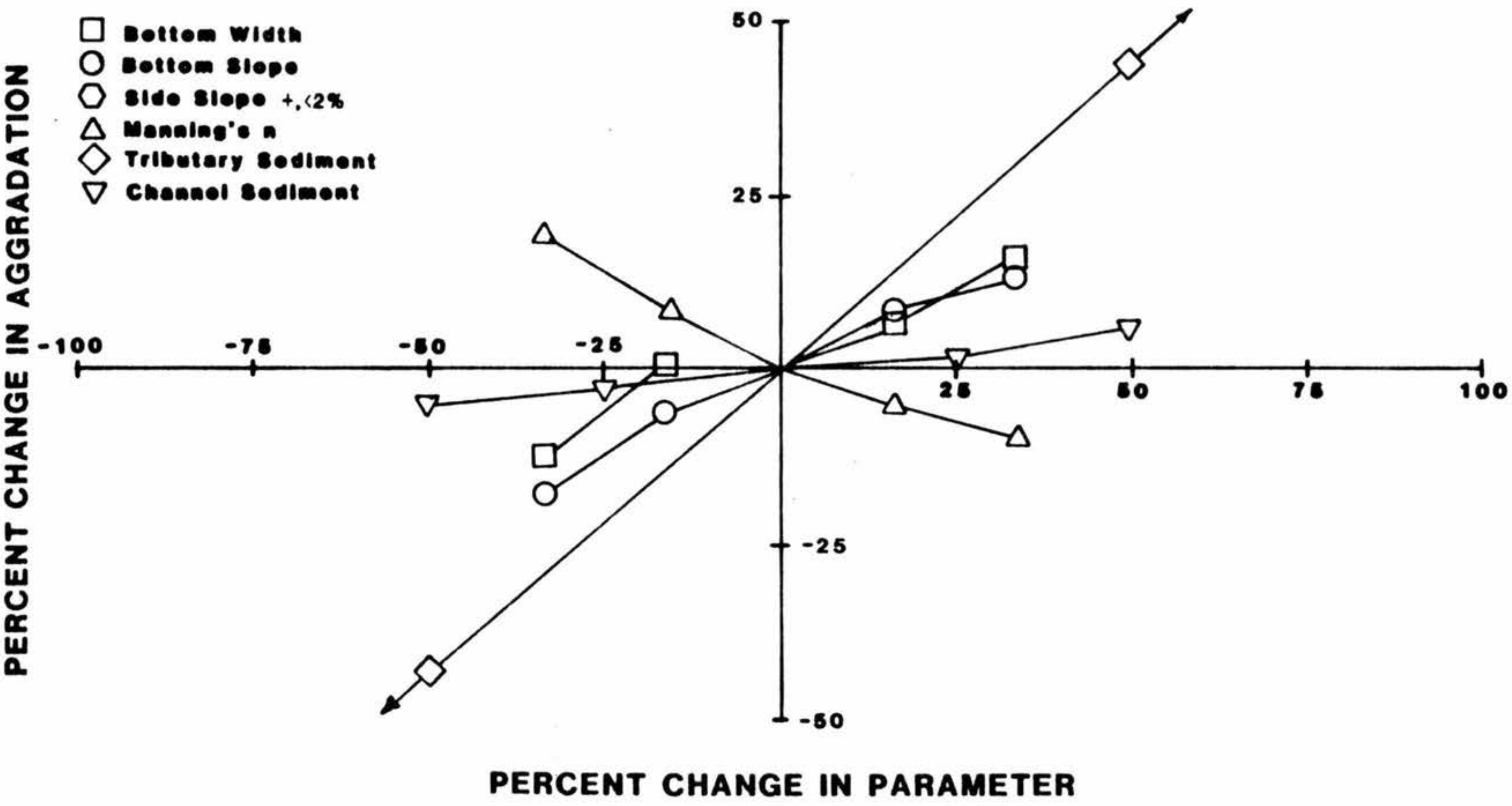

Figure 5.3. Sensitivity of volume of aggradation; short-term, normal flow.

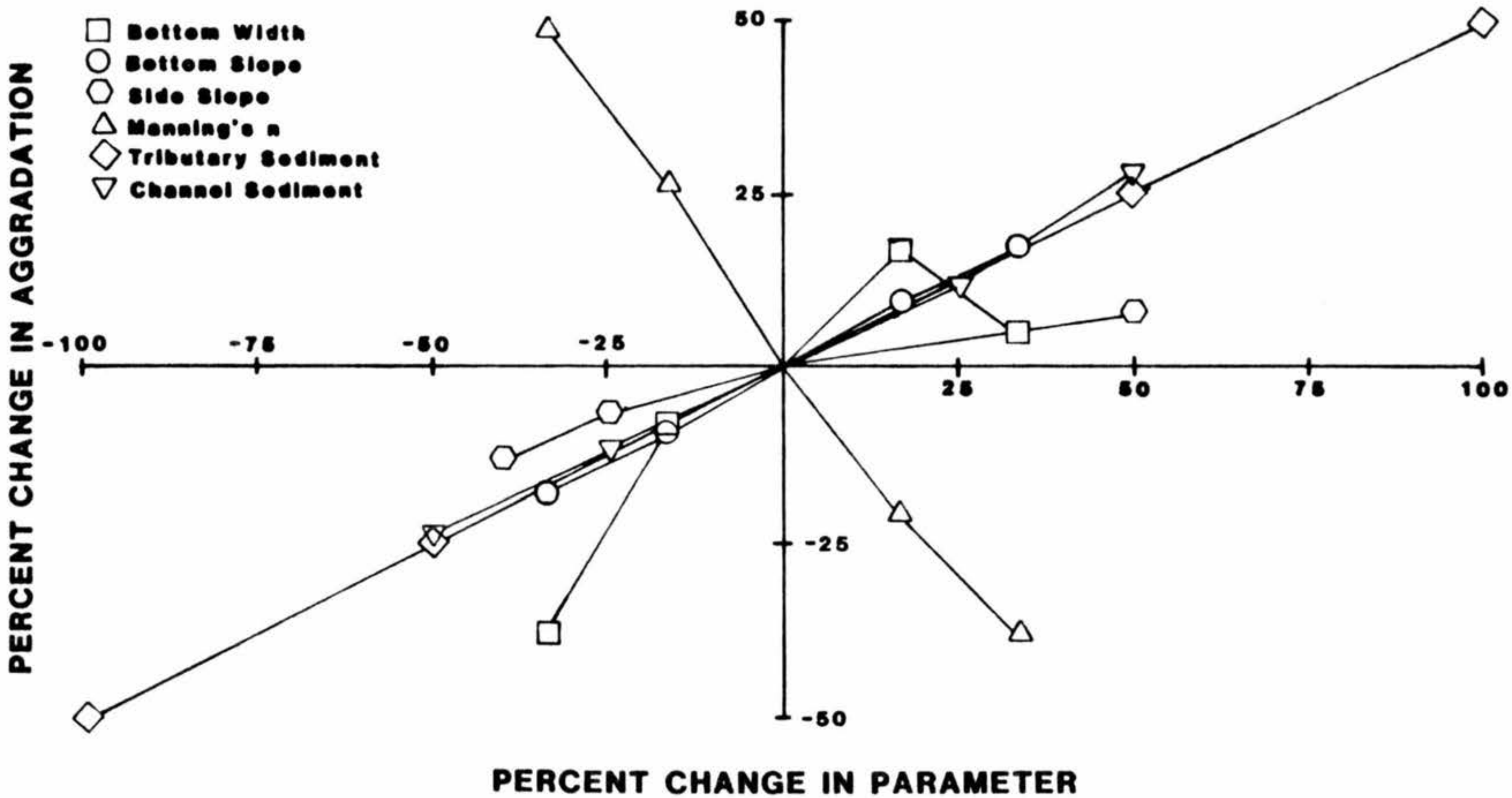

Figure 5.4. Sensitivity of volume of aggradation; short-term, high flow.

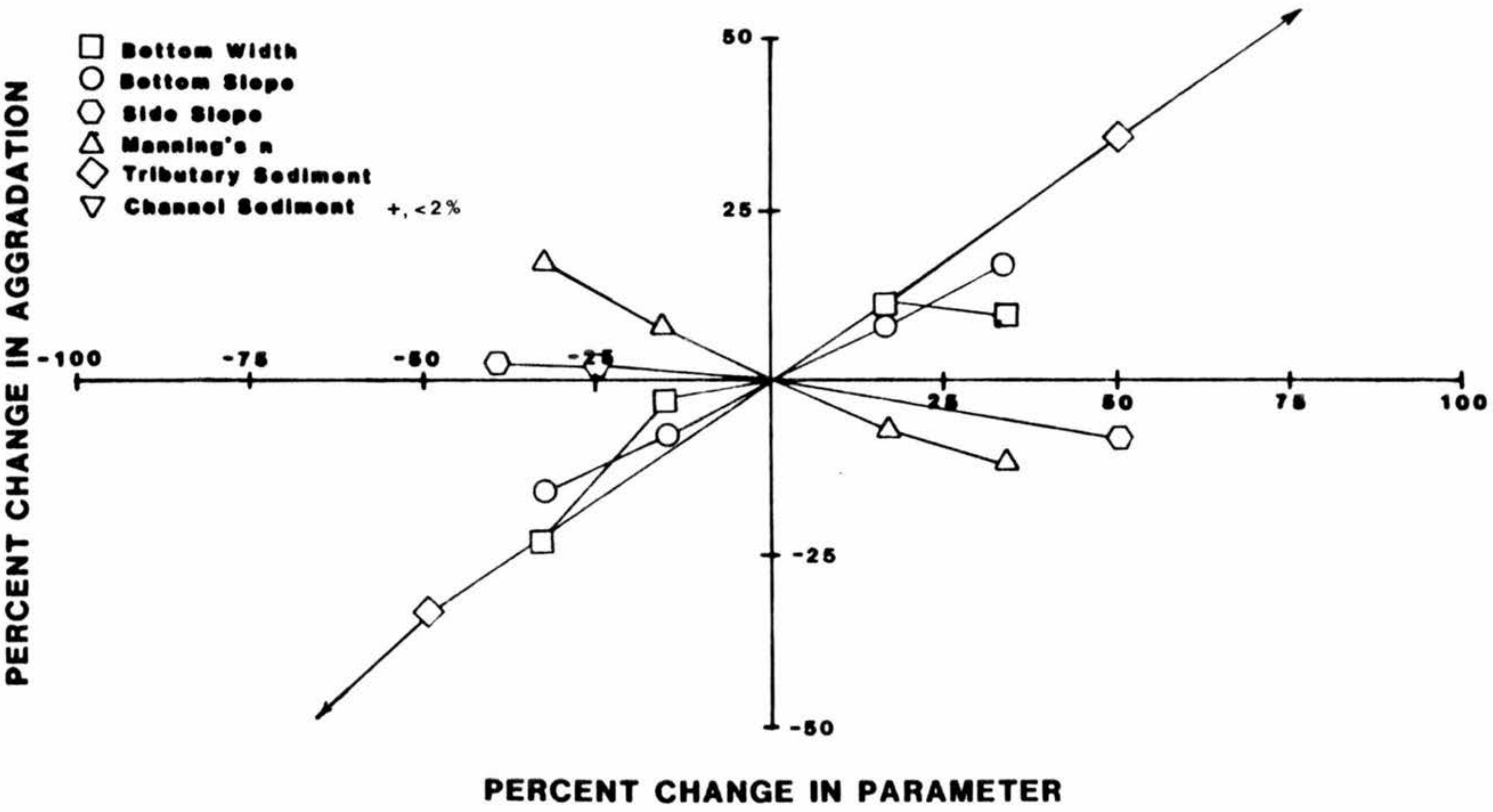

Figure 5.5. Sensitivity of volume of aggradation; long-term.

most dominant followed by bottom width, bottom slope, and Manning's n. Channel sediment transport has only slight impact while side slope has a negligible impact. All of the parameters show uniform, relatively symmetrical curves except for bottom width. Bottom width shows a step function on all curves. It is believed that the steps are caused by the large changes in cross section area which affect flow velocity, sediment transport and the frequency of overbank flow. For short-term, high flow the sensitivity to the parameters are similar except that the sensitivity to Manning's n increases dramically, as could be expected. In addition, sensitivity to side slope is no longer negligible, and sensitivity channel sediment increases. For the long-term period, tributary sediment is the most dominant parameter and channel sediment is the least. Channel side slope reverses but remains only slightly sensitive.

Depth of Flow at Section Seven

Figures 5.6, 5.7 and 5.8 present the sensitivity of the maximum depth of flow at Section 7 for short-term normal and high flows and long-term, respectively. For short-term, normal flow the sensitivity to Manning's n, side slope and tributary sediment have positive slope while bottom width sensitivity has a negative slope. Bottom width and Manning's n are the most dominant parameters while bottom slope and channel sediment are negligible. For short-term, high flow parameter sensitivity is similar to normal flow except tributary sediment sensitivity is negligible. For the long-term period all parameters are significant. While Manning's n and bottom width are still the most dominant, tributary sediment becomes more dominant than the remainder of the parameters.

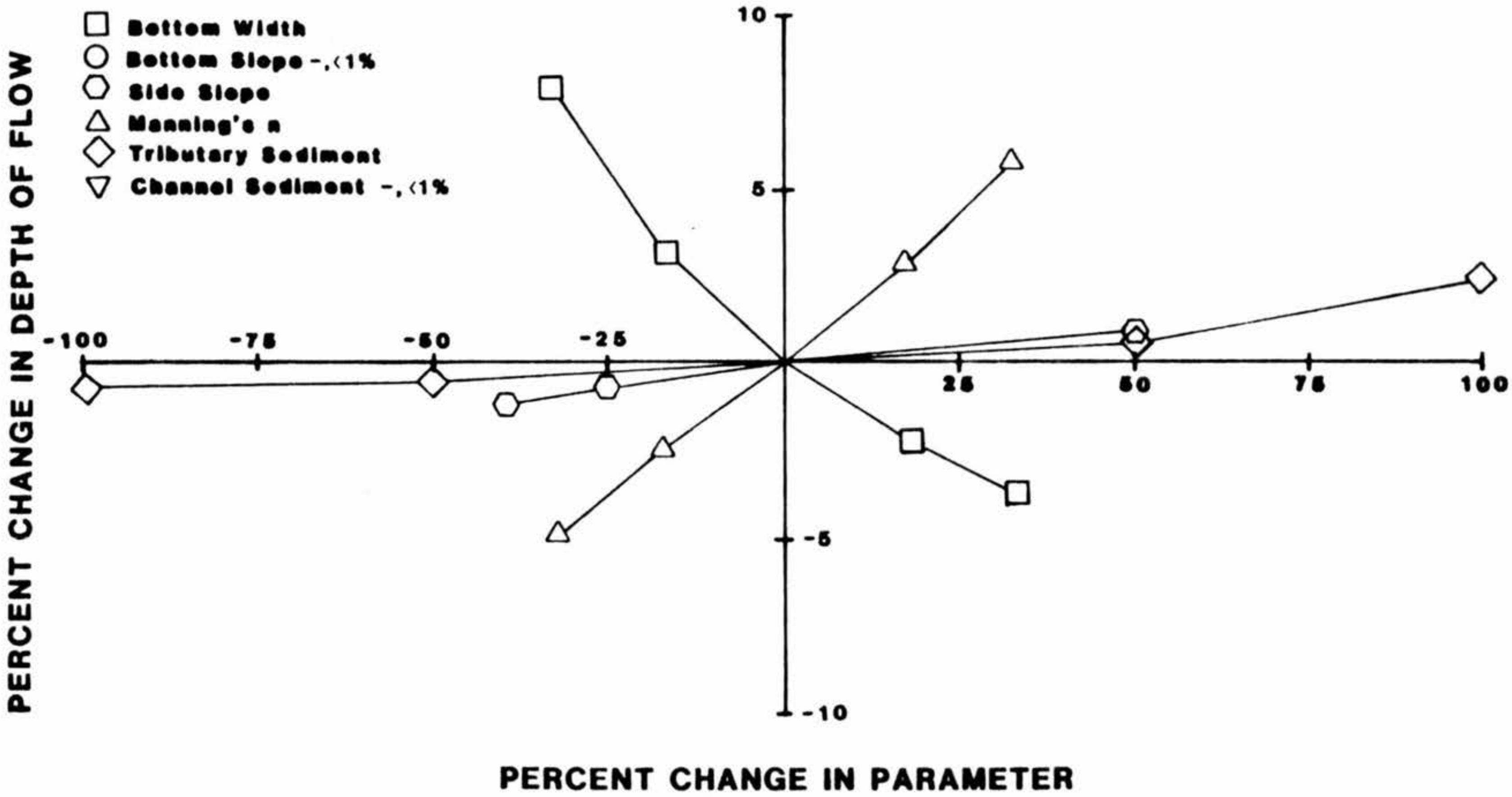

Figure 5.6. Sensitivity of depth of flow at Section 7; short-term, normal flow.

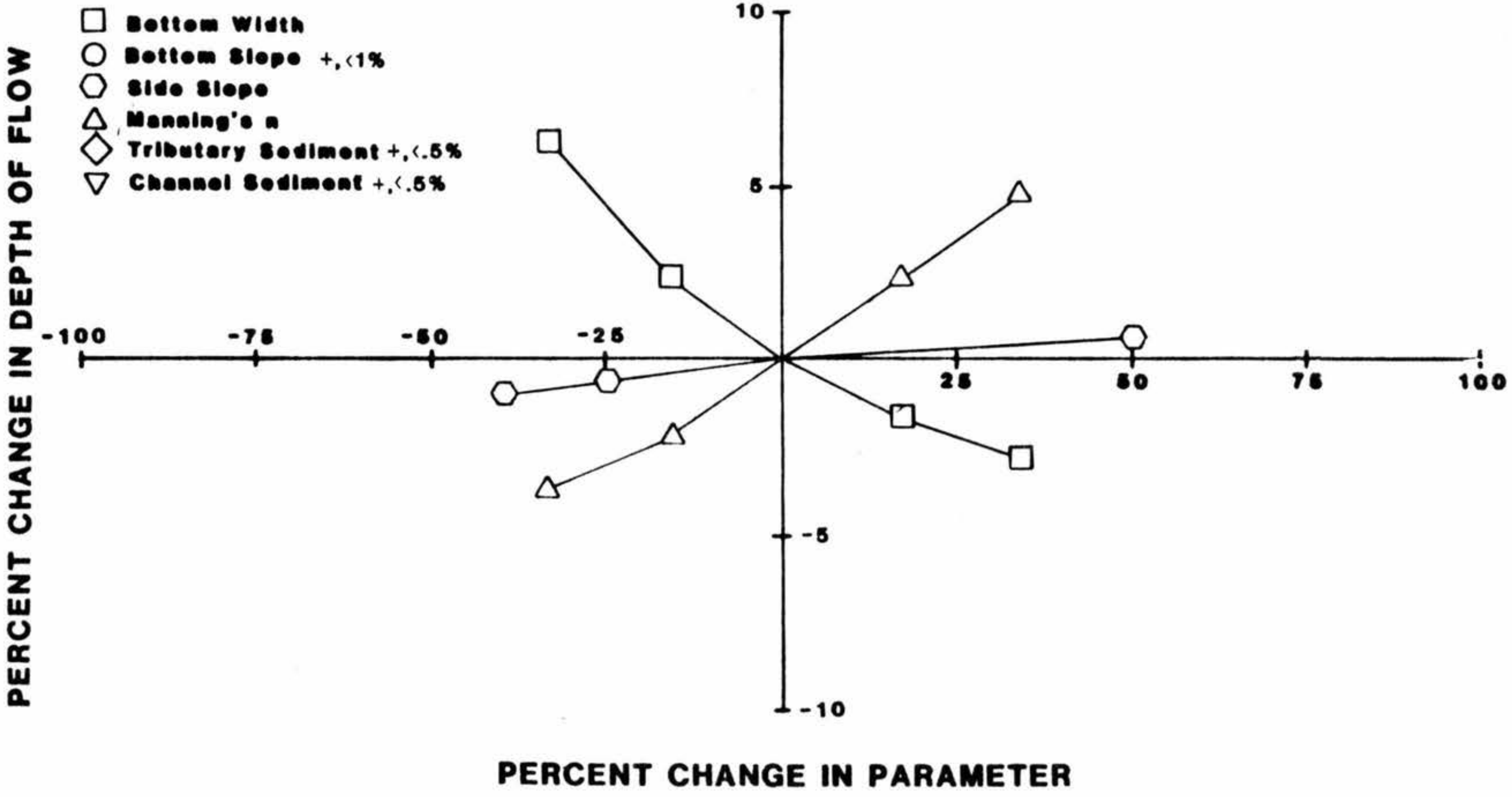

Figure 5.7. Sensitivity of depth of flow at Section 7, short-term, high flow.

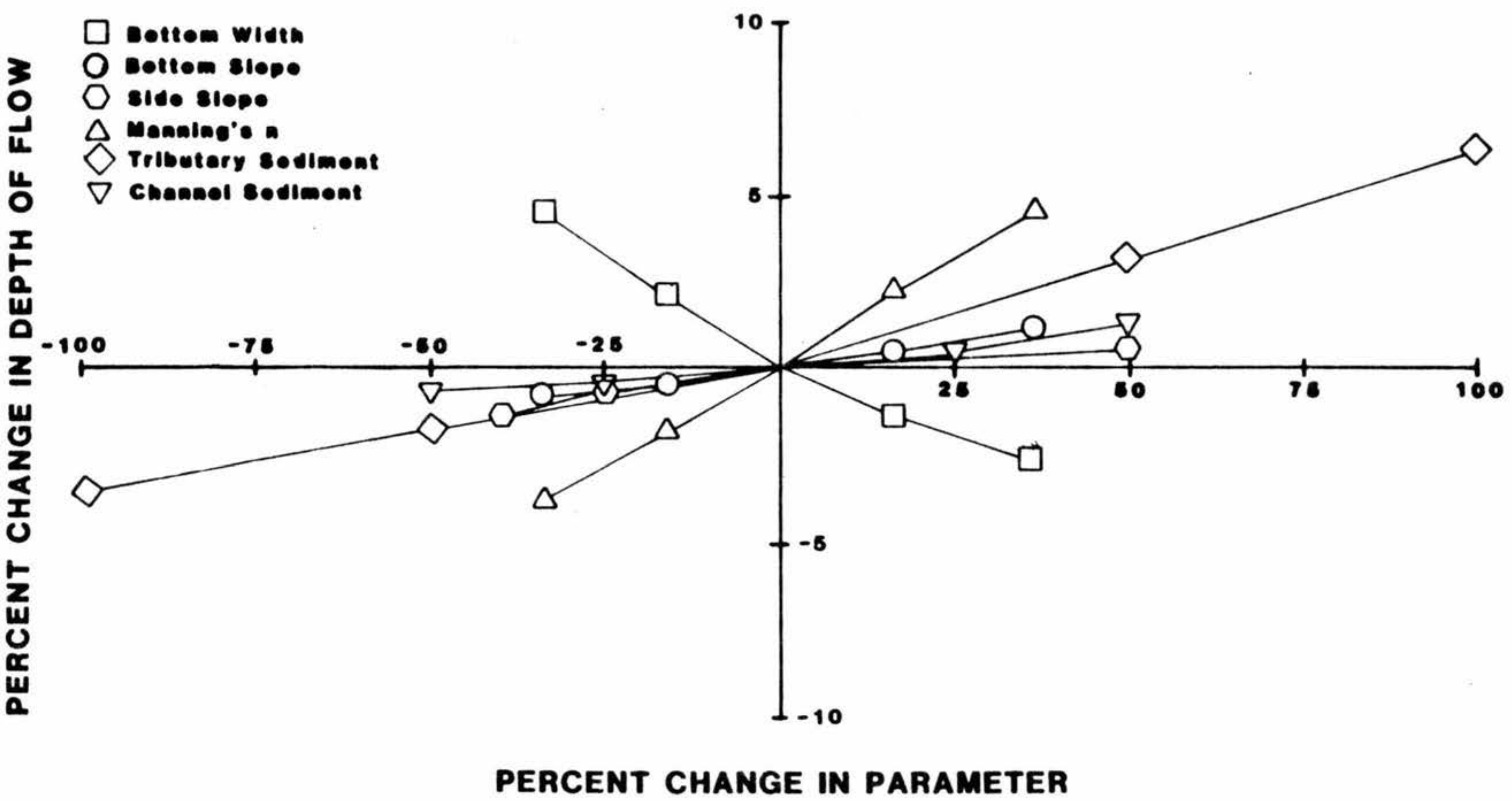

Figure 5.8. Sensitivity of depth of flow at Section 7; long-term.

Depth of Flow At Section Fourteen

Figures 5.9, 5.10 and 5.11 present the sensitivity of the maximum depth of flow at Section 14 for short-term normal, and high flow and long-term, respectively. For short term, normal flow the sensitivity to side slope, Manning's n and tributary sediment have positive slopes while the remainer have negative. Bottom width and Manning's n are the most dominant parameters while channel sediment has negligible effects. For short-term, high flow the parameter sensitivity is similar except that channel sediment reverses slope, but remains negligible and tributary sediment sensitivity becomes negligible. For the long-term period the parameter sensitivity is similar to short-term, high flow except tributary sediment sensitivity increases.

Aggradation at Section Seven

Figures 5.12, 5.13 and 5.14 present the sensitivity of aggradation at Section 7 for short-term normal and high flows and long-term, respectively. For short-term, normal flow all of the parameter sensitivities have positive slope. Tributary sediment has the greatest effect on aggradation while bottom slope and Manning's have negligible impact. For short-term, high flow the sensitivity to the parameters is similar except channel sediment reverses slope. For the long-term period significant changes in sensitivity occur. Bottom width creates the largest impact to aggradation. Side slope reverses its effects, though it remains small in magnitude. All parameters have significant impact.

Aggradation at Section Fourteen

Figure 5.15 presents the sensitivity of aggradation at Section 14. Analyses for the short-term discharge were not performed. No

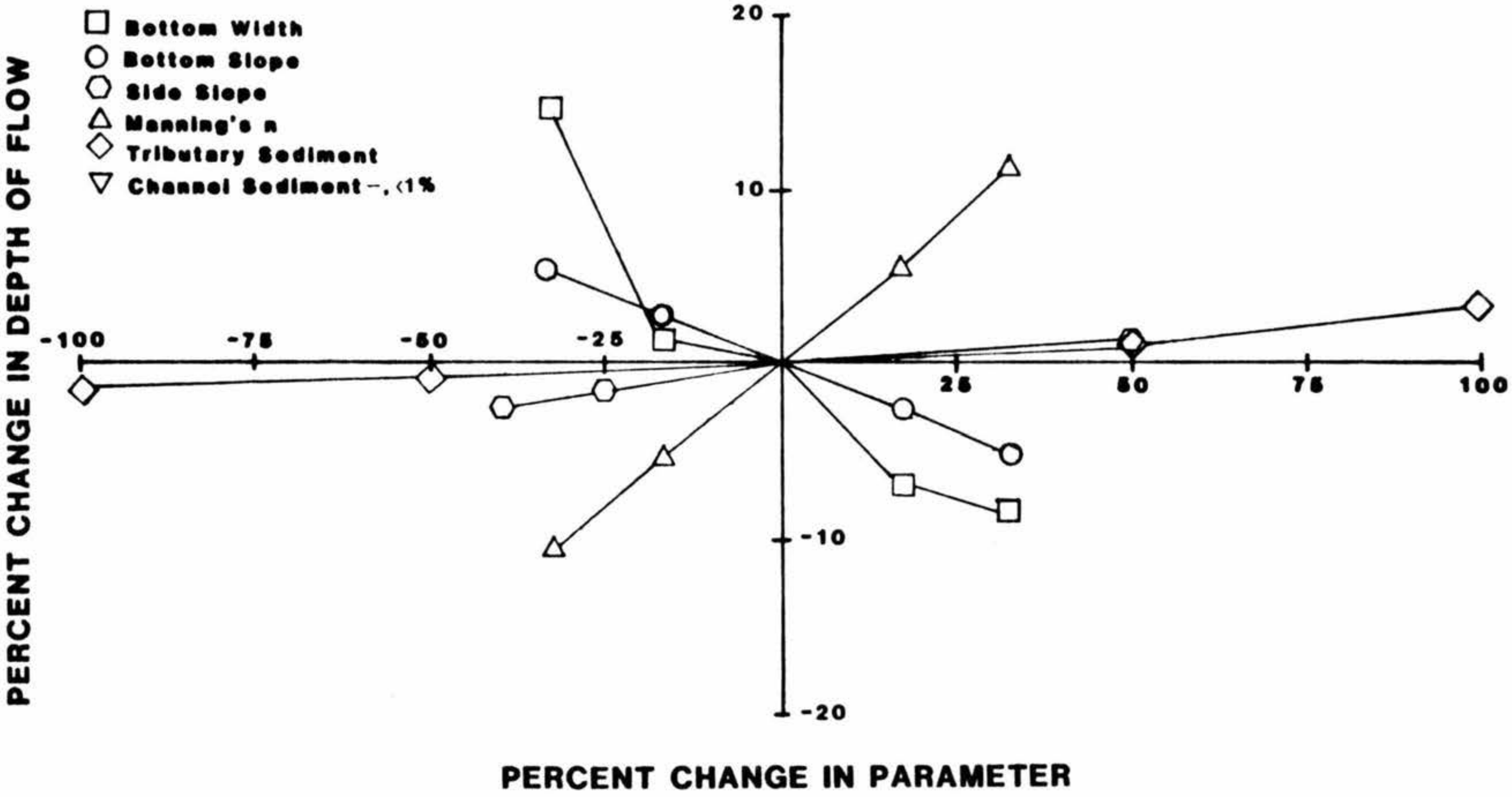

Figure 5.9. Sensitivity of depth of flow at Section 14; short-term, normal flow.

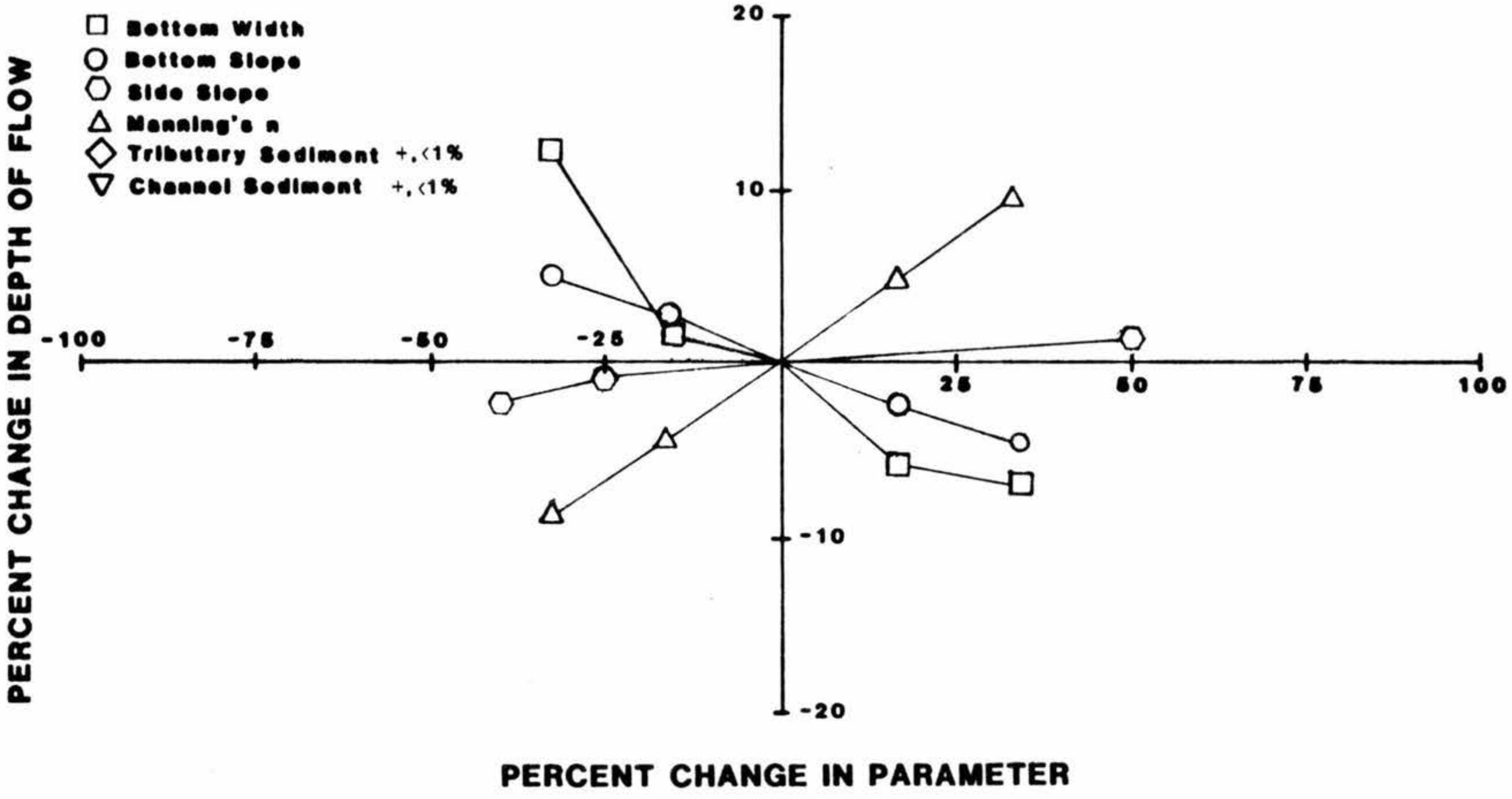

Figure 5.10. Sensitivity of depth of flow at Section 14; short-term, high flow.

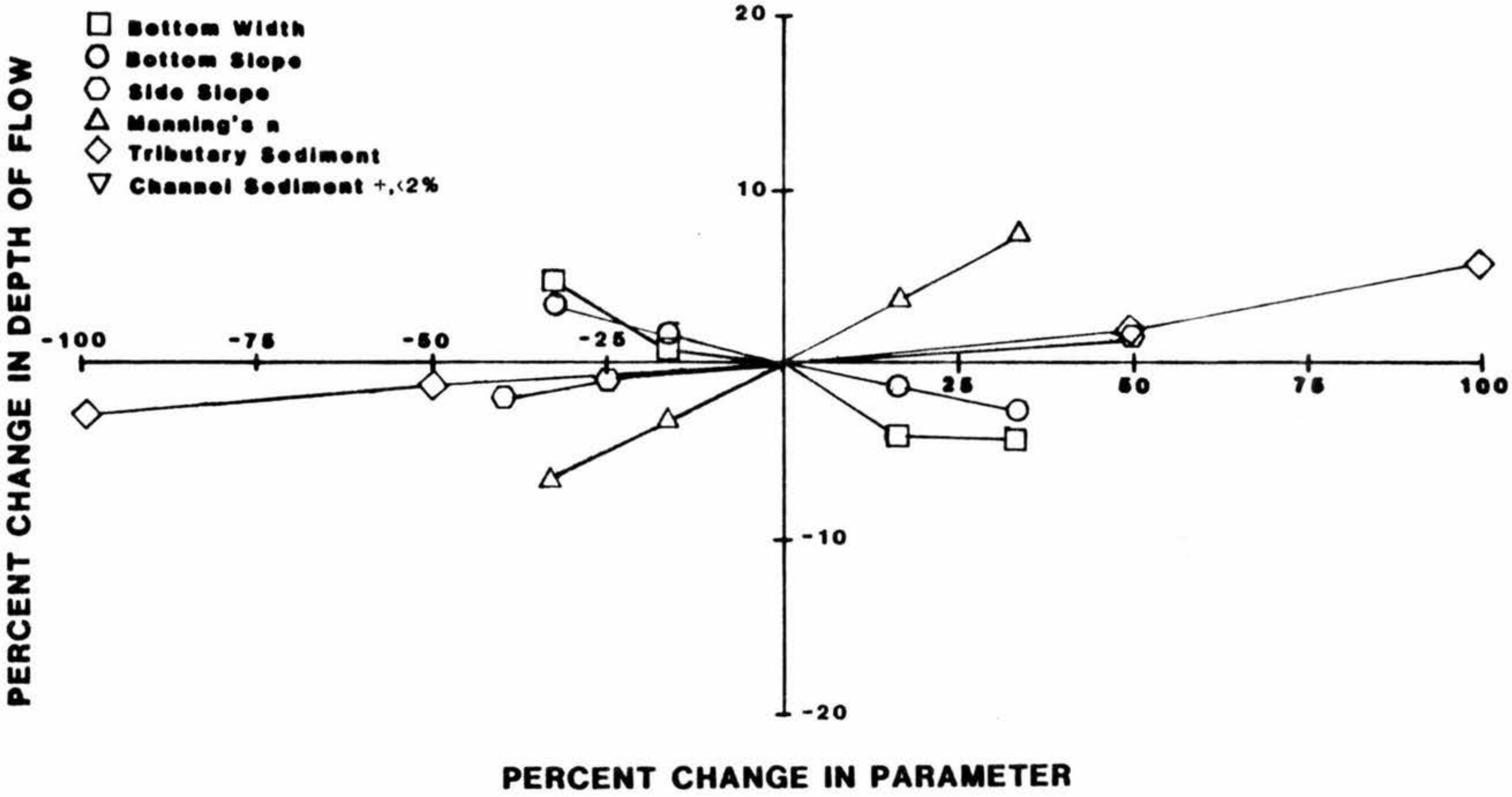

Figure 5.11. Sensitivity of depth of flow at Section 14; long-term.

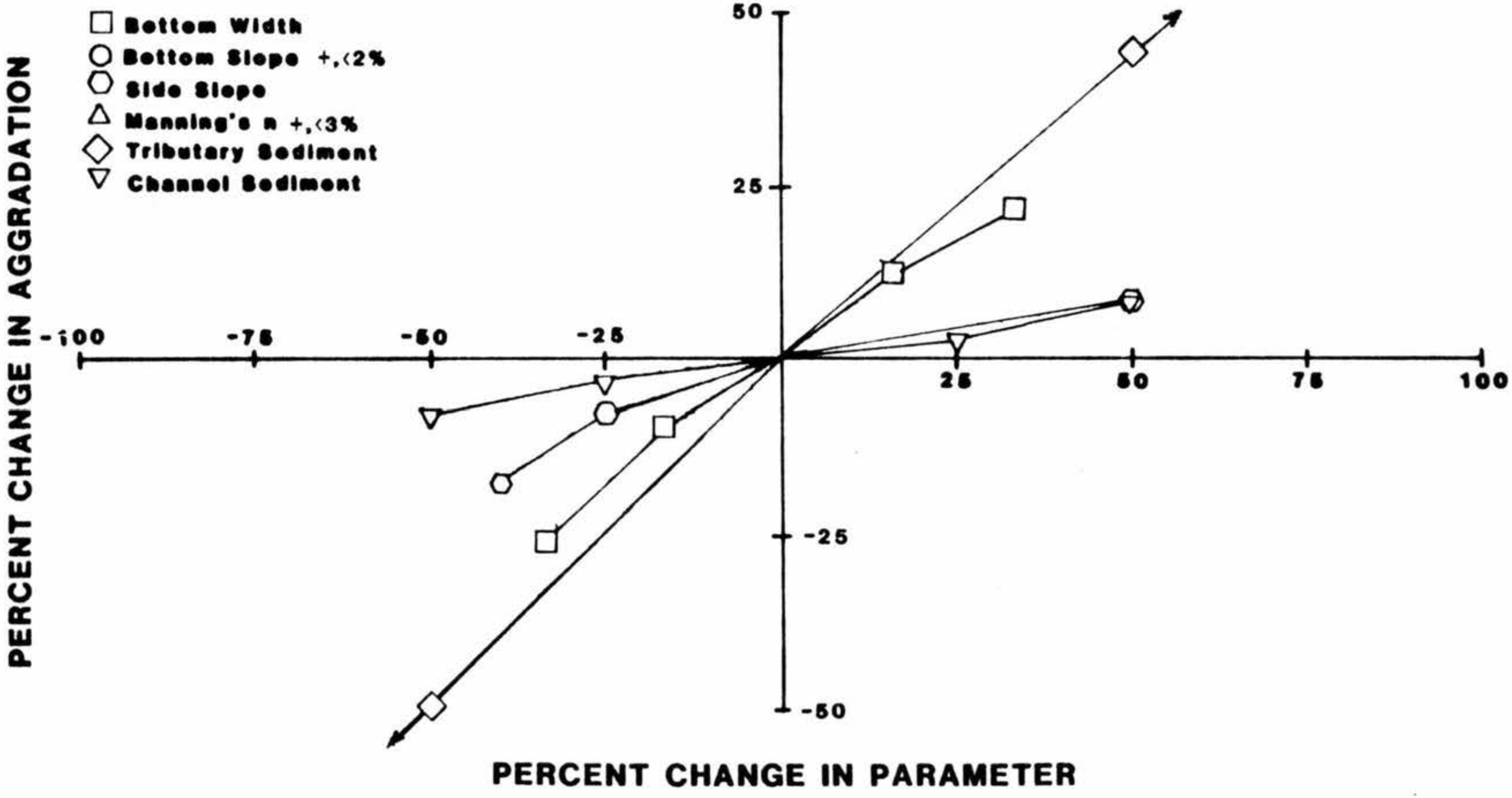

Figure 5.12. Sensitivity of aggradation at Section 7; short-term, normal flow.

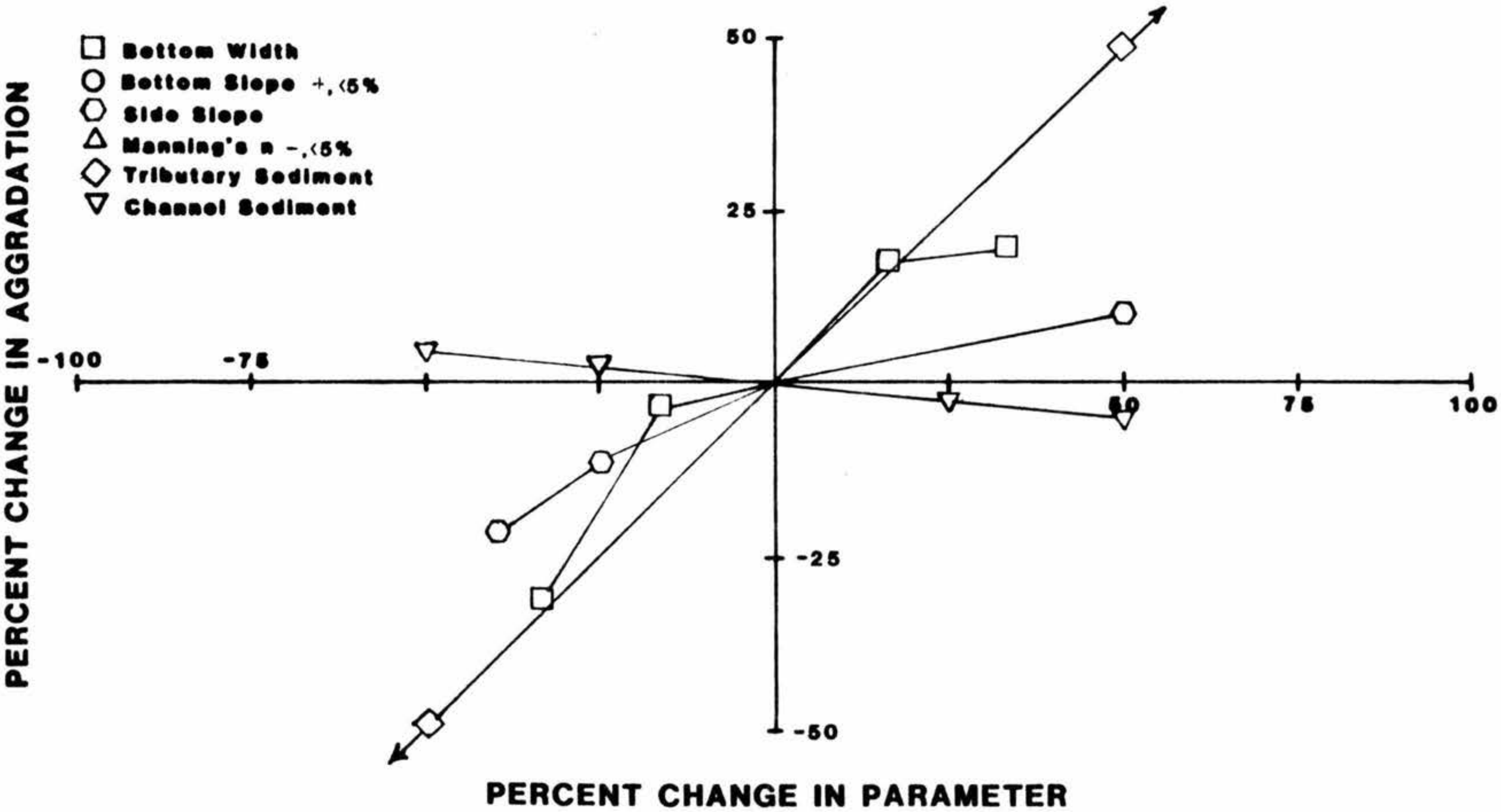

Figure 5.13. Sensitivity of aggradation at Section 7; short-term, high flow.

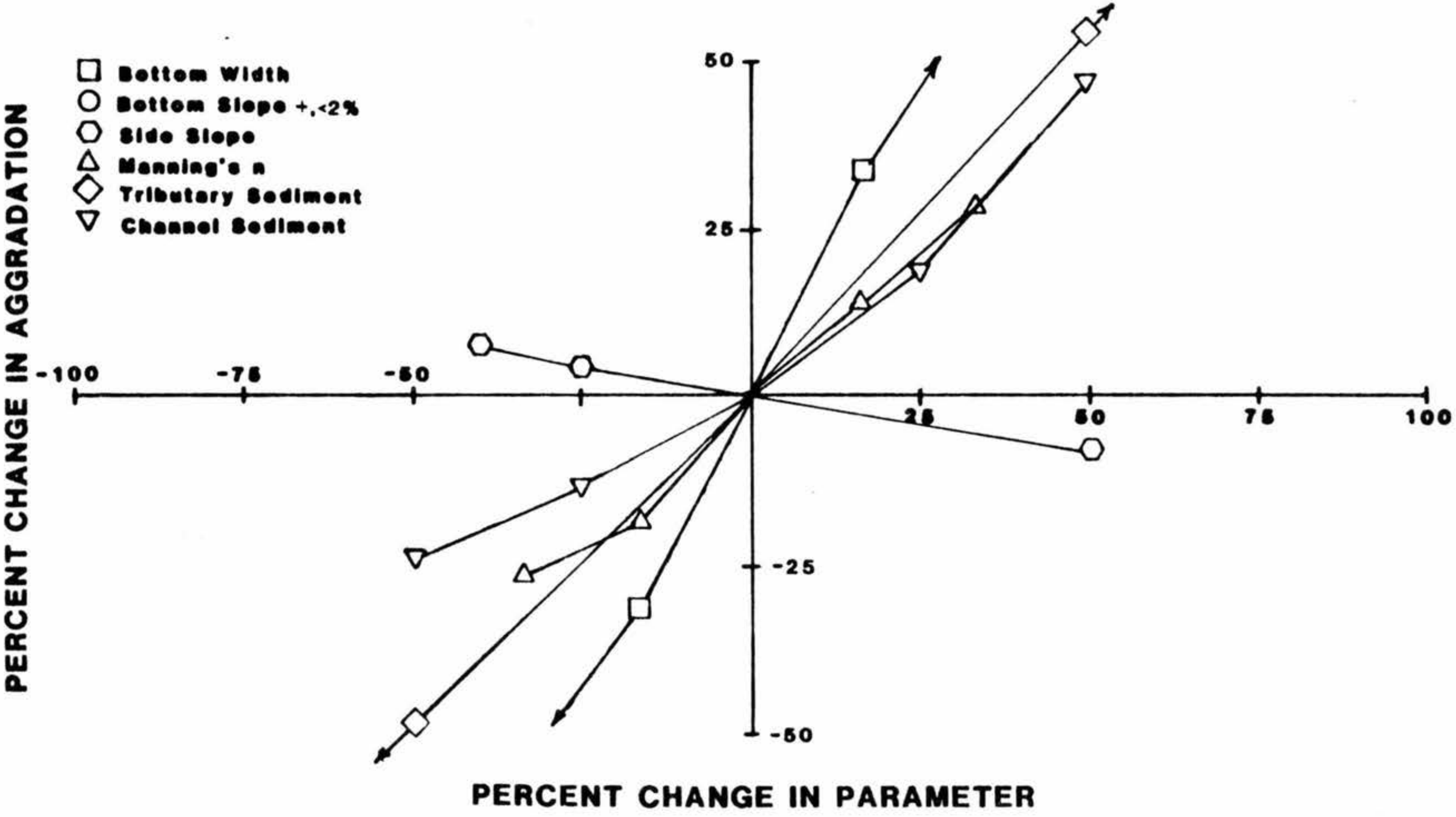

Figure 5.14. Sensitivity of aggradation at Section 7, long-term.

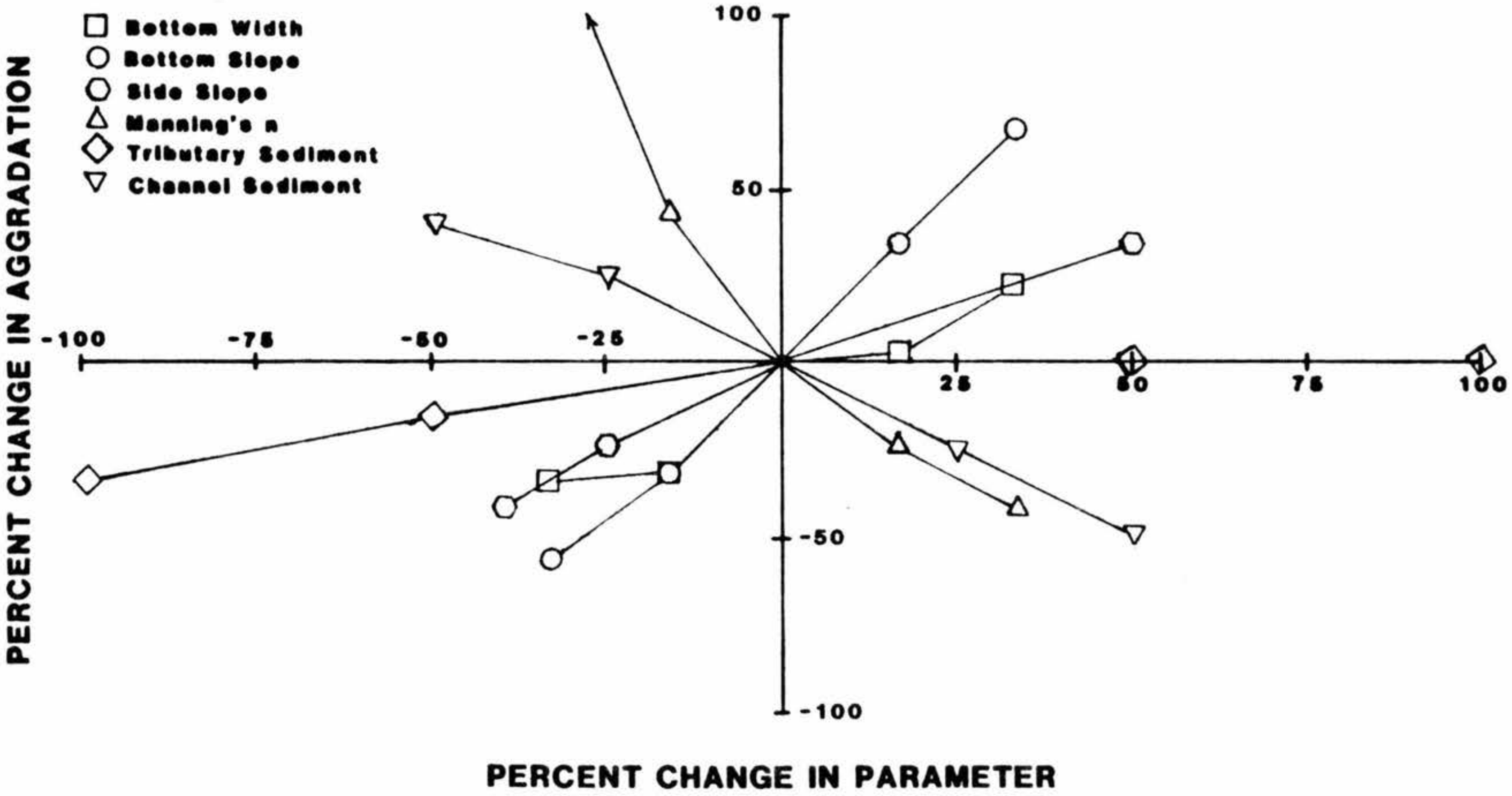

Figure 5.15. Sensitivity of aggradation at Section 14; long-term.

significant aggradation occurs at Section 14 during the one year periods. For the long-term period the sensitivity to Manning's n and channel sediment have negative slopes while the remainder are positive. All parameters are significant with bottom slope and Manning's n having the greatest importance.

Conclusions on Sensitivity

From the results of the sensitivity analysis the following conclusions are made concerning the model sensitivity to variations in the six input and design parameters. Since the results of a sensitivity analysis will vary for different channels and discharges, these conclusions may not be valid for other cases. But the conclusions reached here should provide a general guide for most river environments. The following sections list the conclusion drawn by parameter.

Bottom Width

Bottom width is a moderately to very dominant parameter. Increasing bottom width increases aggradation and decreases depth of flow at all locations. There is no significant variation in the sensitivity to bottom width with the different discharges. Bottom width exhibits a step function which is believed to be caused by the large changes in cross section area that affect flow velocity, sediment transport and the frequency of overbank flow. The steps offer an opportunity to optimize to a fine degree channel design.

Bottom Slope

Bottom slope ranges from a negligible to a very dominant parameter. Increasing bottom slope increases aggradation at the downstream end of the reach. This is due to the increased depth of flow and reduced velocity caused by dropping the thalweg while keeping a constant stage discharge relationship at the downstream control. The significance of bottom slope increase with discharge and time.

Side Slope

Side slope ranges from a negligible to a very dominant parameter. Generally, increasing side slope increases aggradation, but the changes is usually small. The principle exceptions are the long-term aggradation at Sections 7 and 14. At Section 7 the long-term aggradation was reduced slightly by increasing the side slope while at Section 14 side slope is the most significant parameter. As can be expected, increasing the side slope reduces the flow area and increases depth of flow in all cases.

Manning's n

Manning's n is the most dominant parameter for depth of flow. As can be expected, increasing Manning's n increases flow depth in all cases. Manning's n only moderately effects aggradation. It has only a slight to negligible impact on volume of aggradation. Its importance to aggradation increases with the discharge and time period. Increasing Manning's n increases downstream aggradation and decreases aggradation upstream.

Tributary Sediment

Overall, the parameter with the most effect is tributary sediment. Aggradation is most sensitive to it in all cases. The volume of aggradation and the depth of aggradation at Sections 7 and 14 increase with tributary sediment. Depth of flow at both sections also increase significantly with tributary sediment. The effects of tributary sediment are reduced by increases in main channel discharge, time of simulation and distance from the tributary confluence.

Channel Sediment

Overall, the parameter with the least effect is channel sediment. It has only negligible effect on depth of flow and negligible to slight effect on aggradation. The last point is most important, it shows that sediment transport while a very troublesome variable to define, need only be approximately near its true value to produce accurate results in simulation. The effect of channel sediment increase with discharge and time of simulation.

Chapter 6

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Summary

A known discharge, uncoupled, sediment routing model that sequentially solves the equations of sediment continuity and water motion has been developed. The equation of motion is solved by standard step backwater calculations. The backwater calculations have been improved by the addition of a Newton-Raphson method. The method uses channel hydraulic property relationships to improve the depth estimate. This method differs from other Newton-Raphson backwater algorithms in that the change in area and conveyance with depth are incorporated into the estimates. The model also allows for divided flow and flow over weirs. The equation of sediment continuity is solved by a standard, finite difference method. Sediment transport is calculated by a simple power function of velocity and depth. Eroded and deposited sediment are distributed across the cross section by a conveyance weighted method. The model is limited to subcritical flow and cannot predict channel armoring or two-dimensional flow effects. The known discharge, uncoupled formulation limits the model to cases where the change in the bed is small during any one time period, and the rate of change of the water hydrograph is small.

The model was tested against two other models using the frequency and magnitude of error in the prediction of stage at two locations.

The two comparison models were an empirical stage-discharge relationship and a steady, spatially varied flow, rigid boundary model. The comparison of the three models provided a verification of the model developed here and guidelines for application of each model. The known discharge, uncoupled, sediment routing model was applied to the Yazoo River Basin to determine the river response to existing conditions and a proposed channel improvement. The application cases covered over 300 miles of mainstem river and tributaries for a 50 year period. A sensitivity analysis was performed on the model. The sensitivity of five output values to changes in six input parameters was determined.

Conclusions

The known discharge, uncoupled, sediment routing formulation is a viable method of modeling open channel flow in alluvial channels. The uncoupling of the three basic equations of alluvial flow has allowed the development of a relatively simple model. While the model is theorically simple, several features have been incorporated that reduce computer time. The most important time savings features are the hydraulic power relationships, the Newton-Raphson backwater algorithm and the simple sediment transport function. The conveyance-weighted function for distributing erosion and deposition was adequate, but could be improved.

The model had the lowest frequency and magnitude of error in stage prediction of three models tested. The model developed here has much lower error than the empirical stage-discharge relationship for both stable and unstable reaches. The rigid boundary model performed about as well as the sediment routing model in the stable reach. In the unstable reach the sediment routing model which can predict and adapt itself to changes in river conditions related to sediment movement, is superior. The model was successful in its application to the Yazoo River Basin. The model predicted differences in river response between existing conditions and a proposed channel modifications. Because the model is a physical process program it can quantify to the extent of its accuracy, the river environment. This is an important feature. The engineer will not only know that one alternative will have greater channel aggradation than another, but also the volumes, timing and impact to the channel hydraulics at any given location.

The sensitivity analysis showed that tributary sediment has the greatest effect on channel aggradation. This implies that great care should be taken in the estimation of this parameter and that any mainstem sediment management program should look first at the tributaries. Aggradation and depth of flow was moderately to very sensitive to channel bottom width. Bottom width exhibits a step function which is believed to be caused by the frequency of overbank flow. The steps offer an opportunity to optimize channel design. The sensitivity of aggradation to channel bottom slope increases with discharge and time and increases in significance upstream. Increasing channel side slope decreases long-term aggradation and increase depth of flow. Manning's n was very significant to depth of flow, but only moderately effects aggradation. Thus, if only sediment information is needed Manning's n need only be approximated. Channel sediment transport had negligible to slight effect on depth of flow and aggradation. Therefore, the channel sediment transport function need only be somewhere in the same order of magnitude of its true value.

Recommendations

The following work is recommended to extend the applicability of the known discharge, uncoupled sediment routing model:

1. Upgrade the water surface profile algorithm to handle supercritical water surface profiles;
2. Expand the sediment routing routines to account for bed armoring, and control points; and
3. Integrate a storage routing model to supply the known discharge values.

The following work is recommended to improve and better define the performance of the model:

4. Explore the mechanism of lateral erosion and deposition to determine the significance of shear stress and conveyance on erosion and deposition;
5. Perform additional sensitivity analyses to better define factors, such as overbank flow and backwater that effect parameter sensitivity, particularly on bottom width.

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APPENDIX A

KUWASER USER MANUAL

A.1 General

This and the following appendices constitute an user's manual for the program KUWASER. This appendix describes the application theory of temporal and spatial design, program operation and an example application. Appendix B contains program flow charts, Appendix C contains variable definitions, and Appendix D contains a program listing.

A.2 Temporal and Spatial Designs

General

Spatial and temporal designs are necessary to provide a realistic representation of the space-time structure for the simulation model. Information on the river and its tributaries, their location, and the location of all pertinent gaging stations, structures and confluences allow the spatial design of a large river basin to be developed. Spatial designs should also consider the purposes of the study. Temporal design of a system is made using the historic hydrologic records of the watershed or river basin. The records should include water flows, river stages, sediment transport and effect of man's activities, such as reservoir construction, on the hydrologic record. Temporal designs must be compatible with the spatial design. Therefore, only those records pertinent to areas and river reaches included in the spatial design need to be analyzed.

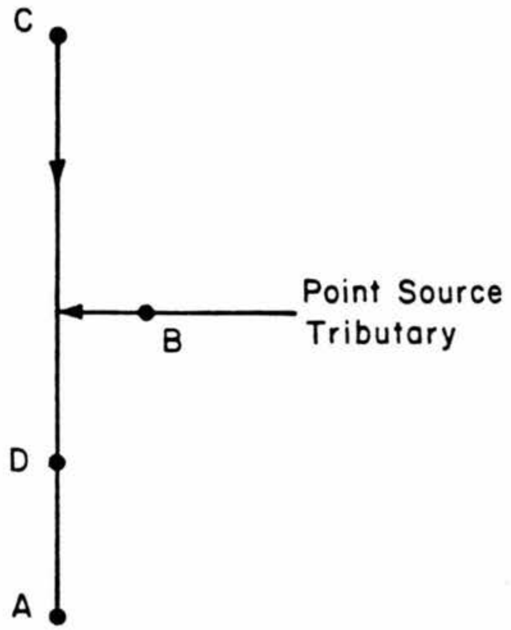


Figure A.1. A typical reach of river.

Temporal Design

General

Temporal design refers to the model's representation of changes in water and sediment input to the river system with time as well as the changes in water discharge and the sediment transport throughout the system. Temporal design should be as realistic as possible considering the system being modeled. Because river systems differ greatly in their temporal characteristics and data availability, KUWASER was designed to require that the user supply the water discharge at each cross section for each time period. While this requires additional user time and effort, it allows the user to more accurately model the system.

While the user may define water discharge in any way he chooses, three methods described here to illustrate temporal designs are, constant discharge, flow continuity, and unsteady flow routing.

The program also requires a reasonably accurate sediment rating curve for each point source tributary. Tributary sediment is determined by measurements or by an empirical method. In addition the relationship between the Manning n-value and the discharge is required. The following describes the three methods of computing water discharge, two methods for determining the tributary rating curve, and an explanation of the determination of the Manning n-value relationship.

Computation of Discharge

Constant Discharge

Figure A.1 shows a typical river reach. The reach extends from A to C and has a point source of water and sediment at B. When steady flow is assumed, the discharge is considered constant in a river reach

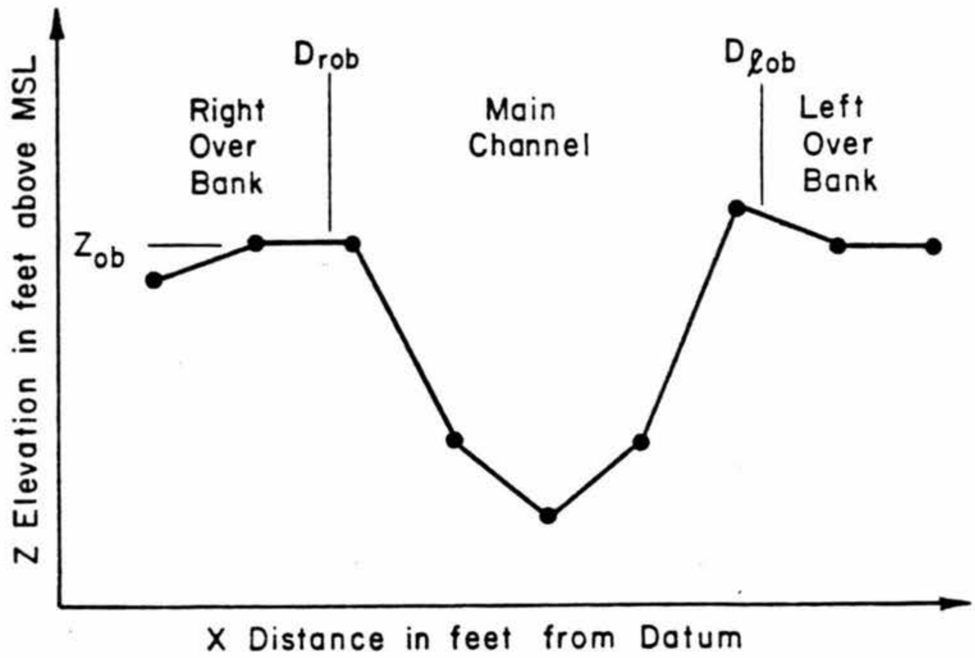


Figure A.2. Typical channel cross section with subdivisions.

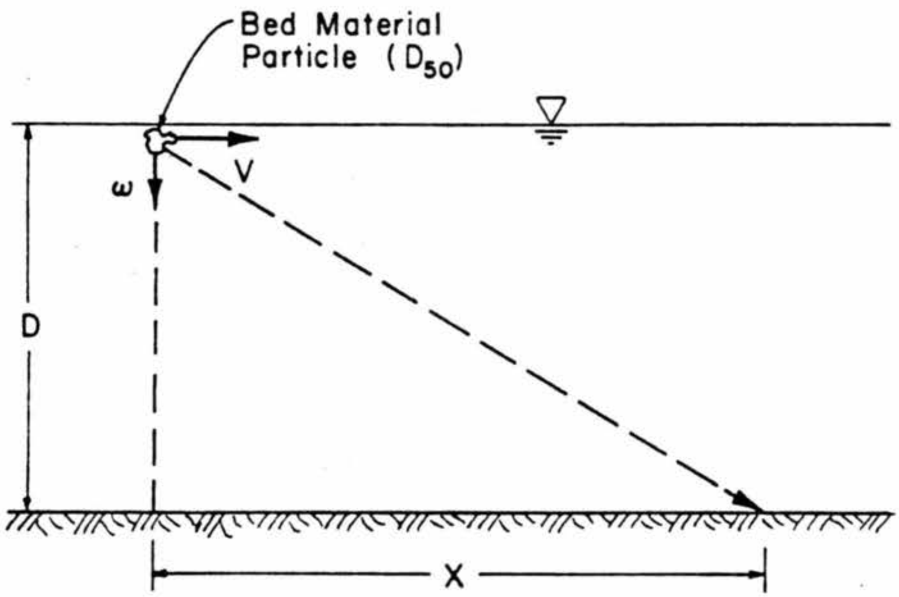


Figure A.3. Particle settling length.

except where lateral inflow occurs. In this example, if the discharge is known at two points, the discharge anywhere in the river can be computed. Thus, if Q_A, Q_B and Q_C are the discharges at the respective points and Q_A and Q_B are known, the discharge in the river from point A to B is equal to Q_A and the discharge from point B to C is equal to Q_C and is computed by:

$$Q_C = Q_A - Q_B \tag{A.1}$$

The flow at each cross section during each time period can thus be determined.

Flow Continuity

If the discharge is known at all three points in Figure 1 the system is over defined for the constant discharge assumption. Usually in a natural river $Q_C + Q_B \neq Q_A$. In these cases a non-point source, Q_{NPS} is defined as:

$$Q_{NPS} = Q_A - (Q_C + Q_B) \tag{A.2}$$

The sign of the non-point source is either positive or negative.

Non-point sources are usually distributed throughout the reach in a uniform manner based on the river distance. If X_A and X_C are the river distances of the respective points and X_D is the river distance of a point D, between points A and B, then the discharge at the point, Q_D is computed by:

$$Q_D = Q_B + Q_C + \frac{X_D - X_A}{X_C - X_A} \quad Q_{NPS} \tag{A.3}$$

This type of approach is used in the Sedimentation Study of the Yazoo River Basin (Simons, Li, Ward, and Duong, 1978), that also contains a detailed description of the temporal design used.

Unsteady Routing

Unsteady flow effects can be modeled if the known discharge model is coupled with an acceptable unsteady flow routing model. Acceptable unsteady models include but are not limited to simple storage routing models and kinematic wave models.

When coupling an unsteady flow model to the known discharge model, the user uses a separate program to generate the discharge at each cross section, for each time period. These discharges are then fed to KUWASER which computes the water surface profile for the particular time interval.

Tributary Sediment Rating Curves

Measured Curves

Each point source tributary requires a sediment rating curve of the form:

$$Q_{s\ell} = a_t \, Q_\ell^{\,b_t} \qquad (A.4)$$

where $Q_{s\ell}$ is the tributary bed material discharge, Q_ℓ is the tributary water discharge and a_t and b_t are the coefficients of the rating curve.

The best way to determine the rating curve for a stream is to take several measurements of the tributary water and sediment discharge, and then determine by least squares analysis the coefficients of the rating curve. When the rating curve is determined in this manner care must be taken to ensure only bed material is included in the sediment discharge and that measurements cover the full range of tributary flows.

Synthetic Curves

When it is not possible to obtain tributary sediment measurements the rating curve for a particular stream can be determined by a theoretical method using the tributary's cross-sectional shape, thalweg slope, bed material size and estimated Manning's n value. There are five basic steps to the process. First the range of the tributaries' water discharge is determined and at least ten flow levels over the whole range are selected. Second, Manning's equation is applied using the channel shape and slope, to determine the depth, width, and velocity of flow for each flow level. Third, using a sediment transport equation such as Einstein's or a combination Meyer-Peter, Müller's and Einstein's the bed material transport is determined for each flow level. Fourth, a curve is fitted either by hand or least squares regression, to the computed sediment and water discharge values to obtain the coefficients a_t and b_t. Finally, the coefficient a_t is calibrated by running the model and observing the short term change in the mainstem bed elevation near the tributary. If the bed degrades then the coefficient a_t is probably too low. However, if the bed aggrades a_t should be decreased.

Manning's n-Value Rating Curve

Manning's n-value for an alluvial stream is not constant but is a function of discharge and depth of flow (Simons and Sentürk, 1977). In the program Manning's n-value is made a simple function of discharge:

$$n = n_o a_n Q^{b_n} \tag{A.5}$$

where n is the actual Manning's n-value, n_o is the initial value of Manning's n that is input with the cross sections, Q is the discharge and a_n and b_n are the coefficients of the relationship. The values of the coefficients are a function of the stream's hydraulics and range of discharge.

To determine the values of a_n and b_n first estimate the n-value for a high and low flow discharge. With the initial n-value, by solving simultaneous equations, the values of a_n and b_n can be determined. The program can then be run for several discharge levels that have known water surface profiles and the error for each discharge determined. New values of a_n and b_n can then be computed which reduce the error in the water surface profile through the whole range of flows. The new value of the coefficients may be either estimated or calculated by least squares analysis. The process is then repeated until no reduction in error is obtained. It should be remembered that from physical significance the coefficient a_n will be a small positive number and the coefficient b_n should be negative.

Spatial Design

General

Spatial design refers to the model's representation of the physical characteristics of the river system. It includes relative information on the location of the various river reaches and tributaries, as well as data on channel properties.

Data required for the spatial design are:

1. digitized channel cross sections with over bank stations and Manning's n values;
2. division of river system into reaches;
3. river distance between cross sections;
4. tributary locations; and
5. locations of any structures.

In addition historical cross section measurements are necessary to calibrate the model.

The following describes considerations to be made in data development of spatial designs.

River Reaches

For program operation a river system is divided into reaches. A river reach is used as a basic computational unit, and as such should represent a single channel with the following hydraulic and sediment properties almost constant:

1. sediment transport,
2. cross section size,
3. channel roughness, and
4. discharge.

The necessary reach divisions required for program operation are described in Section A.4.

Tributaries

Tributaries supply water and sediment input to the mainstem river. The program allows for four different types of tributaries

1. point source in,
2. major tributary in,
3. point source out, and
4. major tributary out.

Point source tributaries are tributaries for which no backwater or sediment routing calculations are made. The water discharge (either in or out) is read, and the tributary sediment is computed using a rating curve.

Major tributaries are tributaries to the mainstem that are separate river reaches on which backwater and sediment transport calculations are conducted. There is no limit on the level of tributaries that can be modeled. Therefore the mainstem may have major tributaries, that in turn have major tributaries, and so on.

For divided flow sediment routing, the model assumes that two reaches act as tributaries to one another. The secondary reach will act as a major tributary out of the primary reach at the top, and as a major tributary into the primary reach at the bottom. Likewise the primary reach will act as a major tributary into and out of the secondary reach.

The discharge for each point source tributary is read with the cross section discharges. Point source discharge locations are termed discharge sections and are defined by the user. Discharge sections do not have digitized cross sections associated with them, only a discharge value in the flow array.

Cross Sections

Geometry

Channel cross sections are defined by (x,z) sets of coordinates. Figure A.2 shows a typical cross section. To allow for different Manning's n-values across the section three subdivisions are made: Right Over Bank, Main Channel, and Left Over Bank. Subsections are divided by two stations D_{lob} and D_{rob}, as shown in Figure A.2. The Manning's n-value at each coordinate point is determined by its location in either the over banks or main channel.

Hydraulic property relationships are computed to relate area of flow, conveyance, alpha, effective depth, and effective width to the thalweg depth. Two separate sets are calculated: one set for main channel flow and a second for overbank flow. The division between main channel and overbank flow is the user defined elevation Z_{ob}. If the overbank elevation is different for the left and right banks the lower of the bank elevations should be used as Z_{ob}. If the water surface is above a coordinate end point (first or last points), the area of flow is determined by extending a vertical line to the water surface from the end point.

Cross Section Spacing

To help maintain numerical stability in the sediment routing, channel cross sections should be evenly spaced. Also it should be remembered that the present model cannot simulate differences among actual and potential sediment transport (calculated by Equation A.6) as well as sediment dispersion and other processes particular to a small simulation space interval. Therefore the minimum cross section spacing should be based on the river hydraulics and bed material size. Dispersion effects are not usually significant when cross section spacing exceeds the average downstream distance when bed material particles settle if released from the water surface. Generally, the following procedure can be used.

Figure A.3 shows the principle used in estimating particle fall distance. First, estimate the average depth and flow velocity. Second, determine fall velocity for the d_{50} bed material particle size (the particle size for which 50% of the sediment mixture is finer). Sediment fall velocity can be determined by several methods (Simons and Suntürk, 1977) but for this purpose Rubey's formula for particles less than 1 mm in size is adequate.

$$w = \frac{\sqrt{\frac{2}{3}\, g(S_s - 1)\, d_{50}^3 + 36\nu^2} \quad -6\nu}{d_{50}} \tag{A.6}$$

where w is the fall velocity, g is the exceleration of gravity, S_s is the sediment specific weight (2.65 for quartz sand), and ν is the kinematic viscosity of water. Particle settling length X, is then computed by:

$$X = \frac{D}{w}\, V \tag{A.7}$$

The above procedure is only general and as such tighter spacing may be used with discretion. There is no numerical upper limit on spacing but for experience, it is recommended that maximum spacing does not exceed 10 X. Cross sections should also be located in areas of interest, control points, and at locations of sudden water surface profile changes.

Weirs

Weirs are represented in the model by a double cross section in a reach with the same river mile. The program computes the water surface elevation at the weir by both backwater and broad crested weir formula, and uses the greater of the two.

The amount of sediment over the weir is computed as a percentage of the sediment transport at the next upstream cross section.

A.3 Input Data

General

This section lists important input variables by the data type along with suggested values, while the next section defines structure and format of actual input files. The suggested values for the input variables are presented to help the first time user in operating the program. With experience, the user should be able to determine the best values of the input for his problem.

The program can operate in either English or Metric (SI) unit systems. When using the English system dimensional input variables should be in feet and seconds, except for river distances of cross sections and tributaries that are in miles and time period lengths that are in days. When using the SI system input variables are in meters and seconds, with the exception of river distances that are in kilometers, and time period lengths that are in days.

General Data

The following variable must be defined for each run.

TITLE	The job title
IPRNT	The print controls, see Section A.5
MST	Maximum number of iterations for backwater calculations (MST $\leq$ 10)
EPS	The convergence limit for backwater (EPS $\leq$ 0.10 ft.)
PORM	The sediment deposit porosity (PORM $\approx$ .3)
CE	The coefficient of expansion losses (CE $\approx$ .3)
CC	The coefficient of contraction losses (CC $\approx$ .1)
IUNIT	The unit system: 1 - ENGLISH; 0 - METRIC

Counters

The following counters must be defined.

NSEC	Number of cross sections
NTIM	Number of time periods
NRIV	Number of river reaches

NQI Number of input discharges

NCALL Number of subroutine calling sequence (see following)

Subroutine Calling Sequence

The program requires the user to input the order in which the various backwater and sediment routing routines are called. The calling sequence is a function of spatial design and thus differs from river to river. To develop the calling sequence the user needs a basic understanding of the program operation. Figure A.4 shows the gross program flow and the order that each operation should be carried out. For each subroutine call (NC) the following variables must be defined.

ICALL(NC,1) Subroutine number code

ICALL(NC,2&3) Dependent on subroutine

Table A.1 gives an explanation of the variable ICALL.

Calling Sequence Order

The following is a set of guidelines for determining the order in which the various subroutines are called. While the sequence of subroutine calls is not strictly order dependent, the user must have a through knowledge of the program operation before attempting to vary from these guidelines.

1. FLOW is called first to determine discharges.

2. The water surface profile is determined by calling SUBPF or DIVDE, starting with the downstream river reach and working upstream. The profile is calculated for the mainstem first and then for any tributaries.

3. After determining the water surface profiles for all reaches SED is called to calculate sediment transport at each cross section.

4. If there are any weirs in the system WEIRS is called after SED.

5. After SED and WEIRS if there are any point source tributaries TRIBS is called to calculate sediment transport in tributaries.

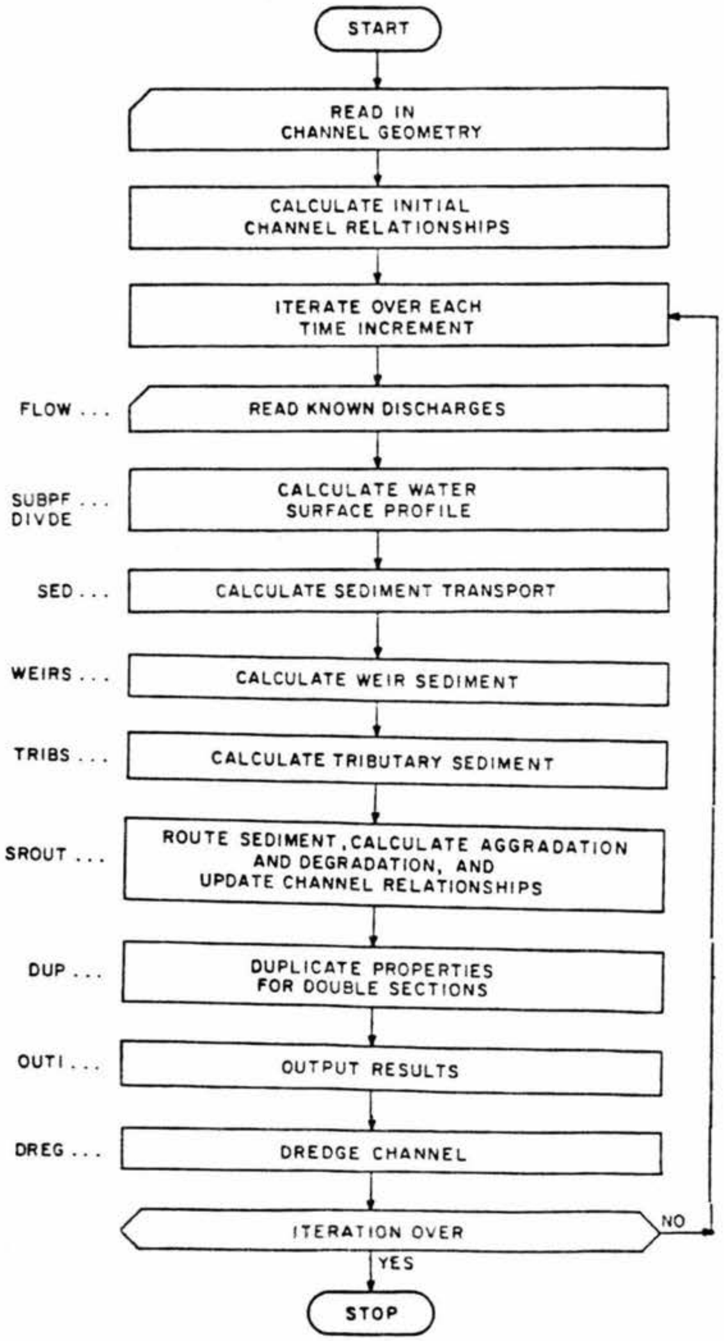


Figure A.4. Subroutine used in operations.

Table A.1. Subroutine Calling Sequence

Value of ICALL(NC,1)	Subroutine Called	Operation Performed	Value of ICALL(NC, 2&3)* 2	3
1	FLOW	Determine discharge at each cross section	---	---
2	SUBPF	Calculate water surface profile	A	---
3	DIVIDE	Compute divided flow reach	B	C
4	SED	Calculate sediment transport at each section	---	---
5	WEIRS	Calculate sediment transport over weir	D	E
6	TRIBS	Calculate all tributary sediment transport	---	---
7	SROUT	Route sediment for reach	F	---
8	DUP	Duplicate properties at double cross sections	G	H
9	OUT1	Output results	---	---
10	DREDG	Dredge river reach	I	---

*Explanation of codes
- A - Number of reach to compute water surface
- B - Number of primary reach in divided flow
- C - Number of secondary reach in divided flow
- D - Number of upstream weir cross section
- E - Number of cross section upstream of weir
- F - Number of reach to compute sediment routing
- G - Number of primary cross section
- H - Number of duplicate cross section
- I - Number of river reach to perform dredging

6. SROUT is called to route the sediment, starting with the downstream reach and working upstream. The sediment for the mainstem is routed first and then for any tributaries. In cases with divided flow, the sediment for the primary reach is routed first and then for the secondary reach.

7. DUP is called after all sediment routing for each duplicated cross section, if any.

8. If routing results are desired OUT1 is called, after SROUT and DUP.

9. After calling OUT1, if any dredging is to be performed DREDG is called.

10. DUP is called again for any duplicate cross sections in dredged reaches.

River Reach

For each river reach the user must define the following variables.

KUP, KDOWN — The numbers for the upstream and downstream cross sections

NTRIB — The number of tributaries to the reach

ICONT — The type of downstream water surface control

1. Stage-discharge relationship
2. Stage-hydrograph
3. Downstream water surface
4. Greatest of #1 and #2
5. Greatest of #1 and #3
6. Normal depth

KCONT — The number of the downstream control cross section (enter 0 if ICONT $\neq$ 3 or 5)

IROUT — The type of downstream cross section sediment routing

1. Fixed section (use on lowermost reach and for secondary reach in divided flow, bed does not aggrade or degrade).
2. Cross section downstream (use when there is another reach directly downstream).

3. Floating section (use for lowest reach on major tributaries).

AX, BX, CX — The coefficients of the downstream control stage-discharge relationship

$$WS = CX + AX\ TQ^{BX}$$

where TQ is the downstream discharge (set to zero if ICONT $\neq$ 1 or 4)

AN, BN — The coefficients of the reach Manning's n-value function

$$n = n_o\ AN\ TQ^{BN}$$

(set, AN = 1.0 and BN = 0.0 if the function is not known and calibrate on known data)

SB — Normal depth slope (use average bed slope).

Tributary

For each tributary the user must define the following variables.

RDT — The mainstem river distance at the tributaries' confluence

ITRIB — The type of tributary

1. Point source in
2. Major tributary in
3. Point source out
4. Major tributary out

KTRIB — The tributary's water discharge section

AT, BT — The coefficient of the point source tributary sediment rating curve

$$QSL = AT\ TQ^{BT}$$

(enter zeros for major tributaries)

Cross Section

For each cross section the following variables must be defined.

ND — The number of cross section points (x,z pairs)

RD — The cross section river distance. Must be measured in upstream direction

X	The array of cross section point stations (horizontal distance) negative values are allowed, but the stations cannot decrease in value from one point to the next
Z	The array of cross section point elevations
DROB, DLOB	Overbank stations. To define the start of overbank conditions, the stations do not have to correspond to points in the x array
FROB, FMC, FLOB	Manning's n-value for right overbank, main channel, and left overbank
ZOB	The overbank elevation used to divide main channel and overbank hydraulic properties relationships

Flow

For each time period the following variables must be defined.

Q	The upstream discharge tons of each cross river reach
QT	The discharge tons of each point source tributary
DT	The time period length in days
STAGE	The stage at the downstream control(set to 0.0 if not used)

A.4 Input Formats

All input data is read into the program from subroutines IN1 and FLOW. The input data are divided into three files to ease the task of assembling and debugging. When the user is evaluating several alternatives, usually only one or two of the input files need changing for each run. The three files are:

I5 - General Data

I7 - Cross Section Data

I8 - Discharge Data

The user must define the device number for each of these files in the main program.

While it is recommended that the input data are kept separate, all three files can be combined into one file as when cards are used as input. To accomplish this the user defines the three files as the same device and then assembles the data cards with the general data first, cross sections second, and discharge data third.

The following describes order and formats for data input.

FILE I5--GENERAL DATA

General Information Cards

Three information cards are required for the title, the print controls, and the convergence limits.

Card Number	Format	Description
1	20A4	(TITLE (M2), M2 = 1, 20) Job Title.
2	8I2	(IPRNT (M1), M1 = 1, 8) Print control (see output section for explanation).
3	I5, 4F10.5, I5	MST, EPS, PORM, CE, CC, IUNIT Maximum number of iterations for the backwater curve (MST); maximum error in total head, (EPS). Sediment deposit porosity (PORM), expansion loss coefficient (CE), contraction loss coefficient (CC), and unit system flag (IUNIT).

Counter Cards

The two counter cards give the number of elements (cross sections), time periods, river reaches etc.) of the system and the computation sequence.

Card Number	Format	Description
1	5I5	NSEC, NTIM, NRIV, NQI, NCALL Number of cross sections (NSEC); number of time periods, NTIM; the number of river reaches, NRIV; number of input discharges, (NQI); number of subroutine (NCALL).

2	3I5	(ICALL (NC, NN), NN = 1, 3) Sequence of subroutine calls. The number of input cards for ICALL is the same as NCALL. Repeat card 2 for each subroutine call, NC.

River Reach Cards

The river reach cards give information on each river reach (number of cross sections, number of tributaries, etc.) and on tributaries. The number of river reach cards depends on the number of river reaches and tributaries.

Card Number	Format	Description
1	6I5, 5F8.4, F8.6	KDOWN (NR), KUP (NR), NTRIB(NR), ICONT(NR), KCONT(NR), IROUT(NR), AX(R), BX(NR), CX(NR), AN(NR), BN(NR), SB(NR). Number of downstream cross section, (KDOWN(NR)); Number of tributaires, (NTRIB(R)); type of downstream control, (ICONT(NR)); number of downstream water surface control cross section, (KCONT (NR)); type of downstream sediment routing (IROUT(NR)); the coefficients of the downstream stage discharge relationship, (AX(NR), BX(NR), CX (NR)); coefficients of the conveyance equation, (AN(NR), BN(NR)), the normal depth slope, (SB(NR)).
2	F10.2, 2I5, 2E10.2	RDT(NR,J), ITRIB(NR,J), KTRIB(NR,J). AT(NR,J), BT(NR,T) Main stem river distance of the confluence, (RDT(NR,J)); type of tributary, (ITRIB(NR,J)); number of discharge cross section for tributary, (KTRIB(NR,)). The coefficients of the tributary sediment input (AT(NR,J), BT(NR,J). (Repeated for each tributary in river reach.) Repeat cards 1 and 2 for each additional river reach.

FILE I7--CROSS SECTION DATA

Cross Section Cards

The cross section cards give information on the river cross sections, including the number of cross section points, river distance, and elevation, and station of each point.

Card Number	Format	Description
1	2X, I3, F7.2	ND(K), RD(K) Number of cross section points in this section, (ND(K)); river distance, (RD(K)).
2	8X, 6(F6.0, F6.1)	X(K,L), Z(K,L) Horizontal distance of cross section points, (X(K,L)); elevation of cross section points (Z(K,L)) in pairs. (Card is repeated for each set of six points.)
3	6F10.4	DROB(K), DLOB(K), FROB, FMC, FLOB, ZOB(K) Distance of right and left overbank, (DROB(K), DLOB(K)); Manning's n for right overbank, main channel, left overbank (FROB, FMC, FLOB); overbank elevation, (ZOB(K)). Repeat card 1 to 3 for each additional cross section.

FILE I8--DISCHARGE DATA

Discharge Cards

The discharge cards give the discharges for each cross section and each discharge section and the time period length.

Card Number	Format	Description
1	Binary	(TQ(K), K = 1,NQI), DT, STAGE, IDREG Discharge TQ(K) for each cross section. Discharge section, and the time period length (DT). If a stage hydrograph is used as a downstream control the stage (STAGE) must be read in. If dredging is to be done the dredging flag IDRG also must be read in.

A.5 Results Outputs

General

Output is user controlled and may vary from no output to output of most intermediate results. Output is controlled by the array IPRNT discussed in this section. An example of the output is found in Section VI. Output is written to three files I6, I9, and I10. File I6 is for printed output and files I9 and I10 are binary files. Binary files are designed so that the detailed intermediate results can be saved. Once the user reviews the printed results, the binary files can be accessed with a used supplied program and any additional information of interest is printed out.

The user must define the device number for each of the output files in the main program.

Print Controls

The following output is controlled by the array IPRNT. The print controls are turned on by inputing a value of 1 for the respective variables. In addition, if at least one of the following print controls IPRNT (1, 2, 3, 4, 5, 6 or 7) is turned on, the title will be printed out.

1. If the print control IPRNT(1) is turned on, all the input data from File I5 and I7 are printed out.

2. If the print control IPRNT(2) is turned on, the following coefficients for the hydraulic properties equation are printed out for the effective width, the effective depth, the total area, the total conveyance, and for alpha, for both flow situations, channel flow and overbank flow:

    - the cross section number
    - the coefficient A of the hydraulic properties equation
    - the power B of the hydraulic property equation
    - the correlation coefficient
    - the standard error

3. If the print control IPRNT(3) is turned on the final bed elevations and the change in elevation at each cross section point is printed out.

4. If the print control IPRNT(4) is turned on, the maximum water elevation and the time period of occurrence at each cross section is output.

5. If the print control IPRNT(5) is turned on, the final minimum bed elevation at each point is printed out.

6. If the print control IPRNT(6) is turned on, following cross section properties are printed out for each cross section and each time period.

   - time period
   - effective width
   - effective depth
   - total area
   - total conveyance
   - alpha
   - velocity
   - water surface
   - discharge
   - sediment transport
   - thalweg elevation

7. If the print control IPRNT (7) is turned on, the following data are output in binary on file I10.

   - effective width
   - effective depth
   - total area
   - total conveyance
   - alpha
   - velocity
   - water surface elevation
   - discharge
   - sediment transport
   - thalweg elevation

   Also the elevation of each cross section point at the end of each year is output in binary to File I9.

8. If the print control IPRNT(8) is turned on, error messages are printed when the backwater or divided flow calculations do not converge. These messages include:

   - the maximum error in total head
   - the number of iterations
   - the cross section number
   - the time period of occurrence

The user should carefully select the desired output and turn off unnecessary output. Generally, print controls IPRNT (1, 2, 6, and 8) provide the best output for initial debugging while IPRNT (1, 3, 4, 5 and 7) provide the best output for production runs. Print controls IPRNT (2 and 6) should not be turned on if a run has several time periods as voluminous amounts of printed output are produced.

A.6 Example Application

General

The following is an example application of the program KUWASER. A portion of the Yazoo River Basin near Greenwood, Mississippi was selected and is shown in Figure A.5. In the example, besides the main-stem calculations, there is a divided flow caused by a cutoff, a major tributary, the Yalobusha River, three-point source tributaries, and a weir. The example gives the step by step procedure necessary to model the case. To simplify the example, actual temporal and spatial designs used in the Sedimentation Study of the Yazoo River Basin for the area were not used.

Temporal Design

The continuity approach is used in the example temporal design. Locations of known discharge are shown in Figure A.6. The discharge at each section (except in divided reaches) is computed by summing all inflows above the section. Any difference between the inflows and the outflows at Belzoni is distributed between Abiaca Creek and Belzoni. Discharge at any cross section between Abiaca Creek and Belzoni is computed by an equation similar to Equation A.2.

Point source sediment routing curves were determined by the empirical method.

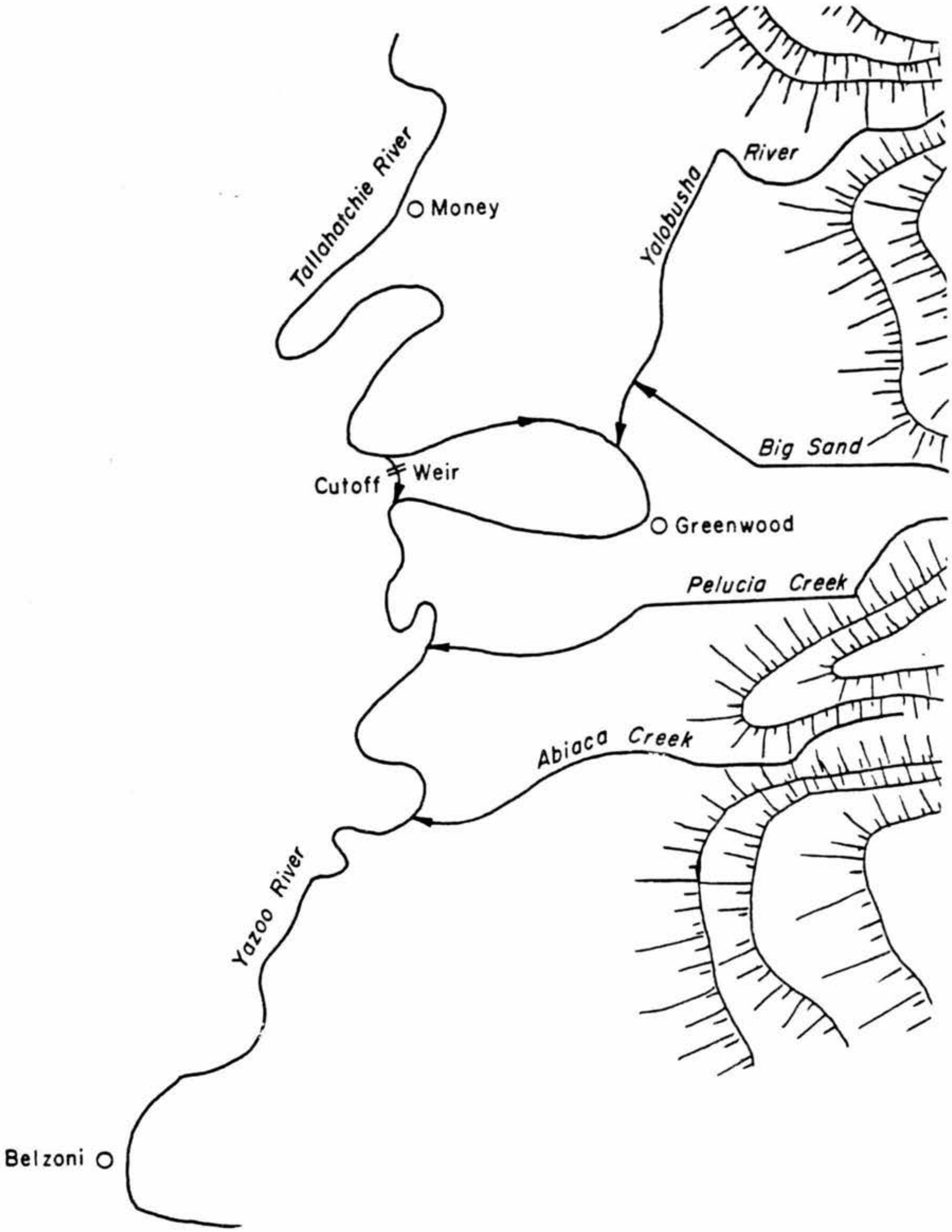


Figure A.5. Example case.

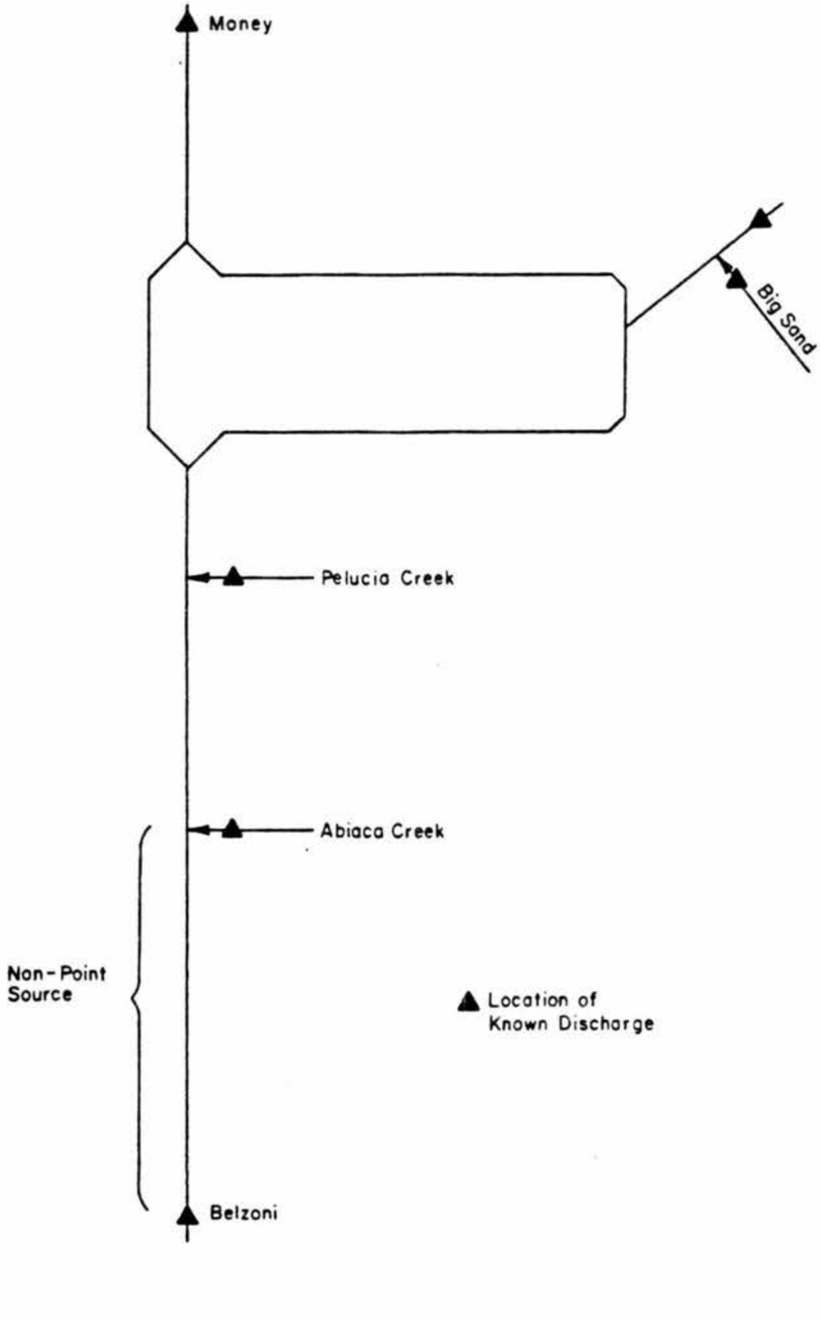


Figure A.6. Example temporal design.

Spatial Design

Figure A.7 shows the example spatial design. The mainstem river is broken into three segments, Reaches I, II, and III. The cutoff where divided flow occurs is Reach IV and the Yalobusha River is Reach V. These divisions were made based on the consistent river characteristics in each reach with consideration of the computational sequence.

Figure A.8 shows the locations of selected cross sections. The cross sections are identified by river mile and number. The cross sections are fairly even spaced at two to three miles except in the cutoff where the short length of the reach has forced tighter spacing. Figure A.8 also shows the discharge section location and number for each point source tributary.

The value of the variables associated with each river reach is shown in Table A.2.

The subroutine computation sequence for the example is

1. call FLOW
2. call SUBPF for Reach I
3. call DIVDE for Reach II and IV
4. call SUBPF for Reach III
5. call SUBPF for Reach V
6. call SED
7. call WEIRS
8. call TRIBS
9. call SROUT for Reach I
10. call SROUT for Reach II
11. call SROUT for Reach IV
12. call SROUT for Reach III

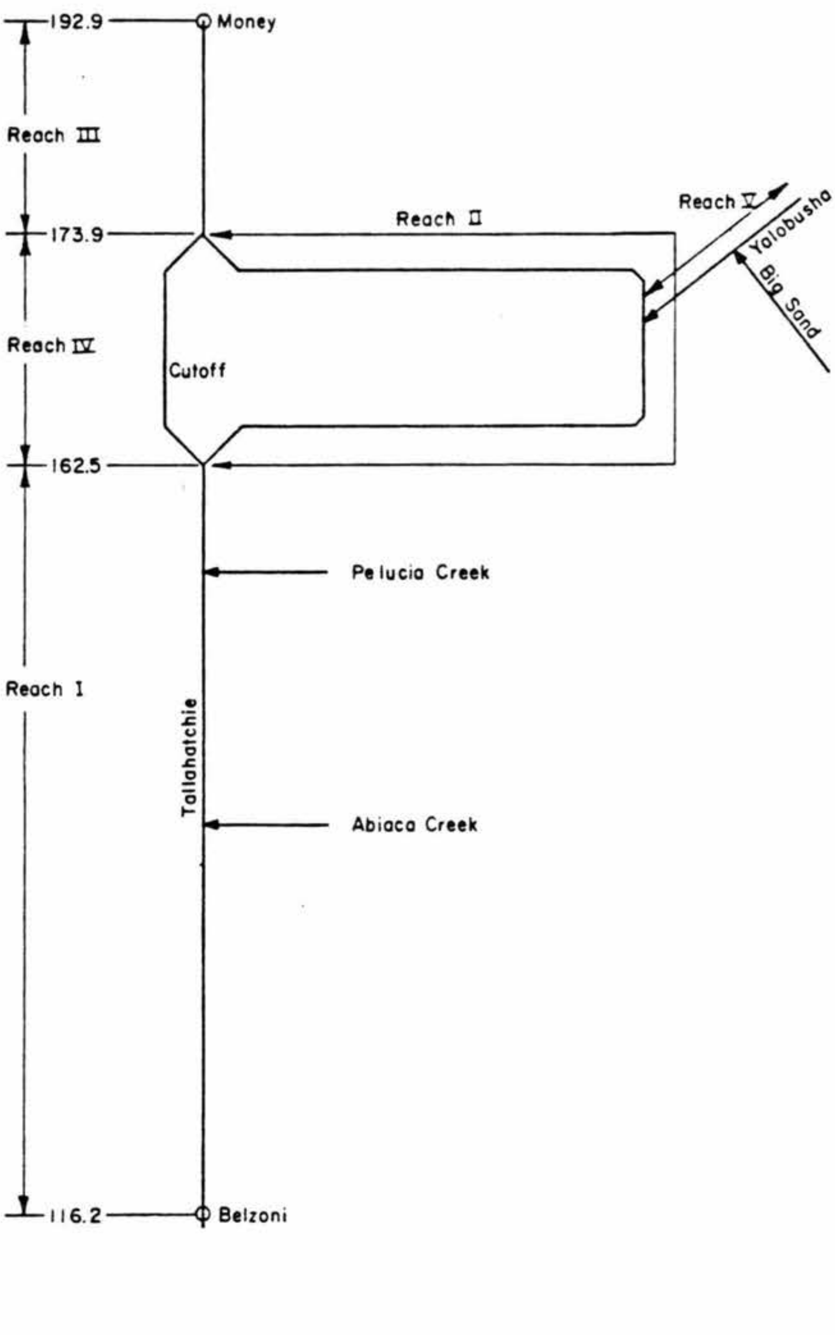


Figure A.7. Example spatial design

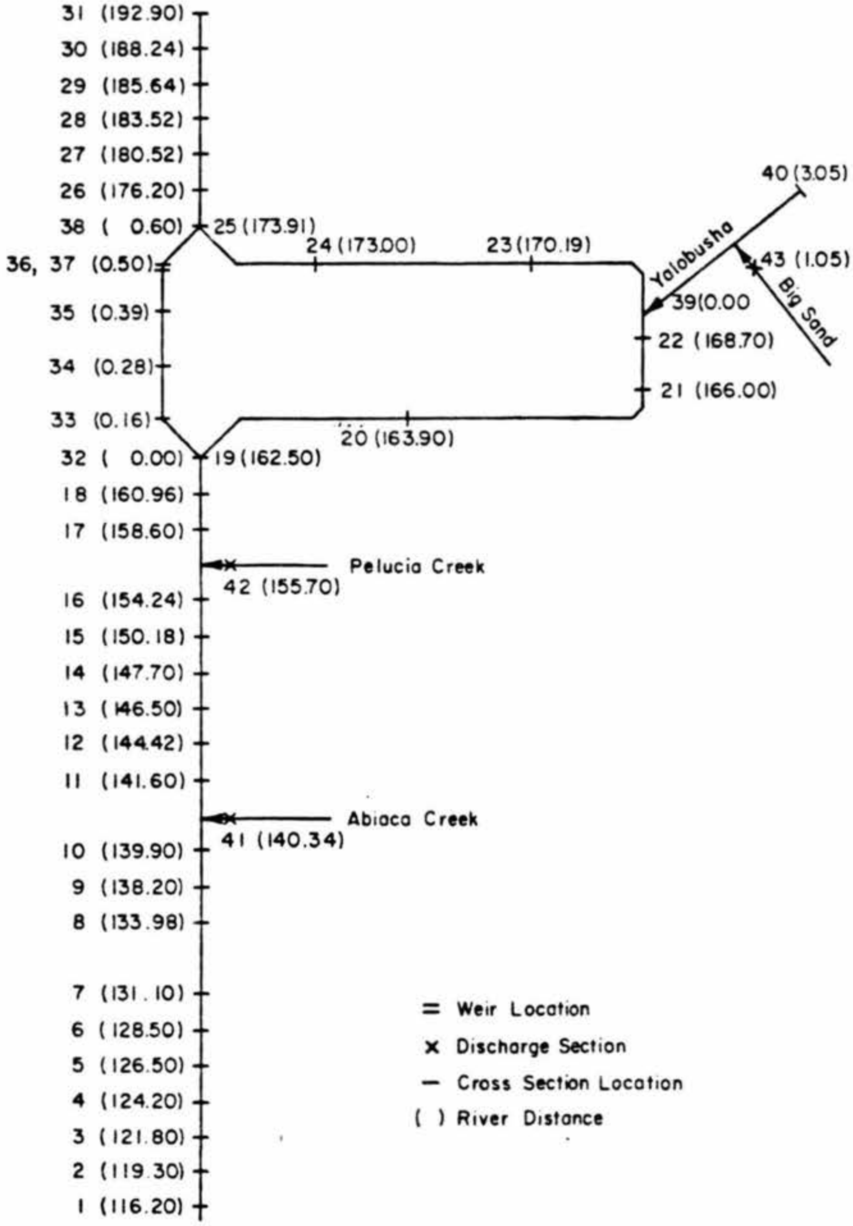


Figure A.8. Example cross section locations.

Table A.2. Input Variable Values for Example

Reach	KDOWN	KUP	NTRIB	ICONT	KCONT	AX, BX, CX	AN, BN	IROUT	SB	TRIBUTARY	RDT	ITRIB	KTRIB	AT
I	1	19	2	1	-	AX = 0.0336	AN = 1.0	1	.00005	1	140.34	1	41	1.5×10^{-8}
						BX = 0.6868	BN = 0.0			2	155.7	1	42	7.5×10^{-8}
						CX = 76.02								
II	19	25	3	3	19	-	AN = 1.0	2	.00007	1	162.51	2	33	-
							BN = 0.0			2	169.0	2	39	-
										3	173.9	4	37	-
III	25	31	0	3	25	-	AN = 1.0	2	.0001	4				
							BN = 0.0							
IV	32	38	2	3	19	-	AN = 1.0	1	.0002	1	0.01	2	20	-
							BN = 0.0			2	0.59	4	24	-
V	39	40	1	3	22		AN = 1.0 BN = 0.0	3	.0002	1	1.05	1	43	1.0×10^{-8}

13. call SROUT for Reach V

14. call DUP for cross sections 19 and 32

15. call DUP for cross sections 28 and 38

16. call OUTPUT

Example Input

Figures A.9, A.10 and A.11, and Table A.3 show input data for the example.

Example Output

Figure A.11 shows the output generated by the example run.

```
          E X A M P L E   F O R   P R O G R A M   K U W A S E R
0 0 1 1 1 1 0 0
  10    0.10000   0.30000   0.30000   0.10000    1
  40      1    5    43   16
   1      0    0
   2      1    0
   3      2    4
   2      3    0
   2      5    0
   4      0    0
   5     37   38
   6      0    0
   7      1    0
   7      2    0
   7      4    0
   7      3    0
   7      5    0
   8     19   32
   8     25   38
   9      0    0
   1     19    2     1     0     1  0.0336  0.6868 76.0200  1.0000  0.0000 .000050
   140.34      1    41  1.50E-08  2.40E+00
   155.70      1    42  1.50E-08  2.40E+00
  19     25    3     3    19     2  0.0000  0.0000  0.0000  1.0000  0.0000 .000070
   162.51      2    33      0.00      0.00
   169.00      2    39      0.00      0.00
   173.90      4    37      0.00      0.00
  25     31    0     3    25     2  0.0000  0.0000  0.0000  1.0000  0.0000 .000100
  32     38    2     3    19     1  0.0000  0.0000  0.0000  1.0000  0.0000 .000200
     0.01      2    20      0.00      0.00
     0.59      4    24      0.00      0.00
  39     40    1     3    22     3  0.0000  0.0000  0.0000  1.0000  0.0000 .000200
     1.05      1    43  1.0E-08   2.4E+00
```

Figure A.9. Example input data for File I5.

```
CS 22 116.20     0  0  0
PI       8086. 108.2 9542. 108.2 9550. 111.0 9598. 118.5 9598. 118.5 9607. 118.4
PI       9658.  99.7 9662.  96.5 9736.  80.9 9806.  76.9 9836.  75.7 9935.  80.5
PI       9946.  84.1 9946.  86.8 9966.  92.0 9970.  99.6 9979. 104.4 9979. 104.4
PI      10026. 105.910034. 101.210045.  98.511586.  98.5
   9660.00   9979.00    .15000    .03000    .15000    104.00
CS 22 119.30     3 29 72
PI      -1510. 110.0 -200. 110.0  -30. 112.0   60. 105.0   75. 106.0   85. 100.0
PI        119. 100.0  160.  76.0  230.  73.0  250.  73.0  280.  76.0  290.  79.0
PI        300.  79.0  310.  81.0  318.  88.0  362.  99.0  380.  98.0  390. 100.0
PI        425.  94.0  475. 105.0  700. 104.0 1990. 104.0
    119.00    385.00    .15000    .03000    .15000    104.00
CS 22 121.80     3 27 72
PI       -692. 107.0 -580. 107.0 -370. 108.0 -340. 107.0 -328. 104.0 -310. 103.0
PI       -272.  82.0 -262.  82.0 -215.  75.0 -192.  74.0 -120.  79.0 -110.  78.0
PI        -95.  81.0  -85.  81.0  -77.  84.0  -25.  95.0   -5. 106.0   15. 109.0
PI         40. 110.0  175. 106.0  300. 109.0  308. 109.0
   -310.00    -10.00    .15000    .03000    .15000    106.00
CS 22 124.20     3 23 72
PI      -1705. 108.0 -560. 108.0 -550. 101.0 -532.  99.0 -508. 100.0 -475.  86.0
PI       -460.  85.0 -445.  72.0 -425.  68.0 -415.  68.0 -405.  67.0 -380.  78.0
PI       -355.  89.0 -340.  92.0 -318. 100.0 -285. 104.0 -260.  99.0 -240.  98.0
PI       -225.  95.0 -205. 100.0 -190. 110.0  895. 110.0
   -500.00   -320.00    .15000    .03000    .15000    100.00
CS 20 126.50     7 26 62
PI      -1520. 108.0 -500. 108.0 -460. 110.0 -440. 104.0 -420. 102.0 -400. 105.0
PI       -360.  82.0 -350.  89.0 -320.  80.0 -300.  80.0 -260.  78.0 -220.  77.0
PI       -180.  84.0 -140. 106.0 -120. 105.0 -100. 108.0  -60. 108.0  -40. 109.0
PI          0. 108.0 1080. 108.0
   -400.00   -140.00    .15000    .03000    .15000    104.00
CS 19 128.50     7 26 62
PI      -1195. 111.0 -200. 111.0  -90. 109.0  -40. 110.0  -40. 107.0    0. 108.0
PI         10. 110.0   40. 111.0   90.  75.0  120.  75.0  140.  76.0  220.  77.0
PI        270.  99.0  300. 102.0  320. 110.0  340. 108.0  400. 108.0  460. 112.0
PI       1405. 112.0
     40.00    270.00    .15000    .03000    .15000    104.00
CS 16 131.10     3 14 72
PI       -130. 111.0 -130. 111.0 -120. 108.0  -60. 108.0  -30. 110.0   20. 108.0
PI         30. 107.0   80.  90.0  200.  80.0  270.  75.0  350. 100.0  350. 112.0
PI        370. 116.0  600. 116.0  670. 123.0  670. 123.0
     30.00    350.00    .15000    .03000    .15000    108.00
CS 16 133.98    11 23 73
PI      -1642. 116.4 -525. 116.4 -357. 111.0 -344. 107.7 -306. 110.5 -263.  84.6
PI       -221.  77.0 -183.  75.5 -163.  77.1 -142.  73.3 -117.  75.8 -101.  82.2
PI        -71.  92.6  -37. 115.5  201. 116.0 1358. 116.0
   -306.00    -37.00    .15000    .03000    .15000    108.00
CS 13 138.20    11 29 73
PI      -1767. 104.1 -516. 104.1 -341. 118.2 -267.  79.1 -141.  86.3  -67.  91.4
PI        -15. 116.3   25. 115.4   29. 113.7  162. 113.7  169. 115.4  264. 115.2
PI       1233. 115.2
   -341.00    -15.00    .15000    .03000    .15000    115.40
CS 18 139.90     3  3 72
PI      -1130. 124.0  -10. 124.0   20. 118.0   40. 119.0  100. 114.0  130. 115.0
PI        210. 113.0  240.  96.0  270.  95.0  370.  75.0  410.  89.0  440.  89.0
PI        460.  98.0  470. 115.0  560. 113.0  590. 115.0  600. 120.0 1870. 120.0
    210.00    470.00    .15000    .03000    .15000    115.00
CS 18 141.60     3  2 72
PI      -1650. 123.0 -390. 123.0 -370. 114.0 -330. 113.0 -300. 102.0 -250.  95.0
PI       -180.  83.0 -170.  85.0 -160.  81.0 -140.  81.0 -140.  84.0 -110.  85.0
PI        -10.  93.0    0.  97.0   30. 111.0   70. 115.0  140. 114.0 1350. 114.0
   -275.00     30.00    .15000    .03000    .15000    105.00
CS 17 144.42     2 25 72
PI      -1206. 117.3  -37. 117.3   58. 114.7  105.  97.9  170.  93.6  268.  75.8
PI        272.  78.8  277.  78.6  294.  75.6  346. 101.3  364. 104.5  371. 103.1
PI        386. 109.4  417. 107.1  451. 119.2  543. 117.3 1794. 117.3
     60.00    346.00    .15000    .03000    .15000    110.00
CS 18 146.50     2 23 72
PI      -1220. 121.0 -100. 121.0  -10. 121.0    0. 124.0    5. 124.0   10. 123.0
PI         70. 113.0  100. 115.0  140. 112.0  210.  85.0  240.  87.0  260.  84.0
PI        280.  83.0  310.  85.0  390.  96.0  420. 114.0  560. 118.0 1780. 118.0
    140.00    420.00    .15000    .03000    .15000    115.00
CS 15 147.70    11 14 73
PI      -1242. 122.8  -96. 122.8  -29. 122.6  -28. 125.3   64. 115.8  159. 113.7
PI        207. 117.2  258.  83.7  329.  86.7  389.  83.8  414.  84.8  493. 117.4
PI        646. 121.6  727. 121.5 1758. 121.5
    207.00    493.00    .15000    .03000    .15000    113.70
CS 16 150.18    11 13 73
PI      -1376. 123.4 -196. 123.4  -16. 124.1   22. 114.7   65. 116.1  124.  84.2
PI        200.  86.5  279.  88.6  290.  85.6  357. 116.8  422. 117.5  450. 115.6
PI        503. 121.5  543. 122.2  595. 114.0 1624. 114.0
     65.00    357.00    .15000    .03000    .15000    114.70
CS 18 154.24     2 11 72
PI      -1670. 122.0 -570. 122.0 -480. 118.0 -460. 116.0 -420. 116.0 -360.  94.0
PI       -310.  94.0 -300.  93.0 -180.  93.0 -170.  91.0 -150.  92.0 -140.  92.0
PI       -100. 108.0  -70. 116.0  -50. 115.0  -20. 118.0    0. 125.0 1330. 125.0
   -420.00   -100.00    .15000    .03000    .15000    116.00
CS 13 158.60     2  4 72
PI      -1540. 117.0 -500. 117.0 -380. 116.0 -350. 121.0 -320. 120.0 -310. 117.0
PI       -300. 116.0 -240. 100.0  -40.  92.0  -10. 101.0   20. 114.0   30. 124.0
PI       1460. 124.0
   -250.00     20.00    .15000    .03000    .15000    115.00
```

Figure A.10. Example input data for File I7.

```
CS 19 160.96     2  1 72
PT      -1700. 133.0 -560. 133.0 -480. 119.0 -420. 118.0 -360.  95.0 -320.  95.0
PT       -270.  94.0 -260.  92.0 -240.  91.0 -230.  93.0 -200.  87.0 -170.  95.0
PT       -140.  96.0  -90. 117.0  -70. 121.0  -40. 119.0  -20. 124.0    0. 127.0
PT       1300. 127.0
  -420.00   -90.00    .15000    .03000    .15000    115.00
CS 19 162.50     3 15 77
PT          0. 130.3   25. 121.3   49. 120.8  101. 125.0  125. 114.8  134. 114.7
PT        145. 103.8  156. 103.7  163.  95.2  215.  83.3  233.  86.9  243.  87.0
PT        294.  92.8  321.  99.7  338. 100.6  347. 103.8  396. 110.9  423. 124.3
PT        448. 122.5
   115.00   396.00    .15000    .03000    .15000    110.00
CS 12 163.9      3 15 77
PT       -443. 121.9 -432. 120.1 -415. 112.1 -406. 109.8 -375.  98.8 -316.  99.4
PT       -264.  96.1 -245.  99.1 -213.  99.9 -207.  97.9 -157. 107.4 -140. 120.8
  -432.00  -140.00    .15000    .03000    .15000    120.00
CS 15 166.0      3 16 77
PT        340. 120.3  364. 108.4  395. 106.1  407. 104.1  437. 104.1  484.  90.0
PT        508.  92.9  526.  90.7  545.  97.2  576.  98.9  616.  98.1  636.  99.0
PT        669. 112.8  674. 121.1  683. 123.4
   340.00   674.00    .15000    .03000    .15000    120.00
CS 12 168.7      3 16 77
PT       -398. 120.3 -360. 123.1 -334. 112.4 -323. 113.2 -297. 102.5 -274. 102.6
PT       -196.  97.4 -162.  97.5 -148.  95.5 -136.  89.2  -84. 125.9    0. 127.3
  -360.00  - 84.00    .15000    .03000    .15000    123.00
CS 12 170.19     3 17 77
PT       -297. 120.3 -290. 120.5 -230. 103.1 -187. 104.6 -181. 102.1 -140. 101.2
PT       -120. 100.0  -76. 101.4  -29. 114.7  -17. 120.7  -11. 118.4    0. 119.0
  -297.00  - 17.00    .15000    .03000    .15000    120.00
CS 12 173.0      3 18 77
PT       -302. 129.0 -286. 121.7 -265. 116.3 -246. 114.7 -217.  99.7 -149.  89.9
PT       -105. 100.1  -66. 104.6  -34. 112.5  -25. 120.2  -15. 123.3    0. 122.8
  -302.00  - 15.00    .15000    .03000    .15000    123.00
CS 15 173.91     3 18 77
PT         21. 125.2   53. 121.1   85. 121.9  124. 108.8  133. 108.8  161.  95.5
PT        228.  94.0  244.  96.5  291.  97.9  311. 101.9  320. 101.2  364. 120.3
PT        374. 120.7  381. 126.2  406. 124.7
    21.00   381.00    .15000    .03000    .15000    125.00
CS 16 176.20     4 13 73
PT      -1040. 126.5  295. 126.5  310. 122.0  360. 122.0  380. 119.0  390. 115.0
PT        400. 107.0  440.  95.0  460.  91.5  500.  92.5  520.  92.5  540.  95.0
PT        560.  95.5  580. 112.0  610. 121.0 1960. 121.0
   385.00   600.00    .15000    .03000    .15000    115.00
CS 16 180.52    12  9 71
PT      -1650. 126.0 -500. 126.0 -390. 127.0 -310. 125.0 -295. 108.0 -250.  99.0
PT       -170.  97.0 -150.  98.0 -130.  97.0 -110.  99.0  -75. 118.0  -50. 122.0
PT        -40. 120.0    0. 132.0  100. 132.0 1350. 132.0
  -300.00  -100.00    .15000    .03000    .15000    115.00
CS 18 183.52    11 30 71
PT      -1180. 136.0 -200. 136.0 -100. 135.0    0. 134.0  140. 124.0  160. 120.0
PT        200. 119.0  235. 122.0  310.  98.0  320.  95.0  390.  99.0  410.  98.0
PT        430. 100.0  480. 122.0  500. 122.0  530. 128.0  600. 127.0 1820. 127.0
   280.00   450.00    .15000    .03000    .15000    120.00
CS 13 185.64    11 22 71
PT      -1338. 124.8  -16. 124.8    1. 125.5   22. 118.4   63. 100.1  111. 100.4
PT        162.  97.5  192.  99.1  222. 115.2  233. 125.8  252. 125.9  270. 124.7
PT       1662. 124.7
     1.00   233.00    .15000    .03000    .15000    124.80
CS 19 188.24    11 16 71
PT      -1270. 135.0 -150. 135.0    0. 132.0   30. 126.0   90. 126.0  120. 122.0
PT        160.  95.0  190.  99.0  210.  92.0  250.  92.0  290. 109.0  300. 108.0
PT        330. 124.0  400. 129.0  430. 125.0  440. 130.0  470. 130.0  500. 129.0
PT       1730. 129.0
   120.00   290.00    .15000    .03000    .15000    125.00
CS 18 192.90    11  8 71
PT      -1323. 130.8   -2. 130.8   35. 127.6   36. 125.5   49. 125.9   65. 119.2
PT         75. 115.7  127.  96.9  177.  96.7  195.  99.6  230.  99.8  264. 115.7
PT        286. 120.4  294. 127.5  336. 126.6  395. 130.0  407. 132.6 1677. 132.6
    -1.00   293.00    .15000    .03000    .15000    125.50
CS 19   0.00     3 15 77
PT          0. 130.3   25. 121.3   49. 120.8  101. 125.0  125. 114.8  134. 114.7
PT        145. 103.8  156. 103.7  163.  95.2  215.  83.3  233.  86.9  243.  87.0
PT        294.  92.8  321.  99.7  338. 100.6  347. 103.8  396. 110.9  423. 124.3
PT        448. 122.5
   115.00   396.00    .15000    .03000    .15000    110.00
CS 14   0.16     5 20 76
PT       -172. 108.0 -165. 105.2 -153. 105.2 -115.  90.7  -93.  89.0  -64.  94.6
PT         -4.  97.5   15.  97.1   57. 100.1   76.  98.0   95.  99.1  128.  96.8
PT        171.  96.5  196. 108.2
  -115.00    57.00    .15000    .03000    .15000     90.70
CS  9   0.28     5 19 76
PT       -152. 109.0  -98. 101.5  -22.  98.8    1.  98.6   55.  99.4  102.  98.9
PT        123.  99.5  129.  98.0  167. 110.1
   -98.00   167.00    .15000    .03000    .15000    101.50
CS 13   0.39     5 18 76
PT        -48. 110.2  -22.  98.8   16.  90.2   32.  82.9   51.  77.7   90.  73.7
PT        100.  77.4  121.  80.8  150.  95.0  160.  94.3  181. 102.6  237. 110.2
PT        264. 131.0
    16.00   121.00    .15000    .03000    .15000     48.00
```

Figure A.10. (Continued).

```
CS 13      .50     WEIR
PT        -491. 131.9 -261. 131.9 -209. 132.8 -169. 119.9 -109. 113.2  -74. 113.3
PT         -50. 110.0   31. 110.0   86. 110.0  133. 110.0  150. 110.0  217. 130.4
PT         251. 130.4
   -150.00    217.00    .15000     .03000     .15000     120.00
CS 13      .50     WEIR
PT        -491. 131.9 -261. 131.9 -209. 132.8 -169. 119.9 -109. 113.2  -74. 113.3
PT         -50. 110.0   31. 110.0   86. 110.0  133. 110.0  150. 110.0  217. 130.4
PT         251. 130.4
   -150.00    217.00    .15000     .03000     .15000     120.00
CS 15     0.60      3 18 77
PT          21. 125.2   53. 121.1   85. 121.9  124. 108.8  133. 108.8  161.  95.5
PT         228.  94.0  244.  96.5  291.  97.9  311. 101.9  320. 101.2  364. 120.3
PT         374. 120.7  381. 126.2  406. 124.7
     21.00    381.00    .15000     .03000     .15000     125.00
CS 18      0.0      3 23 77
PT           1. 129.1   30. 121.6   59. 121.3   88. 105.4  108. 104.2  118.  99.8
PT         138.  97.7  162. 105.1  178. 106.8  192. 106.6  197. 109.2  214. 110.5
PT         231. 116.5  250. 117.2  259. 120.7  273. 122.0  283. 126.2  371. 127.6
     80.00    259.00    .15000     .03000     .15000     105.4
CS 20     3.05      5 19 75
PT        -965. 126.0 -600. 126.0 -485. 126.0 -450. 123.0 -370. 123.0 -325. 122.0
PT        -300. 123.0 -260. 124.0 -250. 124.0 -204. 126.0 -190. 117.0 -180. 114.0
PT         -90. 108.0  -41. 127.0   50. 126.0  150. 123.0  250. 122.0  350. 125.0
PT         460. 126.0  765. 126.0
   -204.00    -41.00    .15000     .03000     .15000     124.00
```

Figure A.10. (Continued).

Table A.3. Example Input Data for File I8 (*the values of the unformated data appear in this column).

Reach	Discharge or cross Section	Name	Water* Discharge in cfs (and time)	Comments
I	1	116.20	11335	
	2	119.30	11423	
	3	121.80	11494	
	4	124.20	11561	
	5	126.50	11626	NPS = -671 cfs
	6	128.50	11683	
	7	131.10	11757	= 28.3 cfs/mile
	8	133.98	11838	
	9	138.20	11958	
	10	139.90	12006	
			←	Abiaca Creek
	11	141.60	11529	
	12	144.42	11529	
	13	146.50	11529	
	14	147.70	11529	
	15	150.18	11529	
			←	Pelucia Creek
	16	154.24	11321	
	17	158.60	11321	
	18	160.96	11321	
II	19	162.50	11321	
	20	163.90	0	
	21	166.00	0	Divided flow
	22	168.70	0	discharges set
	23	170.19	0	to zero
	24	173.00	0	
III	25	173.91	8145	
	26	176.20	8145	
	27	180.52	8145	
	28	183.52	8145	
	29	185.64	8145	
	30	188.24	8145	
	31	192.90	8145	

Table A.3. Continued.

Reach	Discharge or cross Section	Name	Water* Discharge in cfs (and time)	Comments
IV	32	0.00	11321	
	33	0.16	0	
	34	0.28	0	Divided flow
	35	0.39	0	discharges set
	36	0.50	0	to zero
	37	0.50	0	
	38	0.60	8145	
V	39	0.00	3176	
			←	Big Sand Creek
	40	3.05	2884	
	41	ABIACA	477	
	42	PELUCIA	208	
	43	BIG SAND	292	
	DT		7.0	

K U W A S E R
KNOWN DISCHARGE SEDIMENT ROUTING
DEVELOPED BY G.O. BROWN AND R.M. LI
AT COLORADO STATE UNIVERSITY, FOR THE
U.S. ARMY CORPS OF ENGINEERS, VICKSBURG DISTRICT

E X A M P L E   F O R   P R O G R A M   K U W A S E R

C R O S S   S E C T I O N   P R O P E R T I E S
TIME 1 30.00

SECTION	EFFECTIVE WIDTH	EFFECTIVE DEPTH	TOTAL AREA	TOTAL CONVEYANCE	ALPHA	VELOCITY	WATER SURFACE	SEDIMENT TRANSPORT	FLOW	THALWEG ELEVATION
1	234.1	13.1	3169.	843428.	1.1500	2.5247	92.08	.231667	8000.00	75.70
2	169.8	17.9	3247.	976874.	1.2641	2.4635	93.34	.209552	8000.00	72.95
3	208.0	16.2	3532.	1044146.	1.1508	2.2653	94.23	.178957	8000.00	74.26
4	99.2	22.5	2425.	839510.	1.2619	3.2986	94.97	.303801	8000.00	65.79
5	187.8	15.9	3123.	908983.	1.1595	2.5613	96.07	.234519	8000.00	75.84
6	170.7	19.7	3509.	1145920.	1.2176	2.2796	96.74	.180280	8000.00	74.94
7	219.9	15.9	3680.	1074722.	1.1534	2.1742	97.41	.163081	8000.00	75.00
8	185.5	20.5	3963.	1370096.	1.1500	2.0189	98.07	.138711	8000.00	73.32
9	228.7	14.3	3564.	830701.	1.5000	2.2445	99.22	.169945	8000.00	74.22
10	169.2	15.3	2688.	770843.	1.1500	2.9760	100.05	.326801	8000.00	75.67
11	235.6	13.7	3449.	895895.	1.1911	1.9135	100.71	.100967	6600.00	81.19
12	177.0	17.6	3309.	1088136.	1.1691	1.9947	101.42	.110134	6600.00	75.53
13	182.1	15.4	2976.	798814.	1.2908	2.2190	101.99	.139313	6600.00	82.95
14	198.2	16.3	3360.	981287.	1.1860	1.9645	102.42	.109356	6600.00	83.68
15	206.7	15.6	3416.	940701.	1.2550	1.9320	103.01	.103845	6600.00	84.24
16	240.7	10.9	2728.	617949.	1.1518	2.4194	104.53	.174060	6600.00	91.03
17	240.3	10.6	2610.	596463.	1.1500	2.5134	107.12	.191006	6560.00	91.93
18	241.1	14.5	3695.	999643.	1.1746	1.7752	108.02	.086488	6560.00	86.90
19	198.4	22.2	4605.	1696626.	1.1500	1.4245	108.24	.053333	6560.00	81.03
20	195.0	9.1	1830.	364604.	1.1564	2.1083	108.27	.076533	3857.33	96.03
21	182.8	14.0	2792.	600766.	1.5000	1.3817	109.02	.028687	3857.33	89.96
22	172.7	25.5	4805.	1746795.	1.3667	.8028	109.17	.008656	3857.33	78.49
23	130.7	8.8	1224.	236610.	1.1902	.8310	109.17	.002620	1017.33	97.17
24	145.4	13.0	1971.	512664.	1.1500	.5161	109.29	.000947	1017.33	89.98
25	175.0	12.8	2339.	603973.	1.1500	1.5903	109.55	.039325	3720.00	94.01
26	154.3	16.0	2555.	770726.	1.1500	1.4559	110.00	.032493	3720.00	91.51
27	177.6	12.4	2275.	566261.	1.1500	1.6352	110.70	.042100	3720.00	97.01
28	145.5	13.3	2093.	486587.	1.4078	1.7776	111.39	.048243	3720.00	95.03
29	156.1	11.8	1909.	471396.	1.1500	1.9482	112.04	.061743	3720.00	97.46
30	128.5	17.4	2397.	685896.	1.3258	1.5518	112.63	.035874	3720.00	92.00
31	148.3	14.2	2192.	603461.	1.1500	1.6972	113.45	.045086	3720.00	96.70
32	198.4	22.2	4605.	1696626.	1.1500	1.4245	108.24	.053333	6560.00	81.03
33	206.4	19.7	4296.	1436331.	1.1577	.6292	108.24	.003740	2702.67	81.31
34	230.2	9.8	2338.	497816.	1.1500	1.1561	108.24	.014568	2702.67	95.76
35	151.4	14.8	2386.	633882.	1.1500	1.1328	108.26	.013403	2702.67	79.41
36	168.2	2.7	462.	43336.	1.1500	5.8488	108.82	.004028	2702.67	106.00
37	168.2	2.7	462.	43336.	1.1500	5.8488	108.82	.004028	2702.67	106.00
38	174.0	12.8	2331.	601057.	1.1500	1.5956	109.51	.039611	3720.00	94.01
39	77.5	8.0	659.	120300.	1.1636	4.3085	109.17	.257744	2840.00	97.59
40	109.5	6.1	681.	109686.	1.1500	3.8202	117.48	.192739	2600.00	108.00

SECTION	MAX WATER ELEVATION	TIME OF MAX
1	92.08	1
2	93.34	1
3	94.23	1
4	94.97	1
5	96.07	1
6	96.74	1
7	97.41	1
8	98.07	1
9	99.22	1
10	100.05	1
11	100.71	1
12	101.42	1
13	101.99	1
14	102.42	1
15	103.01	1
16	104.53	1
17	107.12	1
18	108.02	1
19	108.24	1
20	108.27	1
21	109.02	1
22	109.17	1
23	109.17	1
24	109.29	1
25	109.55	1
26	110.00	1
27	110.70	1
28	111.39	1
29	112.04	1
30	112.63	1
31	113.45	1
32	108.24	1
33	108.24	1
34	108.24	1
35	108.26	1
36	108.82	1
37	108.82	1
38	109.51	1
39	109.17	1
40	117.48	1

Figure A.11. Example output.

SECTION	MIN BED ELEVATION
1	75.70
2	72.95
3	74.24
4	66.79
5	76.84
6	74.94
7	75.00
8	73.32
9	79.22
10	75.67
11	81.19
12	75.63
13	82.95
14	83.68
15	84.24
16	91.03
17	91.93
18	86.90
19	81.03
20	96.03
21	89.96
22	78.49
23	97.17
24	89.98
25	94.01
26	91.51
27	97.01
28	95.03
29	97.46
30	92.00
31	96.70
32	81.03
33	81.31
34	95.76
35	79.41
36	106.00
37	106.00
38	94.01
39	97.59
40	108.00

FINAL BED ELEVATIONS, AND,
TOTAL CHANGE AT EACH POINT

CROSS SECTION NO. 1

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	8086.	108.2	0.0
2	9542.	108.2	0.0
3	9550.	111.0	0.0
4	9598.	118.5	0.0
5	9598.	118.5	0.0
6	9607.	118.4	0.0
7	9658.	99.7	0.0
8	9662.	96.5	0.0
9	9736.	80.9	0.0
10	9806.	76.9	0.0
11	9836.	75.7	0.0
12	9935.	80.5	0.0
13	9946.	84.1	0.0
14	9946.	86.8	0.0
15	9966.	92.0	0.0
16	9970.	99.6	0.0
17	9979.	104.4	0.0
18	9979.	104.4	0.0
19	10026.	105.9	0.0
20	10034.	101.2	0.0
21	10045.	98.5	0.0
22	11586.	98.5	0.0

Figure A.11. (Continued).

CROSS SECTION NO. 2

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1510.	110.0	0.0
2	-200.	110.0	0.0
3	-30.	112.0	0.0
4	60.	105.0	0.0
5	75.	106.0	0.0
6	85.	100.0	0.0
7	119.	100.0	-.0
8	160.	76.0	-.0
9	230.	73.0	-.0
10	250.	72.9	-.1
11	280.	76.0	-.0
12	290.	79.0	-.0
13	300.	79.0	-.0
14	310.	81.0	-.0
15	318.	88.0	-.0
16	362.	99.0	-.0
17	380.	98.0	0.0
18	390.	100.0	0.0
19	425.	94.0	0.0
20	475.	105.0	0.0
21	700.	104.0	0.0
22	1990.	104.0	0.0

CROSS SECTION NO. 3

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-692.	107.0	0.0
2	-580.	107.0	0.0
3	-370.	108.0	0.0
4	-340.	107.0	0.0
5	-328.	104.0	0.0
6	-310.	103.0	.0
7	-272.	82.0	.0
8	-262.	82.2	.2
9	-215.	75.2	.2
10	-192.	74.2	.2
11	-120.	79.2	.2
12	-110.	78.2	.2
13	-95.	81.2	.2
14	-85.	81.1	.1
15	-77.	84.0	.0
16	-25.	95.0	.0
17	-5.	106.0	0.0
18	15.	109.0	0.0
19	40.	110.0	0.0
20	175.	106.0	0.0
21	300.	109.0	0.0
22	308.	109.0	0.0

CROSS SECTION NO. 4

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1705.	108.0	0.0
2	-560.	108.0	0.0
3	-550.	101.0	0.0
4	-532.	99.0	0.0
5	-508.	100.0	-.0
6	-475.	86.0	-.0
7	-460.	84.9	-.1
8	-445.	71.8	-.2
9	-425.	67.8	-.2
10	-415.	67.7	-.3
11	-405.	66.8	-.2
12	-380.	77.9	-.1
13	-355.	89.0	-.0
14	-340.	92.0	-.0
15	-318.	100.0	-.0
16	-285.	104.0	0.0
17	-260.	99.0	0.0
18	-240.	98.0	0.0
19	-225.	95.0	0.0
20	-205.	100.0	0.0
21	-190.	110.0	0.0
22	895.	110.0	0.0

Figure A.11. (Continued).

CROSS SECTION NO; 5

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1520.	108.0	0.0
2	-500.	108.0	0.0
3	-460.	110.0	0.0
4	-440.	104.0	0.0
5	-420.	102.0	0.0
6	-400.	105.0	-.0
7	-360.	82.0	-.0
8	-350.	88.9	-.1
9	-320.	79.9	-.1
10	-300.	79.8	-.2
11	-260.	77.8	-.2
12	-220.	76.8	-.2
13	-180.	83.9	-.1
14	-140.	106.0	-.0
15	-120.	105.0	0.0
16	-100.	108.0	0.0
17	-60.	108.0	0.0
18	-40.	109.0	0.0
19	0.	108.0	0.0
20	1080.	108.0	0.0

CROSS SECTION NO; 6

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1195.	111.0	0.0
2	-200.	111.0	0.0
3	-90.	109.0	0.0
4	-40.	110.0	0.0
5	-40.	107.0	0.0
6	0.	108.0	0.0
7	10.	110.0	0.0
8	40.	111.0	-.0
9	90.	75.0	-.0
10	120.	74.9	-.1
11	140.	75.9	-.1
12	220.	77.0	-.0
13	270.	99.0	-.0
14	300.	102.0	0.0
15	320.	110.0	0.0
16	340.	108.0	0.0
17	400.	108.0	0.0
18	460.	112.0	0.0
19	1405.	112.0	0.0

CROSS SECTION NO; 7

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-130.	111.0	0.0
2	-130.	111.0	0.0
3	-120.	108.0	0.0
4	-60.	108.0	0.0
5	-30.	110.0	0.0
6	20.	108.0	0.0
7	30.	107.0	-.0
8	80.	90.0	-.0
9	200.	80.0	-.0
10	270.	75.0	-.0
11	350.	100.0	-.0
12	350.	112.0	0.0
13	370.	116.0	0.0
14	600.	116.0	0.0
15	670.	123.0	0.0
16	670.	123.0	0.0

CROSS SECTION NO; 8

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1642.	116.4	0.0
2	-525.	116.4	0.0
3	-357.	111.0	0.0
4	-344.	107.7	0.0
5	-306.	110.5	.0
6	-263.	84.6	.0
7	-221.	77.0	.0
8	-183.	75.5	.0
9	-163.	77.1	.0
10	-142.	73.3	.0
11	-117.	75.8	.0
12	-101.	82.2	.0
13	-71.	92.6	.0
14	-37.	115.5	.0
15	201.	116.0	0.0
16	1358.	116.0	0.0

Figure A.11. (Continued).

CROSS SECTION NO; 9

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1767.	104.1	0.0
2	-516.	104.1	0.0
3	-341.	118.2	.0
4	-267.	79.2	.1
5	-141.	86.4	.1
6	-67.	91.5	.1
7	-15.	116.3	.0
8	25.	115.4	0.0
9	29.	113.7	0.0
10	162.	113.7	0.0
11	169.	115.4	0.0
12	264.	115.2	0.0
13	1233.	115.2	0.0

CROSS SECTION NO; 10

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1130.	124.0	0.0
2	-10.	124.0	0.0
3	20.	118.0	0.0
4	40.	119.0	0.0
5	100.	114.0	0.0
6	130.	115.0	0.0
7	210.	113.0	.0
8	240.	96.0	.0
9	270.	95.5	.5
10	370.	75.7	.7
11	410.	89.6	.6
12	440.	89.3	.3
13	460.	98.1	.1
14	470.	115.0	.0
15	560.	113.0	0.0
16	590.	115.0	0.0
17	600.	120.0	0.0
18	1870.	120.0	0.0

CROSS SECTION NO; 11

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1650.	123.0	0.0
2	-390.	123.0	0.0
3	-370.	114.0	0.0
4	-330.	113.0	0.0
5	-300.	102.0	.0
6	-250.	95.0	.0
7	-180.	83.1	.1
8	-170.	85.2	.2
9	-160.	81.2	.2
10	-140.	81.2	.2
11	-140.	84.1	.1
12	-110.	85.1	.1
13	-10.	93.1	.1
14	0.	97.0	.0
15	30.	111.0	.0
16	70.	115.0	0.0
17	140.	114.0	0.0
18	1350.	114.0	0.0

CROSS SECTION NO; 12

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1206.	117.3	0.0
2	-37.	117.3	0.0
3	58.	114.7	.0
4	105.	97.9	.0
5	170.	93.6	.0
6	268.	75.8	.0
7	272.	78.9	.1
8	277.	78.7	.1
9	294.	75.6	.0
10	346.	101.3	.0
11	364.	104.5	.0
12	371.	103.1	0.0
13	386.	109.4	0.0
14	417.	107.1	0.0
15	451.	119.2	0.0
16	543.	117.3	0.0
17	1794.	117.3	0.0

Figure A.11. (Continued).

CROSS SECTION NO; 13

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1220.	121.0	0.0
2	-100.	121.0	0.0
3	-10.	121.0	0.0
4	0.	124.0	0.0
5	5.	124.0	0.0
6	10.	123.0	0.0
7	70.	113.0	0.0
8	100.	115.0	0.0
9	140.	112.0	-.0
10	210.	85.0	-.0
11	240.	87.0	-.0
12	260.	84.0	-.0
13	280.	83.0	-.0
14	310.	85.0	-.0
15	390.	96.0	-.0
16	420.	114.0	-.0
17	560.	118.0	0.0
18	1780.	118.0	0.0

CROSS SECTION NO; 14

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1242.	122.8	0.0
2	-96.	122.8	0.0
3	-29.	122.6	0.0
4	-28.	125.3	0.0
5	64.	115.8	0.0
6	159.	113.7	0.0
7	207.	117.2	-.0
8	258.	83.7	-.0
9	329.	86.7	-.0
10	389.	83.8	-.0
11	414.	84.8	-.0
12	493.	117.4	-.0
13	646.	121.6	0.0
14	727.	121.5	0.0
15	1758.	121.5	0.0

CROSS SECTION NO; 15

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1376.	123.4	0.0
2	-196.	123.4	0.0
3	-16.	124.1	0.0
4	22.	114.7	0.0
5	65.	116.1	.0
6	124.	84.2	.0
7	200.	86.6	.1
8	279.	88.7	.1
9	290.	85.6	.0
10	357.	116.8	.0
11	422.	117.5	0.0
12	450.	115.6	0.0
13	503.	121.5	0.0
14	543.	122.2	0.0
15	595.	114.0	0.0
16	1624.	114.0	0.0

CROSS SECTION NO; 16

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1670.	122.0	0.0
2	-570.	122.0	0.0
3	-480.	118.0	0.0
4	-460.	116.0	0.0
5	-420.	116.0	.0
6	-360.	94.0	.0
7	-310.	94.0	.0
8	-300.	93.0	.0
9	-180.	93.0	.0
10	-170.	91.0	.0
11	-160.	92.0	.0
12	-140.	92.0	.0
13	-100.	108.0	.0
14	-70.	116.0	0.0
15	-50.	115.0	0.0
16	-20.	118.0	0.0
17	0.	125.0	0.0
18	1330.	125.0	0.0

Figure A.11. (Continued).

CROSS SECTION NO; 17

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1540.	117.0	0.0
2	-500.	117.0	0.0
3	-380.	116.0	0.0
4	-350.	121.0	0.0
5	-320.	120.0	0.0
6	-310.	117.0	0.0
7	-300.	116.0	-.0
8	-240.	99.9	-.1
9	-40.	91.9	-.1
10	-10.	101.0	-.0
11	20.	114.0	-.0
12	30.	124.0	0.0
13	1460.	124.0	0.0

CROSS SECTION NO; 18

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1700.	133.0	0.0
2	-560.	133.0	0.0
3	-480.	119.0	0.0
4	-420.	118.0	-.0
5	-360.	95.0	-.0
6	-320.	94.9	-.1
7	-270.	93.9	-.1
8	-260.	91.9	-.1
9	-240.	90.9	-.1
10	-230.	92.9	-.1
11	-200.	86.9	-.1
12	-170.	94.9	-.1
13	-140.	96.0	-.0
14	-90.	117.0	-.0
15	-70.	121.0	0.0
16	-40.	119.0	0.0
17	-20.	124.0	0.0
18	0.	127.0	0.0
19	1300.	127.0	0.0

CROSS SECTION NO; 19

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1230.	130.0	0.0
2	0.	130.0	0.0
3	10.	129.5	0.0
4	15.	125.0	0.0
5	120.	119.0	.0
6	160.	100.0	.0
7	200.	89.0	.0
8	230.	83.5	.0
9	260.	81.0	.0
10	280.	81.0	.0
11	340.	90.0	.0
12	360.	91.0	.0
13	380.	99.0	.0
14	395.	114.0	.0
15	405.	119.0	0.0
16	1770.	119.0	0.0

CROSS SECTION NO; 20

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1764.	121.9	0.0
2	-443.	121.9	0.0
3	-432.	120.1	0.0
4	-415.	112.1	0.0
5	-406.	109.8	-.0
6	-375.	98.8	-.0
7	-316.	99.3	-.1
8	-264.	96.0	-.1
9	-245.	99.0	-.1
10	-213.	99.8	-.1
11	-207.	97.9	-.0
12	-157.	107.4	-.0
13	-140.	120.8	-.0
14	1236.	120.8	0.0

Figure A.11. (Continued).

CROSS SECTION NO. 21

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1016.	120.3	0.0
2	340.	120.3	-.0
3	364.	108.4	-.0
4	395.	106.1	-.0
5	407.	104.1	-.0
6	437.	104.1	-.0
7	484.	90.0	-.0
8	508.	92.8	-.1
9	526.	90.6	-.1
10	545.	97.2	-.0
11	578.	98.9	-.0
12	616.	98.1	-.0
13	636.	99.0	-.0
14	669.	112.8	-.0
15	674.	121.1	0.0
16	683.	123.4	0.0
17	1984.	123.4	0.0

CROSS SECTION NO. 22

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1797.	125.5	0.0
2	-502.	125.5	0.0
3	-468.	122.5	0.0
4	-461.	112.8	.0
5	-436.	105.2	.0
6	-396.	95.3	.0
7	-386.	94.7	.2
8	-381.	91.1	.3
9	-347.	84.3	.3
10	-337.	83.7	.4
11	-324.	80.1	.5
12	-297.	78.5	.5
13	-267.	82.5	.4
14	-229.	94.7	.1
15	-193.	106.0	.0
16	-165.	108.5	.0
17	-163.	119.8	.0
18	-118.	119.8	0.0
19	-80.	126.1	0.0
20	-1.	127.8	0.0
21	1203.	127.8	0.0

CROSS SECTION NO. 23

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1650.	120.6	0.0
2	-299.	120.6	0.0
3	-290.	120.6	.0
4	-250.	105.9	.0
5	-232.	103.0	.0
6	-204.	105.6	.1
7	-172.	97.9	.1
8	-150.	97.2	.2
9	-122.	103.5	.1
10	-109.	103.6	.1
11	-90.	102.2	.1
12	-81.	104.2	.1
13	-72.	104.2	.0
14	-21.	120.3	.0
15	-5.	119.2	0.0
16	1350.	119.2	0.0

CROSS SECTION NO. 24

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1149.	129.0	0.0
2	-302.	129.0	0.0
3	-286.	121.7	0.0
4	-265.	116.3	0.0
5	-246.	114.7	.0
6	-217.	99.8	.1
7	-149.	90.0	.1
8	-105.	100.1	.0
9	-66.	104.6	.0
10	-34.	112.5	.0
11	-25.	120.2	0.0
12	-15.	123.3	0.0
13	0.	122.8	0.0
14	851.	122.8	0.0

Figure A.11. (Continued).

CROSS SECTION NO; 25

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	21.	125.2	0.0
2	53.	121.1	0.0
3	85.	121.9	.0
4	124.	108.8	.0
5	133.	108.8	.0
6	161.	95.5	.0
7	228.	94.0	.0
8	244.	96.5	.0
9	291.	97.9	.0
10	311.	101.9	.0
11	320.	101.2	.0
12	364.	120.3	.0
13	374.	120.7	0.0
14	381.	126.2	0.0
15	406.	124.7	0.0

CROSS SECTION NO; 26

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1040.	126.5	0.0
2	295.	126.5	0.0
3	310.	122.0	0.0
4	360.	122.0	0.0
5	380.	119.0	0.0
6	390.	115.0	.0
7	400.	107.0	.0
8	440.	95.0	.0
9	460.	91.5	.0
10	500.	92.5	.0
11	520.	92.5	.0
12	540.	95.0	.0
13	560.	95.5	.0
14	580.	112.0	.0
15	610.	121.0	0.0
16	1960.	121.0	0.0

CROSS SECTION NO; 27

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1650.	126.0	0.0
2	-500.	126.0	0.0
3	-390.	127.0	0.0
4	-310.	125.0	.0
5	-295.	108.0	.0
6	-250.	99.0	.0
7	-170.	97.0	.0
8	-150.	98.0	.0
9	-130.	97.0	.0
10	-110.	99.0	.0
11	-75.	118.0	.0
12	-50.	122.0	0.0
13	-40.	120.0	0.0
14	0.	132.0	0.0
15	100.	132.0	0.0
16	1350.	132.0	0.0

CROSS SECTION NO; 28

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1180.	136.0	0.0
2	-200.	136.0	0.0
3	-100.	135.0	0.0
4	0.	134.0	0.0
5	140.	124.0	0.0
6	160.	120.0	0.0
7	200.	119.0	0.0
8	235.	122.0	.0
9	310.	98.0	.0
10	320.	95.0	.0
11	390.	99.0	.0
12	410.	98.0	.0
13	430.	100.0	.0
14	480.	122.0	.0
15	500.	122.0	0.0
16	530.	128.0	0.0
17	600.	127.0	0.0
18	1820.	127.0	0.0

Figure A.11. (Continued).

CROSS SECTION NO. 29

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1338.	124.8	0.0
2	-16.	124.8	0.0
3	1.	125.5	0.0
4	22.	118.4	-.0
5	63.	100.1	-.0
6	111.	100.4	-.0
7	162.	97.5	-.0
8	192.	99.1	-.0
9	222.	115.2	-.0
10	233.	125.8	0.0
11	252.	125.9	0.0
12	270.	124.7	0.0
13	1662.	124.7	0.0

CROSS SECTION NO. 30

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1270.	135.0	0.0
2	-150.	135.0	0.0
3	0.	132.0	0.0
4	30.	126.0	0.0
5	90.	126.0	0.0
6	120.	122.0	.0
7	160.	95.0	.0
8	190.	99.0	.0
9	210.	92.0	.0
10	250.	92.0	.0
11	290.	109.0	.0
12	300.	108.0	.0
13	330.	124.0	.0
14	400.	129.0	0.0
15	430.	125.0	0.0
16	440.	130.0	0.0
17	470.	130.0	0.0
18	500.	129.0	0.0
19	1730.	129.0	0.0

CROSS SECTION NO. 31

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1323.	130.8	0.0
2	-2.	130.8	0.0
3	35.	127.6	0.0
4	36.	125.5	0.0
5	49.	125.9	0.0
6	65.	119.2	0.0
7	75.	115.7	0.0
8	127.	96.9	0.0
9	177.	96.7	0.0
10	195.	99.6	0.0
11	230.	99.8	0.0
12	264.	115.7	0.0
13	286.	120.4	0.0
14	294.	127.5	0.0
15	336.	126.6	0.0
16	395.	130.0	0.0
17	407.	132.6	0.0
18	1677.	132.6	0.0

CROSS SECTION NO. 32

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-1230.	130.0	0.0
2	0.	130.0	0.0
3	10.	129.5	0.0
4	15.	125.0	0.0
5	120.	119.0	.0
6	160.	100.0	.0
7	200.	89.0	.0
8	230.	83.5	.0
9	260.	81.0	.0
10	280.	81.0	.0
11	340.	90.0	.0
12	360.	91.0	.0
13	380.	99.0	.0
14	395.	114.0	.0
15	405.	119.0	0.0
16	1770.	119.0	0.0

Figure A.11. (Continued).

CROSS SECTION NO. 33

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-607.	129.9	0.0
2	-192.	129.9	.0
3	-151.	99.6	.1
4	-107.	81.3	.4
5	-28.	88.4	.4
6	-5.	97.2	.1
7	33.	105.0	.0
8	75.	104.0	.0
9	96.	95.1	.2
10	131.	93.2	.2
11	156.	94.2	.1
12	224.	131.5	.0
13	246.	130.3	0.0
14	393.	130.3	0.0

CROSS SECTION NO. 34

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-393.	127.6	0.0
2	-210.	127.6	0.0
3	-163.	115.7	.0
4	-111.	101.1	.0
5	-51.	99.0	.0
6	-8.	98.9	.0
7	50.	98.1	.1
8	107.	95.8	.1
9	130.	97.7	.0
10	151.	112.1	.0
11	194.	126.3	0.0
12	607.	126.3	0.0

CROSS SECTION NO. 35

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-491.	131.9	0.0
2	-261.	131.9	0.0
3	-209.	132.8	0.0
4	-169.	119.9	0.0
5	-109.	113.2	0.0
6	-74.	113.1	-.2
7	9.	79.4	-.3
8	31.	94.8	-.3
9	86.	101.0	-.1
10	133.	102.0	-.1
11	150.	99.1	-.0
12	217.	130.4	-.0
13	251.	130.4	0.0
14	509.	130.4	0.0

CROSS SECTION NO. 36

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-491.	131.9	0.0
2	-261.	131.9	0.0
3	-209.	132.8	0.0
4	-169.	119.9	0.0
5	-109.	113.2	0.0
6	-74.	113.3	0.0
7	9.	106.0	0.0
8	31.	106.0	0.0
9	86.	106.0	0.0
10	133.	106.0	0.0
11	150.	106.0	0.0
12	217.	130.4	0.0
13	251.	130.4	0.0

CROSS SECTION NO. 37

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-491.	131.9	0.0
2	-261.	131.9	0.0
3	-209.	132.8	0.0
4	-169.	119.9	0.0
5	-109.	113.2	0.0
6	-74.	113.3	0.0
7	9.	106.0	0.0
8	31.	106.0	0.0
9	86.	106.0	0.0
10	133.	106.0	0.0
11	150.	106.0	0.0
12	217.	130.4	0.0
13	251.	130.4	0.0

Figure A.11. (Continued).

CROSS SECTION NO. 38

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	21.	125.2	0.0
2	53.	121.1	0.0
3	85.	121.9	.0
4	124.	108.8	.0
5	133.	108.8	.0
6	161.	95.5	.0
7	228.	94.0	.0
8	244.	96.5	.0
9	291.	97.9	.0
10	311.	101.9	.0
11	320.	101.2	.0
12	364.	120.3	.0
13	374.	120.7	0.0
14	381.	126.2	0.0
15	406.	124.7	0.0

CROSS SECTION NO. 39

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-737.	129.1	0.0
2	1.	129.1	0.0
3	30.	121.6	0.0
4	59.	121.3	-.0
5	88.	105.4	-.0
6	108.	104.2	-.0
7	118.	99.7	-.1
8	138.	97.6	-.1
9	162.	105.0	-.1
10	178.	106.8	-.0
11	192.	106.6	-.0
12	197.	109.2	-.0
13	214.	110.5	0.0
14	231.	116.5	0.0
15	250.	117.2	0.0
16	259.	120.7	0.0
17	273.	122.0	0.0
18	283.	126.2	0.0
19	371.	127.6	0.0
20	1013.	127.6	0.0

CROSS SECTION NO. 40

POINT	HORIZONTAL	ELEVATION	DELTA ELEV.
1	-965.	126.0	0.0
2	-600.	126.0	0.0
3	-485.	126.0	0.0
4	-450.	123.0	0.0
5	-370.	123.0	0.0
6	-325.	122.0	0.0
7	-300.	123.0	0.0
8	-280.	124.0	0.0
9	-250.	124.0	0.0
10	-204.	126.0	0.0
11	-190.	117.0	0.0
12	-180.	114.0	0.0
13	-90.	108.0	0.0
14	-41.	127.0	0.0
15	50.	126.0	0.0
16	150.	123.0	0.0
17	250.	122.0	0.0
18	350.	125.0	0.0
19	460.	126.0	0.0
20	785.	126.0	0.0

Figure A.11. (Continued).

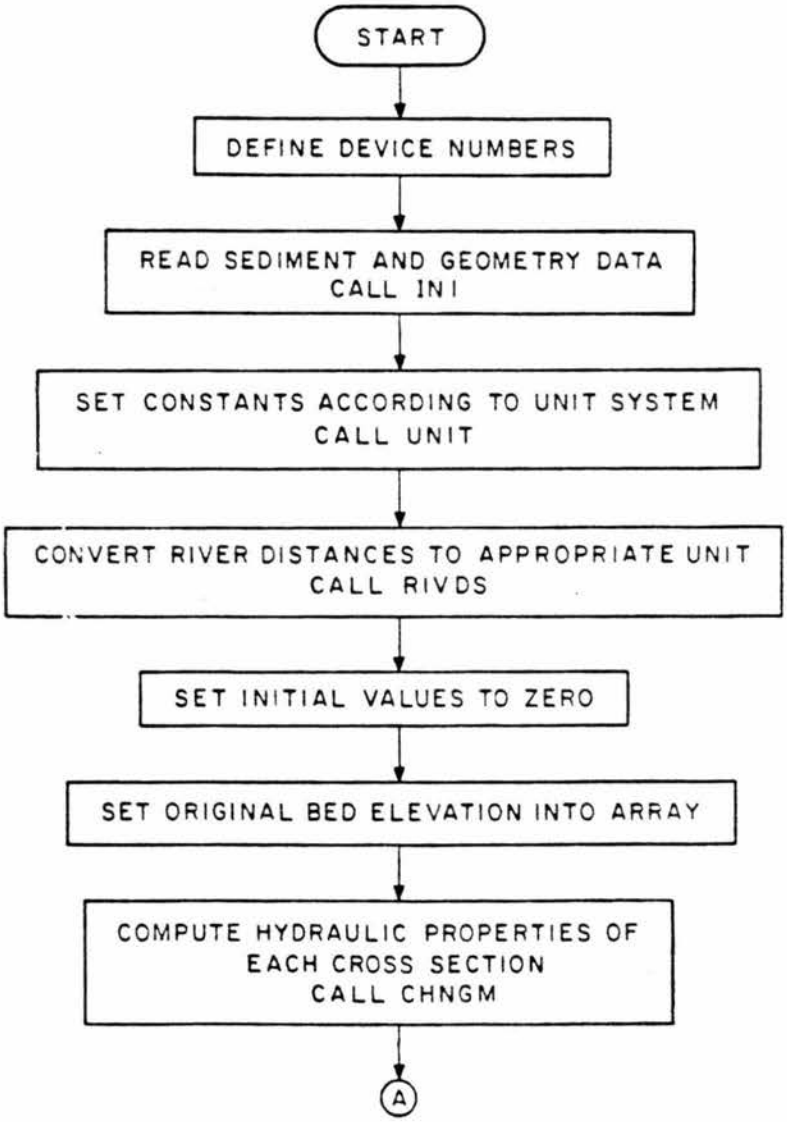
APPENDIX B

PROGRAM FLOW CHARTS

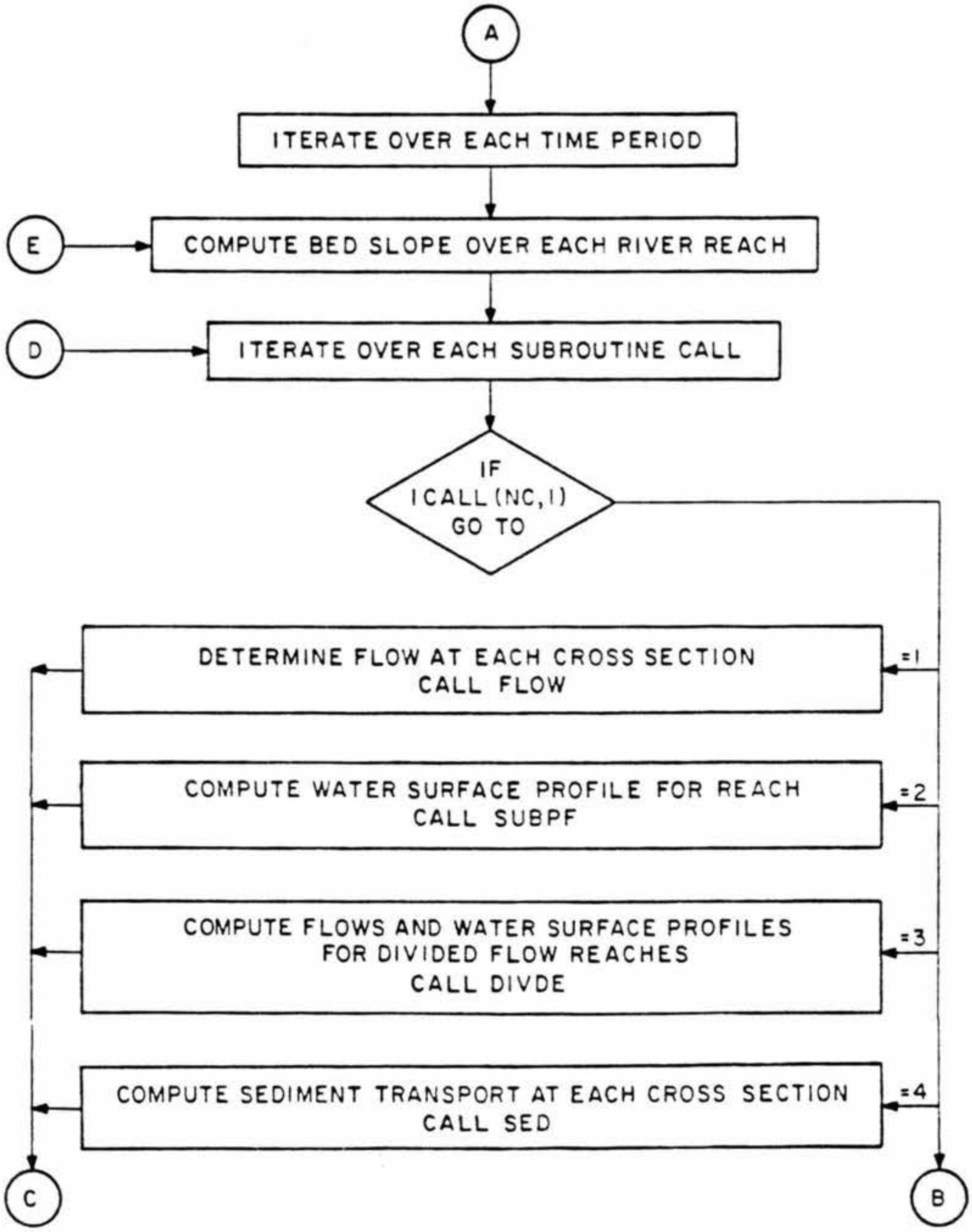
The following are subroutine flow charts for the program KUWASER. To aid the user in understanding the program the flow charts have been designed to show the overall program operation and not the actual FORTRAN statements. The flow charts are presented by subroutine in alphabetical order.

Program KUWASER

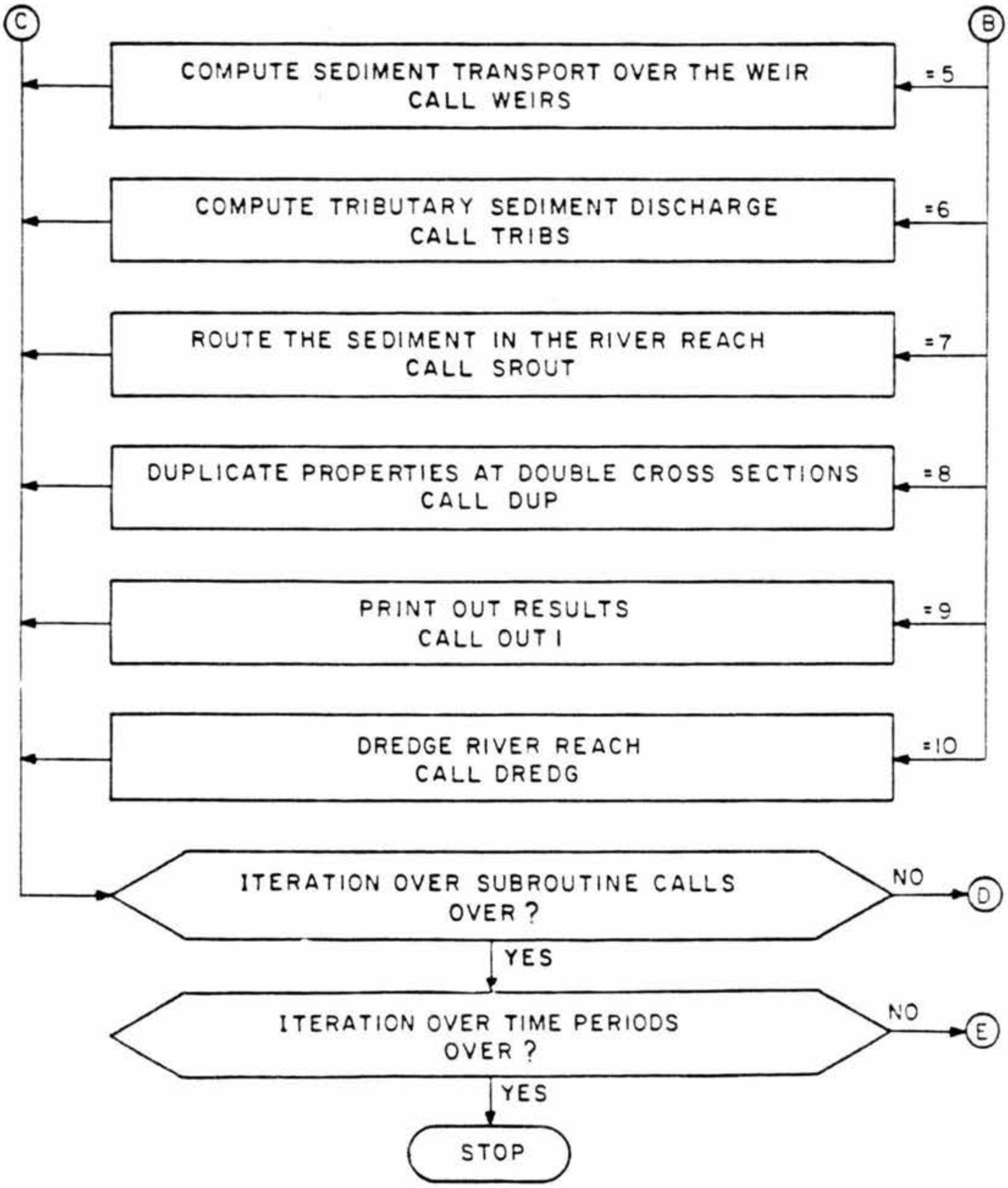
This program is a known discharge, water and sediment routing model.



Program KUWASER (continued)

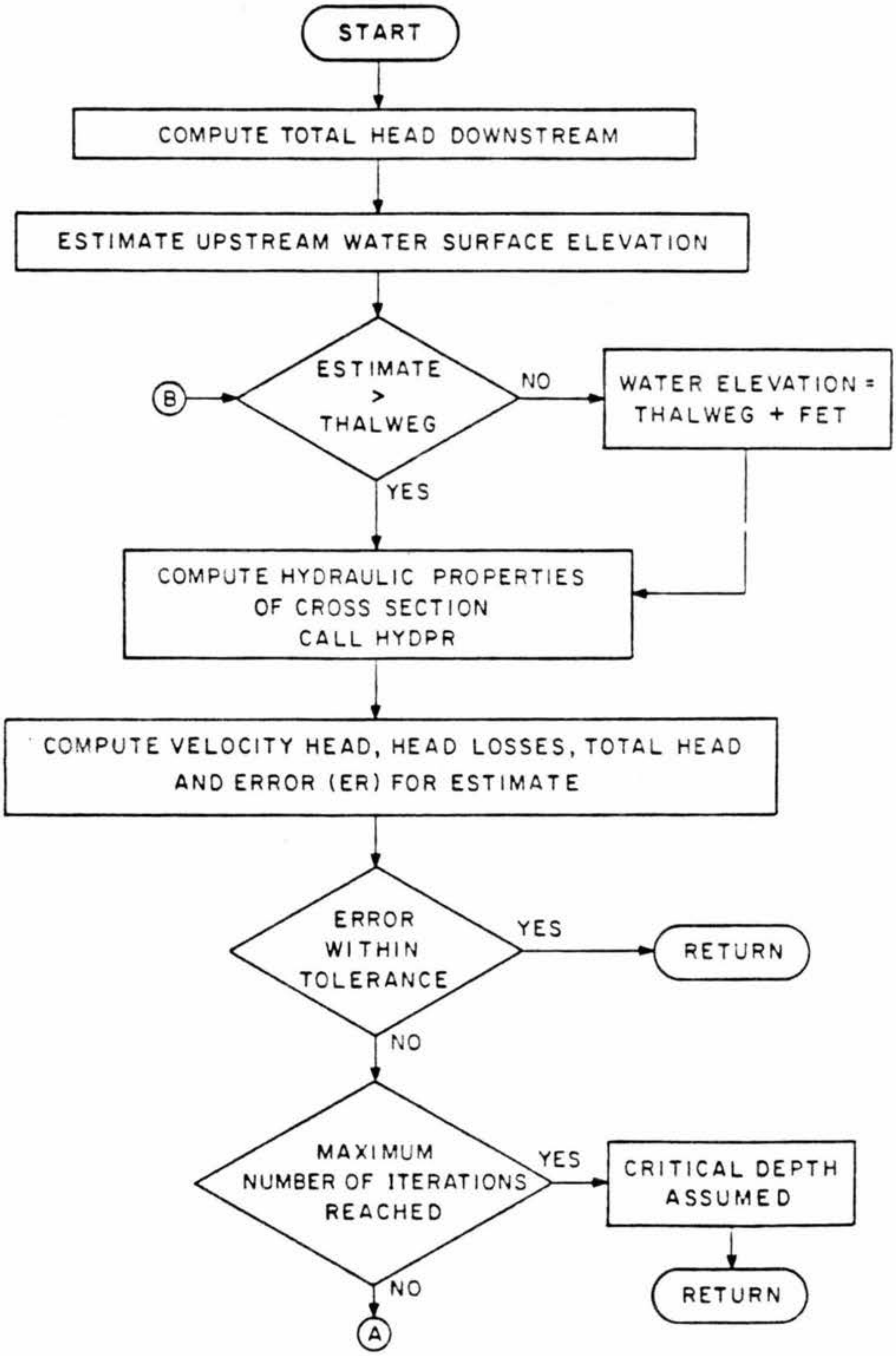


Program KUWASER (continued)

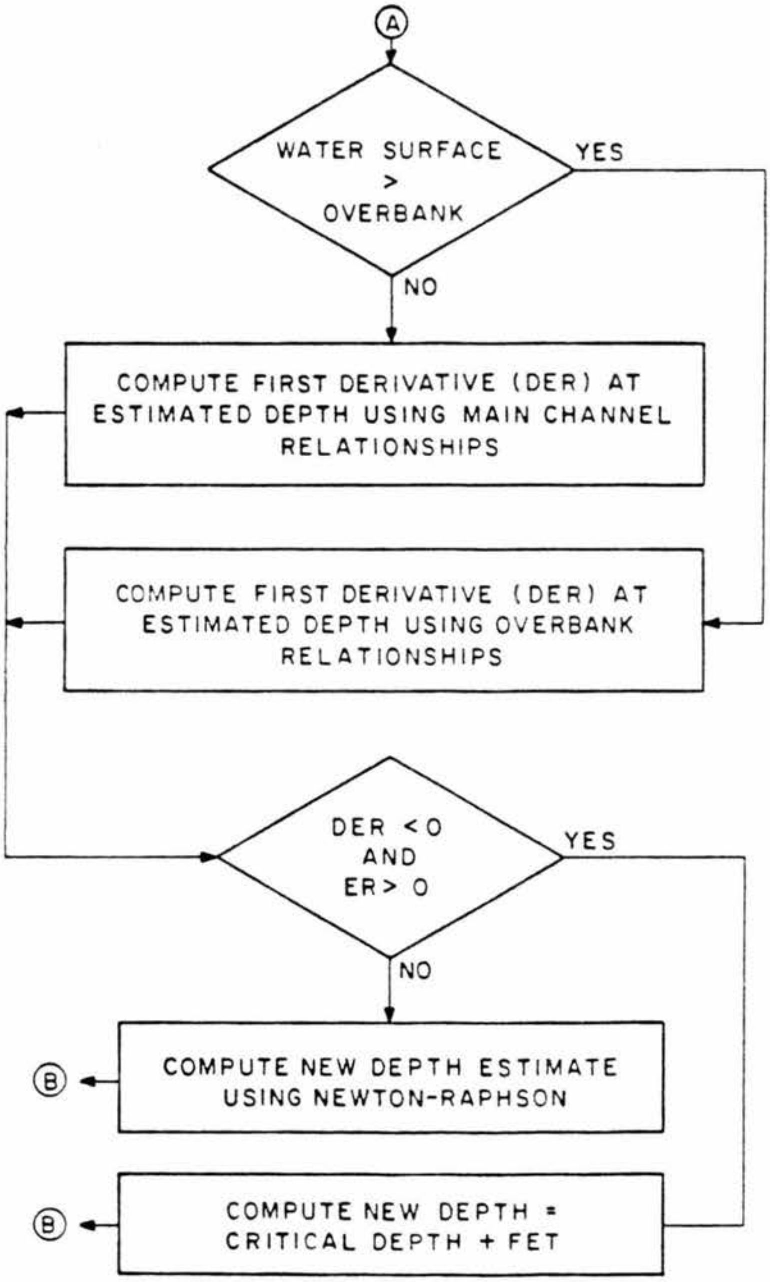


Subroutine BKWAT

This subroutine calculates the water surface elevation at a cross section once the conditions at the downstream section are known. The routine uses a first order Newton-Raphson solution to solve the total head equation.

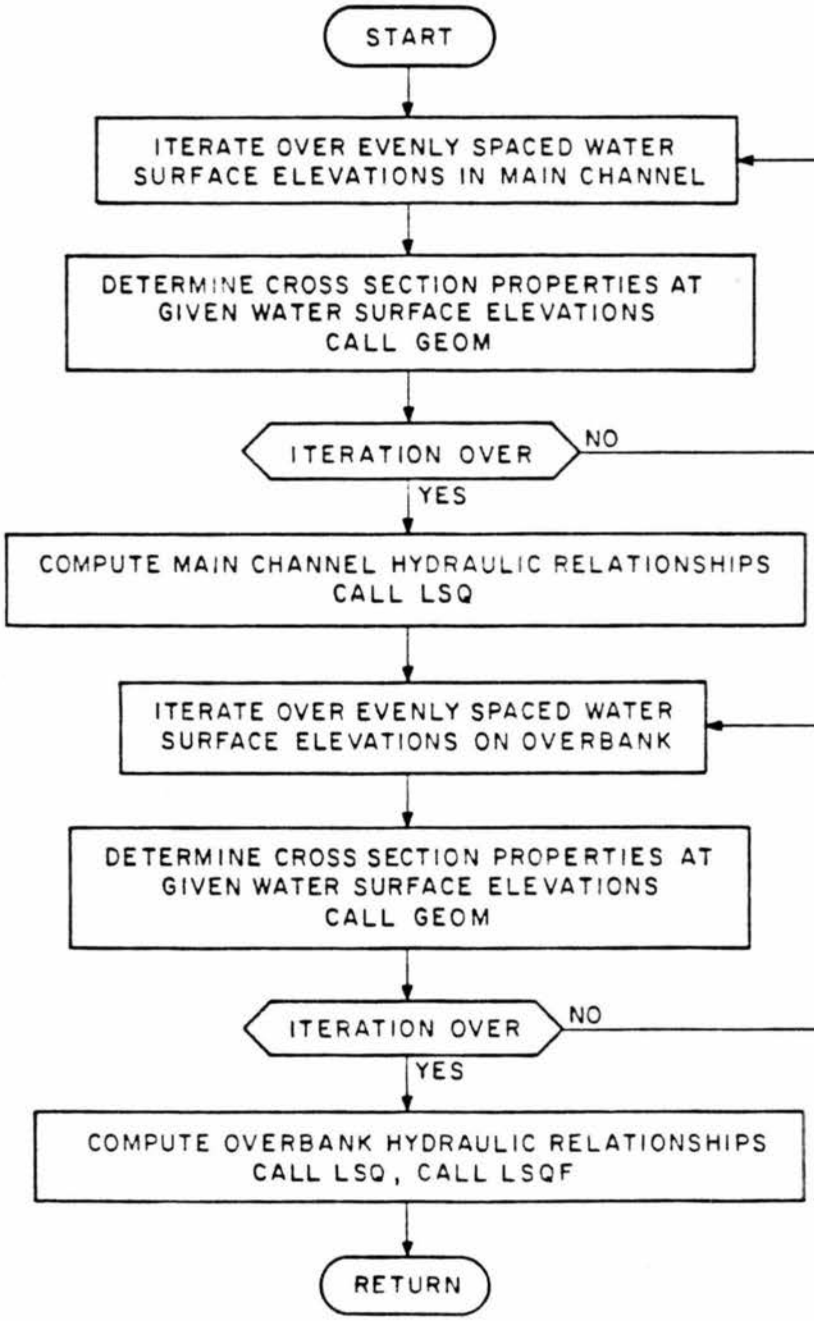


Subroutine BKWAT (continued)



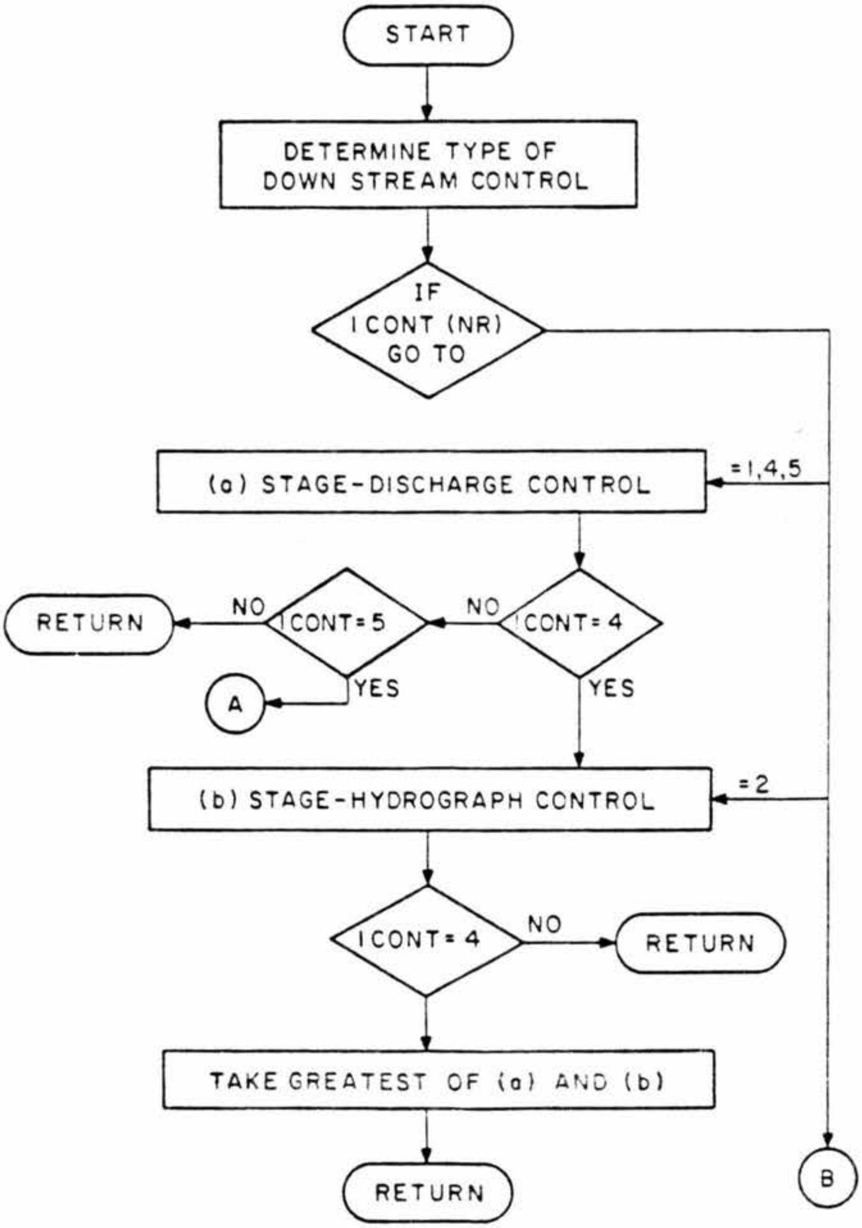
Subroutine CHNGM

This subroutine computes power relations that are used to calculate effective depth (ED), effective width (EW), alpha (ALP), total area (TA), and total conveyance (TK), for a cross section, as a function of the water surface elevation (WS).

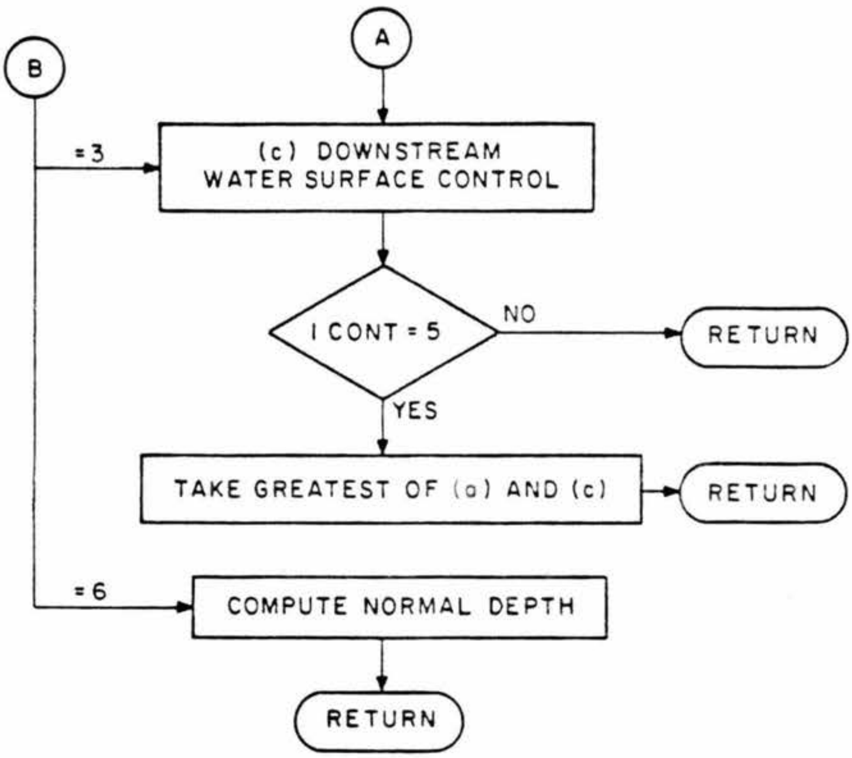


Subroutine CONT

This subroutine is used to compute the water surface elevation at the downstream control.

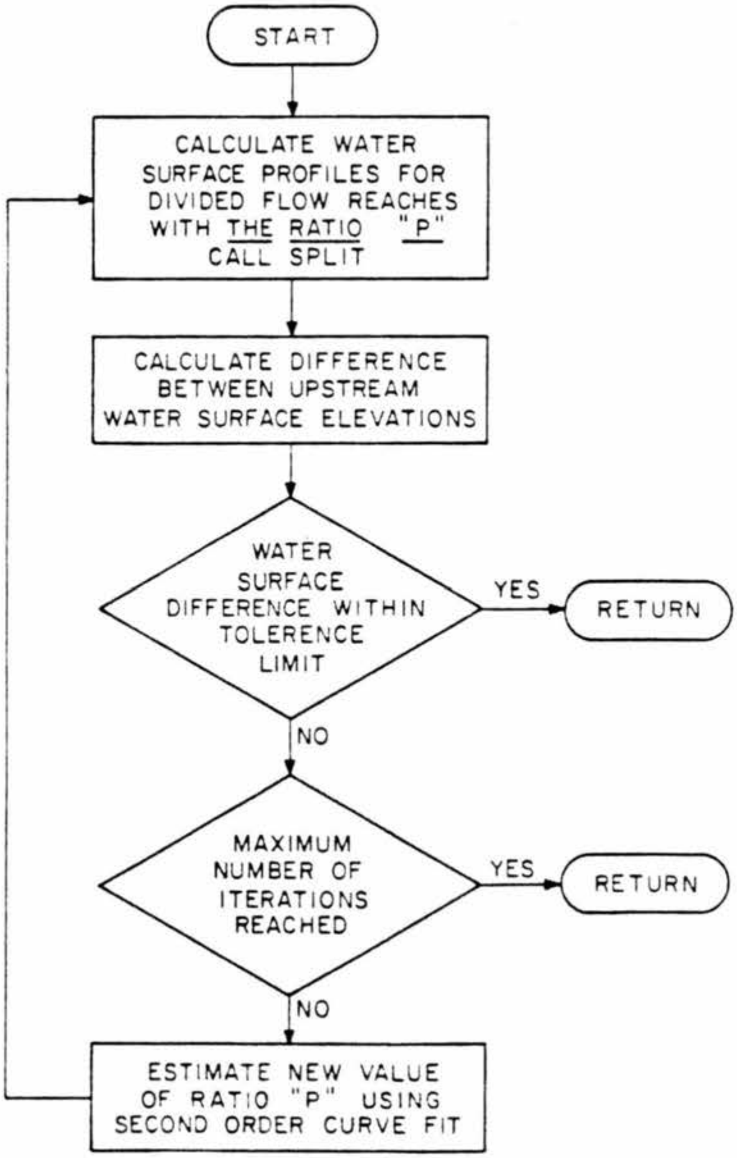


Subroutine CONT (continued)



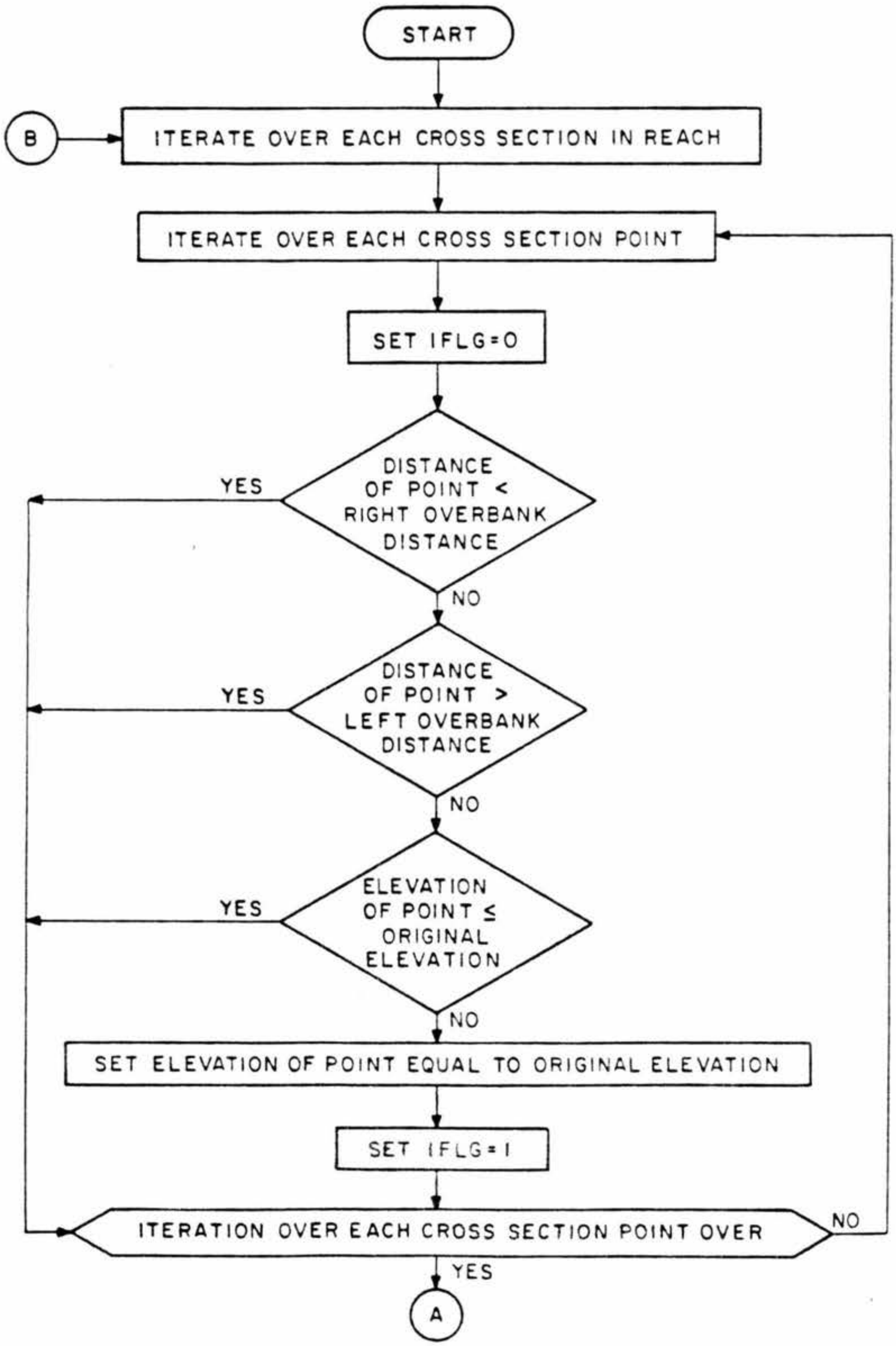
Subroutine DIVDE

This subroutine determines the fraction of the total flow going down each side of divided flow reaches.

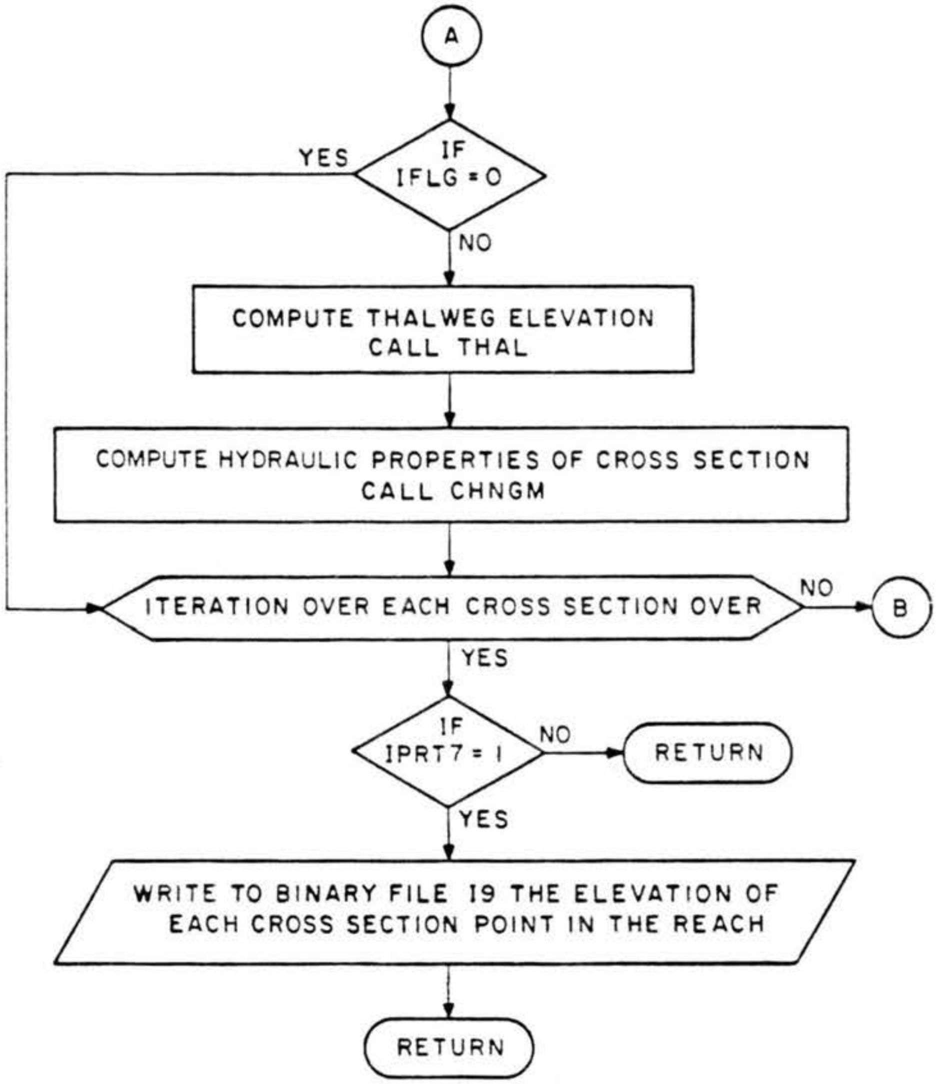


Subroutine DREDG

This subroutine simulates dredging by lowering each cross section in a reach to its original elevation (ZO). Only cross section points in the main channel are lowered.

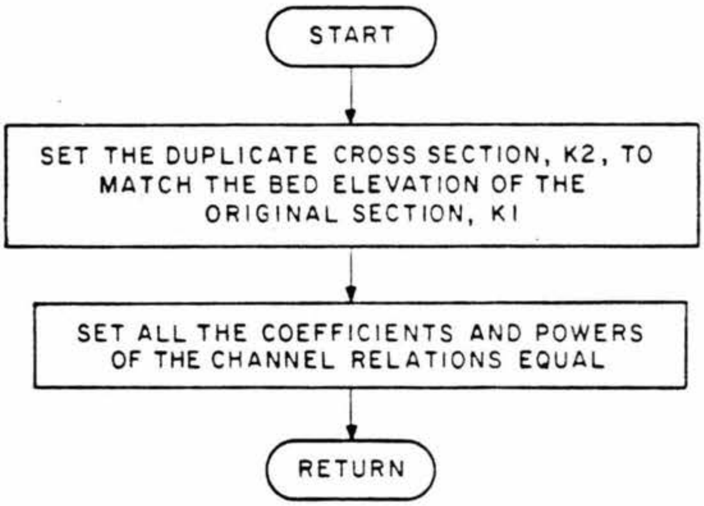


Subroutine DREDG (continued)



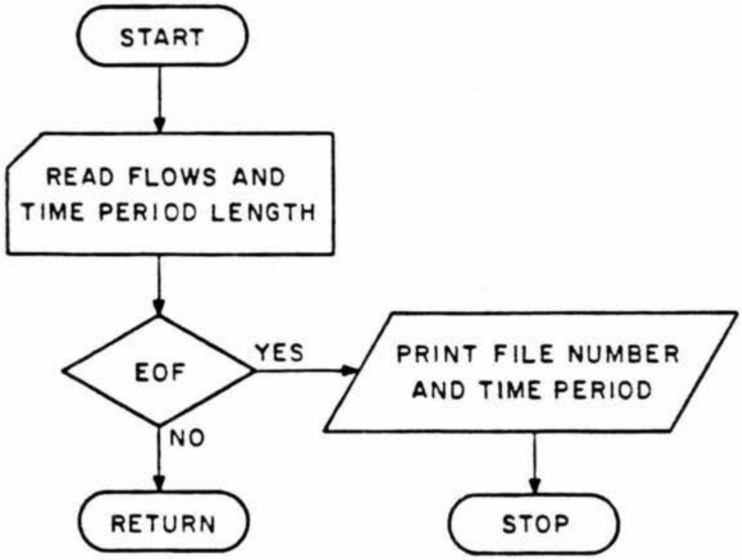
Subroutine DUP

This subroutine is used in divided flow problems when a cross section is used by two different river reaches. It changes the bed elevation of the duplicate cross section, K_2, to match the bed elevation of the original section, (K_1), after sediment routing.



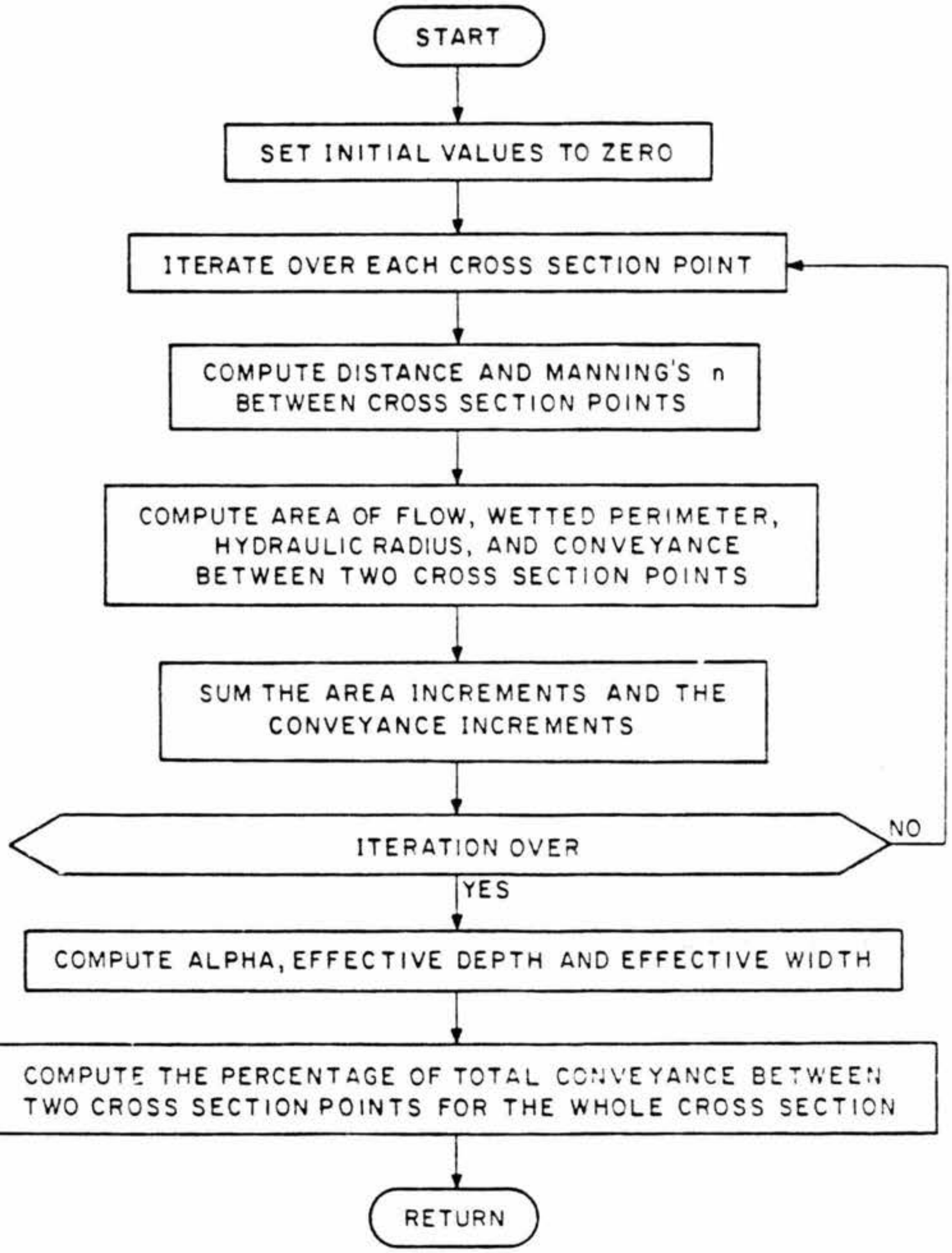
Subroutine FLOW

This subroutine calculates the water discharge at each cross section.



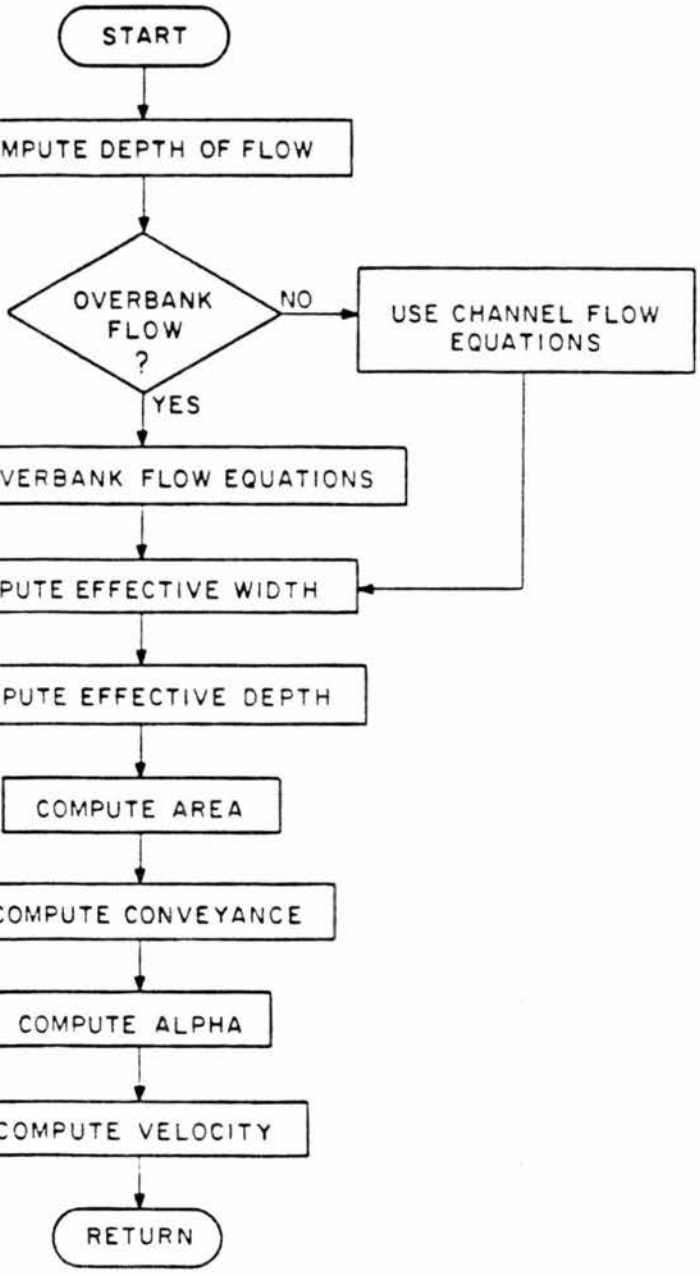
Subroutine GEOM

This subroutine calculates the exact hydraulic properties of a cross section, once given the channel geometry and the water surface elevation.



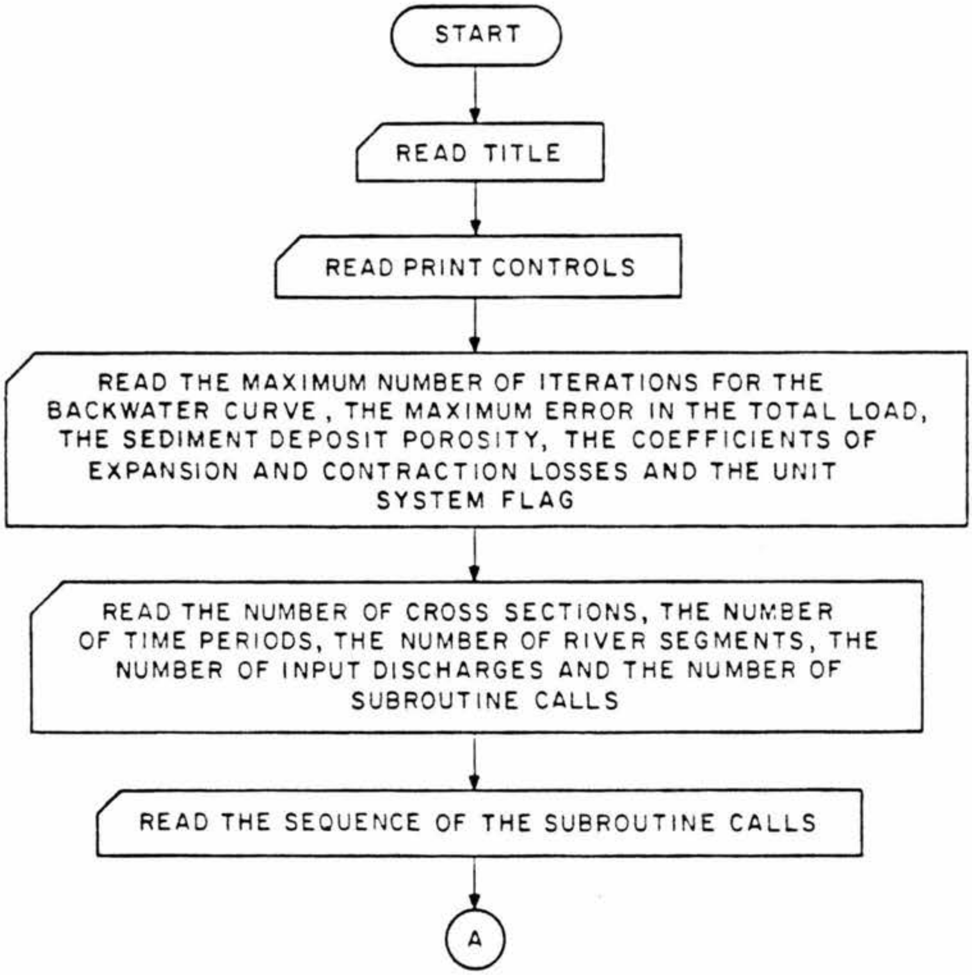
Subroutine HYDPR

This subroutine calculates the hydraulic properties of the (K)TH cross section given the water surface elevation (WS).

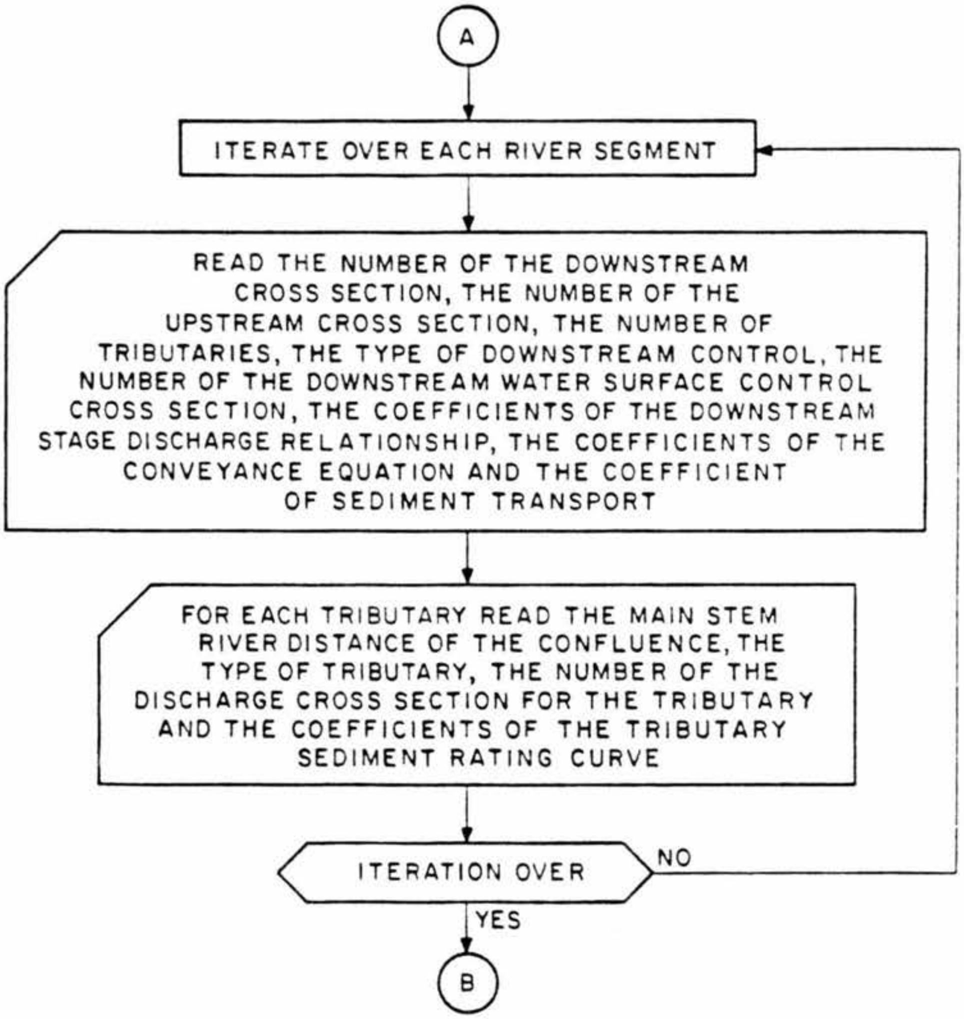


Subroutine IN1

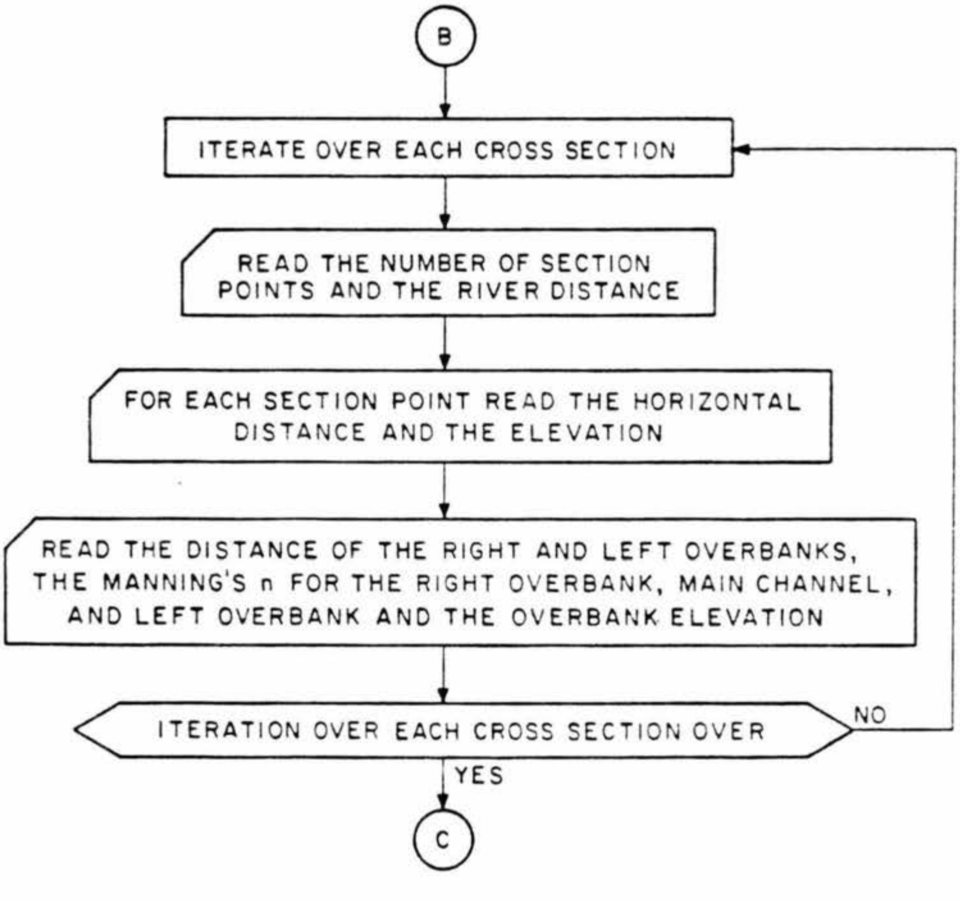
This subroutine reads in the sediment and geometry data.



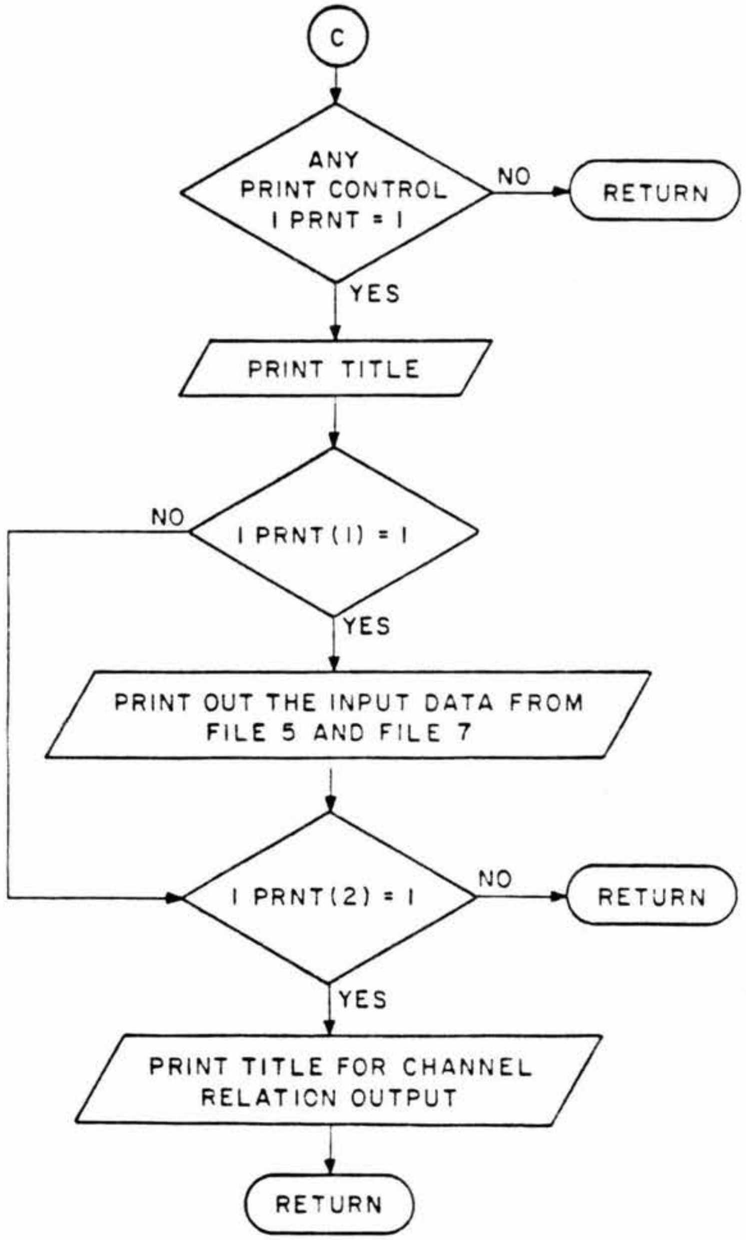
Subroutine IN1 (continued)



Subroutine IN1 (continued)

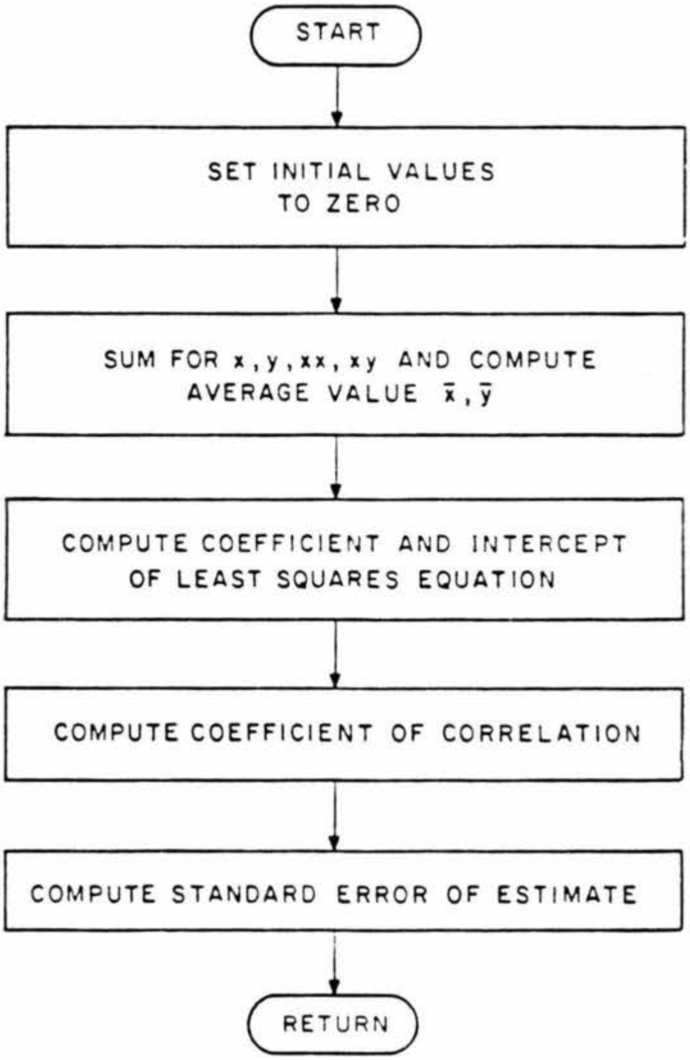


Subroutine IN1 (continued)



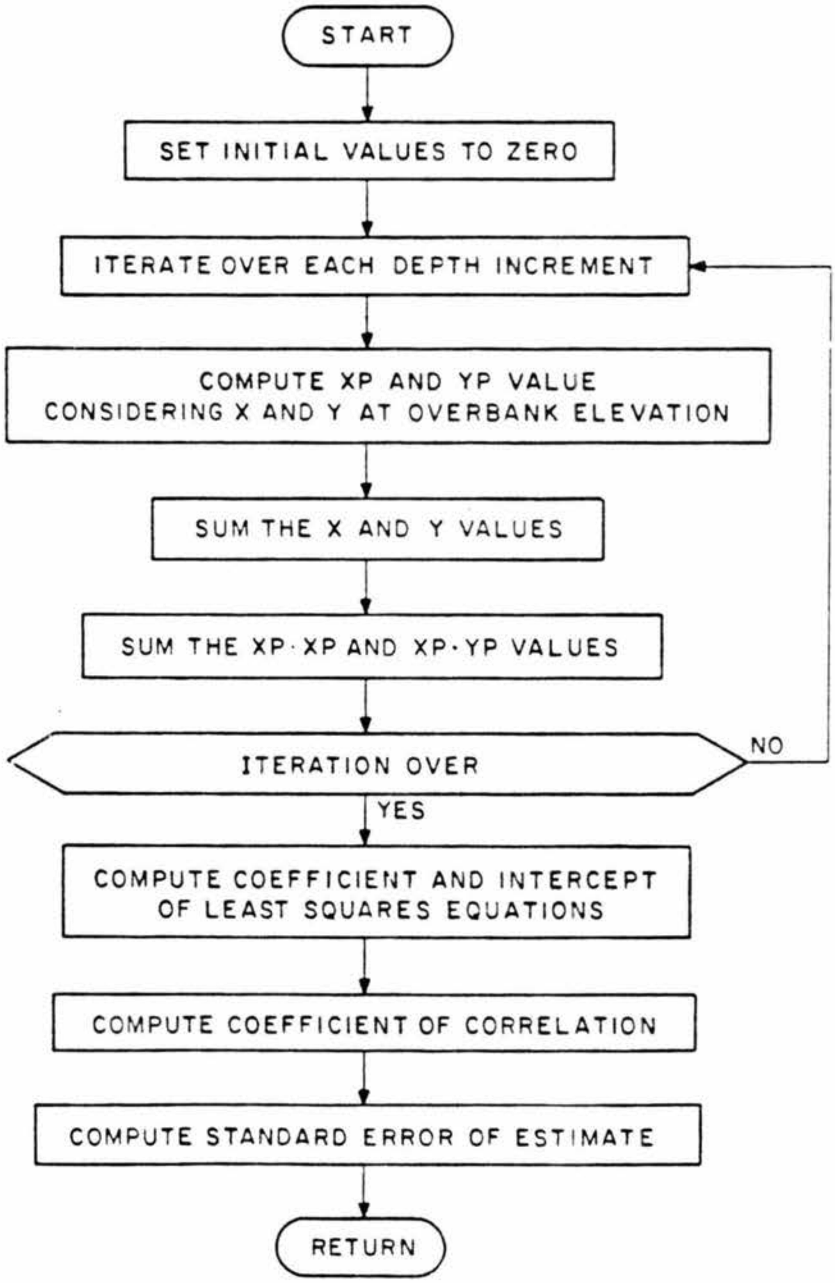
Subroutine LSQ

This subroutine derives the coefficients of the hydraulic power functions, by using a least squares regression.



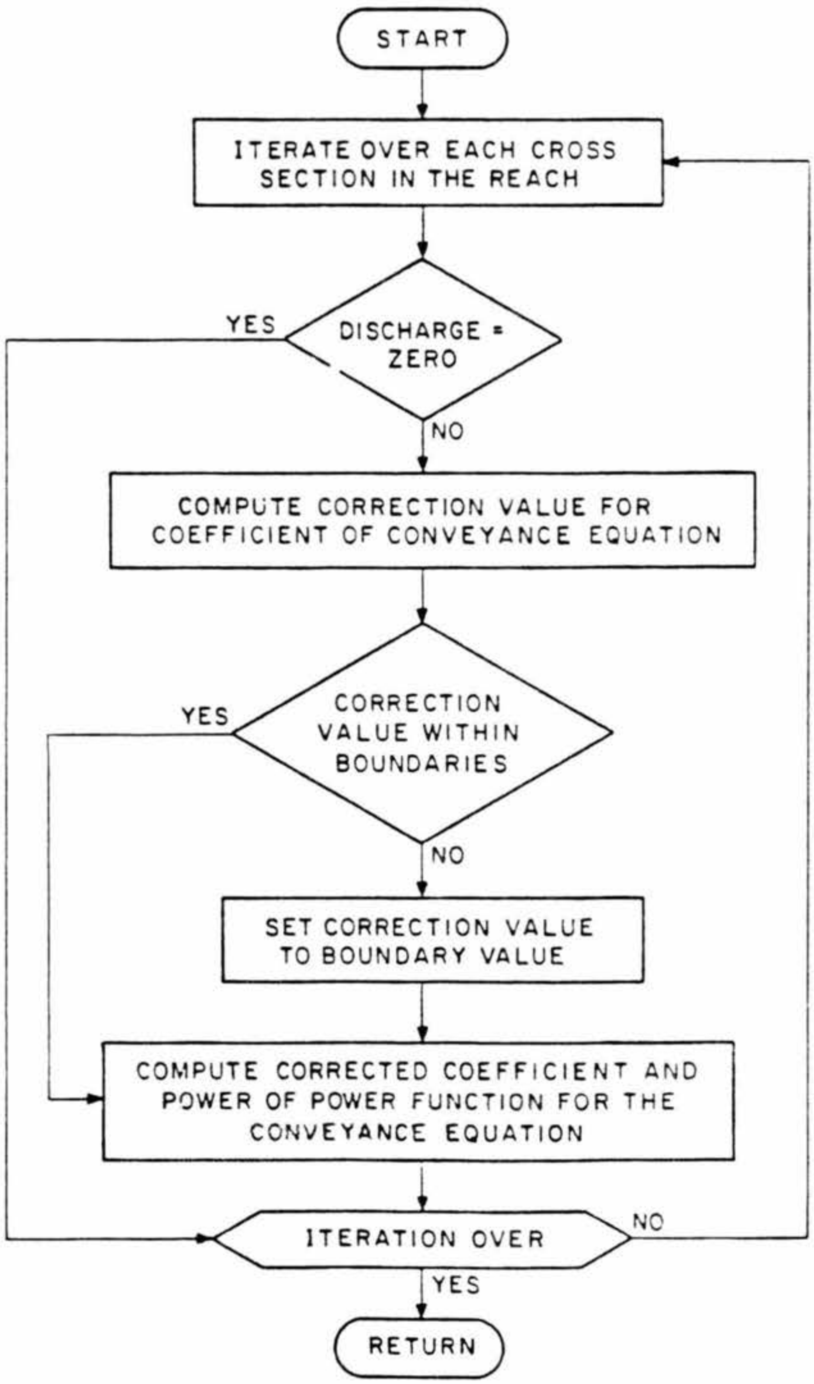
Subroutine LSQF

This subroutine derives the coefficients of the hydraulic power functions, for overbank flow, by using a least squares regression forced through the point (XO, YO).



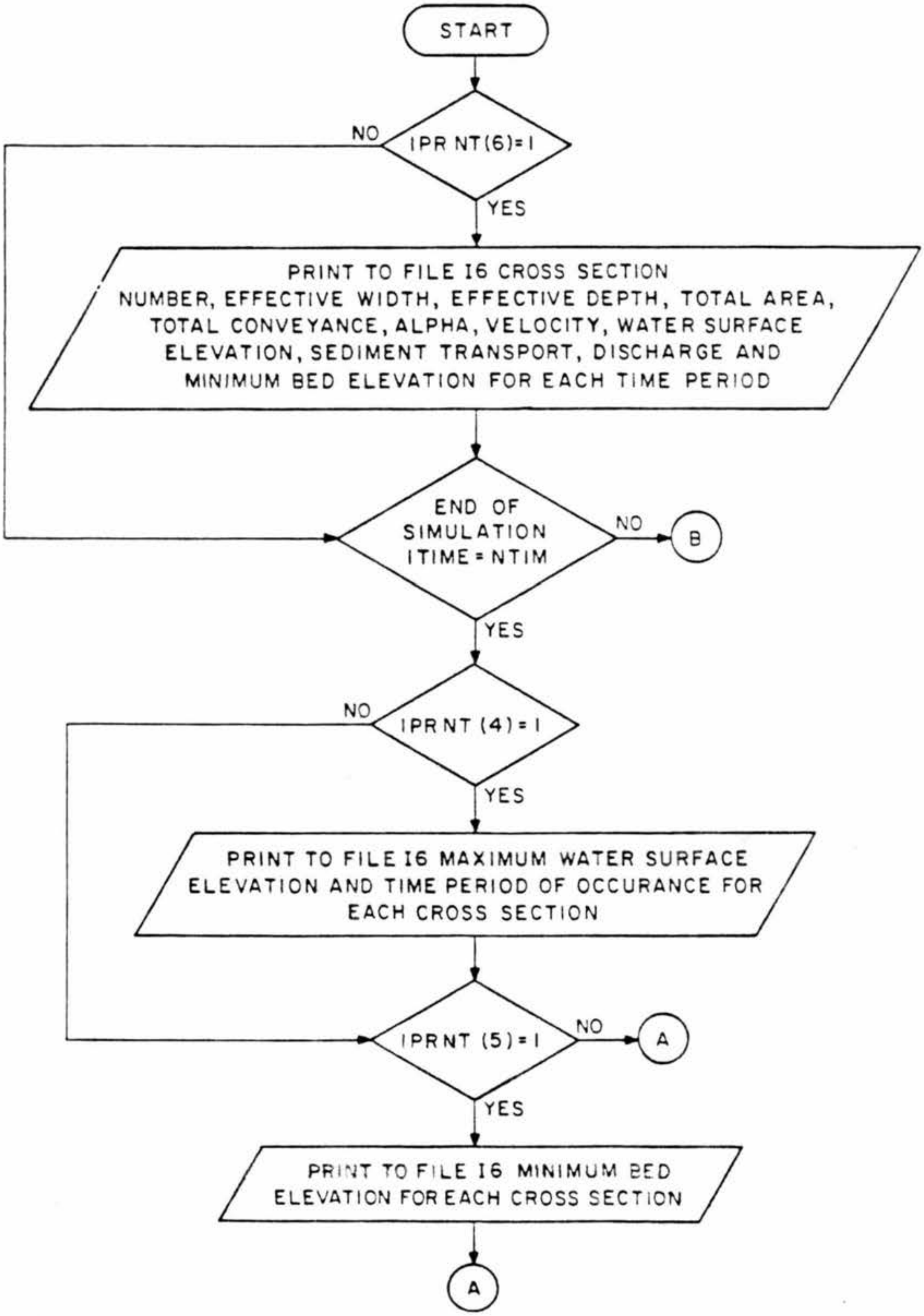
Subroutine NVAL

This subroutine calculates the coefficient of the conveyance equation for the current discharge. This allows Manning's n to be a function of discharge.



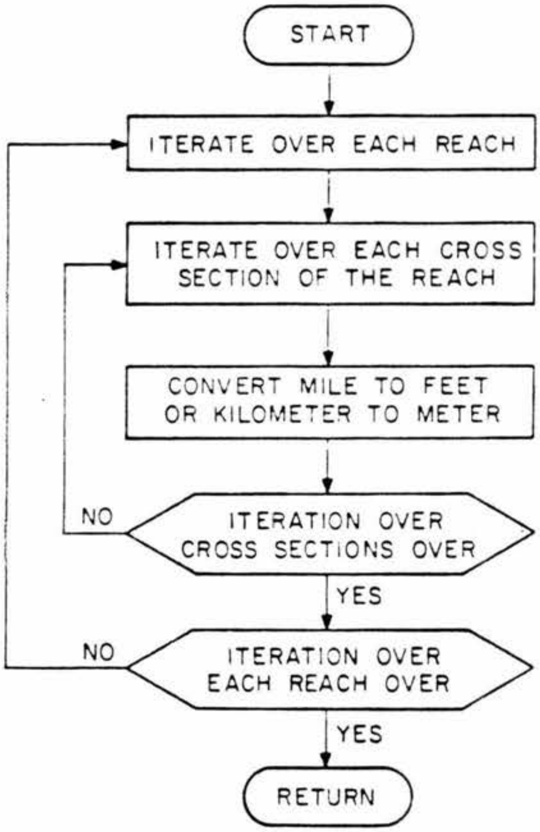
Subroutine OUT1

This subroutine outputs the various results of the simulation model.

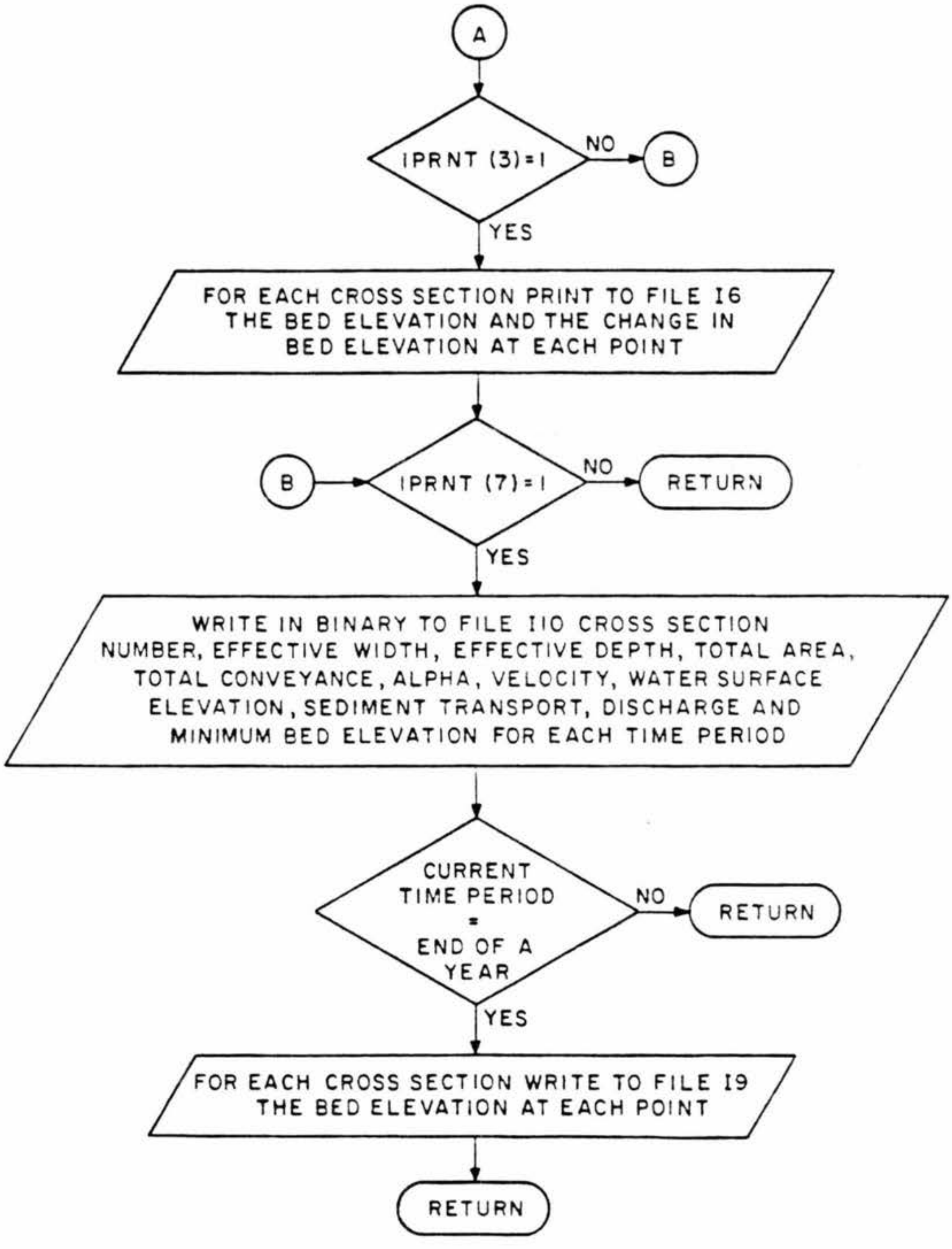


Subroutine RIVDS

This subroutine converts the river distance of each cross section and tributary to proper unit.

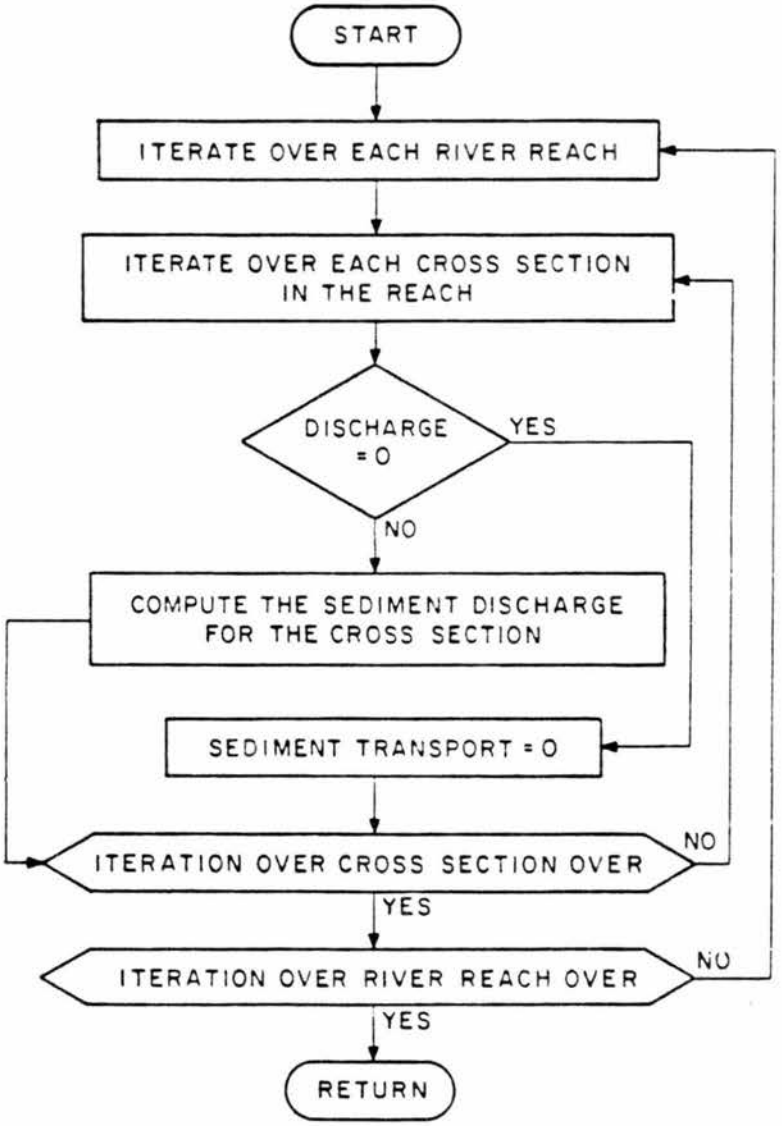


Subroutine OUT 1 (continued)



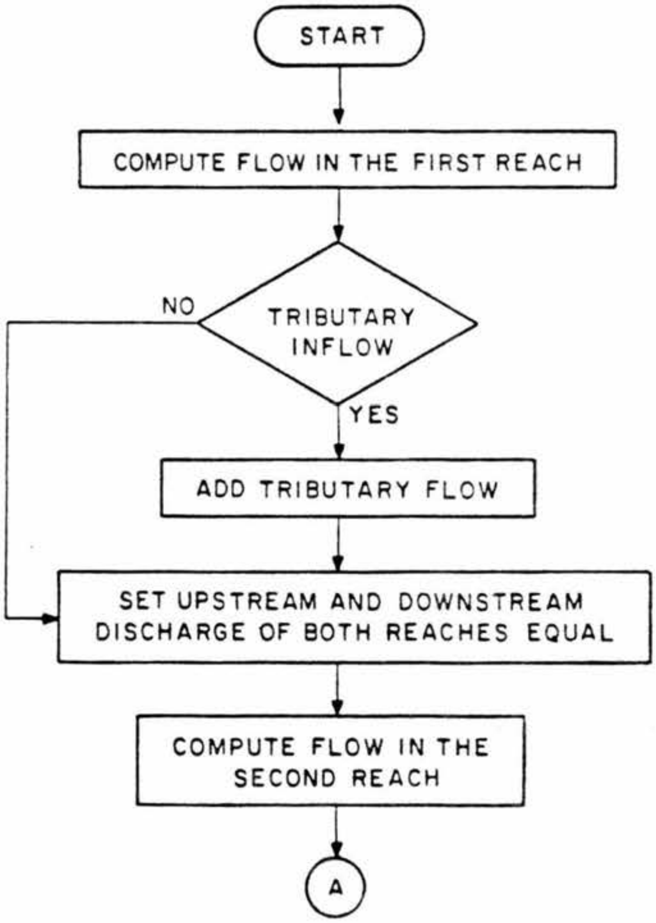
Subroutine SED

This subroutine calculates sediment transport using the generalized formula developed for the Yazoo River.

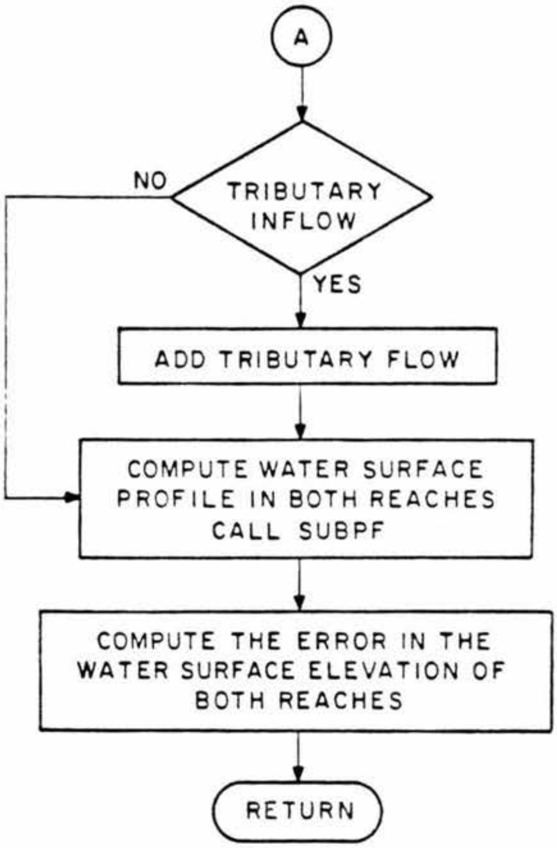


Subroutine SPLIT

This subroutine is used in divided flow problems to split the discharge between two channels.

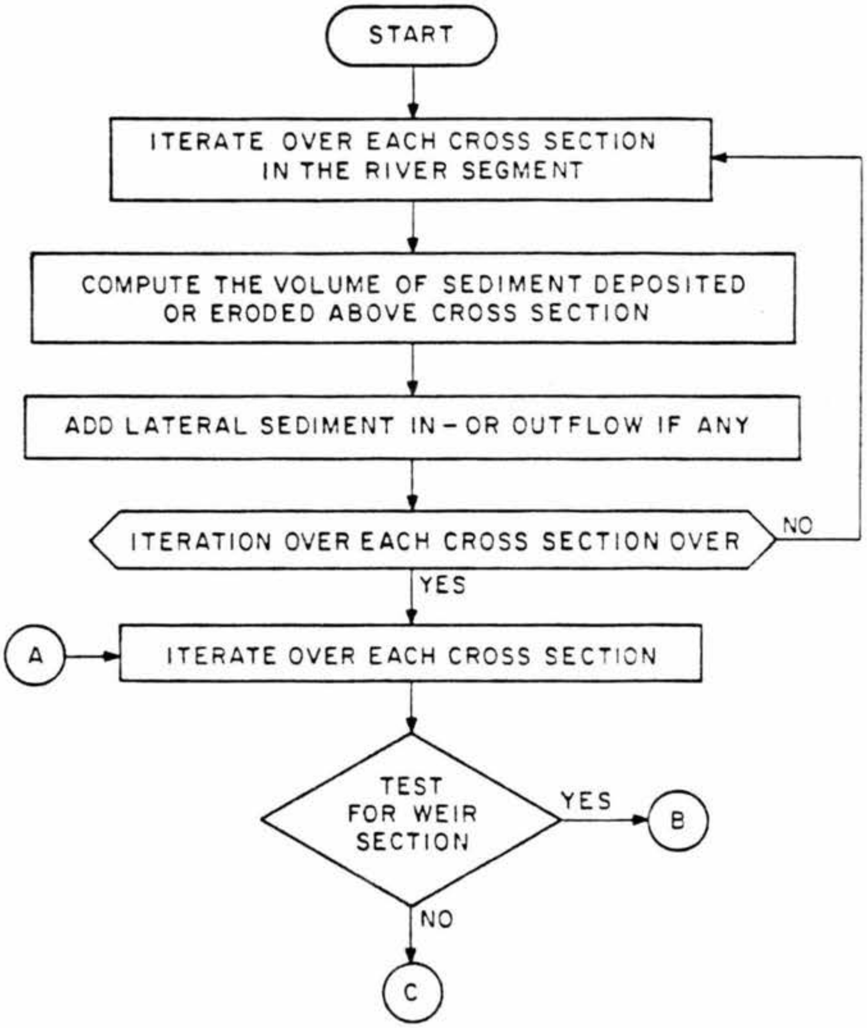


Subroutine SPLIT (continued)

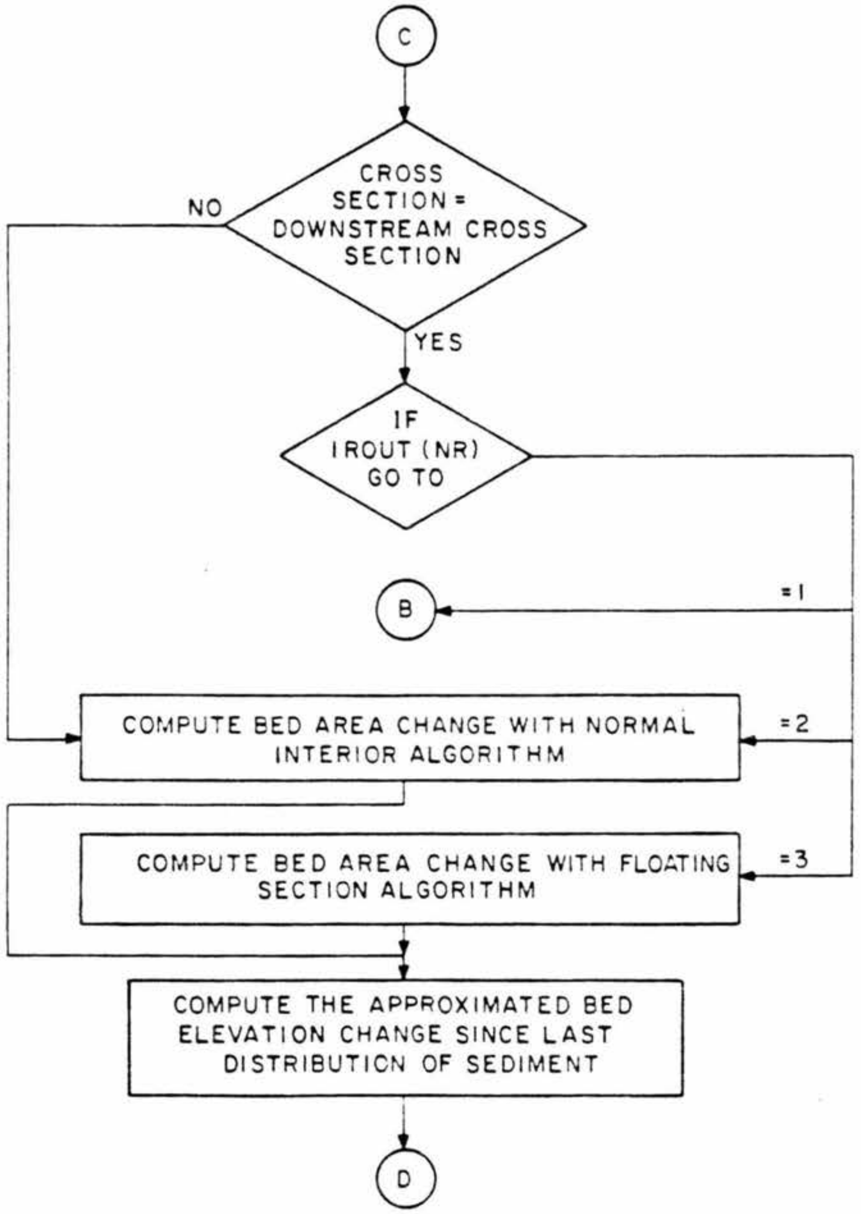


Subroutine SROUT

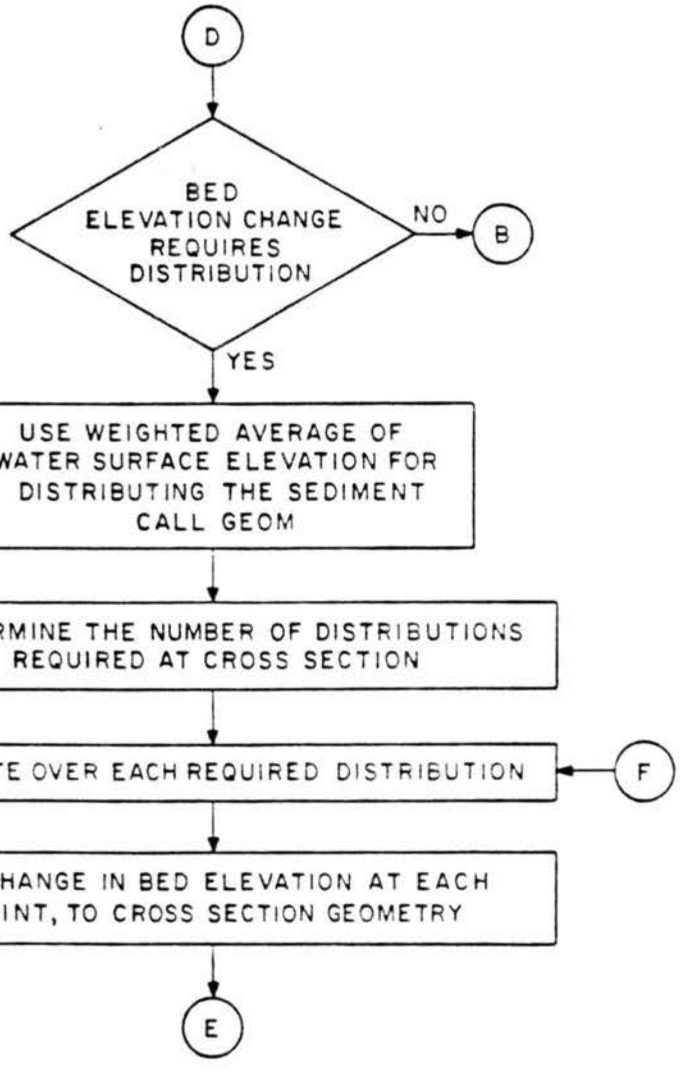
This subroutine routes the sediment, calculates the approximate bed elevation change, and if necessary, distributes the aggradation or degradation through the cross section.



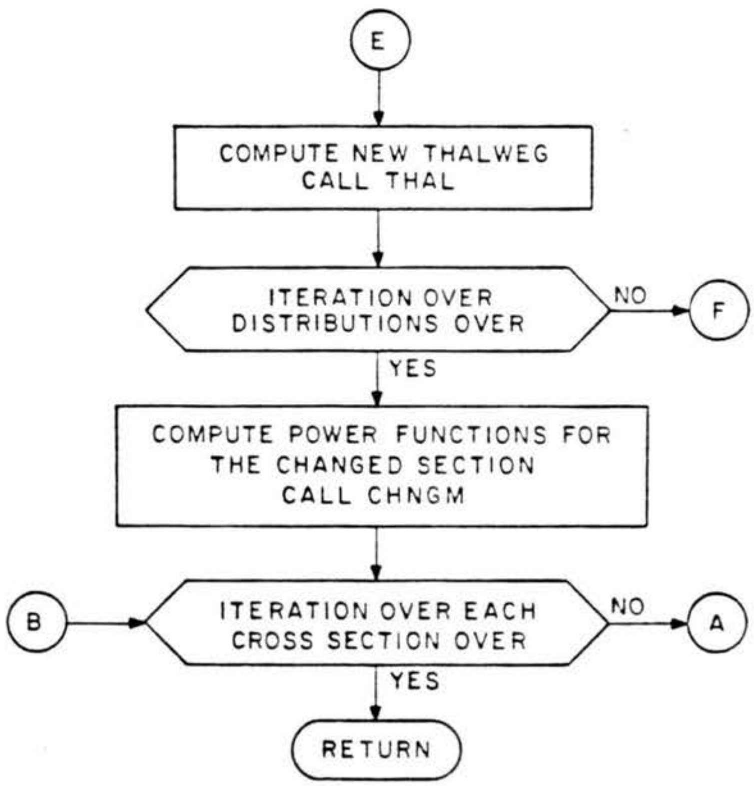
Subroutine SROUT (continued)



Subroutine SROUT (continued)

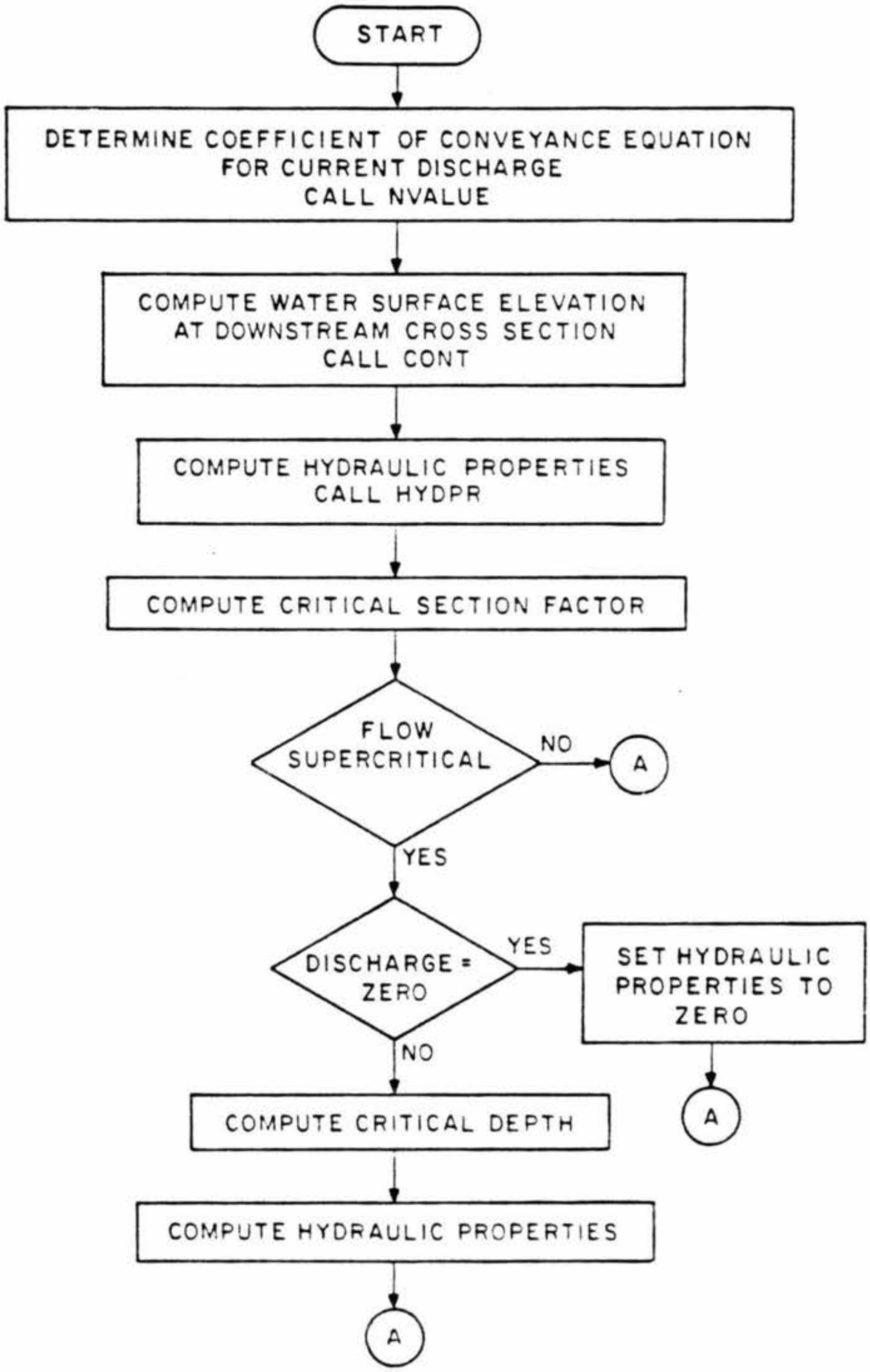


Subroutine SROUT (continued)

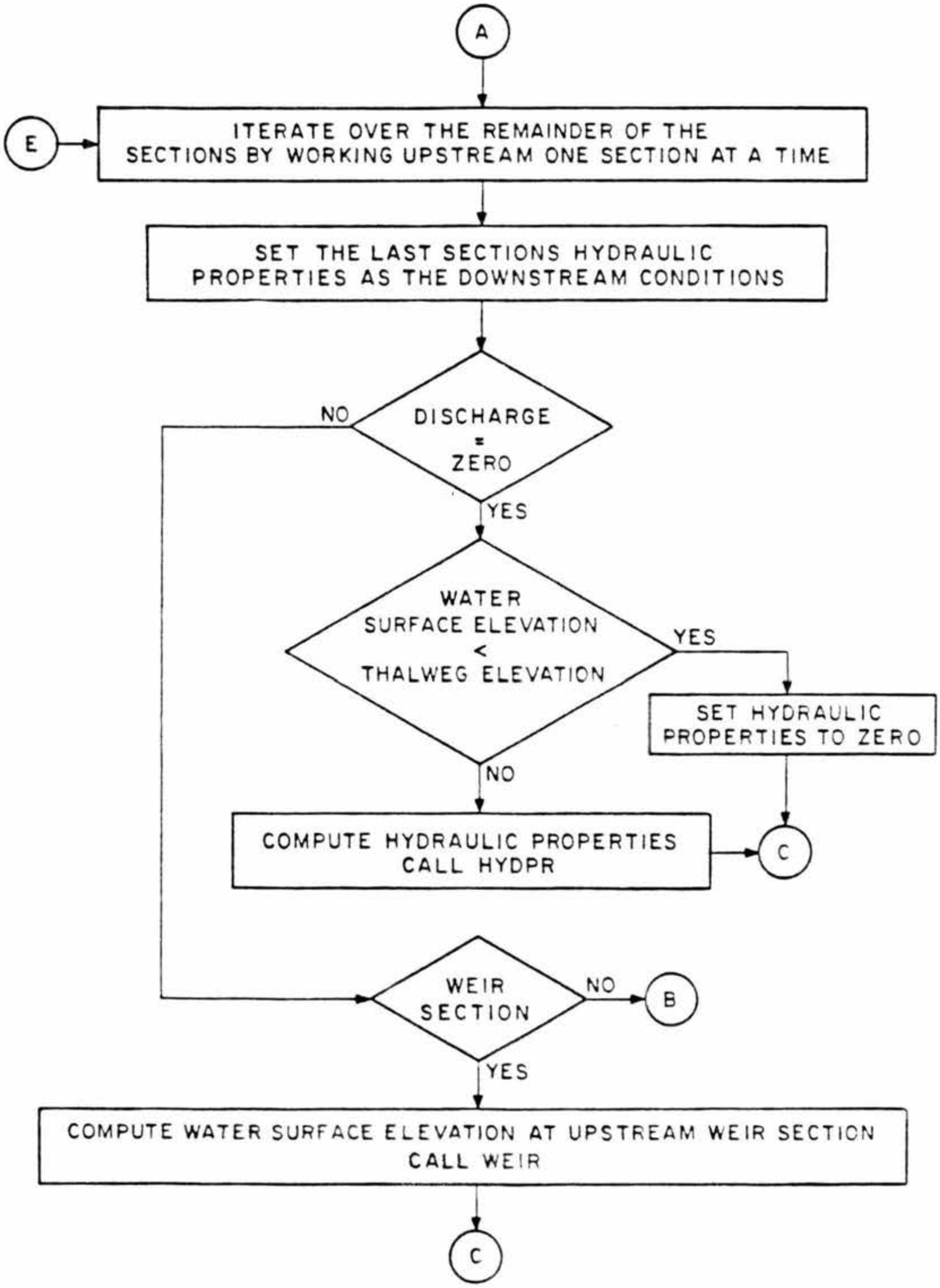


Subroutine SUBPF

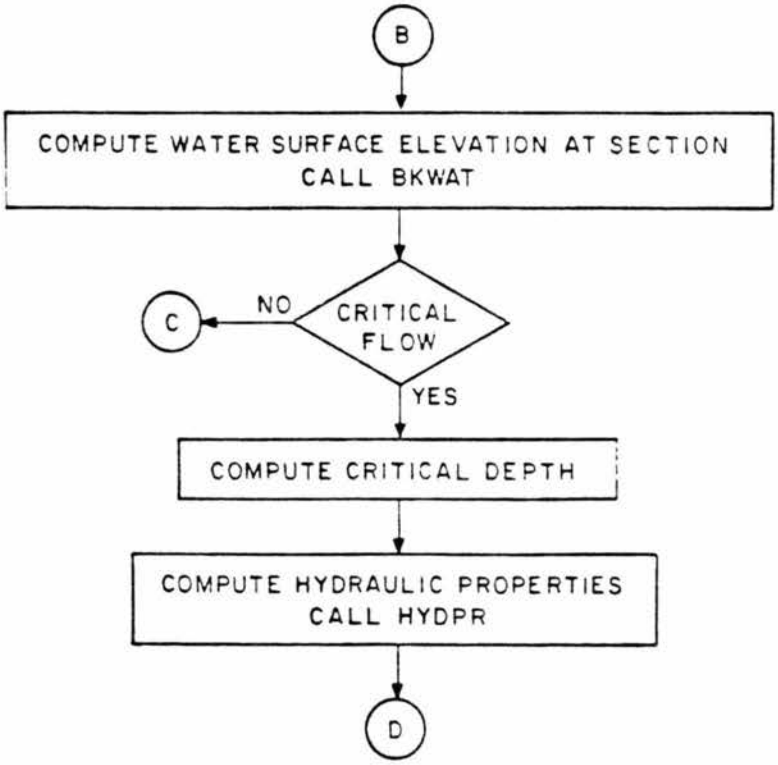
This subroutine calls the various other subroutines needed to calculate the subcritical water surface elevation at each section.



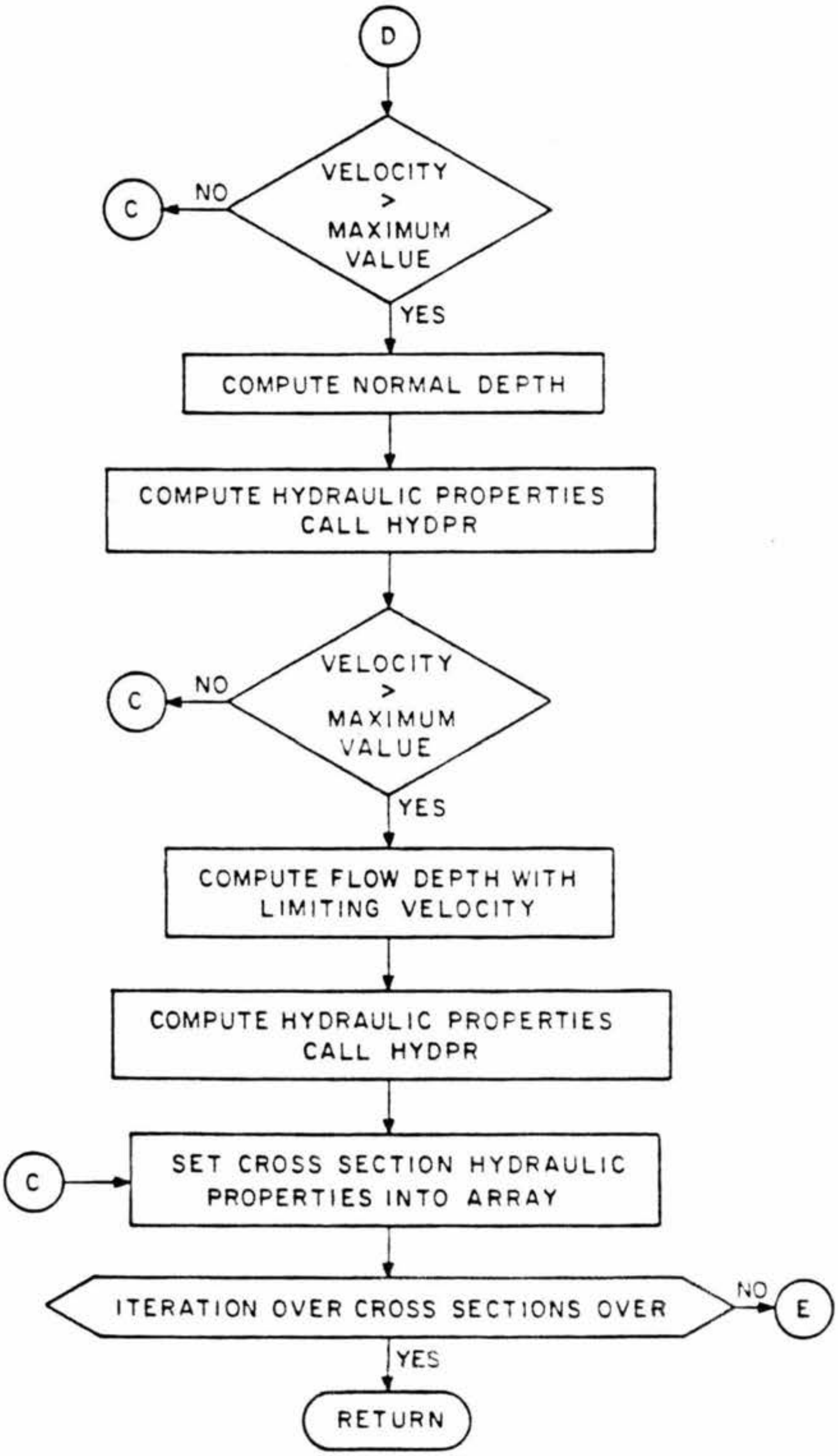
Subroutine SUBPF (continued)



Subroutine SUBPF (continued)

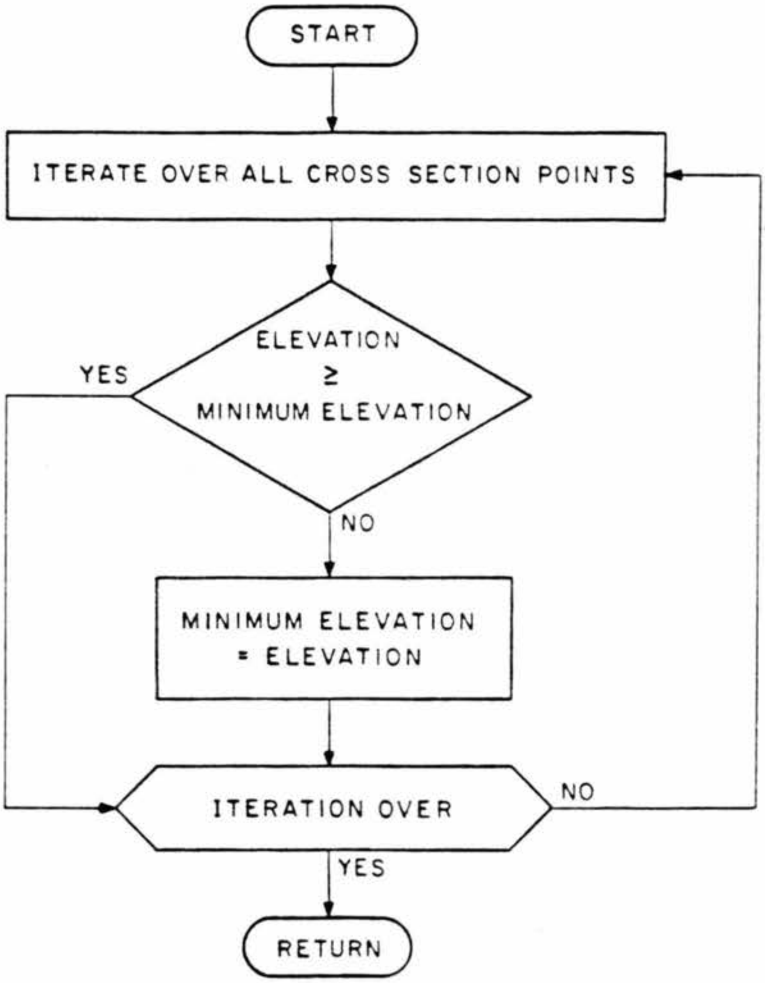


Subroutine SUBPF (continued)



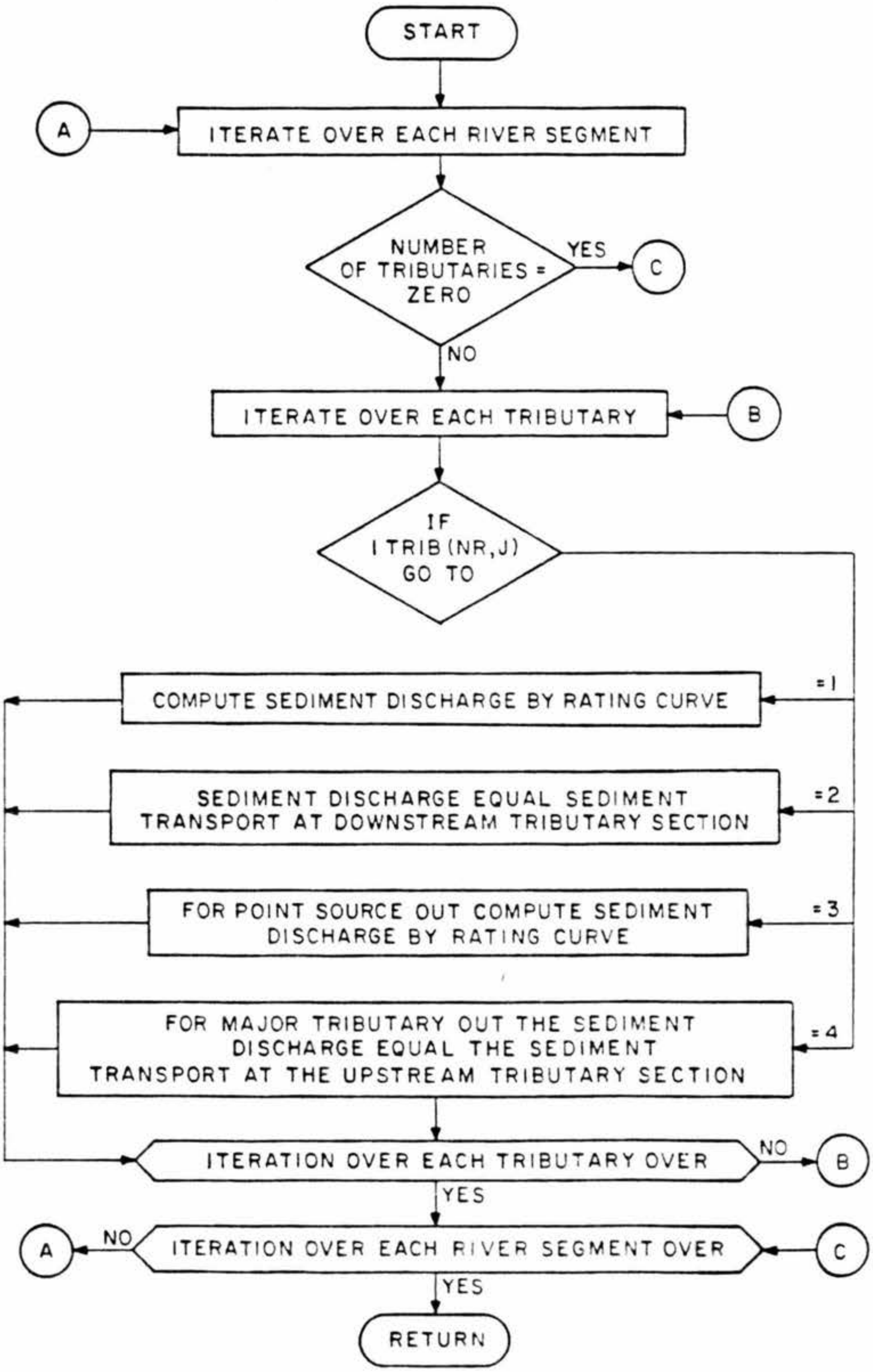
Subroutine THAL

This subroutine determines the cross section thalweg elevation.



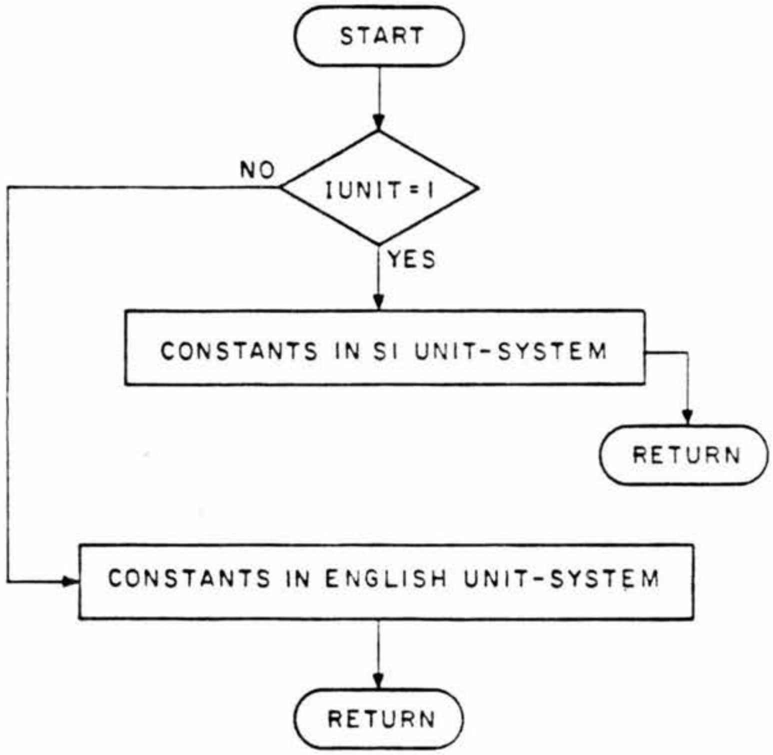
Subroutine TRIBS

This subroutine determines the sediment discharge for each tributary.



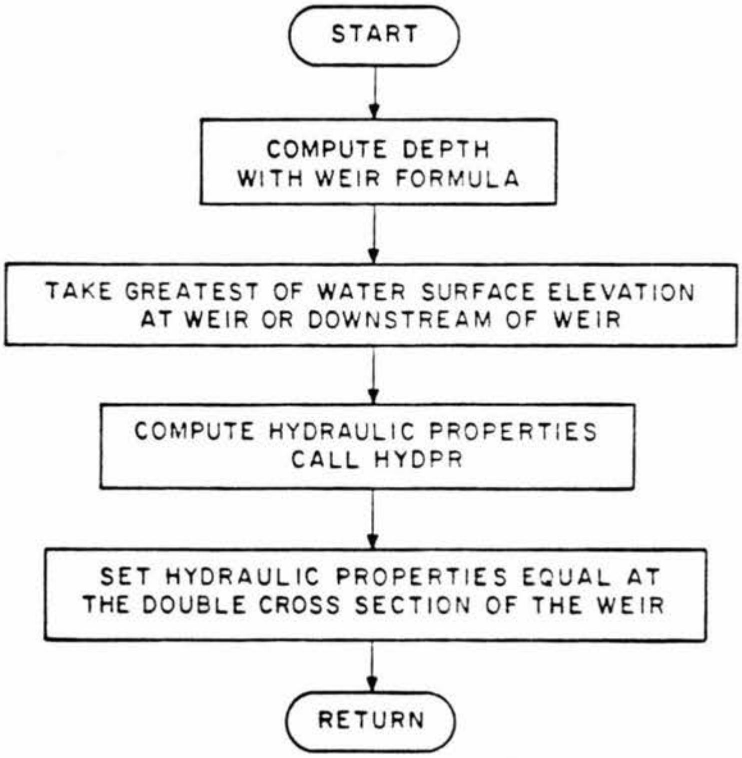
Subroutine UNIT

This subroutine assigns the correct values to the constants according to the unit-system used.



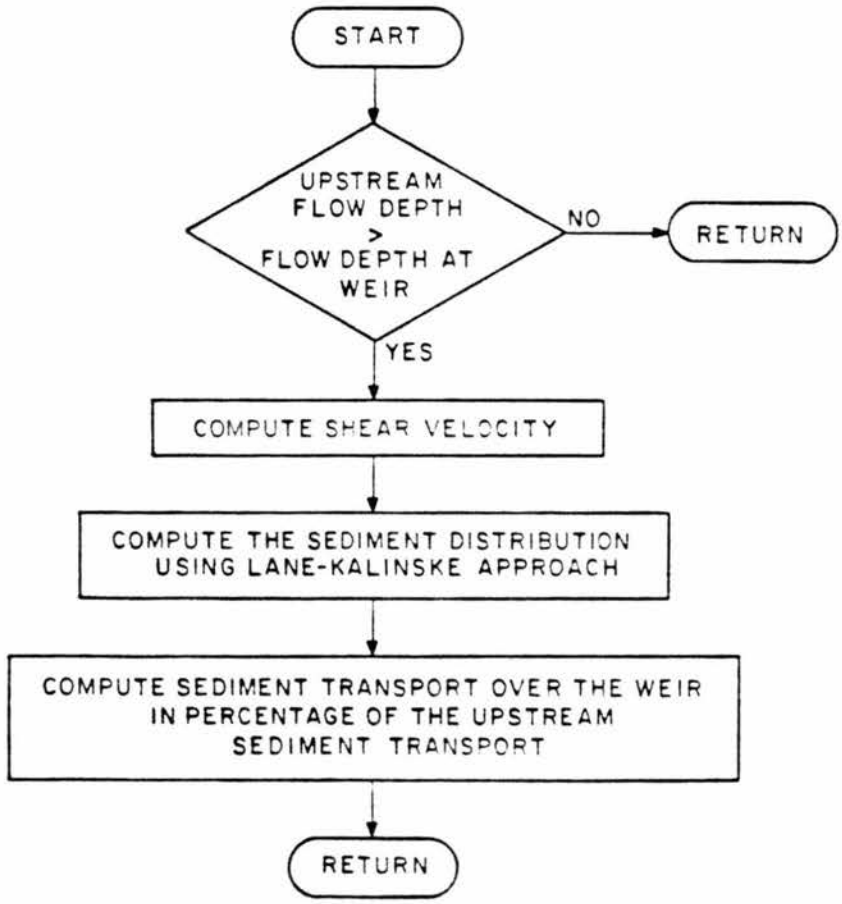
Subroutine WEIR

This subroutine is used to calculate the greater of the downstream water surfaces or the critical depth at the weir.



Subroutine WEIRS

This subroutine calculates the percentage of the upstream sediment transport (G(KU)), which is transported over a weir. It uses the Lane-Kalinske sediment distribution.



APPENDIX C

LIST OF PROGRAM VARIABLES

The following is a list of the variables used in the program KUWASER. For each variable there is a definition, common block name (if applicable) and array size. If the variable is not in a common block, the subroutine(s) in which the variable is used is shown in brackets under the definition. Terms in the definition which appear in parentheses are other variable names. If a variable's dimension is problem dependent, the array size is given as the name of a program variable that the array size should equal or exceed.

Variable	Array size	Common block	Definition
A			Area between two cross section points. [GEOM]
A			Value of intercept. [LSQ]
AALP	10		Log value of alpha (ALP). [CHNGM]
AD	10		Log value of depth. [CHNGM]
ADO			Log value of overbank depth. [CHNGM]
ADZ			Absolute value of approximate bed elevation change. [SROUT]
AED	10		Log value of effective depth (ED). [CHNGM]
AEW	10		Log value of effective width (ED). [CHNCM]
ALP		HYD	Velocity head correction coefficient, for the given cross section and for the given water surface elevation.
ALPK	NSEC	SEC1	Storage array for velocity head correction coefficient.
AN	NRIV	RIV	Coefficient that changes Manning's N value as function of discharge.
AT	NTRIB	RIV	Coefficient of tributary sediment rating curve.
ATA	10		Log value of total area (TA). [CHNGM]
ATAO			Log value of area at overbank elevation. [CHNGM]
ATEMP			Temporary value used in equation of first derivative in Newton-Raphson approximation. [BKWAT]
ATK	10		Log value of total conveyance (TK). [CHNGM]
ATKO			Log value of conveyance at overbank. [CHNGM]

Variable	Array size	Common block	Definition
AX	NRIV	RIV	Coefficient of stage-discharge relationship.
A1	NSEC	SEC1	Coefficient of hydraulic power function for effective width in main channel.
A2	NSEC	SEC1	Coefficient of hydraulic power function for effective depth in main channel.
A3	NSEC	SEC1	Coefficient of hydraulic power function for total area in main channel.
A4	NSEC	SEC1	Coefficient of hydraulic power function for total conveyance in main channel.
A5	NSEC	SEC1	Coefficient of hydraulic power function for alpha in main channel.
A6	NSEC	SEC1	Coefficient of hydraulic power function for effective width for overbank flow.
A7	NSEC	SEC1	Coefficient of hydraulic power function for effective depth for overbank flow.
A8	NSEC	SEC1	Coefficient of hydraulic power function for total area for over-bank flow.
A9	NSEC	SEC1	Coefficient of hydraulic power function for total conveyance for overbank flow.
A10	NSEC	SEC1	Coefficient of hydraulic power function for alpha for overbank flow.
B			Slope of the least squared regression (log value). [LSQ]
BDEN			Temporary value for calculation of first derivative in Newton-Raphson approximation. [BKWAT]
BN	NRIV	RIV	Power that changes Manning's N value as function of discharge.

Variable	Array size	Common block	Definition
BNUM			Temporary value for calculation of first derivative in Newton-Raphson approximation. [BKWAT]
BT	NTRIB	RIV	Power of tributary sediment rating curve.
BX	NRIV	RIV	Power of stage discharge relationship.
B1	NSEC	SEC1	Power of hydraulic power function for effective width in main channel.
B2	NSEC	SEC1	Power of hydraulic power function for effective depth in main channel.
B3	NSEC	SEC1	Power of hydraulic power function for total area in main channel.
B4	NSEC	SEC1	Power of hydraulic power function for total conveyance in main channel.
B5	NSEC	SEC1	Power of hydraulic power function for alpha in main channel.
B6	NSEC	SEC1	Power of hydraulic power function for effective width for overbank flow.
B7	NSEC	SEC1	Power of hydraulic power function for effective depth for overbank flow.
B8	NSEC	SEC1	Power of hydraulic power function for total area for overbank flow.
B9	NSEC	SEC1	Power of hydraulic power function for total conveyance for overbank flow.
B10	NSEC	SEC1	Power of hydraulic power function for alpha for overbank flow.
C			Conveyance. [GEOM], [CONT], [SUBPF]
CC		INF	Coefficient for contraction head loss.
CCE			Coefficient for contraction or expansion losses. [BKWAT]
CE		INF	Coefficient for expansion loss.

Variable	Array size	Common block	Definition
CONV		UNITS	Constant for English or metric units in Manning's N equation.
CORDS		UNITS	Correction factor to convert river distances from miles or kilometer, to feet respectively meters.
CRT			Critical section factor CRT = $Q/\sqrt{g/2}$. [SUBPF]
CX	NRIV	RIV	Constant of stage discharge relationship.
D			Depth used in first derivative calculation in Newton-Raphson approximation. [BKWAT]
D			Critical depth. [SUBPF], [WEIR]
D			Normal depth. [CONT]
DA			Average depth between cross section points. [GEOM]
DA			Change in bed area. [SROUT]
DEPTH			Cross section depth used in calculations for hydraulic properties. [HYDPR]
DER			Temporary derivative value. [BKWAT]
DEX		SYS	River distance between two cross sections.
DEXUP			River distance between cross section and next cross section upstream. [SUBPF]
DF			Darcy-Weisbach fiction factor. [WEIRS]
DIS			River distance. [KUWASER], [RIVDS]
DLOB			Station of left overbank. [IN1]
DO			Depth at overbank. [CHNGM]
DROB			Station of right overbank. [IN1]

Variable	Array size	Common block	Definition
DT		INF	Time period length in days.
DV	NSEC		Change in volume between cross sections. [SROUT]
DWS			Increment of water surface elevation. [CHNGM]
DX			Horizontal distance between cross section points. [SROUT]
DXDWN			River distance to downstream section. [SROUT]
DXUP			River distance to upstream section. [SROUT]
DZF	ND		Change in cross section point elevation. [OUT1]
DZMAX		UNITS	Maximum change in bed elevation allowed before distributing change.
EA			Intercept of the least squared regression (log scale). [LSQ]
ED		HYD	Effective depth for the given cross section and for the given water surface elevation.
EDK	NSEC	SEC1	Storage array for effective depth at cross section.
EL			Error in computed water surface elevation for left point. [DIVDE]
EO			Temporary value. [WEIRS]
EPS		INF	Maximum allowable error in total head used in backwater calculations.
ER			Error in total head for estimation. [BKWAT]
ER			Error in computed water surface elevation for right point. [DIVDE]
ERROR			Difference in water surface elevation in divided flow situation. [SPLIT], [DIVDE], [SPTGW]

Variable	Array size	Common block	Definition
EW		HYD	Effective width for the cross section and for the given water surface elevation.
EY			Temporary value. [WEIRS]
F	NSEC, ND	SEC2	Manning's N for point in a cross section.
FET		UNITS	Increment of water surface elevation.
FLOB			Manning's N for the left overbank. [IN1]
FM			Average Manning's N between cross section points. [GEOM]
FMC			Manning's N for main channel. [IN1]
FROB			Manning's N of right overbank. [IN1]
FX			Average x values in linear regression. [LSQ]
FY			Average y values in linear regression. [LSQ]
G	NSEC	SEC2	Sediment transport in cfs.
GC	NRIV	RIV	Coefficient of sediment transport for river sediment.
GRAV		UNITS	Value of gravitation acceleration.
H			Interval between points. [DIVDE]
HL			Head losses. [BKWAT], [BRIDGE]
HLV			Head losses due to contraction and expansion. [BKWAT]
I			Increment and loop counter.
ICALL	(NCALL, 5)	RIV	Sequence of subroutine calls.
ICAL1			ICAL1 = ICALL (NC, 1). [KUWASER]
ICAL2			ICAL2 = ICALL (NC, 2). [KUWASER]
ICANT			ICANT = ICONT (NR). [CONT]

Variable	Array size	Common block	Definition
ICONT	NRIV	RIV	Type of downstream control.
			= 1 stage-discharge relationship.
			= 2 stage-hydrograph.
			= 3 downstream water surface.
			= 4 greatest of #1 and #2.
			= 5 greatest of #1 and #3.
			= 6 Normal depth.
ICRI			Critical flow flag. [BKWAT], [SUBPF]
			ICRI = 0 subcritical flow.
			ICRI = 1 supercritical flow.
IDRG			Flag for dredging.
			IDRG = 0 no dredging.
			IDRG = 1 dredging.
IDUMB			Dumbie variable. [KUWASER]
IFLAG			Flag for convergence of divided flow calculations. [KUWASER], [DIVDE], [GINBND], [SPLIT], [SPTGW]
			= 1 solution found.
			= 2 no solution found.
			= 3 solution bounded on maximum.
			= 4 solution bounded on minimum.
IFLAG			Flag for over bank flow. [GEOM]
IFLG			Flag for dredging. [DREDG]
II			Increment and loop counter. [GEOM]
IMAX	NSEC	SEC1	Time period of maximum water surface at section.
IM1			Increment and loop counter. [GEOM]

Variable	Array size	Common block	Definition
IPRNT	8	PRT	Print controls. IPRINT (M1) = 0 or 1. If equal 0 no printout, if equal to 1 printout.
			(1) All input data read from files I5 and I7 are printed.
			(2) Coefficients, correlation coefficient, and standard error of each of the hydraulic property equations are printed for each cross section.
			(3) The final elevation and change in elevation at each cross section point are printed.
			(4) Maximum water surface elevation and the time period of occurrence are printed for each cross section.
			(5) Minimum bed elevation at each cross section is printed at the end of simulation.
			(6) Cross section properties (time period, effective width, effective depth, total area, total conveyance, alpha, velocity, water surface, elevation, discharge) are printed.
			(7) Binary output for all data.
			(8) Error messages are printed.
IROOT			IROOT = IROUT (NR). [SROUT]
IROUT	NRIV	RIV	Type of downstream cross section sediment routing or river reach.
			IROUT = 1 fixed, no routing.
			IROUT = 2 cross section downstream, normal routing.
			IROUT = 3 floating, no section downstream
ITIME		INF	Current time period.

Variable	Array size	Common block	Definition
ITRIB	NRIV, NTRIB	RIV	Type of tributary.
			= 1 point source in.
			= 2 major tributary in (considered as a segment).
			= 3 point source out.
			= 4 major tributary out (considered as a segment).
ITRYB			ITRYB = ITRIB (NR, J). [TRIBS]
IUNIT		UNITS	Value for selecting unit-system.
			IUNIT = 1 English.
			IUNIT = 2 Metric-SI.
IZMIN	NSEC	SEC2	Cross section thalweg point.
I5		PRT	File used for general data input.
I6		PRT	File used for output to printer.
I7		PRT	File used for cross sectional data input.
I8		PRT	File used for discharge data input.
I9		PRT	File used for binary output.
I10		PRT	File used for binary output.
J			Increment and loop counter.
JT			Increment and loop counter.
K			Increment and loop counter for cross section number.
K1			Cross section number. [DUP]
K2			Cross section number. [DUP]
KCONT	NRIV	RIV	Number of sections which water surface control.

Variable	Array size	Common block	Definition
KD			Number of downstream cross section. [KUWASER], [CONT], [FLOW], [NVAL], [SUBPF]
KDC			Downstream cross section in cut-off. [SPTGW]
KDCP1			KDPC1 = KDC + 1. [SPTGW]
KDOWN	NRIV	RIV	Downstream cross section for river segment.
KDP1			KDP1 = KD + 1. [SPLIT], [SUBPE], [SROUT]
KDT			Downstream cross section for Tallahatchie River. [STPGW]
KDTP1			KDTP1 = KDT + 1. [STPGW]
KDTR			Downstream cross section for reverse flow Tallahatchie River. [SPTGW]
KDY			Downstream cross section for Yalobusha. [SPTGW]
KDYP1			KDYP1 = KDY + 1. [STPGW]
KDP1			KDP1 = KD +1. [SPLIT], [SYST]
KD1			Number of downstream cross section of first river segment in divided flow situation. [SPLIT]
KD2			Number of downstream cross section of second river segment in divided flow situation. [SPLIT]
KFLAG			Flag for reverse flow in Tallahatchie. [GWBND]
KI			Increment and loop counter. [OUT1]
KM1			KM1 = K - 1. [SUBPF], [WEIR], [SROUT]
KOUNT			Iteration counter. [BKWAT], [DIVDE]
KP1			KP1 = K + 1. [SUBPT], [SROUT]

Variable	Array size	Common block	Definition
KT			KT = KTRIB (NR, J). [FLOW], [SPLIT]
KTR			
KTRIB	NRIV, NTRIB	RIV	Discharge cross section for tributary.
KU			Number of upstream cross section. [KUWASER], [FLOW], [NVAL], [SUBPF], [SED], [SROUT], [WEIR]
KUC			Number of upstream cross section in cut-off. [SPTGW]
KUM1			KUM1 = KU - 1. [SPLIT], [SROUT]
KUP	NRIV	RIV	Upstream cross section for river segment.
KUT			Number of upstream cross section in Tallahatchie River. [GWBND], [SPTGW]
KUTR			Number of upstream cross section in reversed Tallahatchie River. [GWBND], [SPTGW]
KUY			Number of upstream cross section in Yazoo River. [SPTGW]
KU1			Number of upstream cross section for the first river segment in divided flow situation. [DIVDE], [SPLIT]
KU2			Number of upstream cross section for the second river segment in divided flow situation. [DIVDE], [SPLIT]
KW			Weir cross section number. [WEIRS]
KWM1			KW - 1. [WEIRS]
KYAL			Downstream section on Yalobusha. [STPGW]
L			Increment and loop counter for cross sectional points.

Variable	Array size	Common block	Definition
M			Number of cross section points M = ND (K). [IN1], [CHNGM], [KUWASER]
MST		WF	Maximum number of iterations for backwater curve.
M1			Loop counter. [IN1]
M2			Loop counter. [IN1]
N			Increment and loop counter.
NAP			Number of water surface elevations used for linear regression of hydraulic-properties relationship. [CHNGM]
NC			Increment and loop counter. [KUWASER]
NCALL		INF	Number of subroutine calls for each time period.
ND	NSEC	SEC2	Number of points in each cross section.
NDIS			Number of sediment distributions. [SROUT]
NDS			Loop counter. [SROUT]
NN			Increment and loop counter. [IN1]
NP			Number of incremental subareas. [GEOM], [SROUT]
NQI		INF	Number of discharge sections.
NR			Increment and loop counter for number of river segment.
NRC			River segment number for Ft. Pemberton cut-off. [GWBND], [SPTGW]
NRIV		RIV	Number of river segments.
NR1			Number of first river segment in divided flow situation. [DIVDE], [SPLIT]

Variable	Array size	Common block	Definition
NR2			Number of second river segment in divided flow situation. [DIVDE], [SPLIT]
NR3			River segment number. [DIVDE], [SPLIT]
NR4			River segment number. [DIVDE], [SPLIT]
NRM1			NRM1 = NR - 1. [KUWASER], [NVAL]
NRT			River segment number for Tallahatchie. [SPTGW], [GWBND]
NRTR			River segment number for reverse Tallahatchie. [SPTGW], [GWBND]
NRY			River segment number for Yalobusha. [SPTGW], [GWBND]
NSEC		INF	Number of cross sections.
NT			NT = NTRIB (NR). [IN1], [FLOW], [KUWASER], [SPLIT], [TRIBS]
NTIM		INF	Number of time periods.
NTRIB	NRIV	RIV	Number of tributaries for each river segment.
OA4	NSEC	SEC1	Initial value of A4.
OA9	NSEC	SEC1	Initial value of A9.
P			Wetter perimeter. [GEOM]
P			Distribution factor for divide flow situation. [KUWASER], [DIVDE], [IN1], [SPLIT], [GWBND], [SPTGW]
PC			Percentage of sediment going over the weir. [WEIRS]
PMAX			Upper limit for divided flow ratio P. PMAX = 1. [DIVDE]
PMIN			Lower limit for divided flow ratio P. PMIN = 0. [DIVDE]

Variable	Array size	Common block	Definition
PNR	NRIV		Distribution factor for divided flow situation. [KUWASER]
QSL	NRIV, NTRIB	RIV	Tributary sediment discharge
QTAL			Tallahatchie discharge. [SPTGW]
QYAL			Yalobusha discharge. [SPTGW]
R			Hydraulic radius. [GEOM]
RC			Correlation coefficient of hydraulic properties relationships. [CHNGM],[LSQ], [LSQF]
RCK	10		Correlation coefficient for hydraulic properties relationships. [CHNGM]
RD	NSEC	SEC1	River distance for cross section.
RDT	NRIV, NTRIB	RIV	Main stem river distance for each tributary.
RI			RI = FLOAT (I). [OUT1]
RN			Coefficient used to correct the conveyance for the current discharge. [NVAL]
RNMAX			Maximum value for RN. [NVAL]
RNMIN			Minimum value for RN. [NVAL]
SB	NRIV	RIV	Average river segment slope.
SBAR			Standard error of the estimate for hydraulic properties relationship. [LSQ], [CHNGM], [LSQF]
SD	10		
SDA			SDA = SUMDA/NDIS. [SROUT]
SL			Slope used in divided flow. [DIVDE]
SMADA	NSEC	SEC2	Sum of absolute bed area change. [SROUT]
SMZWS	NSEC	SEC2	Sum of water surface elevations. [SROUT]

Variable	Array size	Common block	Definition
STAGE		RIV	Value of stage used for downstream control.
SUMA			Sum used in least squares regression $\Sigma(x_i - \bar{x})\cdot(y_i - \bar{y})$. [LSQ]
SUMB			Sum used in least squares regression $\Sigma(x_i - \bar{x})^2$. [LSQ]
SUMC			Sum used in least squares regression $\Sigma(y_i - \bar{y})^2$. [LSQ]
SUMD			Sum used in least squares regression $\Sigma(y_i - A + Bx_i)^2$ or $\Sigma(y_i - y_{icorr})^2$. [LSQ]
SUMDA	NSEC	SEC2	Sum of the bed area change.
SUMDZ			Approximate change in bed elevation. [SROUT]
SUMD1			Sum used to compute effective depth. [GEOM]
SUMD2			Sum used to compute effective depth. [GEOM]
SUMD3			Sum used to compute effective width. [GEOM]
SUMWK			Sum used to calculate alpha. [GEOM]
SUMX			Sum used in least squares regression. [LSQ]
SUMXX			Sum used in least squares regression. [LSQ]
SUMXY			Sum used in least squares regression. [LSQ]
SUMY			Sum used in least squares regression. [LSQ]
TA		HYD	Total area at the cross section and for the given water surface elevation.
TAK	NSEC	SEC1	Storage array for total area at the cross section (K).

Variable	Array size	Common block	Definition
TED1			Temporary value used for derivation of Newton-Raphson approximation. [BKWAT]
TED2			Temporary value used for derivation of Newton-Raphson approximation. [BKWAT]
THD			Total head downstream. [BKWAT]
TITLE	20		Array for the title of the program run. [IN1]
TK		HYD	Total cross section conveyance at the cross section for the given water elevation.
TKA			Average conveyance used in head-loss calculations. [BKWAT]
TKD		SYS	Total conveyance at downstream cross section.
TKK	NSEC	SEC1	Storage array for total conveyance at cross section.
TOL			Tolerance in divided flow calculations. [DIVDE]
TQ	NQI	SEC	Storage array for total discharge at cross section.
U			Flow velocity. [WEIRS]
US			Shear velocity. [WEIRS]
V		HYD	Velocity.
VH			Velocity head. [BKWAT]
VHD		SYS	Velocity head at downstream cross section.
VK	NSEC	SEC1	Storage array for velocity at cross section.
WE	NSEC	SEC1	Storage array for effective width at cross section.
WID			Width of the weir. [WEIR]

Variable	Array size	Common block	Definition
WS			Water surface elevation. [BKWAT], [CHNGM], [CONT], [GEOM], [HYDPR], [SUBPF], [WEIR]
WSA			Water surface elevation of stage discharge relationship. [CONT]
WSB			Water surface elevation of stage-hydrograph. [CONT]
WSC			Water surface elevation at down-stream cross section. [CONT]
WSD		SYS	Downstream water surface elevation.
WSK	NSEC	SEC1	Storage array for water surface at cross section.
WSMAX	NSEC	SEC1	Maximum water surface elevation at each cross section.
X	NSEC, ND	SEC2	Array of horizontal distances for each cross section.
XB			Increment of channel width. [GEOM]
XO			X-value for forced point. [LSQF]
XL			Temporary value of P used in divided flow situation. [DIVDE]
XP			Temporary value. [LSQF]
XR			Temporary value of P used in divided flow situation. [DIVDE]
XX	10		XX = AD array of log values of depth used in least squares regression. [LSQ]
Y	10		Array used in least squares regression. [LSQ]
YB	10		Array used in least squares regression. [LSQ]
YO			Y-value for forced point. [LSQF]
YP			Temporary value. [LSQF]
Z	NSEC, ND	SEC2	Array of bed elevation.

Variable	Array size	Common block	Definition
ZB			Increment of channel depth. [GEOM]
ZDIF			Difference between maximum and minimum elevation for each cross section. [CHNGM]
ZDIFM		UNITS	Increment of water surface elevation, for the calculation of the hydraulic properties relations.
ZMIN	NSEC	11	Minimum elevation for cross section.
ZMP3			ZMIN + 3.0. [SROUT]
ZO	NSEC, ND	SEC2	Original bed elevations.
ZOB	NSEC	SEC2	Overbank elevation.
ZSQ			Computed section factor. [SUBPF]

APPENDIX D

LISTING OF PROGRAM KUWASER

The following is a list of the program KUWASER. The listings are presented by subroutine in alphabetical order.

Program KUWASER

```
      PROGRAM KUWASER
     1(INPUT,OUTPUT=65,TAPE6=OUTPUT,TAPE5=65,TAPE7=65,TAPE8=513,TAPE9=51
     23,TAPE10=513)
C
C     THIS PROGRAM IS A KNOWN DISCHARGE, WATER AND SEDIMENT ROUTING
C     MODEL.
C
C     THIS MODEL WAS DELVELOPED BY GLENN O. BROWN AND RUH-MING LI,
C     AT THE ENGINEERING RESEARCH CENTER, COLORADO STATE UNIVERSITY,
C     FORT COLLINS, COLORADO.
C
      COMMON /SEC1/           WD(22)        , RD(100)       , TQ(125)       ,
     1           WSK(100)   , WE(100)       , ZMIN(100)     , ZOB(100)      ,
     2           A1(100)    , A2(100)       , A3(100)       , A4(100)       ,
     3           A5(100)    , A6(100)       , A7(100)       , A8(100)       ,
     4           A9(100)    , A10(100)      , B1(100)       , B2(100)       ,
     5           B3(100)    , B4(100)       , B5(100)       , B6(100)       ,
     6           B7(100)    , B8(100)       , B9(100)       , B10(100)      ,
     7           DA4(100)   , DA9(100)      , EDK(100)      , TAK(100)      ,
     8           TKK(100)   , ALPK(100)     , VK(100)       , WSMAX(100)    ,
     9           IMAX(100)
      COMMON /SEC2/           X(100,22)     , Z(100,22)     , ZO(100,22)    ,
     1           DLOB(100)  , DROB(100)     , F(100,22)     , ND(100)       ,
     2           SUMDA(100) , SMADA(100)    , SMZWS(100)    , IZMIN(100)    ,
     3           G(100)
      COMMON /UNITS/          IUNIT         , CORDS         , ZDIFM         ,
     1           GRAV       , VVAL          , FET           , CONV          ,
     2           DZMAX
      COMMON /INF/            NSEC          , NTIM          , DT            ,
     1           IORG       , CE            , CC            , PORM          ,
     2           STAGE      , ITIME         , NQI           , NCALL         ,
     3           ICALL(30,3) , MST          , EPS
      COMMON /RIV/            NRIV          , KUP(10)       , KDOWN(10)     ,
     1           NTRIB(10)  , ICONT(10)     , DSL(10,5)     , AX(10)        ,
     2           BX(10)     , CX(10)        , RDT(10,5)     , ITRIB(10,5)   ,
     3           KTRIB(10,5) , AT(10,5)     , BT(10,5)      , KCONT(10)     ,
     4           IROUT(10)  , SB(10)        , AN(10)        , BN(10)
      COMMON /PRT/            IPRNT(8)      , I5            , I6            ,
     1           I7         , I8            , I9            , I10
      DIMENSION               PNR(10)
C
      DATA IDUMB/0/
C
C     DEFINE DEVICE NUMBERS
C
C        I5         INPUT DEVICE FOR GENERAL DATA FILE
C        I6         DEVICE FOR PRINTED OUTPUT
C        I7         INPUT DEVICE FOR CROSS SECTION FILE
C        I8         INPUT DEVICE FOR DISCHARGE FILE
C        I9         OUTPUT DEVICE FOR YEARLY CROSS SECTION ELEVATIONS,
C                   (BINARY OUTPUT)
C       I10         OUTPUT DEVICE FOR CROSS SECTION HYDRAULIC PRPERTIES,
C                   (BINARY OUTPUT)
      I5 = 5
      I6 = 6
      I7 = 7
      I8 = 8
      I9 = 9
      I10 = 10
C
C     READ IN THE SEDIMENT AND GEOMETRY DATA.
C
      CALL INI
C
C     PUTS CORRECT VALUES TO THE CONSTANTS ACCORDING TO THE UNIT-SYSTEM.
C
      CALL UNIT
C
C     CONVERT RIVER DISTANCES.
C
      DO 130 NR = 1,NRIV
         PNR(NR) = 0.5
         KU = KUP(NR)
         KD = KDOWN(NR)
         IF (NR.EQ.1) GO TO 100
         NRM1 = NR - 1
         IF (KD.EQ.KUP(NRM1)) KD = KD + 1
  100    DO 110 K = KD,KU
            DIS = RD(K)
            CALL RIVDS (NR,DIS)
            RD(K) = DIS
  110    CONTINUE
         NT = NTRIB(NR)
         IF (NT.EQ.0) GO TO 130
         DO 120 J = 1,NT
            DIS = RDT(NR,J)
            CALL RIVDS (NR,DIS)
            RDT(NR,J) = DIS
  120    CONTINUE
```

Program KUWASER continued

```
  130 CONTINUE
C
C     SET INITIAL VALUES
C
      DO 150 K = 1,NSEC
         WSMAX(K) = 0.0
         SMZWS(K) = 0.0
         SMADA(K) = 0.0
         SUMDA(K) = 0.0
         WSK(K) = 0.0
         TO(K) = 0.0
         VK(K) = 0.0
         EDK(K) = 0.0
         WE(K) = 0.0
         ALPK(K) = 0.0
         TKK(K) = 0.0
         TAK(K) = 0.0
C
C     SET ORIGNAL BED ELEVATIONS INTO ZO(K,L) ARRAY.
C
         M = ND(K)
         DO 140 L = 1,M
            ZO(K,L) = Z(K,L)
  140    CONTINUE
  150 CONTINUE
C
C     CALCULATE THE INITIAL HYDRAULIC PROPERTIES OF EACH CROSS SECTION
C
      DO 160 K = 1,NSEC
         CALL THAL  (K)
         CALL CHNGM (K)
  160 CONTINUE
C
C     ITERATE OVER EACH TIME PERIOD.
C
      IF (NTIM.NE.0) GO TO 170
      WRITE (16,310)
      STOP
  170 DO 300 I = 1,NTIM
         ITIME = I
C
C     ITERATE OVER SUBROUTINE CALLS
C
         DO 290 NC = 1,NCALL
            ICAL1 = ICALL(NC,1)
            GO TO (180,190,200,210,220,230,240,250,260,280), ICAL1
C
C     DETERMINE FLOW AT EACH CROSS SECTION
C
  180       CALL FLOW
            GO TO 290
C
C     CALCULATE WATER SURFACE PROFILE FOR REACH
C
  190       CALL SUBPF (ICALL(NC,2))
            GO TO 290
C
C     COMPUTE FLOWS AND WATER SURFACE PROFILES FOR DIVIDED FLOW REACHES
C
  200       ICAL2 = ICALL(NC,2)
            P = PNR(ICAL2)
            IFLAG = 0
            CALL DIVDE (ICALL(NC,2),ICALL(NC,3),IDUMB,IDUMB,IFLAG,P)
            PNR(ICAL2) = P
            GO TO 290
C
C     CALCULATE SEDIMENT TRANSPORT AT EACH CROSS SECTION.
C
  210       CALL SED
            GO TO 290
C
C     CALCULATE SEDIMENT TRANSPORT OVER THE WEIR(S)
C
  220       CALL WEIRS (ICALL(NC,2),ICALL(NC,3))
            GO TO 290
C
C     CALCULATE TRIBUTARY SEDIMENT DISCHARGE.
C
  230       CALL TRIBS
            GO TO 290
C
C     ROUTE THE SEDIMENT IN THE RIVER REACH
C
  240       CALL SROUT (ICALL(NC,2))
            GO TO 290
C
C     DUPLICATE PROPERTIES AT DOUBLE CROSS SECTIONS
C
  250       CALL DUP (ICALL(NC,2),ICALL(NC,3))
            GO TO 290
```

Program KUWASER continued

```
C
C     TEST FOR MAXIMUM WATER SURFACE AT EACH CROSS SECTION
C
  260       DO 270 K = 1,NSEC
               IF (WSK(K).LT.WSMAX(K)) GO TO 270
               WSMAX(K) = WSK(K)
               IMAX(K) = ITIME
  270       CONTINUE
C
C     PRINT OUT THE RESULTS.
C
            CALL OUT1
            GO TO 290
C
C     DREDGE RIVER REACH
C

            CALL DREDG (ICALL(NC,2))
  290    CONTINUE
  300 CONTINUE
C
      STOP
C
C
C
  310 FORMAT (//,10X,38HTHE NUMBER OF TIME PERIODS EQUAL ZERO.,/,10X,42H
     1NO WATER OR SEDIMENT ROUTING IS PERFORMED.)
      END
```

Subroutine BKWAT

```
      SUBROUTINE BKWAT (K,WS,ICRI)
C
C     THIS SUBROUTINE CALCULATES THE WATER SURFACE ELEVATION AT A
C     CROSS SECTION ONCE THE CONDITIONS AT THE DOWNSTREAM SECTION
C     ARE KNOWN.  THE ROUTINE USES A FIRST ORDER NEWTON-RAPHSON
C     SOLUTION TO THE SOLVE THE TOTAL HEAD EQUATION.
C
      COMMON /SEC1/          WD(22)       , RD(100)      , TQ(125)      ,
     1          WSK(100)   , WE(100)      , ZMIN(100)    , ZOB(100)     ,
     2          A1(100)    , A2(100)      , A3(100)      , A4(100)      ,
     3          A5(100)    , A6(100)      , A7(100)      , A8(100)      ,
     4          A9(100)    , A10(100)     , B1(100)      , B2(100)      ,
     5          B3(100)    , B4(100)      , B5(100)      , B6(100)      ,
     6          B7(100)    , B8(100)      , B9(100)      , B10(100)     ,
     7          OA4(100)   , OA9(100)     , EDK(100)     , TAK(100)     ,
     8          TKK(100)   , ALPK(100)    , VK(100)      , WSMAX(100)   ,
     9          IMAX(100)
      COMMON /UNITS/         IUNIT        , CORDS        , ZDIFM        ,
     1          GRAV       , VVAL         , FET          , CONV         ,
     2          DZMAX
      COMMON /INF/           NSEC         , NTIM         , DT           ,
     1          IDRG       , CE           , CC           , PORM         ,
     2          STAGE      , ITIME        , NQI          , NCALL        ,
     3          ICALL(30,3), MST          , EPS
      COMMON /SYS/           VHD          , WSD          , TKD          ,
     1          DEX
      COMMON /HYD/           V            , ED           , EW           ,
     1          ALP        , TK           , TA
      COMMON /PRT/           IPRNT(8)     , I5           , I6           ,
     1          I7         , I8           , I9           , I10
C
C     CALCULATE THE TOTAL HEAD DOWNSTREAM.
C
      THD = VHD + WSD
C
C     ESTIMATE THE UPSTREAM WATER SURFACE ELEVATION BASED ON THE
C     DOWNSTREAM CONDITIONS.
C
      KOUNT = 0
      WS = WSD + DEX * (TQ(K)/TKD) * (TQ(K)/TKD)
  100 KOUNT = KOUNT + 1
C
C     DETERMINE IF ESTIMATE IS GREATER THAN THALWEG.
C
C
      IF (WS.LE.ZMIN(K)) WS = ZMIN(K) + FET
      CALL HYDPR (K,WS)
      IF (KOUNT.GT.MST) GO TO 140
C
C     CALCULATE VELOCITY HEAD, (VH).
C
      VH = ALP * V * V/(2. * GRAV)
      CCE = CE
      IF (VH.LT.VHD) CCE = CC
C
C     CALCULATE THE HEAD LOSS,(HL).
C
      TKA = (TKD + TK)/2.
      HLV = ABS(VH - VHD) * CCE
      HL = DEX * ((TQ(K) * TQ(K))/(TKA * TKA))
C
C     CALCULATE THE ERROR.
C
      ER = VH + WS - HL - THD - HLV
C
C     TEST FOR ERROR TOLERANCE.
C
```

Subroutine BKWAT continued

```
C
      IF (ABS(ER).LE.EPS) GO TO 150
      IF (KOUNT.GT.MST) GO TO 140
C
C     CALCULATE THE FIRST DERIVATIVE AT THE ESTIMATED DEPTH.
C
      D = WS - ZMIN(K)
      IF (WS.GT.ZOB(K)) GO TO 110
      TED1 = B5(K) - 2.0 * B3(K) - 1.0
      ATEMP = (TQ(K) * TQ(K)/(2. * GRAV)) * ((A5(K)/(A3(K) * A3(K))) * (
     1B5(K) - 2.0 * B3(K))) * D *  * TED1
      ATEMP = (1. - CCE) * ATEMP
      TED1 = B4(K) - 1.0
      TED2 = 2.0 * B4(K) - 1.0
      BNUM = (4.0 * DEX * TQ(K) * TQ(K)) * (2.0 * TKD * A4(K) * B4(K) *
     1D *  * TED1 + (A4(K) * A4(K)) * 2.0 * B4(K) * D *  * TED2)
      TED1 = B4(K)
      TED2 = 2.0 * B4(K)
      BDEN = (TKD * TKD) + 2.0 * TKD * A4(K) * D *  * TED1 + (A4(K) *  *
     1 2) * D *  * TED2
      GO TO 120
  110 TED1 = B10(K) - 2.0 * B8(K) - 1.0
      ATEMP = (TQ(K) * TQ(K)/(2. * GRAV)) * ((A10(K)/(A8(K) * A8(K))) *
     1(B10(K) - 2.0 * B8(K))) * D *  * TED1
      ATEMP = (1. - CCE) * ATEMP
      TED1 = B9(K) - 1.0
      TED2 = 2.0 * B9(K) - 1.0
      BNUM = (4.0 * DEX * TQ(K) * TQ(K)) * (2.0 * TKD * A9(K) * B9(K) *
     1D *  * TED1 + (A9(K) * A9(K)) * 2.0 * B9(K) * D *  * TED2)
      TED1 = B9(K)
      TED2 = 2.0 * B9(K)
      BDEN = (TKD * TKD) + 2.0 * TKD * A9(K) * D *  * TED1 + (A9(K) *  *
     1 2) * D *  * TED2
  120 DER = ATEMP + 1.0 + (BNUM/(BDEN *  * 2))
C
C     ESTIMATE CORRECT WATER SURFACE BY NEWTON-RAPHSON METHOD
C
      IF (DER.LT.0.0.AND.ER.GT.0.0) GO TO 130
      WS = WS - ER/ABS(DER)
      GO TO 100
  130 D = (TQ(K) * TQ(K) * A5(K))/(A3(K) * A3(K) * GRAV)
      D = D *  * (1./(1. + 2. * B3(K) - B5(K)))
      WS = D + ZMIN(K) + FET
      IF (WS.LT.ZOB(K)) GO TO 100
      D = (TQ(K) * TQ(K) * A10(K))/(A8(K) * A8(K) * GRAV)
      D = D *  * (1./(1. + 2. * B8(K) - B10(K)))
      WS = D + ZMIN(K) + FET
      GO TO 100
  140 ICRI = 1
      IF (IPRNT(8).NE.1) GO TO 150
      WRITE (I6,160) EPS,MST,K,ITIME
  150 IF (WS.LT.WSD) WS = WSD
      RETURN
C
C
C
  160 FORMAT (/10X,41HBACKWATER CALCULATION DID NOT CONVERGE TO,F4.1,4H
     1IN ,I3,11H ITERATIONS,/,10X,16HAT CROSS SECTION,I4,19H DURING TIME
     2 PERIOD,I5,/,10X,26HCRITICAL DEPTH IS ASSUMED.)
      END
```

Subroutine CHNGM

```
      SUBROUTINE CHNGM (K)
C
C     THIS SUBROUTINE COMPUTES POWER RELATIONS THAT ARE USED TO
C     CALCULATE EFFECTIVE DEPTH (ED), EFFECTIVE WIDTH (EW),
C     ALPHA (ALP), TOTAL AREA (TA), AND TOTAL CONVEYANCE (TK),
C     FOR A CROSS SECTION, AS A FUNCTION OF THE WATER SURFACE
C     ELEVATION (WS).
C
      COMMON /SEC1/          WD(22)        , RD(100)       , TQ(125)       ,
     1        WSK(100)     , WE(100)       , ZMIN(100)     , ZOB(100)      ,
     2        A1(100)      , A2(100)       , A3(100)       , A4(100)       ,
     3        A5(100)      , A6(100)       , A7(100)       , A8(100)       ,
     4        A9(100)      , A10(100)      , B1(100)       , B2(100)       ,
     5        B3(100)      , B4(100)       , B5(100)       , B6(100)       ,
     6        B7(100)      , B8(100)       , B9(100)       , B10(100)      ,
     7        OA4(100)     , OA9(100)      , EDK(100)      , TAK(100)      ,
     8        TKK(100)     , ALPK(100)     , VK(100)       , WSMAX(100)    ,
     9        IMAX(100)
      COMMON /UNITS/         IUNIT         , CORDS         , ZDIFM         ,
     1        GRAV         , VVAL          , FET           , CONV          ,
     2        DZMAX
      COMMON /HYD/           V             , ED            , EW            ,
     1        ALP          , TK            , TA
      COMMON /PRT/           IPRNT(8)      , I5            , I6            ,
     1        I7           , I8            , I9            , I10
      DIMENSION              AEW(10)       , AALP(10)      , AED(10)       ,
     1        ATA(10)      , ATK(10)       , AD(10)        , RCK(10)       ,
     2        SD(10)
      DATA NAP/10/
C
C     CALCULATE THE RELATIONSHIPS FOR THE MAIN CHANNEL.
C
      ZDIF = ZOB(K) - ZMIN(K)
      IF (ZDIF.LT.ZDIFM) ZDIF = ZDIFM
      DWS = ZDIF/FLOAT(NAP)
      WS = ZMIN(K)
C
C     CALCULATE THE EXACT HYDRAULIC PROPERTIES AT (NAP) EVENLY SPACED
C     WATER SURFACE ELEVATIONS.
C
      DO 100 N = 1,NAP
         WS = WS + DWS
         CALL GEOM (K,WS)
C
C     TAKE THE LOG OF THE HYDRAULIC PROPERTIES.  THESE VALUES WILL BE
C     USED IN THE LINEAR REGRESSION SUBROUTINE, SO THAT A POWER
C     FUNCTION WILL BE OBTAINED.
C
         AEW(N) = ALOG(EW)
         AED(N) = ALOG(ED)
         ATA(N) = ALOG(TA)
         ATK(N) = ALOG(TK)
         AALP(N) = ALOG(ALP)
         AD(N) = ALOG(WS - ZMIN(K))
  100 CONTINUE
C
C     CALL LEAST SQUARES LINEAR REGRESSION SUBROUTINE TO CALCULATE
C     THE HYDRAULIC POWER FUNCTIONS.
C
      CALL LSQ (NAP,AD,AEW,A1(K),B1(K),RCK(1),SD(1))
      CALL LSQ (NAP,AD,AED,A2(K),B2(K),RCK(2),SD(2))
      CALL LSQ (NAP,AD,ATA,A3(K),B3(K),RCK(3),SD(3))
      CALL LSQ (NAP,AD,ATK,A4(K),B4(K),RCK(4),SD(4))
      CALL LSQ (NAP,AD,AALP,A5(K),B5(K),RCK(5),SD(5))
      OA4(K) = A4(K)
      IF (RCK(4).GT.0.8) GO TO 110
      A5(K) = 1.25
      B5(K) = 0.01
C
C     CALCULATE THE RELATIONSHIPS FOR OVERBANK FLOW.
C
  110 DD = ZOB(K) - ZMIN(K)
      IF (DD.LE.0.0) DD = 0.1
      ADD = ALOG(DD)
      ATAD = ALOG(A3(K) * DD ** B3(K))
      ATKD = ALOG(A4(K) * DD ** B4(K))
      ZDIF = ZDIFM
      DWS = ZDIF/FLOAT(NAP)
      DO 120 N = 1,NAP
         WS = WS + DWS
         CALL GEOM (K,WS)
C
C     TAKE THE LOG OF THE HYDRAULIC PROPERTIES.  THESE VALUES WILL BE
C     USED IN THE LINEAR REGRESSION SUBROUTINE, SO THAT A POWER
C     FUNCTION WILL BE OBTAINED.
C
         AEW(N) = ALOG(EW)
         AED(N) = ALOG(ED)
         ATA(N) = ALOG(TA)
         ATK(N) = ALOG(TK)
         AALP(N) = ALOG(ALP)
         AD(N) = ALOG(WS - ZMIN(K))
```

Subroutine CHNGM continued

```
  120 CONTINUE
C
C     CALL LEAST SQUARES LINEAR REGRESSION SUBROUTINE TO CALCULATE
C     THE HYDRAULIC POWER FUNCTIONS.
C
      CALL LSQ (NAP,AD,AEW,A6(K),B6(K),RCK(6),SD(6))
      CALL LSQ (NAP,AD,AED,A7(K),B7(K),RCK(7),SD(7))
      CALL LSQF (NAP,AD,ATA,A8(K),B8(K),RCK(8),SD(8),ADO,ATAO)
      CALL LSQF (NAP,AD,ATK,A9(K),B9(K),RCK(9),SD(9),ADO,ATKO)
      CALL LSQ (NAP,AD,AALP,A10(K),B10(K),RCK(10),SD(10))
      OA9(K) = A9(K)
      IF (RCK(9).GT.0.8) GO TO 130
      A10(K) = 1.25
      B10(K) = 0.01
  130 IF (IPRNT(2).NE.1) RETURN
      WRITE (I6,140) K,A1(K),B1(K),RCK(1),SD(1),A2(K),B2(K),RCK(2),SD(2)
     1,A3(K),B3(K),RCK(3),SD(3),A4(K),B4(K),RCK(4),SD(4),A5(K),B5(K),RCK
     2(5),SD(5)
      WRITE (I6,140) K,A6(K),B6(K),RCK(6),SD(6),A7(K),B7(K),RCK(7),SD(7)
     1,A8(K),B8(K),RCK(8),SD(8),A9(K),B9(K),RCK(9),SD(9),A10(K),B10(K),R
     2CK(10),SD(10)
C
      RETURN
C
C
C
  140 FORMAT (2X,I3,1X,F6.2,1X,F4.2,1X,F4.2,1X,F5.3,1X,1H1,F7.2,1X,F4.2,
     11X,F4.2,1X,F5.3,1X,1H1,F7.2,1X,F4.2,1X,F4.2,1X,F5.3,1X,1H1,F7.0,1X
     2,F4.2,1X,F4.2,1X,F5.3,1X,1H1,F7.3,1X,F4.2,1X,F4.2,1X,F5.3,1X,1H1)
      END
```

Subroutine CONT

```
      SUBROUTINE CONT (NR,WS)
C
C     THIS SUBROUTINE IS USED TO COMPUTE THE WATER SURFACE ELEVATION
C     AT THE DOWNSTREAM CONTROL
C
C       DEFINITIONS OF TYPES OF CONTROL
C
C       ICONT(NR)             TYPE OF CONTROL
C
C           1                 STAGE-DISCHARGE RELATIONSHIP
C           2                 STAGE HYDROGRAPH
C           3                 DOWNSTREAM WATER SURFACE
C           4                 GREATEST OF #1 AND #2
C           5                 GREATEST OF #1 AND #3
C           6                 NORMAL DEPTH
C
      COMMON /SEC1/           WD(22)        , RD(100)      , TQ(125)      ,
     1          WSK(100)    , WE(100)       , ZMIN(100)    , ZOB(100)     ,
     2          A1(100)     , A2(100)       , A3(100)      , A4(100)      ,
     3          A5(100)     , A6(100)       , A7(100)      , A8(100)      ,
     4          A9(100)     , A10(100)      , B1(100)      , B2(100)      ,
     5          B3(100)     , B4(100)       , B5(100)      , B6(100)      ,
     6          B7(100)     , B8(100)       , B9(100)      , B10(100)     ,
     7          DA4(100)    , DA9(100)      , EDK(100)     , TAK(100)     ,
     8          TKK(100)    , ALPK(100)     , VK(100)      , WSMAX(100)   ,
     9          IMAX(100)
      COMMON /RIV/            NRIV          , KUP(10)      , KDOWN(10)    ,
     1          NTRIB(10)   , ICONT(10)     , DSL(10,5)    , AX(10)       ,
     2          BX(10)      , CX(10)        , RDT(10,5)    , ITRIB(10,5)  ,
     3          KTRIB(10,5) , AT(10,5)      , ST(10,5)     , KCONT(10)    ,
     4          IROUT(10)   , SB(10)        , AN(10)       , BN(10)
      COMMON /INF/            NSEC          , VTIM         , DT           ,
     1          IORG        , CE            , CC           , PORM         ,
     2          STAGE       , ITIME         , NQI          , NCALL        ,
     3          ICALL(30,3) , MST           , EPS
C
C     DETERMINE TYPE OF DOWNSTREAM CONTROL.
C
      ICANT = ICONT(NR)
      GO TO (100,110,120,100,100,170), ICANT
C
C     STAGE-DISCHARGE CONTROL
C
  100 KD = KDOWN(NR)
      WSA = CX(NR) + AX(NR) * TQ(KD) ** BX(NR)
      IF (ICONT(NR).EQ.4) GO TO 110
      IF (ICONT(NR).EQ.5) GO TO 120
      WS = WSA
      RETURN
C
C     STAGE HYDROGRAPH CONTROL.
C
  110 WSB = STAGE
      IF (ICONT(NR).EQ.4) GO TO 130
      WS = WSB
      RETURN
C
C     DOWNSTREAM WATER SURFACE CONTROL.
C
  120 KD = KCONT(NR)
      WSC = WSK(KD)
      IF (ICONT(NR).EQ.5) GO TO 150
      WS = WSC
      RETURN
  130 IF (WSB.GT.WSA) GO TO 140
      WS = WSA
      RETURN
  140 WS = WSB
      RETURN
  150 IF (WSC.GT.WSA) GO TO 160
      WS = WSA
      RETURN
  160 WS = WSC
      RETURN
C
C     NORMAL DEPTH CALCULATIONS.
C
  170 KD = KDOWN(NR)
      C = TQ(KD)/(SB(NR) ** 0.5)
      D = (C/A4(KD)) ** (1./B4(KD))
      WS = ZMIN(KD) + D
      IF (WS.LE.ZOB(KD)) RETURN
      C = TQ(KD)/(SB(NR) ** 0.5)
      D = (C/A9(KD)) ** (1./B9(KD))
      WS = ZMIN(KD) + D
C
      RETURN
      END
```

Subroutine DIVDE

```
      SUBROUTINE DIVDE (NR1,NR2,P)
C
C     THIS SUBROUTINE USES A SECOND ORDER CURVE FITTING ALGORITHM
C     TO FIND THE RATIO OF FLOWS "P" IN DIVIDED FLOW REACHSES
C
      COMMON /INF/           NSEC        , NTIM       , DT        ,
     1        IORG         , CE          , CC         , PORM      ,
     2        STAGE        , ITIME       , NOI        , NCALL     ,
     3        ICALL(30,3)  , MST         , EPS
      COMMON /PRT/           IPRNT(8)    , I5         , I6        ,
     1        I7           , I8          , I9         , I10
      DIMENSION              E(3)        , Y(3)
      DATA XLOL,XUPL/0.0,1.0/
      XA = P
      NC = 0
      DX = 0.2
  100 CALL SPLIT (NR1,NR2,VALUE,XA)
      P = XA
      IF (VALUE.LE.(EPS * EPS)) GO TO 270
      A = VALUE
      XB = XA + DX
      IF (XB.LE.XUPL) GO TO 110
      XB = XUPL
  110 CALL SPLIT (NR1,NR2,VALUE,XB)
      P = XB
      IF (VALUE.LE.(EPS * EPS)) GO TO 270
      B = VALUE
C
C     DETERMINE THE THIRD POINT REQUIRED FOR APPROXIMATION
C
      IF (A.GT.B) GO TO 150
  120 XC = XA - DX
      IF (XC.GE.XLOL) GO TO 130
      XC = XLOL
  130 CALL SPLIT (NR1,NR2,VALUE,XC)
      P = XC
      IF (VALUE.LE.(EPS * EPS)) GO TO 270
      C = VALUE
      Y(1) = XC
      Y(2) = XA
      Y(3) = XB
      E(1) = C
      E(2) = A
      E(3) = B
      IF (C.LT.A) GO TO 140
      XINF = XA
      FINF = A
      GO TO 180
  140 XINF = XC
      FINF = C
      GO TO 180
  150 XC = XA + 2. * DX
      IF (XC.LE.XUPL) GO TO 160
      XC = XUPL
  160 CALL SPLIT (NR1,NR2,VALUE,XC)
      P = XC
      IF (VALUE.LE.(EPS * EPS)) GO TO 270
      C = VALUE
      Y(1) = XA
      Y(2) = XB
      Y(3) = XC
      E(1) = A
      E(2) = B
      E(3) = C
      IF (C.LT.B) GO TO 170
      XINF = XB
      FINF = B
      GO TO 180
  170 XINF = XC
      FINF = C
C
C     ELIMINATE PREMATURE TERMINATION DUE TO EQUAL VALUES AT TWO END
C     POINTS IN THE FIRST SEARCH
C
  180 YDEF = Y(3) - 2. * Y(2) + Y(1)
      EDEF = E(1) - E(3)
      IF (NC.GT.0.OR.ABS(YDEF).GT.EPS.OR.ABS(EDEF).GT.EPS) GO TO 190
      DX = 0.5 * DX
      Y(2) = Y(1) + DX
      IF (Y(2).GT.XUPL) Y(2) = XUPL
      CALL SPLIT (NR1,NR2,VALUE,Y(2))
      P = Y(2)
C
      IF (VALUE.LE.(EPS * EPS)) GO TO 270
      E(2) = VALUE
      Y(3) = XINF
      E(3) = FINF
      EDEF = E(1) - E(3)
      IF (E(2).GT.FINF) GO TO 190
      XINF = Y(2)
      FINF = E(2)
```

Subroutine DIVDE continued

```
C
C      CHECK THE CONVEXITY OF THE QUADRATIC FUNCTION
C
  190 A1 = (Y(1) - Y(2)) * (Y(2) - Y(3)) * (Y(1) - Y(3))
      IF (ABS(A1).EQ.0.) GO TO 200
      A2 = E(1) * (Y(2) - Y(3)) + E(2) * (Y(3) - Y(1)) + E(3) * (Y(1) -
     1Y(2))
      SA = A2/A1
      IF (SA.GE.0.0) GO TO 210
      DX = Y(3) - Y(1)
      XA = Y(1)
      A = E(1)
      XB = Y(3)
      B = E(3)
      IF (EDEF.GT.0.) GO TO 150
      GO TO 120
  200 XSTA = XINF
      FSTA = FINF
      GO TO 270
C
C      DETERMINE THE MINIMUM OF THE QUADRATIC FUNCTION
C
  210 SB = (E(1) - E(2))/(Y(1) - Y(2)) - SA * (Y(1) + Y(2))
      XSTA =  - SB/(2. * SA)
      IF (XSTA.GE.XLOL.AND.XSTA.LE.XUPL) GO TO 230
      IF (EDEF.GT.0.0) GO TO 220
      XSTA = XLOL
      GO TO 230
  220 XSTA = XUPL
  230 NC = NC + 1
      CALL SPLIT (NR1,NR2,VALUE,XSTA)
      P = XSTA
      IF (VALUE.LE.(EPS * EPS)) GO TO 270
      FSTA = VALUE
      XTEM = XSTA
      FTEM = FSTA
      IF (FSTA.LE.FINF) GO TO 240
      XTEM = XINF
      FTEM = FINF
  240 IF (ABS(1. - FSTA/FINF).GT.EPS) GO TO 250
      XSTA = XTEM
      FSTA = FTEM
      GO TO 270
  250 IF (NC.LT.MST) GO TO 260
      IF (IPRNT(8).NE.1) RETURN
      WRITE (16,280) NR1,NR2,ITIME
      RETURN
  260 DL = ABS(XINF - XSTA)
      IF (DL.LT.DX) DX = DL
      XA = XTEM
      GO TO 100
C
C      A MINIMUM HAS BEEN FOUND
C
  270 RETURN
C
C
C
  280 FORMAT (/,10X, 42HDIVIDED FLOW CALCULATIONS DID NOT CONVERGE,/,10X
     1, 11HFOR REACHES,I3,  4H AND,I3, 18HDURING TIME PERIOD,I3)
      END
```

Subroutine DREDG

```
      SUBROUTINE DREDG (NR)
C
C     THIS SUBROUTINE SIMULATES DREDGING BY LOWERING EACH CROSS
C     SECTION IN A REACH TO ITS ORIGINAL ELEVATION (ZO).  ONLY
C     CROSS SECTION POINTS IN THE MAIN CHANNEL ARE LOWERED.
C
      COMMON /SEC1/          WD(22)       , RD(100)      , TW(125)      ,
     1         WSK(100)    , WE(100)      , ZMIN(100)    , ZOB(100)     ,
     2         A1(100)     , A2(100)      , A3(100)      , A4(100)      ,
     3         A5(100)     , A6(100)      , A7(100)      , A8(100)      ,
     4         A9(100)     , A10(100)     , B1(100)      , B2(100)      ,
     5         B3(100)     , B4(100)      , B5(100)      , B6(100)      ,
     6         B7(100)     , B8(100)      , B9(100)      , B10(100)     ,
     7         DA4(100)    , DA9(100)     , EDK(100)     , TAK(100)     ,
     8         IKR(100)    , ALPR(100)    , VK(100)      , WSMAX(100)   ,
     9         IMAX(100)
      COMMON /SEC2/          X(100,22)    , Z(100,22)    , ZO(100,22)   ,
     1         DLOB(100)   , DROB(100)    , F(100,22)    , ND(100)      ,
     2         SUMDA(100)  , SMADA(100)   , SMZWS(100)   , IZMIN(100)   ,
     3         G(100)
      COMMON /RIV/           NRIV         , KUP(10)      , KDOWN(10)    ,
     1         NTRIB(10)   , ICONT(10)    , QSL(10,5)    , AX(10)       ,
     2         BX(10)      , CX(10)       , ROT(10,5)    , ITRIB(10,5)  ,
     3         KTRIB(10,5) , AT(10,5)     , BT(10,5)     , KCONT(10)    ,
     4         IROUT(10)   , SB(10)       , AN(10)       , BN(10)
      KU = KUP(NR)
      KD = KDOWN(NR)
      DO 110 K = KD,KU
         IFLG = 0
         M = ND(K)
         DO 100 L = 1,M
            IF (X(K,L).LT.DROB(K)) GO TO 100
            IF (X(K,L).GT.DLOB(K)) GO TO 100
            IF (Z(K,L).LE.ZO(K,L)) GO TO 100
            Z(K,L) = ZO(K,L)
            IFLG = 1
  100    CONTINUE
         IF (IFLG.EQ.0) GO TO 110
         CALL THAL (K)
         CALL CHNGM (K)
  110 CONTINUE
      WRITE (I9) ((Z(K,L),K = KD,KU),L = 1,22)
      RETURN
C
      END
```

Subroutine DUP

```
      SUBROUTINE DUP (K1,K2)
C
C     THIS SUBROUTINE IS USED IN DIVIDE FLOW PROBLEMS WHEN
C     A CROSS SECTION IS USED BY TWO DIFFERENT RIVER REACHES.
C     IT CHANGES THE BED ELEVATION OF THE DUPLICATE CROSS
C     SECTION, (K2), TO MATCH THE BED ELEVATION OF THE ORIGINAL
C     SECTION, (K1), AFTER SEDIMENT ROUTING.
C
      COMMON /SEC1/          WD(22)      , RD(100)     , TW(125)     ,
     1       WSK(100)      , WE(100)     , ZMIN(100)   , ZOB(100)    ,
     2       A1(100)       , A2(100)     , A3(100)     , A4(100)     ,
     3       A5(100)       , A6(100)     , A7(100)     , A8(100)     ,
     4       A9(100)       , A10(100)    , B1(100)     , B2(100)     ,
     5       B3(100)       , B4(100)     , B5(100)     , B6(100)     ,
     6       B7(100)       , B8(100)     , B9(100)     , B10(100)    ,
     7       OA4(100)      , OA9(100)    , EDK(100)    , TAK(100)    ,
     8       IKK(100)      , ALPK(100)   , VK(100)     , WSMAX(100)  ,
     9       IMAX(100)
      COMMON /SEC2/          X(100,22)   , Z(100,22)   , ZO(100,22)  ,
     1       DLOB(100)     , DROB(100)   , F(100,22)   , ND(100)     ,
     2       SUMDA(100)    , SMADA(100)  , SMZWS(100)  , IZMIN(100)  ,
     3       G(100)
C
      N = ND(K1)
      DO 100 L = 1,N
         Z(K2,L) = Z(K1,L)
  100 CONTINUE
      ZMIN(K2) = ZMIN(K1)
      IZMIN(K2) = IZMIN(K1)
      A1(K2) = A1(K1)
      A2(K2) = A2(K1)
      A3(K2) = A3(K1)
      A4(K2) = A4(K1)
      A5(K2) = A5(K1)
      A6(K2) = A6(K1)
      A7(K2) = A7(K1)
      A8(K2) = A8(K1)
      A9(K2) = A9(K1)
      A10(K2) = A10(K1)
      B1(K2) = B1(K1)
      B2(K2) = B2(K1)
      B3(K2) = B3(K1)
      B4(K2) = B4(K1)
      B5(K2) = B5(K1)
      B6(K2) = B6(K1)
      B7(K2) = B7(K1)
      B8(K2) = B8(K1)
      B9(K2) = B9(K1)
      B10(K2) = B10(K1)
      OA4(K2) = OA4(K1)
      OA9(K2) = OA9(K1)
C
      RETURN
      END
```

Subroutine FLOW

```
      SUBROUTINE FLOW
C
C     THIS SUBROUTINE CALCULATES THE WATER DISCHARGE AT EACH
C     CROSS SECTION.
C
      COMMON /SEC1/         WD(22)        , RD(100)       , TQ(125)       ,
     1         WSK(100)   , WE(100)       , ZMIN(100)     , ZOB(100)      ,
     2         A1(100)    , A2(100)       , A3(100)       , A4(100)       ,
     3         A5(100)    , A6(100)       , A7(100)       , A8(100)       ,
     4         A9(100)    , A10(100)      , B1(100)       , B2(100)       ,
     5         B3(100)    , B4(100)       , B5(100)       , B6(100)       ,
     6         B7(100)    , B8(100)       , B9(100)       , B10(100)      ,
     7         DA4(100)   , DA9(100)      , EDK(100)      , TAK(100)      ,
     8         TKK(100)   , ALPK(100)     , VK(100)       , WSMAX(100)    ,
     9         IMAX(100)
      COMMON /INF/          NSEC          , NTIM          , DT            ,
     1         IORG       , CE            , CC            , PORM          ,
     2         STAGE      , ITIME         , NQI           , NCALL         ,
     3         ICALL(30,3), MST           , EPS
      COMMON /PRT/          IPRNT(8)      , I5            , I6            ,
     1         I7         , I8            , I9            , I10
C
C     READ IN THE FLOWS AND THE TIME PERIOD LENGTH.
C
      READ (I8) (TQ(K),K = 1,NQI),DT
      IF (EOF(I8)) 100,110,100
  100 WRITE (I6,120) I8,ITIME
      STOP
  110 RETURN
C
C
C
  120 FORMAT (10X, 24HEND OF FILE READ ON FILE,I2, 18HDURING TIME PERIOD
     1,I5)
      END
```

Subroutine GEOM

```
      SUBROUTINE GEOM (K,WS)
C
C     THIS SUBROUTINE CALCULATES THE EXACT HYDRAULIC PROPERTIES OF
C     A CROSS SECTION, ONCE GIVEN THE CHANNEL GEOMETRY AND THE
C     WATER SURFACE ELEVATION.
C
      COMMON /SEC1/             WD(22)        , RD(100)       , TQ(125)       ,
     1          WSK(100)      , WE(100)       , ZMIN(100)     , ZOB(100)      ,
     2          A1(100)       , A2(100)       , A3(100)       , A4(100)       ,
     3          A5(100)       , A6(100)       , A7(100)       , A8(100)       ,
     4          A9(100)       , A10(100)      , B1(100)       , B2(100)       ,
     5          B3(100)       , B4(100)       , B5(100)       , B6(100)       ,
     6          B7(100)       , B8(100)       , B9(100)       , B10(100)      ,
     7          DA4(100)      , DA9(100)      , EDK(100)      , TAK(100)      ,
     8          TKK(100)      , ALPK(100)     , VK(100)       , WSMAX(100)    ,
     9          IMAX(100)
      COMMON /SEC2/             X(100,22)     , Z(100,22)     , ZO(100,22)    ,
     1          DLOB(100)     , DROB(100)     , F(100,22)     , ND(100)       ,
     2          SUMDA(100)    , SHADA(100)    , SMZWS(100)    , IZMIN(100)    ,
     3          G(100)
      COMMON /UNITS/            IUNIT         , CORDS         , ZDIFM         ,
     1          GRAV          , VVAL          , FET           , CONV          ,
     2          DZMAX
      COMMON /INF/              NSEC          , NTIM          , DT            ,
     1          IORG          , CE            , CC            , PORM          ,
     2          STAGE         , ITIME         , NOI           , NCALL         ,
     3          ICALL(30,3)   , MST           , EPS
      COMMON /HYD/              V             , ED            , EW            ,
     1          ALP           , TK            , TA
C
      N = ND(K)
      NP = N - 1
      IFLAG = 0
      TA = 0.
      TK = 0.
      SUMWK = 0.
      SUMD1 = 0.
      SUMD2 = 0.
      SUMD3 = 0.
      DO 100 I = 1,NP
        WD(I) = 0.
  100 CONTINUE
C
C     ITERATE OVER EACH CROSS SECTION POINT
C
      DO 170 I = 2,N
C
C     CALCULATE DISTANCE AND MANNINGS N BETWEEN CROSS SECTION POINTS.
C
         XB = (X(K,I) - X(K,I - 1)) + 1.0E - 6
         FM = 0.5 * (F(K,I - 1) + F(K,I))
         IF (Z(K,I).GE.WS) GO TO 120
         IF (Z(K,I - 1).GE.WS) GO TO 110
C
C     CALCULATE AREA OF FLOW, WETTED PERIMETER, AND DEPTH.
C
         DA = WS - 0.5 * (Z(K,I - 1) + Z(K,I))
         A = XB * DA
         ZB = ABS(Z(K,I) - Z(K,I - 1))
         P = SQRT(XB * XB + ZB * ZB)
         GO TO 160
  110    ZB = WS - Z(K,I)
         XB = XB * ZB/(Z(K,I - 1) - Z(K,I))
         GO TO 150
  120    IF (Z(K,I - 1).GE.WS) GO TO 170
         IF (I.LT.IZMIN(K).AND.WS.LT.(ZOB(K) + .001)) GO TO 130
         ZB = WS - Z(K,I - 1)
         XB = XB * ZB/(Z(K,I) - Z(K,I - 1))
         IF (WS.LT.(ZOB(K) + .001)) IFLAG = 1
         GO TO 150
C
C     SET OVER BANK FLOWS TO ZERO IF WATER SURFACE IS NOT ABOVE
C     OVER BANK ELEVATION.
C
  130    TA = 0
         TK = 0.
         SUMWK = 0.
         SUMD1 = 0.
         SUMD2 = 0.
         SUMD3 = 0.
         IM1 = I - 1
         DO 140 II = 1,IM1
            WD(II) = 0.
  140    CONTINUE
         GO TO 170
  150    A = 0.5 * XB * ZB
         P = SQRT(XB * XB + ZB * ZB)
         DA = 0.5 * ZB
  160    R = A/P
         C = 1.486 * A * R * * (2./3.)/FM
C
C     SUM FLOWS BETWEEN CROSS SECTION POINTS.
C
```

Subroutine GEOM continued

```
            TA = TA + A
            TK = TK + C
            SUMWK = SUMWK + C ** 3./A ** 2.
            SUMD1 = SUMD1 + DA ** 1.6666 * A/FM
            SUMD2 = SUMD2 + DA ** 0.6666 * A/FM
            SUMD3 = SUMD3 + DA ** 0.6666 * A
            WD(I - 1) = C
            IF (IFLAG.EQ.1) GO TO 180
  170 CONTINUE
  180 ALP = SUMWK * TA ** 2./TK ** 3.
      ED = SUMD1/SUMD2
      EW = SUMD3/ED ** 1.6666
      DO 190 I = 1,NP
         WD(I) = WD(I)/TK
  190 CONTINUE
C
      RETURN
      END
```

Subroutine HYDPR

```
      SUBROUTINE HYDPR (K,WS)
C
C     THIS SUBROUTINE CALCULATES THE HYDRAULIC PROPERTIES OF THE
C     (K)TH CROSS SECTION GIVEN THE WATER SURFACE ELEVATION (WS).
C
      COMMON /SEC1/          WD(22)        , RD(100)       , TQ(125)       ,
     1          WSK(100)     , WE(100)       , ZMIN(100)     , ZOB(100)      ,
     2          A1(100)      , A2(100)       , A3(100)       , A4(100)       ,
     3          A5(100)      , A6(100)       , A7(100)       , A8(100)       ,
     4          A9(100)      , A10(100)      , B1(100)       , B2(100)       ,
     5          B3(100)      , B4(100)       , B5(100)       , B6(100)       ,
     6          B7(100)      , B8(100)       , B9(100)       , B10(100)      ,
     7          UA4(100)     , UA9(100)      , EDK(100)      , TAK(100)      ,
     8          TKK(100)     , ALPK(100)     , VK(100)       , WSMAX(100)    ,
     9          IMAX(100)
      COMMON /HYD/           V             , ED            , EW            ,
     1          ALP          , TK            , TA
C
C      CALCULATE THE DEPTH OF FLOW
C
      DEPTH = WS - ZMIN(K)
C
C     IF THE WATER SURFACE IS ABOVE THE OVERBANK
C     USE THE OVERBANK EQUATIONS.
C
      IF (WS.GT.ZOB(K)) GO TO 100
C
C     CALCULATE EFFECTIVE WIDTH
C
      EW = A1(K) * DEPTH *  * B1(K)
C
C     CALCULATE EFFECTIVE DEPTH
C
      ED = A2(K) * DEPTH *  * B2(K)
C
C     CALCULATE AREA
C
      TA = A3(K) * DEPTH *  * B3(K)
C
C     CALCULATE THE CONVEYANCE
C
      TK = A4(K) * DEPTH *  * B4(K)
C
C     CALCULATE ALPHA
C
      ALP = A5(K) * DEPTH *  * B5(K)
      IF (ALP.GT.1.5) ALP = 1.5
      IF (ALP.LT.1.15) ALP = 1.15
C
C     CALCULATE THE VELOCITY
C
      V = TQ(K)/TA
      RETURN
C
C     CALCULATE EFFECTIVE WIDTH
C
  100 EW = A6(K) * DEPTH *  * B6(K)
C
C     CALCULATE EFFECTIVE DEPTH
C
      ED = A7(K) * DEPTH *  * B7(K)
C
C     CALCULATE AREA
C
      TA = A8(K) * DEPTH *  * B8(K)
C
C     CALCULATE THE CONVEYANCE
C
      TK = A9(K) * DEPTH *  * B9(K)
C
C     CALCULATE ALPHA
C
      ALP = A10(K) * DEPTH *  * B10(K)
      IF (ALP.GT.1.5) ALP = 1.5
      IF (ALP.LT.1.15) ALP = 1.15
C
C     CALCULATE THE VELOCITY
C
      V = TQ(K)/TA
C
      RETURN
      END
```

Subroutine IN1

```
      SUBROUTINE IN1
C
C     THIS SUBROUTINE READS IN THE SEDIMENT AND GEOMETRY DATA.
C
      COMMON /SEC1/            WD(22)        , RD(100)       , TQ(125)       ,
     1       WSK(100)      , WE(100)       , ZMIN(100)     , ZOB(100)      ,
     2       A1(100)       , A2(100)       , A3(100)       , A4(100)       ,
     3       A5(100)       , A6(100)       , A7(100)       , A8(100)       ,
     4       A9(100)       , A10(100)      , B1(100)       , B2(100)       ,
     5       B3(100)       , B4(100)       , B5(100)       , B6(100)       ,
     6       B7(100)       , B8(100)       , B9(100)       , B10(100)      ,
     7       DA4(100)      , DA9(100)      , EDK(100)      , TAK(100)      ,
     8       TKK(100)      , ALPK(100)     , VK(100)       , WSMAX(100)    ,
     9       IMAX(100)
      COMMON /SEC2/            X(100,22)     , Z(100,22)     , ZO(100,22)    ,
     1       DLOB(100)     , DROB(100)     , F(100,22)     , ND(100)       ,
     2       SUMDA(100)    , SMADA(100)    , SMZWS(100)    , IZMIN(100)    ,
     3       G(100)
      COMMON /INF/             NSEC          , NTIM          , DT            ,
     1       IORG          , CE            , CC            , PORM          ,
     2       STAGE         , ITIME         , NQI           , NCALL         ,
     3       ICALL(30,3)   , MST           , EPS
      COMMON /PRT/             IPRNT(8)      , I5            , I6            ,
     1       I7            , I8            , I9            , I10
      COMMON /RIV/             NRIV          , KUP(10)       , KDOWN(10)     ,
     1       NTRIB(10)     , ICONT(10)     , JSL(10,5)     , AX(10)        ,
     2       BX(10)        , CX(10)        , RDT(10,5)     , ITRIB(10,5)   ,
     3       KTRIB(10,5)   , AT(10,5)      , BT(10,5)      , KCONT(10)     ,
     4       IROUT(10)     , SB(10)        , AN(10)        , BN(10)
      COMMON /UNITS/           IUNIT         , CORDS         , ZDIFM         ,
     1       GRAV          , VVAL          , FET           , CONV          ,
     2       DZMAX
      DIMENSION                TITLE(20)
C
C     READ TITLE
C
      READ (I5,260) (TITLE(M2),M2 = 1,20)
C
C     READ THE PRINT CONTROL.
C
      READ (I5,270) (IPRNT(M1),M1 = 1,8)
C
C     READ IN THE MAXIMUM NUMBER OF ITERATIONS FOR THE BACKWATER
C     CURVE, (MST), THE MAXIMUM ERROR IN TOTAL HEAD, (EPS),
C     THE SEDIMENT DEPOSIT POROSITY, (PORM), THE COEFFICIENTS OF
C     EXPANSION AND CONTRACTION LOSSES, (CE,CC), AND THE UNIT SYSTEM
C     FLAG, (IUNIT).
C
      READ (I5,280) MST,EPS,PORM,CE,CC,IUNIT
C
C     READ IN THE NUMBER OF CROSS SECTIONS, (NSEC), THE NUMBER
C     OF TIME PERIODS,(NTIM), THE NUMBER OF RIVER SEGMENTS,
C     (NRIV), THE NUMBER OF INPUT DISCHARGES, (NQI), AND THE
C     NUMBER OF SUBROUTINE CALLS, (NCALL).
C
      READ (I5,290) NSEC,NTIM,NRIV,NQI,NCALL
C
C     READ IN THE SEQUENCE OF THE SUBROUTINE CALLS, (ICALL(NC,NN)).
C
      DO 100 NC = 1,NCALL
         READ (I5,290) (ICALL(NC,NN),NN = 1,3)
  100 CONTINUE
C
C     ITERATE OVER EACH RIVER SEGMENT.
C
      DO 120 NR = 1,NRIV
C
C     FOR EACH RIVER SEGMENT, READ IN THE NUMBER OF THE DOWNSTREAM
C     CROSS SECTION, (KDOWN(NR)), THE NUMBER OF THE UPSTREAM
C     CROSS SECTION, (KUP(NR)), THE NUMBER OF TRIBUTARIES,
C     (NTRIB(NR)), THE TYPE OF DOWNSTREAM CONTROL, (ICONT(NR)),
C     THE NUMBER OF THE DOWNSTREAM WATER SURFACE CONTROL CROSS SECTION,
C     (KCONT(NR)), THE COEFFICIENTS OF THE DOWNSTREAM STAGE DISCHARGE
C     RELATIONSHIP, (AX(NR),BX(NR),CX(NR)), THE COEFFICIENTS OF THE
C     CONVAYENCE EQUATION, (AN(NR),BN(NR)), AND THE NORMAL DEPTH
C     SLOPE, (SB(NR)).
C
         READ (I5,300) KDOWN(NR),KUP(NR),NTRIB(NR),ICONT(NR),KCONT(NR),I
     1   ROUT(NR),AX(NR),BX(NR),CX(NR),AN(NR),BN(NR),SB(NR)
         NT = NTRIB(NR)
C
C     FOR EACH TRIBUTARY READ IN THE MAINSTEM RIVER DISTANCE
C     OF THE CONFLUENCE, (RDT(NR,J)), THE TYPE OF TRIBUTARY,
C     (ITRIB(NR,J)), THE NUMBER OF THE DISCHARGE CROSS SECTION
C     FOR THE TRIBUTARY, (KTRIB(NR,J)), AND THE COEFFICIENTS OF THE
C     TRIBUTARYS SEDIMENT RATING CURVE, (AT(NR,J),BT(NR,J)).
C
         IF (NT.EQ.0) GO TO 120
         DO 110 J = 1,NT
            READ (I5,310) RDT(NR,J),ITRIB(NR,J),KTRIB(NR,J),AT(NR,J),BT(
     1      NR,J)
  110    CONTINUE
```

Subroutine IN1 continued

```
  120 CONTINUE
C
C     FOR EACH CROSS SECTION READ IN THE NUMBER OF SECTION POINTS,
C     (ND(K)), AND THE RIVER DISTANCE, (RD(K)).
C
      DO 160 K = 1,NSEC
         READ (I7,320) ND(K),RD(K)
         M = ND(K)
C
C     FOR EACH SECTION POINT READ IN THE HORIZONTAL DISTANCE,
C     (X(K,L)), AND THE ELEVATION, (Z(K,L)).
C
         READ (I7,330) (X(K,L),Z(K,L),L = 1,M)
C
C     FOR EACH CROSS SECTION READ IN THE DISTANCE OF THE RIGHT AND LEFT
C     OVERBANKS, (DROB,DLOB), THE MANINGS N FOR THE RIGHT
C     OVERBANK, MAIN CHANNEL, AND LEFT OVERBANK, (FROB,FMC,FLOB).
C     AND THE OVERBANK ELEVATION, (ZOB(K)).
C
         READ (I7,340) DROB(K),DLOB(K),FROB,FMC,FLOB,ZOB(K)
         DO 150 L = 1,M
            IF (X(K,L).GT.DROB(K)) GO TO 130
            F(K,L) = FROB
            GO TO 150
  130       IF (X(K,L).GE.DLOB(K)) GO TO 140
            F(K,L) = FMC
            GO TO 150
  140       F(K,L) = FLOB
  150    CONTINUE
  160 CONTINUE
C
C     PRINT OUT INPUT DATA
C
      IF (IPRNT(1).EQ.1) GO TO 170
      IF (IPRNT(2).EQ.1) GO TO 170
      IF (IPRNT(3).EQ.1) GO TO 170
      IF (IPRNT(4).EQ.1) GO TO 170
      IF (IPRNT(6).EQ.1) GO TO 170
      GO TO 180
  170 WRITE (I6,350)
      WRITE (I6,360) (TITLE(M2),M2 = 1,20)
  180 CONTINUE
      IF (IPRNT(1).NE.1) GO TO 240
      WRITE (I6,370) MST,EPS,PORM,CC,CE,IUNIT
      WRITE (I6,380) NSEC,NTIM,NRIV,NQI,NCALL
      WRITE (I6,390)
      DO 190 NC = 1,NCALL
         WRITE (I6,400) NC,(ICALL(NC,NN),NN = 1,3)
  190 CONTINUE
      DO 210 NR = 1,NRIV
         WRITE (I6,410) NR,KDOWN(NR),KUP(NR),NTRIB(NR),ICONT(NR),KCONT(N
     1   R),IROUT(NR),AX(NR),BX(NR),CX(NR),AN(NR),BN(NR),SB(NR)
         NT = NTRIB(NR)
         IF (NT.EQ.0) GO TO 210
         DO 200 J = 1,NT
            WRITE (I6,420) J,ROT(NR,J),ITRIB(NR,J),KTRIB(NR,J),AT(NR,J),
     1      BT(NR,J)
  200    CONTINUE
  210 CONTINUE
      DO 230 K = 1,NSEC
         WRITE (I6,430) K,ND(K),RD(K),ZOB(K)
         WRITE (I6,440)
         M = ND(K)
         DO 220 L = 1,M
            WRITE (I6,450) L,X(K,L),Z(K,L),F(K,L)
  220    CONTINUE
  230 CONTINUE
  240 CONTINUE
      IF (IPRNT(2).NE.1) GO TO 250
      WRITE (I6,460)
      WRITE (I6,470)
C
  250 RETURN
C
C
C
  260 FORMAT (20A4)
  270 FORMAT (6I2)
  280 FORMAT (I5,4F10.5,I5)
  290 FORMAT (5I5)
  300 FORMAT (6I5,5F8.4,F8.6)
  310 FORMAT (F10.2,2I5,2E10.2)
  320 FORMAT (2X,I3,F7.2)
  330 FORMAT ((8X,6(F6.0,F6.1)))
  340 FORMAT (6F10.4)
  350 FORMAT (1H1,/,10X,13HK U W A S E R,/,10X,25HKNOWN DISCHARGE SEDIME
     1NT ,/7HROUTING,/,10X,35HDEVELOPED BY J.J. BROWN AND R.M. LI,/,10X,3
     27HAT COLORADO STATE UNIVERSITY, FOR THE,/,10X,48HU.S. ARMY CORPS O
```

Subroutine IN1 continued

```
     3F ENGINEERS, VICKSBURG DISTRICT)
 360 FORMAT (///,10X,82(1H*),/,10X,1H*,80X,1H*,/,10X,1H*,20A4,1H*,/,10X
     1,1H*,80X,1H*,/,10X,82(1H*),///)
 370 FORMAT (10X,47HMAXIMUM NUMBER OF ITERATIONS FOR CALCULATIONS, ,6HM
     1ST = ,I5,,/,10X,42HACCURACY OF BACKWATER CALCULATIONS, EPS = ,F10.
     25,,/,10X,38HPOROSITY OF SEDIMENT DEPOSITS, PORM = ,F10.5,,/,10X, 5
     33HCOEFFICIENT OF CONTRAITION VELOCITY HEAD LOSSES, CC = ,F10.5,/,10
     4X, 52HCOEFFICIENT OF EXPANSION VELOCITY HEAD LOSSES, CE = ,F10.5/,
     510X, 26HUNIT SYSTEM FLAG, IUNIT = ,I5,//)
 380 FORMAT (10X,31HTHE NUMBER OF RIVER SECTIONS = ,I5,/,10X,29HTHE NUM
     1BER OF TIME PERIODS = ,I5,/10X,28HTHE NUMBER OF RIVER SEGMENTS,3H
     2= ,I5,/,10X,33HTHE NUMBER OF INPUT DISCHARGES = ,I5,/,10X,33HTHE N
     3UMBER OF SUBROUTINE CALLS = ,I5)
 390 FORMAT (/,10X,22HNCALL  ICALL (1 TO 3),)
 400 FORMAT (8X,4(2X,I5))
 410 FORMAT (//,10X,14HRIVER SEGMENT ,I5,/,10X,34HDOWNSTREAM CROSS SECT
     1ION NUMBER = ,I5,/,10X,32HUPSTREAM CROSS SECTION NUMBER = ,I5,/,10
     2X,24HNUMBER OF TRIBUTARIES = ,I5,/,10X,39HTYPE OF WATER SURFACE CO
     3NTROL, ICONT = ,I5,/,10X,35HNUMBER OF CONTROL SECTION, KCONT = ,I5
     4,/,10X,44HTYPE OF DOWNSTRAM SEDIMENT ROUTING, IROUT = ,I5,/,10X,45
     5HCOEFFICIENTS OF STAGE DISCHARGE RELATIONSHIP,,/,13X,5HAX = ,F8.4,
     6/,13X,5HBX = ,F8.4,/,13X,5HCX = ,F8.4,/,10X,40HCOEFFICIENTS OF MAN
     7NINGS N RELATIONSHIP,,/,13X,5HAN = ,F8.4,/,13X,5HBN = ,F8.4,/,10X,
     8 25HNORMAL DEPTH SLOPE, SB = ,F8.6)
 420 FORMAT (/,15X,10HTRIBUTARY ,I5,/,15X,17HRIVER DISTANCE = ,F7.2,/,1
     15X,27HTYPE OF TRIBUTARY, ITRIB = ,I5,/,15X,47HNUMBER OF TRIBUTARY
     2DISCHARGE SECTION, KTRIB = ,I5,/,15X,47HCOEFFICIENTS OF TRIBUTARY
     3SEDIMENT RELATIONP   ,/,18X,5HAT = ,E10.3,/,18X,5HBT = ,F10.2)
 430 FORMAT (///,10X,15HSECTION NUMBER ,I4,/,10X,40HTHE NUMBER OF CROSS
     1 SECTION POINTS IS = ,I2,/,10X,24HTHE RIVER DISTANCE IS = ,F10.2,/
     2,10X,25HTHE OVERBANK ELEVATION = ,F10.2)
 440 FORMAT (/,10X,48HPOINT     HORIZONTAL       ELEVATION         N VALUE,
     1/)
 450 FORMAT (10X,I5,3(5X,F10.4))
 460 FORMAT (1H1,/,4H NO.,4X,15HEFFECTIVE WIDTH,6X,1HI,4X,15HEFFECTIVE
     1DEPTH,5X,1HI,6X,10HTOTAL AREA,8X,1HI,4X,16HTOTAL CONVEYANCE,4X,1H
     2,8X,5HALPHA,11X,1HI/29X,1HI,4(24X,1HI))
 470 FORMAT (5X,5(4X,1HA,4X,1HB,2X,13HCO COEF ERR I)/29X,1HI,4(24X,1HI)
     1)
      END
```

Subroutine LSQ

```
      SUBROUTINE LSQ (N,XX,Y,EA,B,RC,SBAR)
C
C     THIS SUBROUTINE DERIVES THE COEFFICIENTS OF THE HYDRAULIC
C     POWER FUNCTIONS, BY USING A LEAST SQUARES REGRESSION.
C
      DIMENSION            XX(10)      , Y(10)        , YB(10)
C
      SUMX = 0.
      SUMXX = 0.
      SUMY = 0.
      SUMXY = 0.
      SUMA = 0.
      SUMB = 0.
      SUMC = 0.
      SUMD = 0.
      DO 100 I = 1,N
         SUMX = SUMX + XX(I)
         SUMY = SUMY + Y(I)
         SUMXX = SUMXX + XX(I) * XX(I)
         SUMXY = SUMXY + XX(I) * Y(I)
  100 CONTINUE
      FX = SUMX/FLOAT(N)
      FY = SUMY/FLOAT(N)
C
C     DERIVE THE EQUATION.
C
      B = (SUMXY - FLOAT(N) * FX * FY)/(SUMXX - FLOAT(N) * FX * FX)
      A = FY - B * FX
C
C     RAISE E TO THE (A) POWER.  THE VALUE, (EA), WILL BE USED IN
C     THE POWER FUNCTIONS.
C
      EA = EXP(A)
C
C     CALCULATE THE COEFFICIENT OF CORRELATION.
C
      DO 110 I = 1,N
         SUMA = SUMA + (XX(I) - FX) * (Y(I) - FY)
         SUMB = SUMB + (XX(I) - FX) *  * 2
         YB(I) = A + B * XX(I)
         SUMC = SUMC + (Y(I) - FY) *  * 2
  110 CONTINUE
      RC = SUMA/SQRT(SUMB * SUMC)
C
C     CALCULATE THE STANDARD ERROR OF ESTIMATE.
C
      DO 120 I = 1,N
         SUMD = SUMD + (Y(I) - YB(I)) *  * 2
  120 CONTINUE
      SBAR = SQRT(SUMD/(FLOAT(N) - 2.))
C
      RETURN
      END
```

Subroutine LSQF

```
      SUBROUTINE LSQF (N,XX,Y,EA,B,RC,SBAR,XO,YO)
C
C     THIS SUBROUTINE DERIVES THE COEFFICIENTS OF THE HYDRAULIC
C     POWER FUNCTIONS, FOR OVERBANK FLOW, BY USING A LEAST
C     SQUARES REGRESSION FORCED THROUGH THE POINT (XO,YO).
C
      DIMENSION              XX(10)      , Y(10)       , YB(10)
C
      SUMX = 0.
      SUMXX = 0.
      SUMY = 0.
      SUMXY = 0.
      SUMA = 0.
      SUMB = 0.
      SUMC = 0.
      SUMD = 0.
      DO 100 I = 1,N
         XP = XX(I) - XO
         YP = Y(I) - YO
         SUMX = SUMX + XX(I)
         SUMY = SUMY + Y(I)
         SUMXX = SUMXX + XP * XP
         SUMXY = SUMXY + XP * YP
  100 CONTINUE
      FX = SUMX/FLOAT(N)
      FY = SUMY/FLOAT(N)
C
C     DERIVE THE EQUATION.
C
      B = SUMXY/SUMXX
      A = YO - B * XO
C
C     RAISE E TO THE (A) POWER.  THE VALUE, (EA), WILL BE USED IN
C     THE POWER FUNCTIONS.
C
      EA = EXP(A)
C
C     CALCULATE THE COEFFICIENT OF CORRELATION.
C
      DO 110 I = 1,N
         SUMA = SUMA + (XX(I) - FX) * (Y(I) - FY)
         SUMB = SUMB + (XX(I) - FX) * * 2
         YB(I) = A + B * XX(I)
         SUMC = SUMC + (Y(I) - FY) * * 2
  110 CONTINUE
      RC = SUMA/SQRT(SUMB * SUMC)
C
C     CALCULATE THE STANDARD ERROR OF ESTIMATE.
C
      DO 120 I = 1,N
         SUMD = SUMD + (Y(I) - YB(I)) * * 2
  120 CONTINUE
      SBAR = SQRT(SUMD/(FLOAT(N) - 1.))
C
      RETURN
      END
```

Subroutine NVAL

```
      SUBROUTINE NVAL (NR)
C
C     THIS SUBROUTINE CALCULATES THE COEFFICIENT OF THE
C     CONVEYENCE EQUATION FOR THE CURRENT DISCHARGE.
C
C     THIS ALLOWS MANNING"S N TO BE A FUNCTION OF DISCHARGE.
C
      COMMON /SEC1/          WD(22)        , RD(100)      , TQ(125)      ,
     1       WSK(100)      , WE(100)       , ZMIN(100)    , ZOB(100)     ,
     2       A1(100)       , A2(100)       , A3(100)      , A4(100)      ,
     3       A5(100)       , A6(100)       , A7(100)      , A8(100)      ,
     4       A9(100)       , A10(100)      , B1(100)      , B2(100)      ,
     5       B3(100)       , B4(100)       , B5(100)      , B6(100)      ,
     6       B7(100)       , B8(100)       , B9(100)      , B10(100)     ,
     7       OA4(100)      , OA9(100)      , EDK(100)     , TAK(100)     ,
     8       TKK(100)      , ALPK(100)     , VK(100)      , WSMAX(100)   ,
     9       IMAX(100)
      COMMON /RIV/           NRIV          , KUP(10)      , KDOWN(10)    ,
     1       NTRIB(10)     , ICONT(10)     , BSC(10,5)    , AX(10)       ,
     2       BX(10)        , CX(10)        , RDT(10,5)    , ITRIB(10,5)  ,
     3       KTRIB(10,5)   , AT(10,5)      , BT(10,5)     , KCONT(10)    ,
     4       IROUT(10)     , SB(10)        , AN(10)       , BN(10)
      DATA RNMAX,RNMIN/1.4,0.67/
C
      KD = KDOWN(NR)
      KU = KUP(NR)
C
C     ITERATE OVER EACH CROSS SECTION IN THE REACH
C
      DO 100 K = KD,KU
         IF (TQ(K).EQ.0.0) GO TO 100
C
C     CALCULATE CORRECTION FACTOR
C
         RN = AN(NR) * ABS(TQ(K)) *  * BN(NR)
C
C     TEST FOR VALUE WITHIN LIMITS
C
         IF (RN.GT.RNMAX) RN = RNMAX
         IF (RN.LT.RNMIN) RN = RNMIN
         A4(K) = OA4(K)/RN
         A9(K) = OA9(K)/RN
  100 CONTINUE
C
      RETURN
      END
```

Subroutine OUT1

```
      SUBROUTINE OUT1
C
C     THIS SUBROUTINE OUTPUTS THE VARIOUS RESULTS OF THE SIMULATION
C     MODEL.
C
      COMMON /SEC1/          WD(22)        , RD(100)       , TQ(125)       ,
     1         WSK(100)    , WE(100)       , ZMIN(100)     , ZOB(100)      ,
     2         A1(100)     , A2(100)       , A3(100)       , A4(100)       ,
     3         A5(100)     , A6(100)       , A7(100)       , A8(100)       ,
     4         A9(100)     , A10(100)      , B1(100)       , B2(100)       ,
     5         B3(100)     , B4(100)       , B5(100)       , B6(100)       ,
     6         B7(100)     , B8(100)       , B9(100)       , B10(100)      ,
     7         DA4(100)    , DA9(100)      , EDK(100)      , TAK(100)      ,
     8         TKK(100)    , ALPK(100)     , VK(100)       , WSMAX(100)    ,
     9         IMAX(100)
      COMMON /SEC2/          X(100,22)     , Z(100,22)     , ZO(100,22)    ,
     1         DLOB(100)   , DROB(100)     , F(100,22)     , ND(100)       ,
     2         SUMDA(100)  , SMADA(100)    , SMZWS(100)    , IZMIN(100)    ,
     3         G(100)
      COMMON /INF/           NSEC          , NTIM          , DT            ,
     1         IORG        , CE            , CC            , PORM          ,
     2         STAGE       , ITIME         , NQI           , NCALL         ,
     3         ICALL(30,3) , MST           , EPS
      COMMON /RIV/           NRIV          , KUP(10)       , KDOWN(10)     ,
     1         NTRIB(10)   , ICONT(10)     , QSL(10,5)     , AX(10)        ,
     2         BX(10)      , CX(10)        , RDT(10,5)     , ITRIB(10,5)   ,
     3         KTRIB(10,5) , AT(10,5)      , BT(10,5)      , KCONT(10)     ,
     4         IROUT(10)   , SB(10)        , AN(10)        , BN(10)
      COMMON /PRT/           IPRNT(8)      , I5            , I6            ,
     1         I7          , I8            , I9            , I10
      DIMENSION              DZF(22)
C
C     PRINT CROSS SECTION NUMBER,EFFECTIVE WIDTH,EFFECTIVE DEPTH,TOTAL
C     AREA,TOTAL CONVEYANCE,ALPHA,VELOCITY,WATER SURFACE ELEVATION,
C     SEDIMENT TRANSPORT,DISCHARGE AND MINIMUM BED ELEVATION FOR EACH
C     TIME PERIOD.
C
      IF (IPRNT(6).NE.1) GO TO 100
      WRITE (I6,200) ITIME,DT
      WRITE (I6,210) ((K,WE(K),EDK(K),TAK(K),TKK(K),ALPK(K),VK(K),WSK(K)
     1,G(K),TQ(K),ZMIN(K),K = 1,NSEC))
  100 IF (ITIME.NE.NTIM) GO TO 150
C
C     PRINT MAXIMUM WATER SURFACE ELEVATION AND THE TIME PERIOD OF
C     OCCURANCE FOR EACH CROSS SECTION.
C
      IF (IPRNT(4).NE.1) GO TO 110
      WRITE (I6,220)
      WRITE (I6,230) (K,WSMAX(K),IMAX(K),K = 1,NSEC)
C
C     PRINT MINIMUM BED ELEVATION FOR EACH CROSS SECTION AT THE END OF
C     THE SIMULATION.
C
  110 IF (IPRNT(5).NE.1) GO TO 120
      WRITE (I6,240)
      WRITE (I6,250) (K,ZMIN(K),K = 1,NSEC)
  120 CONTINUE
C
C     PRINT OUT THE BED ELEVATION AND THE CHANGE IN BED ELEVATION
C     AT EACH POINT.
C
C
      IF (IPRNT(3).NE.1) GO TO 150
      WRITE (I6,190)
      DO 140 K = 1,NSEC
         N = ND(K)
         DO 130 L = 1,N
            DZF(L) = Z(K,L) - ZO(K,L)
  130    CONTINUE
         WRITE (I6,170) K
         WRITE (I6,180) (L,X(K,L),Z(K,L),DZF(L),L = 1,N)
  140 CONTINUE
C
C     BINARY OUTPUT.
C
  150 IF (IPRNT(7).NE.1) RETURN
C
C     PRINT CROSS SECTION NUMBER,EFFECTIVE WIDTH,EFFECTIVE DEPTH,TOTAL
C     AREA,TOTAL CONVEYANCE,ALPHA,VELOCITY,WATER SURFACE ELEVATION,
C     SEDIMENT TRANSPORT,DISCHARGE AND MINIMUM BED ELEVATION FOR EACH
C     TIME PERIOD.
C
      WRITE (I10) ((WE(K),EDK(K),TAK(K),TKK(K),ALPK(K),VK(K),WSK(K),G(K)
     1,ZMIN(K),K = 1,NSEC),(TQ(KI),KI = 1,NQI),((QSL(N,J),J = 1,5),N = 1
     2,NRIV))
C
C     DETERMINE IF THE CURRENT TIME PERIOD IS THE END OF A YEAR
C
      RI = FLOAT(ITIME)
      RI = RI/52.18
      RI = RI - AINT(RI)
      IF (RI.GT.0.9904.OR.RI.LT.0.00958) GO TO 160
      RETURN
```

Subroutine OUT1 continued

```
C
C     PRINT OUT THE BED ELEVATION AT EACH POINT
C
  160 CONTINUE
      WRITE (I9) ((Z(K,L),K = 1,NSEC),L = 1,22)
      RETURN
C
C
C
  170 FORMAT (//,20X,17HCROSS SECTION NO.,I5,/,20X,17HPOINT   HORIZONTAL,
     124H  ELEVATION  DELTA ELEV.,/)
  180 FORMAT (21X,I2,5X,F6.0,5X,F6.1,5X,F6.1)
  190 FORMAT (1H1,19X,46HF I N A L   B E D   E L E V A T I O N S,   A N D,/
     1,20X,47HT O T A L   C H A N G E   A T   E A C H   P O I N T)
  200 FORMAT (//,10X,47HC R O S S   S E C T I O N   P R O P E R T I E S,
     1/,10X,4HTIME,I5,F10.2,/,12X,108HSECTION  EFFECTIVE EFFECTIVE    TOT
     2AL       TOTAL      ALPHA    VELOCITY      WATER    SEDIMENT     FLOW
     3 THALWEG,/,23X,37HWIDTH        DEPTH        AREA    CONVEYANCE,22X,17HSUR
     4FACE TRANSPORT,15X,9HELEVATION,/)
  210 FORMAT (13X,I3,3X,2F10.1,2F10.0,2F10.4,F10.2,F10.6,F10.2,F10.2)
  220 FORMAT (1H1,8X,3HMAX,7X,4HTIME,/,9X,5HWATER,5X,2HOF,/,8H SECTION,1
     1X,9HELEVATION,1X,3HMAX,/)
  230 FORMAT (1H ,I4,F10.2,2X,I4)
  240 FORMAT (1H1,8X,3HMIN,/,9X,3HBED,/,18H SECTION ELEVATION,/)
  250 FORMAT (1H ,I4,3X,F10.2)
      END
```

Subroutine RIVDS

```
      SUBROUTINE RIVDS (NR,DIS)
C
C     THIS SUBROUTINE CONVERTS THE RIVER DISTANCE FROM MILES TO FEET,
C     OR FROM KILOMETERS TO METERS.
C
C     IT MAY BE USED TO CORRECT FOR CUTTOFFS OR CHANGES IN RIVER
C     ALIGNMENT BY ADDING SPECIAL LOGIC.
C
      COMMON /UNITS/        IUNIT        , CORDS         , ZDIFM         ,
     1        GRAV         , VVAL         , FET           , CONV          ,
     2        DZMAX
C
      DIS = DIS * CORDS
C
      RETURN
      END
```

Subroutine SED

```
      SUBROUTINE SED
C
C     THIS SUBROUTINE CALULATES SEDIMENT TRANSPORT USING THE
C     GENERALIZED FORMULA DEVELOPED FOR THE YAZOO RIVER.
C
      COMMON /SEC1/          WD(22)        , RD(100)       , TQ(125)       ,
     1        WSK(100)     , WE(100)       , ZMIN(100)     , ZOB(100)      ,
     2        A1(100)      , A2(100)       , A3(100)       , A4(100)       ,
     3        A5(100)      , A6(100)       , A7(100)       , A8(100)       ,
     4        A9(100)      , A10(100)      , B1(100)       , B2(100)       ,
     5        B3(100)      , B4(100)       , B5(100)       , B6(100)       ,
     6        B7(100)      , B8(100)       , B9(100)       , B10(100)      ,
     7        DA4(100)     , DA9(100)      , EDK(100)      , TAK(100)      ,
     8        TKK(100)     , ALPK(100)     , VK(100)       , WSMAX(100)    ,
     9        IMAX(100)
      COMMON /SEC2/          X(100,22)     , Z(100,22)     , ZO(100,22)    ,
     1        DLOB(100)    , DROB(100)     , F(100,22)     , ND(100)       ,
     2        SUMDA(100)   , SMADA(100)    , SMZWS(100)    , IZMIN(100)    ,
     3        G(100)
      COMMON /RIV/           NRIV          , KUP(10)       , KDOWN(10)     ,
     1        NTRIB(10)    , ICONT(10)     , DSL(10,5)     , AX(10)        ,
     2        BX(10)       , CX(10)        , ROT(10,5)     , ITRIB(10,5)   ,
     3        KTRIE(10,5)  , AT(10,5)      , BT(10,5)      , KCONT(10)     ,
     4        IROUT(10)    , SB(10)        , AN(10)        , BN(10)
      COMMON /INF/           NSEC          , NTIM          , DT            ,
     1        IORG         , CE            , CC            , PORM          ,
     2        STAGE        , ITIME         , NQI           , NCALL         ,
     3        ICALL(30,3)  , MST           , EPS
C
C     ITERATE OVER EACH RIVER REACH
C
      DO 120 NR = 1,NRIV
          KU = KUP(NR)
          KD = KDOWN(NR)
C
C     ITERATE OVER EACH CROSS SECTION IN THE REACH
C
         DO 110 K = KD,KU
            IF (TQ(K).EQ.0.0) GO TO 100
C
C     CALCLATE THE SEDIMENT TRANSPORT FOR THE CROSS SECTION
C
            G(K) = ABS(VK(K)) ** 3.16 * EDK(K) ** 0.96 * WE(K) ** 4.4
     1      8E - 06
            IF (TQ(K).LT.0.0) G(K) =  - 1. * G(K)
            GO TO 110
  100       G(K) = 0.
  110    CONTINUE
  120 CONTINUE
C
      RETURN
      END
```

Subroutine SPLIT

```
      SUBROUTINE SPLIT (NR1,NR2,ERROR,P)
C
C     THIS SUBROUTINE IS USED IN DIVIDED FLOW PROBLEMS TO SPLIT THE
C     DISCHARGE BETWEEN TWO CHANNELS.
C
      COMMON /SEC1/          WD(22)       , RD(100)       , TQ(125)       ,
     1           WSK(100)  , WE(100)      , ZMIN(100)     , ZOB(100)      ,
     2           A1(100)   , A2(100)      , A3(100)       , A4(100)       ,
     3           A5(100)   , A6(100)      , A7(100)       , A8(100)       ,
     4           A9(100)   , A10(100)     , B1(100)       , B2(100)       ,
     5           B3(100)   , B4(100)      , B5(100)       , B6(100)       ,
     6           B7(100)   , B8(100)      , B9(100)       , B10(100)      ,
     7           UA4(100)  , UA9(100)     , EDK(100)      , TAK(100)      ,
     8           IKK(100)  , ALPK(100)    , VK(100)       , WSMAX(100)    ,
     9           IMAX(100)
      COMMON /RIV/           NRIV         , KUP(10)       , KDOWN(10)     ,
     1           NTRIB(10) , ICONT(10)    , DSL(10,5)     , AX(10)        ,
     2           BX(10)    , CX(10)       , RDT(10,5)     , ITRIB(10,5)   ,
     3           KTRIB(10,5), AT(10,5)    , BT(10,5)      , KCONT(10)     ,
     4           IROUT(10) , SB(10)       , AN(10)        , BN(10)
C
  100 KD1 = KDOWN(NR1)
      KU1 = KUP(NR1)
      KUP1 = KU1 + 1
      KUM1 = KD1 - 1
      NT = NTRIB(NR1)
C
C     CALCULATE THE FLOW IN THE FIRST REACH
C
      DO 130 K = KUP1,KUM1
         TQ(K) = TQ(KU1) * P
         IF (NT.LE.2) GO TO 130
C
C     ADD TRIBUTARY FLOW IF ANY
C
         NTM1 = NT - 1
         DO 120 J = 2,NTM1
            IF (RD(K).GE.RDT(NR1,J)) GO TO 120
            KT = KTRIB(NR1,J)
            IF (ITRIB(NR1,J).GT.2) GO TO 110
            TQ(K) = TQ(K) + TQ(KT)
            GO TO 120
  110       TQ(K) = TQ(K) - TQ(KT)
  120    CONTINUE
  130 CONTINUE
C
C     SET THE UPSTREAM AND DOWNSTREAM FLOW OF BOTH REACHES EQUAL
C
      KD2 = KDOWN(NR2)
      KU2 = KUP(NR2)
      KUP1 = KU2 + 1
      KUM1 = KD2 - 1
      TQ(KD2) = TQ(KD1)
      TQ(KU2) = TQ(KU1)
      NT = NTRIB(NR2)
C
C     CALCULATE THE FLOW IN THE SECOND REACH
C
      DO 160 K = KUP1,KUM1
         TQ(K) = TQ(KU1) * (1. - P)
         IF (NT.LE.2) GO TO 160
C
C     ADD TRIBUTARY FLOW IF ANY
C
         NTM1 = NT - 1
         DO 150 J = 2,NTM1
            IF (RD(K).GE.RDT(NR2,J)) GO TO 150
            KT = KTRIB(NR2,J)
            IF (ITRIB(NR2,J).GT.2) GO TO 140
            TQ(K) = TQ(K) + TQ(KT)
            GO TO 150
  140       TQ(K) = TQ(K) - TQ(KT)
  150    CONTINUE
  160 CONTINUE
C
C     CALCULATE THE WATER PROFILE IN BOTH REACHES
C
      CALL SUBPF (NR1)
      CALL SUBPF (NR2)
C
C     COMPUTE THE ERROR IN THE WATER SURFACE ELEVATIONS
C
      ERROR = WSK(KU1) - WSK(KU2)
      ERROR = ERROR * ERROR
C
      RETURN
      END
```

Subroutine SROUT

```
      SUBROUTINE SROUT (NR)
C
C     THIS SUBROUTINE ROUTES THE SEDIMENT, CALCULATES THE
C     APPROXIMATE BED ELEVATION CHANGE, AND IF NECESSARY,
C     DISTRIBUTES THE AGGRADATION OR DEGRADATION THROUGH
C     THE CROSS SECTION.
C
      COMMON /SEC1/        WD(22)       , RD(100)      , TQ(125)      ,
     1        WSK(100)    , WE(100)      , ZMIN(100)    , ZOB(100)     ,
     2        A1(100)     , A2(100)      , A3(100)      , A4(100)      ,
     3        A5(100)     , A6(100)      , A7(100)      , A8(100)      ,
     4        A9(100)     , A10(100)     , B1(100)      , B2(100)      ,
     5        B3(100)     , B4(100)      , B5(100)      , B6(100)      ,
     6        B7(100)     , B8(100)      , B9(100)      , B10(100)     ,
     7        DA4(100)    , DA9(100)     , EDK(100)     , TAK(100)     ,
     8        TKK(100)    , ALPK(100)    , VK(100)      , WSMAX(100)   ,
     9        IMAX(100)
      COMMON /SEC2/        X(100,22)    , Z(100,22)    , ZO(100,22)   ,
     1        DLOB(100)   , DROB(100)    , F(100,22)    , ND(100)      ,
     2        SUMDA(100)  , SMADA(100)   , SMZWS(100)   , IZMIN(100)   ,
     3        G(100)
      COMMON /UNITS/       IUNIT        , CORDS        , ZDIFM        ,
     1        GRAV        , VVAL         , FET          , CONV         ,
     2        DZMAX
      COMMON /INF/         NSEC         , NTIM         , DT           ,
     1        IORG        , CE           , CC           , PORM         ,
     2        STAGE       , ITIME        , NQI          , NCALL        ,
     3        ICALL(30,3) , MST          , EPS
      COMMON /RIV/         NRIV         , KUP(10)      , KDOWN(10)    ,
     1        NTRIB(10)   , ICONT(10)    , QSL(10,5)    , AX(10)       ,
     2        BX(10)      , CX(10)       , RDT(10,5)    , ITRIB(10,5)  ,
     3        KTRIB(10,5) , AT(10,5)     , BT(10,5)     , KCONT(10)    ,
     4        IROUT(10)   , SB(10)       , AN(10)       , BN(10)
      DIMENSION            DV(100)
C
C     ITERATE OVER EACH CROSS SECTION IN THE RIVER SEGMENT.
C
      KU = KUP(NR)
      KD = KDOWN(NR)
      KDP1 = KD + 1
      KUM1 = KU - 1
      DO 120 K = KD,KUM1
         KP1 = K + 1
C
C     CALCULATE  THE VOLUME OF SEDIMENT DEPOSITED, OR ERODED.
C
         DV(K) = (G(KP1) - G(K)) * 86400. * DT
C
C     ADD IN LATERAL SEDIMENT INFLOW IF ANY.
C
         JT = NTRIB(NR)
         IF (JT.EQ.0) GO TO 110
         DO 100 J = 1,JT
            IF (RDT(NR,J).GE.RD(K).AND.RDT(NR,J).LT.RD(KP1)) DV(K) = DV(
     1      K) + QSL(NR,J) * 86400.0 * DT
  100    CONTINUE
  110    DV(K) = (DV(K)/(1.0 - PORM))
  120 CONTINUE
      DXUP = RD(KDP1) - RD(KD)
      DO 210 K = KD,KUM1
         IF (ABS(TQ(K)).EQ.0.0) GO TO 210
         KM1 = K - 1
         KP1 = K + 1
         DXDWN = DXUP
         DXUP = RD(KP1) - RD(K)
C
C      TEST FOR WEIRS
C
         IF ((DXDWN/CORDS).LT.0.001) GO TO 210
         IF ((DXUP/CORDS).LT.0.001) GO TO 210
         IF (K.GT.KD) GO TO 140
         IROUT = IROUT(NR)
         GO TO (210,140,130), IROUT
C
C     CALCULATE THE CHANGE IN THE AREA AT THE CROSS SECTION
C
  130    DA = 0.5 * DV(K)/DXUP
         GO TO 150
  140    DA = (1.5 * DV(K) + 0.5 * DV(KM1))/(DXUP + DXDWN)
  150    SUMDA(K) = SUMDA(K) + DA
C
C     CALCULATE THE APPROXIMATE BED ELEVATION CHANGE
C     SINCE THE LAST DISTRIBUTION OF SEDIMENT.
C
         SUMDZ = SUMDA(K)/WE(K)
         SMZWS(K) = SMZWS(K) + WSK(K) * ABS(DA)
         SMADA(K) = SMADA(K) + ABS(DA)
```

Subroutine SROUT continued

```
C
C     TEST TO SEE IF THE BED ELEVATION HAS CHANGED ENOUGH TO
C     REQUIRE DISTRIBUTING THE SEDIMENT THROUGH THE CROSS SECTION
C
         ADZ = ABS(SUMDZ)
         IF (ADZ.GT.DZMAX) GO TO 160
         IF (ITIME.EQ.NTIM.AND.SMADA(K).GT.0.) GO TO 160
         GO TO 210
C
C     USE THE WEIGHTED AVERAGE OF WATER SURFACE ELEVATIONS FOR
C     DISTRIBUTING THE SEDIMENT.
C
  160    NDIS = IFIX(ADZ/(DZMAX * 4.))
         IF (NDIS.LT.1) NDIS = 1
         SDA = SUMDA(K)/FLOAT(NDIS)
         WS = SMZWS(K)/SMADA(K)
         DO 200 NDS = 1,NDIS
            ZMP3 = ZMIN(K) + 3.
            IF (WS.LT.ZMP3) WS = ZMP3
            CALL GEOM (K,WS)
            N = ND(K)
            NP = N - 1
C
C     ADD IN CHANGE IN BED ELEVATION, AT EACH POINT, TO
C     CROSS SECTION GEOMETRY.
C
            DX = X(K,2) - X(K,1)
            IF (DX.EQ.0.) GO TO 170
            Z(K,1) = Z(K,1) + (SDA * WD(1))/DX
  170       DO 180 L = 2,NP
               DX = X(K,L + 1) - X(K,L - 1)
               IF (DX.EQ.0.) GO TO 180
               Z(K,L) = Z(K,L) + (SDA * (WD(L) + WD(L - 1)))/DX
  180       CONTINUE
            DX = X(K,N) - X(K,N - 1)
            IF (DX.EQ.0.) GO TO 190
            Z(K,N) = Z(K,N) + (SDA * WD(N - 1))/DX
  190       CALL THAL (K)
  200    CONTINUE
C
C     CALCULATE POWER FUNCTIONS FOR THE CHANGED SECTION.
C
         CALL CHNGM (K)
C
C     SET SMADA (K),SMZWS (K), AND SUMDA(K), BACK TO ZERO.
C
         SMADA(K) = 0.
         SMZWS(K) = 0.
         SUMDA(K) = 0.
  210 CONTINUE
C
      RETURN
      END
```

Subroutine SUBPF

```
      SUBROUTINE SUBPF (NR)
C
C     THIS SUBROUTINE CALLS THE VARIOUS OTHER SUBROUTINES NEEDED TO
C     CALCULATE THE SUBCRITICAL WATER SURFACE ELEVATION AT EACH SECTION.
C
      COMMON /SEC1/          WD(22)         , RD(100)        , TQ(125)         ,
     1       WSK(100)      , WE(100)        , ZMIN(100)      , ZOB(100)        ,
     2       A1(100)       , A2(100)        , A3(100)        , A4(100)         ,
     3       A5(100)       , A6(100)        , A7(100)        , A8(100)         ,
     4       A9(100)       , A10(100)       , B1(100)        , B2(100)         ,
     5       B3(100)       , B4(100)        , B5(100)        , B6(100)         ,
     6       B7(100)       , B8(100)        , B9(100)        , B10(100)        ,
     7       DA4(100)      , DA9(100)       , EDK(100)       , TAK(100)        ,
     8       TKK(100)      , ALPK(100)      , VK(100)        , WSMAX(100)      ,
     9       IMAX(100)
      COMMON /UNITS/         IUNIT          , CORDS          , ZDIFM           ,
     1       GRAV          , VVAL           , FET            , CONV            ,
     2       DZMAX
      COMMON /HYD/           V              , ED             , EW              ,
     1       ALP           , TK             , TA
      COMMON /RIV/           NRIV           , KUP(10)        , KDOWN(10)       ,
     1       NTRIB(10)     , ICONT(10)      , DSL(10,5)      , AX(10)          ,
     2       BX(10)        , CX(10)         , ROT(10,5)      , ITRIB(10,5)     ,
     3       KTRIB(10,5)   , AT(10,5)       , BT(10,5)       , KCONT(10)       ,
     4       IROUT(10)     , SB(10)         , AN(10)         , BN(10)
      COMMON /SYS/           VHD            , WSD            , TKD             ,
     1       DEX
C
C     DETERMINE THE COEFFICIENT OF THE CONVAYENCE EQUATION
C     FOR THE CURRENT DISCHARGE.
C
      CALL NVAL (NR)
C
C     CALCULATE THE WATER SURFACE ELEVATION AT THE DOWNSTREAM
C     SECTION, BY USING THE CONTROL CONDITIONS.
C
      CALL CONT (NR,WS)
      K = KDOWN(NR)
      IF (WS.LE.ZMIN(K)) GO TO 100
      CALL HYDPR (K,WS)
C
C     TEST FOR CRITICAL FLOW.
C
      CRT = TQ(K)/((GRAV/ALP) * * .5)
      ZSU = TA * ((TA/EW) * * .5)
      IF (CRT.LT.ZSU) GO TO 130
C
C     CALCULATE CRITICAL DEPTH.
C
  100 IF (TQ(K).GT.0.0) GO TO 110
      WS = WSD
      ED = 0.0
      EW = 0.0
      TA = 0.0
      TK = 0.0
      ALP = 0.0
      V = 0.0
  110 D = (TQ(K) * TQ(K) * A5(K))/(A3(K) * A3(K) * GRAV)
      D = D * * (1./(1. + 2. * B3(K) - B5(K)))
      WS = D + ZMIN(K)
      IF (WS.LT.ZOB(K)) GO TO 120
      D = (TQ(K) * TQ(K) * A10(K))/(A8(K) * A8(K) * GRAV)
      D = D * * (1./(1. + 2. * B8(K) - B10(K)))
      WS = D + ZMIN(K)
C
C     CALCULATE THE HYDRAULIC PROPERTIES OF THE SECTION.
C
  120 CALL HYDPR (K,WS)
  130 WE(K) = EW
      WSK(K) = WS
      TAK(K) = TA
      EDK(K) = ED
      TKK(K) = TK
      ALPK(K) = ALP
      VK(K) = V
C
C     ITERATE OVER THE REMAINDER OF THE SECTIONS, BY WORKING
C     UPSTREAM ONE SECTION AT A TIME.
C
      KU = KUP(NR)
      K2 = KDOWN(NR) + 1
      DO 220 K = K2,KU
         ICRI = 0
C
C     SET THE LAST SECTIONS HYDRAULIC PROPERTIES AS THE DOWNSTREAM
C     CONDITIONS.
C
```

Subroutine SUBPF continued

```
      KD = K - 1
      DEX = RD(K) - RD(KD)
      VHD = ALP * V * V/(2. * GRAV)
      WSD = WS
      TKD = TK
      IF (TQ(K).GT.0.0) GO TO 150
      WS = WSD
      IF (WS.LE.ZMIN(K)) GO TO 140
      CALL HYDPR (K,WS)
      GO TO 210
  140 EW = 0.0
      ED = 0.0
      TA = 0.0
      TK = 0.0
      ALP = 0.0
      V = 0.0
      GO TO 210
C
C     TEST FOR WEIR SECTION
C
  150 IF (DEX.GT.0.00001) GO TO 160
C
C     CALCULATE THE WATER SURFACE AT THE UPTREAM WEIR SECTION
C
      CALL WEIR (K,WS)
      GO TO 210
C
C     CALCULATE THE WATER SURFACE ELEVATION AT SECTION(K).
C
  160 CALL BKWAT (K,WS,ICRI)
C
C     TEST FOR CRITICAL FLOW.
C
      IF (ICRI.EQ.1) GO TO 170
      CRT = TQ(K)/((GRAV/ALP) * * .5)
      ZSQ = TA * ((TA/EW) * * .5)
      IF (CRT.LE.ZSQ) GO TO 210
C
C     CALCULATE CRITICAL DEPTH.
C
  170 D = (TQ(K) * TQ(K) * A5(K))/(A3(K) * A3(K) * GRAV)
      D = D * * (1./(1. + 2. * B3(K) - B5(K)))
      WS = D + ZMIN(K)
      IF (WS.LT.Z08(K)) GO TO 180
      D = (TQ(K) * TQ(K) * A10(K))/(A8(K) * A8(K) * GRAV)
      D = D * * (1./(1. + 2. * B8(K) - B10(K)))
      WS = D + ZMIN(K)
  180 IF (WS.LT.WSD) WS = WSD
      CALL HYDPR (K,WS)
      IF (V.LT.VVAL) GO TO 210
C
C      CALCULATE NORMAL DEPTH
C
      C = TQ(K)/SB(NR) * * .5
      D = (C/A4(K)) * * (1./B4(K))
      WS = ZMIN(K) + D + .001
      IF (WS.LT.WSD) WS = WSD
      IF (WS.LT.Z08(K)) GO TO 190
      D = (C/A9(K)) * * (1./B9(K))
      WS = ZMIN(K) + D + .001
      IF (WS.LT.WSD) WS = WSD
  190 CALL HYDPR (K,WS)
      IF (V.LT.VVAL) GO TO 210
C
C     VELOCITY LIMITED
C
      D = (TQ(K)/(VVAL * A3(K))) * * (1.0/B3(K))
      WS = D + ZMIN(K)
      IF (WS.LT.WSD) WS = WSD
      IF (WS.LT.Z08(K)) GO TO 200
      D = (TQ(K)/(VVAL * A8(K))) * * (1.0/B8(K))
      WS = D + ZMIN(K)
      IF (WS.LT.WSD) WS = WSD
  200 CALL HYDPR (K,WS)
C
C     SET CROSS SECTION HYDRAULIC PROPERTIES INTO ARRAYS
C
  210 WE(K) = EW
      WSK(K) = WS
      TAK(K) = TA
      EDK(K) = ED
      TKK(K) = TK
      ALPK(K) = ALP
      VK(K) = V
  220 CONTINUE
C
      RETURN
      END
```

Subroutine THAL

```
      SUBROUTINE THAL (K)
C
C     THIS SUBROUTINE DETERMINES THE CROSS SECTION THALWEG ELEVATION
C
      COMMON /SEC1/         WD(22)      , RD(100)     , TQ(125)     ,
     1       WSK(100)     , WE(100)     , ZMIN(100)   , ZOB(100)    ,
     2       A1(100)      , A2(100)     , A3(100)     , A4(100)     ,
     3       A5(100)      , A6(100)     , A7(100)     , A8(100)     ,
     4       A9(100)      , A10(100)    , B1(100)     , B2(100)     ,
     5       B3(100)      , B4(100)     , B5(100)     , B6(100)     ,
     6       B7(100)      , B8(100)     , B9(100)     , B10(100)    ,
     7       DA4(100)     , DA9(100)    , EDK(100)    , TAK(100)    ,
     8       TKK(100)     , ALPK(100)   , VK(100)     , WSMAX(100)  ,
     9       IMAX(100)
      COMMON /SEC2/         X(100,22)   , Z(100,22)   , ZD(100,22)  ,
     1       DLOB(100)    , DROB(100)   , F(100,22)   , ND(100)     ,
     2       SUMDA(100)   , SMADA(100)  , SMZWS(100)  , IZMIN(100)  ,
     3       G(100)
C
C     TEST EACH CROSS SECTION POINT TO FIND THE MINIMUM ELEVATION,
C     (ZMIN(K)).
C
      M = ND(K)
      ZMIN(K) = Z(K,1)
      IZMIN(K) = 1
      DO 100 L = 1,M
         IF (Z(K,L).GE.ZMIN(K)) GO TO 100
         ZMIN(K) = Z(K,L)
         IZMIN(K) = L
  100 CONTINUE
C
      RETURN
      END
```

Subroutine TRIBS

```
      SUBROUTINE TRIBS
C
C     THIS SUBROUTINE DETERMINES THE SEDIMENT DISCHARGE FOR
C     EACH TRIBUTARY.
C
C     DEFINITIONS OF TYPES OF TRIBUTARIES
C
C     ITRIB(NR,J)         TYPE OF TRIBUTARY
C
C         1                  POINT SOURCE IN
C         2                  MAJOR TRIBUTARY IN
C         3                  POINT SOURCE OUT
C         4                  MAJOR TRIBUTARY OUT
C
      COMMON /SEC1/         WD(22)        , HD(100)       , TQ(125)       ,
     1          WSK(100)    , WE(100)     , ZMIN(100)     , ZOB(100)      ,
     2          A1(100)     , A2(100)     , A3(100)       , A4(100)       ,
     3          A5(100)     , A6(100)     , A7(100)       , A8(100)       ,
     4          A9(100)     , A10(100)    , B1(100)       , B2(100)       ,
     5          B3(100)     , B4(100)     , B5(100)       , B6(100)       ,
     6          B7(100)     , B8(100)     , B9(100)       , B10(100)      ,
     7          QA4(100)    , QA9(100)    , EDK(100)      , TAK(100)      ,
     8          TKK(100)    , ALPK(100)   , VK(100)       , WSMAX(100)    ,
     9          TMAX(100)
      COMMON /SEC2/         X(100,22)     , Z(100,22)     , ZO(100,22)    ,
     1          DLOB(100)   , DROB(100)   , F(100,22)     , NO(100)       ,
     2          SUMDA(100)  , SMADA(100)  , SMZWS(100)    , IZMIN(100)    ,
     3          G(100)
      COMMON /INF/          NSEC          , NTIM          , DT            ,
     1          IORG        , CE          , CC            , PORM          ,
     2          STAGE       , ITIME       , NQI           , NCALL         ,
     3          ICALL(30,3) , MST         , EPS
      COMMON /RIV/          NRIV          , KUP(10)       , KDOWN(10)     ,
     1          NTRIB(10)   , ICONT(10)   , QSL(10,5)     , AX(10)        ,
     2          BX(10)      , CX(10)      , QOT(10,5)     , ITRIB(10,5)   ,
     3          KTRIB(10,5) , AT(10,5)    , BT(10,5)      , KCONT(10)     ,
     4          IROUT(10)   , SB(10)      , AN(10)        , BN(10)
C
C     ITERATE OVER EACH RIVER SEGMENT.
C
      DO 150 NR = 1,NRIV
         NT = NTRIB(NR)
         IF (NT.EQ.0) GO TO 150
         DO 140 J = 1,NT
            K = KTRIB(NR,J)
C
C     DETERMINE THE TYPE OF TRIBUTARY.
C
            ITRYB = ITRIB(NR,J)
            GO TO (100,110,120,130), ITRYB
C
C     CALCULATE SEDIMENT DISCHARGE FOR POINT SOURCE BY RATING CURVE
C
  100       QSL(NR,J) = AT(NR,J) * TQ(K) ** BT(NR,J)
            GO TO 140
C
C     FOR MAJOR TRIBUTARIES SET SEDIMENT DISCHARGE TO SEDIMENT
C     TRANSPORT AT DOWNSTREAM TRIBUTARY SECTION.
C
  110       QSL(NR,J) = G(K)
            GO TO 140
C
C     POINT SOURCE OUT
C
  120       QSL(NR,J) = AT(NR,J) * TQ(K) ** BT(NR,J)
            QSL(NR,J) =  - QSL(NR,J)
            GO TO 140
C
C     MAJOR TRIBUTARY OUT
C
  130       QSL(NR,J) =  - G(K)
  140    CONTINUE
  150 CONTINUE
C
      RETURN
      END
```

Subroutine UNIT

```
      SUBROUTINE UNIT
C
C     THIS SUBROUTINE ASSIGNES THE CORRECT VALUES TO THE
C     CONSTANTS ACCORDING TO THE UNIT-SYSTEM USED.
C
C        IUNIT        UNITS
C
C         1          ENGLISH UNIT-SYSTEM
C         2          METRIC SYSTEM (SI)
C
C
      COMMON /UNITS/          IUNIT        , CORDS        , ZDIFM        ,
     1        GRAV         , VVAL         , FET          , CONV         ,
     2        DZMAX
C
      IF (IUNIT.NE.1) GO TO 100
      CORDS = 5280.
      ZDIFM = 10.
      GRAV = 32.2
      VVAL = 10.
      FET = 5.
      CONV = 1.486
      DZMAX = 0.5
      RETURN
  100 CORDS = 1000.
      ZDIFM = 3.0
      GRAV = 9.81
      VVAL = 3.3
      FET = 1.5
      CONV = 1.0
      DZMAX = 0.15
C
      RETURN
      END
```

Subroutine WEIR

```
      SUBROUTINE WEIR (K,WS)
C
C     THIS SUBROUTINE IS USED TO CALCULATE THE GREATER OF THE DOWNSTREAM
C     WATER SURFACE OR THE CRITICAL DEPTH AT THE WEIR.
C
      COMMON /SEC1/           WD(22)        , RD(100)       , TQ(125)       ,
     1        WSK(100)      , WE(100)       , ZMIN(100)     , ZOB(100)      ,
     2        A1(100)       , A2(100)       , A3(100)       , A4(100)       ,
     3        A5(100)       , A6(100)       , A7(100)       , A8(100)       ,
     4        A9(100)       , A10(100)      , B1(100)       , B2(100)       ,
     5        B3(100)       , B4(100)       , B5(100)       , B6(100)       ,
     6        B7(100)       , B8(100)       , B9(100)       , B10(100)      ,
     7        DA4(100)      , DA9(100)      , EDK(100)      , TAK(100)      ,
     8        TKK(100)      , ALPK(100)     , VK(100)       , WSMAX(100)    ,
     9        IMAX(100)
      COMMON /SYS/            VHD           , WSD           , TKD           ,
     1        DEX
      COMMON /HYD/            V             , ED            , EW            ,
     1        ALP           , TK            , TA
      DATA WID/200.0/
C
C     CALCULATE DEPTH BY WEIR EQUATION
C
      D = (TQ(K)/(WID * 2.85)) ** 0.666666
      WS = D + ZMIN(K) + 0.001
C
C     TEST FOR GREATEST OF DOWNSTREAM OR WEIR WATER SURFACE ELEVATIONS
C
      IF (WS.LT.WSD) WS = WSD
      CALL HYDPR (K,WS)
C
C     SET HYDRAULIC PROPERTIES EQUAL AT THE WEIR CROSS SECTIONS
C
      KM1 = K - 1
      WSK(KM1) = WS
      VK(KM1) = V
      TAK(KM1) = TA
      TKK(KM1) = TK
      EDK(KM1) = ED
      WE(KM1) = EW
      ALPK(KM1) = ALP
C
      RETURN
      END
```

APPENDIX E

KUWASER PROGRAM SPECIFICATIONS

KUWASER PROGRAM SPECIFICATIONS

A. TITLE OF PROGRAM: KUWASER, known discharge, uncoupled, sediment routing.

B. AUTHOR: Glenn O. Brown

C. DATE PROGRAM COMPLETED: July 1979

D. PURPOSE OF PROGRAM: To compute the spatially varied flow profile, sediment transport, and aggradation-degradation in rivers. The program can be applied to a single stream or an entire river basin, accomodating divided flow rivers.

E. EQUIPMENT REQUIREMENTS:

Language: ANSI FORTRAN IV

Central Memory Core Storage: Problem dependent (61000_8 on CDC 172, with arrays dimensioned for 100 cross sections, 10 river reaches and 10 tributaries per reach.)
Central-Processor Time: 0.01 to 0.05 c.p. seconds on CDC 172 for one time-space calculation.

Peripheral Equipment: Minimum requirements are printer and one input device, such as a card reader. The program has refined input-output features which utilize up to three input devices and three output devices.

F. SIZE OF OBJECT CODE: 2300, 80 character lines.

G. INPUT: Requirements include digitized channel cross sections, Manning's n values for channel and overbank, river system configuration, downstream stage control, point source tributary sediment rating curves and the known discharge at each cross section.

H. OUTPUT: Output is user controlled. Output options include aggradation-degradation at each cross section, maximum water surface elevations, minimum bed elevations, and cross section hydraulic properties (area, depth, conveyance, velocity, sediment transport, alpha, discharge and water surface elevation) for each time period.

I. GENERAL EQUATIONS: The program solves the spatially varied flow, Manning's and sediment continuity equations. These equations are:

Spatially Varied Flow equation

$$\frac{dD}{dx} = S_o - S_h - \frac{d}{dx}\left(\alpha \frac{V^2}{2g}\right)$$

Manning's Equation

$$V = \frac{1.486}{n} R^{2/3} S_f^{1/2} \quad \text{(English Units)}$$

Sediment Continuity

$$\frac{\partial Q_s}{\partial x} + (1 - \rho) \frac{\partial A_b}{\partial t} = q_{s\ell}$$

Where D is the thalweg depth, x is the distance along the channel, S_o is the bed slope, S_h is the total head slope, α is the velocity distribution coefficient, g is the acceleration of gravity, V is the average flow velocity, n is the Manning's n-value, S_f is the friction slope, R is the hydraulic radius, Q_s is the volume rate of sediment transport, ρ is the sediment deposit porosity, A_b is the area of bed area deposited or eroded, t is time, and $q_{s\ell}$ is the lateral sediment input per unit length of channel.

J. RANGE OF APPLICATION: The program is presently limited to subcritical flow, and cannot predict channel armoring or two-dimensional flow effects. The known discharge uncoupled formulation limits the model to cases where the change in the bed is small during any one time period, and the rate of change of the water hydrograph is small.

K. DIMENSION SYSTEM: English or SI.