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APPLICATION OF THE U.S.G.S. DIFFUSION HYDRODYNAMIC MODEL FOR URBAN FLOODPLAIN ANALYSIS[1]

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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 environmental concerns of flood protection and high flow velocities can be better studied with the help of the two-dimensional DHM flow model than by use of the one-dimensional modeling techniques. In the test case, 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 use of the one-dimensional approaches.
(KEY TERMS: two-dimensional unsteady flow, floodplain analysis.)

INTRODUCTION

The main objective of this report 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) (Hromadka, 1985; Hromadka *et al*, 1985; Guyman and Hromadka, 1986; Hromadka and Durbin, 1986; Hromadka and Nestlinger, 1985; Hromadka and Yen, 1986; Hromadka and Yen, 1987). 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 using HEC-2 (1982).

The application study site is in the City of Garden Grove, California (see Figure 1). The local terrain slopes southwesterly at a mild gradient (i.e., 0.4%) and is fully developed with mixed residential and commercial developments. The freeways form barriers through the study site so that all flows are laterally constrained with outlets at railroads and major streets crossing under the freeways. Consequently, in this region the flood water would 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 cross-sectional terrain, the floodplain analysis needs to include the two-dimensional unsteady flow approaches.

Because the DHM provides a two-dimensional hydrodynamic response, use of the model eliminates 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 on an urban area the selection becomes somewhat arbitrary. Additionally, the DHM accommodates both backwater effects and unsteady flow, both of which are typically neglected in HEC-2 (1973) floodplain analysis.

DESCRIPTION OF THE DHM

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, 1973) may be inappropriate. The two-dimensional unsteady flow equations consist of one equation of continuity

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

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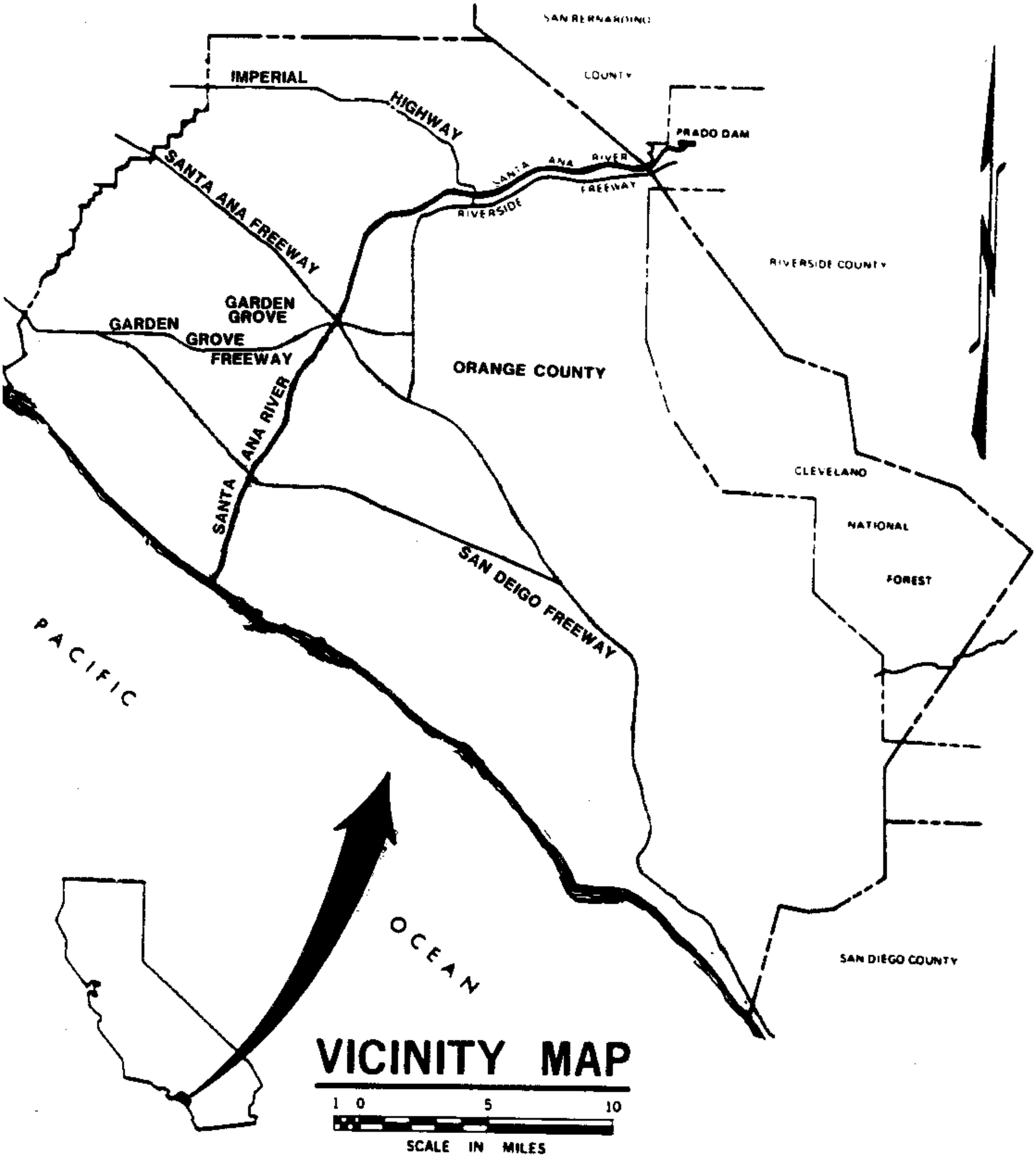

Figure 1. Vicinity Map.

and two equations of motion

$$\frac{\partial Q_x}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q_x^2}{Ax}\right) + \frac{\partial}{\partial y}\left(\frac{Q_x Q_y}{Ax}\right) +$$

$$gAx\left[S_{fx} + \frac{\partial h}{\partial x}\right] = 0 \qquad (2a)$$

$$\frac{\partial Q_y}{\partial t} + \frac{\partial}{\partial y}\left(\frac{Q_y^2}{Ay}\right) + \frac{\partial}{\partial x}\left(\frac{Q_x Q_y}{Ay}\right) +$$

$$gAy\left[S_{fy} + \frac{\partial h}{\partial y}\right] = 0 \qquad (2b)$$

in which t is time, x and y (and the subscripts) are the orthogonal directions in the horizontal plane; q_x and q_y are the flow rates per unit width in the x and y directions; z is the depth of water; Q_x and Q_y are the flow rates in the x and y directions, respectively; h is the water surface elevation measured vertically from a horizontal datum; g is the acceleration of gravity; Ax and Ay are the cross-sectional areas; and S_{fx} and S_{fy} are the friction slopes in the x,y-directions. The DHM .tilizes the uniform grid element to model the two-dimensional unsteady flow; therefore, Ax and Ay are defined as the length of uniform grid element times the depth of water.

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

$$S_{fx} = \frac{n^2 Q_x^2}{C^2 Ax^2 R_x^{4/3}} \qquad (3a)$$

and

$$S_{fy} = \frac{n^2 Q_y^2}{C^2 Ay^2 R_y^{4/3}} \qquad (3b)$$

in which n is the Manning's roughness coefficient; Rx, Ry are the hydraulic radii in the x,y-directions; and the constant C=1 for SI units and 1.486 for U.S. Customary units.

In the DHM, the local and convective acceleration terms in the momentum equation (i.e., the first three terms of Eq. 2) are neglected (Akan and Yen, 1981). "hus, Eq. (2) is simplified as

$$S_{fx} = -\frac{\partial h}{\partial x} \qquad (4a)$$

and

$$S_{fy} = -\frac{\partial h}{\partial y} \qquad (4b)$$

Combining Eqs. (3) and (4) yields

$$Q_x = \frac{C}{n} Ax\ R_x^{2/3} \frac{\left(-\frac{\partial h}{\partial x}\right)}{\left|\frac{\partial h}{\partial x}\right|^{1/2}} \qquad (5a)$$

$$Q_y = \frac{C}{n} Ay\ R_y^{2/3} \frac{\left(-\frac{\partial h}{\partial y}\right)}{\left|\frac{\partial h}{\partial y}\right|^{1/2}} \qquad (5b)$$

which may account 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 Eq. (5) as

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

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

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

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

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

or

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

where

$$K_x = \frac{C}{n} Z\, R_x^{2/3} \Big/ \left|\frac{\partial h}{\partial x}\right|^{1/2}$$

and

$$K_y = \frac{C}{n} Z\, R_y^{2/3} \Big/ \left|\frac{\partial h}{\partial y}\right|^{1/2}$$

The numerical algorithms used for solving Eq. (7) are fully discussed by Guymon and Hromadka (1986) and in the U.S.G.S. Water Resources Investigation Report, 87-4137 (Hromadka and Yen, 1987). The data preparation needs for a floodplain analysis is also discussed in the U.S.G.S. Water Resources Investigation Report (Hromadka and Yen, 1987).

Two new features have recently been added to the DHM: (1) a flow-path reduction factor and (2) the effective grid area. The flow-path reduction factor is used to effectively block flows across some grid boundaries and allow limited or full flow across other grid boundaries (see Figure 2). The effective grid area allows the available storage of a particular grid to be varied (see Figure 2). The flow-path reduction factor and effective grid area permit the more realistic representation of the field conditions. An aerial photograph or a field investigation is needed to determine the proper flow-path reduction factors and effective grid areas for the study area.

APPLICATION OF THE DHM TO THE STUDY AREA

The Santa Ana River, at its present capacity, is not able to adequately convey the 100-year return period design flow. Overflows of the river will occur at several locations within developed portions of Orange County. The DHM was applied, in this study, to the overflow of the river through the City of Garden Grove, California. Using U.S.G.S. topographic quadrangle maps, a 1000-foot grid discretization was prepared (Figure 3). Ground elevations for each grid were estimated from the maps.

A global Manning's roughness coefficient of n = 0.045 was initially used in this study, except at major obstructions, such as freeways. Roughness coefficients for freeway undercrossings were assumed to be n = 0.020. Effective grid areas were also assigned to the elements that are adjacent to the freeways. This decreased the available storage of the particular grid For example, grid element number 213 (see Figure 3) is split by the 22 Freeway, and flow from the north side of the freeway cannot cross the freeway at this location, so only one-half of the grid was used for storage of runoff volume. The net effects of using the flow path reduction factor and the decreased available storage is to achieve more realistic results for the floodplain analysis.

Based upon an aerial photograph and a field investigation of the study area, it was assumed that the flood flows will mostly be contained within flow-paths in which streets exist. On the average, widths of these flow-paths comprise one-fifth of a typical cross-section, i.e., the flow-path reduction factor is 0.8 (1–0.2). An average effective grid area was also found from the aerial photograph. Buildings occupied thirty-five percent of the photographed area. In this study all buildings were assumed to be excluded from available storage. Therefore, a global effective area factor of 0.65 was applied to the entire study area.

A 100-year frequency runoff hydrograph of the Santa Ana River at Imperial Highway (see Figure 1) was generated by the U.S. Army Corps of Engineers, Los Angeles District (1987). Since there are no breakout points between the Imperial Highway and Katella Avenue, this hydrograph was used in the subject DHM model, by dividing the runoff hydrograph into segments (see Figure 4) according to the peak breakout flowrates estimated in the referenced Corps of Engineers' study,

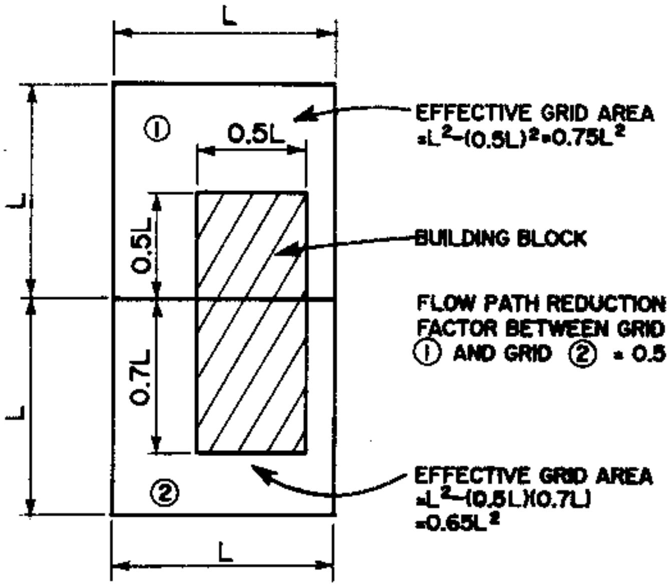

Figure 2. Flow Path Reduction Factor and Effective Grid Area Features.

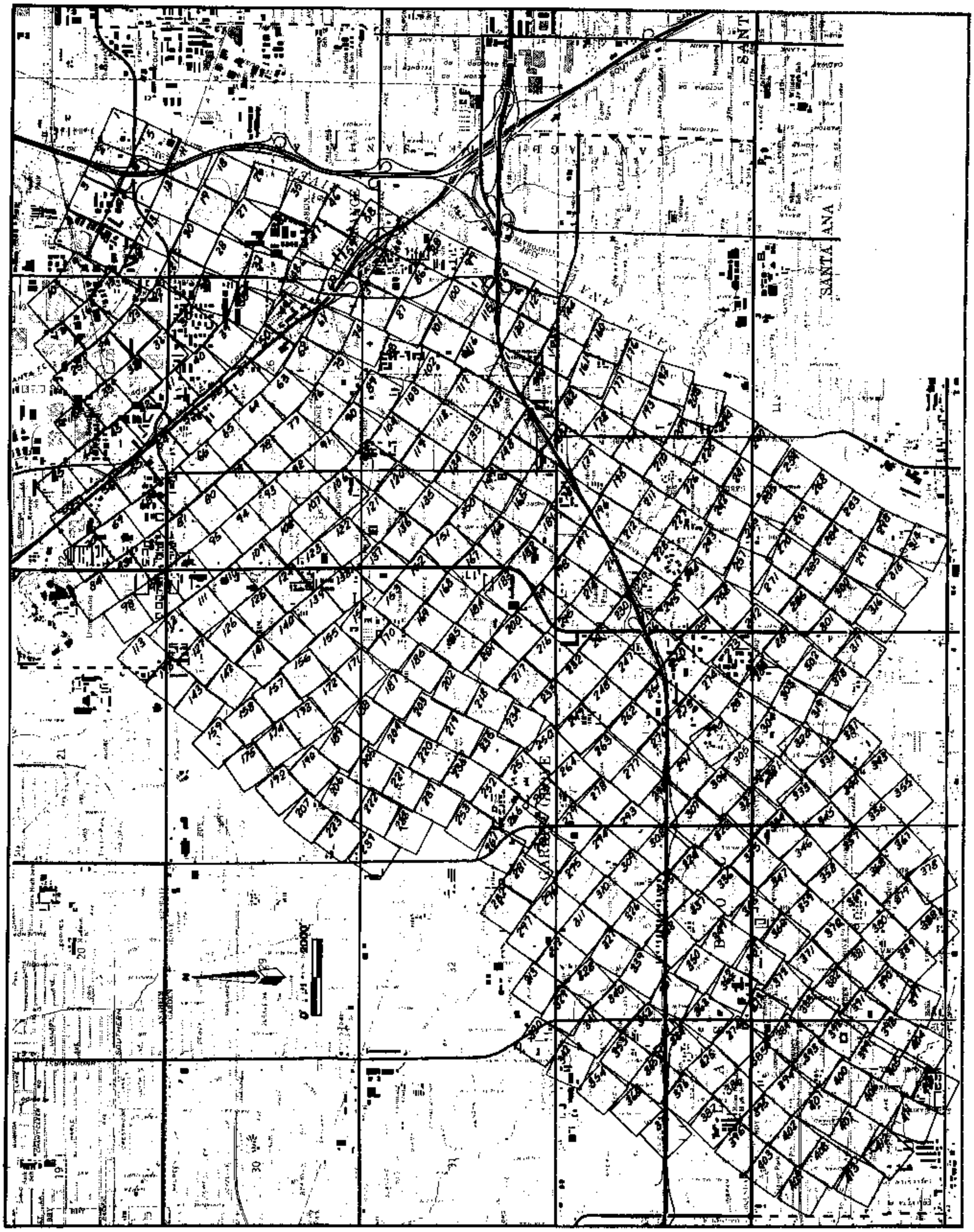

Figure 3. Model Schematic.

and applied at the various breakout points along the river. The peak 5000 cfs was applied at Katella Avenue (grid #1). The next 19,000 cfs breaks out just north of the Garden Grove Freeway (grid #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 #114). An underlying assumption in the Corps' breakout analysis was that the eastern overflows return to the river downstream from the study site, so the overflow is ignored in this model. Finally, 3000 cfs overflows the west bank at Fairview Street (grid #240). As seen from Figure 4 this overflow actually occurs first and is the longest in duration.

The overflows were assumed to occur as shown in Figure 4 because of the diminishing capacity of the river moving downstream from Katella Avenue. According to a channel-capacity analysis in the Corps of Engineers' study (1982), the channel capacity upstream of Katella Avenue is in excess of 50,000 cfs, but,is reduced to about 16,000 cfs at Fairview Street.

The maximum flood depths calculated using the DHM are shown in Figure 5. These depths occur at various times throughout the total simulation time of 24 hours, although depths close to the maximum depths will remain for hours before and after the peaks. The floodplain boundary (see Figure 6) is derived from the maximum flood depths and the ground elevations. The floodplain boundary on the Flood Insurance Rate Map (FIRM) (1982) for the area is also shown in Figure 6. The maximum water surface elevation contours from the DHM analysis and from the FIRM map are also depicted on Figure 6. The DHM analysis resulted in a wider floodplain and consequently, lower maximum water-surface elevations at some locations than the results from the FIRM (1982). The maximum deviation in the predicted maximum water surface elevation between these two approaches is 6 feet (see Figure 6). This is primarily due to the difference in HEC-2 (1982) and DHM approaches. The DHM analysis allows the water to move in both longitudinal and lateral directions, but the HEC-2 (1982) analysis is the unidirectional approach. This is shown on Figure 6 where the FIRM floodplain boundary is more or less parallel to the Santa Ana River. Also, the DHM is unsteady-state flow analysis and the HEC-2 (1982) is steady-state flow analysis. No field data are available to verify both the results from the FIRM (1982) and the DHM analysis.

In general, the DKM 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

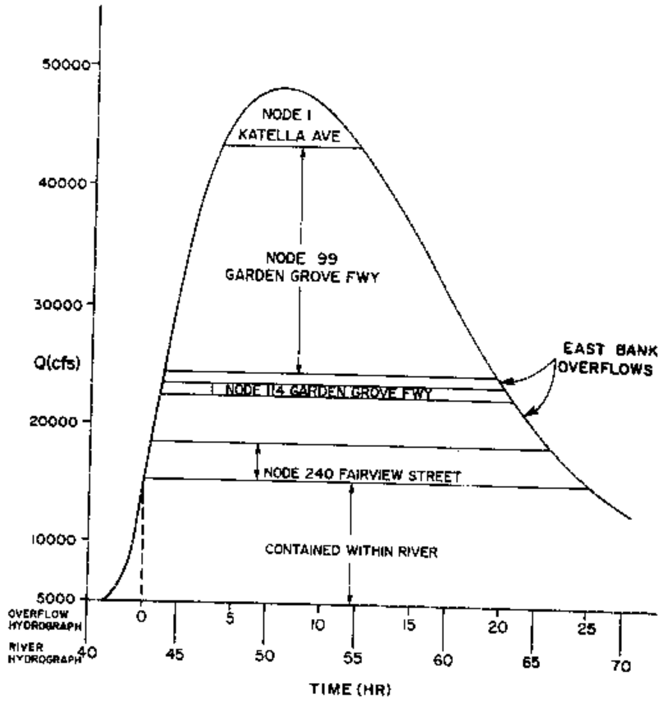

Figure 4. Segmented Santa Ana River 100-Year Runoff Hydrograph at Imperial Highway.

culverts, bridges, freeway underpasses, and so forth, unsteady flow overland flow across watershed boundaries due to backwater and ponding flow effects. In general, several important types of information can be generated from the DHM analysis. These include (1) the time versus flood depth relationship; (2) the flood wave arrival time; (3) the maximum flood depth arrival time; (4) the direction and magnitude of the flood wave; (5) the stage versus discharge relationship; and (6) the outflow hydrograph at any specified grid element within the study area.

CONCLUSIONS

The DHM (1987), which provides another tool for floodplain management, was published by the U.S. Geological Survey as a Water Resources Investigations Report (87-4137). The flow-path reduction factor and the effective grid area were added to the DHM (1987) for a more realistic representation of the field conditions.

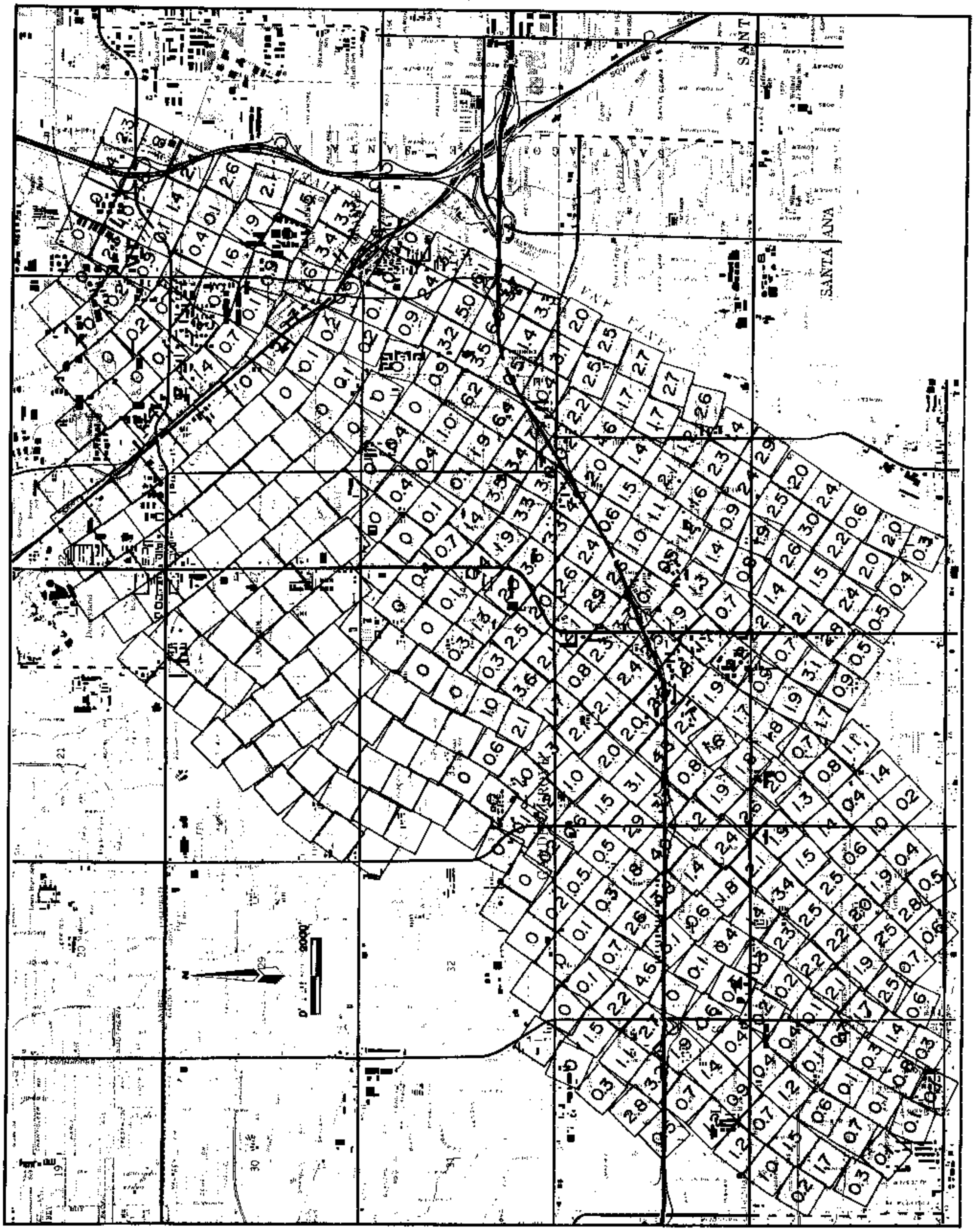

Figure 5. Simulated Maximum Flood Depths.

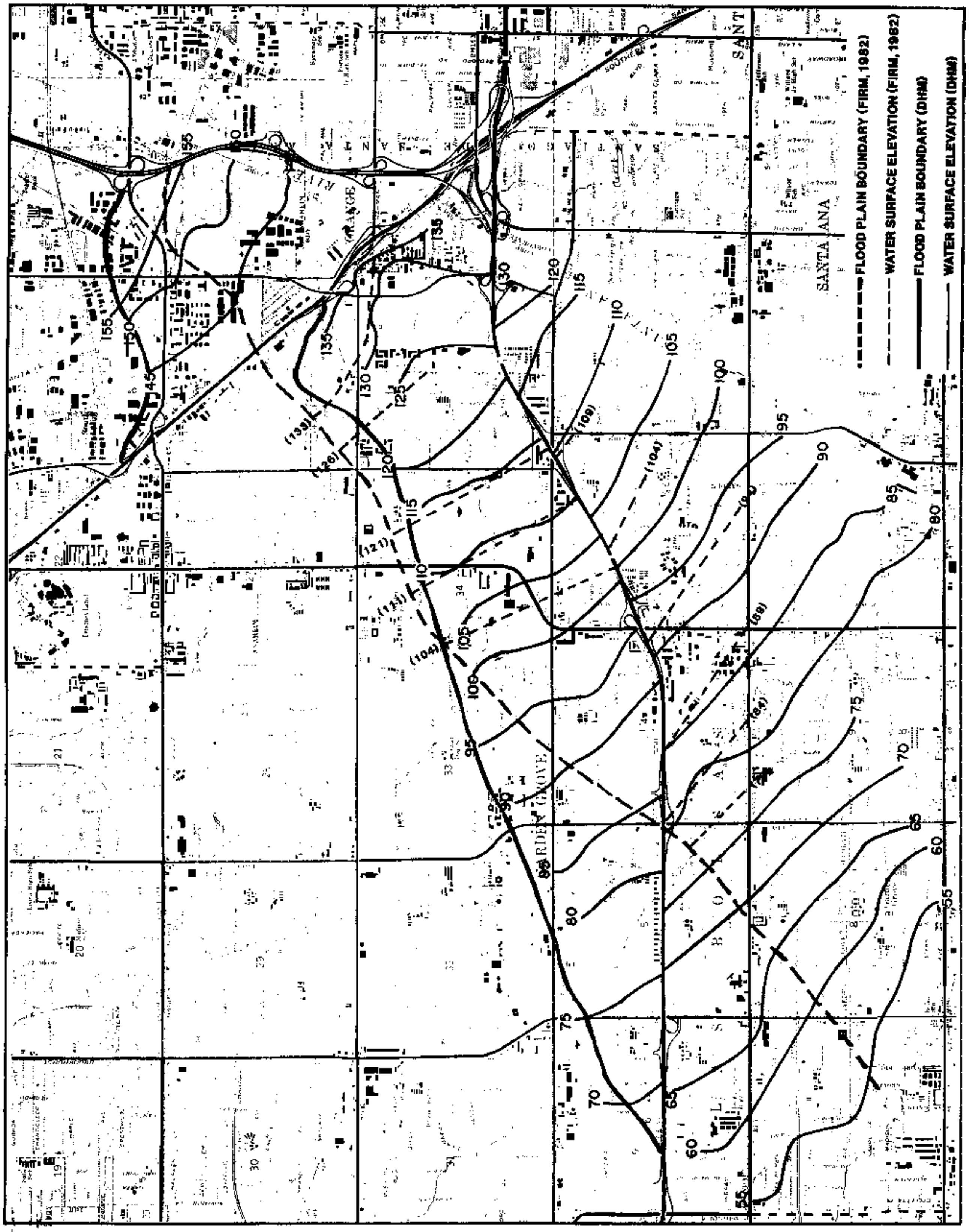

Figure 6. Comparison of the Floodplain Boundaries and Maximum Water Surface Elevations.

In this study, the DHM analysis resulted in a wider floodplain and, consequently, lower water-surface elevations at some locations than the results from the FIRM (1982). Because both of the results were based on different modeling approaches, and no field data and no data are available to validate either model, applications of both models to some recorded flooding areas are needed in order to properly compare the accuracy between both modeling approaches.

NOTICE

The computational results shown in this paper are to be used for research purposes only. No governmental approval of the results shown are to be construed nor implied.

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