

# A diffusion hydrodynamic model (DHM)

T. V. Hromadka II and C. C. Yen

Hydrologists, Williamson and Schmid, 17782 Sky Park Boulevard, Irvine, CA 92714, USA

A diffusion hydrodynamic model of coupled two-dimensional overland flow and one-dimensional open channel flow (DHM) is developed. Because of the diffusion form of the governing flow equations is used in this model, several important hydraulic effects are accommodated which are incapable of being handled by the often-used kinematic routing techniques which are used in most watershed models; namely, backwater effects, channel overflow, combined overland flow and storage effects, and ponding. Because these often ignored hydraulic effects are important in drainage studies involving flood control channel deficiencies and subtle grade differences between watershed boundaries (e.g. alluvial fan hydrology), the DHM approach affords the practicing hydrologist a new tool for drainage system evaluations.

## INTRODUCTION

A diffusion hydrodynamic model of coupled two-dimensional overland flow and one-dimensional open channel flow (DHM) is developed. Because the diffusion form of the governing flow equations is used in this model, several important hydraulic effects are accommodated which are incapable of being handled by the often-used kinematic routing techniques which are used in most watershed models; namely, backwater effects, channel overflow, combined overland flow and storage effects, and ponding. Because these often ignored hydraulic effects are important in drainage studies involving flood control channel deficiencies and subtle grade differences between watershed boundaries (e.g. alluvial fan hydrology), the DHM approach affords the practicing hydrologist a new tool for drainage system evaluations.

This paper is organized into six working sections as follows:

Section Number	Description
I	DHM model theoretical development
II	verification of the DHM model
III	program description for DHM
IV	applications of DHM
V	comparison between DHM model and kinematic routing technique
VI	appendix - program listings and an example input file

In this paper, the pertinent literature is cited as needed in the text. However, for a general overview, the reader is referred to the Two-Dimensional Flow Modeling Conference Proceedings of the U.S. Army Corps of Engineers (1981).

Because the DHM computer code is surprisingly small, and can be easily handled by most current FORTRAN home computers, FORTRAN listings (and documentation) are included for the reader's convenience.

Accepted February 1986. Discussion closes November 1986.

0309-1708/86/030118-5\$2.00

© 1986 Computational Mechanics Publications

118 *Adv. Water Resources*, 1986, Volume 9, September

In typical applications involving large scale problems, pre- and post-processors should be developed to ease the data entry demands, and graphically display the tremendous amount of modelling results generated by the computer models.

Ample applications are included in this paper which hopefully demonstrate the utility of this modelling approach in many civil engineering drainage problems. Problems considered in this paper include: (1) large scale flood plain dam-break analysis; (2) small scale dam-break analysis within a municipality; (3) temporary flood-control debris-basin failure onto a broad plain; (4) dam-break flood flows around a landfill site; (5) rainfall-runoff modelling; (6) development of synthetic S-graphs for unit hydrograph studies; and (7) flooding of a watershed due to open channel deficiencies. Finally, kinematic routing technique is applied to one-dimensional problems. This study indicates that the DHM model is more stable than the kinematic routing technique.

## I. MODEL DEVELOPMENT

### INTRODUCTION

Many flow phenomena of great importance to the engineer are unsteady in character, and cannot be reduced to steady flow by changing the view-point of the observer. A complete theory of unsteady flow is therefore required, and will be reviewed in this section. The equations of motion are not soluble in the most general case, but approximations and numerical methods can be developed which yield solutions of satisfactory accuracy.

### I.1 EQUATION OF CONTINUITY

The law of continuity for unsteady flow may be established by considering the conservation of mass in an infinitesimal space between two channel sections (Fig. 1). In unsteady flow, the discharge changes with distance at a rate  $\partial Q/\partial x$ , and the depth changes with time at a rate  $\partial y/\partial t$ . The change in discharge through space in the time  $dt$  is  $(\partial Q/\partial x) dx dt$ . The corresponding change in channel

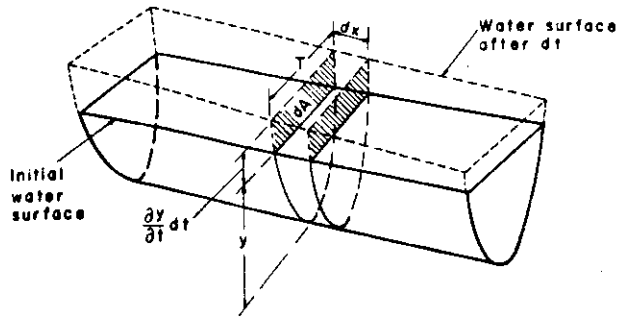


Fig. 1. Continuity of unsteady flow

storage in space is  $T dx (\partial y/\partial t) dt = dx (\partial A/\partial t) dt$ . Since water is incompressible, the net change in discharge plus the change in storage should be zero; that is

$$\left(\frac{\partial Q}{\partial x}\right) dx dt + T dx \left(\frac{\partial y}{\partial t}\right) dt = \left(\frac{\partial Q}{\partial x}\right) dx dt + dx \left(\frac{\partial A}{\partial t}\right) dt = 0$$

Simplifying,

$$\frac{\partial Q}{\partial x} + T \frac{\partial y}{\partial t} = 0 \tag{1}$$

or

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \tag{2}$$

At a given section,  $Q = VA$ ; thus equation (1) becomes

$$\frac{\partial(VA)}{\partial x} + T \frac{\partial y}{\partial t} = 0 \tag{3}$$

or

$$A \frac{\partial V}{\partial x} + V \frac{\partial A}{\partial x} + T \frac{\partial y}{\partial t} = 0 \tag{4}$$

Since the hydraulic depth  $D = A/T$  and  $\partial A = T \partial y$ , the above equation may be written

$$D \frac{\partial V}{\partial x} + V \frac{\partial y}{\partial x} + \frac{\partial y}{\partial t} = 0 \tag{5}$$

The above equations are all forms of the continuity equation for unsteady flow in open channels. For a rectangular channel of infinite width, equation (1) may be written

$$\frac{\partial q}{\partial x} + \frac{\partial y}{\partial t} = 0 \tag{6}$$

where  $q$  is the discharge per unit width.

### 1.2 EQUATION OF MOTION

In a steady, uniform flow problem, the gradient,  $dH/dx$ , of the total energy line is equal in magnitude and opposite in sign to the 'friction slope'  $S_f = v^2/(C^2R)$ . Indeed this statement was in a sense taken as the definition of  $S_f$ ; however in the present context we have to consider the

more general case in which the flow is nonuniform and the velocity may therefore be changing in the downstream direction. The net force, shear force and pressure force, is no longer zero, since the flow is accelerating. Therefore, the equation of motion becomes

$$-\gamma A \Delta h - \tau_0 P \Delta x = \rho A \Delta x \left( v \frac{\partial v}{\partial x} + \frac{\partial v}{\partial t} \right)$$

i.e.

$$\begin{aligned} \tau_0 &= -\gamma R \left( \frac{\partial h}{\partial x} + \frac{v}{g} \frac{\partial v}{\partial x} + \frac{1}{g} \frac{\partial v}{\partial t} \right) \\ &= -\gamma R \left( \frac{\partial H}{\partial x} + \frac{1}{g} \frac{\partial v}{\partial t} \right) \end{aligned} \tag{7}$$

where  $\tau_0$  is the shear stress,  $\gamma$  is the specific weight of fluid,  $R$  is the mean hydraulic radius, and  $\rho$  is the fluid density. Substituting  $\tau_0 = \gamma v^2/C^2$  into equation (7), we obtain

$$\frac{\partial H}{\partial x} + \frac{1}{g} \frac{\partial v}{\partial t} + \frac{v^2}{C^2 R} = 0 \tag{8}$$

and this equation may be written as

$$S_e + S_a + S_f = 0 \tag{9}$$

where the three terms of equation (9) are called the energy slope, the acceleration slope, and the friction slope respectively. Fig. 2 depicts the simplified representation of energy in unsteady flow.

By substituting  $H = v^2/2g + y + z$  and the bed slope  $S_0 (-\partial z/\partial x)$  into equation (8), we obtain

$$\begin{aligned} \frac{\partial H}{\partial x} &= \frac{\partial z}{\partial x} + \frac{\partial y}{\partial x} + \frac{v}{g} \frac{\partial v}{\partial x} \\ &= -S_0 + \frac{\partial y}{\partial x} + \frac{v}{g} \frac{\partial v}{\partial x} \\ &= -\frac{1}{g} \frac{\partial v}{\partial t} - S_f \end{aligned} \tag{10}$$

Hence equation (8) can be written as

$$S_f = S_0 - \frac{\partial y}{\partial x} - \frac{v}{g} \frac{\partial v}{\partial x} - \frac{1}{g} \frac{\partial v}{\partial t} = \frac{v^2}{C^2 R}$$

steady uniform flow → |  
 steady nonuniform flow → |  
 unsteady nonuniform flow → |

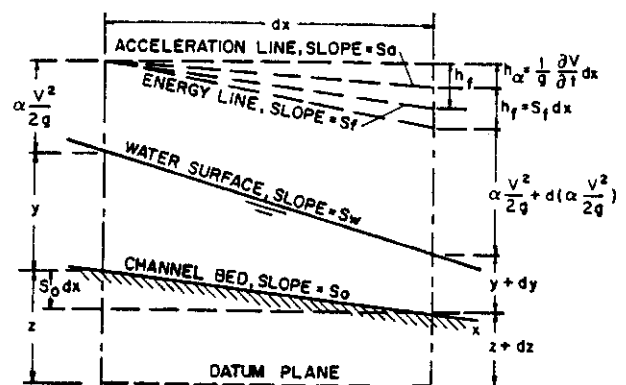


Fig. 2. Simplified representation of energy in unsteady flow

This equation may be applicable as indicated. This arrangement shows how the nonuniformity and unsteadiness of flows introduce extra terms into the governing dynamic equation. For example, it is noted that the steady-flow equation is valid only when the pressure distribution is hydrostatic; that is, when the vertical components of acceleration are negligible.

### 1.3 DIFFUSION HYDRODYNAMIC MODEL (DHM)

#### 1.3.0 One-dimensional diffusion hydrodynamic model

The mathematical relationships in a one-dimensional diffusion hydrodynamic (DHM) model are based upon the flow equations of continuity (2) and momentum (11) which can be rewritten (Akan and Yen, 1981) as

$$\frac{\partial Q_x}{\partial x} + \frac{\partial A_x}{\partial t} = 0 \tag{12}$$

$$\frac{\partial Q_x}{\partial t} + \frac{\partial(Q_x^2/A_x)}{\partial x} + gA_x \left( \frac{\partial H}{\partial x} + S_{fx} \right) = 0 \tag{13}$$

where  $Q_x$  is the flowrate;  $x, t$  are spatial and temporal coordinates;  $A_x$  is the flow area;  $g$  is gravity;  $H$  is the water surface elevation; and  $S_{fx}$  is a friction slope. It is assumed that  $S_{fx}$  is approximated from Manning's equation for steady flow by (e.g. Akan and Yen, 1981)

$$Q_x = \frac{1.486}{n} A_x R^{2/3} S_{fx}^{1/2} \tag{14}$$

where  $R$  is the hydraulic radius; and  $n$  is a friction factor which may be increased to account for other energy losses such as expansions and bend losses. Letting  $m_x$  be a momentum quantity defined by

$$m_x = \left( \frac{\partial Q_x}{\partial t} + \frac{\partial(Q_x^2/A_x)}{\partial x} \right) / gA_x \tag{15}$$

then equation (13) can be rewritten as

$$S_{fx} = - \left( \frac{\partial H}{\partial x} + m_x \right) \tag{16}$$

In equation (15), the subscript  $x$  included in  $m_x$  indicates the directional term. The expansion of equation (13) to the two-dimensional case leads directly to the terms ( $m_x, m_y$ ) except that now a cross-product of flow velocities are included, increasing the computational effort considerably.

Rewriting equation (14) and including equations (15) and (16), the directional flow rate is computed by

$$Q_x = -K_x \left( \frac{\partial H}{\partial x} + m_x \right) \tag{17}$$

where  $Q_x$  indicates a directional term, and  $K_x$  is a type of conduction parameter defined by

$$K_x = \frac{1.486}{n} A_x R^{2/3} \left| \frac{\partial H}{\partial x} + m_x \right|^{1/2} \tag{18}$$

In equation (18),  $K_x$  is limited in value by the denominator term being checked for a smallest allowable magnitude.

Substituting the flow rate formulation of equation (17) into equation (12) gives a diffusion type of relationship

$$\frac{\partial}{\partial x} K_x \left( \frac{\partial H}{\partial x} + m_x \right) = \frac{\partial A_x}{\partial t} \tag{19}$$

The one-dimensional diffusion model of Akan and Yen (1981) assumes  $m_x = 0$  in equation (18). Thus, the one-dimensional DHM is given by

$$\frac{\partial}{\partial x} K_x \frac{\partial H}{\partial x} = \frac{\partial A_x}{\partial t} \tag{20}$$

where  $K_x$  is now simplified as

$$K_x = \frac{1.486}{n} A_x R^{2/3} \left| \frac{\partial H}{\partial x} \right|^{1/2} \tag{21}$$

For a constant channel width,  $W$ , equation (20) reduces to

$$\frac{\partial}{\partial x} K_x \frac{\partial H}{\partial x} = W \frac{\partial H}{\partial t} \tag{22}$$

However, it is noted that a family of models is given by equation (19) where  $m_x$  is defined by selecting from the possibilities

$$m_x = \begin{cases} \frac{\partial(Q_x^2/A_x)}{\partial x} / gA_x, & \text{(convective acceleration model)} \\ \frac{\partial Q_x}{\partial t} / gA_x, & \text{(local acceleration model)} \\ \left( \frac{\partial Q_x}{\partial t} + \frac{\partial(Q_x^2/A_x)}{\partial x} \right) / gA_x, & \text{(coupled model)} \\ 0, & \text{(DHM)} \end{cases} \tag{23}$$

#### 1.3.1 Two-dimensional diffusion hydrodynamic model

The set of (fully dynamic) 2-D unsteady flow equations consists of one equation of continuity

$$\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} + \frac{\partial H}{\partial t} = 0 \tag{24}$$

and two equations of motion

$$\frac{\partial q_x}{\partial t} + \frac{\partial}{\partial x} \left( \frac{q_x^2}{h} \right) + \frac{\partial}{\partial y} \left( \frac{q_x q_y}{h} \right) + gh \left( S_{fx} + \frac{\partial H}{\partial x} \right) = 0 \tag{25}$$

$$\frac{\partial q_y}{\partial t} + \frac{\partial}{\partial y} \left( \frac{q_y^2}{h} \right) + \frac{\partial}{\partial x} \left( \frac{q_x q_y}{h} \right) + gh \left( S_{fy} + \frac{\partial H}{\partial y} \right) = 0 \tag{26}$$

in which  $q_x, q_y$  are flow rates per unit width in the  $x, y$ -directions;  $S_{fx}, S_{fy}$  represent friction slopes in  $x, y$ -directions;  $H, h, g$  stand for, respectively, water-surface elevation, flow depth, and gravitational acceleration; and  $x, y, t$  are for spatial and temporal coordinates.

The above equation set is based on the assumptions of constant fluid density with zero sources or sinks in the flow field, of hydrostatic pressure distributions, and of relatively uniform bottom slopes.

The local and convective acceleration terms can be grouped together such that equations (24), (25), and (26) are rewritten as

$$m_z + \left( S_{fz} + \frac{\partial H}{\partial z} \right) = 0, \quad z = x, y \quad (27)$$

where  $m_z$  represents the sum of the first three terms in equations (25) and (26) divided by  $gh$ . Assuming the friction slope to be approximated by steady flow conditions, the Manning's formula in the U.S. customary units can be used to estimate

$$q_z = \frac{1.486}{n} h^{5/3} S_{fz}^{1/2}, \quad z = x, y \quad (28)$$

Equation (28) can be rewritten as

$$q_z = -K_z \frac{\partial H}{\partial z} - K_z m_z, \quad z = x, y \quad (29)$$

where

$$K_z = \frac{1.486}{n} h^{5/3} \left| \frac{\partial H}{\partial S} + m_S \right|^{1/2}, \quad z = x, y \quad (30)$$

The symbol  $S$  indicates the flow direction which makes an angle of  $\theta = \tan^{-1}(q_y/q_x)$  in the positive  $x$ -direction.

Values of  $m$  are assumed negligible by several investigators (Akan and Yen, 1981, Hromadka, 1984, and Xanthopoulos and Koutitas, 1976), resulting in the simple diffusion model,

$$q_z = -K_z \frac{\partial H}{\partial z}, \quad z = x, y \quad (31)$$

The proposed 2-D DHM is formulated by substituting equation (31) into equation (24)

$$\frac{\partial}{\partial x} K_x \frac{\partial H}{\partial x} + \frac{\partial}{\partial y} K_y \frac{\partial H}{\partial y} = \frac{\partial H}{\partial t} \quad (32)$$

If the momentum term groupings were retained, equation (32) would be written as

$$\frac{\partial}{\partial x} K_x \frac{\partial H}{\partial x} + \frac{\partial}{\partial y} K_y \frac{\partial H}{\partial y} + S = \frac{\partial H}{\partial t} \quad (33)$$

where

$$S = \frac{\partial}{\partial x} (K_x m_x) + \frac{\partial}{\partial y} (K_y m_y)$$

and  $K_x, K_y$  are also functions of  $m_x, m_y$  respectively.

#### 1.4 NUMERICAL APPROXIMATION

##### 1.4.0 Numerical solution algorithm

The following steps are taken in the computer program where the flow path is assumed initially discretized by equally spaced nodal points with a Manning's  $n$ , an elevation, and an initial flow depth (usually zero) defined:

- (1) between nodal points, compute an average Manning's  $n$ , and average geometric factors,

- (2) assuming  $m_x = 0$ , estimate the nodal flow depths for the next timestep,  $(t + \Delta t)$  by using equations (20) and (21) explicitly,
- (3) using the flow depths at time  $t$  and  $(t + \Delta t)$ , estimate the midtimestep value of  $m_x$  selected from equation (23),
- (4) recalculate the conductivities  $K_x$  using the appropriate  $m_x$  values,
- (5) determine the new nodal flow depths at time  $(t + \Delta t)$  using equation (19), and
- (6) return Step (3) until  $K_x$  matches midtimestep estimates.

The above algorithm steps can be used regardless of the choice of definition for  $m_x$  from equation (23). Additionally, the above program steps can be directly applied to a two-dimensional diffusion model with the selected  $(m_x, m_y)$  relations incorporated.

##### 1.4.1 Numerical model formulation (grid elements)

For uniform grid elements, the integrated finite difference version of the nodal domain integration (NDI) method is used. For grid elements, the NDI nodal equation is based on the usual system shown in Fig. 3. Flow rates along the boundary  $\Gamma$  are estimated using a linear trial function assumption between nodal points.

For a square grid of width  $\delta$ ,

$$q|_{\Gamma_E} = -(K_x|_{\Gamma_E})(H_E - H_C)/\delta \quad (34)$$

where

$$K_x|_{\Gamma_E} = \begin{cases} \left( \frac{1.486}{n} h^{5/3} \right)_{\Gamma_E} \left| \frac{H_E - H_C}{\delta \cos \theta} \right|^{1/2}; & \bar{h} > 0 \\ 0; & \bar{h} \leq 0 \text{ or } |H_E - H_C| < 10^{-3} \end{cases} \quad (35)$$

In equation (35),  $\bar{h}$  and  $n$  are both the average of the values of  $C$  and  $E$ , i.e.,  $\bar{h} = (h_C + h_E)/2$  and  $n = (n_C + n_E)/2$ . (Additionally, the denominator of  $K_x$  is checked such that  $K_x$  is set to zero if  $|H_E - H_C|$  is less than a tolerance such as  $10^{-3}$  ft.)

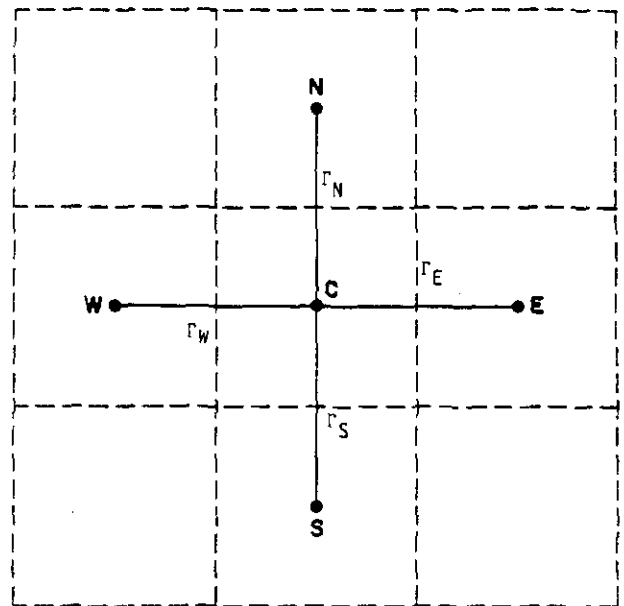


Fig. 3. Two-dimensional finite difference analog

The model advances in time by an explicit approach

$$\mathbf{H}^{i+1} = \mathbf{K}^i \mathbf{H}^i \quad (36)$$

where the assumed input flood flows are added to the specified input nodes at each timestep. After each timestep, the hydraulic conduction parameters of equation (35) are reevaluated, and the solution of equation (36) reinitiated. Using grid sizes with uniform lengths of 1000-feet, timesteps of size 5.0 sec were found satisfactory. Verification of the 2-D DHM is given in section II for the class of problems involving severe peaked flood hydrographs such as those resulting from dam-breaks.

#### 1.4.2 Model timestep selection

The sensitivity of the model to timestep selection is dependent upon the slope of the hydrograph ( $\partial Q/\partial t$ ) and the grid spacing. Increasing the grid spacing size introducing additional storage to a corresponding increase in nodal point flood depth values. Similarly, a decrease in timestep size allows a refined calculation of inflow and outflow values and a smoother variation in nodal point flood depths with respect to time. The computer algorithm may self-select a timestep by increments of halving (or doubling) the selected timestep size so that a proper balance of inflow-outflow to control volume storage variation is achieved. In order to avoid a matrix solution for flood depths, an explicit timestepping algorithm is used to solve for the time derivation term. For large timesteps or a rapid variation in the dam-break hydrograph (i.e.,  $\partial Q/\partial t$  is large), a large accumulation of flow volume will occur at the most upstream nodal point. That is, at the dam-break reservoir nodal point, the lag in outflow from the control volume can cause unacceptable error in the computation of the flood depth. One method which offset this error is the program to self-select the timestep until the difference in the rate of volume accumulation is within a specified tolerance. For the example problems considered, a timestep of about 5 to 10 seconds was found adequate.

Due to the form of the DHM in equation (22), the model can be extended into an implicit technique. However, this extension would require a matrix solution process which may become unmanageable for two-dimensional models which utilize hundreds of nodal points.

## II. VERIFICATION OF DIFFUSION HYDRODYNAMIC MODEL (DHM)

### INTRODUCTION

An unsteady flow hydraulic problem of considerable interest is the analysis of dam-breaks and the subsequent floodplain evolution.

The use of numerical methods to approximately solve the flow equations for the propagation of a flood wave due to an earthen dam failure has been the subject of several studies reported in the literature. Generally, the one-dimensional flow is modelled wherever there is no significant lateral variation in the flow. Land (1980 a, b) examines four such dam-break models in their prediction of flooding levels and flood wave travel time, and compares the results against observed dam failure information. In dam-break analysis studies, an assumed dam-break failure mode (which may be part of the solution) is used to develop an inflow hydrograph to the

down-stream flood plain. Consequently, it is noted that a considerable sensitivity in modelling results is attributed to the dam-break failure rate assumptions. Ponce and Tsivoglou (1981) examine the gradual failure of an earthen embankment (caused by an overtopping flooding event) and present a detailed model of the total system: sediment transport, unsteady channel hydraulics, and earth embankment failure. In this section, the main objective is to evaluate the diffusion form of the flow equations for the estimation of flood depths (and the flood plain) resulting from a specified dam-break hydrograph. Consequently the dam-break failure mode is not considered in this section. Rather, the dam-break failure mode may be included as part of the model solution (such as for a sudden breach) or specified as a reservoir outflow hydrograph.

In another study, Rajar (1978) studies a one-dimensional flood wave propagation from an earthen dam failure. His model solves the St. Venant equations by means of either a first-order diffusive or a second-order Lox-Wendroff numerical scheme. A review of the literature indicates that the most often-used numerical scheme is the method of characteristics (to solve the governing flow equations) such as described in Sakkas and Strelkoff (1973), and Chen (1980 a, b).

Although many dam-break studies involve flood flow regimes which are truly two-dimensional (in the horizontal plane), the two dimensional case has not received much attention in the literature. Katopodes and Strelkoff (1978) use the method of bicharacteristics to solve the governing equations of continuity and momentum. The model utilizes a moving grid algorithm to follow the flood wave propagation, and also employs several interpolation schemes to approximate the nonlinearity effects. In a much simpler approach, Xanthopoulos and Koutitas (1976) use a diffusion model (i.e., the inertia terms are assumed negligible in a comparison to the pressure, friction, and gravity components) to approximate a two-dimensional flow field. The model assumes that the flood plain flow regime is such that the inertia terms (local and convective acceleration) are negligible. In a one-dimensional model, Akan and Yen (1981) also use the diffusion approach to model hydrograph confluences at channel junctions. In the latter study, comparisons of modelling results were made between the diffusion model, a complete dynamic wave model solving the total equation system, and the basic kinematic wave equation model (i.e., the inertia and pressure terms are assumed negligible in comparison to the friction and gravity terms). The comparisons between the diffusion model and the dynamic wave model were very favourable, showing only minor discrepancies.

The kinematic-wave flow model has been recently used in the computation of dam-break flood waves (Hunt, 1982). Hunt concludes in his study that the kinematic-wave solution is asymptotically valid. Since the diffusion model has a wider range of applicability of bed slopes and wave periods than the kinematic model (Ponce *et al.*, 1978), then the diffusion model approach should provide an extension to the referenced kinematic model.

Because the diffusion modelling approach leads to an economic two-dimensional dam-break model (with numerical solutions based on the usual integrated finite-difference or finite element techniques), the need to include the extra components in the momentum equation must be ascertained. For example, evaluating the convective

acceleration terms in a two-dimensional flow model requires approximately an additional 50-percent of the computational effort required in solving the entire two-dimensional model with the inertia terms omitted. Consequently, including the local and convective acceleration terms increases the computer execution costs significantly. Such increases in computational effort may not be significant for one-dimensional case studies; however, two-dimensional case studies necessarily involve considerably more computational effort and any justifiable simplifications of the governing flow equations is reflected by a significant decrease in computer software requirements, costs and computer execution time.

Ponce (1981) examines the mathematical expressions of the flow equations which lead to wave attenuation in prismatic channels. It is concluded that the wave attenuation process is caused by the interaction of the local acceleration term with the sum of the terms of friction slope and channel slope. When local acceleration is considered negligible, wave attenuation is caused by the interaction of the friction slope and channel slope terms with the pressure gradient or convective acceleration terms (or a combination of both terms). Other discussions of flow conditions and the sensitivity to the various terms of the flow equations are given in Miller and Cunge (1975), Morris and Woolhiser (1980), and Henderson (1963).

It is stressed that the ultimate objective of this paper is to develop a two-dimensional diffusion model for use in estimating floodplain evolution such as occurs due to drainage system deficiencies. Prior to finalizing such a model, the requirement of including the inertia terms in the unsteady flow equations needs to be ascertained. The strategy used to check on this requirement is to evaluate the accuracy in predicted flood depths produced from a one-dimensional diffusion model with respect to the one-dimensional U.S.G.S. K-634 dam-break model which includes all of the inertia term components.

## II.1 ONE-DIMENSIONAL ANALYSIS

### II.1.0 Study approach

In order to evaluate the accuracy of the diffusion model of equation (22) in the prediction of flood depths, the U.S.G.S. fully dynamic flow model K-634 (Land, 1980, a, b) is used to determine channel flood depths for comparison purposes. The K-634 model solves the coupled flow equations of continuity and momentum by an implicit finite difference approach and is considered to be a highly accurate model for many unsteady flow problems. The study approach is to compare predicted: (1) flood depths, and (2) discharge hydrographs from both the K-634 and the DHM (equation (22)) for various channel slopes and inflow hydrographs.

It should be noted that different initial conditions are used for these two models. The U.S.G.S. K-634 model requires a base flow to start the simulation, i.e., the initial depth of water can not be zero. Next, the normal depth assumption is used to generate an initial water depth before the simulation starts. These two steps are not required by the DHM.

In this case study, two hydrographs are assumed; namely, peak flows of 120 000 cfs and 600 000 cfs. Both hydrographs are assumed to increase linearly from zero to the peak flow rate at time of 1-hour, and then decrease linearly to zero time of 6-hours (see Fig. 4 inset). The study

channel is assumed to be a uniform rectangular section of Manning's  $n$  equal to 0.040, and various slopes  $S_0$  in the range of  $0.001 \leq S_0 \leq 0.01$ . Figure 4 shows the comparison of modelling results. From the figure, various flood depths are plotted along the channel length of up to 10-miles. Two reaches of channel lengths of up to 30-miles are also plotted in Fig. 4 which correspond to a slope  $S_0 = 0.0020$ . In all tests, grid spacing was set at 1000-foot intervals.

From Fig. 4 it is seen that the diffusion model provides estimates of flood depths that compare very well to the flood depths predicted from the K-634 model. Differences in predicted flood depths are less than 3-percent for the various channel slopes and peak flow rates considered.

Figures 5 and 6 show good comparisons between the DHM and the K-634 model for water depths and outflow hydrographs at 5 and 10 miles downstream from the dam-break site.

### II.1.1 Grid spacing selection

The choice of timestep and grid size for an explicit time advancement is a relative matter and is theoretically based on the well-known Courant condition (Basco, 1978). The choice of grid size usually depends on available topographic data for nodal elevation determination and the size of the problem. The effect of the grid size (for constant timestep for 7.2 seconds) on the diffusion model accuracy can be shown by example where nodal spacings of 1000, 2000 and 5000-feet are considered. The predicted flood depths varied only slightly from choosing the grid size between 1000-feet and 2000-feet. However, an increased variation in results occurs when a grid size of 500-feet is selected. Figure 7 shows the computed flood

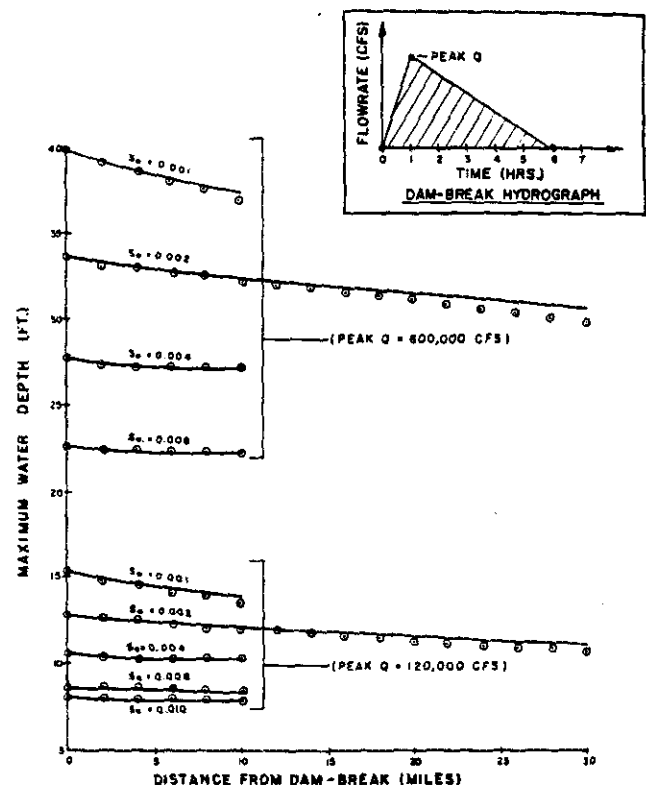


Fig. 4. Diffusion model (○) and K-634 model results (solid line) for 1000-foot width channel, Manning's  $n=0.040$ , and various channel slopes,  $S_0$

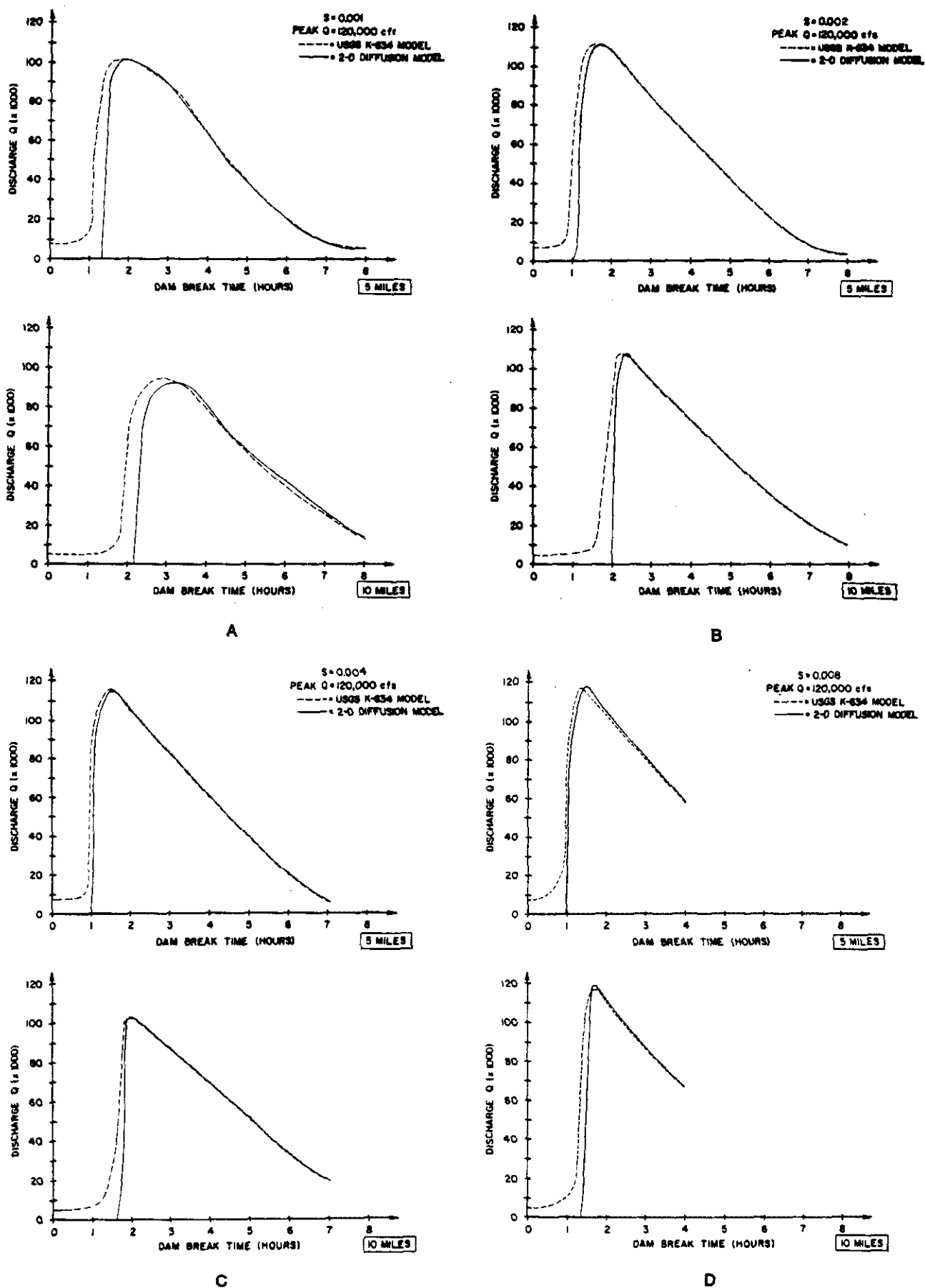
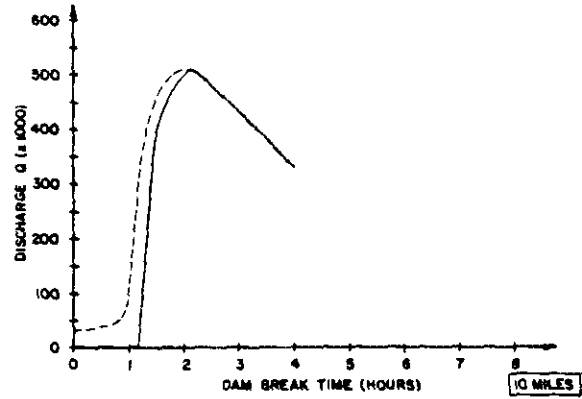
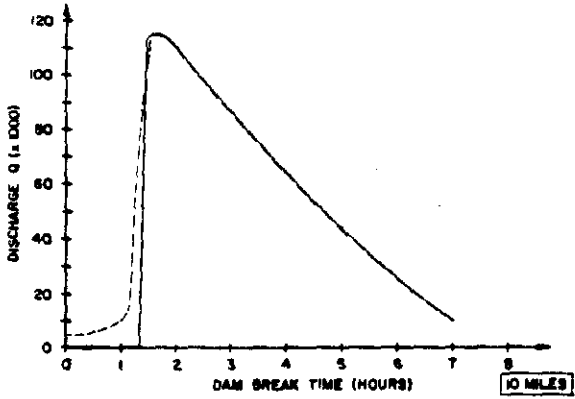
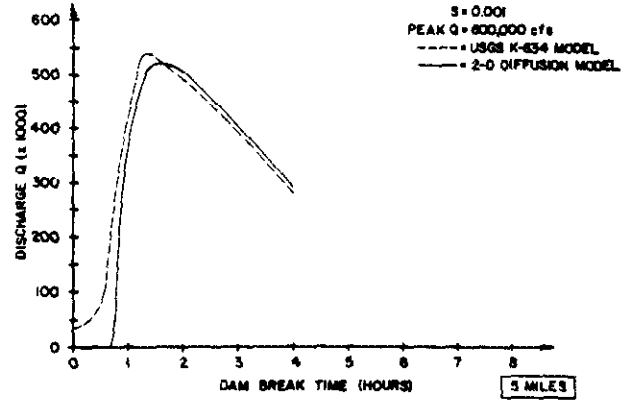
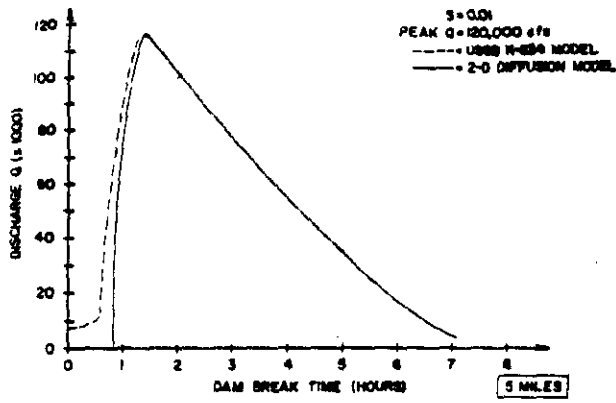
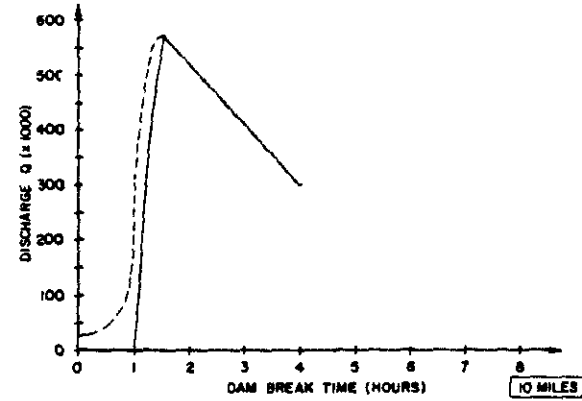
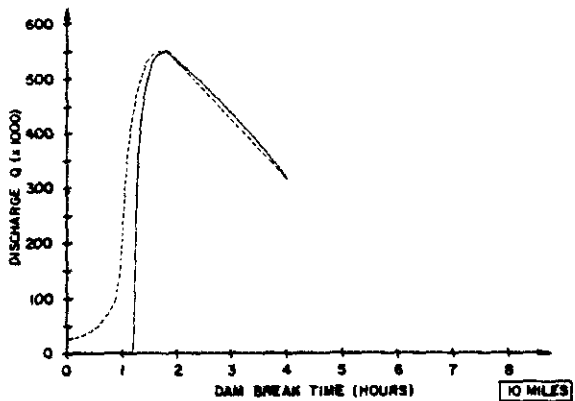
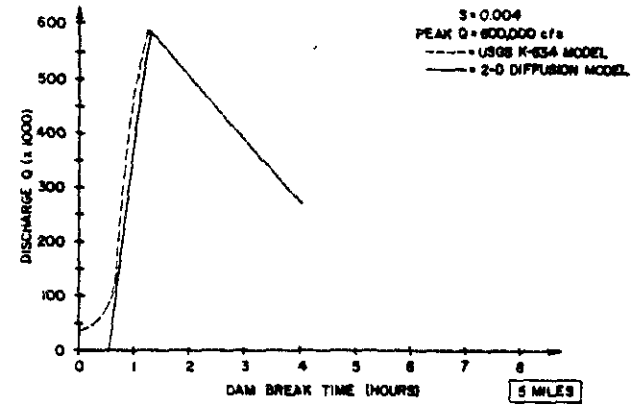
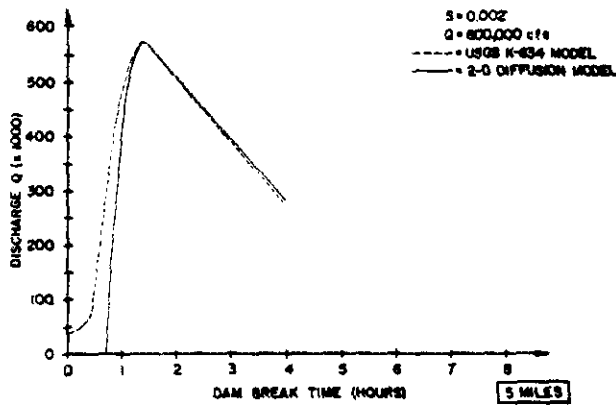


Fig. 5. Comparisons of outflow hydrographs at 5 and 10 miles downstream from the dam-break site



E

F



G

H

Fig. 5.—cont.



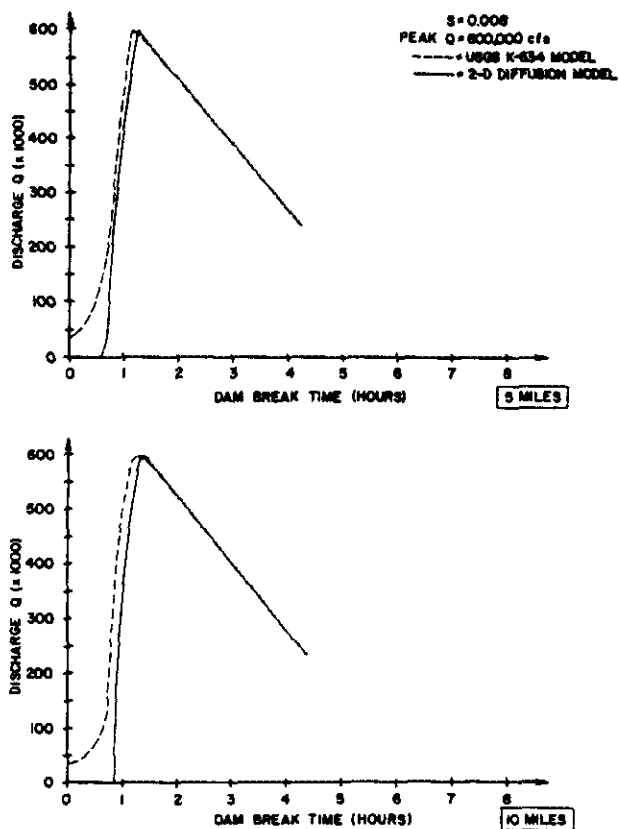


Fig. 5.—cont.

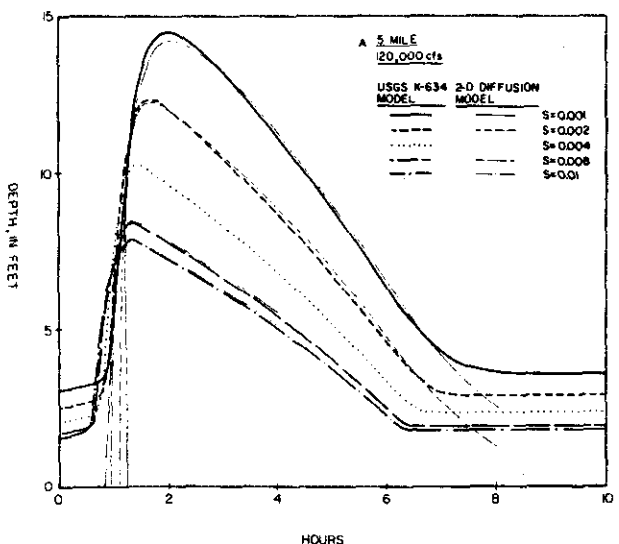


Fig. 6. Comparisons of depths of water at 5 and 10 miles downstream from the dam-break site

depths in comparison to the K-634 modelling results (Fig. 4) for the considered grid sizes, and the peak flow rate test hydrograph of 600 000 cfs.

Because the algorithm presented is based upon an explicit timestepping technique, the modelling results may become inaccurate should the timestep size *versus* grid size ratio become large. A simple procedure to eliminate

this instability is to half the timestep size until convergence in computed results is achieved. Generally, such a timestep adjustment may be directly included in the dam-break model computer program. For the cases considered in this section, timestep sizes of 7.2 seconds was found to be adequate when using the 1000-foot to 5000-foot grid sizes.

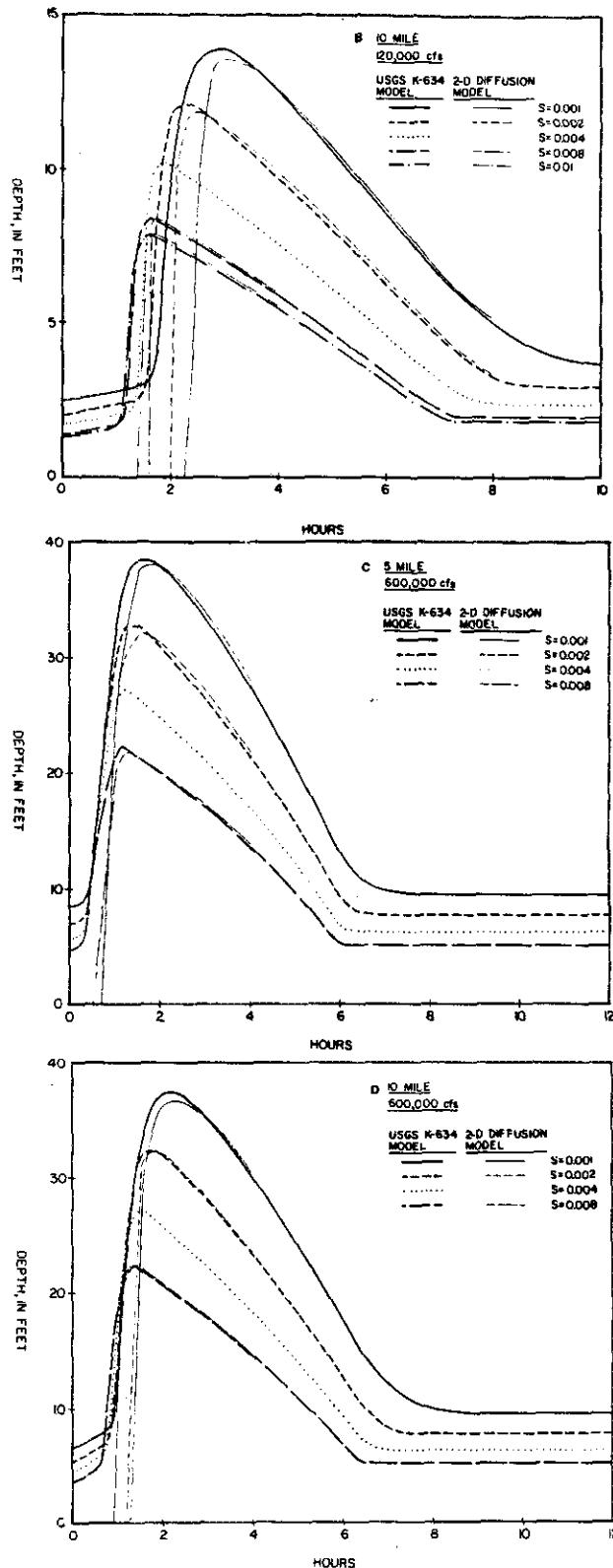


Fig. 6.—cont.

II.1.2 Conclusions and discussions

For the dam-break hydrographs considered and the range of channel slopes modelled, the simple diffusion dam-break model of equation (22) provides estimates of flood depths and outflow hydrographs which compare favourably to the results determined by the well-known

K-634 one-dimensional dam-break model. Generally speaking, the difference between the two modelling approaches is found to be less than a 3 percent variation in predicted flood depths.

The presented diffusion dam-break model is based upon a straightforward explicit timestepping method which allows the model to operate upon the nodal points without the need to use large matrix systems. Consequently, the model can be implemented on most currently available microcomputers.

The diffusion model of equation (22) can be directly extended to a two-dimensional model by adding the *y*-direction terms which are computed in a similar fashion as the *x*-direction terms. The resulting two-dimensional diffusion model is tested by modelling the considered test problems in the *x*-direction, the *y*-direction, and along a 45-degree trajectory across a two-dimensional grid aligned with the *x*-*y* coordinate axis. Using a similar two-dimensional model, Xanthopoulos and Koutitas (1976) conceptually verify the diffusion modelling technique by considering the evolution of a two-dimensional flood plain which propagates radially from the dam-break site.

From the above conclusions, use of the diffusion approach of equation (22) in a two-dimensional DHM may be justifiable due to the low variation in predicted

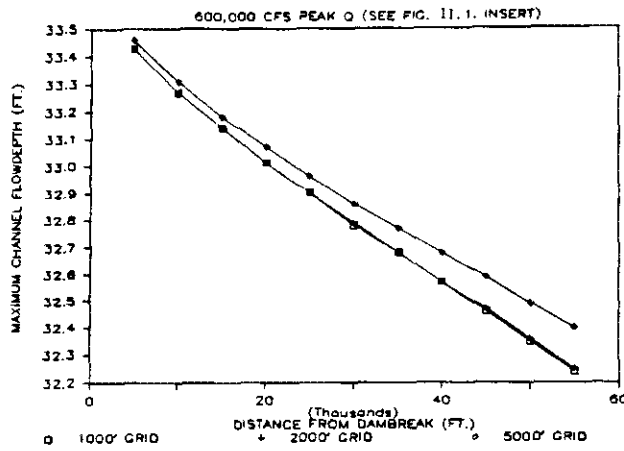


Fig. 7. Effects of differing grid size

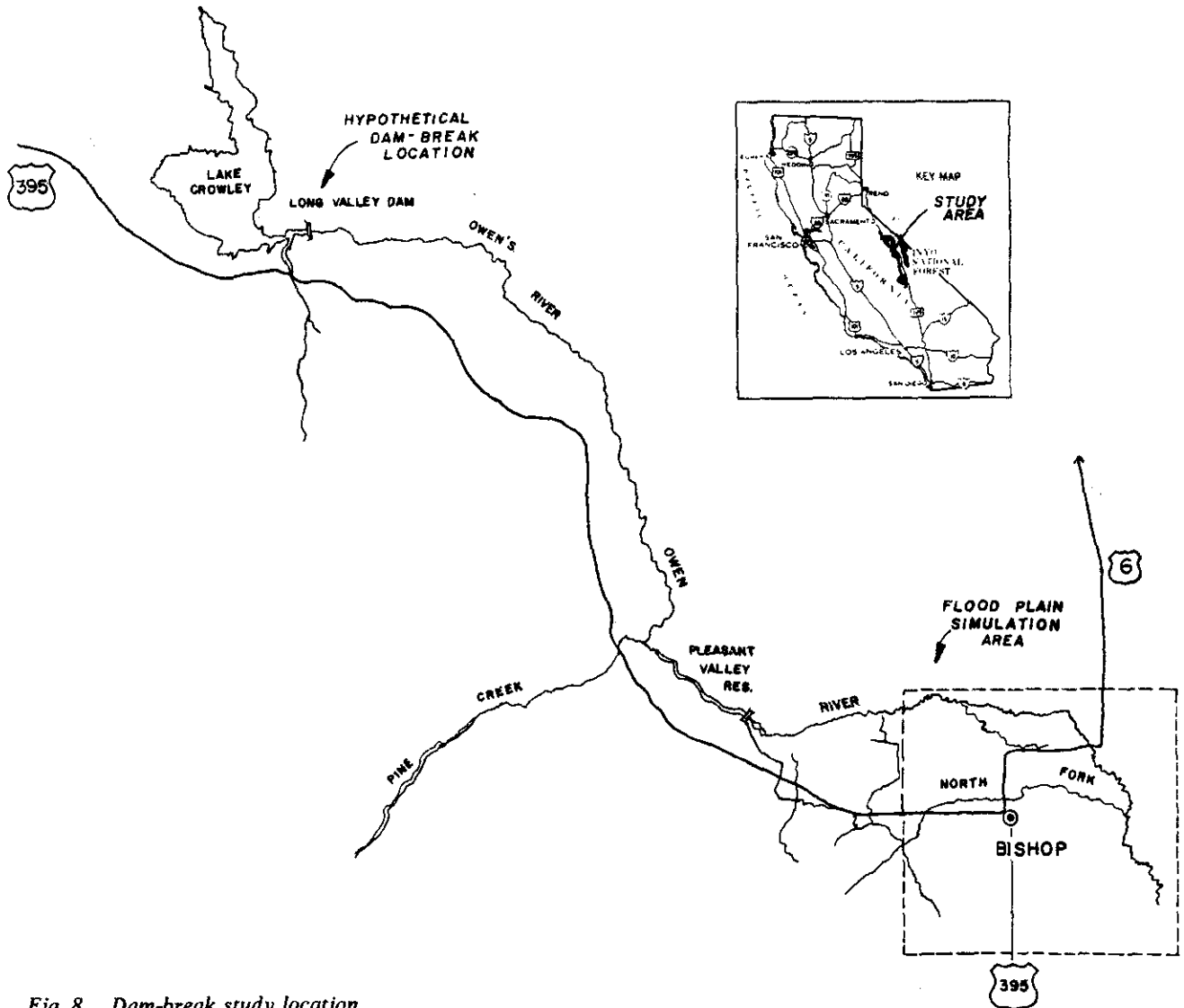


Fig. 8. Dam-break study location

flooding depths (one-dimensional) with the exclusion of the inertia terms. Generally speaking, a two-dimensional model would be employed when the expansive nature of flood flows is anticipated. Otherwise, one of the available one-dimensional models would suffice for the analysis.

## II.2 TWO-DIMENSIONAL ANALYSIS

### II.2.0 Introduction

In this section, a two-dimensional DHM is developed. The model is based on a diffusion approach where gravity, friction, and pressure forces are assumed to dominate the flow equations. Such an approach has been used earlier by Xanthopoulos and Koutitas (1975) in the prediction of dam-break flood plains in Greece. In those studies, good results were also obtained in the use of the two-dimensional model in predicting one-dimensional flow quantities. The preceding section considers a one-dimensional diffusion model and concludes that for most velocity flow regimes (i.e., less than approximately 25 feet/sec), the diffusion model is a reasonable approximation of the full dynamic wave formulation.

An integrated finite difference grid model is developed which equates each cell-centered node to a function of the four neighbouring cell nodal points. To demonstrate the model's flood plain predictive capacity, a study of a hypothetical dam-break of the Crowley Lake dam near the City of Bishop, California (Fig. 8) is considered (Hromadka et al., 1985).

### II.2.1 K-634 modelling results and discussion

Using the K-634 model for computing the two-dimensional flow was attempted by means of the one-dimensional nodal spacing shown in Fig. 9. Cross sections were obtained by field survey, and the elevation data used to construct nodal point flow-width versus stage diagrams. A constant Manning's friction factor of 0.04 was assumed for study purposes. The assumed dam-break failure reached a peak flow rate of 420 000 cfs within one hour, and returned to zero flow 9.67 hours later. The resulting K-634 flood plain limits is shown in Fig. 10. To model the flow break-out, a slight gradient was assumed for the topography perpendicular to the main channel. The motivation for such a lateral gradient is to limit the channel floodway section in order to approximately conserve the one-dimensional momentum equations. Consequently, fictitious channel sides are included in the K-634 model study which results in an artificial confinement of the flows. Hence, a narrower flood plain is delineated (such as shown in Fig. 10) where the flood flows are falsely retained within a hypothetical channel confine. An examination of the flood depths given in Fig. 12 indicates that at the widest flood plain expanse of Fig. 10, the flood depth is about 6-feet, yet the flood plain is not delineated to expand southerly, but is modelled to terminate based on the assumed gradient of the topography towards the channel. Such complications in accommodating an expanding flood plain when using a one-dimensional model are obviously avoided by using a two-dimensional approach.

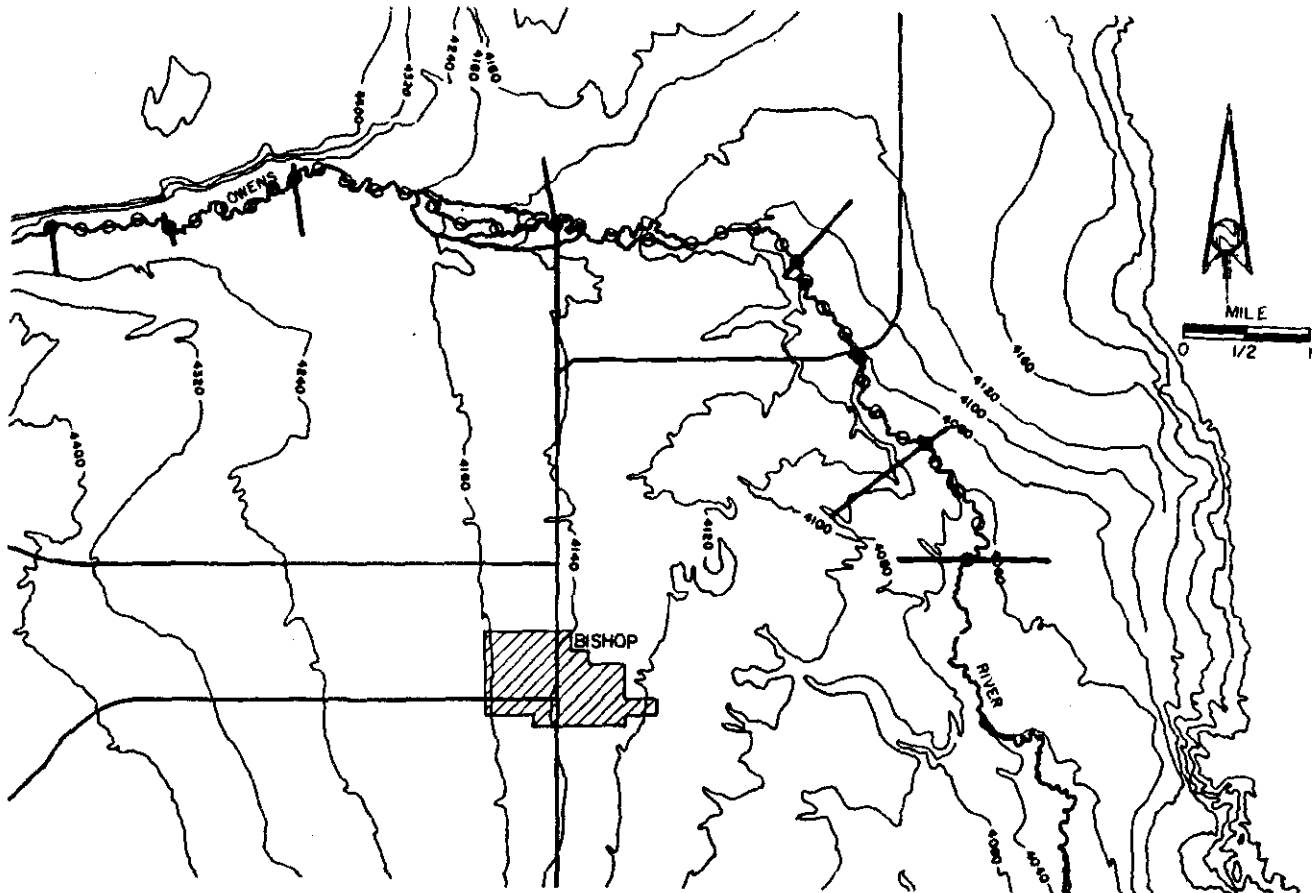


Fig. 9. Surveyed cross section locations on Owens River for use in K-634 model

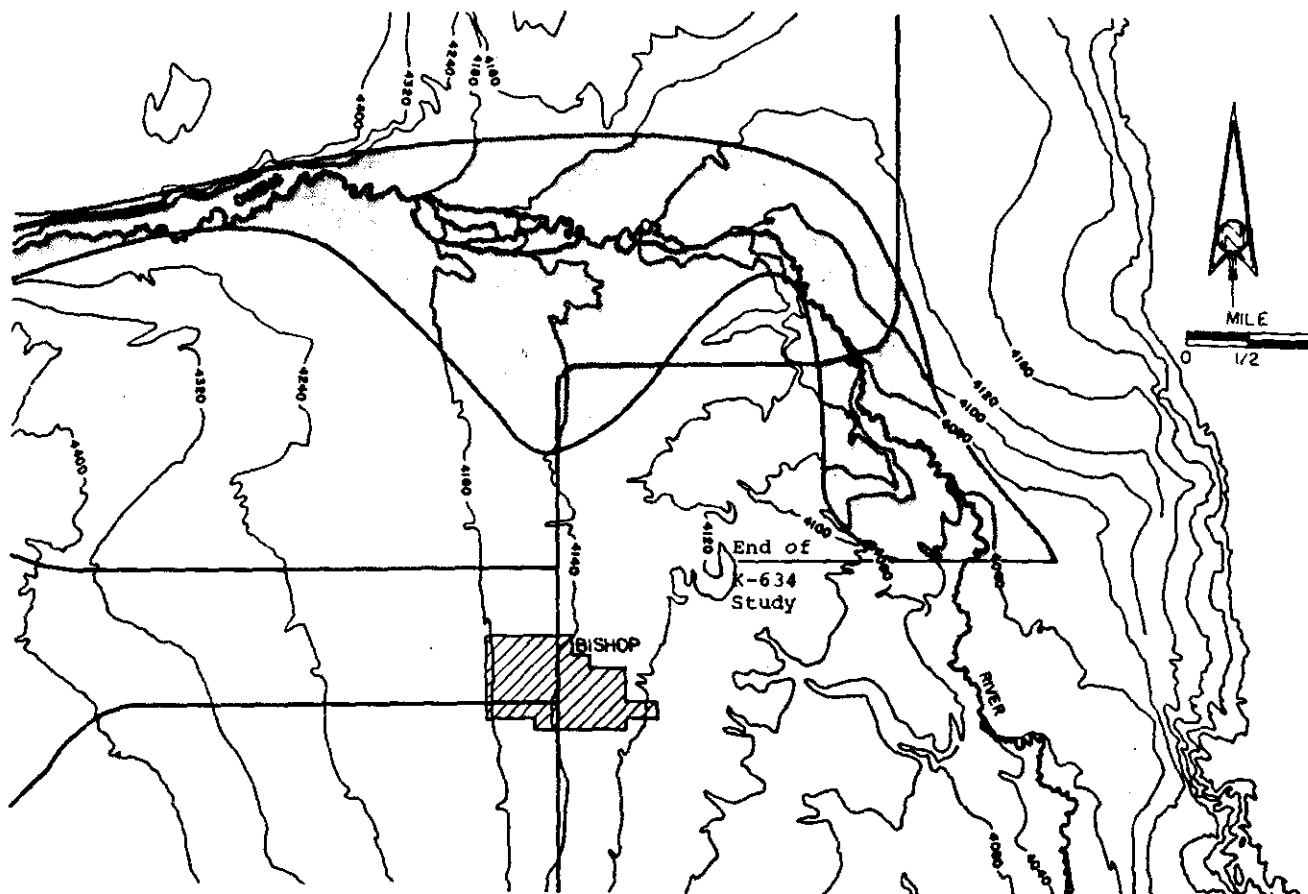


Fig. 10. Floodplain computed from K-634 model

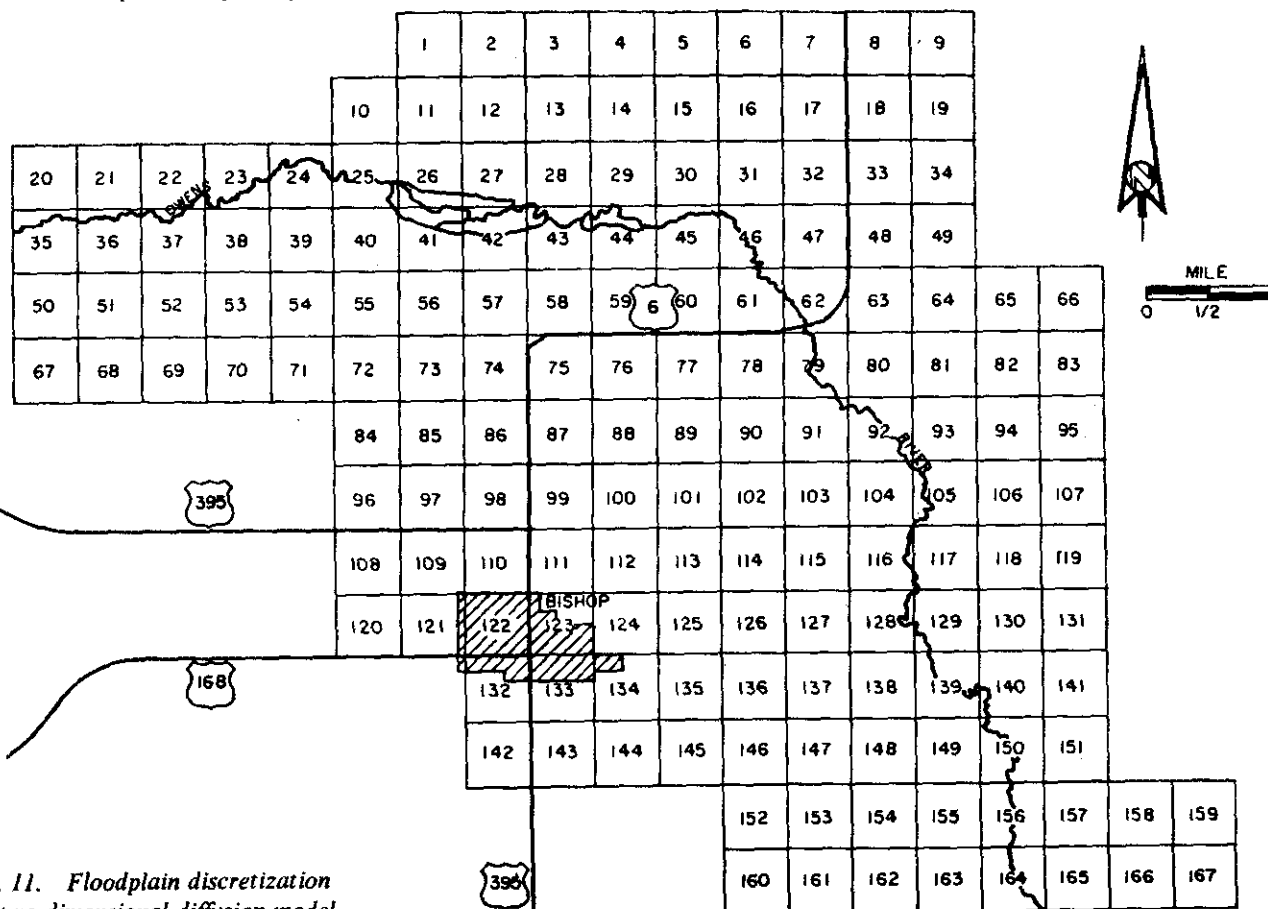


Fig. 11. Floodplain discretization for two-dimensional diffusion model

The two-dimensional DHM is now applied to the hypothetical dam-break problem using the grid discretization shown in Fig. 11. The same inflow hydrograph produced by the K-634 model is used for the DHM. Again, the Manning's friction factor of 0.04 was used. The resulting flood plain is shown in Fig. 13 for the 1/4-square-mile grid model.

Comparisons of predicted maximum water elevations are shown in Fig. 12 which plots K-634 modelling results and the two-dimensional modelling results. The two approaches are comparable except at points shown as A

and B in the figure. Point A corresponds to the predicted breakout of flows away from the Owens River channel with flows travelling southerly towards the City of Bishop. As discussed previously, the K-634 predicted flood depth corresponds to a flow depth of 6 feet (above natural ground) which is actually unconfined by the channel. The natural topography will not support such a flood depth and, consequently, there should be southerly breakout flows such as predicted by the two-dimensional model. With such breakout flows included, it is reasonable that the two-dimensional model would predict a lower flow depth at point A.

At point B, the K-634 model predicts a flood depth of approximately 2 feet less than the two-dimensional model. However at this location, the K-634 modelling results are based on cross-sections which traverse a 90-degree bend. In this case K-634 model will over-estimate the true channel storage, resulting in an underestimation of flow depths.

In comparing the various model predicted flood depths and delineated plains, it is seen that the two-dimensional DHM produced more reasonable predictions of the flood plain characteristics which are associated with broad, flat plains such as found at the study site than the one-dimensional model. The DHM model affords approximation of channel bends, channel expansions and contractions, flow breakouts, and the general area of inundation. Additionally, the DHM approach allows for the inclusion of return flows (to the main channel) which result due to upstream channel breakout flows, and other

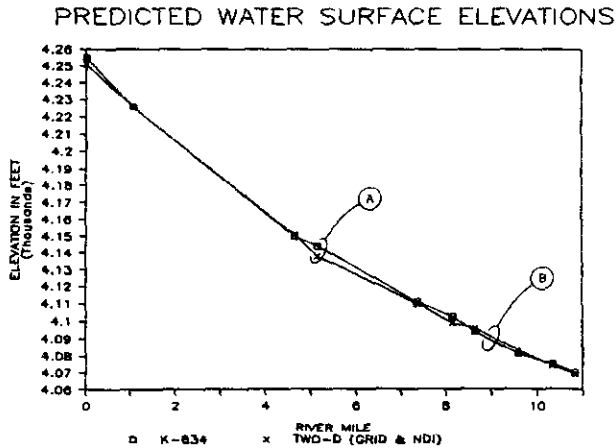


Fig. 12. Comparison of modelled water surface elevations

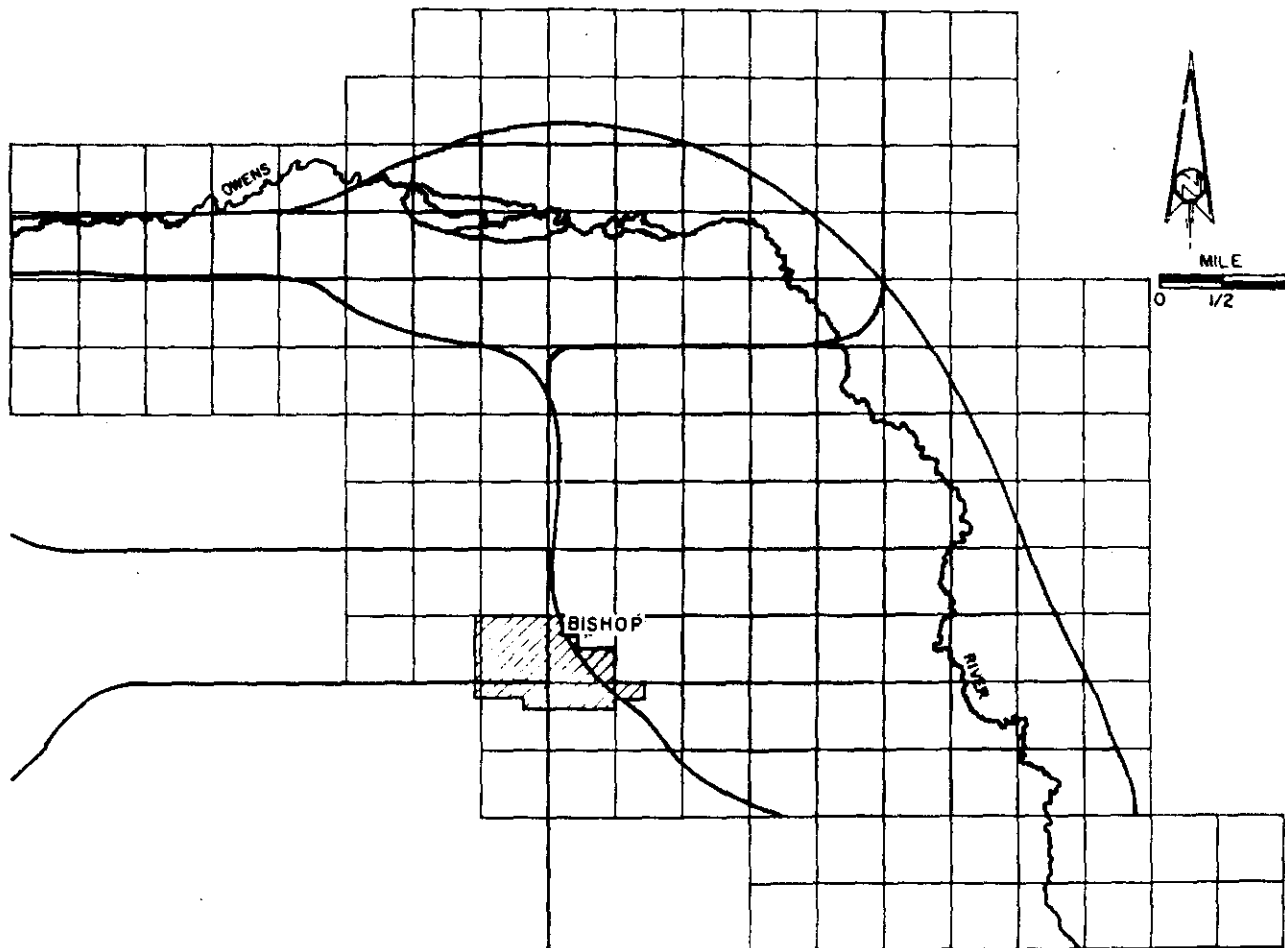


Fig. 13. Floodplain for two-dimensional diffusion model

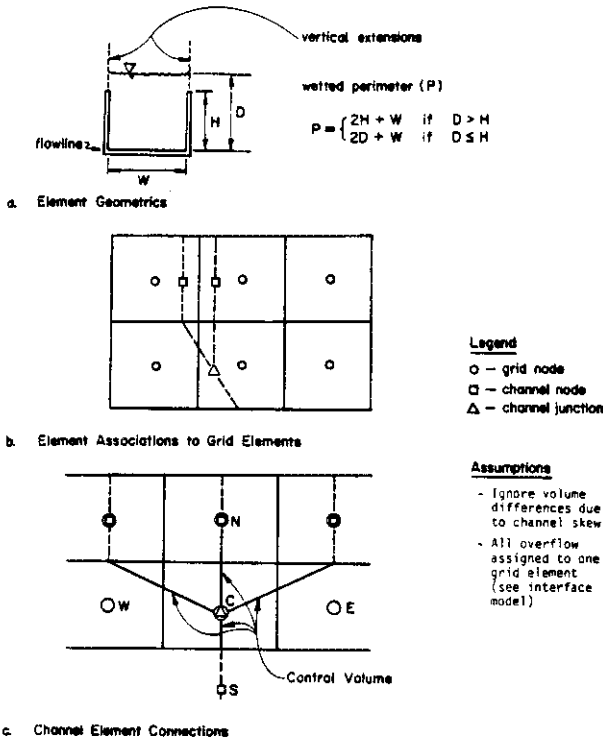


Fig. 14. DHM one-dimensional channel elements

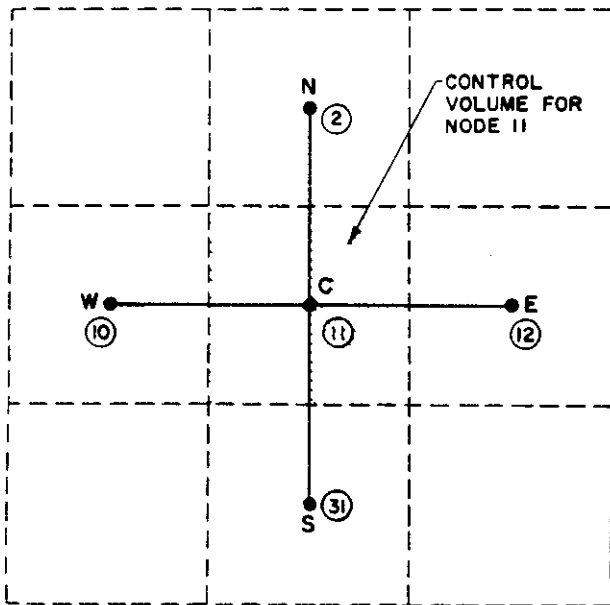


Fig. 15. Grid element nodal molecule

two-dimensional flow effects without the need for special modelling accommodations which would be required when using a one-dimensional model.

### III. PROGRAM DESCRIPTIONS OF DHM MODEL

#### INTRODUCTION

A computer program for the two-dimensional diffusion hydrodynamic model which is based on the diffusion form

of the St. Vincent equations where gravity, friction, and pressure forces are assumed to dominate the flow equation will be discussed in this section.

The DHM model consists of a 1-D channel and 2-D flood plain models, and an interface sub-model. The one-dimensional channel element utilizes the following assumptions:

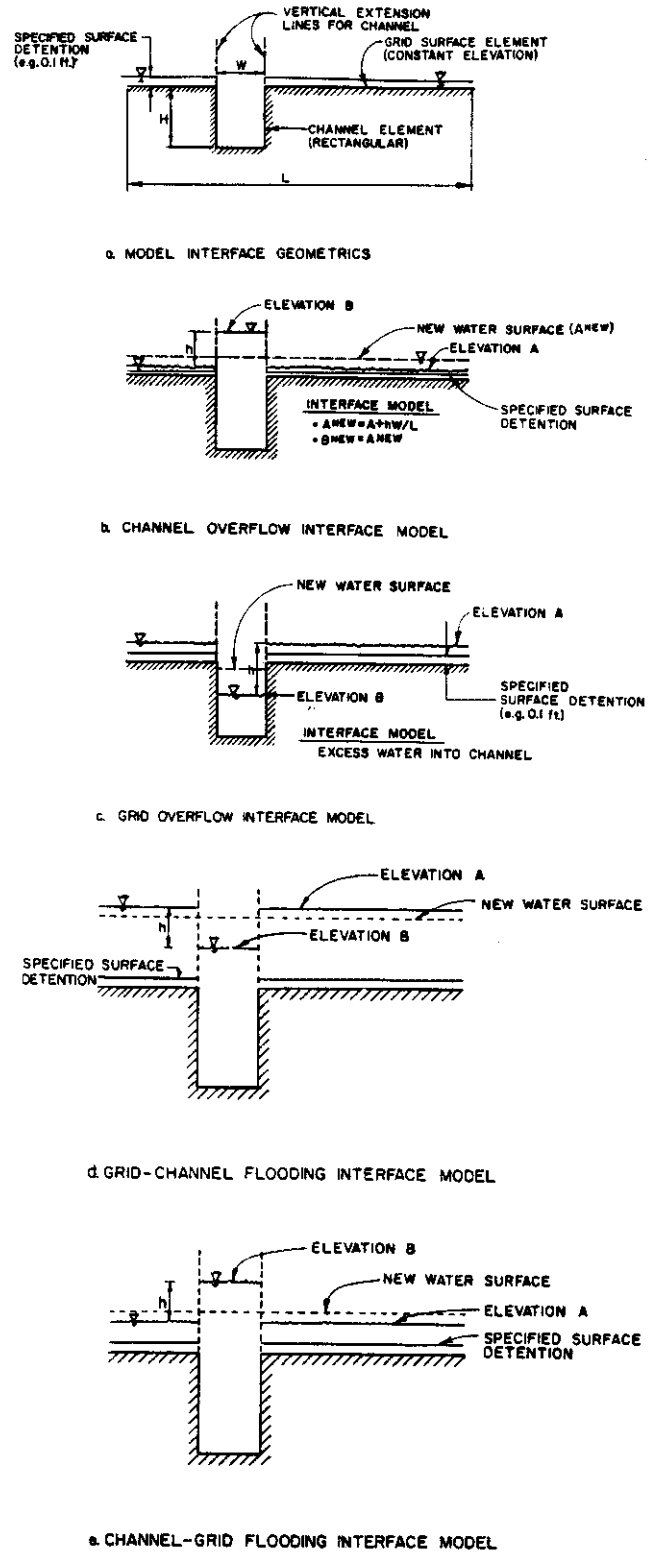


Fig. 16. DHM interface model

- (1) infinite vertical extensions on channel walls (Fig. 14a),
- (2) wetted perimeter is calculated as shown on Fig. 14b,
- (3) volumes due to channel skew is ignored (Fig. 14c), and
- (4) all overflow water is assigned to one grid element (Fig. 15).

The interface model calculates the excess amount of water either from the channel element or from the flood plain element. This excess water is redistributed to the flood plain element or the channel element according to the water surface elevation.

This FORTRAN program has the capabilities to simulate both one- and two-dimensional surface flow problems, such as the presented one-dimensional open channel flow and two-dimensional dam-break problems. Engineering applications of the program will be presented in the next section.

### III.1 INTERFACE MODEL

#### Introduction

The interface model modifies the water surface elevations of grid (flood plain) and channel elements at specified time intervals (update intervals). There are three cases of interface situations: (1) channel overflow, (2) grid overflow, and (3) flooding of channel and grid elements.

#### III.1.1 Channel overflow

When the channel is overflowing, the excess water is temporarily stored in the vertical extension area (Fig. 16b). This excess water ( $h \cdot w \cdot L$ ) is subsequently uniformly

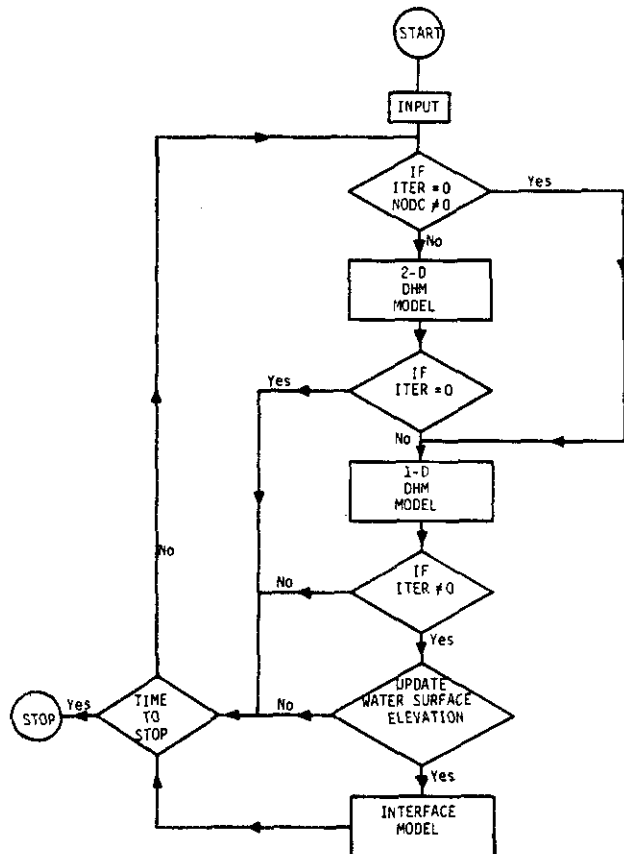


Fig. 17a. Flow chart for DHM model

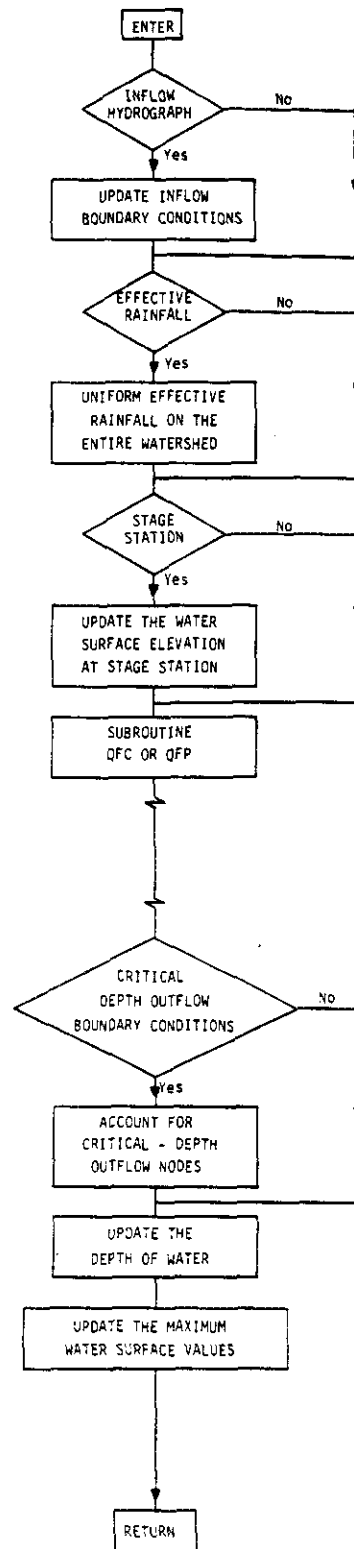


Fig. 17b. Flow chart for channel and floodplain DHM models

distributed over the grid element. In other words, the new grid water surface elevation is equal to the old water surface elevation plus a depth of  $hw/L$ , and the channel water surface elevation now matches the parent grid water surface elevation.

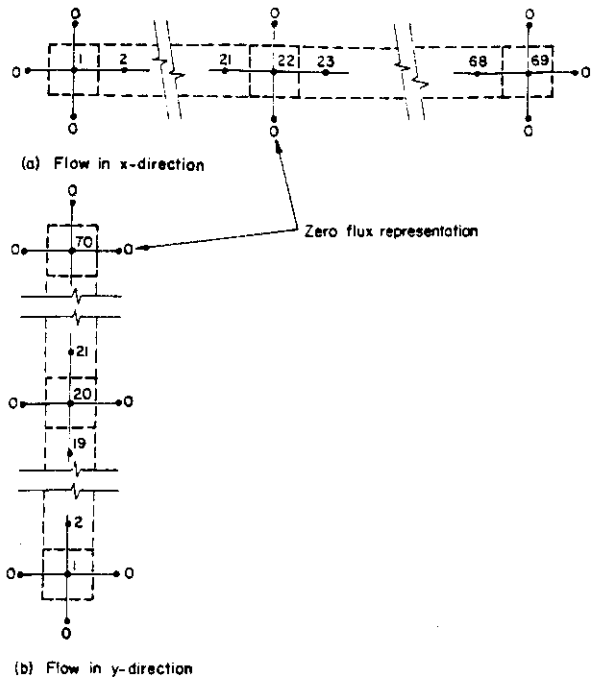


Fig. 18. One-dimensional grid element

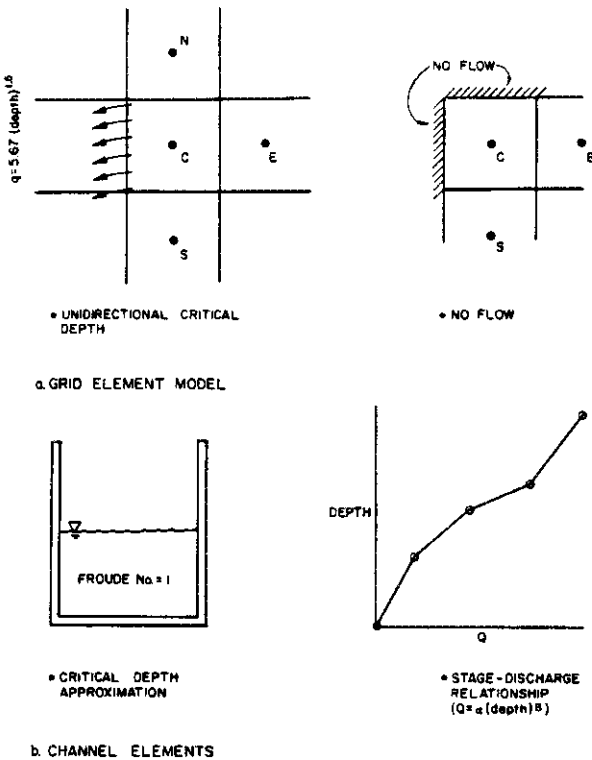


Fig. 19. DHM boundary conditions models

III.1.2 Grid overflow

When the surface water elevation of the grid element is greater than a specified surface detention (Fig. 16a), the excess water drains into the channel element and the new surface water elevation is changed according to the following two conditions (Fig. 16c), (a) If  $v > v'$ , where  $v$  denotes the excess volume of water per unit length and  $v'$  denotes the available volume per unit length, the new

water surface of the grid element is  $A^{NEW} = A^{OLD} - (v - v')/L$  and the new water surface elevation of the channel element is also equal to  $A^{NEW}$ ; (b) If  $v \leq v'$ , the new water surface elevation of the grid element is  $A^{NEW} = A^{OLD} - h$  and the new water surface elevation of the channel element is  $B^{NEW} = B^{OLD} + v/w$ .

III.1.3 Flooding of channel and grid

When flooding occurs, the water surface elevations of the grid and channel elements are both greater than the specified surface detention elevation. Two cases have to be considered as follows:

- (1) If  $A > B$  (Fig. 16d), the new water surface elevation of the grid element is  $A^{NEW} = A^{OLD} - h(L - w)/L$  and the new water surface elevation of the channel element is equal to  $A^{NEW}$ .
- (2) If  $A < B$  (Fig. 16e), the new water surface elevation of the grid element is  $A^{NEW} = A^{OLD} + h \cdot w/L$  and the new water surface elevation of the channel element is equal to  $A^{NEW}$ .

III.2 COMPUTER PROGRAM

III.2.0 Introduction

Figures 17a and 17b depict the simple flow chart for the DHM Model. The listings of the computer program and an example input file are included in the appendix.

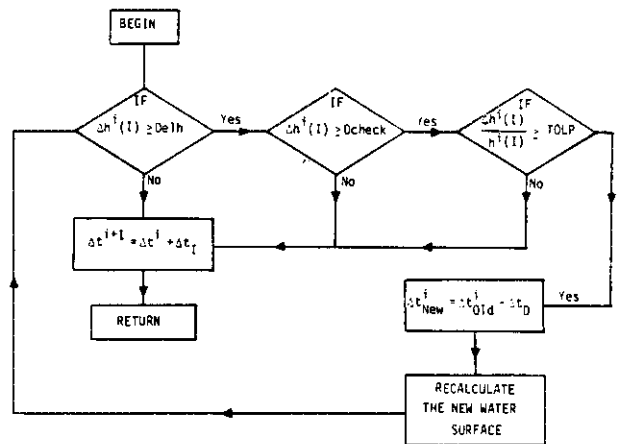


Fig. 20. Algorithm for the variable time step

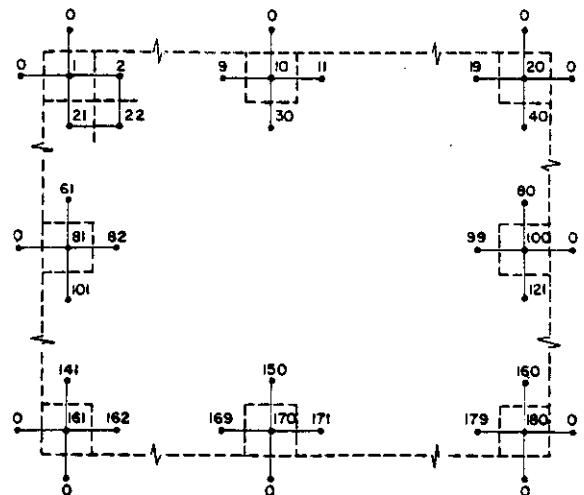


Fig. 20. No flux boundary nodes



III.2.1 Input file descriptions

The DHM model calls for the following data entries:

Line	Variables
1	DTMIN,DTMAX,DTI,DTD,SIMUL,ITER,TOUT,KODE,KMO DEL
2	NNOD,NODC,SIDE,TOL,DTOL,DTOLP,PI,TO
3	FP(1,J),J = 1,7
...	...
NNOD + 2	FP(NNOD,J)J = 1,7
NNOD + 3	NERI
NNOD + 4	(R(I,J),J = 1,2),I = 1,NERI
NNOD + 5	NFPI,NPFPI
NNOD + 6	KINP(1),(HP(1,J,1),HP(1,J,2),J = 1,NPFPI)
...	...
NNOD + 5 + NFPI	KINP(NFPI),(HP(NFPI,J,1),HP(NFPI,J,2),J = 1,NPFPI)
NNOD + NFPI + 6	NDC
NNOD + NFPI + 7	NODDC(I),I = 1,NDC
NNOD + NFPI + 8	NFLUX,NFOUT
NNOD + NFPI + 9	NODFX(I),I = 1,NFLUX
NNOD + NFPI + 10	KK,(FC(KK,J),J = 1,5)
...	...
NNOD + NFPI + NODC + 9	KK,(FC(KK,J),J = 1,5)
NNOD + NFPI + NODC + 10	NCHI,NPCHI,NCHO,NPCHO,NSTA,NPSTA
NNOD + NFPI + NODC + 11	KIN(1),(H(1,J,1),H(1,J,2)),J = 1,NPCHI)
...	...
NNOD + NFPI + NODC + NCHI + 10	KIN(NCHI),(H(NCHI,J,1),H(NCHI,J,2)),J = 1,NPCHI)
NNOD + NFPI + NODC + NCHI + 11	KOUT(1),(HOUT(1,J,1),HOUT(1,J,2), HOUT(1,J,3),J = 1,NPCHO)
...	...
NNOD + NFPI + NODC + NCHI + NCHO + 10	KOUT(NCHO),(HOUT(NCHO,J,1),HOUT(NCHO,J,2), HOUT(NCHO,J,3),J = 1,NPCHO)
NNOD + NFPI + NODC + NCHI + NCHO + 11	NOSTA(1),(STA(1,J,1),STA(1,J,2),J = 1, NPSTA)
...	...
NNOD + NFPI + NODC + NCHI + NCHO + 10 + NSTA	NOSTA(NSTA),(STA(NSTA,J,1),STA(NSTA,J,2), J = 1,NPSTA)

where

DTMIN	is the minimum allowable timestep in second,
DTMAX	is the maximum allowable timestep in second,
DTI	is the increment of timestep in second,
DTD	is the decrement of timestep in second,
SIMUL	is the total simulation time in hour,
ITER	is the update interval (timestep) that interface model is called,
TOUT	is the output period in hour,
KODE	{ 0, suppress the efflux velocities 1, output the efflux velocities
KMODEL	{ 1, kinematic routing technique otherwise, diffusion hydrodynamic model
NNOD	is the total number of nodal points for floodplain,
NODC	is the number of channel element,
SIDE	is the dimension of the uniform grid side in feet,
TOL	is the specified surface detention in feet,
DTOL	is the minimum change of water depth in feet for each timestep,
DTOLP	is defined as
	$DTOLP = \frac{\text{change of water depth}}{\text{pervious water depth}} \times 100\%$
FP(I,1)	is the northern nodal point of node I,
FP(I,2)	is the eastern nodal point of node I,
FP(I,3)	is the southern nodal point of node I,
FP(I,4)	is the western nodal point of node I,

FP(I,5)	is the averaged Manning's friction factor for node I,
FP(I,6)	is the averaged surface elevation for node I in feet,
FP(I,7)	is the initial water depth for node I in feet,
NERI	is the number of uniform effective rainfall rate data pairs,
R(I,1)	is the time (hour) corresponding to the effective rainfall rate,
R(I,2)	is the effective rainfall intensity (in/hr) ordinate for effective rainfall rate,
NFPI	is the number of input nodal points for the flood plain,
NPFPI	is the number pair of inflow hydrograph rate entires,
KINP(I)	is the array that stores the inflow boundary condition nodal points,
HP(I,J,1)	is the time (hour) corresponding to the inflow hydrograph,
HP(I,J,2)	is the inflow rate (cfs) ordinate for the inflow hydrograph,
NDC	is the number of critical-depth outflow nodal points,
NODDC(I)	is the array which stores the critical-depth outflow nodal points,
NFLUX	is the number of nodal points where outflow hydrographs are being printed,
NFOUT	is the interval for outflow hydrograph (in timesteps),
NODF(I)	is the array which stores the nodal points where outflow hydrographs are being printed,
KK	is the nodal point for channel element,
FC(KK,1)	is the array which stores the averaged Manning's coefficient of the channel elements,
FC(KK,2)	is the array which stores the width of the channel elements,
FC(KK,3)	is the array which stores the depth of the channel elements,
FC(KK,4)	is the array which stores the bottom elevation of the channel elements,
FC(KK,5)	is the array which stores the initial water depth of the channel elements,
NCHI	is the number of the inflow boundary conditions for the channel system,
NPCHI	is the pair of inflow hydrograph entries of the channel system,
NCHO	is the number of the outflow boundary conditions for the channel system,
NPCHO	is the pair of outflow hydrograph entries of the channel system,
NSTA	is the number of the stage station nodal points,
NPSTA	is the pair of stage curve entries,
KIN(I)	is the array which stores the nodes of inflow hydrograph of the channel system,
H(I,J,1)	is the time (hour) corresponding to the inflow hydrograph for the channel system,
H(I,J,2)	is the inflow rate (cfs) ordinate for the inflow hydrograph for the channel system,
KOUT(I)	is the array which stores the nodes of outflow hydrograph of the channel system,
HOUT(I,1)	is the array which stores the depth that a specified stage-discahrge curve is used,
HOUT(I,2)	is the array which stores the multiplier of a stage-discharge curve,
HOUT(I,3)	is the array which stores the exponent of a stage-discharge curve,
NOSTA(I)	is the array which stores the node of stage curve for the channel system,
STA(I,J,1)	is the array which stores the time (hour) corresponding to the time-stage curve,
STA(I,J,2)	is the array which stores the water surface elevation (feet) of the time-stage curve.

**Note:**

1. If any value of NERI, NFPI, NDC, NFLUX and NODC is equal to zero, then the values for the corresponding array need not be entered in the input file.
2. If NODC equals to zero, then entire channel element information need not be entered in the input file.

**III.3 USER'S INSTRUCTIONS**

*Introduction*

The DHM model has the capabilities to perform: (1) one-dimensional analysis, (2) two-dimensional analysis and (3) one- and two-dimensional interface analysis.

*III.3.1 One-dimensional analysis*

For one-dimensional analysis, a zero value should be entered for variable ITER. The entries for array FP(I,J) should reflect the one-dimensional representation as shown in Fig. 18.

*III.3.2 Two-dimensional analysis*

For two-dimensional analysis, zero values should be assigned to variables ITER and NODC. The entire data entries for the channel system can be neglected in the input file.

*III.3.3 One- and two-dimensional interface model*

When variables ITER and NODC are not equal to zero, the interface model is called at each update interval to calculate the new surface water elevations for both the grid and channel elements. A negative sign should be used in the Manning's coefficient to indicate a channel element passing through a grid element.

*III.3.4 Inflow boundary conditions*

Inflow boundary conditions are described by a linear time-inflow rate hydrograph for each specified inflow grid or channel element.

*III.3.5 Outflow boundary conditions*

Outflow boundary conditions for channel element (Fig. 19a) are:

- (1) unidirectional critical depth assumption, i.e., discharge per unit length is  $q = 5.67 (\text{depth})^{1.5}$ , and
- (2) no flow boundary conditions (Fig. 20).

Outflow boundary condition for channel system is described by the following equation (Fig. 19b) as:

$$Q = \begin{cases} 0 & \text{If } 0 \leq \text{depth of water} \leq \text{specified surface detention} \\ \alpha_1 (\text{depth})^{\beta_1} & \text{If specified surface detention} < \text{depth of water} \leq d_1 \\ \alpha_2 (\text{depth})^{\beta_2} & \text{If } d_1 < \text{depth of water} \leq d_2 \\ \vdots & \vdots \end{cases}$$

where  $d_1, d_2, \dots$ , are the pre-determined values from a stage-discharge station and up to 10 set of data can be used to represent the stage-discharge relationship for each station.

### III.3.6 Variable time step

Variable time step dramatically reduces the computational time. The algorithm of the variable time step is depicted in Fig. 20.

where

- $\Delta h^i(I)$  is the change of water depth for Node I at timestep  $i$ ,
- Delh is the user specified tolerance,
- $\Delta t^i$  is the interval for timestep  $i$ ,
- $\Delta t_i$  is the user specified incremental time interval,
- $\Delta t_D$  is the user specified decremental time interval,
- TOLP is the user specified percentage of water depth, and
- Dcheck is the user specified percentage of water depth, and Dcheck is defined as Delh/TOLP.

### III.3.7 Kinematic routing technique

The kinematic routing technique is also included in the DHM model. By setting KMODEL to 1, the kinematic routing is evoked. A comparison study between the kinematic routing technique and the diffusion model is presented in section V.

## IV. APPLICATIONS OF THE DHM ONE-DIMENSIONAL MODEL

### Application 1: Steady flow in an open channel

Because the DHM is anticipated for use in modelling watershed phenomena, it is important that the channel models represent known flow characteristics. Unsteady flow is examined in the previous section. For steady flow, a steady-state, gradually varied flow problem is simulated by the 2-D diffusion model. Figure 21 depicts both the water levels from the 2-D diffusion model and from the flow resistance equation. For an 8000 cfs constant inflow rate, the surface water profiles from both the 2-D diffusion model and the flow resistance equation match quite well. The discrepancies of these profiles occur at the break points where the upstream channel slope and downstream channel slope is equal to 0.001 and the downstream channel slope is equal to 0.005, the surface water level is assumed to be equal to the critical depth. However, Henderson (1966), notes that brink flow is typically less than the critical depth ( $D_c$ ). The DHM water surface closely matches the 0.72  $D_c$  brink depth.

It is clear to see that the DHM cannot simulate the hydraulic jump, but rather smooths out the usually assumed 'shock front'. However, when considering unsteady flow, the DHM may be a reasonable approach for approximating jump profile. For a higher inflow rate, 20,000 cfs, the surface water levels differ in the most upstream reach. Again, this is due to the downstream control, critical depth, of the flow resistance equation.

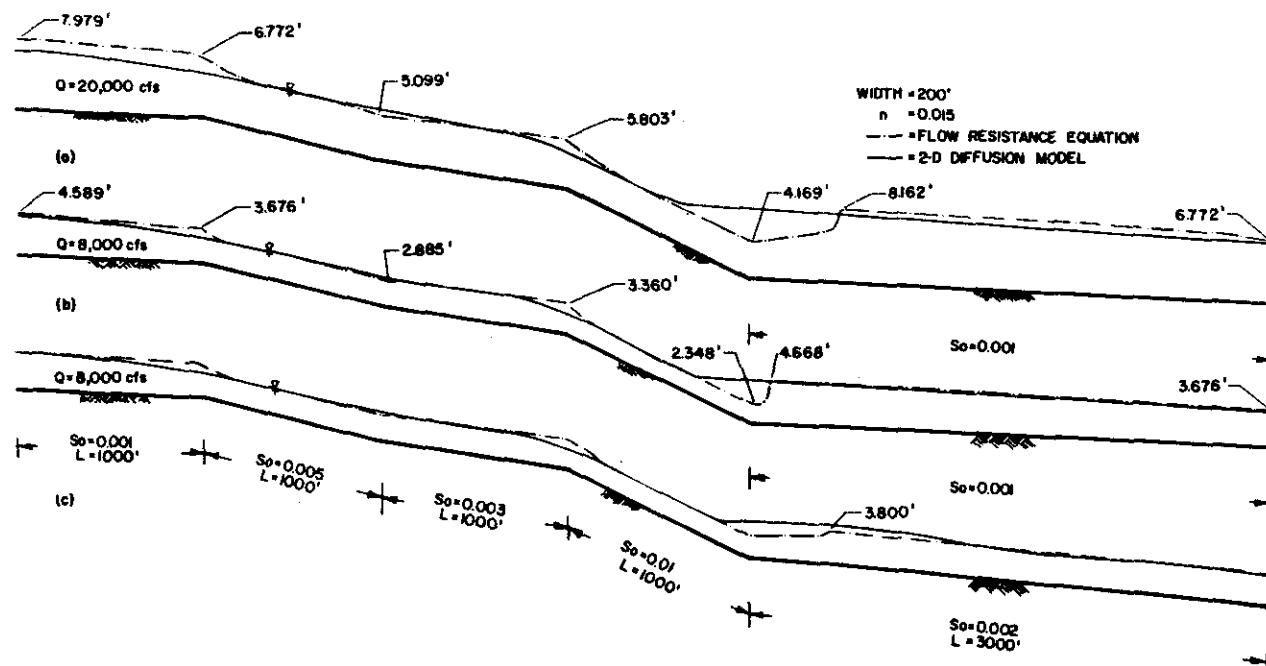


Fig. 21. Gradually varied flow profiles (Note DHM smoothing of hydraulic jumps and 'brink' flow depths less than critical depth)

IV.1 TWO-DIMENSIONAL APPLICATIONS

Application 2: Rainfall-runoff model

The DHM can be used to develop a runoff hydrograph given the time distribution of effective rainfall. To demonstrate the DHM runoff hydrograph generation, the DHM is used to develop a synthetic S-graph for a watershed where overland flow is the dominating flow effect.

To develop the S-graph, a uniform effective rainfall is assumed to uniformly occur over the watershed. For each timestep (e.g., 5-seconds), an incremental volume of water is added directly to each grid-element based on the assumed constant rainfall intensity, resulting in an equivalent increase in the nodal point depth of water. Runoff flows to the point of concentration according to two-dimensional diffusion hydrodynamics.

The following applications show S-graphs developed for several hypothetical watersheds with various cross-slopes, channel slopes, areas, and friction factors. Figure 22 shows the watershed discretization used for the S-graph development shown in Fig. 23. Included for comparison purposes in Fig. 23 are the S.C.S. S-graphs for a triangular and a curvilinear unit hydrograph representations. It is seen that the DHM runoff S-graph closely matches the equivalent S.C.S. S-graph. Fig. 24 shows other S-graphs developed for different watershed configurations and conditions. From the figure, all S-graphs have a strong similarity to the S.C.S. S-graph.

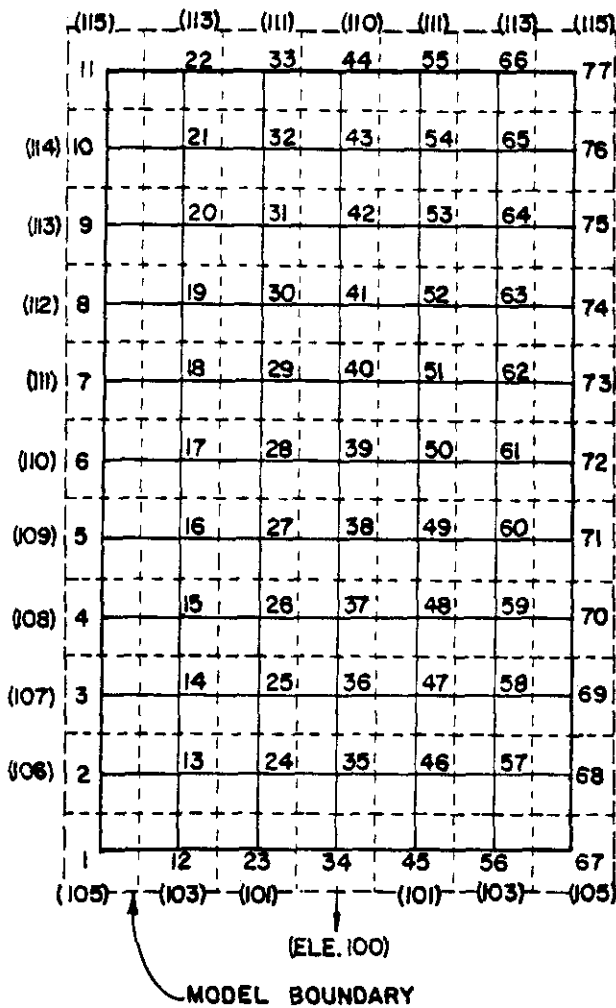


Fig. 22. Test watershed discretization

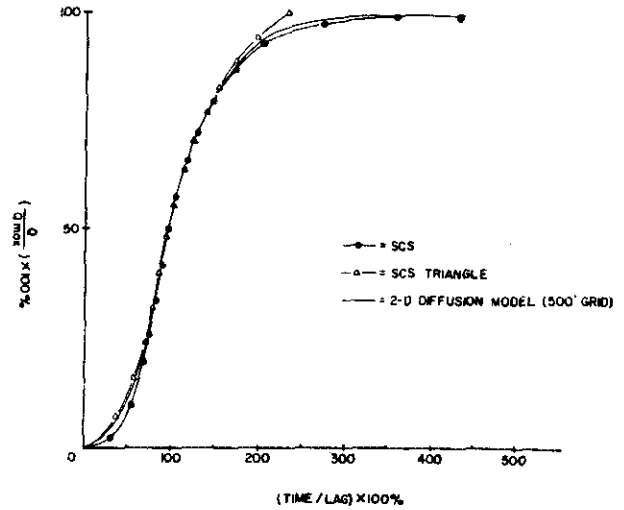


Fig. 23. Diffusion model produced S-graph and S.C.S. S-graphs

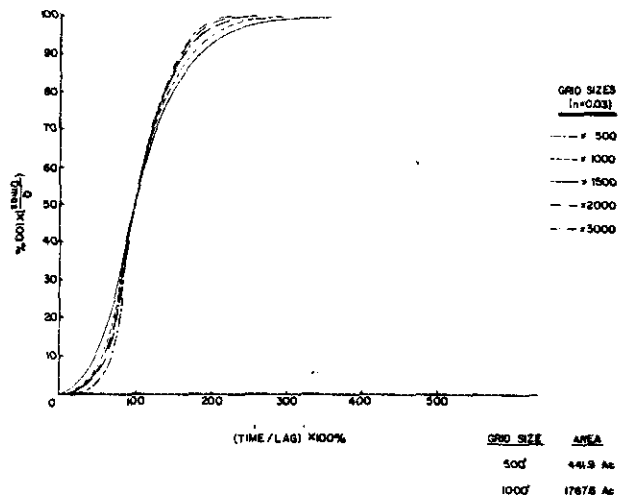


Fig. 24a. Diffusion model produced S-graphs for various grid sizes (nodal elevations held constant)

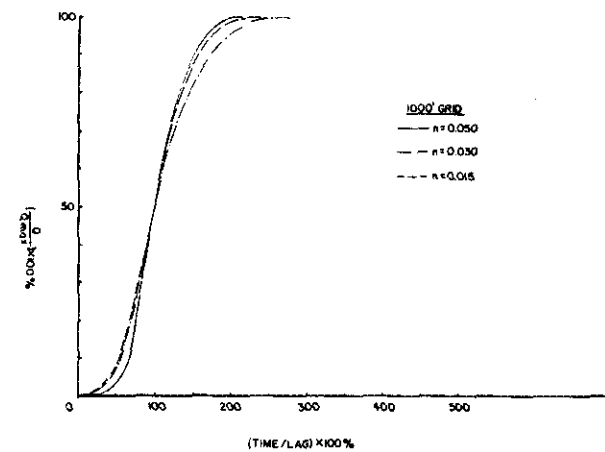


Figure 24b. Diffusion model produced S-graph for various Manning's friction factors

Application 3: Rainfall-runoff model

The 10 square mile Cucamonga Creek watershed (California) is shown discretized by 1000-foot grid elements in Fig. 25. A design storm (Fig. 26a) applied by the U.S. Army Corps of Engineers and resulting runoff

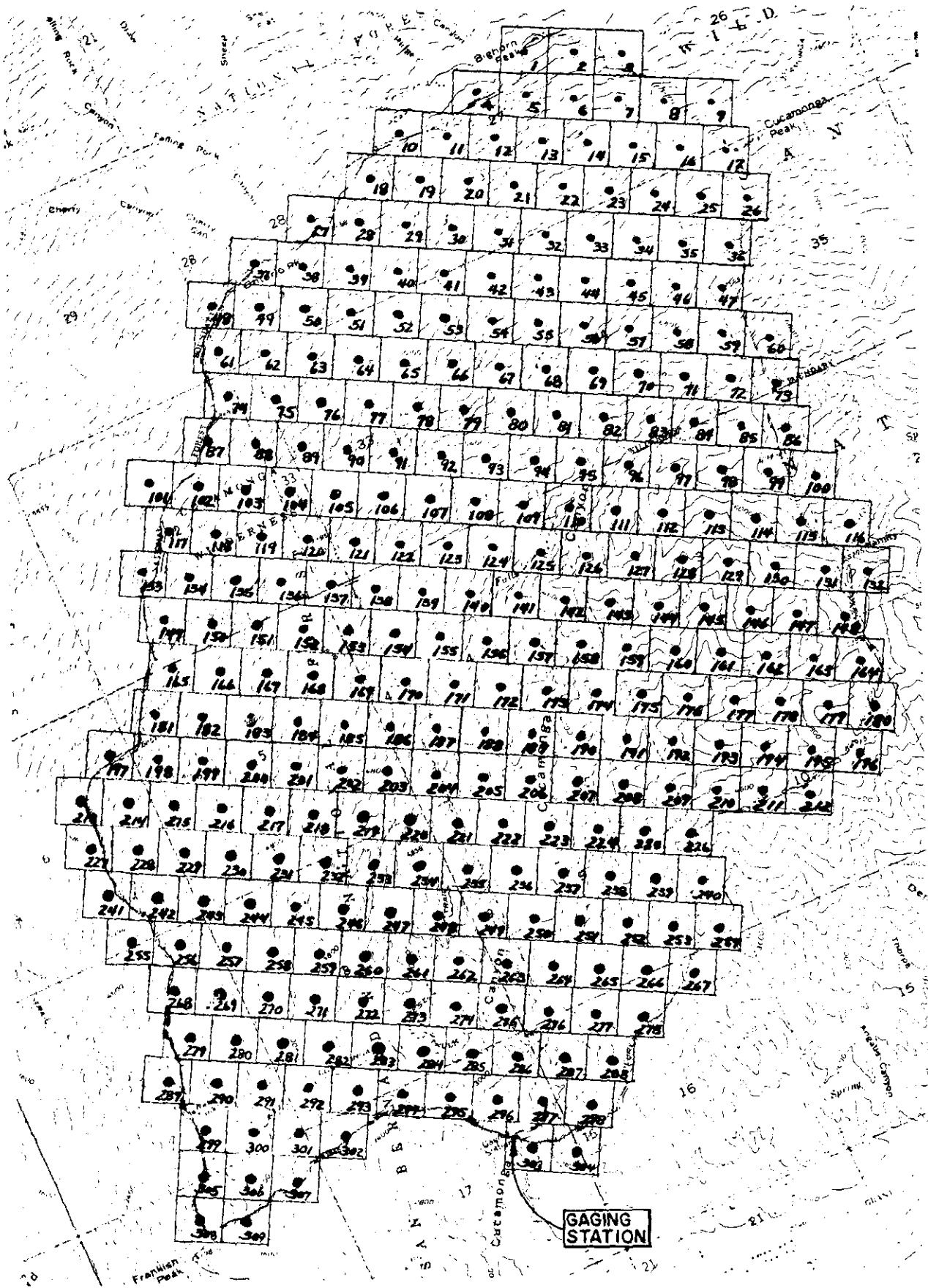


Fig. 25. Cucamonga Creek discretization

hydrograph is shown in Fig. 26b. Also shown in Fig. 26b is the corresponding DHM runoff hydrograph. From the figure, the diffusion model develops runoff quantities which are in good agreement with the values computed using unit hydrograph (S-graph) derived from stream gage data.

**Application 4: Small-scale dam-break floodplain analysis**

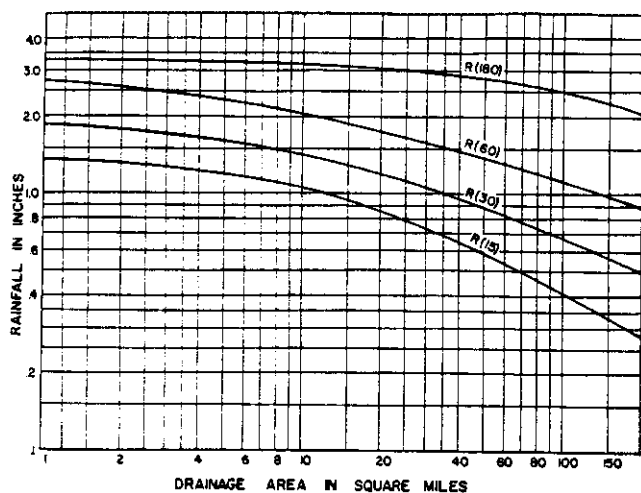
The DHM is applied to a hypothetical dam-failure of the Orange County Reservoir located north of the City of Brea, Orange County, California (see Fig. 27). The study site (Fig. 28) includes the area between the Orange County Reservoir (north of the City of Brea) and the proposed Brea Mall development. This application of the DHM illustrates its use in a municipal setting where flood flow patterns are impacted by railroads, bridge under-crossings, and other man-made obstacles to flow.

Using current U.S.G.S. topographic quadrangle maps (photo-revised, 1981) a 500-foot grid control volume discretization was constructed as shown in Fig. 29.

In each grid, an area-averaged ground elevation was estimated based on the topographic map. A Manning's friction factor of  $n = 0.040$  was used throughout the study. (Canyon reaches,  $n = 0.030$ ; grassy plains,  $n = 0.050$ ).

Major assumptions used in this DHM application are as follows:

- (1) Friction effects are modelled by Manning's equation as used in the DHM.
- (2) All storm drain systems provide negligible draw off of the dam-break flows. This assumption accom-



HYETOGRAPH COMPUTATION

UNIT PERIOD	AMOUNT
1	.27 (R(180)-R(60))
2	.25 (R(180)-R(60))
3	.11 (R(180)-R(60))
4	.25 (R(180)-R(60))
5	.20 (R(180)-R(60))
6	.22 (R(180)-R(60))
7	.14 (R(180)-R(60))
8	.16 (R(180)-R(60))
9	.48 (R(60)-R(30))
10	.32 (R(60)-R(30))
11	1.00 (R(15))
12	1.00 (R(30)-R(15))

LOCAL PROJECT STORM  
DEPTH AREA DURATION CURVES

Fig. 26a. Design storm

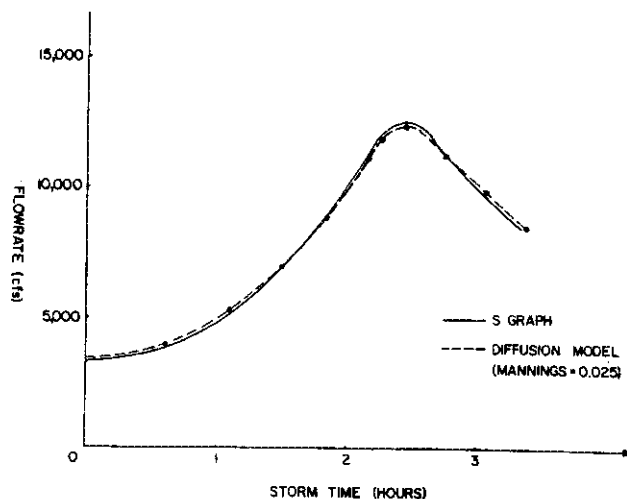


Fig. 26b. Modelled runoff hydrographs

modates a design storm in progress during the dam failure. This assumption also implies that storm water runoff provides a negligible increase to the dam-break flow hydrograph.

- (3) All canyon damming effects due to culvert crossings provide negligible attenuation of dam-break flows. This assumption is appropriate due to the concurrent design storm assumption, and due to sediment deposition from transport of the reservoir earthen dam materials.
- (4) The reservoir failure conforms to an outflow hydrograph such as shown in Fig. 30.

The Orange County Reservoir is an earthen dam lined (along the interior) with concrete. In the improbable event of a failure, an erosive process will initiate which allows the escape flowrate to increase gradually rather than suddenly as would occur due to failure of a rigid structure.

Due to the low volume (200 acre-feet) retained in the reservoir, the outlet hydrograph assumed is a significant function of the remaining reservoir storage. That is, the reservoir volume is quickly depleted by low-to-moderate flows out of the reservoir.

To estimate a reasonable peak outlet flow,  $Q_p$ , an iteration method is used until a balance between the estimated outlet hydrograph  $Q_p$  is made to the resulting flowrate as a function of the remaining stored waters.

The ultimate outlet geometry is assumed to be a V-shaped massive failure with side slopes at a 45-degree incline. Flows are then based on critical depth, with a free outlet to the steep downstream canyon reaches. Backwater effects to the dam outlet are assumed negligible due to the steep terrain, and to also assume a more conservative condition.

Based on the above assumptions, the outlet flowrate for a ponded depth  $H$  (feet) is given (for the ultimate dam-break failure geometry) by  $Q_p = 2.472 H^{2.5}$  cfs. The reservoir rating curve relating basin depth to volume is shown in Table 1. For the assumed outlet hydrograph shape width  $Q_p$ , occurring at 20-minutes after dam-failure, the volume drained by time 20-minutes is given by  $V_d = 0.01377 Q_p$  (acre-feet). The estimate of  $Q_p$  is provided by the iteration shown in Table 2.

From the table, it is seen that  $Q_p$  is strongly influenced by the quick depletion of the reservoir's stored waters. It



schematic representation of the problem domain and the reasonableness in modelling results. The model discretization was adjusted when considered appropriate to better represent field conditions and subsequent modelling results rechecked by additional field investigations.

In order to develop more refined modelling results, detailed survey information would be required to reduce the uncertainty in elevations determined from the cited topographic maps.

Comparison of the flood plain to a previous study (Metropolitan Water District of Southern California, 1973) is shown in Fig. 32. The main differences in

estimated flood plains is due to the dynamic nature of the diffusion model which accounts for the storage effects due to flooding, and the attenuation of a flood wave due to two-dimensional routing effects.

To examine the sensitivity in modelling results, the dam-break was assumed to occur at node 6 (neglecting canyon routing), and also the peak outflow was doubled to  $Q_p = 18\,000$  cfs (at a time of 20 minutes). To allow this new  $Q_p$  to occur, the basin volume was doubled to over 400 acre-feet. The resulting flood plain is shown in Fig. 33. From this figure, it is seen that with doubling the basin capacity, only the northeastern portion of the Brea Mall site is still estimated to be affected by the hypothetical dam-

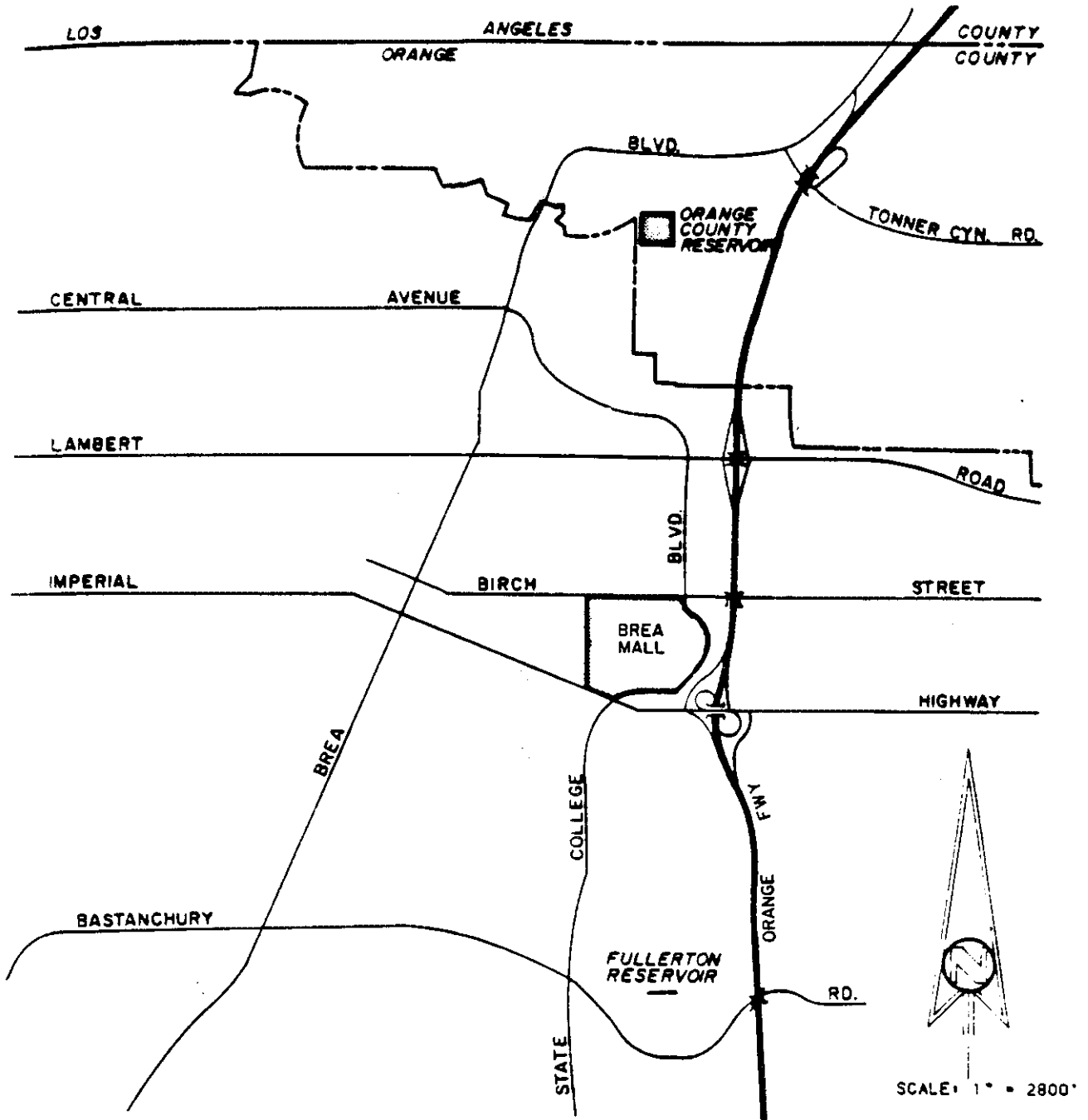


Fig. 28. Location map for the Orange County reservoir dam-break problem





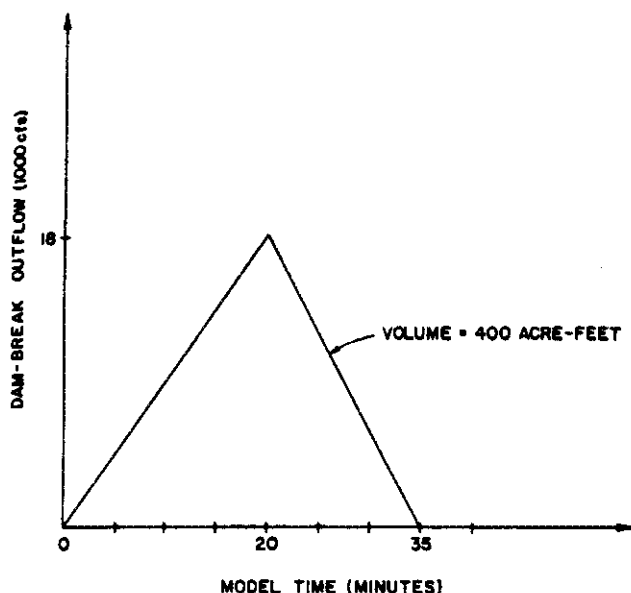


Fig. 30. Study dam-break outflow hydrograph

failure of the Orange County Reservoir. In this second analysis, the flooding depth is estimated to increase to about 2.5 feet.

Figure 34 shows lines of arrival times for the second (volume doubled) basin test study.

*Application 5: Small-scale flows onto a flat plain*

A common civil engineering problem is the use of temporary detention basins to offset the effect of urbanization on watershed runoff. A problem, however, is the analysis of the basin failure; especially, when the floodflows enter a wide expanse land surface with several small channels. This application is to present study conclusions in estimating the flood plain which may result from a hypothetical dam-failure of the L02P30 Temporary Retarding Basin. The results of this study are to be used to study the potential impacts of the area if the retention basis berm were to fail.

The study site includes the area south of Plano Trabuco, Phase I. It is bounded on the north by L02P30 Retarding Basin Berm, on the east and south of Portola Parkway and on the west by the Arroyo Trabuco bluffs (see Fig. 35).

Using a 1" = 300' topographic map, a 200-foot grid control volume discretization was constructed as shown in Fig. 36. In each grid, an area-averaged ground elevation was estimated based on the topographic map. A Manning's friction factor of  $n=0.030$  was used throughout the study.

Due to the low volume (80.5 acre-feet) retained in the reservoir, the outlet hydrograph assumed is a significant function of the remaining reservoir storage. That is, the reservoir volume is quickly depleted by low-to-moderate flows out of the reservoir.

To estimate a reasonable peak outlet flow,  $Q_p$ , an iteration method is used until a balance between the estimated outlet hydrograph  $Q_p$  is made to the resulting flowrate as a function of the remaining stored waters.

The ultimate outlet geometry is assumed to be a 90-foot wide rectangular broad crested spillway.

Based on the above assumptions, the outlet flowrate for a ponded depth  $H$  (feet) is given (for the ultimate dam-break failure geometry) by  $Q_p = 270 H^{1.5}$  cfs. The rating curve relating basin depth to volume is shown in Table 3. For the assumed outlet hydrograph shape width  $Q_p$  occurring at 15-minutes after dam-failure, the volume drained by time 15-minutes is given by  $V_d = 0.01033 Q_p$  (acre-feet). The estimate of  $Q_p$  is provided by the iteration shown in Table 4.

The dam-break hydrograph of Fig. 37 was used as seen in the inflow hydrograph to the grid network of Fig 36. The resulting flood-plain using the dam-break model is shown in Fig. 38. It is seen from Fig. 38 that the floodplain continues south across the Portola Parkway in some areas and also spreads westerly and eventually flows down the Arroyo Trabuco bluffs.

The profile of Portola varies approximately 2 feet above and below the adjacent land. Consequently, minor ponding may occur where Portola Parkway is high and sheet flow across Porthola Parkway will occur at low points. It should be noted that depths along Portola Parkway are less than 1 foot. Figure 39 shows lines of arrival times for the basin study.

Directly below the location of the dam-break at grid no. 3, the water depth is the greatest reading 1.3 feet, but the maximum depth of Portola Parkway is 0.8 feet. It is

Table 1. Orange County reservoir volume and dam-break outflow

Depth (ft)	Volume (af)	$Q_p$ (Dam-break) (cfs)
2	4.6	14
4	9.5	79
6	14.8	218
8	20.3	447
10	26.4	782
12	32.7	1233
14	39.4	1813
16	46.5	2530
18	54.1	3400
20	62.0	4420
22	70.3	5611
24	79.2	7000
26	88.5	8520
28	96.2	10300
30	108.4	12200
32	119.1	14300
34	130.2	16700
36	141.9	19200
38	154.0	22000
40	166.7	25000
42	179.9	28300
44	193.8	31800
46	211.0	35500

Table 2. Estimating dam-break  $Q_p$  for Orange County reservoir (20-minute peak time)

Assumed depth (ft)	$Q_p^1$ (cfs)	Volume drained <sup>2</sup> (af)	Volume left <sup>3</sup> (af)	Depth (ft)
20.0	4420	60.9	151.0	37.0
26.0	8520	117.3	94.7	27.5
26.5	8936	123.1	88.9	26.0

NOTES:

- 1:  $Q_p = 2.472 H^{2.5}$
- 2:  $V_d = 0.1377 Q_p \text{ AF}$
- 3:  $V_{\text{left}} = (212 - V_d) \text{ AF}$

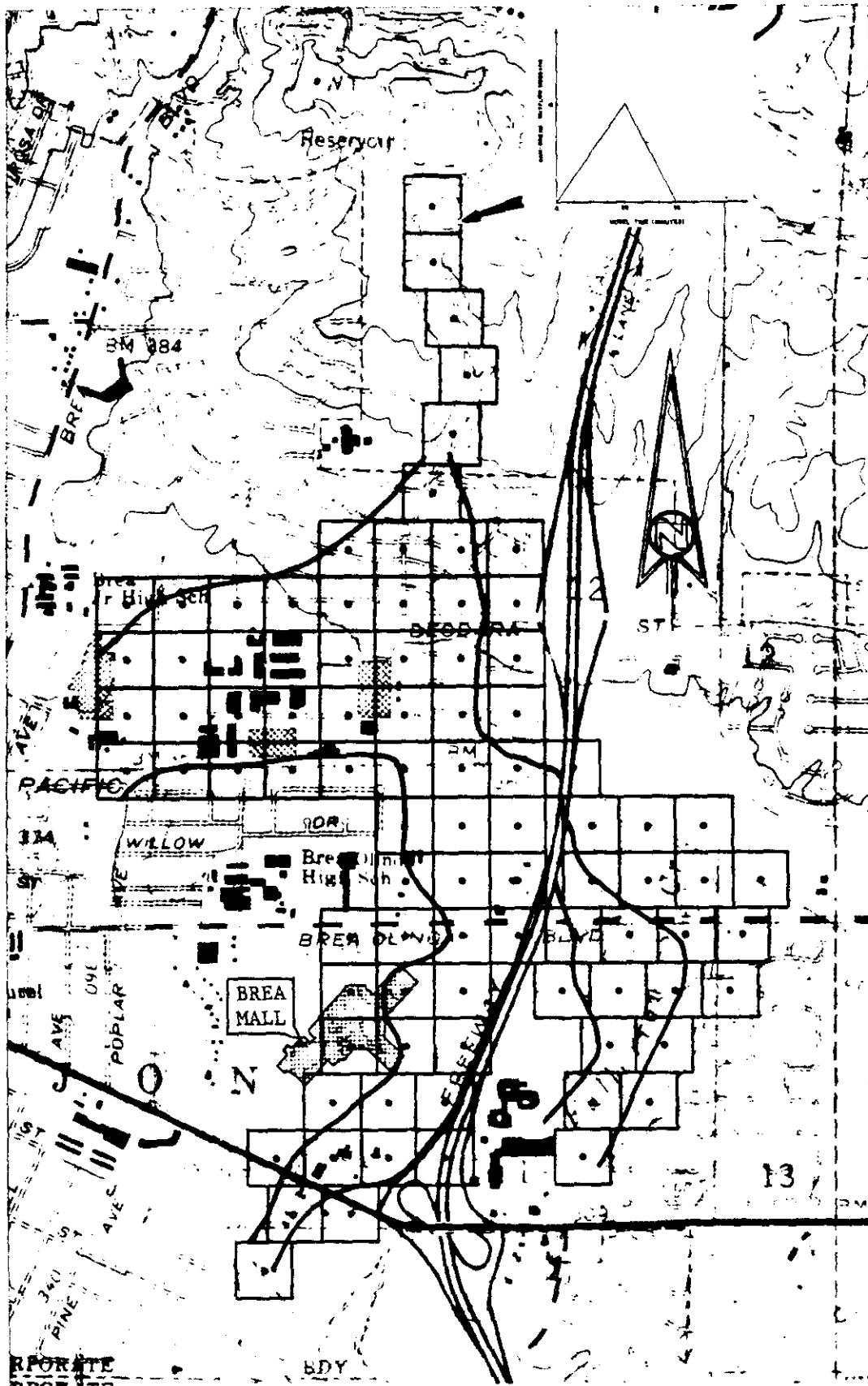


Fig. 31. Flood plain for 200 A.F. basin test for Orange County reservoir

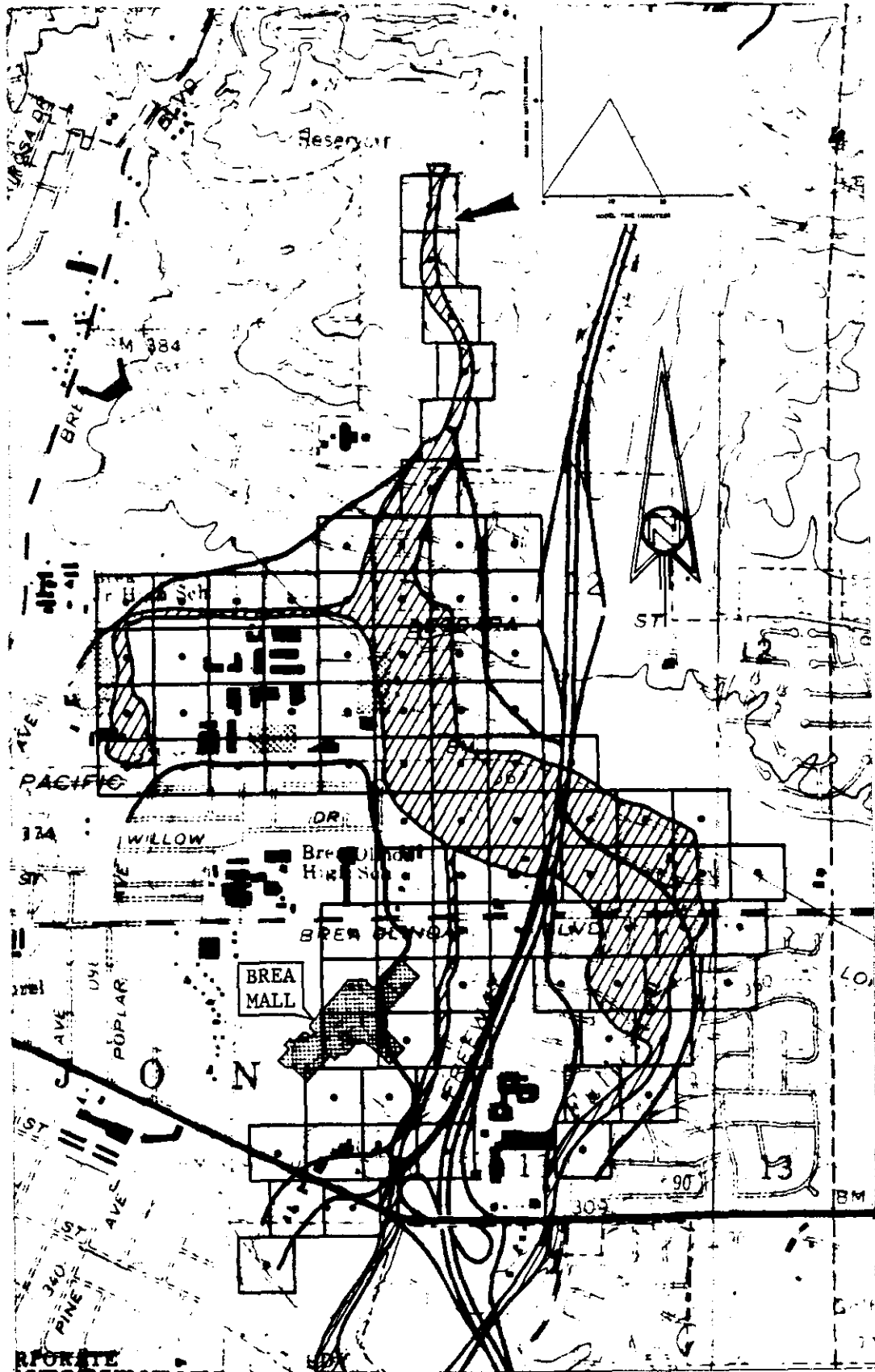


Fig. 32. Comparison of flood plain results

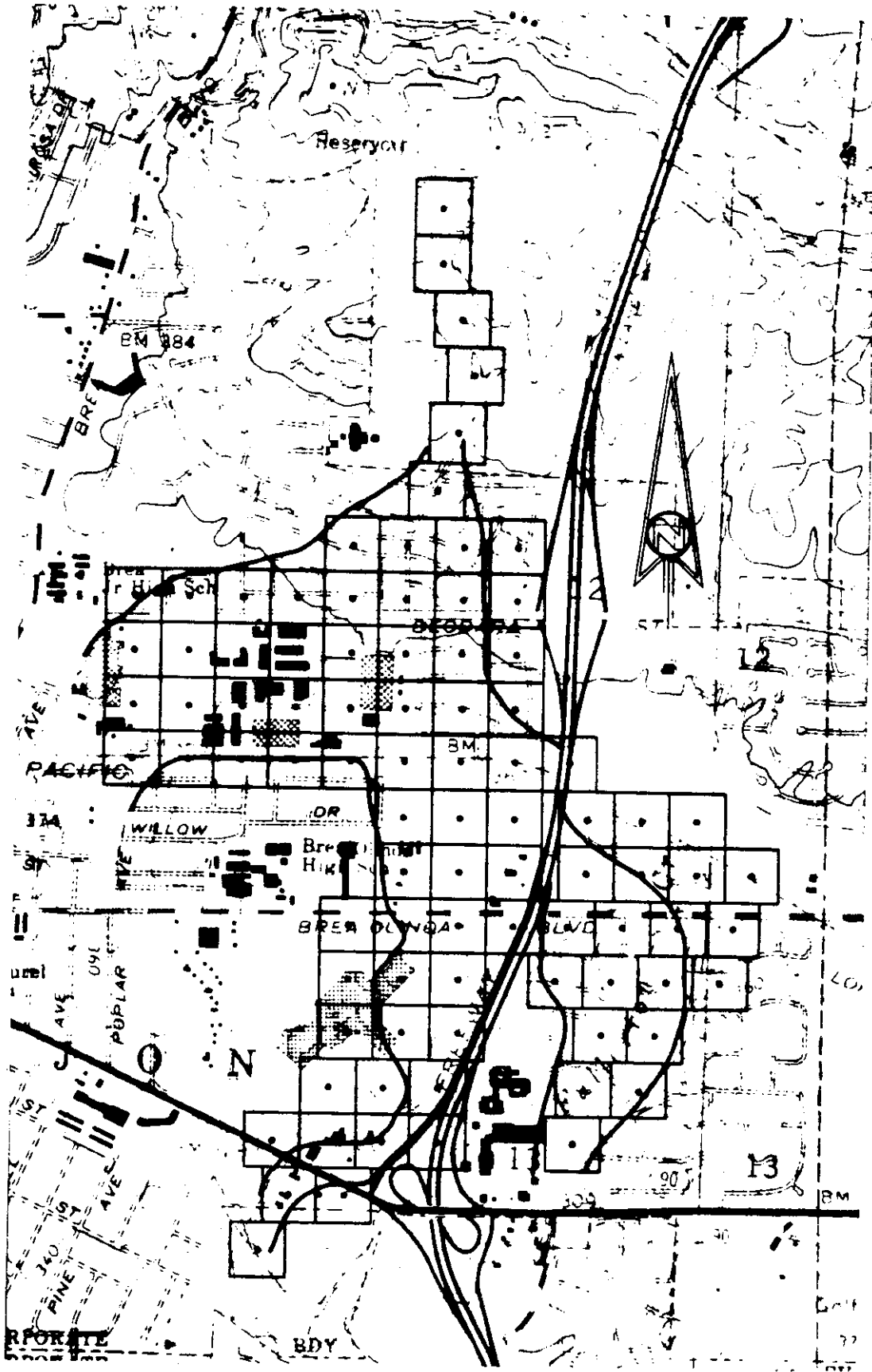


Fig. 33. Flood plain for 400 A.F. basin test for Orange County reservoir

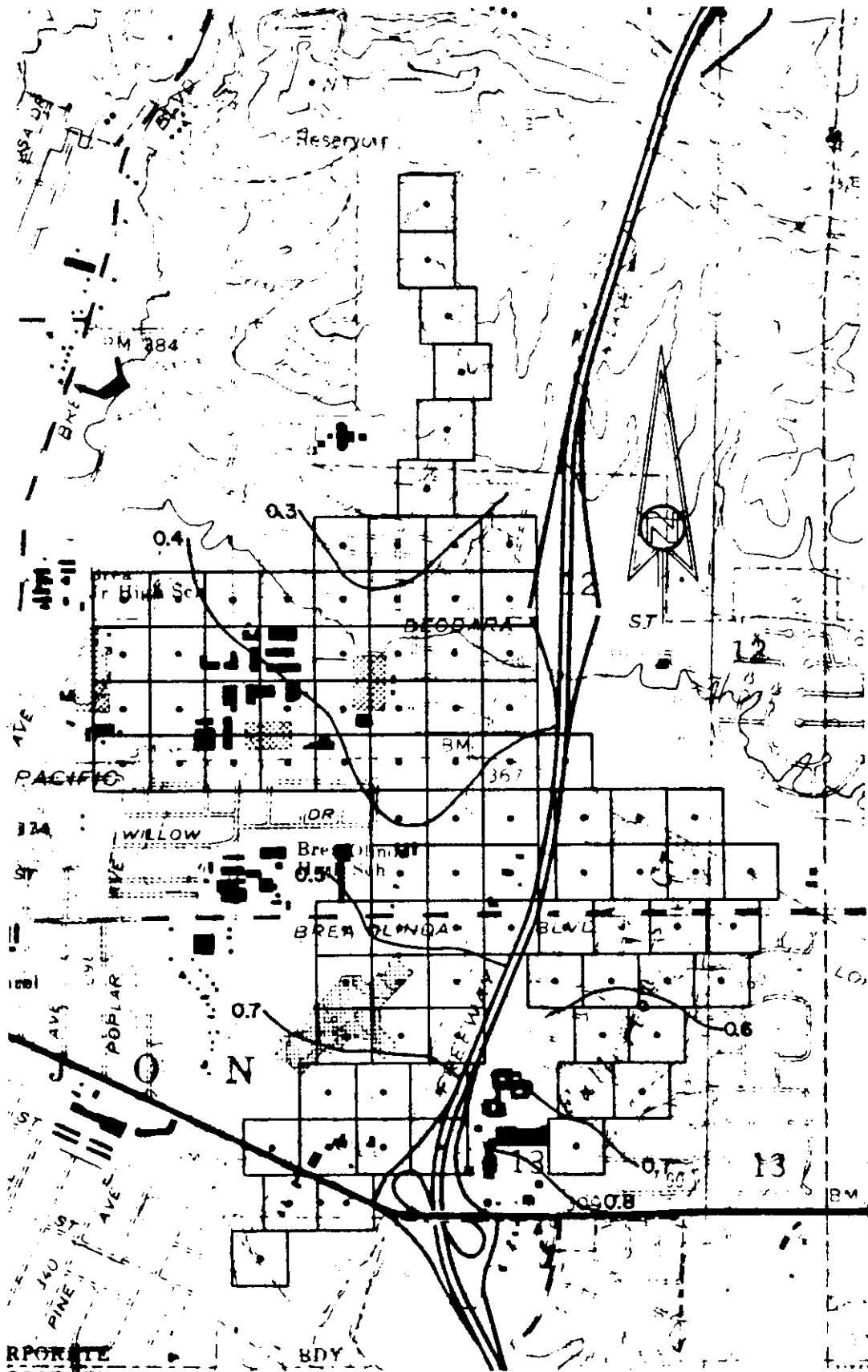


Fig. 34. Time (hours) of maximum flooding depth (400 A.F. basin test) for Orange County reservoir

concluded that Portola Parkway is essentially unaffected by a hypothetical failure of the L02P30 Temporary Retarding Basin.

*Application 6: Two-dimensional floodflows around a large obstruction*

In another temporary detention basin site, floodflows (from a dam-break) would pond upstream of a landfill site,

and then split, when waters are deep enough, to flow on either side of the landfill. An additional complication is a railroad berm located downstream of the landfill, which forms a channel for floodflows. The study site (see Fig. 40) is bounded on the north by a temporary berm approximately 300 feet north of the Union Pacific Railroad, bounded on the east by Milliken Avenue, bounded on the south by the Union Pacific Railroad and bounded on the west by Lower Deer Creek.

In this problem, a four-foot berm is to represent the

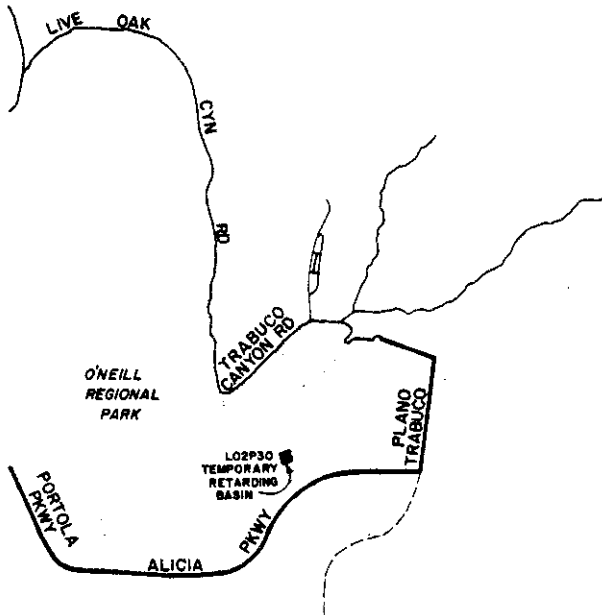


Fig. 35. Location map for L02P30 Temporary Retarding Basin

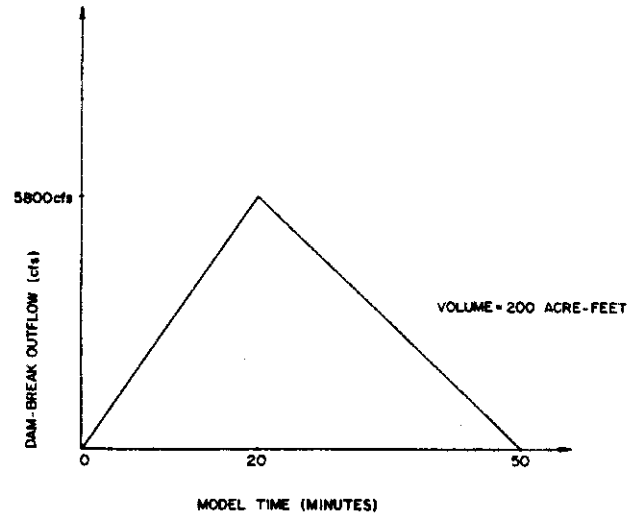


Fig. 37. Study dam break outflow hydrograph for the L02P30 Temporary Retarding Basin

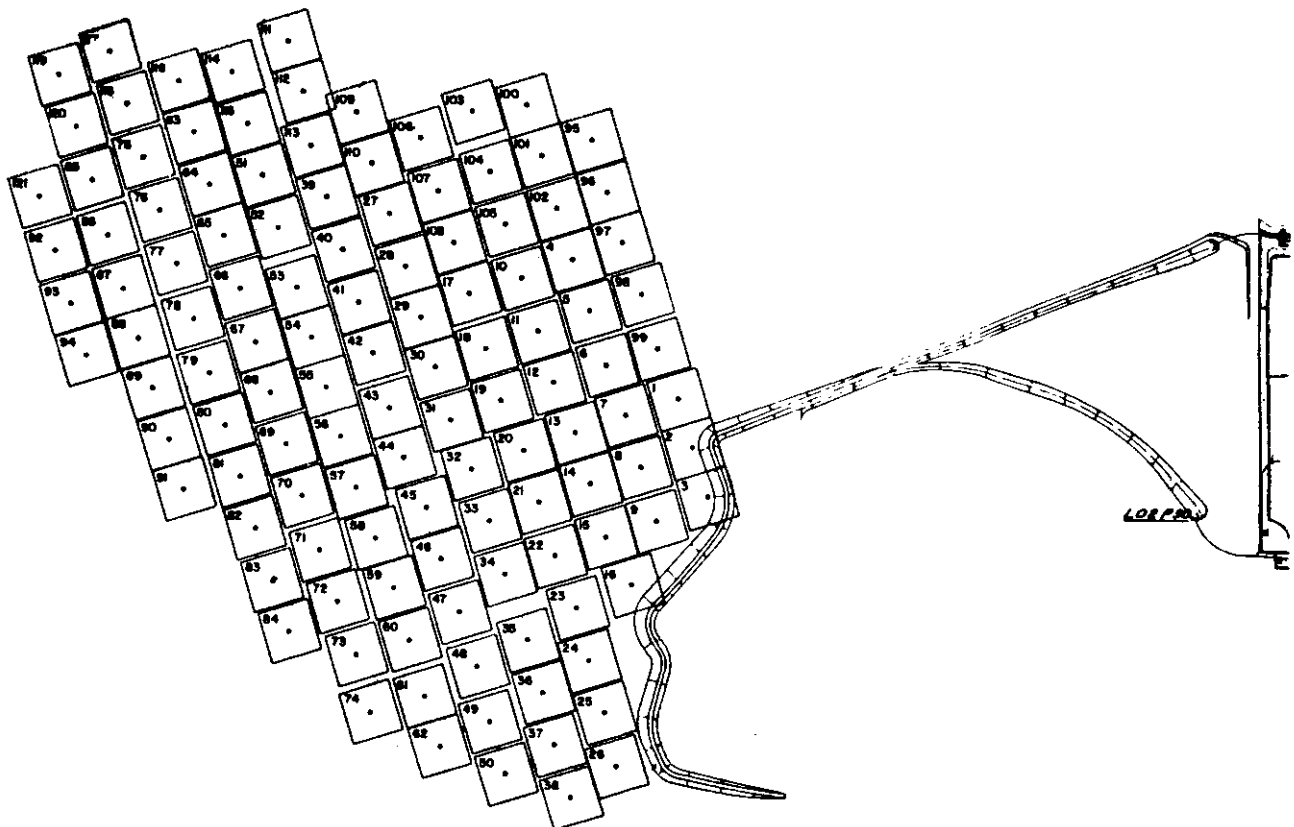


Fig. 36. Domain discretization of L02P30 Temporary Retarding Basin

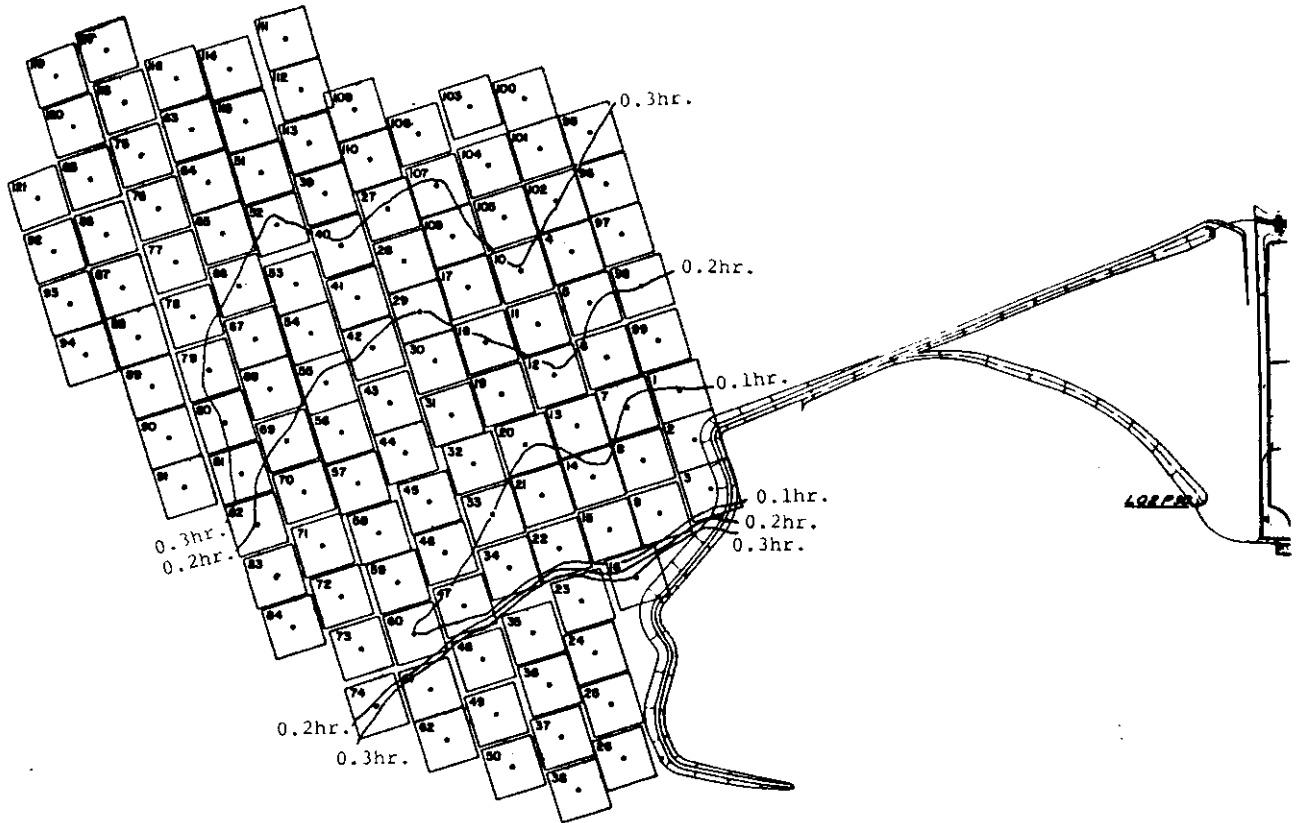


Fig. 38. Time of maximum flooding depth (80.5 A.F. basin test) for L02P30 Temporary Retarding Basin

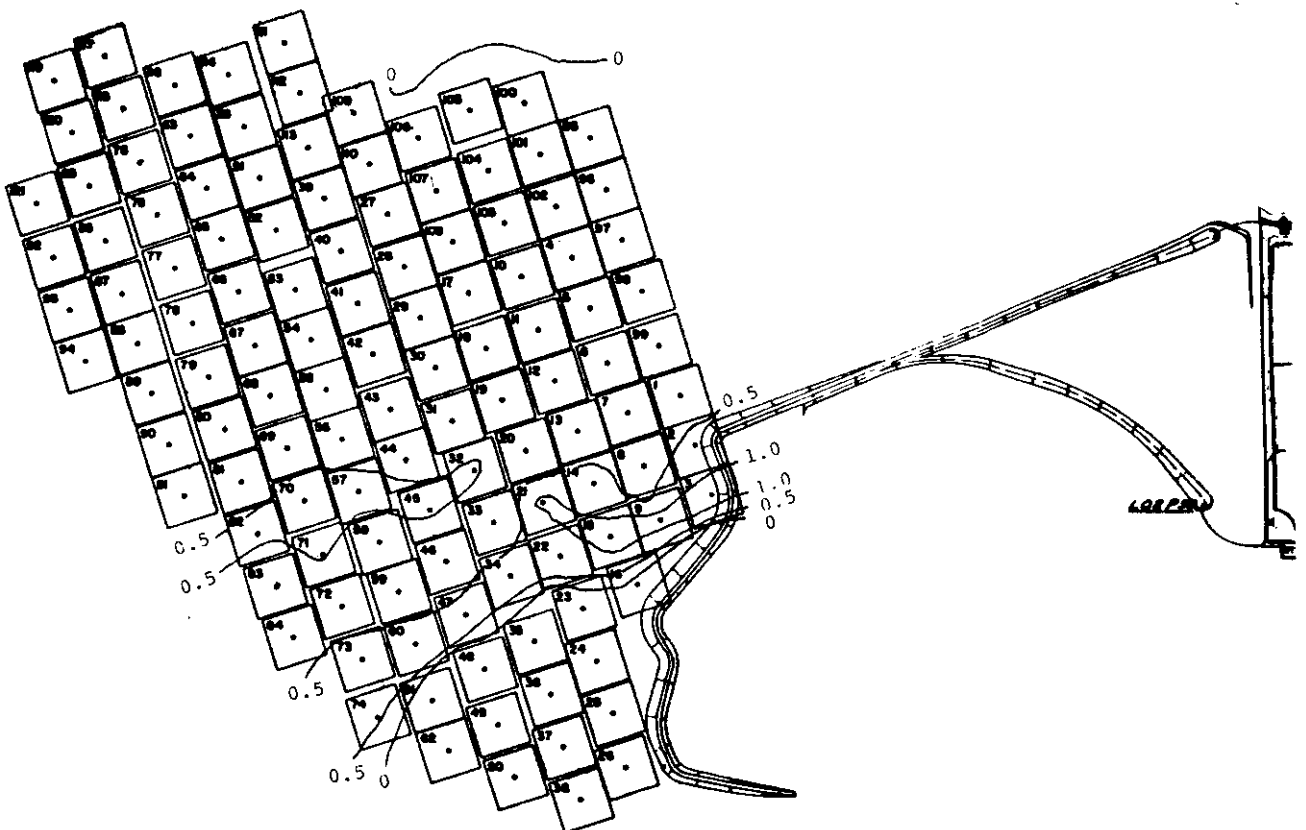


Fig. 39. Flood plain for 80.5 A.F. basin test for L02P30 Temporary Retarding Basin



railroad's floodplain capacity. A 200-foot grid control volume discretization was constructed as shown in Fig. 41.

In each grid, an area-averaged ground elevation was estimated based on the topographic map, except for the

Table 3. L02P30 temporary retarding basin volume and dam-break outflow

Depth (ft)	Volume (af)	$Q_p$ (Dam-break) (cfs)
0.5	9.9	95
1.0	15.1	270
1.5	20.3	496
2.0	25.5	764
2.5	30.7	1 067
3.0	37.6	1 403
3.5	44.6	1 768
4.0	51.5	2 160
4.5	58.4	2 577
5.0	65.4	3 019
5.5	72.3	3 483
6.0	80.5	3 968

railroad boundary where a four-foot berm is imposed. A Manning's friction factor of  $n=0.030$  was used throughout the study.

The O.I.P. Temporary Detention Basin is an earthen dam approximately 15 feet high and 30 feet deep. It is assumed that the flowrate reaches its peak in 20 minutes and then residues over the proceeding 30 minutes. The estimated outflow hydrograph is shown in Fig. 42.

Table 4. Estimating dam-break  $Q_p$  for L02P30 temporary retarding basin (15-minute peak time)

Assumed depth (ft)	$Q_p^1$ (cfs)	Volume drained <sup>2</sup> (af)	Volume left <sup>3</sup> (af)	Depth (ft)
4.0	2 160	22.3	58.2	4.5
4.5	2 577	26.6	53.9	4.2
4.3	2 407	24.9	55.6	4.3

NOTES:

$$Q_p = 3LH^{1.5} \text{ where } L=90'$$

$$Q_p = 270H^{1.5} \text{ cfs}$$

$$2: V_d = 0.01033Q_p \text{ AF}$$

$$V_{\text{left}} = (80.5 - V_d) \text{ AF}$$

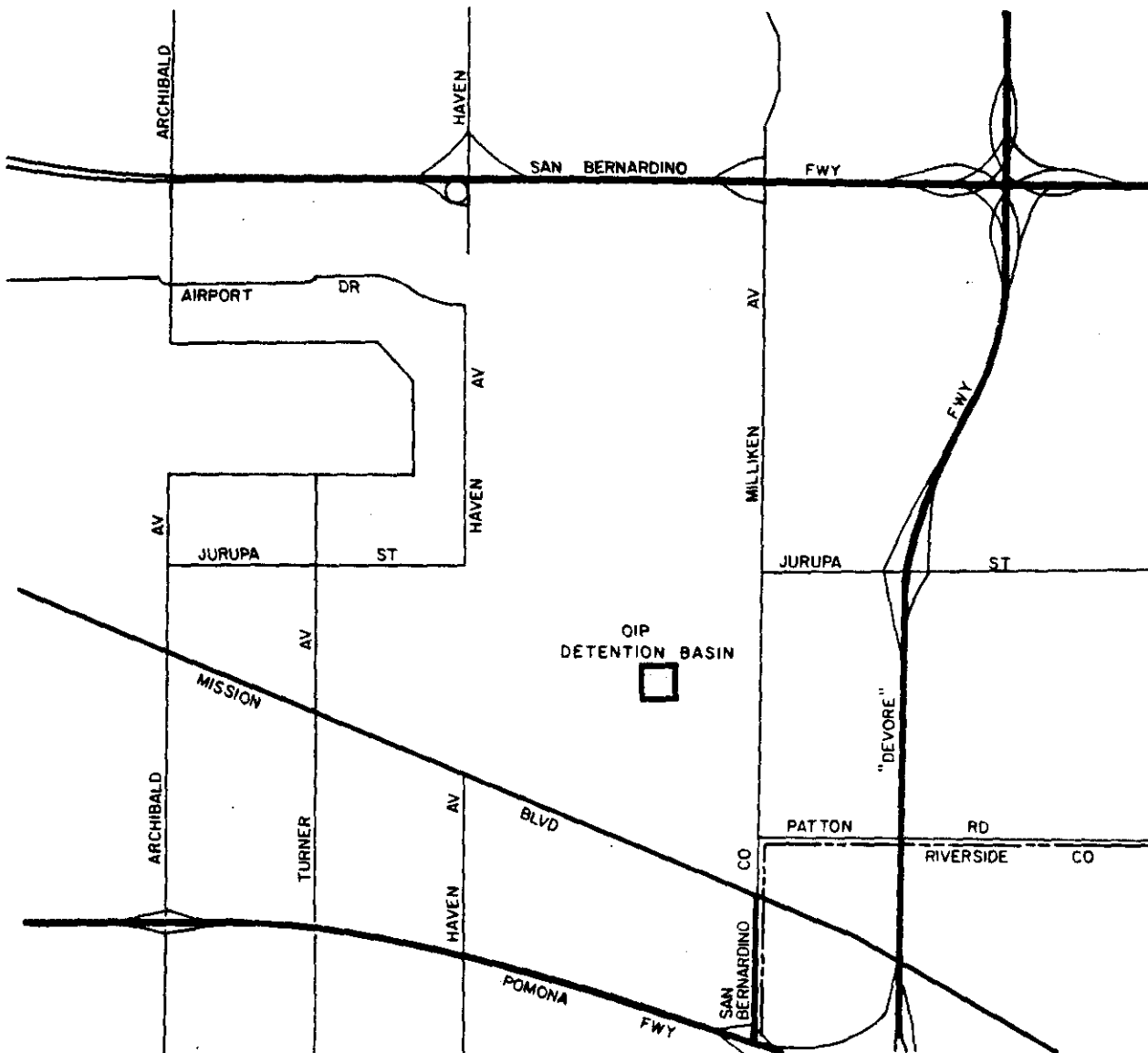


Fig. 40. Location map for O.I.P. Temporary Detention Basin

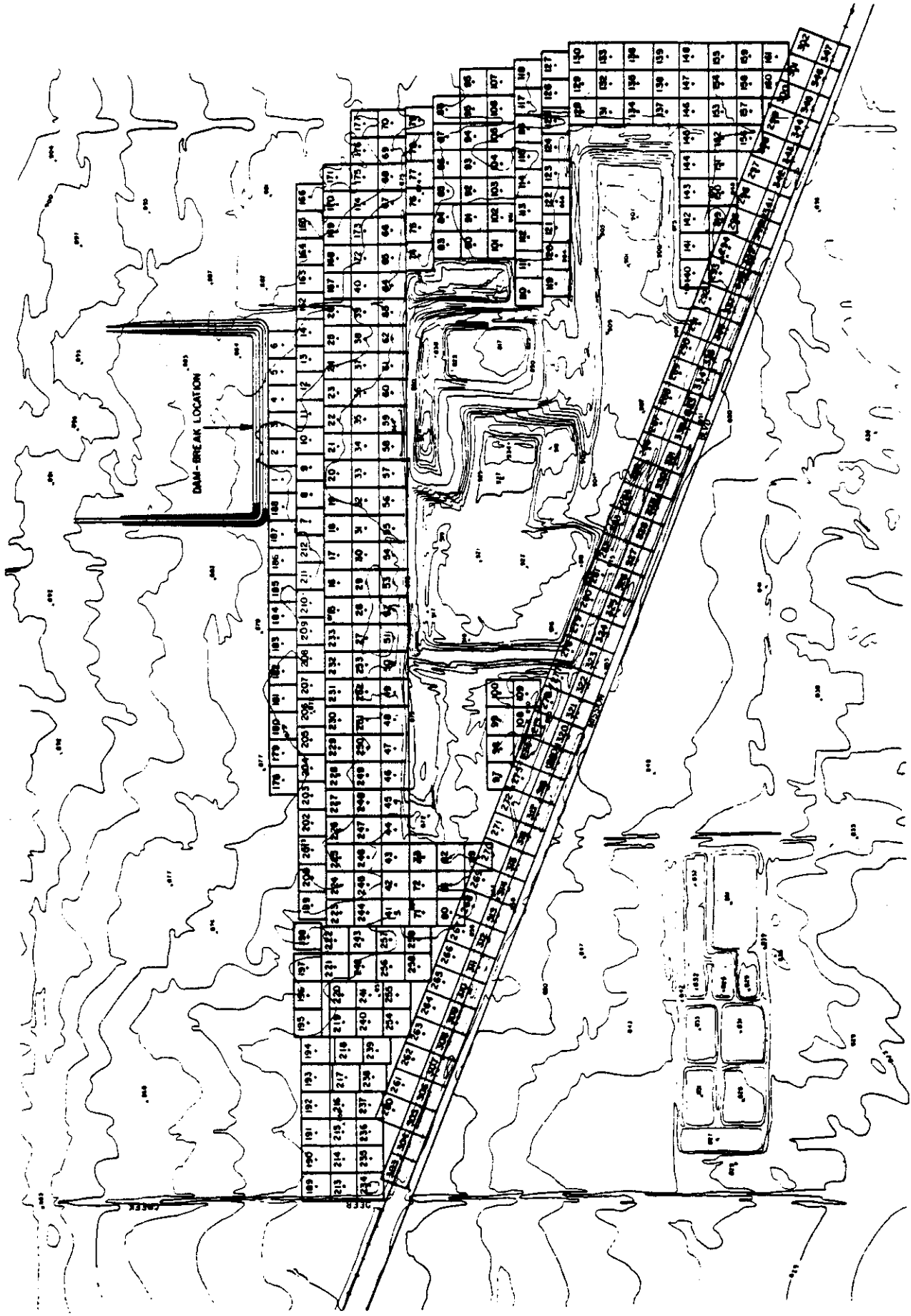


Fig. 41. Domain discretization for O.I.P. Detention Basin

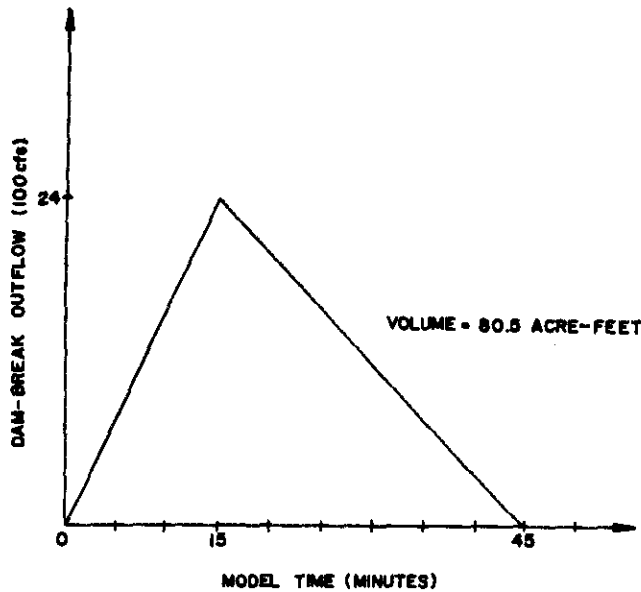


Fig. 42. Study dam-break outflow hydrograph for O.I.P. Temporary Detention Basin

The ultimate outflow geometry is assumed to be a 40 foot break in a 15 foot high, 30 foot wide berm. Based on the said conditions  $Q_p = 3.09 LH^{1.5}$  cfs. For the assumed outlet hydrograph shape with  $Q_p$  occurring at 20-minutes after the dam failure, the volume drained by peak time is given by  $V_d = 0.01377Q_p$  (acre feet).

From the table, it is seen that  $Q_p$  is strongly influenced by the quick depletion of the reservoir's stored waters. It is also noted that the  $Q_p$  is assumed to occur at model time of 20-minutes which indicates a severe erosion rate that destroys a substantial earthen berm structure. Consequently, the estimated  $Q_p$  may be considered conservative.

The dam-break hydrograph of Fig. 42 is used for the inflow hydrograph to the grid network of Fig. 41. The resulting floodplain (using the dam-break model) is shown in Fig. 43. (This floodplain is simply representative of the flow 2.5 hours after the dam-break.) From Fig. 43, it is seen that the floodplain spreads out laterally and flows around the landfill. The flow ponds up around the landfill; along the north side of the landfill, the water ponds as high as 9.2 feet, and along the sides of the landfill, the water ponds up to 5.1 feet high. As the flow travels south, it ponds up to a depth of 4.8 feet against the railroad near Milliken Avenue. Because the water spreads laterally, Milliken Avenue runs the risk of becoming flooded; however, the water only ponds to 0.6 feet along the street. A more in-depth study is needed to see if the water would remain in the gutter or flood Milliken Avenue.

By observing the arrival times of the floodplain in Fig. 44, it is seen that the floodplain changes very little on the west side of the landfill once it reaches the railroad (0.6 hours after the dam-break). But on the east side of the landfill it takes 2.0 hours to reach the railroad.

*Application 7: Estuary modelling*

A hypothetical bay is shown in Fig. 45 and is schematized in Fig. 46. Stage hydrographs are available at seven stations as marked in Fig. 45 and are numbered 1 through 7 (counterclockwise). Stage values in this application are expressed by sinusoidal equations which

are shown in Table 5. Some DHM-predicted flow patterns in the estuary are shown in Figs. 47 to 49. The flow patterns appear reasonable by comparing the fluctuations of the water surface to the stage hydrographs. DHM computed flow patterns compare well to a similar study prepared by Lai (1976).

**IV.2 APPLICATION FOR CHANNEL AND FLOODPLAIN INTERFACE MODEL**

*Application 8: Channel-floodplain model*

Figure 50 depicts a discretization of a two-dimensional hypothetical watershed with three major channels crossing through the floodplain.

Figure 51 depicts the inflow and outflow boundary conditions for the hypothetical watershed model. Figures 52 through 57 show the evolutions of the floodplain. The shaded areas indicate which grid element are flooded. From Fig. 52, it is seen that the outflow rate is less than the inflow rate which results in a flooding situation adjacent to the neighbouring grid elements and the junction of channel B and B' is also flooded. At the end of the peak inflow rate (Fig. 55), about 1/3 of the floodplain is flooded. After 10 hours of simulation, Fig. 57 indicates a flooding situation along bottom of the model. Figure 58 shows the maximum depth of water at 4 downstream cross-sections. It is needed to point out that the water surface elevations are not necessarily incurred at the same time. Finally, Figs 59 and 60 show the outflow hydrographs for both the channel system and the floodplain system.

**IV.3 DISCUSSION**

As shown in the previous examples, the DHM model is capable to simulate the one- and two-dimensional problems, separately. Up till now, no existing numerical model can be used successfully to simulate or predict the evolution of the channel-flood plain interface problem. The proposed DHM model uses a simple diffusion model and interface model to simulate the channel-flood plain interface problem. Results from the hypothetical channel-flood plain interface model show an acceptable floodplain evolution.

Table 5. Boundary values for flow computation in a hypothetical bay

Node	a (ft)	$\xi$ (sec)	M (ft)
63	5	0	0
70	4.95	60	0
74	4.85	180	0
75	4.85	180	0
46	4.75	1200	0.3
39	4.725	1260	0.35
33	4.7	1320	0.4
5	4.5	1800	0.7
4	4.45	1860	0.75

Boundary value equation:

$$z = a \sin \left[ \frac{2\pi(t - \xi)}{T} \right] + M + 100$$

in which

- a = amplitude, t = time, in second.
- $\xi$  = phase lag, T = tidal period = 12.4 hr.
- M = mean water level, = 44640 sec.

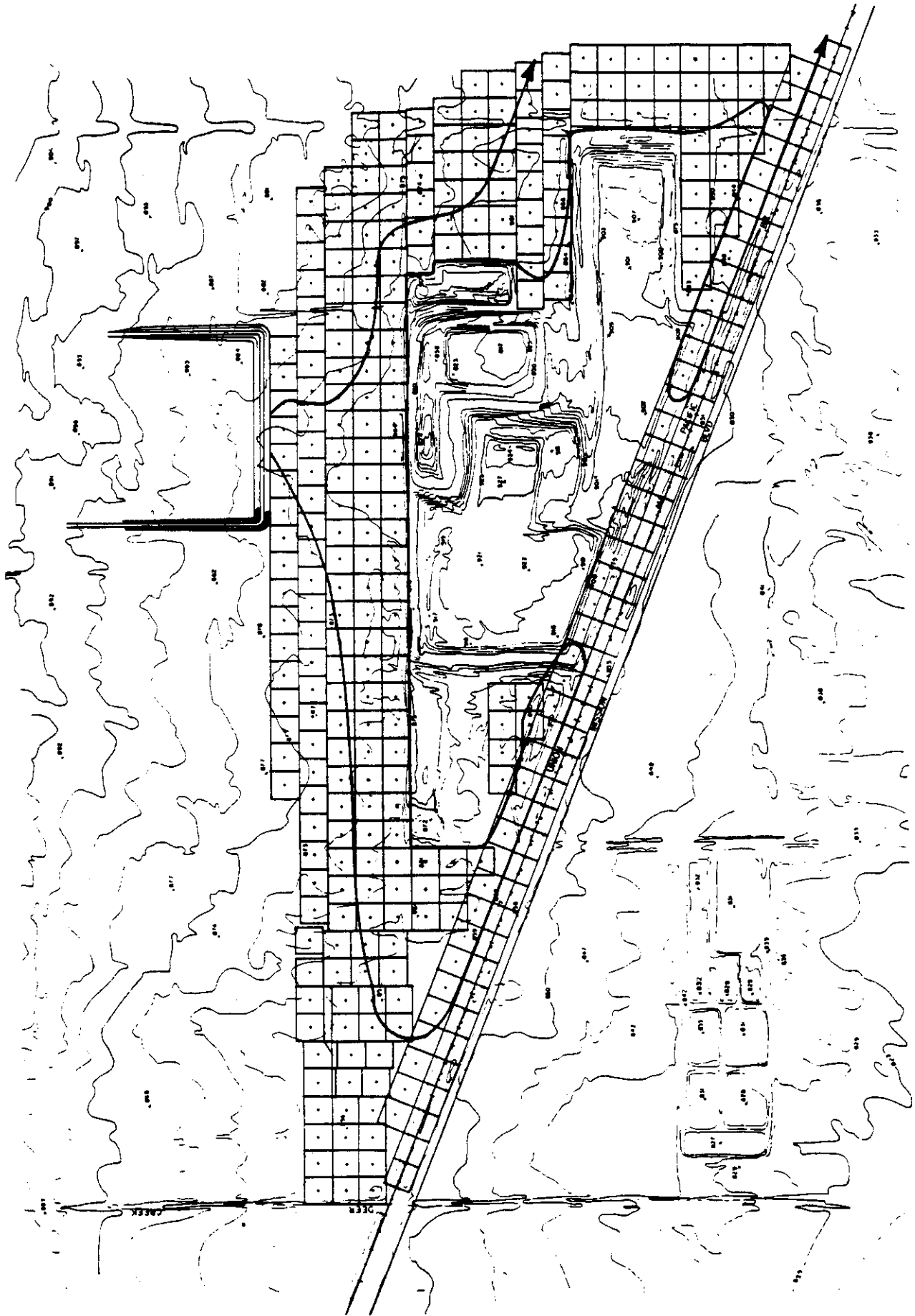


Fig. 43. Time (hours) of maximum flooding depth for O.I.P. Detention Basin



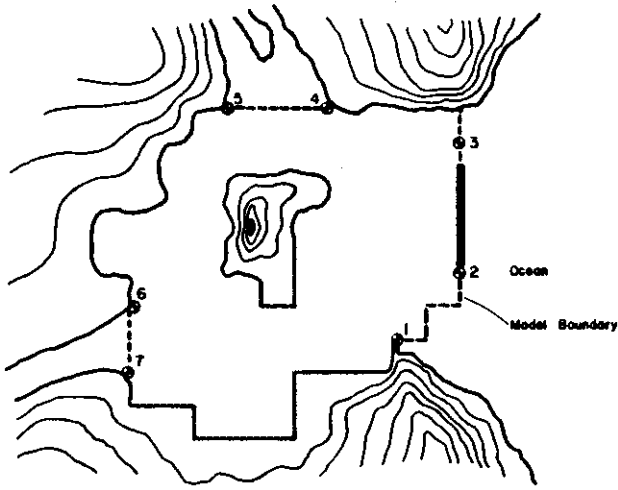


Fig. 45. A hypothetical bay

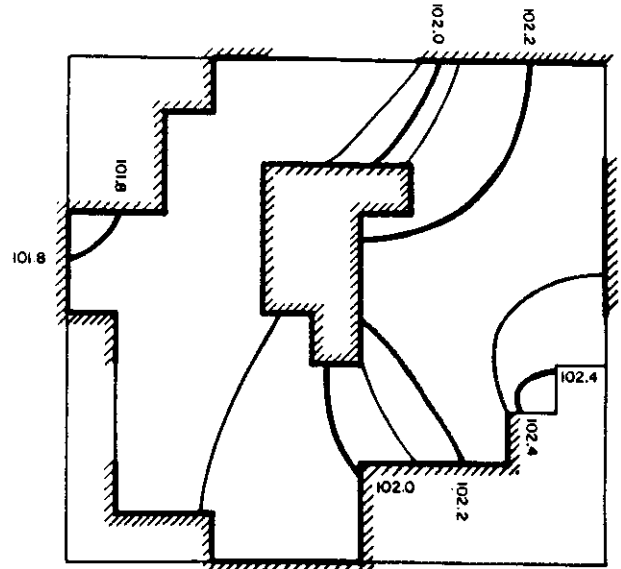


Fig. 47b. Mean water surface profile at 1-hour

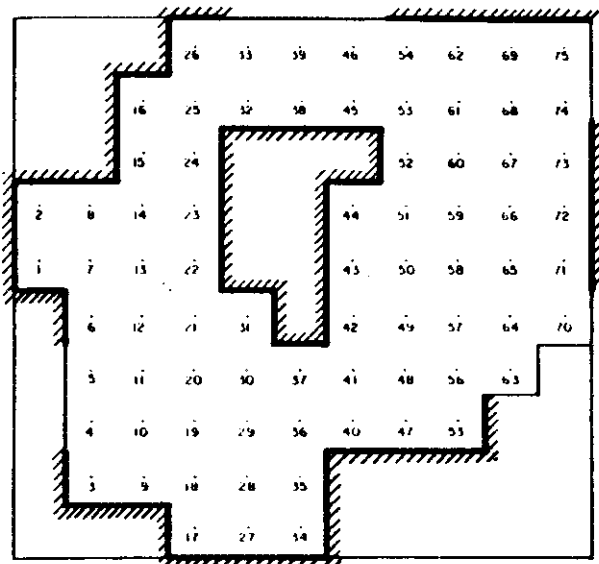


Fig. 46. The schematization of a hypothetical bay shown in Fig. 45

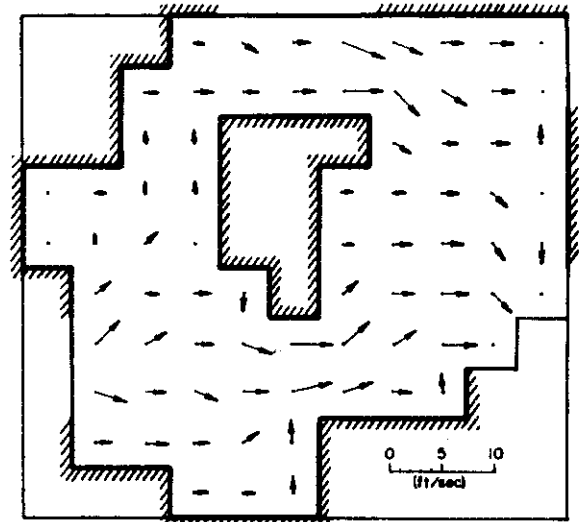


Fig. 48a. Mean velocity profile at 5-hours

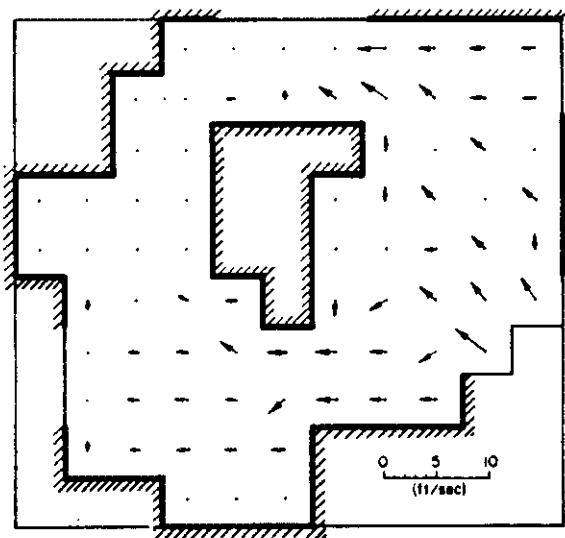


Fig. 47a. Mean velocity at 1-hour

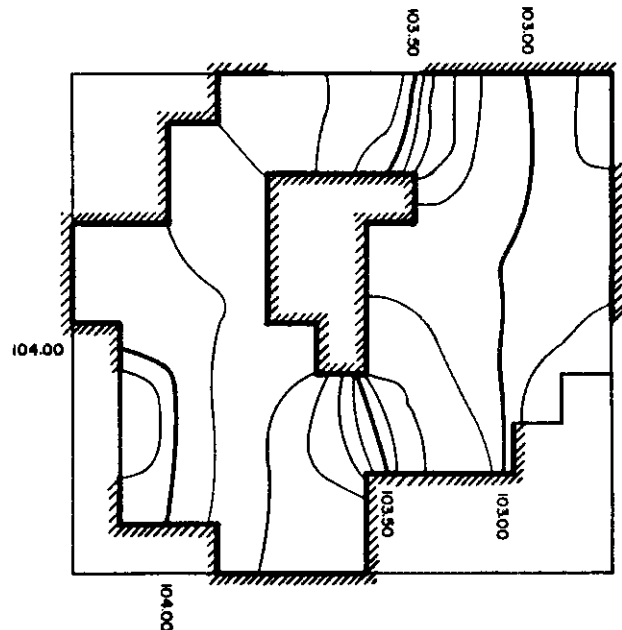


Fig. 48b. Mean water surface profile at 5-hours



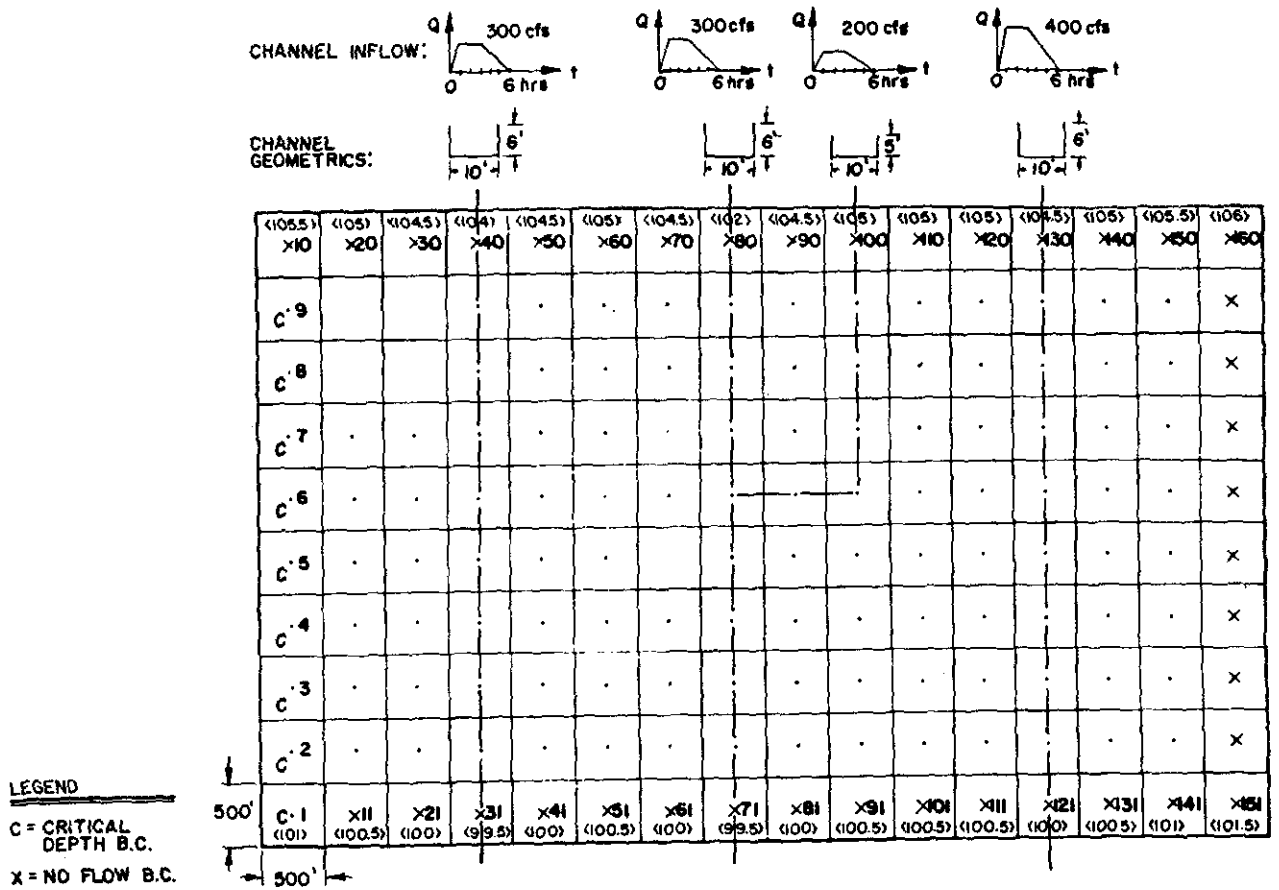


Fig. 51. Inflow and outflow boundary conditions for the hypothetical watershed model

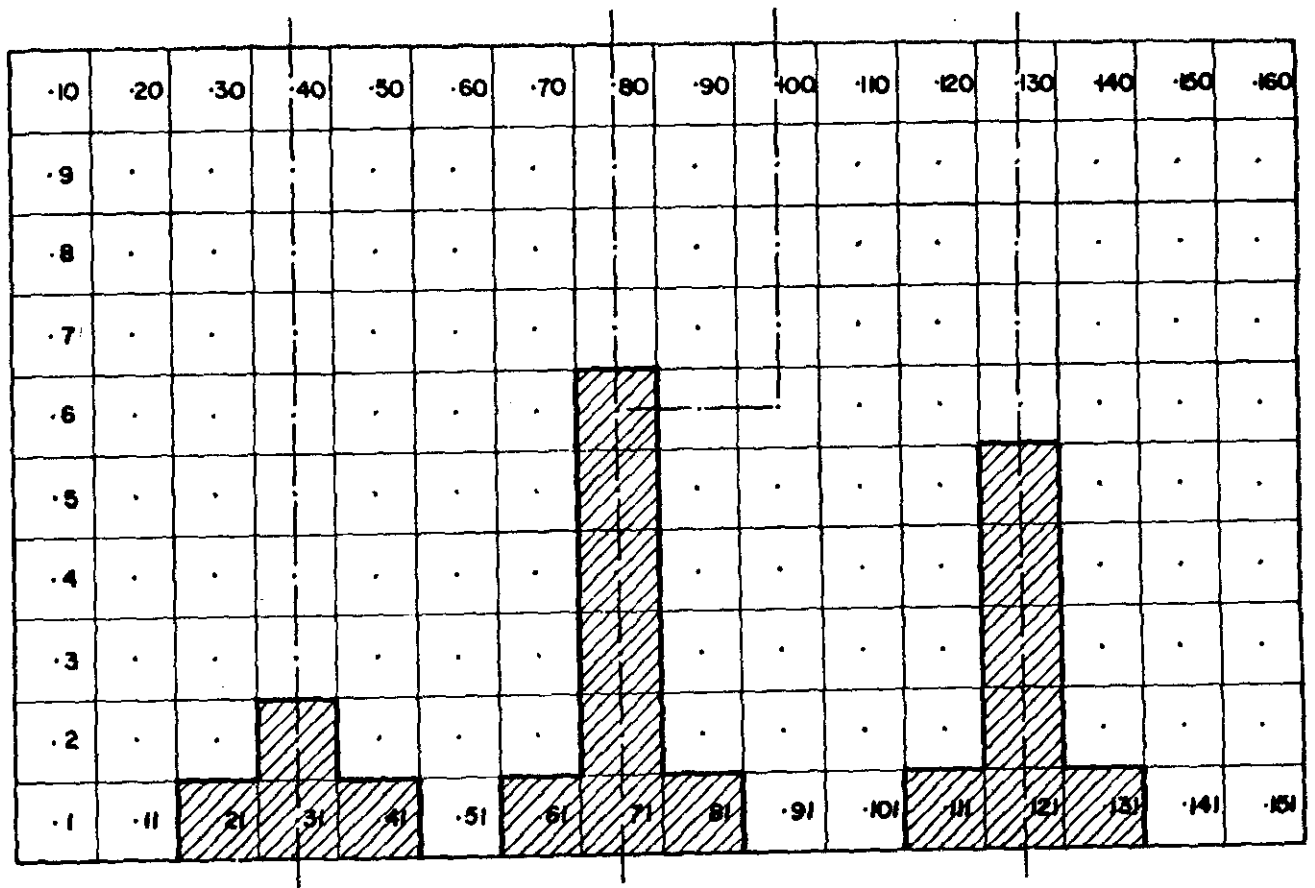


Fig. 52. DHM modelled floodplain at time = 1-hour



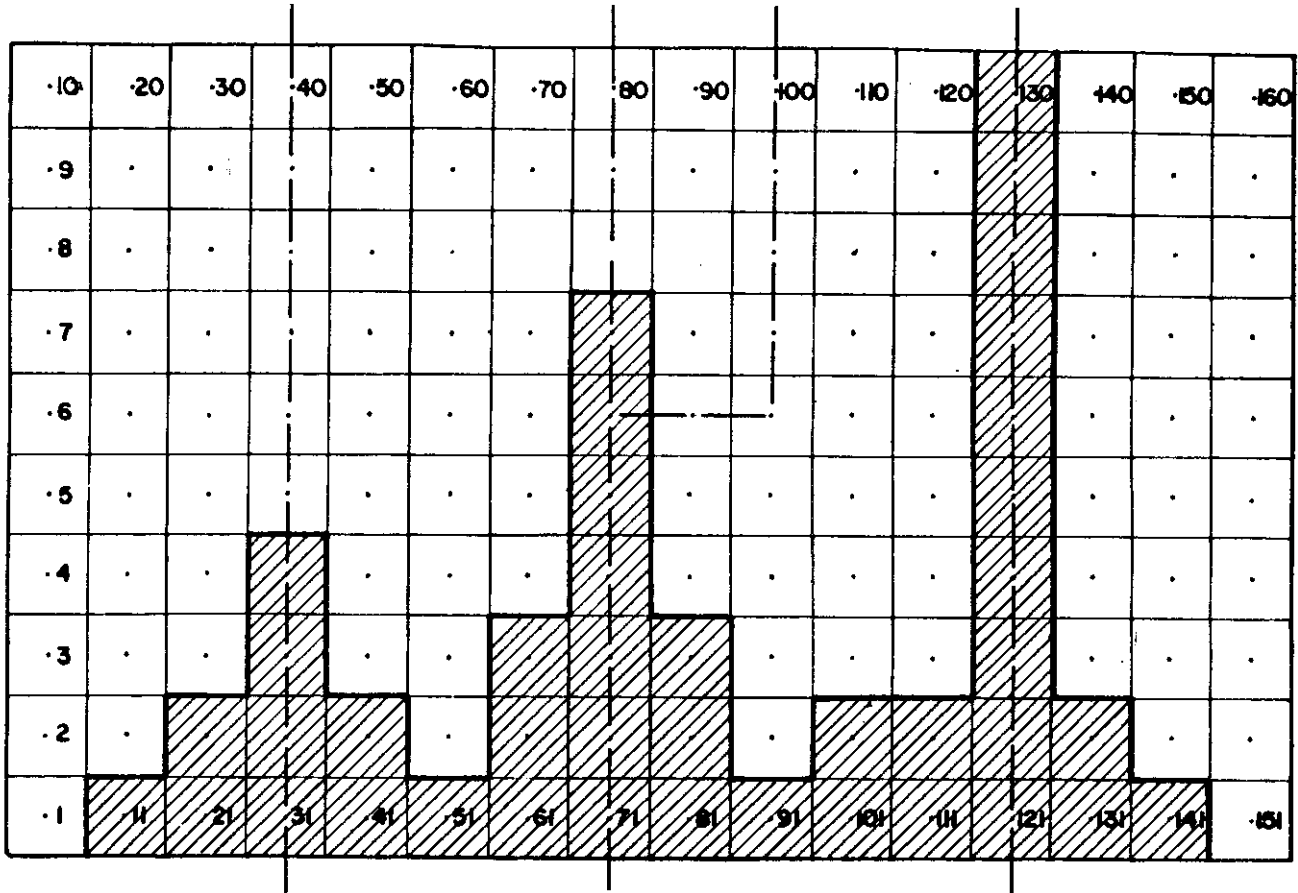


Fig. 53. DHM modelled floodplain at time = 2-hours

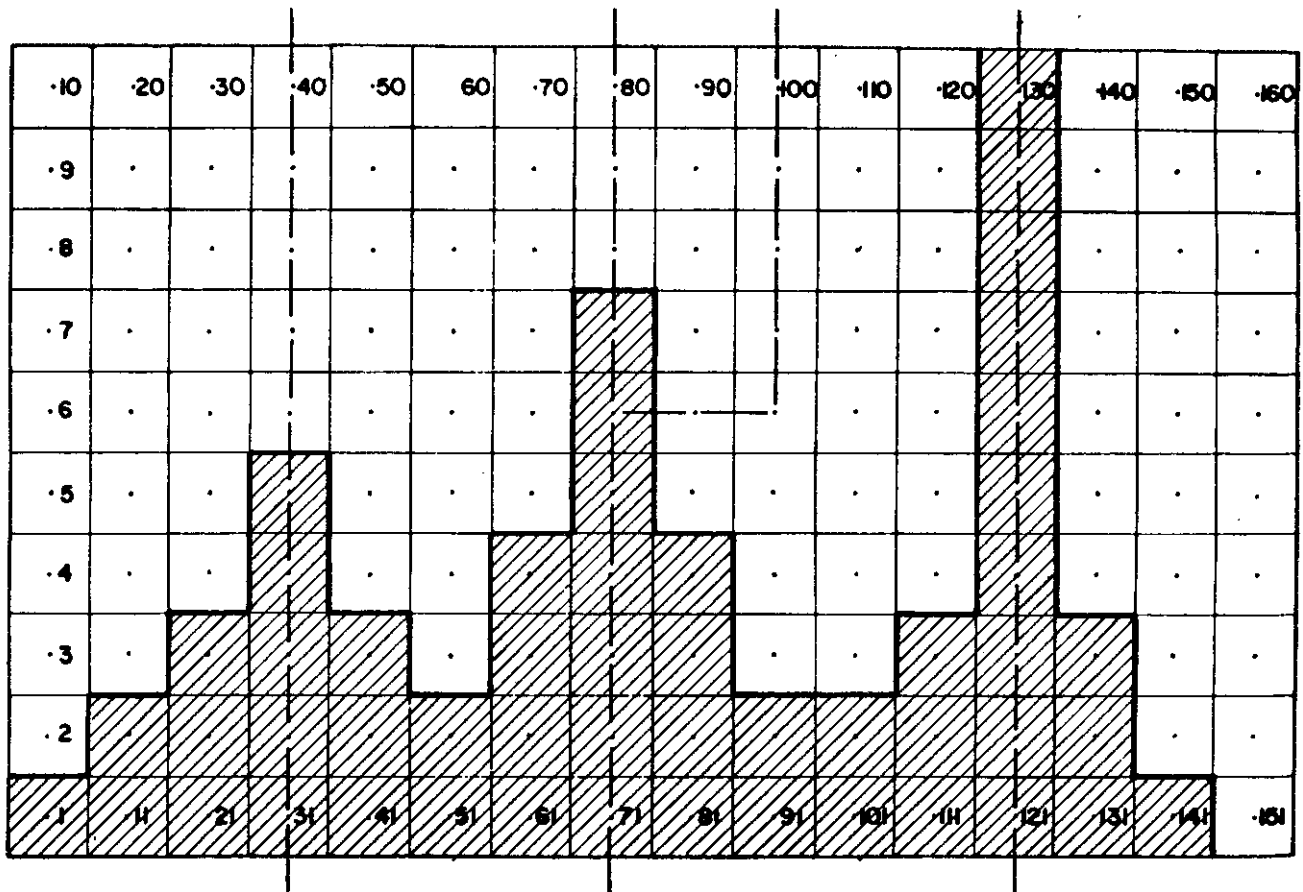


Fig. 54. DHM modelled floodplain at time = 3-hours

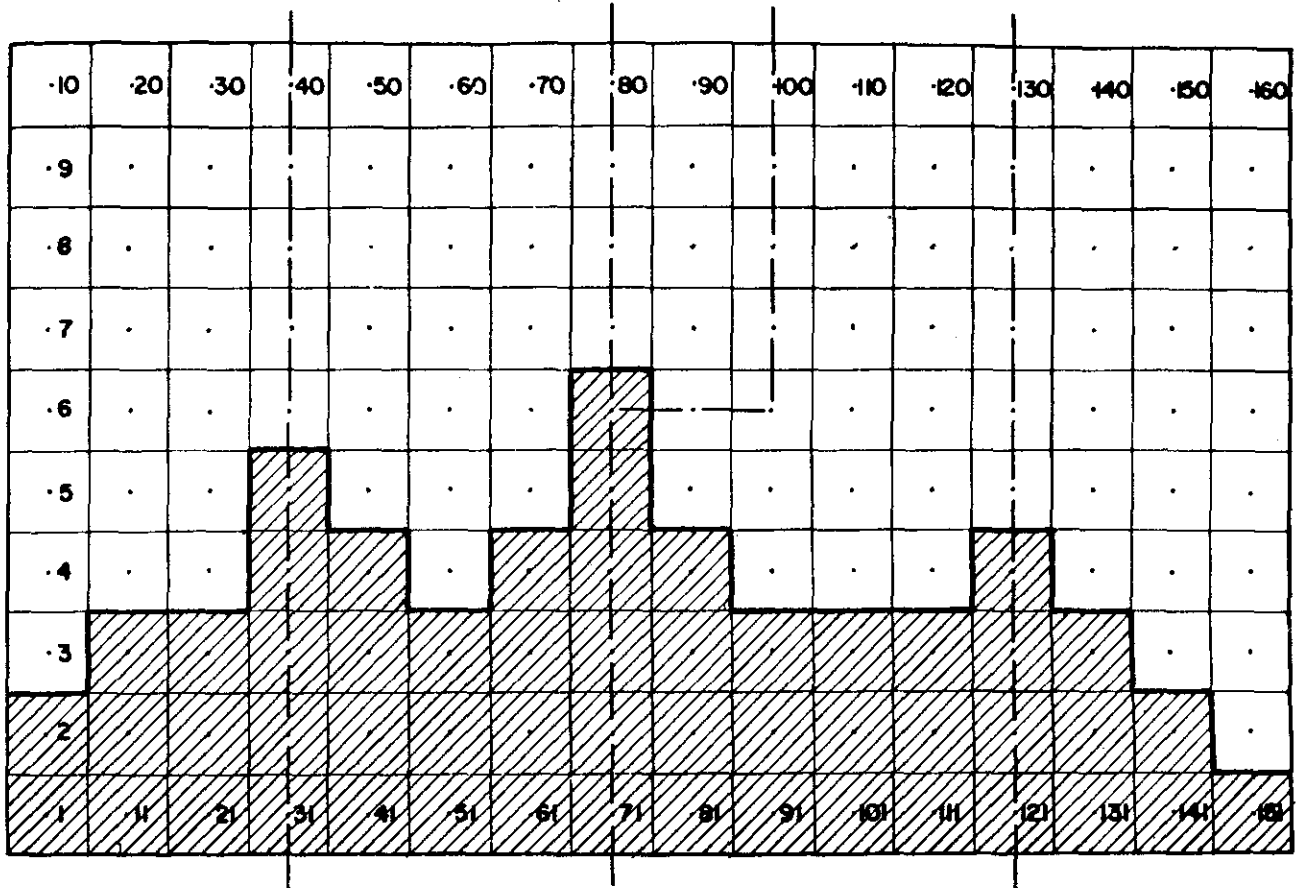


Fig. 55. DHM modelled floodplain at time = 5-hours

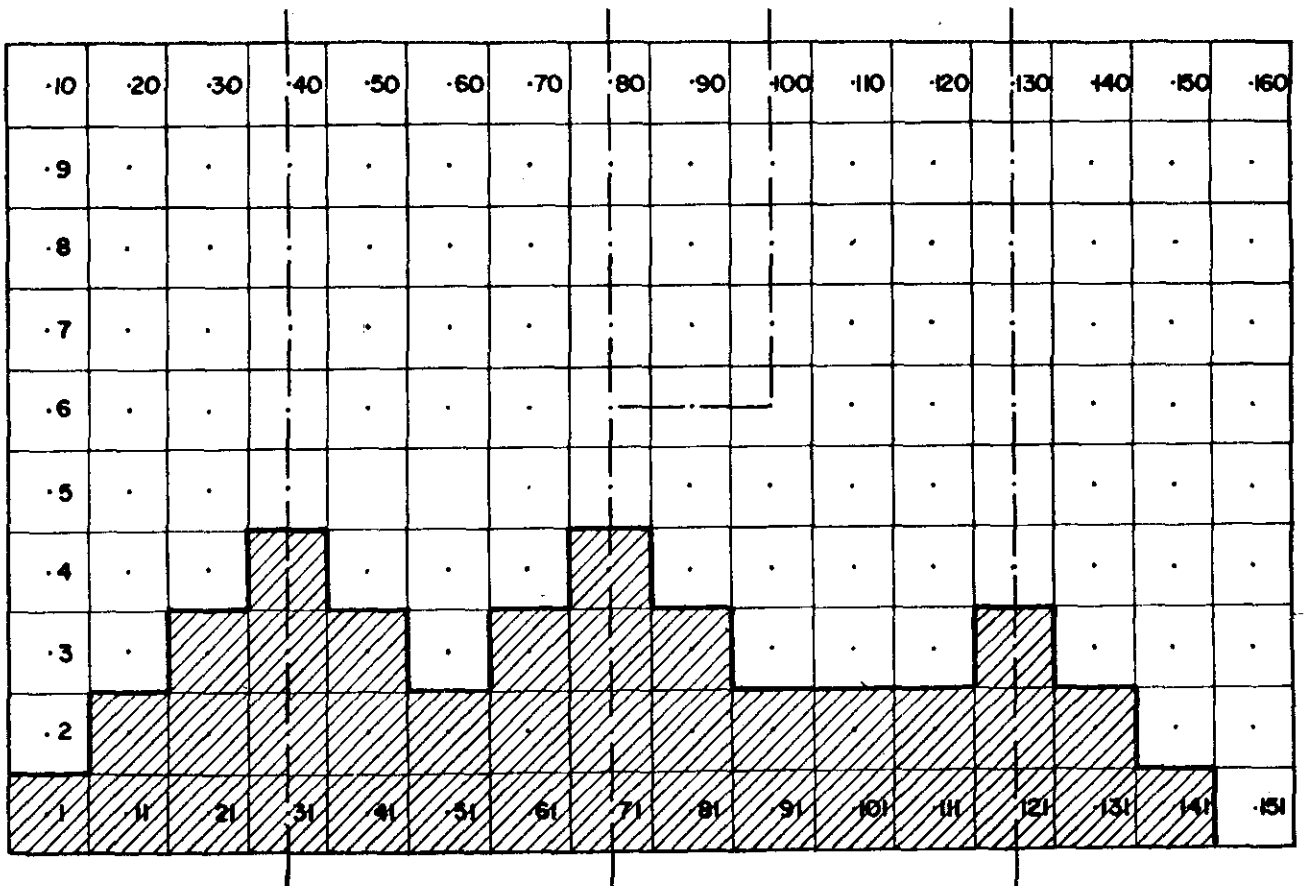


Fig. 56. DHM modelled floodplain at time = 7-hours

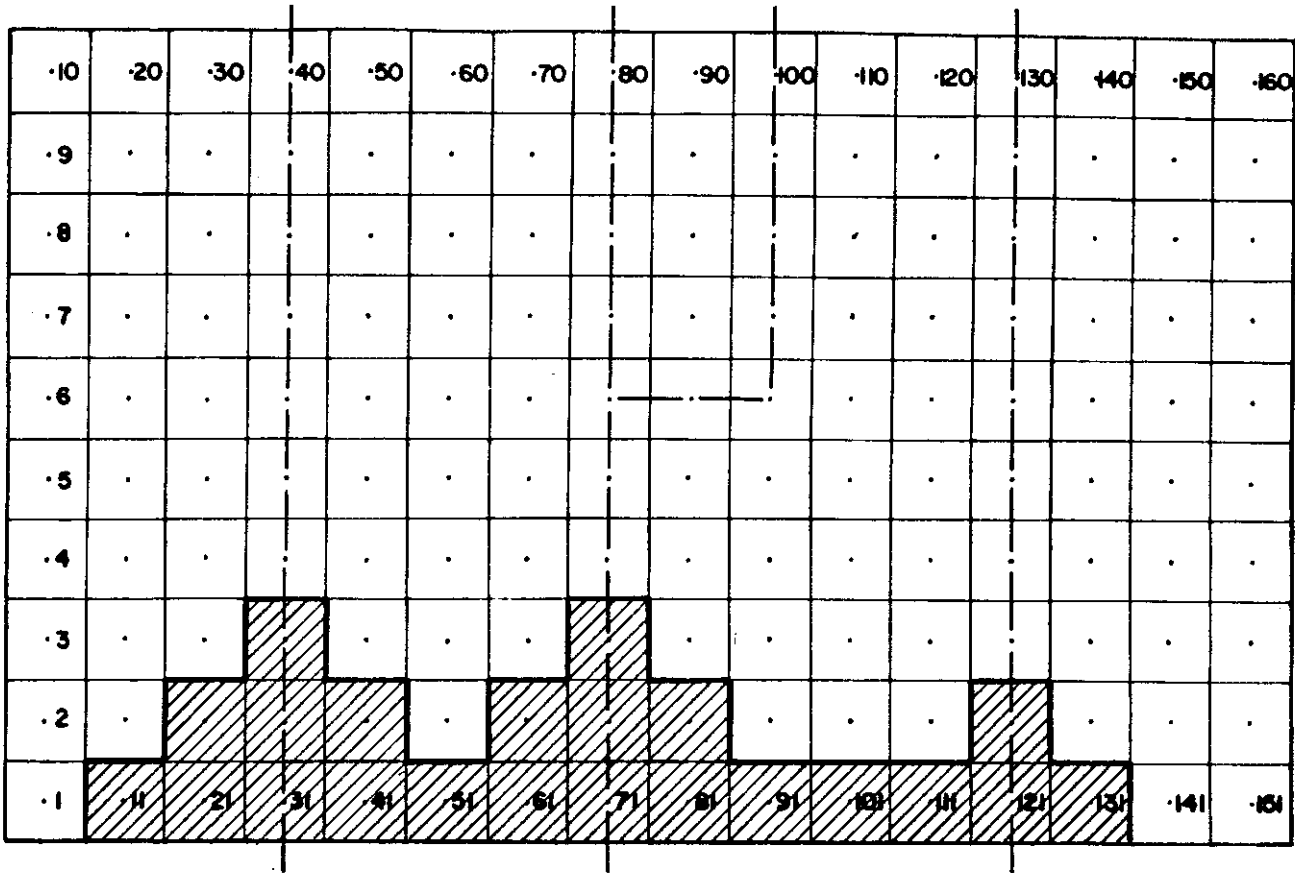


Fig. 57. DHM modelled floodplain at time = 10-hours

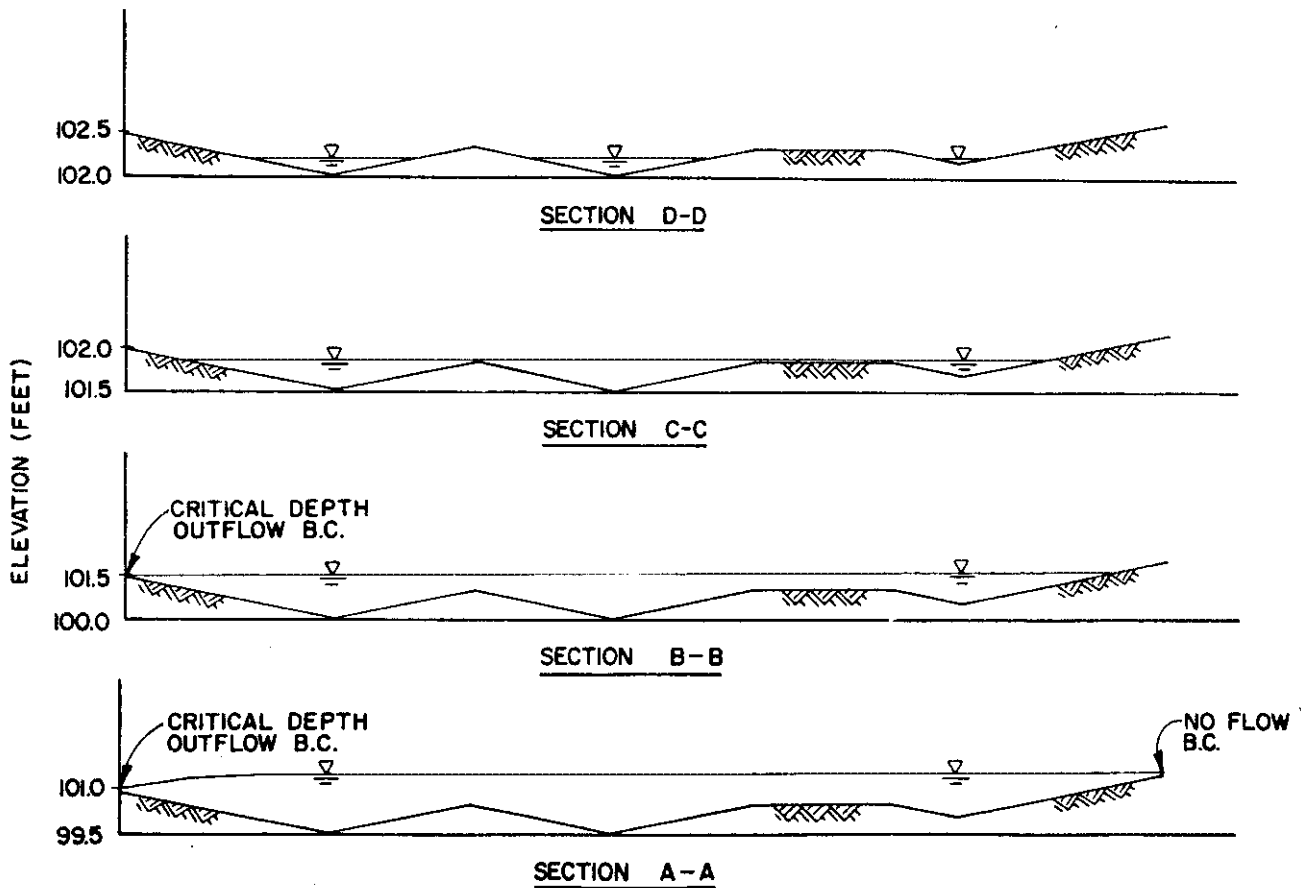
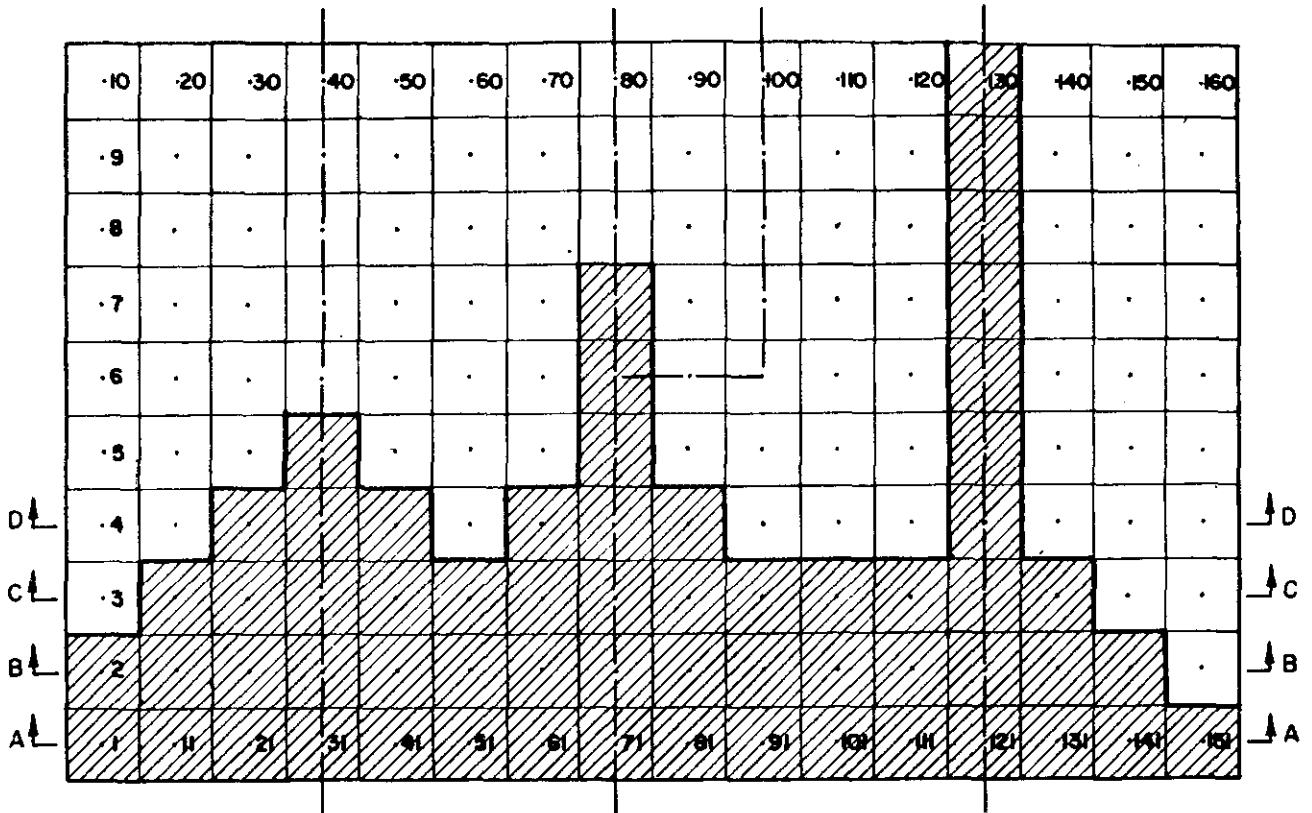


Fig. 58. Maximum water depth at different cross-sections



(a) Maximum Floodplain

Fig. 58.—cont.

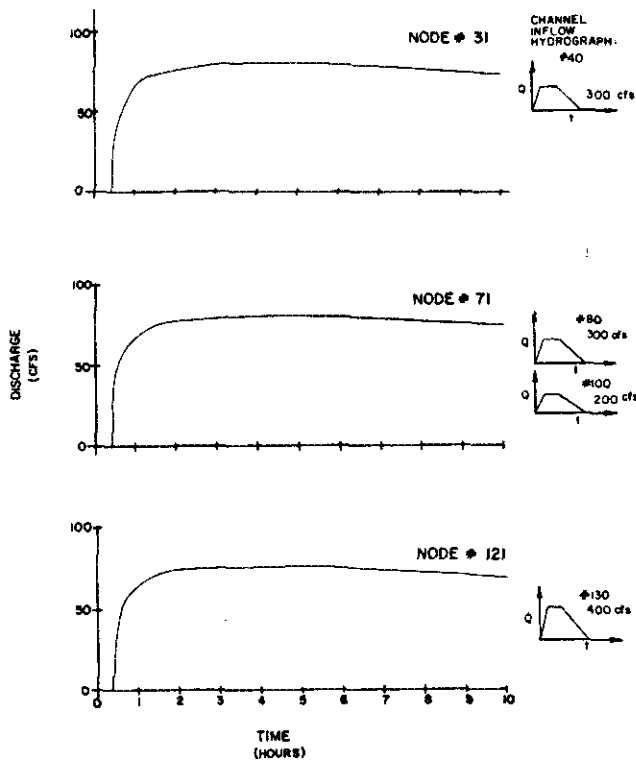


Fig. 59. Bridge flow hydrographs assumed outflow relation: ( $Q=10d$ )

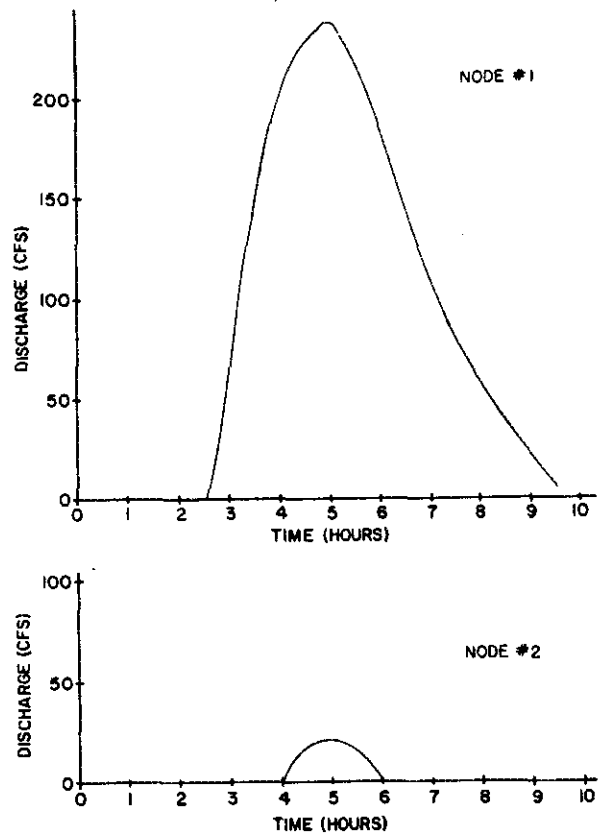


Fig. 60. Critical outflow hydrographs for flood plain

modelled in the open channels due to constrictions which, typically, are the source of flood system deficiencies.

In a recent report by Doyle *et al.* (1983), an examination of approximations of the one dimensional flow equation is presented. The authors write:

'The kinematic wave and diffusion wave approximations of the momentum equation provide simpler and faster computer solutions than the full dynamic equation and therefore are often used instead of the complete dynamic model. The choice of the approximations depends on which terms must be retained in the flow equation to accurately describe the stream system. Henderson (1966) gives the following values for terms of the momentum equation taken from a fast-rising flood for an actual river in steep alluvial country:

$$S_0, \quad \frac{\partial y}{\partial x}, \quad \frac{V \partial V}{g \partial x}, \quad \frac{1}{g} \frac{\partial V}{\partial t}$$

Feet per/mile 26,     1/2,   1/8 to 1/4,   1/20

These figures were computed for a flood in which the discharge increased from 10 000 ft<sup>3</sup>/s to 150 000 ft<sup>3</sup>/s and decreased again to 10 000 ft<sup>3</sup>/s within 24 hours. Even in this case, where the acceleration terms were comparatively large, they still are not as important as the bed slope term ( $S_0$ ). In some situations, however, the discharge and bed slope can determine the magnitude of the other terms. On very small slopes ( $S_0$  small) the pressure term might be the same order of magnitude as  $S_0$ . If the discharge rises fast, then all terms may be important (especially on flat to moderate slopes). Omitting even small terms (in these situations) from the equation can introduce errors into the solution.

It has been shown repeatedly in flow-routing applications that the kinematic wave approximation always predicts a steeper wave with less dispersion and attenuation than may actually occur. This can be traced to the approximations made in the development of the kinematic wave equations wherein the momentum equation is reduced to a uniform flow equation of motion that simply states the friction slope is equal to the bed slope. If the pressure term is retained in the momentum equation (diffusion wave method), then this will help to stop the accumulation of error that occurs when the kinematic wave approximation procedure is applied.'

### V.1 Results

The one-dimensional channel problem of Chapter 1 is used to compare the results between the DHM model and the kinematic routing. For the steep channel, both techniques show comparative results up to 10 miles for the maximum water depth (Fig. 61) and discharge rates at 5 and 10 miles (Figs 62 and 63). For the mild channel, the maximum water surface and discharge rates deviate more and more as the water flows downstream.

### V.2 Conclusions

The DHM can be reduced to use the kinematic routing approximation of the complete flow equations. The simplified model, however, loses the capability to approximate backwater effects, ponding, channel overflow, flow over adverse gradients, and other flow

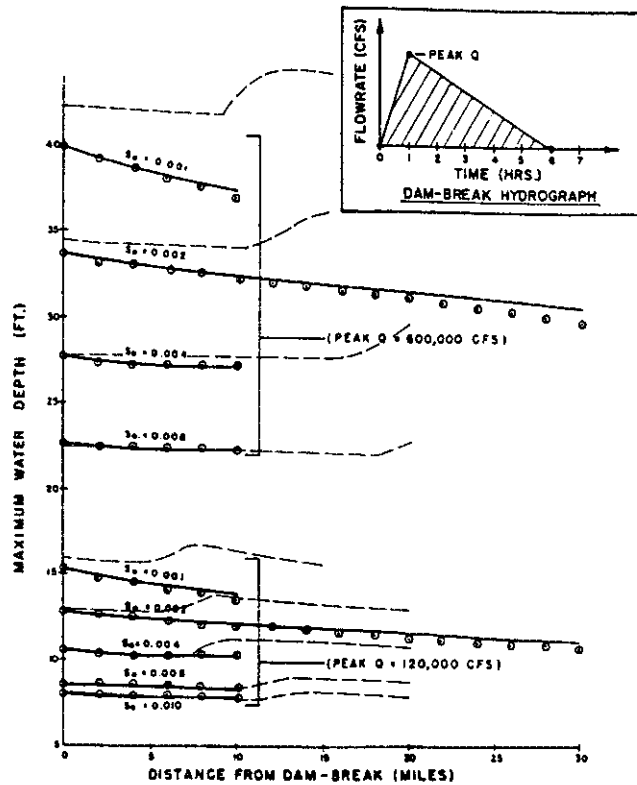


Fig. 61. Diffusion model (⊙), kinematic routing (dashed line) and K-634 model results (solid line) for 1000-foot width channel, Manning's  $n=0.040$ , and various channel slopes,  $S_0$

effects which are important in flood channel system deficiency analysis.

For one-dimensional unsteady flow channel routing problems where backwater effects are negligible, the provided comparisons between the diffusion and kinematic routing approximations show significant differences which may be important to watershed models based on the kinematic routing technique. Because the diffusion routing technique is simple to implement, and due to the additional hydraulic approximation effects provided by diffusion routing, it is recommended that all kinematic wave based hydrologic models be modernized by use of the diffusion routing technique.

## VI. CONCLUSIONS

A diffusion hydrodynamic model (DHM) is developed for use in civil engineering flood control studies. The DHM capabilities provides the practicing engineer with a flood control modelling capability not previously available, and only at a price of a home computer.

Although several applications are provided in the paper, further research is required as to the verification of predicted flooding depths, travel times, and other important hydraulic information.

## VII. ACKNOWLEDGEMENTS

Acknowledgements are paid to United States Geological Survey, Sacramento, California, for their time and computational assistance with several sections of this report.

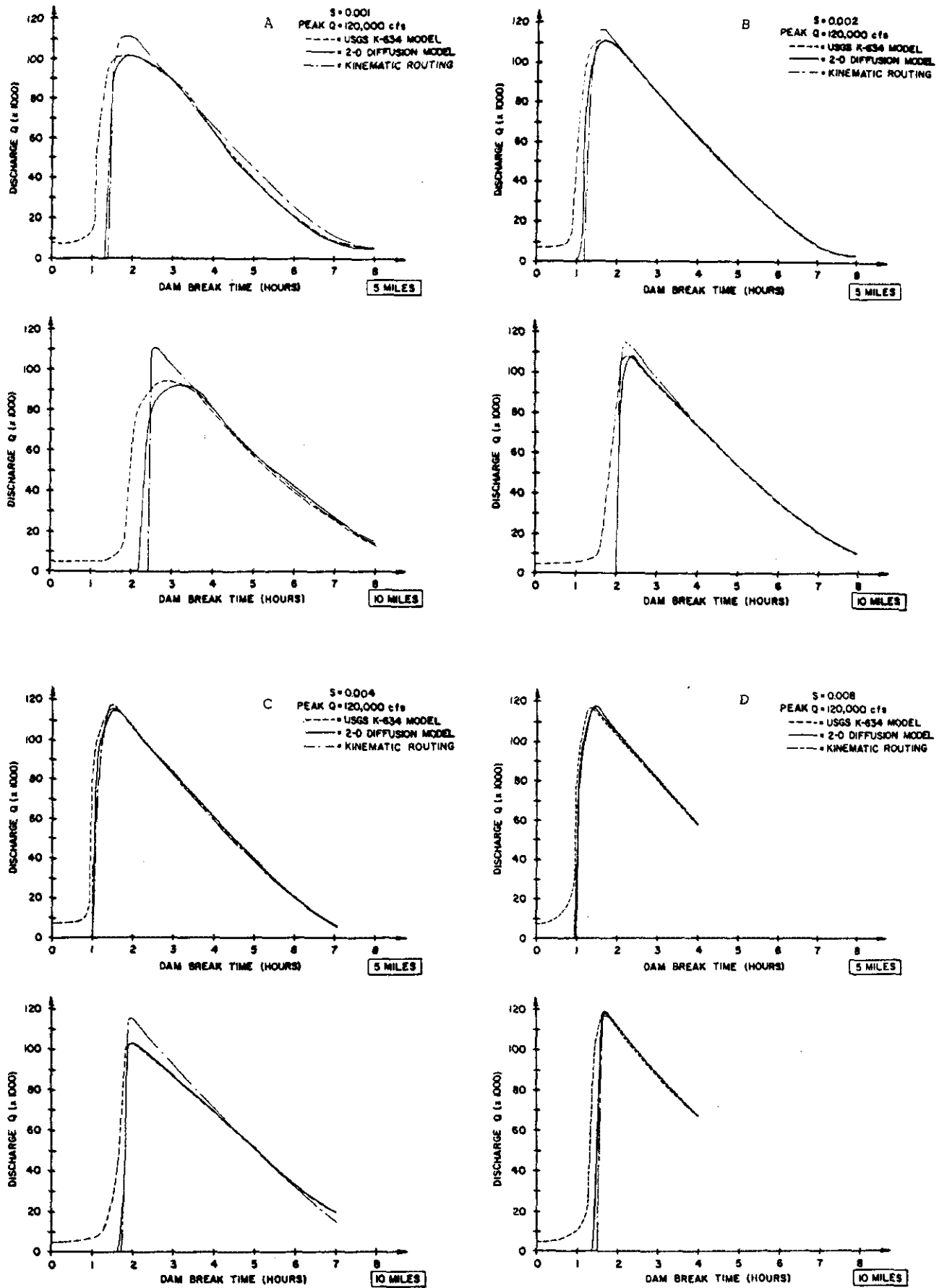


Fig. 62. Comparisons of outflow hydrographs at 5 and 10 miles downstream from the dam-break site

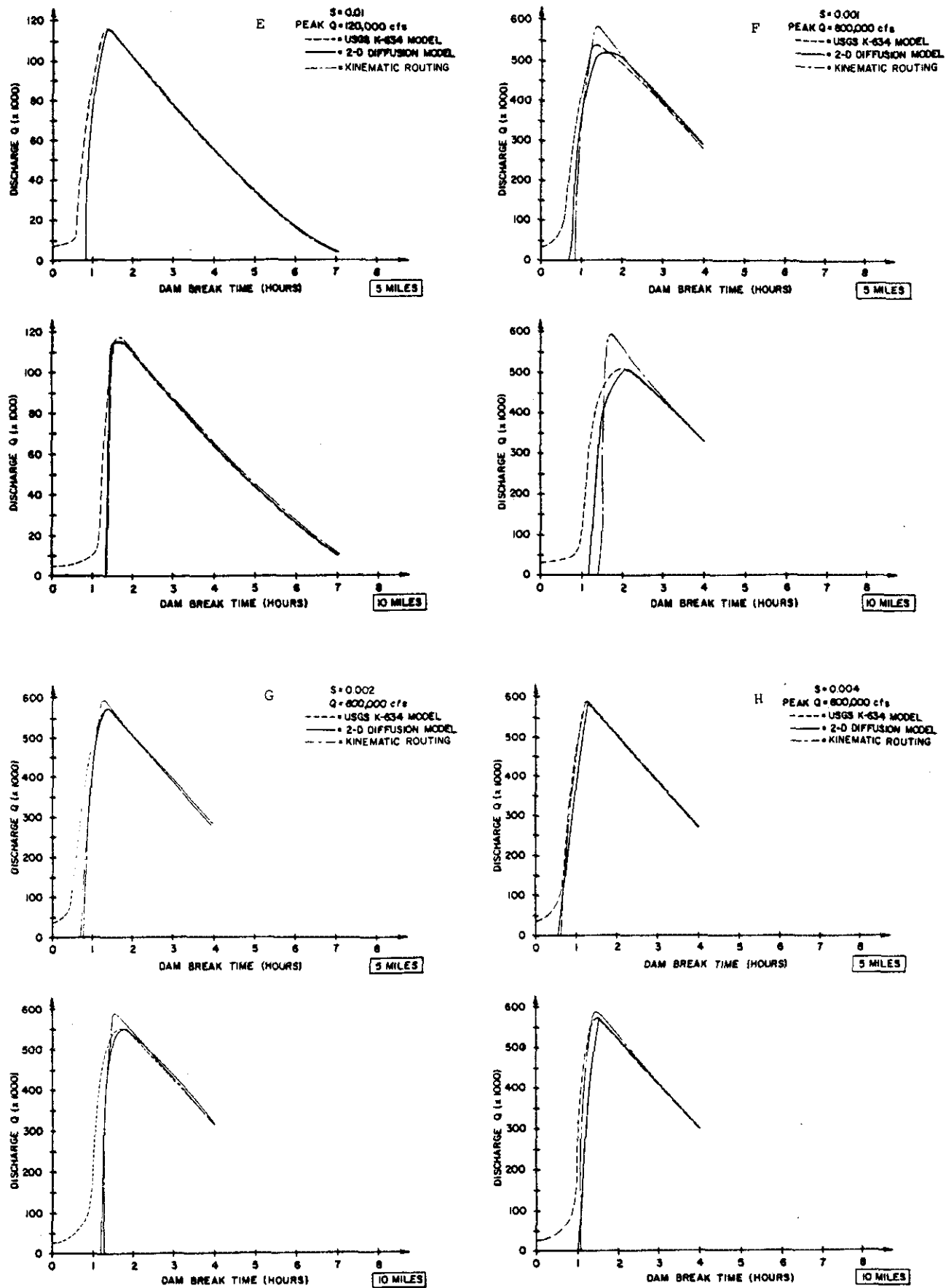


Fig. 62.—cont.

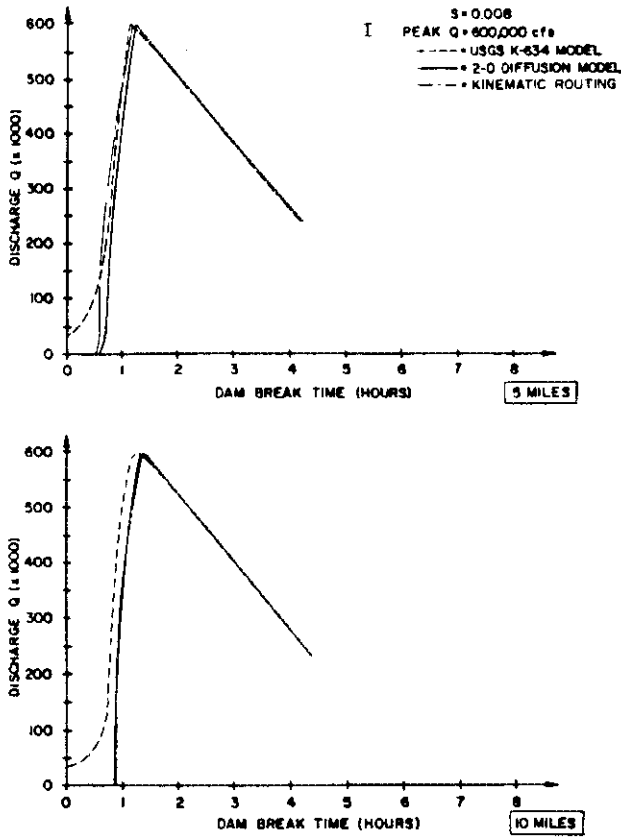


Fig. 62.—cont:

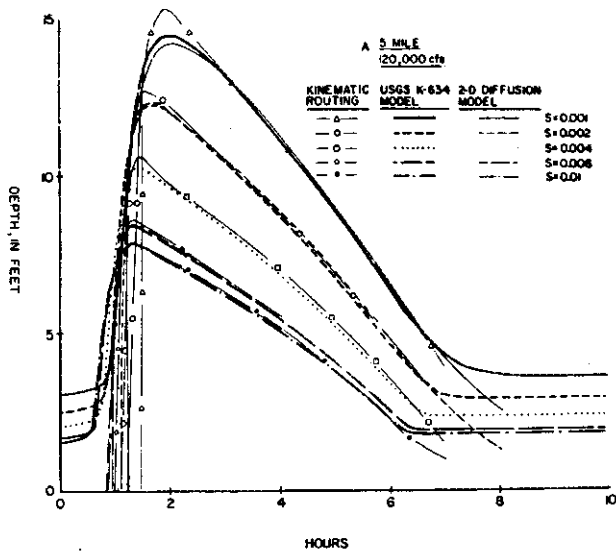
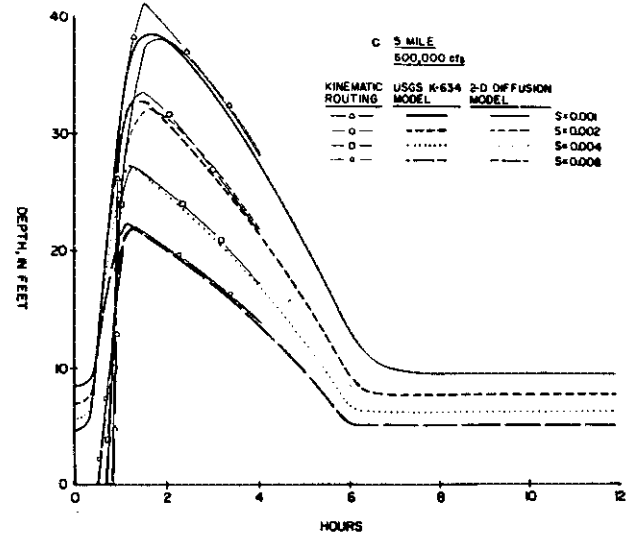
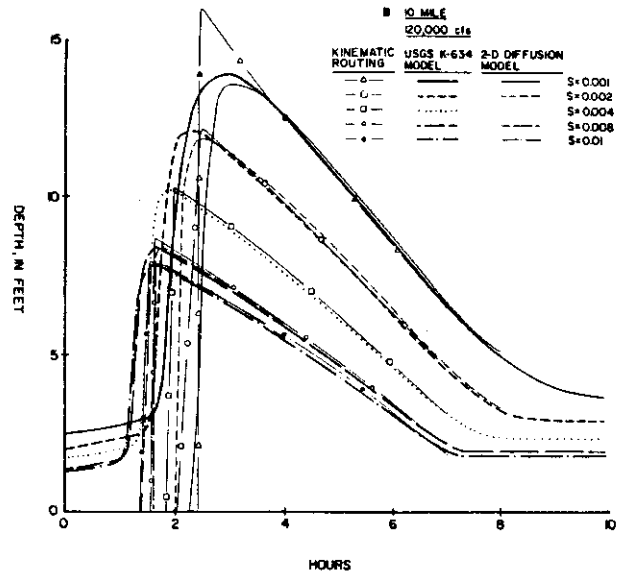


Fig. 63. Comparisons of depths of water at 5 and 10 miles downstream from the dam-break site

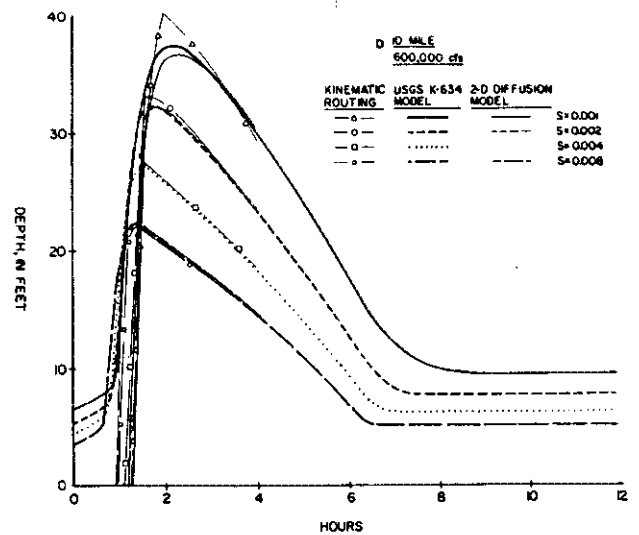


Fig. 63.—cont.





```

6      FORMAT(130(' '))
      WRITE(NWRITE,12)
12     FORMAT(//,10X,'NODAL POINT DATA ENTRY:',//,
C      7X,'*** FLOOD PLAIN INFORMATION ***',//,
C      10X,'NC = CENTRAL GRID NODE',//,
C      10X,'NN,NE,NS,NW = NORTH, EAST, SOUTH, WEST NODAL POINTS',//,
C      10X,'NBAR = NODAL POINT MANNINGS FRICTION FACTOR',//,
C      12X,'(NEGATIVE SIGN INDICATES A CHANNEL PASSING THROUGH)',//,
C      10X,'ELEV = NODAL POINT ELEVATION',//,
C      10X,'DEPTH = INITIAL WATER DEPTH AT NODE',//)
13     FORMAT(11X,' NC NN NE NS NW NBAR ELEV. DEPTH')
      WRITE(NWRITE,13) (I,(PP(I,J),J=1,7),I=1,NNOD)
15     FORMAT(10X,S14,1X,P6.4,2X,P6.1,1X,P5.1)
      WRITE(NWRITE,6)
      IF(NERI.LT.1)GOTO 60
      WRITE(NWRITE,22)NERI
22     FORMAT(//,10X,'NUMBER OF EFFECTIVE RAINFALL INTENSITY ',
C      'ENTRIES = ',I2,/,4X,'LINEAR FUNCTION IN EFFECTIVE RAINFALL',
C      ' INTENSITY (IN/HR) ON WATERSHED:',//,10X,'(HOOR INTENSITY)')
      WRITE(NWRITE,23)((R(I,J),J=1,2),I=1,NERI)
23     FORMAT(8X,P6.2,4X,P6.2)
      WRITE(NWRITE,6)
60     IF(NPFI.LT.1)GOTO 62
      DO 64 I=1,NPFI
      WRITE(NWRITE,117)KINP(I)
      DO 64 J=1,NPFI
      WRITE(NWRITE,18)HP(I,J,1),HP(I,J,2)
64     CONTINUE
      WRITE(NWRITE,6)
62     IF(NDC.LT.1)GOTO 61
      WRITE(NWRITE,21)NDC
21     FORMAT(//,10X,'NUMBER OF CRITICAL-DEPTH OUTFLOW NODES = ',I4,/,
C      10X,'CRITICAL-DEPTH OUTFLOW NODE NUMBERS:')
      WRITE(NWRITE,38)(NODC(I),I=1,NDC)
38     FORMAT(10X,I3,1X,I3)
7      FORMAT(130(' '))
      WRITE(NWRITE,6)
61     IF(NODC.LT.1)GOTO 260
      WRITE(NWRITE,17)
17     FORMAT(//,7X,'***CHANNEL INFORMATION***',//,
C      10X,'NODE NBAR WIDTH DEPTH BOTTOM INITIAL DEPTH')
      WRITE(NWRITE,16) (I,(FC(I,J),J=1,5),I=1,NNOD)
16     FORMAT(10X,I3,1X,P5.4,1X,P7.1,1X,P7.1,1X,P7.1,5X,P7.1)
      WRITE(NWRITE,6)
      IF(NCHI.LT.1)GOTO 116
      DO 51 I=1,NCHI
      WRITE(NWRITE,117)KIN(I)
      DO 51 J=1,NPCHI
      WRITE(NWRITE,18)H(I,J,1),H(I,J,2)
51     CONTINUE
117    FORMAT(//,10X,'INFLOW HYDROGRAPH AT NODE #',I3,/,
C      12X,'(HOUR CFS)')
18     FORMAT(10X,P5.1,4X,P7.0)
      WRITE(NWRITE,6)
116    IF(NCHO.LT.1)GOTO 119
      WRITE(NWRITE,19)
19     FORMAT(10X,'OUTFLOW IS APPROXIMATED AS THE FOLLOWING EQUATION:',
C      '/,12X,'QOUT = ALPHA*(DEPTH)**BETA')
      DO 250 I=1,NCHO
      WRITE(NWRITE,20)HOUT(I)
      FORMAT(10X,'OUTFLOW NODE # ',I3,
C      '/,9X,'(DEPTH LESS THAN',
C      '/,9X,' OR EQUAL TO ALPHA BETA')
      DO 250 J=1,NPCHO
      WRITE(NWRITE,25)HOUT(I,J,1),HOUT(I,J,2),HOUT(I,J,3)
25     FORMAT(15X,P4.1,6X,P7.3,1X,P7.3)
      CONTINUE
      WRITE(NWRITE,6)
119    IF(NSTA.LT.1)GOTO 260
      DO 52 I=1,NSTA
      WRITE(NWRITE,118)NSTA(I)
      DO 52 J=1,NPSTA
      WRITE(NWRITE,39)STA(I,J,1),STA(I,J,2)
39     FORMAT(10X,P5.1,4X,P7.3)
52     CONTINUE
118    FORMAT(//,10X,'STAGE CURVE AT NODE #',I3,/,
C      12X,'(HOUR FEET)')
      WRITE(NWRITE,6)
260    CONTINUE
C
C
C      MAIN PROGRAM
C
C..... INITIALIZE CONSTANTS
      DSEC=DTMIN
      DT=DTMIN/3600.
      DTOLP=DTOLP*.01
      CHECKD=DTOL/DTOLP
      TTIME=0.
      QBC=0.
      QTEMP=0.
      KK=0
      TTOUT=TOUT
      TTFOOT=TFOUT
      KIT=0
      TIME=0.
      DO 40 J=1,NNOD
      DHAX(J,1)=0.
      TIMEX(J,1)=0.
      DHAX(J,2)=0.
      TIMEX(J,2)=0.
      FP(J,8)=0.
40     CONTINUE
C
C..... MAIN LOOP FOR MODEL
C
10000  KROUT=0
      THIN=99.
      THX=-99.
      TMEAN=0.
C
C..... FLOODPLAIN MODEL
C
2100   KFLUX=0
      IKODE=0
      TIME=TIME+DT
      FPMAX=0.
      FCMAX=0.
      IJR=0
      IF(NODC.NE.0 .AND. ITER.EQ.0)TTIME=DSEC
      IF(ITER.EQ.0 .AND. NODC.NE.0)GO TO 7777
C..... UPDATE TIME AND BOUNDARY CONDITION VALUES
      IF(NPFI.LT.1)GOTO 711
      DO 695 J=1,NPFI
      DO 710 I=2,NPFI
      IF(TIME.GT.HP(J,I,1))GOTO 710
      QTEMP=HP(J,I-1,2)*(HP(J,I,2)-HP(J,I-1,2))*(TIME-HP(J,I-1,1))
C      (HP(J,I,1)-HP(J,I-1,1))
710    GO TO 730
      CONTINUE
700    QTEMP=HP(J,I,2)
730    QBC=QTEMP/SIDE
      IF(QBC.LT.0.)QBC=0.
      JJ=KINP(J)
      FP(J,8)=FP(J,8)+QBC
695    CONTINUE
C..... INCLUDE THE EFFECTIVE RAINFALL ON THE WATERSHED
711    IF(NERI.LT.1)GOTO 555
      DO 680 J=2,NERI
      IF(TIME.GT.R(J,1))GOTO 680
      RRATE=R(J-1,2)+(R(J,2)-R(J-1,2))*(TIME-R(J-1,1))/
      (R(J,1)-R(J-1,1))
C      GO TO 660
      CONTINUE
680    QRAIN=RRATE*SIDE*SIDE/(12.*3600.)
      DO 697 J=1,NNOD
      FP(J,8)=FP(J,8)+QRAIN/SIDE
      CONTINUE
697    IF(NFLUX.EQ.0)GOTO 560
555    IF(TIME.LT.TTFOOT)GOTO 560
      TTFOOT=TTFOOT+TTFOOT
      IF(ITER.EQ.0 .AND. NODC.NE.0)GO TO 560
      WRITE(NWRITE,6)
      WRITE(NWRITE,1300)TIME
      IF(RRATE.NE.0.)WRITE(NWRITE,1301)RRATE
      IJK=1
      WRITE(NWRITE,213)
213    FORMAT(//,5X,'AVERAGE FLOW RATE FOR SPECIFIED FLOOD PLAIN ',
C      'NODES :',//,10X,'(NODE',5X,'(QN',9X,'(QE',9X,'(QS',9X,'(QW')')
560    CONTINUE
C..... CALCULATE FLOW VELOCITIES AND FLOWRATES
      DO 1000 I=1,NNOD
      DO 690 II=1,4
      QQ=0.
      NQ=FP(I,II)
      IF(NQ.EQ.0.)GOTO 690
      CALL QFP(I,NQ,SIDE,QQ,II,VV,TOL,RMODEL)
      IF(II.EQ.1)GOTO 9999
690    Q(II)=QQ
C..... ADJUST FLOWRATES FOR DIRECTION
      Q(3)=-Q(3)
      Q(4)=-Q(4)
C..... ESTIMATE ACCUMULATION OF INFLOW
      QNET=Q(3)+Q(4)-Q(1)-Q(2)
      IF(NFLUX.EQ.0)GOTO 1010
      IF(IJK.NE.1)GOTO 1010
      QN=Q(1)*SIDE
      QE=Q(2)*SIDE
      QS=Q(3)*SIDE
      QW=Q(4)*SIDE
      DO 540 J=1,NFLUX
      IF(I.EQ.NODFX(J))WRITE(NWRITE,24)I,QN,QE,QS,QW
24     FORMAT(10X,I4,(2X,E9.3))
540    CONTINUE
1010   FP(I,8)=QNET+FP(I,8)
1000   CONTINUE
C..... ACCOUNT FOR CRITICAL-DEPTH OUTFLOW NODES
      IF(NDC.LT.1)GOTO 1201
      DO 1100 J=1,NDC
      JJ=MODDC(J)
      QOUT=5.67*(FP(JJ,7)**0.5)*(FP(JJ,7)-TOL)
      IF(FP(JJ,7).LT.TOL)QOUT=0.
      FP(JJ,8)=FP(JJ,8)-QOUT
1100   CONTINUE
C..... UPDATE CHANGE OF WATER DEPTH
1201   DO 1200 J=1,NNOD
      FP(J,8)=FP(J,8)+DSEC/SIDE
      TEMP=ABS(FP(J,8))
      IF(TEMP.LT.DTOL)GOTO 1200
      IF(FP(J,7).LT.CHECKD)FPMAX=99.
      IF(FP(J,7).LT.CHECKD)GOTO 1231
      TOLP=TEMP/FP(J,7)
      IF(TOLP.GE.DTOLP)FPMAX=99.
      IF(TOLP.GE.DTOLP)GOTO 1231
1200   CONTINUE
C..... CALCULATE THE EFFLUX VELOCITIES
      DO 691 J=1,NNOD
      IF(KODE.NE.1)GOTO 692
      DO 691 II=1,4
      QQ=0.
      NQ=FP(J,II)
      IF(NQ.EQ.0.)GOTO 691
      CALL QFP(J,NQ,SIDE,QQ,II,VV,TOL,RMODEL)
      VEL(J,II)=VV
691    CONTINUE
C..... CHECK INTERFACE MODEL UPDATE REQUEST
692    IF(IKODE.EQ.0)KIT=KIT+1
      IF(IKODE.EQ.0)TTIME=TTIME+DSEC
      IF(KIT.NE.ITER)GOTO 1231
      IF(NODC.LT.1)GOTO 1231
C
C..... UPDATE WATER DEPTH FOR CHANNEL
C
7777   CALL FLOODC(TIME,TTIME,NNOD,NCHI,NCHO,NPCHI,NPCHO,SIDE,NSTA,
C      NPSTA,NODC,TOL,DTOL,DTOLP,NFLUX,KFLUX,ITER,FCMAX,NWRITE,CHECKD,
C      RMODEL)
C..... UPDATE NEW TIMESTEP SIZE
1231   DD=AMAX1(FPMAX,FCMAX)
      IF(DD.GT.0.)DSECP=DSEC-DTD
      IF(DD.LE.0.)DSECP=DSEC+DTI
      IF(DSECP.LT.DTMIN)DSECP=DTMIN
      IF(DSECP.GT.DTMAX)DSECP=DTMAX
      DTD=DSECP/3600.
      IF(DD.LE.DTOL)GOTO 1255
      IF(DSEC.EQ.DTMIN)IKODE=1+IKODE
      IF(DSEC.NE.DTMIN)IKODE=1
      IF(IKODE.GE.3)GOTO 999
      TIME=TIME-DT+DTT
      TTIME=TTIME-DSEC+DSECP
      DO 1257 J=1,NNOD
      FP(J,8)=0.
1257   CONTINUE
      DT=DTD
      DSEC=DSECP
      GO TO 2110
C..... UPDATE DEPTH OF WATER
1255   DO 1259 J=1,NNOD
      FP(J,7)=FP(J,7)+FP(J,8)
      IF(FP(J,7).LT.0.)FP(J,7)=0.
      FP(J,8)=0.
      IF(NODC.LT.1)GOTO 1259
      FC(J,5)=FC(J,5)+PC(J,6)
      IF(FC(J,5).LT.0.)FC(J,5)=0.
      FC(J,6)=0.
1259   CONTINUE
      IF(DSEC.GT.TMAX)TMAX=DSEC

```

```

IF (DSEC.LT.TMIN) TMIN=DSEC
C..... INTERFACE BETWEEN FLOOD PLAIN AND CHANNEL DEPTHS
IF (KIT.NE.ITER) GOTO 1239
IF (NODC.LT.1) GOTO 1239
IF (ITER.NE.0) CALL CHANPL (NNOD, SIDE, TOL)
TIME=0.
KIT=0.
C..... CHECK OUTPUT REQUEST
1239 IF (TIME.LT.TTOUT) GOTO 1252
C..... USE FC(I,6) AND FP(I,8) TO STORE WATER SURFACE ELEVATIONS
DO 1253 J=1, NNOD
IF (NODC.LT.1) GOTO 1254
FC(J,6) = FC(J,5) + FC(J,4)
IF (ITER.EQ.0) GOTO 1253
FP(J,8) = FP(J,7) + FP(J,6)
1253 CONTINUE
C..... UPDATE MAXIMUM WATER SURFACE VALUES
1252 DO 1230 J=1, NNOD
TEMP = FP(J,7)
TEST = DMAX(J,1)
IF (TEMP.LT.TEST) GOTO 1230
DMAX(J,1) = TEMP
TIMEX(J,1) = TIME
CONTINUE
1230 IF (NODC.LT.1) GOTO 1232
DO 1237 J=1, NNOD
TEMP = FC(J,5)
TEST = DMAX(J,2)
IF (TEMP.LT.TEST) GOTO 1237
DMAX(J,2) = TEMP
TIMEX(J,2) = TIME
CONTINUE
1237 TMEAN = TMEAN + DSEC
KKOUT = KKOUT + 1
DT = DTT
DSEC = DSECP
IF (TIME.GE.TI .AND. TIME.LE.TO) GOTO 9999
IF (TIME.LT.TTOUT) GOTO 2100
C..... STORE FLOODPLAIN AND CHANNEL RESULTS IN OUTPUT FILE
9999 WRITE (NWRITE,6)
WRITE (NWRITE,1300) TIME
WRITE (NWRITE,1300) TIME
IF (RARC.NE.0) WRITE (NWRITE,1301) RRATE
FORMAT (//,5X, 'MODEL TIME (HOURS) = ', F10.2)
1300 FORMAT (11X, 'EFFECTIVE RAINFALL (IN/HR) = ', F6.2, //)
1301 IF (ITER.EQ.0 .AND. NODC.NE.0) GOTO 2001
WRITE (NWRITE,26)
26 FORMAT (7X, '***FLOOD PLAIN RESULTS***')
IF (NPFI.LT.1) GOTO 821
DO 810 J=1, NPFI
DO 820 I=1, NPFI
IF (TIME.GT.HP(J,I,1)) GOTO 820
QIN = HP(J,I,1,2) + (HP(J,I,2) - HP(J,I,1,2)) * (TIME - HP(J,I,1,1)) /
(HP(J,I,1) - HP(J,I,1,1))
C
GO TO 830
820 CONTINUE
830 WRITE (NWRITE,28) KINP(J), QIN
810 CONTINUE
821 KO=1
IO=1
JO=10
200 WRITE (NWRITE,1) (J,J=IO,JO)
WRITE (NWRITE,2) (FP(J,7), J=IO,JO)
WRITE (NWRITE,3) (FP(J,8), J=IO,JO)
IF (KODE.EQ.1) WRITE (NWRITE,31) (VEL(J,1), J=IO,JO)
IF (KODE.EQ.1) WRITE (NWRITE,74) (VEL(J,2), J=IO,JO)
IF (KODE.EQ.1) WRITE (NWRITE,35) (VEL(J,3), J=IO,JO)
IF (KODE.EQ.1) WRITE (NWRITE,36) (VEL(J,4), J=IO,JO)
33 FORMAT (5X, 'VEL-W', 10(3X, F8.3))
74 FORMAT (5X, 'VEL-E', 10(3X, F8.3))
35 FORMAT (5X, 'VEL-S', 10(3X, F8.3))
36 FORMAT (5X, 'VEL-W', 10(3X, F8.3))
1 FORMAT (//,5X, 'NODE', 7X, 10(I3, EX))
2 FORMAT (5X, 'DEPTH', 10(3X, F8.3))
3 FORMAT (3X, 'ELEVATION', F9.3, 10(2X, F9.3))
KO=KO+1
IO=IO+10
JO=10*KO
IF (JO.LE.NNOD) GOTO 200
IF (JO>NNOD.GE.10) GOTO 100
JO=NNOD
GO TO 200
100 DO 1360 J=1, NNOD
1360 FP(J,8) = 0.
C..... OUTPUT OUTFLOW RATE AT CRITICAL-DEPTH NODES
IF (NDC.LT.1) GOTO 2001
WRITE (NWRITE,9)
9 FORMAT (//,5X, 'OUTFLOW RATE AT CRITICAL-DEPTH NODES:',
/, 10X, 'NODE OUTFLOW RATE (CFS)')
DO 1400 J=1, NDC
JJ = NODDC(J)
QOUT = 5.67 * (FP(JJ,7) ** 0.5) * SIDE * (FP(JJ,7) - TOL)
IF (FP(JJ,7).LT.TOL) QOUT = 0.
WRITE (NWRITE,8) JJ, QOUT
FORMAT (10X, I4, 5X, F10.2)
CONTINUE
1400 WRITE (NWRITE,6)
2001 IF (NODC.LT.1) GOTO 2000
WRITE (NWRITE,27)
27 FORMAT (//,7X, '***CHANNEL RESULTS***, //)
IF (NCHI.LT.1) GOTO 321
DO 310 J=1, NCHI
DO 320 I=2, NPCHI
IF (TIME.GT.H(J,I,1)) GOTO 320
QIN = H(J,I,1,2) + (H(J,I,2) - H(J,I,1,2)) * (TIME - H(J,I,1,1)) /
(H(J,I,1) - H(J,I,1,1))
C
GO TO 330
320 CONTINUE
330 WRITE (NWRITE,28) KIN(J), QIN
28 FORMAT (10X, 'INFLOW RATE AT NODE ', I3, ' IS EQUAL TO ', F10.2)
CONTINUE
310 IF (NCHO.LT.1) GOTO 341
DO 340 J=1, NCHO
JJ = KOUT(J)
DO 345 KJ=1, NPCHO
IF (FC(JJ,5).GT.HOUT(J,KJ,1)) GOTO 345
QOUT = HOUT(J,KJ,2) * (FC(JJ,5) ** HOUT(J,KJ,3))
IF (FC(JJ,5).LT.TOL) QOUT = 0.
GO TO 346
345 CONTINUE
346 WRITE (NWRITE,29) JJ, QOUT
29 FORMAT (10X, 'OUTFLOW RATE AT NODE ', I3, ' IS EQUAL TO ', F10.2)
CONTINUE
340 CONTINUE
341 KO=1
IO=1
JO=10
201 WRITE (NWRITE,1) (J,J=IO,JO)
WRITE (NWRITE,2) (FC(J,5), J=IO,JO)
WRITE (NWRITE,3) (FC(J,6), J=IO,JO)
KO=KO+1
IO=IO+10
JO=10*KO
IF (JO.LE.NNOD) GOTO 201
IF (JO>NNOD.GE.10) GOTO 1001
JO=NNOD
GO TO 200
1001 WRITE (NWRITE,10001)
FORMAT (///,10X, 'MAXIMUM WATER SURFACE VALUES FOR FLOOD',
' PLAIN', //)
KO=1
IO=1
JO=10
300 WRITE (NWRITE,1) (J,J=IO,JO)
WRITE (NWRITE,2) (DMAX(J,1), J=IO,JO)
WRITE (NWRITE,4) (TIMEX(J,1), J=IO,JO)
4 FORMAT (5X, 'TIME ', 10(3X, F8.3))
KO=KO+1
IO=IO+10
JO=10*KO
IF (JO.LE.NNOD) GOTO 300
IF (JO>NNOD.GE.10) GOTO 400
JO=NNOD
GO TO 300
400 WRITE (NWRITE,7)
6666 IF (NODC.LT.1) GOTO 4400
WRITE (NWRITE,10002)
10002 FORMAT (///,10X, 'MAXIMUM WATER SURFACE VALUES FOR CHANNEL', //)
KO=1
IO=1
JO=10
3300 WRITE (NWRITE,1) (J,J=IO,JO)
WRITE (NWRITE,2) (DMAX(J,2), J=IO,JO)
WRITE (NWRITE,4) (TIMEX(J,2), J=IO,JO)
KO=KO+1
IO=IO+10
JO=10*KO
IF (JO.LE.NNOD) GOTO 3300
IF (JO>NNOD.GE.10) GOTO 4400
JO=NNOD
GO TO 3300
4400 WRITE (NWRITE,7)
C..... END OF PROGRAM
IF (ID.EQ.1) WRITE (NWRITE,909)
909 FORMAT (2X, '*** DEPTH OF WATER IS EITHER GREATER THAN',
' 150 OR LESS THAN 0 *** / 2X, '*** PROGRAM STOP ***')
IF (IKODE.GE.3) WRITE (NWRITE,907) DSEC
907 FORMAT (2X, '*** MINIMUM TIMESTEP ', F4.1, ' SEC. IS TOO LARGE!!',
' / 2X, '*** A SMALLER TIMESTEP SHOULD BE USED ***')
STOP
END
SUBROUTINE FLOODC (TIME, TTIME, NNOD, NCHI, NCHO, NPCHI, NPCHO, SIDE,
NSTA, NPSTA, NODC, TOL, DTOL, DTOLX, NPLUX, KPLUX, ITER, FCMA, NWRITE,
C CHECKD, KMODEL)
C
C THIS SUBROUTINE CALCULATES THE DEPTH OF WATER FOR
C THE CHANNEL MODEL
C
COMMON/BLK 1/FP(200,8), FC(200,6)
COMMON/BLK 2/RIN(10), H(10,10,2), KOUT(10), HOUT(10,10,3)
COMMON/BLK 3/NOSTA(10), STA(10,10,2), NODFX(50)
COMMON/BLK 4/DMAX(200,2), TIMEX(200,2)
DIMENSION Q(4)
C
DEFINITIONS
FC(I,J) = HANNINGS, WIDTH, DEPTH, ELEV, INITIAL DEPTH, TEMPORARY MEMORY
RIN(I) = ARRAY OF INFLOW NODE
H(I,J,2) = TIME COORDINATE FOR INFLOW RATE IN HOUR
H(I,J,3) = INFLOW RATE (CFS)
KOUT(I) = ARRAY OF OUTFLOW NODE
HOUT(I,J) = PARAMETERS FOR OUTFLOW NODE
Q(I) = VOLUME OF FLOW
NOSTA(I) = ARRAY OF STAGE STATION
STA(I,J,1) = TIME COORDINATE FOR STAGE CURVE
STA(I,J,2) = DEPTH OF WATER IN FEET
C
CHANNEL MODEL
C..... INITIALIZE CONSTANTS
QBC=0.
QTEMP=0.
DO 40 J=1, NNOD
FC(J,6) = 0.
40 CONTINUE
IF (KPLUX.EQ.1 .AND. ITER.EQ.0) WRITE (NWRITE,212) TIME
212 FORMAT (//,130(' '), 5X, 'MODEL TIME (HOUR) = ', F10.2, //)
IF (KPLUX.EQ.1) WRITE (NWRITE,213)
C..... MAIN LOOP FOR CHANNEL MODEL
C..... UPDATE TIME AND BOUNDARY CONDITION VALUES
IF (NCHI.LT.1) GOTO 711
DO 695 J=1, NCHI
DO 710 I=2, NPCHI
IF (TIME.GT.H(J,I,1)) GOTO 710
QTEMP = H(J,I,1,2) + (H(J,I,2) - H(J,I,1,2)) * (TIME - H(J,I,1,1)) /
(H(J,I,1) - H(J,I,1,1))
GO TO 730
710 CONTINUE
730 QBC = QTEMP * TTIME
IF (QBC.LT.0.) QBC = 0.
C..... UPDATE INFLOW BOUNDARY CONDITION NODES
JJ = IN(J)
FC(IJ,6) = QBC
695 CONTINUE
C..... CALCULATE FLOW VELOCITIES AND FLOWRATES
DO 1000 I=1, NNOD
ONET = 0.
IF (FP(I,5).GT.0.) GOTO 1000
DO 690 II=1,4

```

```

        QQ=0.
        NQ=FP(1,1)
        IF (NQ,5).GT.0.1NQ=0
        IF (NQ,5).EQ.0. GOTO 690
        CALL QFC(I,NQ,QQ,SIDE,TOL,KMODEL)
690      Q(I)=NQ
C.....ADJUST FLOWRATES FOR DIRECTION
        Q(3)=-Q(3)
        Q(4)=-Q(4)
C.....ESTIMATE ACCUMULATION OF INFLOW
        QNET=(Q(3)*Q(4)-Q(1)-Q(2))*TTIME
        IF (NPLUX,5).EQ.0 GOTO 1000
        IF (KFLUX,5).EQ.0 GOTO 1000
213     FORMAT(1X,'AVERAGE FLOW RATE FOR SPECIFIED CHANNEL NODES :',
           C  ' /,10X,'NODE',5X,'QM',9X,'QE',9X,'QS',9X,'QW')
        DO 540 J=1,NPLUX
        IF (I.NE.NODFX(J)) GOTO 540
        WRITE(NWRITE,24) I,Q(1),Q(2),Q(3),Q(4)
24      FORMAT(10X,I4,4(2X,E9.3))
540     CONTINUE
1000    FC(I,6)=QNET+FC(I,5)
C.....ACCOUNT DISCHARGE AT OUTFLOW NODES
        IF (NCHO,LT.1) GOTO 741
        DO 1100 J=1,NCHO
        JJ=KOUT(J)
        DO 1110 K=1,NPCHO
        IF (FC(JJ,5).GT.HOUT(J,K,1)) GOTO 1110
        QOUT=HOUT(J,K,2)*FC(JJ,5)**HOUT(J,K,3)*TTIME
        IF (FC(JJ,5).LT.TOL) QOUT=0.
        GO TO 1111
1110    CONTINUE
1111    FC(JJ,6)=FC(JJ,6)-QOUT
1100    CONTINUE
C.....UPDATE THE WATER ELEVATIONS AT STAGE STATIONS
741     IF (NSTA,LT.1) GOTO 1201
        DO 740 I=1,NSTA
        NN=NOSTA(I)
        DO 750 J=2,NPSTA
        IF (TIME,GT.STA(I,J,1)) GOTO 750
        DE=STA(I,J,2)+(STA(I,J,2)-STA(I,J-1,2))*(TIME-STA(I,J-1,1))
           C  / (STA(I,J,1)-STA(I,J-1,1))
        GO TO 760
750     CONTINUE
760     FCMAX=ABS(DE-FC(NN,5))-FC(NN,4)
        FC(NN,5)=DE-FC(NN,4)
        FC(NN,6)=0.
740     CONTINUE
C.....CHECK MAXIMUM CHANGE OF WATER DEPTH
1201    DO 1200 J=1,NNOD
        IF (NSTA,LT.1) GOTO 1253
        DO 1252 JJ=1,NSTA
        IF (J,GT.NOSTA(JJ)) GOTO 1200
        CONTINUE
1252    IF (FP(J,5).GT.0.) GOTO 1200
        A=0.
        KCO=0
        DO 1251 JJ=1,4
        NQ=FP(J,JJ)
        IF (FP(NQ,5).GT.0.) GOTO 1251
        A=A+(.25*FC(NQ,2)+.75*FC(J,2))*5*SIDE
        KCO=KCO+1
1251    CONTINUE
        IF (KCO,GT.1) A=2.*A
        FC(J,6)=FC(J,6)/A
1200    CONTINUE
        DO 1255 I=1,NNOD
        TEMP=ABS(FC(I,6))
        IF (TEMP,LT.DTOL) GOTO 1255
        IF (FC(I,5).LT.CHECKD) FCMAX=99.
        IF (FC(I,5).LT.CHECKD) RETURN
        TOLP=TEMP/FC(I,5)
        IF (TOLP,GE.DTOLP) FCMAX=99.
        IF (TOLP,GE.DTOLP) RETURN
1255    CONTINUE
        RETURN
        END
    
```

```

SUBROUTINE CHANPL(MNOD,SIDE,TOL)
C
C THIS SUBROUTINE UPDATES THE WATER SURFACE ELEVATION
C BETWEEN THE FLOODPLAIN AND CHANNEL MODELS
C
COMMON/BLK 1/FP(200,8),FC(200,6)
DO 100 I=1,NNOD
C.....CHECK INTERFACE BETWEEN CHANNEL AND FLOOD PLAIN
        IF (FP(I,5).GT.0.) GOTO 100
C.....A IS WATER LEVEL AT FLOOD PLAIN
C.....B IS WATER LEVEL AT CHANNEL
C.....FC(I,3) IS THE DEPTH OF CHANNEL
        A=FP(I,6)-FP(I,7)
        B=FC(I,4)+FC(I,5)
        IF (A,GT.B) GOTO 110
C.....FLOODING OF CHANNEL, B > A
        FP(I,7)=FP(I,7)+(B-A)*FC(I,2)/SIDE
        FC(I,5)=FP(I,7)+FC(I,3)
        GO TO 100
C.....FLOW INTO CHANNEL FROM GRID ELEMENT, A > B
110     IF (FC(I,3).LT.FC(I,5)) GOTO 120
        VAL=(FC(I,3)-FC(I,5))*TOL/FC(I,2)
        VW=(SIDE-FC(I,2))*(FP(I,7)-TOL)
C.....CASE 1 - NO FLOW INTO CHANNEL
        IF (VW,LT.0.) GOTO 100
        IF (VAL,GE.VW) GOTO 130
C.....CASE 2 - CHANNEL IS FULLED AFTER FILLING
        FP(I,7)=TOL+(VW-VAL)/SIDE
        FC(I,5)=FC(I,3)+FP(I,7)
        GO TO 100
C.....CASE 3 - FC(I,3) > FC(I,5)
130     FC(I,5)=FC(I,5)+VW/FC(I,2)
        FP(I,7)=TOL
        GOTO 100
C.....CASE 4 - FC(I,5) > FC(I,3)
120     FP(I,7)=B+(A-B)*(SIDE-FC(I,2))/SIDE-FP(I,6)
        FC(I,5)=FP(I,7)+FC(I,3)
100     CONTINUE
        RETURN
        END
C
SUBROUTINE QFP(I,NQ,SIDE,QQ,ID,VEL,TOL,KMODEL)
C
C THIS SUBROUTINE CALCULATES THE EFFLUX PER UNIT WIDTH
C WHICH FLOWS ACROSS THE ADJACENT CONTROL VOLUMES
C
COMMON/BLK 1/FP(200,8),FC(200,6)
VEL=0.
ID=0
QQ=0.
H=FP(I,7)+FP(I,6)
    
```

```

        IF (KMODEL,5).EQ.1) H=FP(I,6)
        IF (FP(I,7).EQ.0. .AND. FP(NQ,7).EQ.0.) GOTO 2002
C.....DEPTHS ARE NONZERO
        HN=FP(NQ,7)+FP(NQ,6)
        IF (KMODEL,5).EQ.1) HN=FP(NQ,6)
        GRAD=(HN-H)/SIDE
        HBAR=.5*(FP(I,7)+FP(NQ,7))
        IF (GRAD) 150,2002,170
        H > HN
150     IF (FP(I,7).LT.TOL) GOTO 2002
        YBAR=FP(I,7)-TOL
        GOTO 180
C.....HN > H
170     IF (FP(NQ,7).LT.TOL) GOTO 2002
        YBAR=FP(NQ,7)-TOL
        IF (YBAR,LT.TOL) GOTO 2002
        XNBAR=.5*(ABS(FP(I,5))+ABS(FP(NQ,5)))
        AGRAD=ABS(GRAD)
        IF (AGRAD,GT..00001) GOTO 185
        QQ=0.
        GOTO 2002
185     XK=1.486/XNBAR*YBAR*HBAR**.667/SQRT(AGRAD)
        IF (HBAR,LT.0.) ID=1
        IF (HBAR,GT.150.) ID=1
        QQ=XK*GRAD
        VEL=QQ/YBAR
2002    CONTINUE
        RETURN
        END
SUBROUTINE QFC(I,NQ,QQ,SIDE,TOL,KMODEL)
C
C THIS SUBROUTINE CALCULATES VOLUME OF WATER THAT
C FLOWS ACROSS THE ADJACENT CONTROL VOLUMES
C
COMMON/BLK 1/FP(200,8),FC(200,6)
QQ=0.
DCH=.5*(FC(I,3)+FC(NQ,3))
WID=.5*(FC(I,2)+FC(NQ,2))
H=FC(I,4)+FC(I,5)
IF (KMODEL,5).EQ.1) H=FC(I,4)
IF (FC(I,5).EQ.0. .AND. FC(NQ,5).EQ.0.) GOTO 2002
C.....DEPTHS ARE NONZERO
HN=FC(NQ,4)+FC(NQ,5)
IF (KMODEL,5).EQ.1) HN=FC(NQ,4)
GRAD=(HN-H)/SIDE
IF (GRAD) 150,2002,170
H > HN
150     IF (FC(I,5).LT.TOL) GOTO 2002
        YBAR=FC(I,5)
        GOTO 180
C.....HN > H
170     IF (FC(NQ,5).LT.TOL) GOTO 2002
        YBAR=FC(NQ,5)
180     HBAR=.5*(FC(I,5)+FC(NQ,5))
        WETT=2.*HBAR+WID
        WETC=2.*DCH+WID
        WET=AMIN1(WETC,WETT)
        A=WID*HBAR
        R=A/WET
        IF (HBAR,LT.TOL) GOTO 2002
        XNBAR=.5*(FC(I,1)+FC(NQ,1))
        AGRAD=ABS(GRAD)
        IF (AGRAD,GT..00001) GOTO 185
        QQ=0.
        GOTO 2002
185     XK=1.486/XNBAR*R**.667/SQRT(AGRAD)
        VEL=XK*GRAD
        QQ=VEL*WID*YBAR
2002    CONTINUE
        RETURN
        END
    
```

A.1 EXAMPLE INPUT FILE

1.	30.	1.	10.	10.	1.	.5	0	2
160	36	500	.0001	1	10.	0	0	
2	11	0	0	.040	101.000	0.		
3	12	1	0	.040	101.500	0.		
4	13	2	0	.040	102.000	0.		
5	14	3	0	.040	102.500	0.		
6	15	4	0	.040	103.000	0.		
7	16	5	0	.040	103.500	0.		
8	17	6	0	.040	104.000	0.		
9	18	7	0	.040	104.500	0.		
10	19	8	0	.040	105.000	0.		
0	20	9	0	.040	105.500	0.		
12	21	0	1	.040	100.500	0.		
13	22	11	2	.040	101.000	0.		
14	23	12	3	.040	101.500	0.		
15	24	13	4	.040	102.000	0.		
16	25	14	5	.040	102.500	0.		
17	26	15	6	.040	103.000	0.		
18	27	16	7	.040	103.500	0.		
19	28	17	8	.040	104.000	0.		
20	29	18	9	.040	104.500	0.		
0	30	19	10	.040	105.000	0.		
22	31	0	11	.040	100.000	0.		
23	32	21	12	.040	100.500	0.		
24	33	22	13	.040	101.000	0.		
25	34	23	14	.040	101.500	0.		
26	35	24	15	.040	102.000	0.		
27	36	25	16	.040	102.500	0.		
28	37	26	17	.040	103.000	0.		
29	38	27	18	.040	103.500	0.		
30	39	28	19	.040	104.000	0.		
0	40	29	20	.040	104.500	0.		
32	41	0	21	-.040	99.500	0.		
33	42	31	22	-.040	100.000	0.		
34	43	32	23	-.040	100.500	0.		
35	44	33	24	-.040	101.000	0.		
36	45	34	25	-.040	101.500	0.		
37	46	35	26	-.040	102.000	0.		
38	47	36	27	-.040	102.500	0.		
39	48	37	28	-.040	103.000	0.		
40	49	38	29	-.040	103.500	0.		
0	50	39	30	-.040	104.000	0.		
42	51	0	31	-.040	100.000	0.		
43	52	41	32	.040	100.500	0.		
44	53	42	33	.040	101.000	0.		
45	54	43	34	.040	101.500	0.		
46	55	44	35	.040	102.000	0.		
47	56	45	36	.040	102.500	0.		
48	57	46	37	.040	103.000	0.		

49	58	47	38	.040	103.500	0.				
50	59	48	39	.040	104.000	0.				
0	60	49	40	.040	104.500	0.				
52	61	0	41	.040	100.500	0.				
53	62	51	42	.040	101.000	0.				
54	63	52	43	.040	101.500	0.				
55	64	53	44	.040	102.000	0.				
56	65	54	45	.040	102.500	0.				
57	66	55	46	.040	103.000	0.				
58	67	56	47	.040	103.500	0.				
59	68	57	48	.040	104.000	0.				
60	69	58	49	.040	104.500	0.				
0	70	59	50	.040	105.000	0.				
62	71	0	51	.040	100.000	0.				
63	72	61	52	.040	100.500	0.				
64	73	62	53	.040	101.000	0.				
65	74	63	54	.040	101.500	0.				
66	75	64	55	.040	102.000	0.				
67	76	65	56	.040	102.500	0.				
68	77	66	57	.040	103.000	0.				
69	78	67	58	.040	103.500	0.				
70	79	68	59	.040	104.000	0.				
0	80	69	60	.040	104.500	0.				
72	81	0	61	-.040	99.500	0.				
73	82	71	62	-.040	100.000	0.				
74	83	72	63	-.040	100.500	0.				
75	84	73	64	-.040	101.000	0.				
76	85	74	65	-.040	101.500	0.				
77	86	75	66	-.040	102.000	0.				
78	87	76	67	-.040	102.500	0.				
79	88	77	68	-.040	103.000	0.				
80	89	78	69	-.040	103.500	0.				
0	90	79	70	-.040	104.000	0.				
82	91	0	71	.040	100.000	0.				
83	92	81	72	.040	100.500	0.				
84	93	82	73	.040	101.000	0.				
85	94	83	74	.040	101.500	0.				
86	95	84	75	.040	102.000	0.				
87	96	85	76	-.040	102.500	0.				
88	97	86	77	.040	103.000	0.				
89	98	87	78	.040	103.500	0.				
90	99	88	79	.040	104.000	0.				
0	100	89	80	.040	104.500	0.				
92	101	0	81	.040	100.500	0.				
93	102	91	82	.040	101.000	0.				
94	103	92	83	.040	101.500	0.				
95	104	93	84	.040	102.000	0.				
96	105	94	85	.040	102.500	0.				
97	106	95	86	-.040	103.000	0.				
98	107	96	87	-.040	103.500	0.				
99	108	97	88	-.040	104.000	0.				
100	109	98	89	-.040	104.500	0.				
0	110	99	90	-.040	105.000	0.				
102	111	0	91	.040	101.000	0.				
103	112	101	92	.040	101.500	0.				
104	113	102	93	.040	102.000	0.				
105	114	103	94	.040	102.500	0.				
106	115	104	95	.040	103.000	0.				
107	116	105	96	.040	103.500	0.				
108	117	106	97	.040	104.000	0.				
109	118	107	98	.040	104.500	0.				
110	119	108	99	.040	105.000	0.				
0	120	109	100	.040	105.500	0.				
112	121	0	101	.040	100.500	0.				
113	122	111	102	.040	101.000	0.				
114	123	112	103	.040	101.500	0.				
115	124	113	104	.040	102.000	0.				
116	125	114	105	.040	102.500	0.				
117	126	115	106	.040	103.000	0.				
118	127	116	107	.040	103.500	0.				
119	128	117	108	.040	104.000	0.				
120	129	118	109	.040	104.500	0.				
0	130	119	110	.040	105.000	0.				
122	131	0	111	-.040	100.000	0.				
123	132	121	112	-.040	100.500	0.				
124	133	122	113	-.040	101.000	0.				
125	134	123	114	-.040	101.500	0.				
126	135	124	115	-.040	102.000	0.				
127	136	125	116	-.040	102.500	0.				
128	137	126	117	-.040	103.000	0.				
129	138	127	118	-.040	103.500	0.				
130	139	128	119	-.040	104.000	0.				
0	140	129	120	-.040	104.500	0.				
132	141	0	121	.040	100.500	0.				
133	142	131	122	.040	101.000	0.				
134	143	132	123	.040	101.500	0.				
135	144	133	124	.040	102.000	0.				
136	145	134	125	.040	102.500	0.				
137	146	135	126	.040	103.000	0.				
138	147	136	127	.040	103.500	0.				
139	148	137	128	.040	104.000	0.				
140	149	138	129	.040	104.500	0.				
0	150	139	130	.040	105.000	0.				
142	151	0	131	.040	101.000	0.				
143	152	141	132	.040	101.500	0.				
144	153	142	133	.040	102.000	0.				
145	154	143	134	.040	102.500	0.				
146	155	144	135	.040	103.000	0.				
147	156	145	136	.040	103.500	0.				
148	157	146	137	.040	104.000	0.				
149	158	147	138	.040	104.500	0.				
150	159	148	139	.040	105.000	0.				
0	160	149	140	.040	105.500	0.				
152	0	0	141	.040	101.500	0.				
153	0	151	142	.040	102.000	0.				
154	0	152	143	.040	102.500	0.				
155	0	153	144	.040	103.000	0.				
156	0	154	145	.040	103.500	0.				
157	0	155	146	.040	104.000	0.				
158	0	156	147	.040	104.500	0.				
159	0	157	148	.040	105.000	0.				
160	0	158	149	.040	105.500	0.				
0	0	159	150	.040	106.000	0.				
0	0	0	0	0	0	0				
1	2	3	4	5	6	7	8	9		
0	0	0	0	0	0	0	0	0		
31	.015	10.	6.	93.5	0.					
32	.015	10.	6.	94.0	0.					
33	.015	10.	6.	94.5	0.					
34	.015	10.	6.	95.0	0.					
35	.015	10.	6.	95.5	0.					
36	.015	10.	6.	96.0	0.					
37	.015	10.	6.	96.5	0.					
38	.015	10.	6.	97.0	0.					
39	.015	10.	6.	97.5	0.					
40	.015	10.	6.	98.0	0.					
71	.015	10.	6.	93.5	0.					
72	.015	10.	6.	94.0	0.					
73	.015	10.	6.	94.5	0.					
74	.015	10.	6.	95.0	0.					
75	.015	10.	6.	95.5	0.					
76	.015	10.	6.	96.0	0.					
77	.015	10.	6.	96.5	0.					
78	.015	10.	6.	97.0	0.					
79	.015	10.	6.	97.5	0.					
80	.015	10.	6.	98.0	0.					
86	.015	10.	5.5	97.0	0.					
96	.015	10.	5.	98.0	0.					
97	.015	10.	5.	98.5	0.					
98	.015	10.	5.	99.0	0.					
99	.015	10.	5.	99.5	0.					
100	.015	10.	5.	100.0	0.					
121	.015	10.	6.	94.0	0.					
122	.015	10.	6.	94.5	0.					
123	.015	10.	6.	95.0	0.					
124	.015	10.	6.	95.5	0.					
125	.015	10.	6.	96.0	0.					
126	.015	10.	6.	96.5	0.					
127	.015	10.	6.	97.0	0.					
128	.015	10.	6.	97.5	0.					
129	.015	10.	6.	98.0	0.					
130	.015	10.	6.	98.5	0.					
4	5	3	1	0	0					
40	0	0	1	300	3	300	5	0	12	0
80	0	0	1	300	3	300	5	0	12	0
100	0	0	1	200	3	200	5	0	12	0
130	0	0	1	400	3	400	5	0	12	0
31	30	30	1							
71	30	30	1							
121	30	30	1							