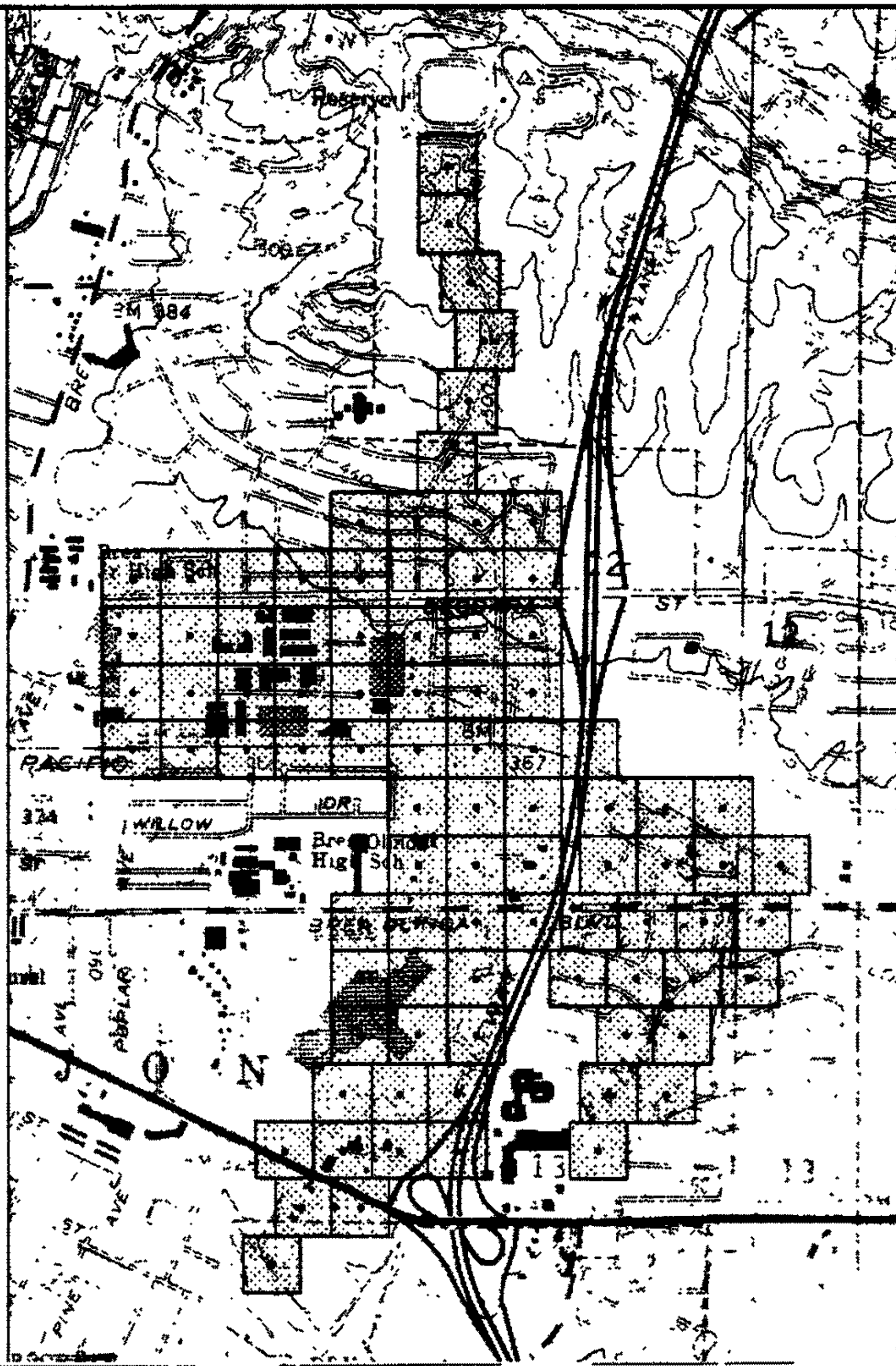
A TWO-DIMENSIONAL DAM-BREAK MODEL OF THE ORANGE COUNTY RESERVOIR

PREPARED FOR:

LSA

OUR JOB NO:
84166

JUNE 11, 1984



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A TWO-DIMENSIONAL
DAM-BREAK MODEL OF
THE ORANGE COUNTY RESERVOIR

ESPECIALLY PREPARED FOR

L.S.A.

OUR JOB NO. 84166

JUNE 7, 1984

WILLIAMSON AND SCHMID

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I. INTRODUCTION

I.1 PURPOSE OF REPORT

The purpose of this report is to present study conclusions in estimating a flood plain which may result from a hypothetical dam-failure of the Orange County Reservoir located north of the City of Brea. The results of this study are to be used as partial fulfillment of the necessary environment impact investigations needed for development of the Brea Mall, in the City of Brea.

I.2 PROJECT SITE

The study site includes the area between the Orange County Reservoir (north of the City of Brea) and the proposed Brea Mall development.

The Brea Mall is located in the City of Brea, westerly of the Orange Freeway (57) and is bounded on the north by Birch Street, on the east by State College Boulevard, on the south by Imperial Highway, and on the west by Randolph Avenue (see Fig. 1).

A field examination of the project site is documented by the photo-log contained in Appendix A of this report.

I.3 STUDY METHODOLOGY

A convenient approach to study a hypothetical dam-failure is to simply estimate a maximum possible flowrate and route this flow as a steady state discharge through the downstream reaches. This method is excessively conservative in that all effects due to the time variations in channel storage, routing, and the slow failure of an earthen dam structure.

To account for these dynamic processes, a two-dimensional diffusion dam-break model originally developed for the U.S.G.S. (Hromadka, 1984) is used to estimate the flood plain resulting from a hypothetical failure of the subject Orange County Reservoir site.

I.4 ORANGE COUNTY RESERVOIR

Orange County Reservoir, which was constructed during 1940-41, is located approximately 2.5 miles northeast of the City of Brea in Orange County. It has a storage capacity of 212 acre-feet and serves primarily as a regulating facility on the Orange County Feeder. This reservoir also provides water storage which can permit continued service for the cities of Fullerton, Anaheim, and Santa Ana when the northerly portion of the Orange County Feeder is shut down.

The reservoir is of the cut and fill type, rectangular in shape with rounded

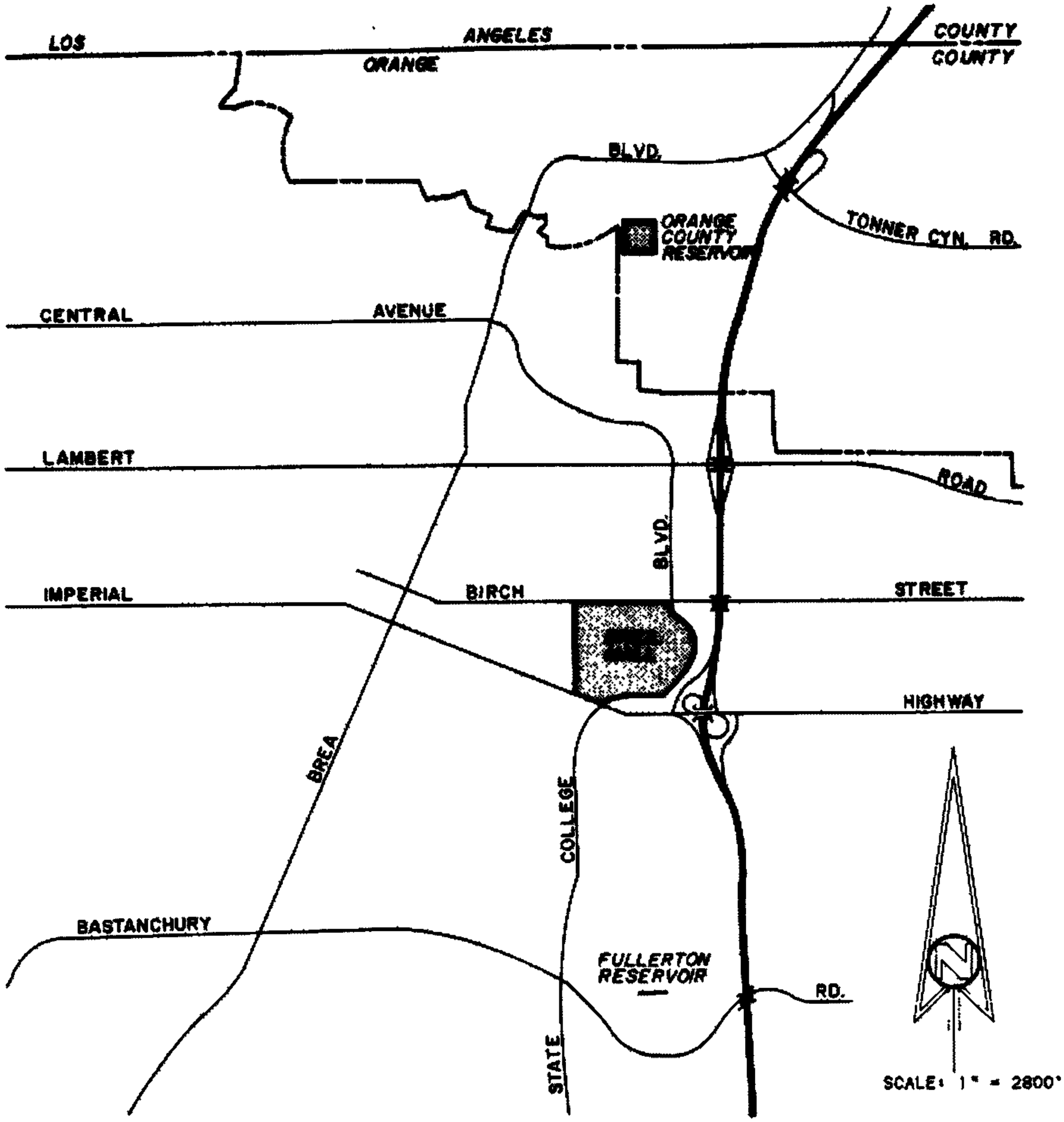

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FIGURE 1

LOCATION MAP

SCALE	1" = 2800'
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corners, and has sides that are constructed on a 3 to 1 slope. Construction of the bottom and side slopes of reservoir included (1) the placement of a 5 foot thick blanket of selected earth fill to insure maximum stability and imperviousness; and (2) the placement of 2-inch thick, wire mesh reinforced gunite lining to protect against erosion.

Orange County Reservoir is connected to the Orange County Feeder through both a reservoir inlet, spillway and by-pass structure on the northeast side and a separate outlet facility on the southeast side of the reservoir. The inlet and outlet facilities are connected by a segment of the Orange County Feeder which forms a reservoir by-pass and permits the entire flow of the Orange County Feeder to be diverted around the reservoir under unusual operating conditions.

I.5 REPORT ORGANIZATION

This report is organized into four sections as follows:

SECTION	DESCRIPTION
I	General Introduction
II	Develops and verifies a one-dimensional diffusion dam-break model
III	Develop and verifies a two-dimensional diffusion dam-break model
IV	Applies the two-dimension diffusion dam-break model to the Orange County Reservoir, north of The City of Brea.

I.6 PROJECT STAFF

Engineers participating in this study include:

Ted Hromadka, Ph.D., R.C.E.
Mike McGovern, M.S., R.C.E.
Barry Beech, R.C.E.

II. ONE-DIMENSIONAL DIFFUSION DAM-BREAK MODEL

II.1 INTRODUCTION

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 recent studies. Generally, the one-dimensional flow is modeled wherever there is no significant lateral variation in the flow. Land (1980a,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 downstream flood plain. Consequently, it is noted that a considerable sensitivity in modeling 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 of the report, 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 (1980a,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 later study, comparisons of modeling 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 favorable, 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 modeling approach leads to an economic two-dimentional 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 studied. 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 is to develop a two-dimensional diffusion model for use in estimating dam-break flood plains. Prior to finalizing such a model, however, 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.2 DIFFUSION MODEL

The mathematical relationships used in a one-dimensional diffusion model dam-break formulation are based upon the flow equations of continuity and momentum (Akan and Yen, 1981)

$$\frac{\partial Q_x}{\partial x} + \frac{\partial A_x}{\partial t} = 0 \qquad (1)$$

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

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)

$$Q_x = \frac{1.486}{n} A_x R^{2/3} S_{fx}{}^{1/2} \qquad (3)$$

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

$$m_x = \left[\frac{\partial Q_x}{\partial t} + \frac{\partial (Q_x^2/A_x)}{\partial x} \right] \Big/ gA_x \qquad (4)$$

then Eq. (2) can be rewritten as

$$S_{fx} = - \left[\frac{\partial H}{\partial x} + m_x \right] \qquad (5)$$

In Eq. (4), the subscript x included in m_x indicates the directional term. The expansion of Eq. (2) 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 Eq. (3) and including Eqs. (4) and (5), the directional flow rate is computed by

$$Q_x = - K_x \left(\frac{\partial H}{\partial x} + m_x \right) \tag{6}$$

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

$$K_x = \frac{1.486}{n} A_x R^{2/3} \bigg/ \left| \frac{\partial H}{\partial x} + m_x \right|^{1/2} \tag{7}$$

In Eq. (7), K_x is limited in value by the denominator term being checked for a smallest allowable magnitude.

Substituting the flow rate formulation of Eq. (6) into Eq. (1) gives a diffusion type of relationship

$$\frac{\partial}{\partial x} K_x \left(\frac{\partial H}{\partial x} + m_x \right) = \frac{\partial A_x}{\partial t} \tag{8}$$

The one-dimensional diffusion model of Akan and Yen (1981) assumes $m_x = 0$ in Eq. (7). Likewise, the two-dimensional diffusion model of Xanthopoulos and Koutitas (1976) assumes $m_x = m_y = 0$. Thus, the one-dimensional diffusion model is given by

$$\frac{\partial}{\partial x} K_x \frac{\partial H}{\partial x} = \frac{\partial A_x}{\partial t} \tag{9}$$

where K_x is now simplified as

$$K_x = \frac{1.486}{n} A_x R^{2/3} \Big/ \left| \frac{\partial H}{\partial x} \right|^{1/2} \qquad (10)$$

For a constant channel width, W, Eq. (9) reduces to

$$\frac{\partial}{\partial x} K_x \frac{\partial H}{\partial x} = W \frac{\partial H}{\partial t} \qquad (11)$$

The remainder of this report section considers the use of Eq. (8) as an alternative to the solution of the coupled system of Eqs. (1) and (2). However, it is noted that a family of models is given by Eq. (8) where m_x is defined by selecting from the possibilities

$$m_x = \begin{cases} \dfrac{\partial(Q_x^2/A_x)}{\partial x} \Big/ gA_x, & \text{(convective acceleration model)} \\ \dfrac{\partial Q_x}{\partial t} \Big/ gA_x, & \text{(local acceleration model)} \\ \left(\dfrac{\partial Q_x}{\partial t} + \dfrac{\partial(Q_x^2/A_x)}{\partial x} \right) \Big/ gA_x, & \text{(coupled model)} \\ 0\,, & \text{(diffusion model)} \end{cases} \qquad (12)$$

II.3 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 Mannings n, an elevation, and an initial flow depth (usually zero) defined:

(1) between nodal points, compute an average Mannings n, and average geometric factors

(2) assuming $m_x = 0$, estimate the nodal flow depths for the next

timestep, $(t + \Delta t)$ by using Eqs. (9) and (10) explicitly

(3) using the flow depths at time t and $(t + \Delta t)$, estimate the midtimestep value of m_x selected from Eq. (12).

(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 Eq. (8)

(6) return to 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 Eq. (12). Additionally, the above program steps can be directly applied to a two-dimensional diffusion model with the selected (m_x, m_y) relations incorporated.

II.4 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 introduces 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. $\partial Q/\partial t$ 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 diffusion model in Eq. (11), 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.5 STUDY APPROACH

In order to evaluate the accuracy of the diffusion model of Eq. (11) in the prediction of flood depths, the U.S.G.S. fully dynamic flow model K-634 (Land, 1980a,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 flood depths predicted from both the K-634 and the diffusion (Eq. 11) model for various channel slopes and inflow hydrographs.

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 at time of 6-hours (see Fig 2 inset). The study channel is assumed to be a uniform rectangular section of Manning's n equal to 0.040, and various slopes S_o in the range of $0.001 \leq S_o \leq 0.01$. Figure 2 shows the comparison of modeling 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. 2 which correspond to a slope $S_o = 0.0020$. In all tests, grid spacing was set at 1000-foot intervals.

From Fig. 2 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.

II.6 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. 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 of 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 5000-feet is selected. Figure 3 shows the computed flood depths in comparison to the K-634 modeling results (Fig.2) 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 modeling 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

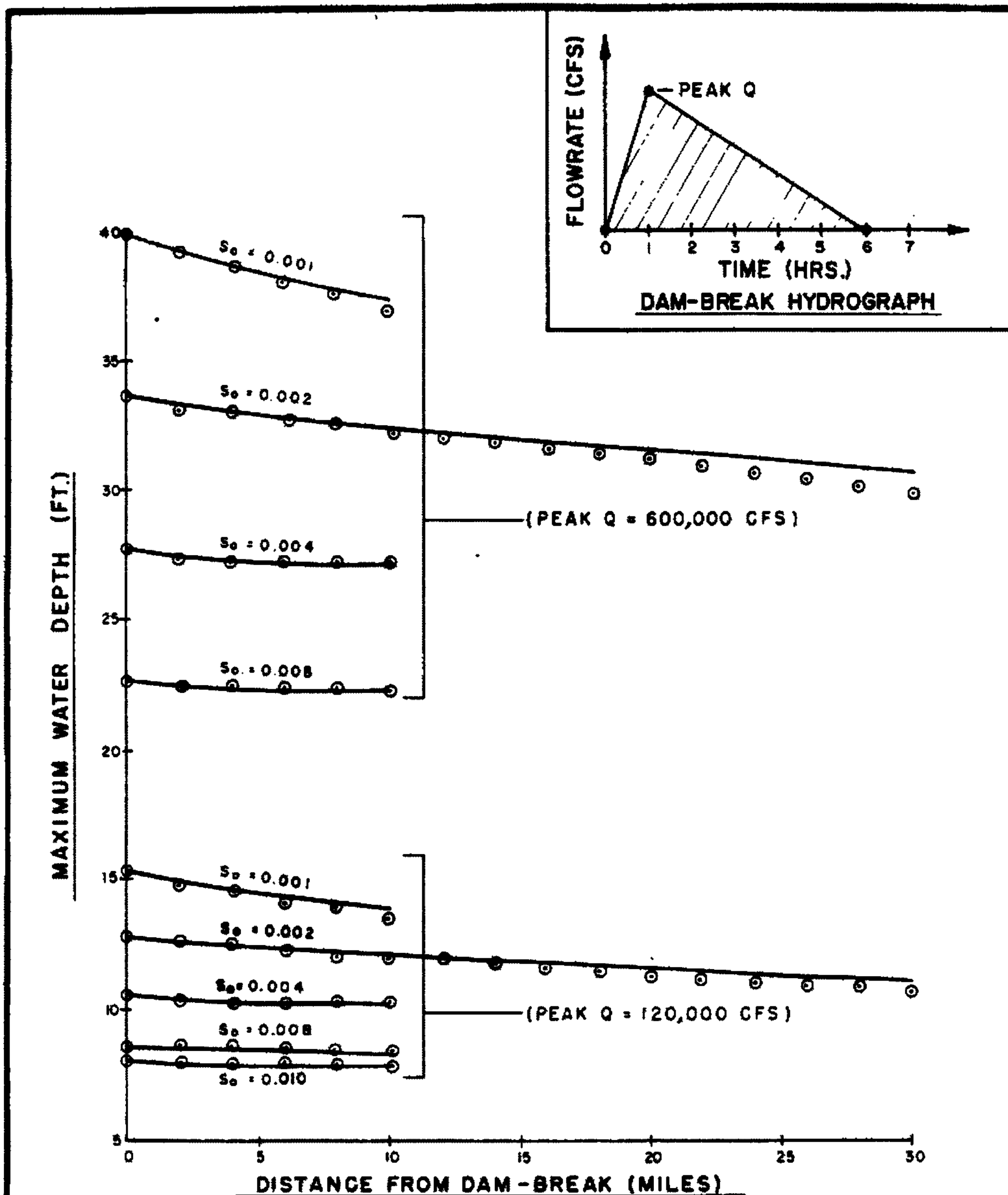


FIG. 2. DIFFUSION MODEL (⊙) AND K-634 MODEL RESULTS (SOLID LINE) FOR 1,000-FOOT WIDTH CHANNEL, MANNING'S n = 0.040, AND VARIOUS CHANNEL SLOPES, S_o.

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 paper, timestep sizes of 7.2 seconds was found to be adequate when using the 1000-feet to 5000-feet grid sizes.

II.7 CONCLUSIONS

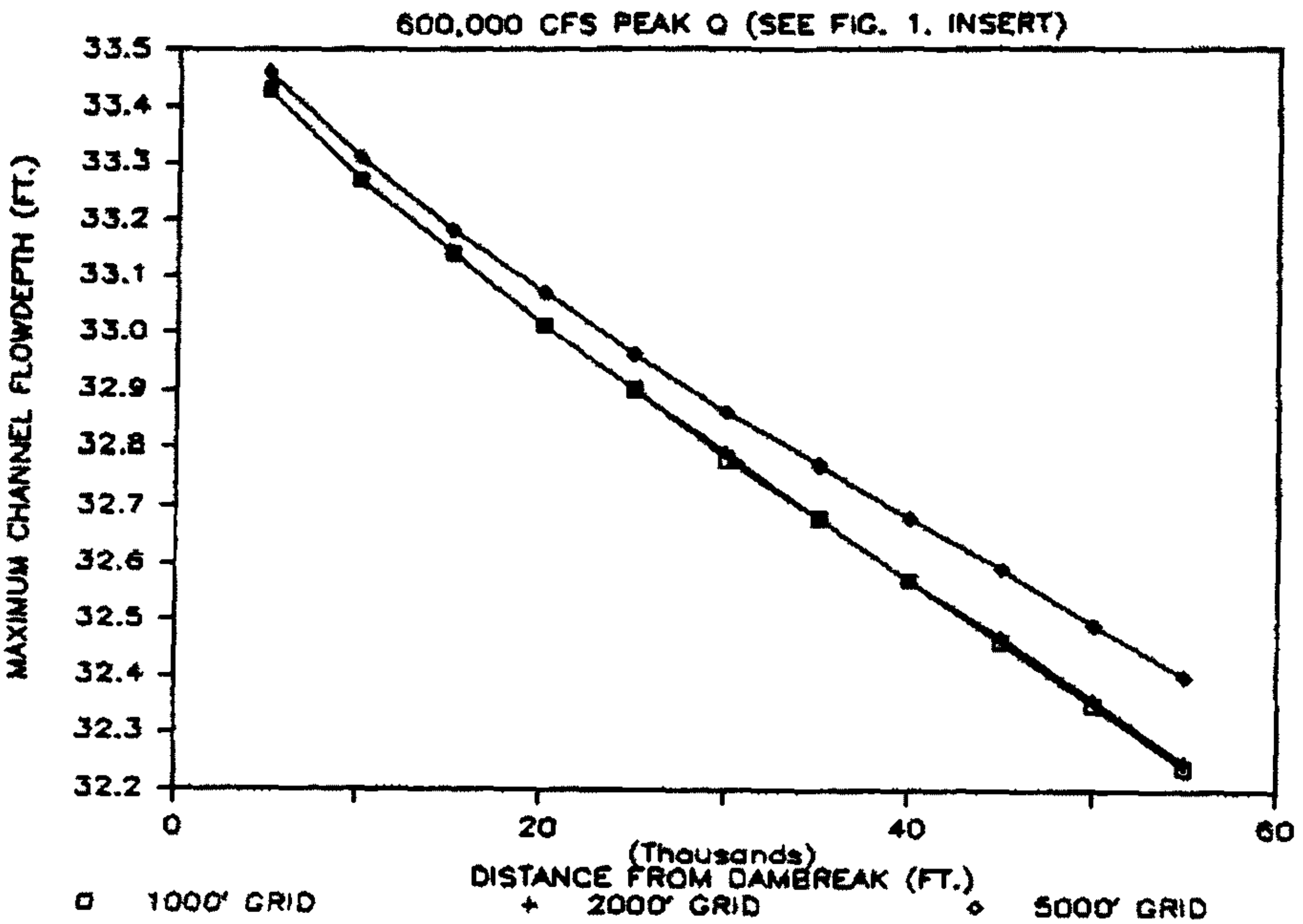
For the dam-break hydrographs considered and the range of channel slopes modeled, the simple diffusion dam-break model of Eq. (11) provides estimates of flood depths which compare favorably to the flood depths determined by the well-known K-634 one-dimensional dam-break model. Generally speaking, the difference between the two modeling 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 Eq. (11) 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 dam-break model is tested by modeling 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 modeling technique by considering the evolution of a two-dimensional flood plain which propagates radially from the dam-break site.

II.8 DISCUSSION

From the above conclusions, use of the diffusion approach of Eq. (11) in a two-dimensional dam-break model may be justifiable due to the low variation in predicted 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. Assuming a two-dimensional diffusion dam-break model is applied to such an analysis, the deviation in predicted flood depths by not including the inertia terms may be considerably offset by the uncertainty associated with an assumed dam-break failure rate, and the assumed deterioration shape of the dam (e.g. Xanthopoulos and Koutitas, 1976). An additional major uncertainty is the assumed friction factors assigned to the nodal points. Because such uncertainties remain with a predictive dam-break analysis, further refinement of the diffusion modeling approach by including the inertia terms may not be needed.



Effects of Differing Grid Size

Fig. 3.

III. A TWO-DIMENSIONAL DIFFUSION DAM-BREAK MODEL

III.1 INTRODUCTION

The probable flooding damages which may occur due to a dam failure is of concern to many civil engineers and planners. Not only do such studies provide a source of information for insurance and flood control studies, but the actual planning process for the construction of a dam site can be modified by the results of such a predictive analysis.

Generally, such dam-break studies can be completed by either scaled hydraulic models or by use of computer simulation. The most cost effective approach is to approximately solve the governing flow equations of momentum and continuity by computer simulation, where several situations can be considered by a fraction of the cost of a scaled prototype model.

For water courses in which flows can be classified as one-dimensional, several models are presented and verified in the literature. One widely used one-dimensional model which is utilized in this study is the U.S.G.S. K-634 model version developed by Land (1980a,b).

However for flow conditions which are truly two-dimensional, such as occur when massive dam-break flows exceed the water course channel capacities and excess flows break out and travel away from the water course, then a one-dimensional approximation may be inappropriate.

In this section of the report, a two-dimensional dam-break model 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 of this report considers a one-dimensional diffusion model and concludes that for low-to-moderate velocity flow regimes (i.e., less than approximately 25 feet/sec), the diffusion model is a reasonable approximation of the full dynamic wave formulation.

In this section, a finite difference grid model is developed which equates each cell-centered node to a function of the four neighboring cell nodal points. To verify 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) is considered.

III.2 STUDY AREA DESCRIPTION

The Owens River drains the rugged eastern slopes of the Sierra Nevada Range, the Benton Range, and the western slopes of White Mountains located in Inyo County, California (Fig 4). The head waters of the Owens River flows into Lake Crowley (formed by the Long Valley dam). The dam is an earthen type, 126 ft in height with a crest length of 595 ft, and a total storage capacity of 183,465 acre-feet of water. Downstream from Long Valley Dam, the Owens River is constrained in an incised canyon of volcanic Bishop tuff formation. The river flows through the gorge to the canyon mouth, seventeen miles downstream from the dam. The river encounters several hydroelectric power plants in reaching the canyon mouth. However in this study, the effects of these obstructions in a dam-break analysis is assumed minor due to the relative magnitude of the flood volume compared to the volume afforded by each power plant.

Through the first several miles downstream from the canyon mouth, the river meanders through alluvial fans with a slope of 50 ft per mile. Four miles downstream from the canyon mouth, the river enters Owens Valley which is a flat alluvial valley with a slope of only 10 ft per mile. (The basin elevations range from 4300 ft to 3800 ft). The river curves around the City of Bishop about 17 miles downstream from the canyon mouth, and then meanders through Owens Valley for another 40 miles until it reaches Tinemaka Reservoir. Figures 4 and 5 illustrate the study location and vicinity.

III.3 U.S.G.S. K-634 ONE-DIMENSIONAL ANALYSIS APPROACH

In this section, the one-dimensional dynamic wave, implicit dam-break model K-634 developed by Land (1980a,b) is used to investigate a hypothetical dam failure and resulting flooding along the Owens River.

The K-634 model routes a flood hydrograph through a reservoir with a water level initially below the position where a breach in the retaining dam is assumed to occur. The routing of this hydrograph is accomplished by one of two procedures; namely, a hydrologic method (storage-continuity), or a hydraulic method such as discussed in Land (1980a,b). When the water level in the reservoir reaches a preselected level, a breach in the dam is assumed to begin. The breach begins with a zero width, and widens and deepens during a specified time of failure. At the end of the failure, the breach is fully developed and is assumed to remain constant for the remainder of the simulation. The breach flow rates are computed based on a trapezoidal-shaped critical flow area. The outflow hydrograph at the dam is computed as the sum of the initial flow through dam structures and the flow through the breach.

The dam-break flood is then mathematically routed downstream by solving the one-dimensional Saint Venant flow equations using a nonlinear implicit finite-difference algorithm.

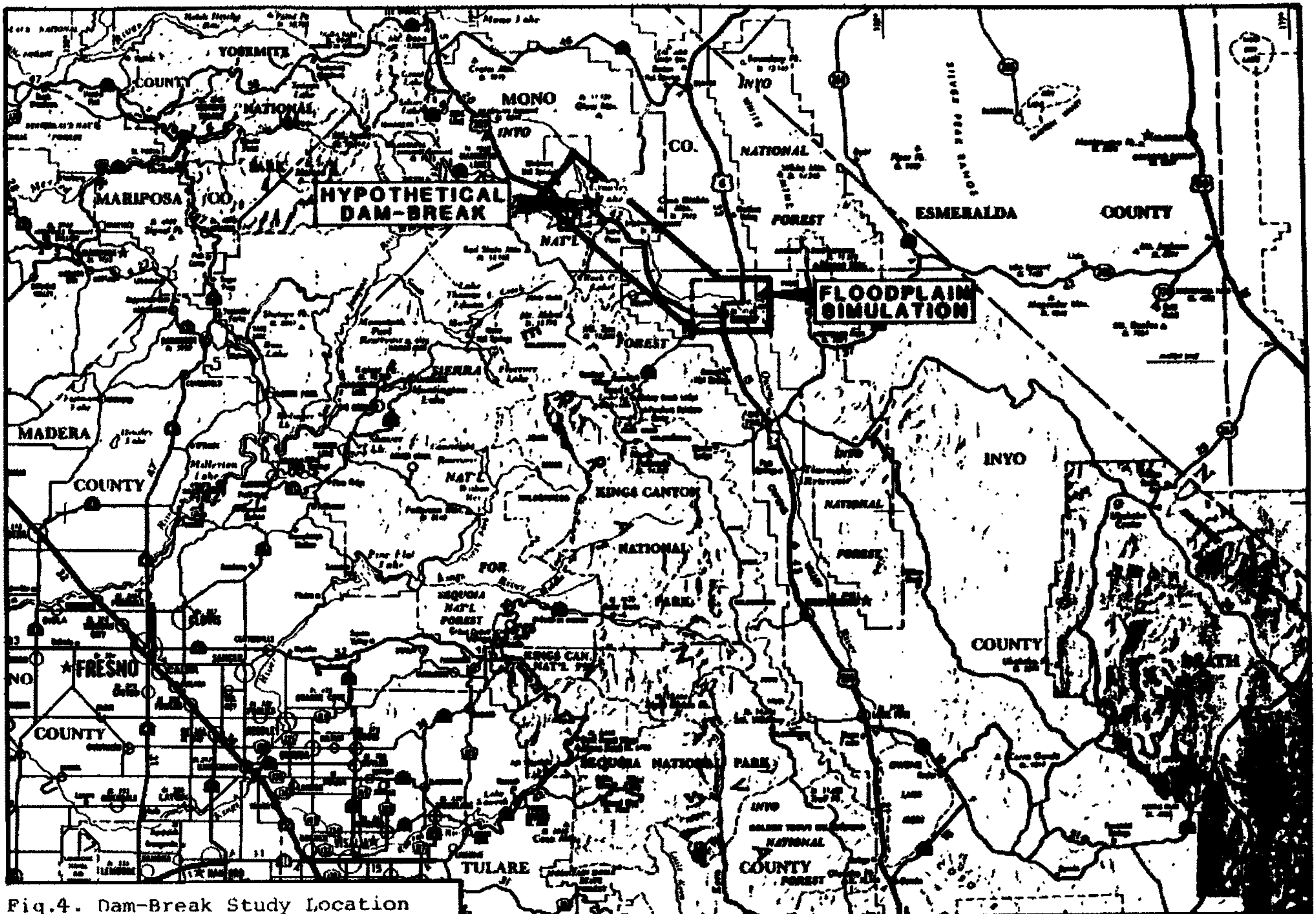


Fig.4. Dam-Break Study Location

III.4 ONE-DIMENSIONAL MODEL DATA REQUIREMENTS

The K-634 data requirements fall into three categories. (1) In the reservoir: surface area versus elevation tables, or channel geometry and roughness; (2) at the dam: breach shape, duration of breach development, dam outlet structures stage-outflow ratings, and water-surface elevation when failure begins to occur; and (3) in the stream: channel geometry, roughness and state of flow (e.g. subcritical or supercritical). The upstream boundary condition for routing the flood downstream is the dam-break outflow hydrograph. The downstream boundary condition is a dynamic state-discharge relation which is computed from the full dynamic flow equations.

The one-dimensional model grid schemetization is shown in Fig. 5. The channel geometry of the reach of river was defined by field-surveyed cross sections where, for each section, a maximum of eight elevation versus top width data pairs were specified. Due to the relatively flat topography perpendicular to the main channel (see fig 5), the upper most portions of the channel cross-sections were assumed to have a mild gradient. This modeling assumption results in a false confinement of channel flows due to a fictitious boundary being assumed on both sides of the channel. Consequently, predicted flood plains will be approximate at best, and requires additional interpretation. The effects of this modeling assumption will be further discussed in a subsequent section.

III.5 MATHEMATICAL FORMULATION FOR TWO-DIMENSIONAL MODEL

The Saint Venant equations for two-dimensional flow may be written as

$$\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} + \frac{\partial H}{\partial t} = 0 \qquad (1)$$

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

$$\frac{\partial Q_y}{\partial t} + \frac{\partial}{\partial y} Q_y^2/A_y + \frac{\partial}{\partial x} Q_y Q_x/A_x + gA_y \left[S_{fy} + \frac{\partial H}{\partial y} \right] = 0 \qquad (3)$$

where

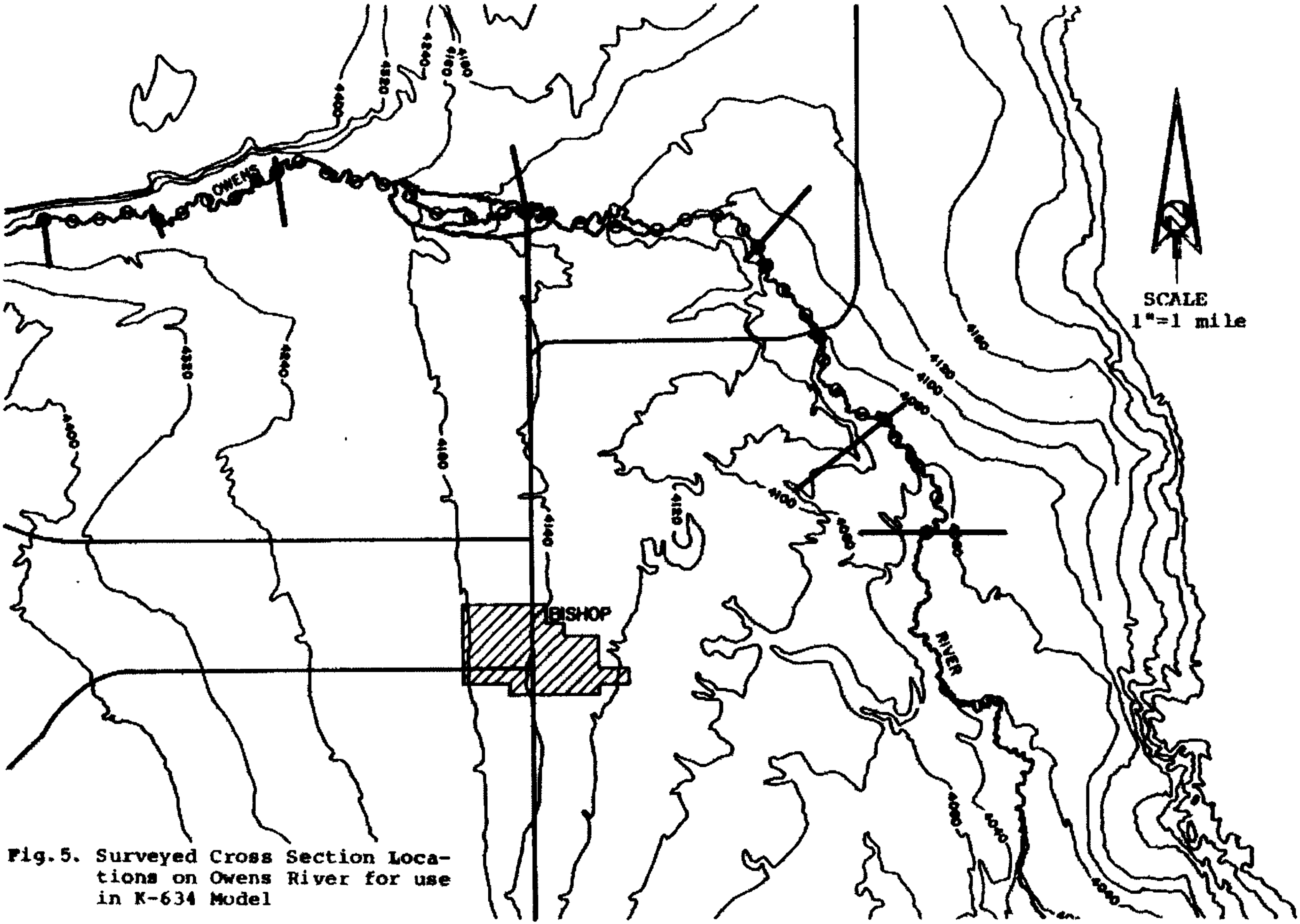


Fig. 5. Surveyed Cross Section Locations on Owens River for use in K-634 Model

Q_x, Q_y	=	flow rates in x,y – directions
q_x, q_y	=	flow rates/length in x,y – directions
A_x, A_y	=	directional cross-section areas
S_{fx}, S_{fy}	=	directional friction slopes
x, y, t	=	spatial and temporal coordinates
g	=	gravity
H	=	water surface elevation

where, for a grid discretization (Fig 6), Q_x equals the product of q_x and the grid width, δ. The equation of continuity (Eq. 1) is based on the assumption of constant fluid density with zero sources or sinks in the flow field. The x- and y-direction momentum relations (Eqs. 2 and 3) assume hydrostatic pressure distributions.

The local and convective acceleration momentum terms can be grouped together such that Eq. 1, 2 and 3 are rewritten as

$$m_z + \left(S_{fz} + \frac{\partial H}{\partial z} \right) = 0 , \quad z = x,y \qquad (4)$$

where m_z represents the sum of the first three terms in Eqs. (2) and (3), and divided by gA_z. Assuming the friction slope to be approximated by steady flow conditions, the Manning's equation in inch-pound units can be used to estimate

$$Q_z = \frac{1.486}{n} A_z R_z^{2/3} S_{fz}^{1/2} \qquad (5)$$

where

A_x, A_y = directional flow areas
R_x, R_y = directional hydraulic radii

Equation 5 can be rewritten as

$$Q_z = - K_z \frac{\partial H}{\partial z} - K_z m_z , \quad z = x,y \tag{6}$$

where

$$K_z = \frac{1.486}{n} A_z R_z^{2/3} \Big/ \left| \frac{\partial H}{\partial z} + m_z \right|^{1/2} , \quad z = x,y \tag{7}$$

Hromadka (1984), Akan and Yen (1981), and Xanthopousos and Koutitas (1976) assume m_x and m_y are both negligible, resulting in the simple diffusion model (see preceding report section)

$$Q_z = - K_z \frac{\partial H}{\partial z} \tag{8}$$

In Eq.(6), m_z can be retained in either the full form, or can include either the local or the convective acceleration component. However, Hromadka (1984) showed that for one-dimensional dam-break flood waves on flat plains where flow velocities are generally below about 25 ft/sec m_z is generally small, and the loss of accuracy incurred by setting $m_z = 0$ may be considered acceptable when considering the modeling errors due to the wide range of model parameter variability which occurs in this type of analysis as a result of the assumed dam-break mode of failure, failure rate and shape, watershed friction factor distribution and variation during the dam-break, and other factors. Nevertheless, m_z can be included in Eq. 6 resulting in the complete Saint Venant formulation; however, the resulting computer computation effort increases by approximately 50-percent due to the cross-product evaluation of the convective acceleration terms and the local acceleration term approximations.

The proposed two-dimensional dam-break model is formulated by substituting Eq. 8 into the continuity equation giving

$$\frac{\partial}{\partial x} K_x \frac{\partial H}{\partial x} + \frac{\partial}{\partial y} K_y \frac{\partial H}{\partial y} = \frac{\partial H}{\partial t} \qquad (9)$$

If the momentum term groupings were retained, Eq. 9 would be written as

$$\frac{\partial}{\partial x} K_x \frac{\partial H}{\partial x} + \frac{\partial}{\partial y} K_y \frac{\partial H}{\partial y} + S = \frac{\partial H}{\partial t} \qquad (10)$$

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.

III.6 NUMERICAL MODEL FORMULATION (GRID ELEMENTS)

For uniform grid elements, the numerical modeling approach used is the integrated finite difference version of the nodal domain integration (NDI) method. For grid elements, the NDI nodal equation is based on the usual nodal system shown in Fig. 6 Flow rates along the boundary Γ are estimated using a linear trial function assumption between nodal points.

For a square grid of width δ

$$Q\Big|_{\Gamma_E} = -\left(K_x\Big|_{\Gamma_E}\right)\left(H_E - H_C\right) \Big/ \delta \qquad (11)$$

where

$$K_x\Big|_{\Gamma_E} = \begin{cases} 1.486\left(A\,R^{2/3}/n\right)_{\Gamma_E} \Big/ \left|\left(H_E - H_C\right)\Big/\delta\right|^{1/2} ; \; \bar{D} > 0 \\ 0 \; ; \; \bar{D} \leq 0 \text{ or } |H_E - H_C| < 10^{-3} \end{cases} \qquad (12)$$

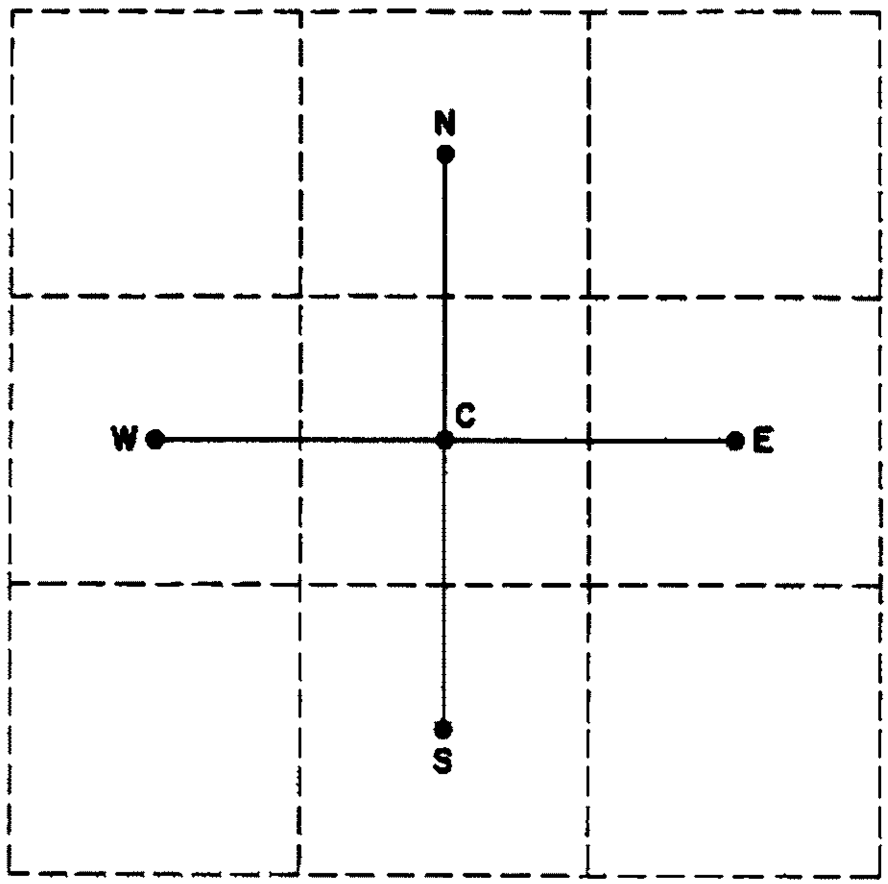


Fig 6. Grid Element Nodal Molecule

In Eq. 12, the terms A, R, and n are evaluated at the average flow depth defined by $D = (D_E + D_C)/2$, and $n = (n_E + n_C)/2$. Additionally, the denominator of K_x is checked such that K_x is set to zero of $|H_E - H_C|$ is less than a tolerance such as 10^{-3} ft.

The model advances in time by an explicit approach

$$\underset{\sim}{H}^{i+1} = \underset{\sim}{K}^{i}\ \underset{\sim}{H}^{i} \tag{13}$$

where the assumed dam-break flows are added to the specified input nodes at each timestep. After each timestep, the conduction parameters of Eq. 12 are reevaluated, and the solution of Eq. 13 reinitiated. Using grid sizes with uniform lengths of one-half mile, timesteps of size 3.6 sec were found satisfactory.

III.7 TWO-DIMENSIONAL MODEL DATA REQUIREMENTS

The two-dimensional model requires only the natural ground topography and estimated Manning's friction factor for data input. Consequently, the diffusion model will provide useful dam-break flood-plain estimates with only minimum data requirements which are usually readily available. For the grid model, a nodal molecule associates the central node to the north, east, south, and west nodal points. Zero flow boundary conditions are easily accommodated by entering a zero for the appropriate nodal molecule position. The nodal point elevation is based on an estimated average elevation for the assumed grid cell.

The model includes a critical depth (specified) boundary condition whereby flow depths may be assumed to correspond to critical flow conditions.

Finally, the inflow hydrograph may be specified at one or more nodal points. In this study, the form of this hydrograph was specified as the change in flow rate per 5-minute unit period. This time derivative of inflow is used to provide a smooth inflow hydrograph stepped according to the specified model time step.

III.8 CASE-STUDY RESULTS AND DISCUSSION

The K-634 model was initially applied to the steep canyon reach immediately downstream of the assumed dam-break. The modeling results indicated negligible attenuation of the flood wave peak and, consequently, subsequent studies of the two-dimensional plain were assumed to have the dam-break located at the downstream point of the canyon, neglecting the effects of the long canyon reach.

Applying the K-634 model to computing the two-dimensional flow was attempted by means of the one-dimensional nodal spacing shown in Fig. 5.

Cross sections were obtained by field survey, and the elevation data used to construct nodal point flow-width versus stage diagrams for use in the K-634 model. A constant friction factor (Manning's) 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. 7. As discussed in a previous section, a slight gradient was assumed for the topography perpendicular to the main channel. The motivation for specifying such a gradient was to limit the channel floodway section in order to approximately conserve the one-dimensional momentum equations. As a result of this assumption, fictitious channel sides are included in the K-634 model study which results in an artificial confinement of the flows. Thusly, a narrower flood plain is delineated (such as shown in Fig. 7) where the flood flows are falsely retained within a hypothetical channel confine. An examination of the flood depths given in Fig. 9 indicates that at the widest flood plain expanse of Fig. 7, the flood depth is about 6-feet, yet the flood plain is not delineated to expand southerly, but is modeled 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.

The two-dimensional diffusion model was applied to the total flood plain area using the grid discretization shown in Fig. 8. The same inflow hydrograph produced by the K-634 model was used for both two-dimensional simulations. Again, the Manning's friction factor of 0.04 was used. The resulting flood plain is shown in Fig. 8 for the ¼-square-mile grid model.

Comparisons of predicted maximum water elevations are shown in Fig. 9 which plots K-634 modeling results and the two-dimensional modeling 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 traveling 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 modeling results are based on cross-sections which traverse a 90-degree bend. In this case the 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,

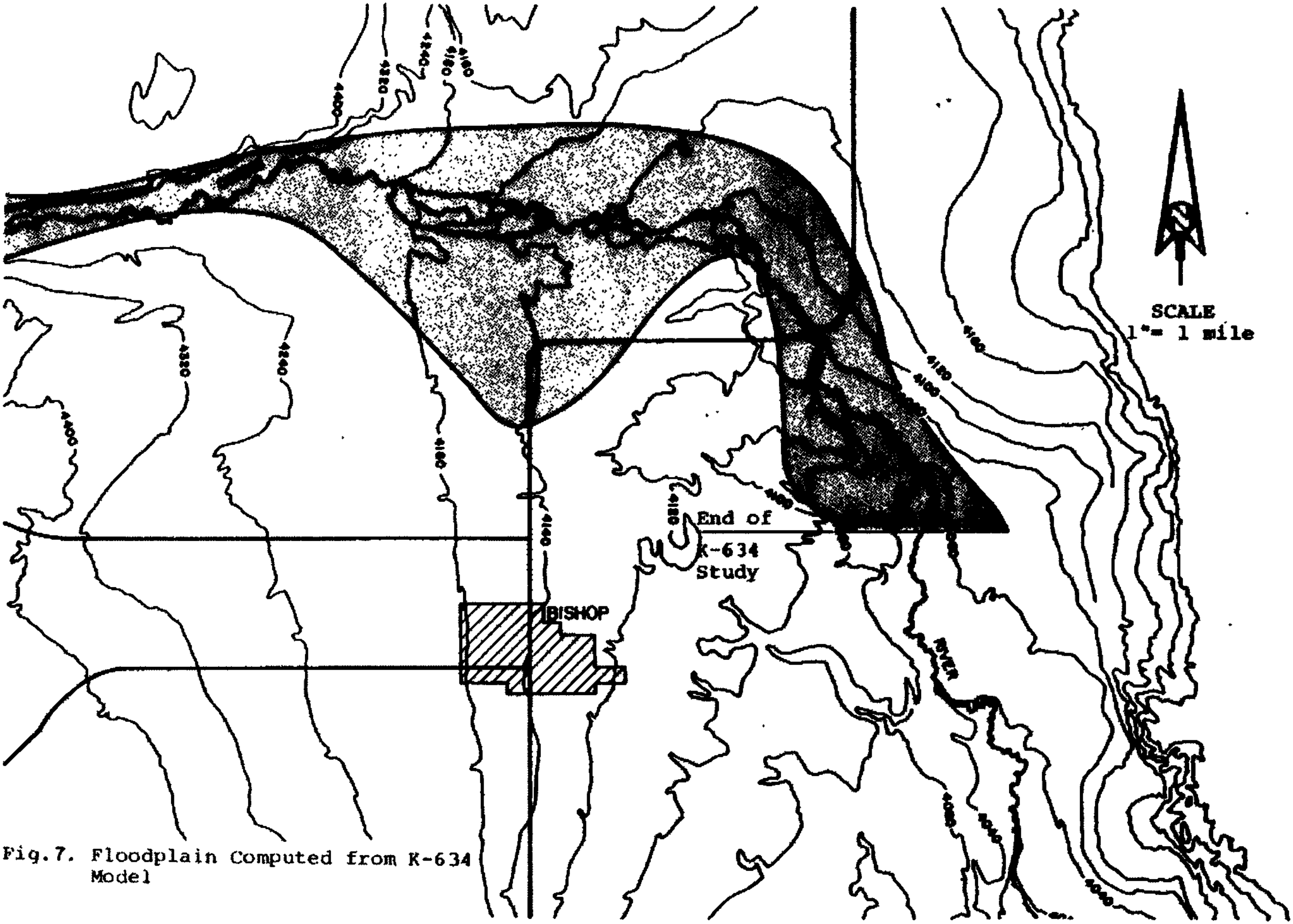


Fig.7. Floodplain Computed from K-634 Model

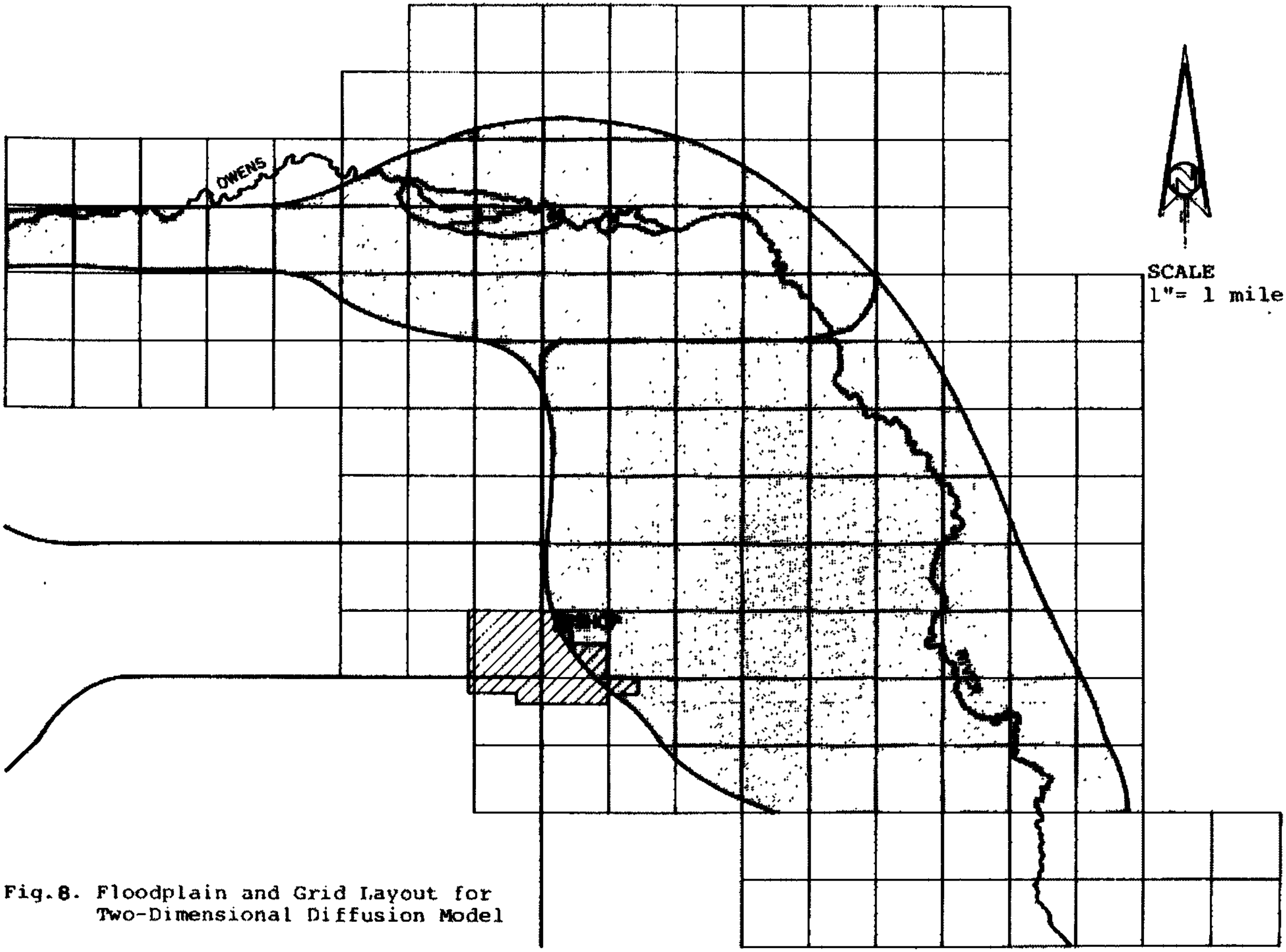


Fig.8. Floodplain and Grid Layout for Two-Dimensional Diffusion Model

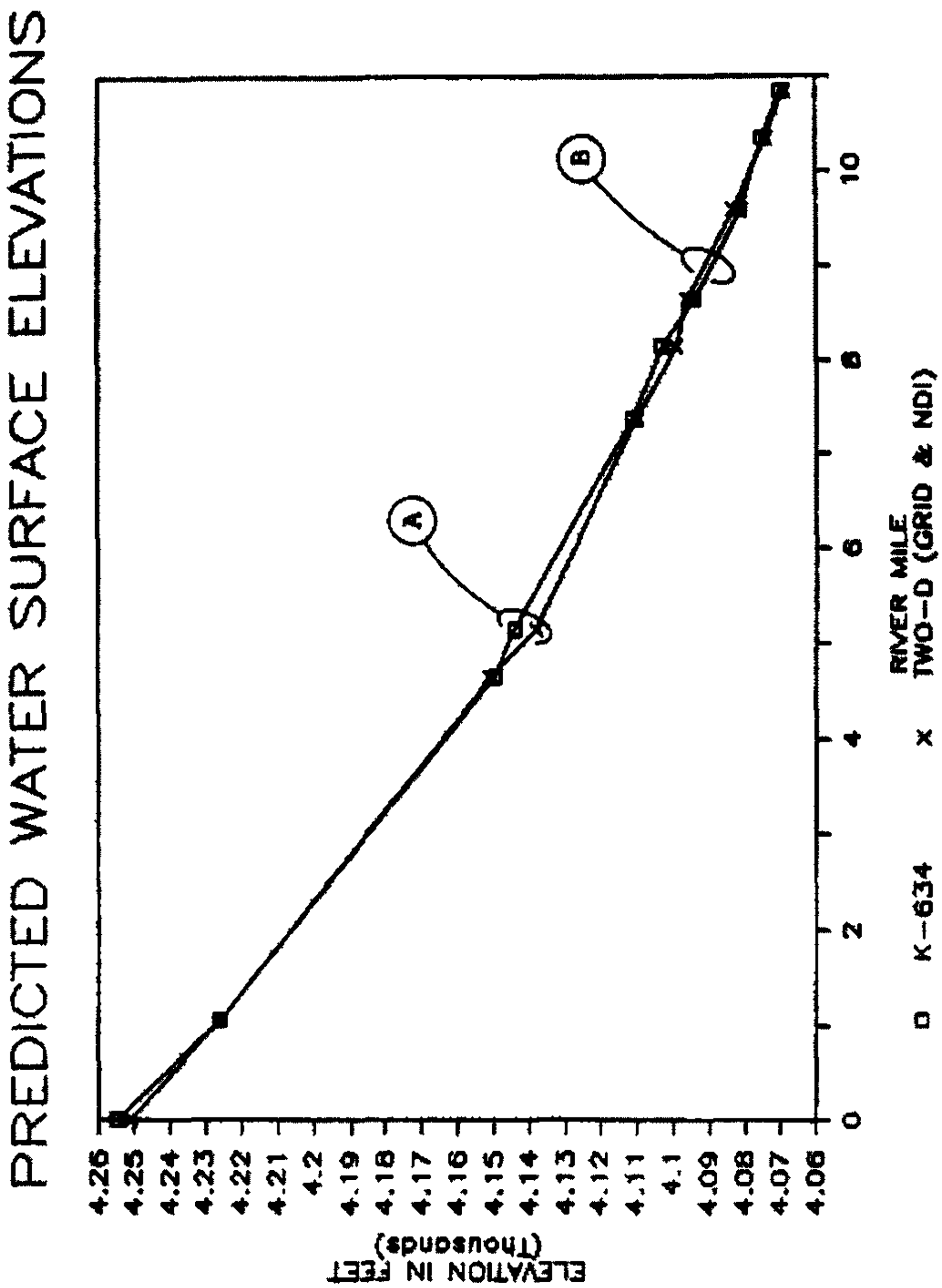


Fig.9. Comparison of Modeled Water
Surface Elevations

it is seen that the two-dimensional diffusion model produced more reasonable predictions of the two-dimensional flood plain characteristics which are associated with broad, flat plains such as found at the study site than the one-dimensional model. The two-dimensional model affords approximation of channel bends, channel expansions and contractions, flow breakouts, and the general area of inundation. Additionally, the two-dimensional modeling approach allows for the inclusion of return flows (to the main channel) which result due to upstream channel breakout flows, and other two-dimensional flow effects without the need for special modeling accommodations which would be required when using a one-dimensional model.

III.9 MODEL SENSITIVITY

The sensitivity of the overall modeling approach is illustrated for the case of K-634 by examining the three parameters of nodal spacing, Manning's n, and dam-break time of failure. The base run values are a nodal spacing of 0.25 mile, Manning's friction factor of 0.04, and a 1-hour dam-failure duration.

It can be seen in Table 1 that the most sensitive parameter is the friction factor, followed by the relatively insensitive parameters of time of failure and the nodal spacing. Similar sensitivity is found for the diffusion model.

TABLE I

K-634 SENSITIVITY ANALYSIS

	RIVER MILE			
	2.0		8.4	
	MAXIMUM FLOWRATE (cfs)	MAXIMUM ELEVATION	MAXIMUM FLOWRATE (cfs)	MAXIMUM ELEVATION
NODE SPACING				
0.25 mi*	408,038	4208.80	384,717	4098.58
0.30 mi	412,266	4208.98	386,164	4098.60
0.40 mi	412,082	4209.11	385,878	4098.59
0.50 mi	412,129	4209.12	384,828	4098.99
TIME OF FAILURE				
0.50 hr	420,301	4209.04	388,965	4098.64
1.0 hr*	408,038	4208.80	384,717	4098.58
1.5 hr	400,352	4208.66	380,050	4098.49
1.75 hr	399,380	4208.64	377,584	4098.45
2.0 hr	394,846	4208.56	374,338	4098.39
MANNING'S n				
0.04*	408,038	4208.80	384,717	4098.58
0.045	411,221	4209.82	382,273	4099.35
0.05	410,386	4210.74	378,310	4100.05

* base run used for flood plain study purposes

III.10 CONCLUSIONS

A simple two-dimensional diffusion model was developed which can be based on a finite-difference grid discretization. The keystone of the two-dimensional model is the diffusion version of the Saint Venant flow equations. Application of the model to the prediction of two-dimensional dam-break flood waves indicate that the diffusion model provides a significant advantage over the corresponding one-dimensional models in the study of flood plains such as occurs from a dam-break. The two-dimensional model is simple to use, requires readily available data, and does not need special modeling techniques to approximate two-dimensional flood flow effects.

IV. APPLICATION OF A DAM-BREAK MODEL TO THE ORANGE COUNTY RESERVOIR

IV.1 INTRODUCTION

The main objective of this report section is to develop a probable dam-break flood plain which may occur from a hypothetical failure of the Orange County Reservoir.

The method used to estimate this hypothetical dam-break flood plain is the two-dimensional diffusion dam-break model of section III.

IV.2 FLOOD PLAIN DISCRETIZATION AND PARAMETERS

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. 10.

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, $n = 0.050$).

IV.3 MODELING ASSUMPTIONS

Major assumptions used in this study are as follows:

(1) Friction effects are modeled by Manning's equation as used in the two-dimensional diffusion model.

(2) All storm drain systems provide negligible draw off of the dam-break flows. This assumption accommodates 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 material.

(4) The reservoir failure conforms to an outflow hydrograph such as shown in Fig. 11.

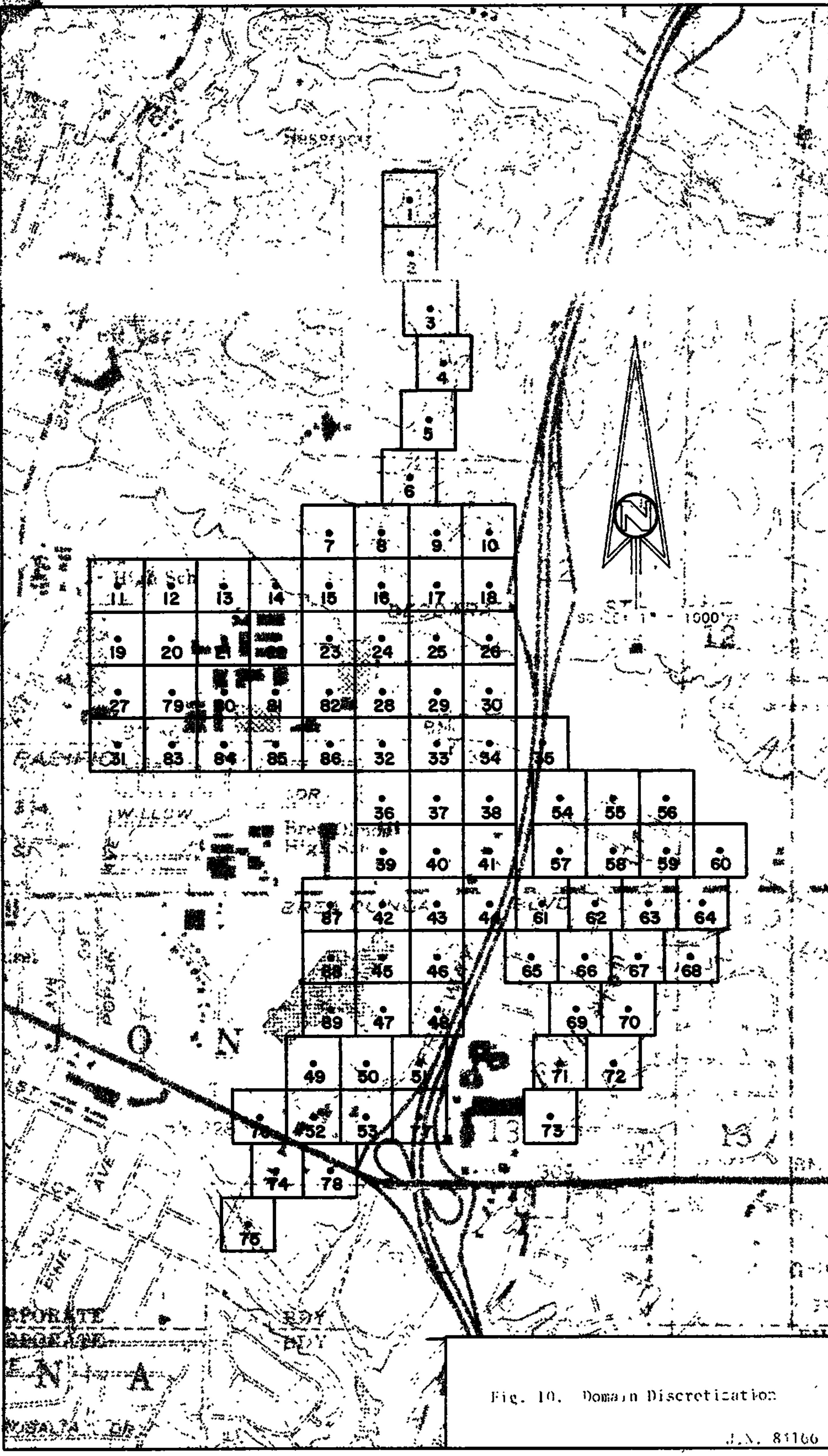


Fig. 10. Domain Discretization

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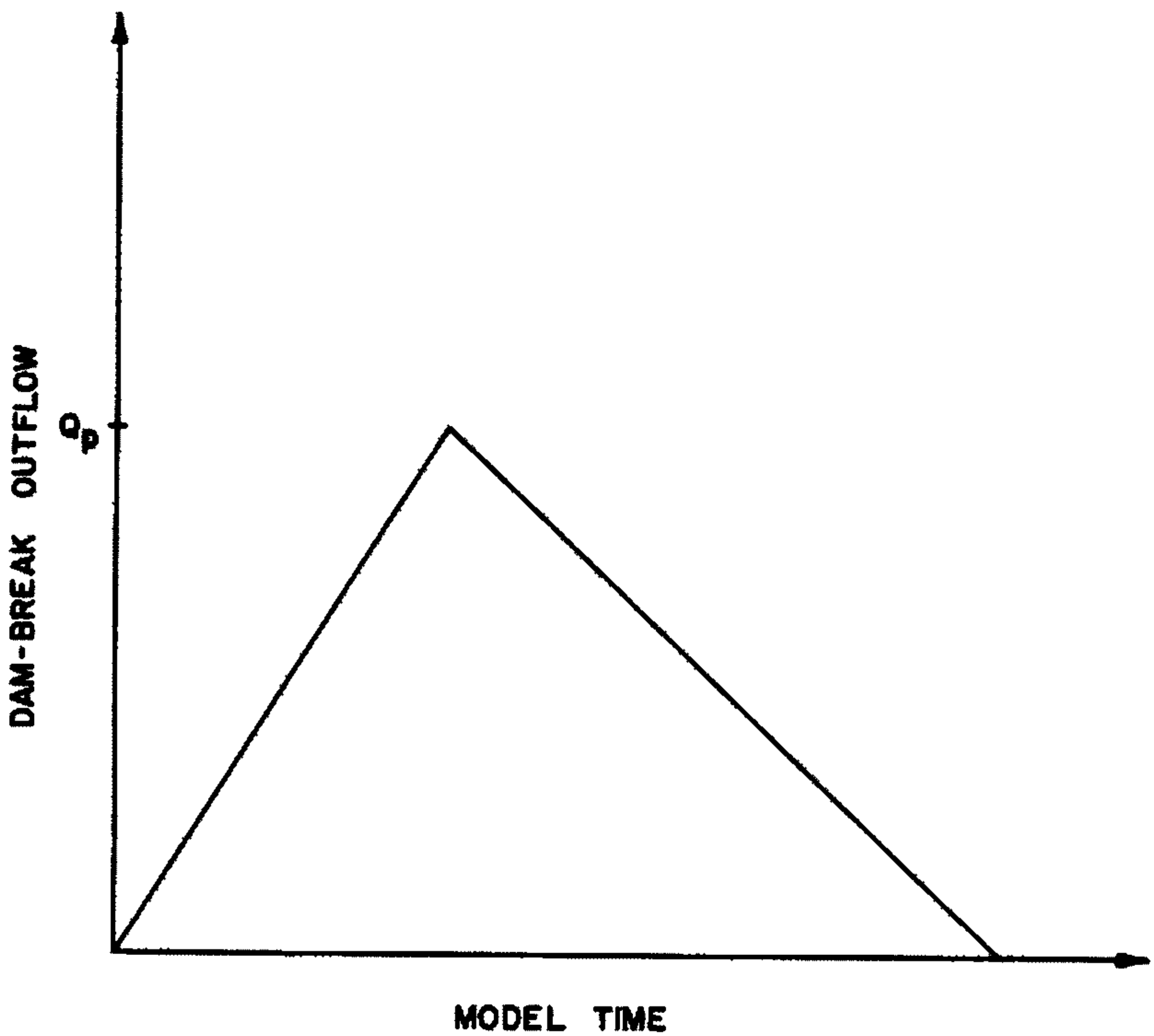

Fig. 11. Example Outflow Hydrograph

IV.4 DAM-BREAK FAILURE MODE

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 2. 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 3.

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 peak 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 (lined with concrete on the basin interior) with flow velocities less than about 30 fps. Consequently, the estimated Q_p may be considered conservative.

IV.5 DAM-BREAK FLOOD PLAIN ESTIMATION

The dam-break hydrograph of Fig. 12 was used as on the inflow hydrograph to the grid network of Fig. 10. The resulting flood-plain (using the dam-break model) is shown in Fig. 13. From Figure 13 it is seen that the northeasterly portion of the Brea Mall is predicted to be subject to approximately 1.5-foot depth of flooding. This portion of the mall represents a lower level of the complex with parking lots located between the mall and State College Boulevard.

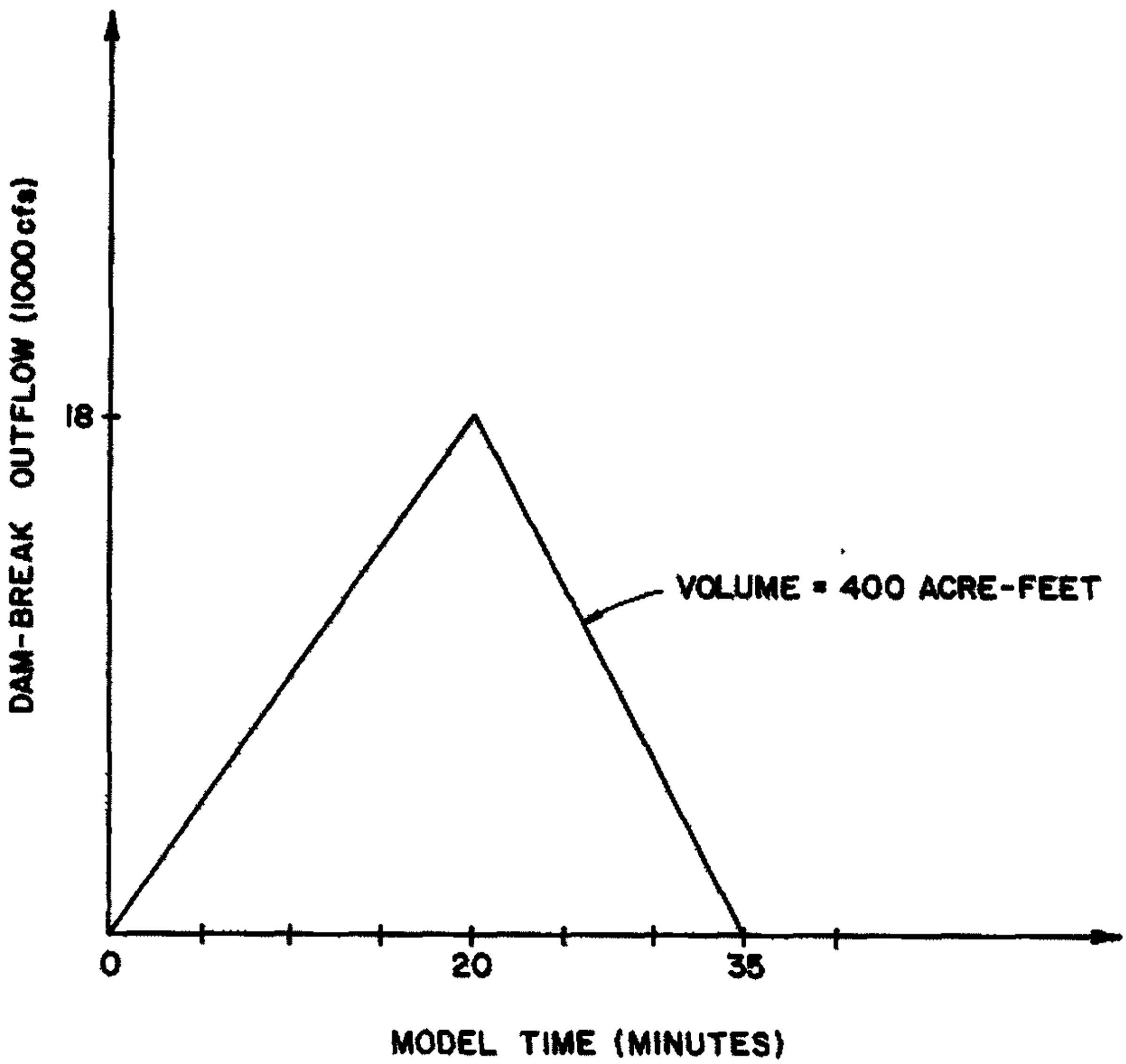

Fig. 12. Study Dam-Break Outflow Hydrograph

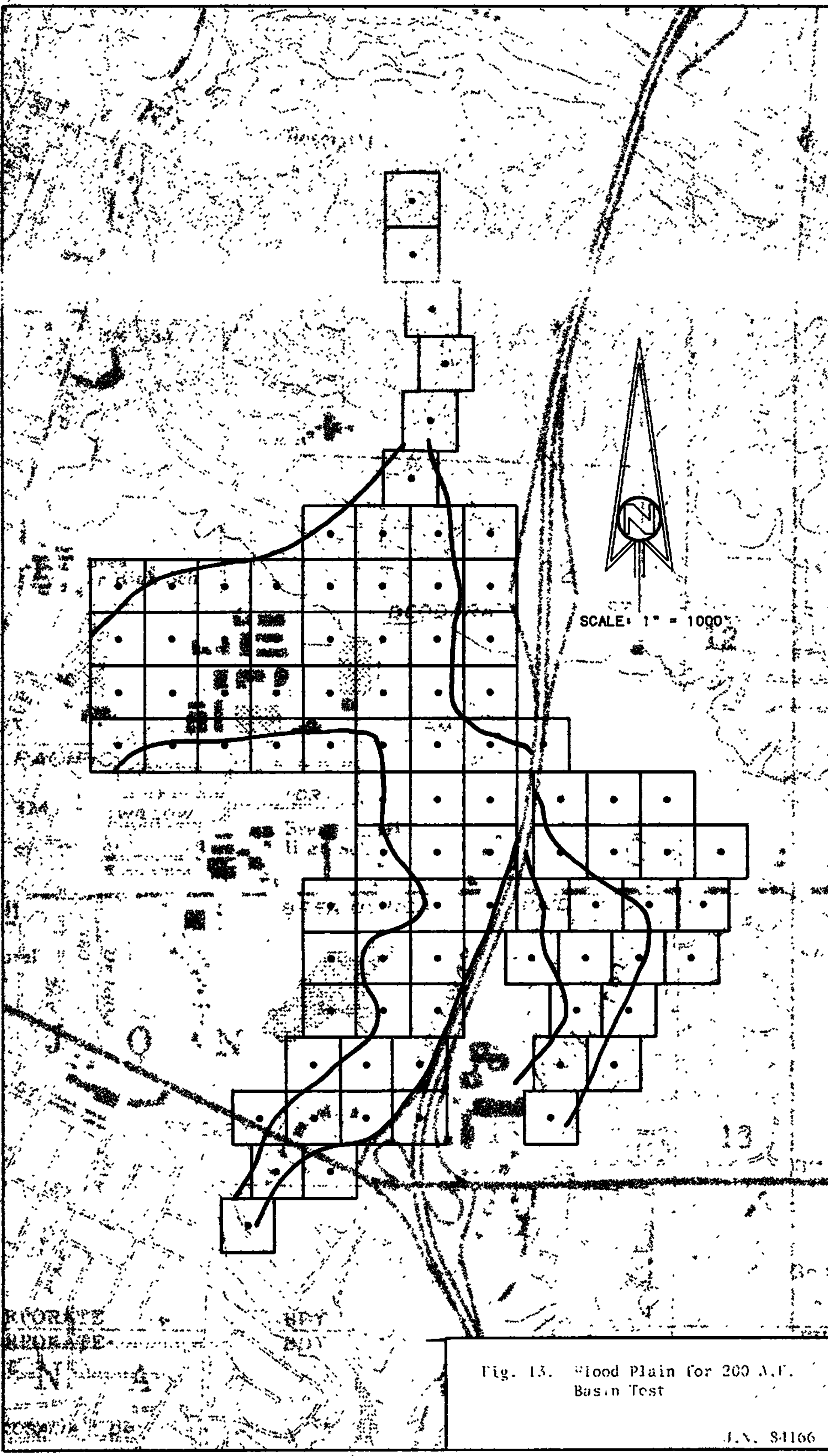

Fig. 13. Flood Plain for 200 A.F. Basin Test

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It is noted, however, that the uncertainty in modeling results may be significant due to the modeling effort being based upon 20-foot contour U.S.G.S. topographic maps. To aid in reducing this uncertainty, several site examinations were conducted in order to verify the grid schematic representation of the problem domain and the reasonableness in modeling results. The model discretization was adjusted when considered appropriate to better represent field conditions and subsequent modeling results rechecked by additional field investigations.

In order to develop more refined modeling 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 14. 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 modeling 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 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. 15. 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 hypothetic dam-failure of the Orange County Reservoir. In this second analysis, the flooding depth is estimated to increase to about 2.5 feet.

Figure 16 shows lines of arrival times for the second (volume doubled) basin test study. Computer results for this second test study are provided in Appendix B of this report.

IV.6 CONCLUSIONS

A two-dimensional diffusion dam-break model (section III) is used to estimate the flood plain resulting from a hypothetical failure of the Orange County Reservoir north of the City of Brea. From the study, it is concluded that the estimated flood plain is reasonable, and is based upon a modeling approach which accounts for the time-varying effects of routing, storage, and an earthen dam-failure process. It is also concluded that the Brea Mall site is essentially unaffected by a hypothetical failure of the Orange County Reservoir, except for the northeastern portion of the complex where the lower level is predicted to be flooded by about 1 to 2 feet of water.

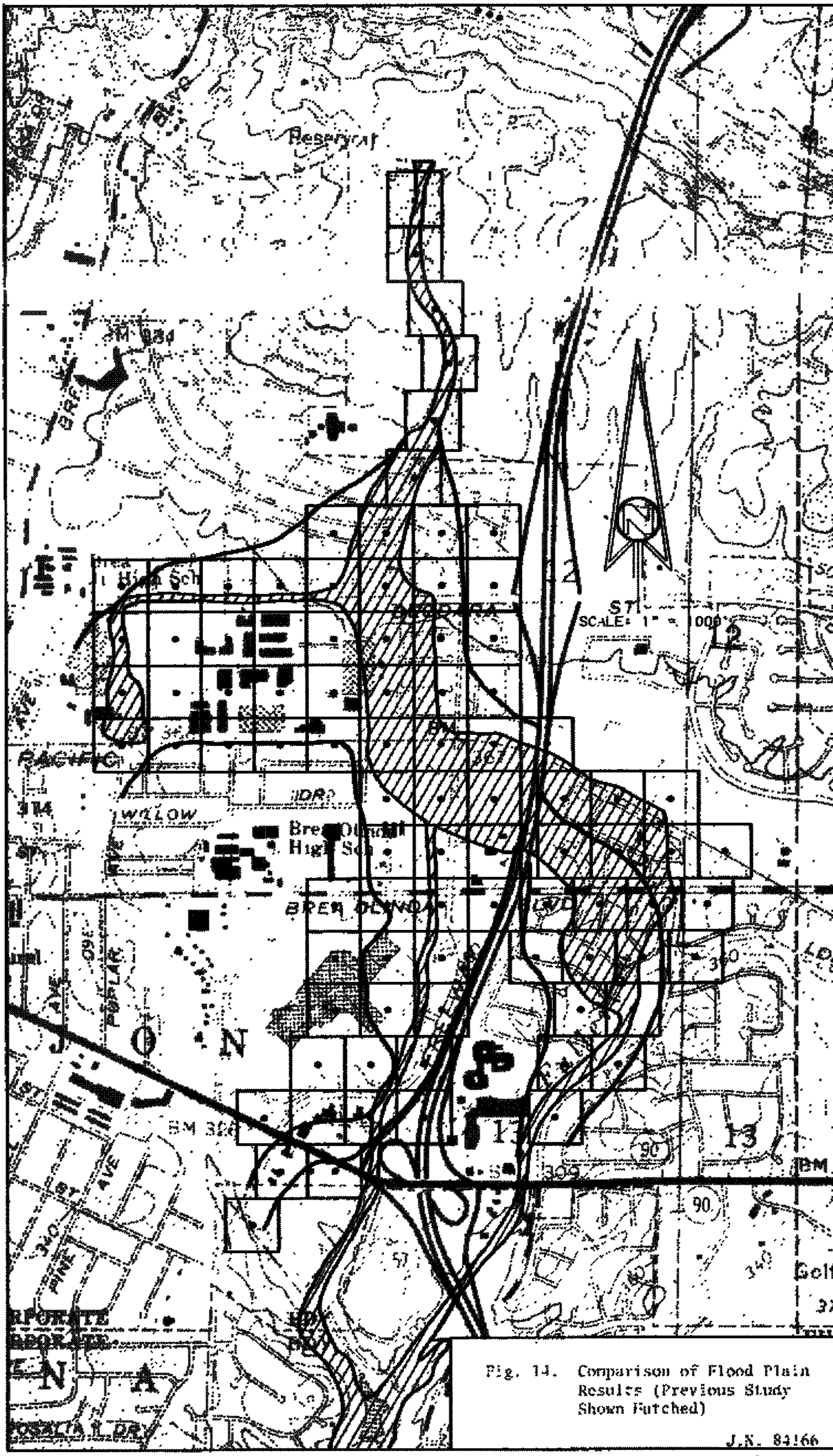

Fig. 14. Comparison of Flood Plain Results (Previous Study Shown Hatched)

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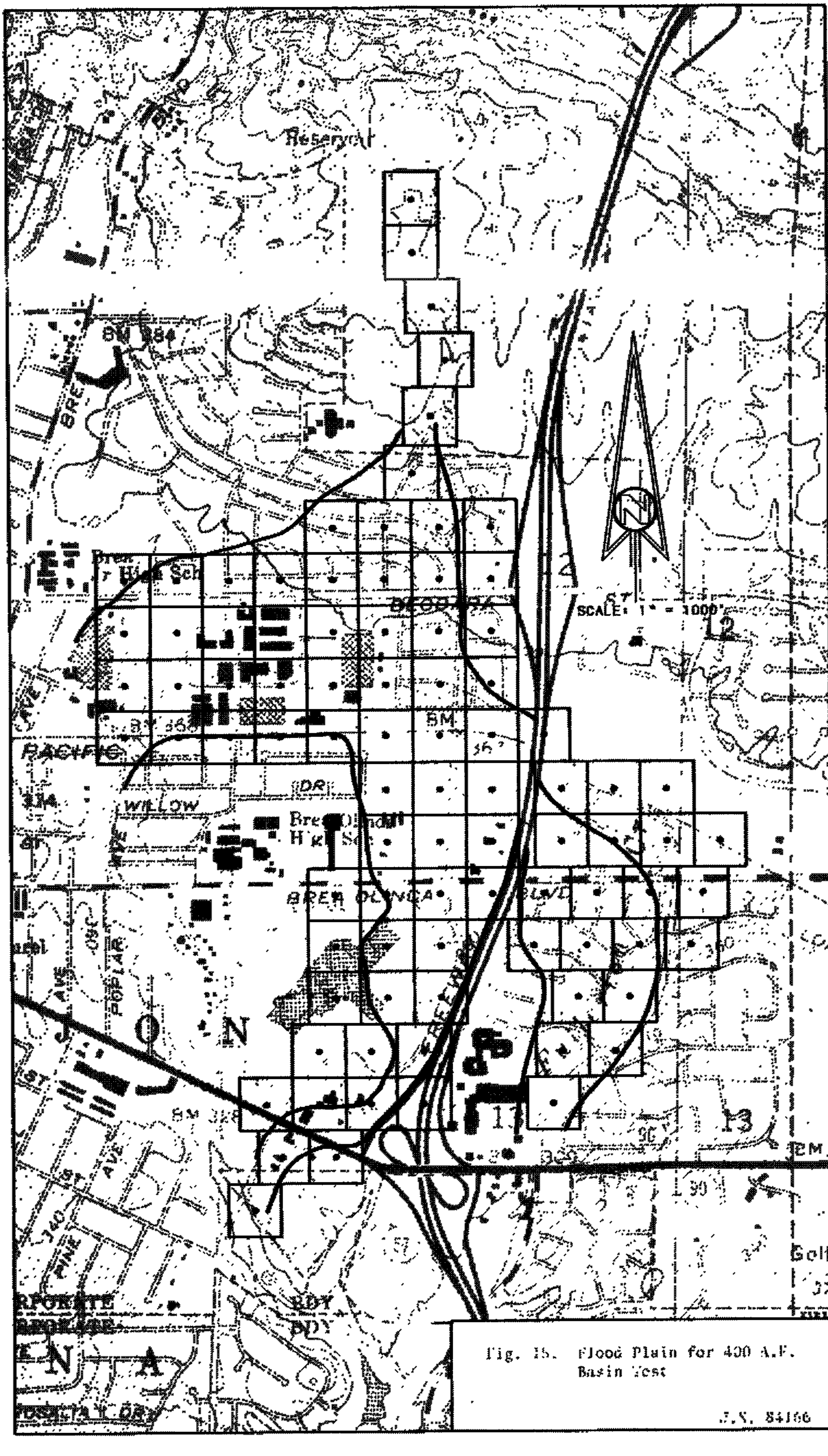

Fig. 15. Flood Plain for 400 A.F. Basin Test

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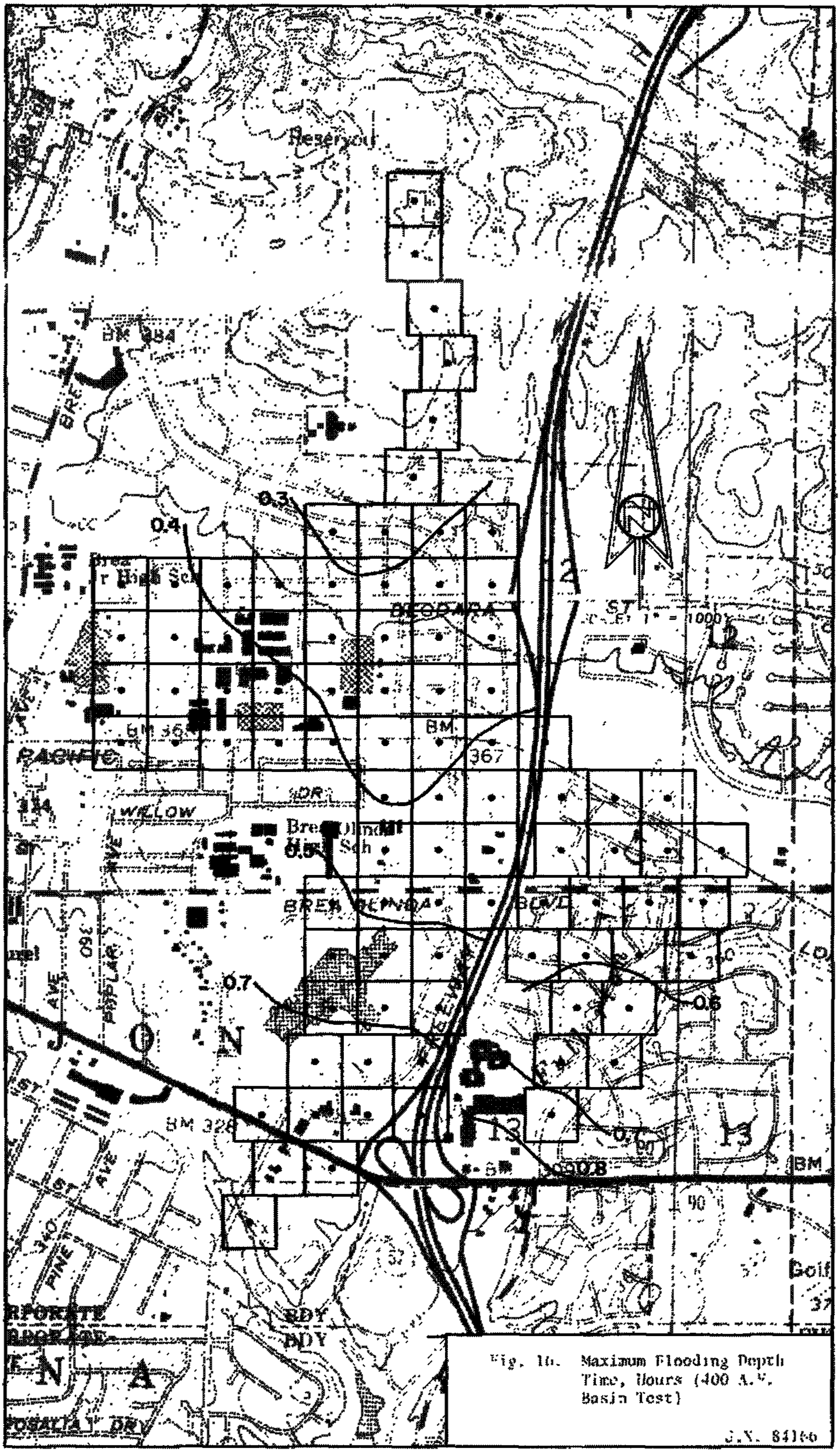

Fig. 1b. Maximum Flooding Depth Time, Hours (400 A.F. Basin Test)

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TABLE 2. ORANGE COUNTY RESERVIOR 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	1,233
14	39.4	1,813
16	46.5	2,530
18	54.1	3,400
20	62.0	4,420
22	70.3	5,611
24	79.2	7,000
26	88.5	8,520
28	96.2	10,300
30	108.4	12,200
32	119.1	14,300
34	130.2	16,700
36	141.9	19,200
38	154.0	22,000
40	166.7	25,000
42	179.9	28,300
44	193.8	31,800
46	211.0	35,500

TABLE 3. ESTIMATING DAM-BREAK Q_p
(20-Minute Peak Time)

Assumed Depth (ft)	Q_p[1] (cfs)	Volume Drained[2] (af)	Volume Left[3] (af)	Depth (ft)
20.0	4,420	60.9	151.0	37.0
26.0	8,520	117.3	94.7	27.5
26.5	8,936	123.1	88.9	26.0

NOTES:

1: $Q_p = 2.472H^{2.5}$

2: $V_d = 0.01377Q_p$ AF

3: $V_{left} = (212 - V_d)$ AF

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