ANALYSE D'UN CHAMP D'INONDATION URBAIN PAR MODELISATION SOUS LA FORME D'UN SYSTEME DE RESERVOIRS

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RESUME

Le Modèle Hydrodynamique de Diffusion (MHD) bidimensionnel est appliqué à l'évaluation des profondeurs du champ d'inondation résultant du débordement d'une rivière canalisée. Le MDH modélise mathématiquement le champ d'inondation sous la forme d'un système de réservoirs interreliés, avec des entrées et des sorties calculées selon la forme à moment d'inertie nul des équations de Saint-Venant. Les préoccupations environnementales que suscitent les crues et les vitesses d'écoulement élevées peuvent être étudiées plus efficacement au moyen du modèle d'écoulement bidimensionnel MHD que par les techniques de modélisation unidimensionnelle. Dans l'exemple présenté, les différences entre les profondeurs d'inondation prévues au moyen du MHD et celles mesurées par la méthode unidimensionnelle (HEC-2) sont importantes. Le MHD génère beaucoup d'information, mais il est facile à utiliser et n'exige pas plus d'expertise que les méthodes unidimensionnelles.

URBAN FLOODPLAIN ANALYSIS BY USE OF A SYSTEM OF RESERVOIRS

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ABSTRACT

The two-dimensional Diffusion Hydrodynamic Model, DHM, is applied to the evaluation of floodplain depths resulting from an overflow of a leveed river. The DHM mathematically models the floodplain as a system of interconnected reservoirs, with inflows and outflows computed according to the zero inertia form of the St. Venant equations. The environmental concerns of flooding and high flow velocities can be more effectively studied using the two-dimensional DHM flow model than by one-dimensional modeling techniques. In the example presented, some of the predicted flood depth differences between the DHM and the one-dimensional approach (i.e, HEC-2) are found to be significant. Although the DHM generates considerable information, it is easy to use and does not require expertise beyond that required for one-dimensional approaches.

INTRODUCTION

The main objective of this paper is to summarize the findings of a detailed study of the Santa Ana River 100-year event floodplain in the City of Garden Grove, California, using the two-dimensional Diffusion Hydrodynamic Model (DHM) [3, 5, 6, 7, 8, 9, 10]. In this study, the two-dimensional unsteady flow analysis results are compared with the one-dimensional modeling results obtained in a typical Federal Emergency Management Agency (FEMA) flood-insurance study [2] using HEC-2. Accordingly, this paper compares a modeling approach based upon use of a system of interconnected reservoirs to a model based on the very widely used steady-state flow profile program, HEC-2 [4].

The application study site is in the City of Garden Grove, California (see Figure 1). The local terrain slopes southwesterly at a mild gradient (about 0.4%), and the site is fully developed with mixed residential and

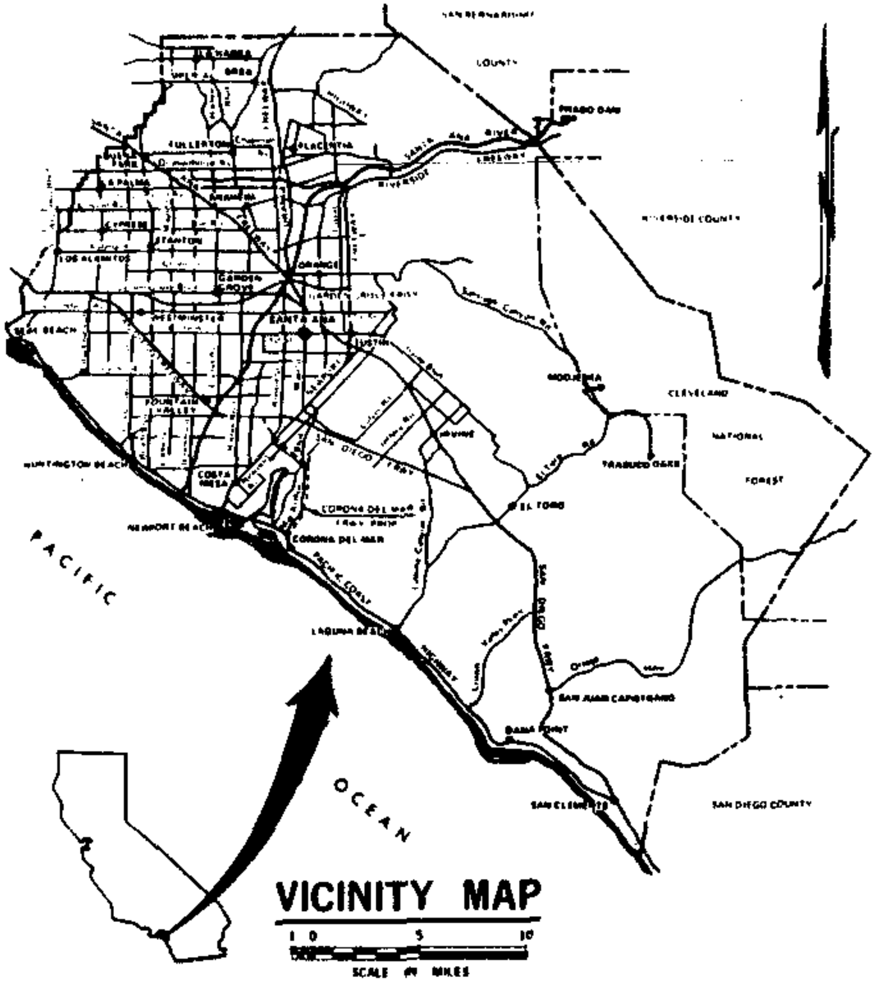

Figure 1. Vicinity Map.

commercial developments. Freeways form barriers across the study site so that all flows are laterally constrained with outlets at railroads and major streets crossing under the freeways. In this region flood waters flow southwesterly from the Santa Ana River, partially diverted by the Garden Grove Freeway. Because of the large quantity of flood flow conveyed through the floodplain and the mild terrain slope, it was necessary to use modeling of two-dimensional unsteady flow for the floodplain analysis.

Because the DHM provides a two-dimensional hydrodynamic response, its use reduces the uncertainty in predicted flood depths due to the variability in the choice of cross-sections used in the one-dimensional models. That is, model users might select a cross-section perpendicular to the direction of flow, but for an urban area this selection becomes somewhat arbitrary. Additionally, the DHM accommodates both backwater effects and unsteady flow. The latter is neglected in HEC-2 floodplain analysis.

MATHEMATICAL MODEL OF SYSTEM OF RESERVOIRS

The DHM provides the capability to model two-dimensional unsteady flow where storage effects and diverging flow paths are important, and hence, the steady state one-dimensional flow approach (such as HEC-2 [4]) may be inappropriate. This topographic flow model describes two-dimensional flow characteristics by means of a system of interconnected reservoirs.

The two-dimensional unsteady flow equations consist of one equation of continuity

$$\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} + \frac{\partial z}{\partial t} = 0 \tag{1}$$

and two equations of motion

$$\frac{\partial Q_x}{\partial t} + \frac{\partial}{\partial x}\left[\frac{Q_x^2}{Ax}\right] + \frac{\partial}{\partial y}\left[\frac{Q_x Q_y}{Ax}\right] + gAx\left[S_{fx} + \frac{\partial h}{\partial x}\right] = 0 \tag{2a}$$

$$\frac{\partial Q_y}{\partial t} + \frac{\partial}{\partial y}\left[\frac{Q_y^2}{Ay}\right] + \frac{\partial}{\partial x}\left[\frac{Q_x Q_y}{Ay}\right] + gAy\left[S_{fy} + \frac{\partial h}{\partial y}\right] = 0 \tag{2b}$$

in which t is time, x and y (and the subscripts) refer to the orthogonal directions in the horizontal plane; q_x and q_y are flow rates per unit width in the x and y-directions; z is the depth of water; Q_x and Q_y are flow rates in the x and y-directions, respectively; h is the water surface elevation measured vertically from a horizontal datum; g is acceleration of gravity; Ax and Av are the cross-sectional area; and S_{fx} and S_{fy} are the friction slopes in the x,y-directions. The DHM utilizes the uniform grid element as a reservoir for use two-dimensional unsteady flow. Therefore, Ax and Ay are defined as the length of the uniform grid element times the depth of water. Hence, the storage indication curve for each grid reservoir is readily computed.

The friction slopes S_{fx} and S_{fy} can be estimated by using Manning's formula

$$S_{fx} = \frac{n^2 Q_x^2}{C^2 Ax^2 R_x^{4/3}} \quad , \qquad S_{fy} = \frac{n^2 Q_y^2}{C^2 Ay^2 R_y^{4/3}} \tag{3}$$

in which n is the Manning's roughness factor; R_x, R_y are hydraulic radiuses in x,y-directions; and the constant C=1 for SI units and 1.486 for U.S. Customary units.

For diffusion wave modeling the local and convective acceleration terms in the momentum equation (i.e., the first three terms of Eq. 2) can be neglected [1]. Thus, Eq. (2) is simplified to

$$S_{fx} = -\frac{\partial h}{\partial x} \quad , \qquad S_{fy} = -\frac{\partial h}{\partial y} \tag{4}$$

Combining Eqs. (3) and (4) yields

$$Q_x = \frac{C}{n} Ax\, R_x^{2/3} \frac{\left(-\frac{\partial h}{\partial x}\right)}{\left|\frac{\partial h}{\partial x}\right|^{1/2}} \quad , \qquad Q_x = \frac{C}{n} Ay\, R_y^{2/3} \frac{\left(-\frac{\partial h}{\partial y}\right)}{\left|\frac{\partial h}{\partial y}\right|^{1/2}} \tag{5}$$

which accounts for flows in both positive and negative x and y-directions. The flow rates per unit width in the x and y-directions can be obtained from Eqs. (5) as

$$q_x = \frac{C}{n} Z\, R_x^{2/3} \frac{\left(-\frac{\partial h}{\partial x}\right)}{\left|\frac{\partial h}{\partial x}\right|^{1/2}} \quad , \qquad q_y = \frac{C}{n} Z\, R_y^{2/3} \frac{\left(-\frac{\partial h}{\partial x}\right)}{\left|\frac{\partial h}{\partial x}\right|^{1/2}} \tag{6}$$

Substituting Eq. (6) into Eq. (1), gives

$$\frac{\partial}{\partial x}\left[\frac{C}{n} Z\, R_x^{2/3} \frac{\left(-\frac{\partial h}{\partial x}\right)}{\left|\frac{\partial h}{\partial x}\right|^{1/2}}\right] + \frac{\partial}{\partial y}\left[\frac{C}{n} Z\, R_x^{2/3} \frac{\left(-\frac{\partial h}{\partial y}\right)}{\left|\frac{\partial h}{\partial y}\right|^{1/2}}\right] + \frac{\partial h}{\partial t} = 0$$

$$\frac{\partial}{\partial x}\left[K_x \frac{\partial h}{\partial x}\right] + \frac{\partial}{\partial y}\left[K_y \frac{\partial h}{\partial y}\right] = \frac{\partial h}{\partial t} \tag{7}$$

where

$$K_x = \frac{C}{n} Z\, R_x^{2/3} \Big/ \left|\frac{\partial h}{\partial x}\right|^{1/2} \quad \text{and} \quad K_y = \frac{C}{n} Z\, R_r^{2/3} \Big/ \left|\frac{\partial h}{\partial y}\right|^{1/2} \tag{8}$$

Equation (6) provides the inflow and outflow relationships for each grid reservoir used in the model. The numerical algorithms used for solving Eq. (7) are fully discussed by [3] and in the U.S.G.S. Water Resources Investigation Report, 87-4137 [10]. Data preparation needs for a flood-plain analysis are also discussed in this latter publication.

APPLICATION OF THE DHM

The Santa Ana River is not able to adequately convey the 100-year return period flood. Overflows from the river will occur at several locations. This section describes the application of the DHM to the overflow of the river as it flows through the City of Garden Grove, California.

U.S. Geological Survey topographic maps were used to provide mean elevation data for definition of the DHM topographic data (Fig. 2). Based upon aerial photographs and a field investigation of the study area, it was concluded that flood flows will mostly be contained within flow-paths in streets. An average effective grid area (i.e., that area which contributes to rapid storage of water) was also found from the aerial photographs. An average effective flow path for each grid was also estimated. Buildings occupy about thirty-five percent of the modeled area. For this analysis, all buildings were assumed to be excluded from available storage, and a global effective area factor of 0.65 was applied to the entire study area.

The 100-year frequency hydrograph of the Santa Ana River at Imperial Highway (see Fig. 3) was determined by the U.S. Army Corps of Engineers, Los Angeles District [11]. Because there are no breakout points between the Imperial Highway and Katella Avenue, this hydrograph was used in the DHM model. The flows breaking out of the river from Katella Avenue and downstream were estimated by dividing the runoff hydrograph into segments (see Fig. 3) according to the peak breakout flowrates estimated in the Corps of Engineers' study. These flows were applied at the various breakout points along the river. An initial 5000 cfs from the hydrograph peak was applied at Katella Avenue (Grid No. 1). The next 19,000 cfs breaks out just north of the Garden Grove Freeway (Grid No. 99). Immediately south of the Garden Grove Freeway, 1000 cfs breaks out on the west and east banks. Only the west bank overflow was applied to the model (Grid No. 114). An underlying assumption in the Corps' breakout analysis was that eastern overflows return to the river downstream from the study site, so this overflow is ignored in the DHM model. Finally, 3000 cfs overflows the west bank at Fairview Street (Grid No. 240). As seen from Fig. 3 this overflow begins first and is the longest in duration.

The overflows were assumed to occur as shown in Fig. 3 as a consequence of the diminishing capacity of the river downstream from Katella Avenue. According to the Corps of Engineers' study [11], the capacity of the channel upstream of Katella Avenue is more than 50,000 cfs, but is only about 16,000 cfs at Fairview Street.

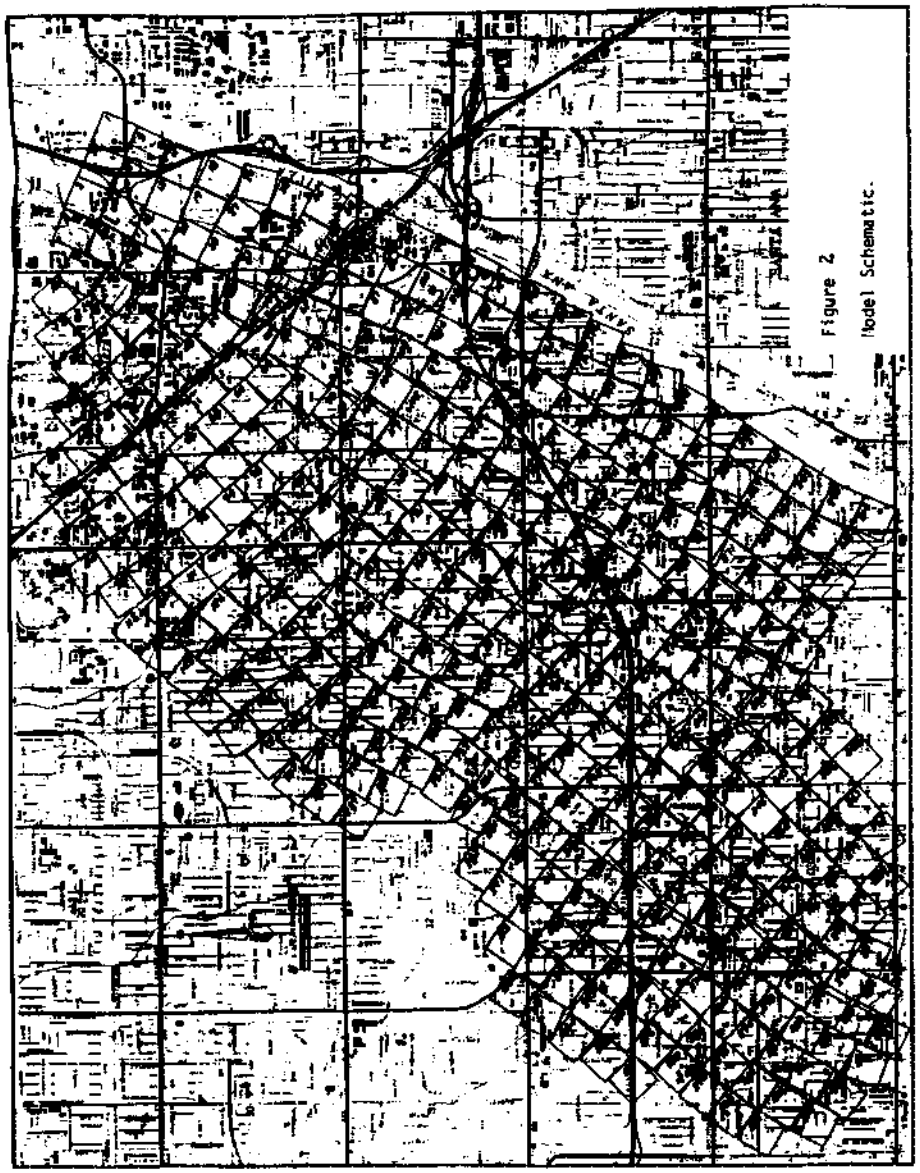

Figure 2 Model Schematic.

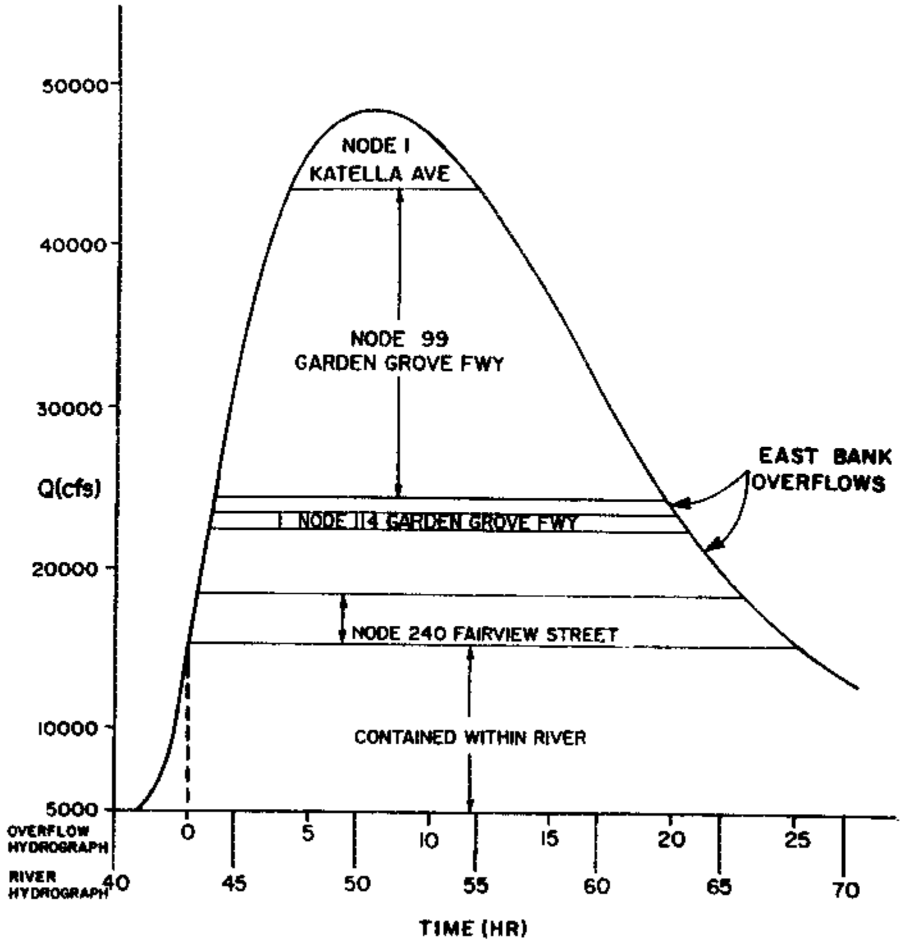

Figure 3. Segmented Santa Ana River 100-Year Runoff Hydrograph at Imperial Highway.

The maximum flood depths calculated using the DHM are shown in Fig. 4. These depths occur at various times throughout the 24 hour simulation period, although depths close to the maximum depths will occur for hours before and after the peaks. The floodplain boundary (see Fig. 5) is derived from maximum flood depths and ground surface elevations by interpolation of the results. The floodplain boundary on the Flood Insurance Rate Map (FIRM) [2] for the area is also shown in Fig. 5. It was computed using HEC-2. The maximum water surface elevation contours from both the DHM analysis and the FIRM map are depicted on Fig. 5. The DHM analysis indicates a wider floodplain and, consequently, lower maximum water-surface elevations at some locations than are indicated from the FIRM results. The largest deviation between the two approaches in the predicted maximum water surface elevation is 6 feet (see Fig. 5). This large difference is a direct result of the differences in HEC-2 and DHM approaches. The DHM analysis assumes that water moves in both longitudinal and lateral directions, while the HEC-2 analysis is a one-directional approach. This is reflected in Fig. 5 where the FIRM floodplain boundary is more or less parallel to the Santa Ana River. Also, the two methods differ in that the DHM provides an unsteady-state flow analysis while HEC-2 is a steady-state flow computation. However, in this study, unsteady flow effects were found to be minor in comparison to the two-dimensional flow features.

In general, the DHM analysis will provide a better floodplain analysis because this approach is capable of handling unsteady backwater effects in overland flow, unsteady overland flow due to constrictions (such as culverts, bridges, freeway underpasses, and so forth), as well as unsteady overland flow across watershed boundaries due to backwater and ponding effects. In general, several important types of information can be generated from the DHM analysis. These include (1) time versus flood depth relationships; (2) flood wave arrival times; (3) maximum flood depth arrival times; (4) direction and magnitude of the flood wave; (5) stage versus discharge relationships; and (6) outflow hydrographs at any specified grid element within the study area.

CONCLUSIONS

The Diffusion Hydrodynamic Model provides an effective tool for flood plain management, especially when the flow is two-dimensional in nature. In comparison to the commonly used HEC-2 model for floodplain analysis, the DHM allows unsteady flows to be analyzed so that floodplain storage effects can be evaluated accurately. This permits the determination of the time of occurrence of flood peak flow, stage versus time relationships, and other time-related relationships. In general, a more accurate definition of the flooding limits can be obtained with the DHM. Although the DHM generates considerable information, it is easy to use and does not require expertise beyond that required for one-dimensional approaches.

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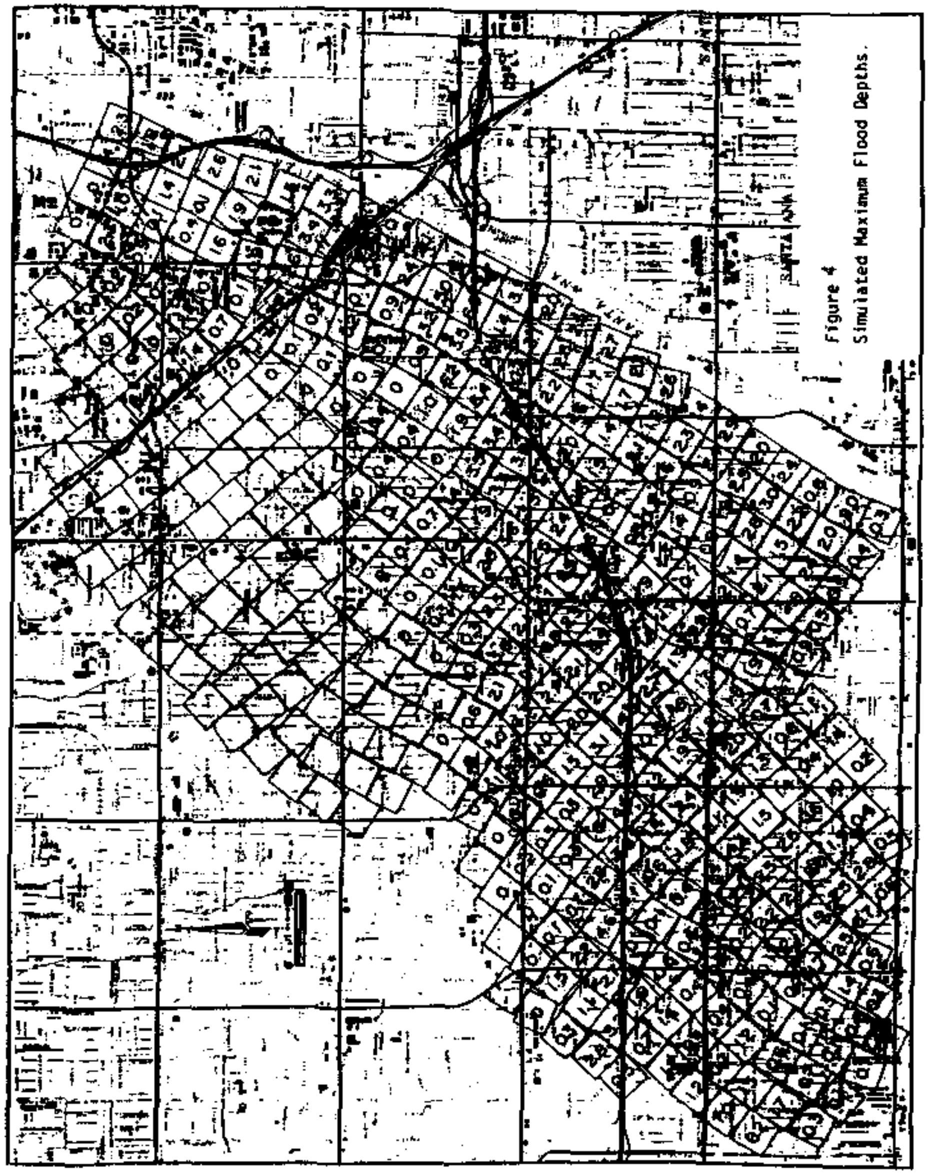

Figure 4
Simulated Maximum Flood Depths.

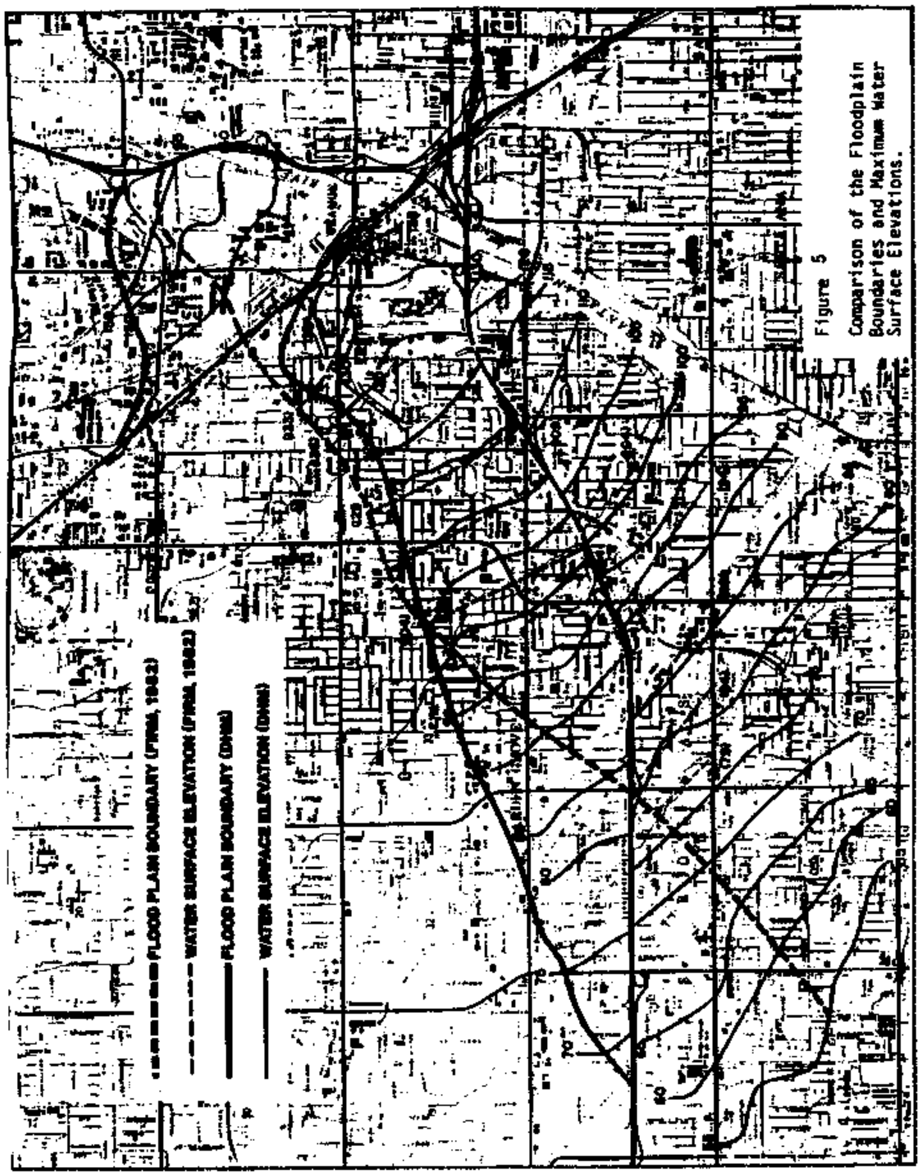

Figure 5
Comparison of the Floodplain Boundaries and Maximum Water Surface Elevations.

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