EAST GARDEN GROVE - WINTERSBURG (C05)
AND OCEANVIEW (C06) CHANNELS
INUNDATION STUDIES

(EXISTING CHANNEL CONDITIONS RESULTS)

VOLUME 1. MAIN REPORT

APPENDICES A-C

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WILLIAMSON & SCHMID
CONSULTING CIVIL ENGINEERS AND LAND SURVEYORS
15101 Red Hill Avenue • Tustin, California 92680 • 714/259-7900 • 714/259-0210 FAX

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INUNDATION STUDIES

(EXISTING CHANNEL CONDITIONS RESULTS)

Submitted by: ______________________

T.V. Hromadka II, Ph.D., Ph.D., P.H., RCE
Project Manager

Prepared by: ______________________

Chung-Cheng Yen, Ph.D.
Project Engineer

August, 1992

WILLIAMSON & SCHMID CONSULTING CIVIL ENGINEERS AND LAND SURVEYORS

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I. INTRODUCTION

I.1. STUDY AREA

The Orange County Flood Control District facilities C05, East Garden Grove-Wintersburg Channel and its tributary C06, Ocean View Channel, drain an area of approximately 18,000 acres (28.13 square miles) within the cities of Anaheim, Fountain Valley, Garden Grove, Huntington Beach, Orange, Santa Ana, and Westminster. The watersheds (see Figures 1.1 and 1.2) lie on a flat coastal plain and are generally bounded by the Santa Ana River to the east, Talbert Valley watersheds (facilities D01, D02, and D05) to the south, the Pacific Ocean to the west and the Westminster Channel watershed (facility C04) to the north.

The C05 and C06 Channels, originally built to interim standards as part of the 1956 Bond Act, require extensive improvements to meet currently acceptable flood protection standards. During the storms of December 4, 1974, March 1, 1983, and February 12, 1992 these channels overflowed in several locations.

The C05 Channel is a leveed earthen channel from the ocean outlet to approximately Golden West Street, a trapezoidal earth channel with riprap slope protection upstream to approximately Bolsa Avenue, and a concrete trapezoidal channel with several reaches of covered conduit to its terminus at Chapman Avenue where it splits off into County storm drains C05P21 and C05P22.

There are two retarding basins within the C05 watershed, Haster Basin (C05B02) on the C05 mainline north of Garden Grove Boulevard and West Street Basin (C05B01) on the C05S10 tributary system. These basins are known to have inadequate capacity.

Ocean View Channel (C06) is generally an incised trapezoidal earth channel in the lower reaches from the C05 confluence to Mile Square Park with the exception of rectangular concrete channel and covered conduit reaches downstream of the San Diego Freeway (I-405), a riprap greenbelt channel through Mile Square Park changing again to a trapezoidal earth channel to its terminus at Newhope Street.

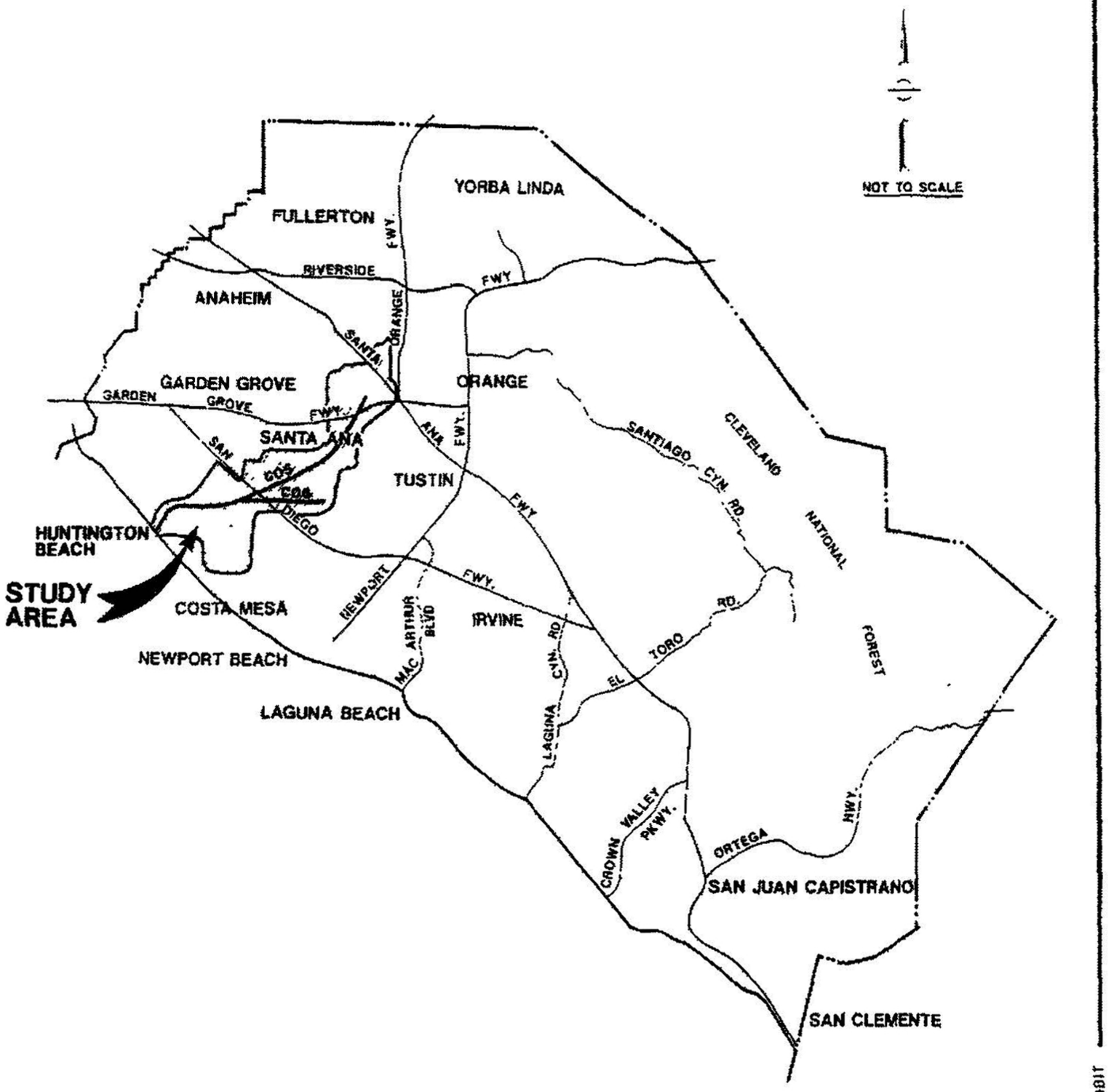

FIGURE 1.1: REGIONAL LOCATION MAP

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SCALE 1"=5000'

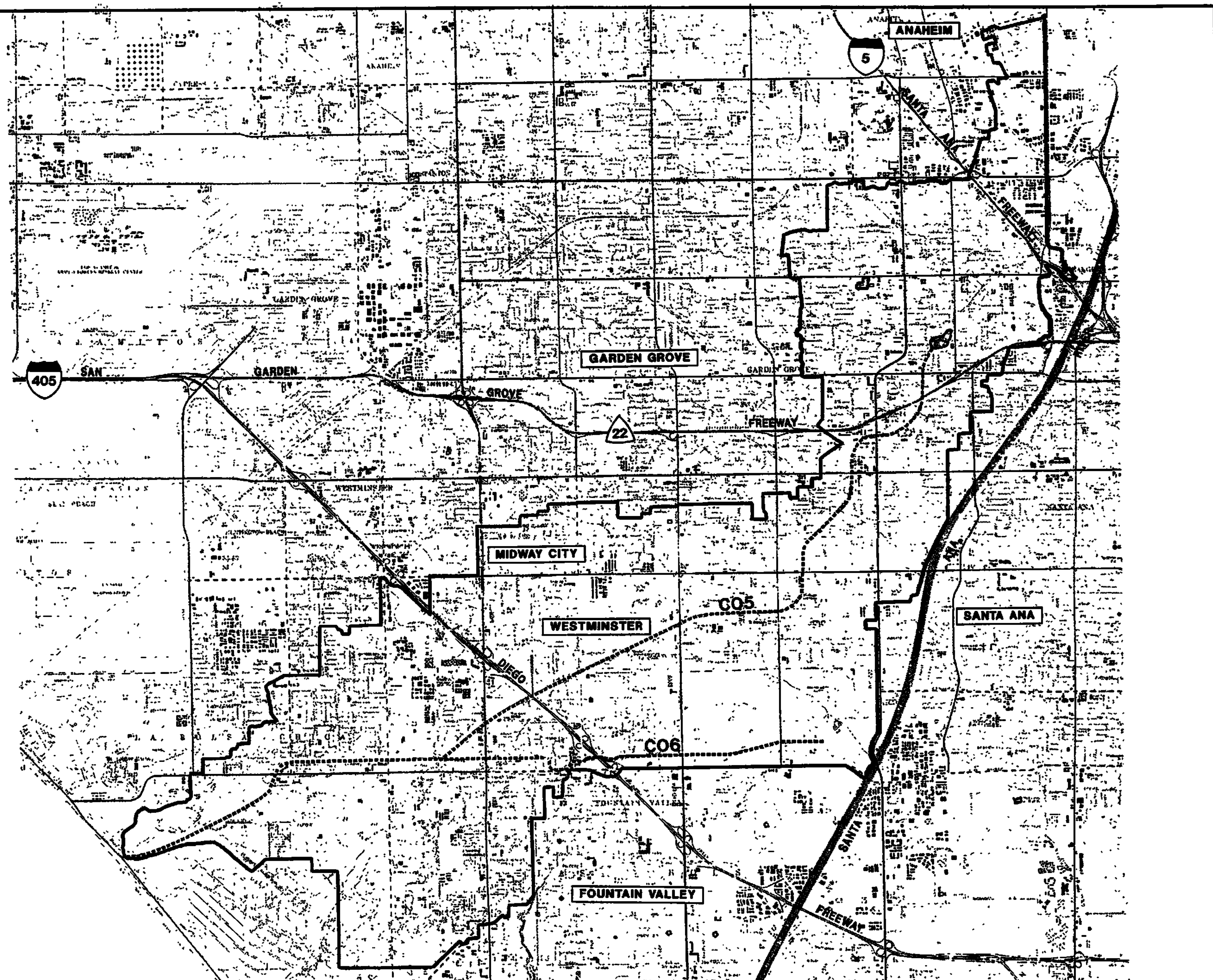

FIGURE 1.2
VINCINITY MAP

I.2. PURPOSE OF STUDY

There are three main objectives of the ongoing East Garden Grove-Wintersburg and Ocean View Channels (C05/C06) Project Report and Inundation Study:

1. Define an ultimate channel system alternative that minimizes total cost and environmental damages.

2. Develop a construction phasing program that prioritizes the construction of the recommended ultimate channel system components in the order of decreasing benefits to the entire system. That is, fix first what will benefit the system the most, and so on.

3. Perform inundation studies to potentially provide information sufficient to facilitate the development of a possible assessment fee program to pay for the ultimate channel system, based on "direct benefits received" and/or contributory factors.

This report presents the study methodology, procedures and results obtained for Objective #3, Inundation Studies.

I.3. REPORT ORGANIZATION

Reviews of previous floodplain studies and hydrologic analyses on the tributary Santa Ana River floodplain and the C05/C06 watershed are provided in Section II.

A brief discussion of the U.S.G.S. Diffusion Hydrodynamic Model and its modeling components are provided in Section III. Application of the DHM to the Study Area are fully discussed in Section IV. Results from the DHM analyses are summarized in Section V.

The Report organization is illustrated by the flow chart in Figure 1.3. The individual tasks, as shown in the flow chart, are further described in the linked chapters and appendices.

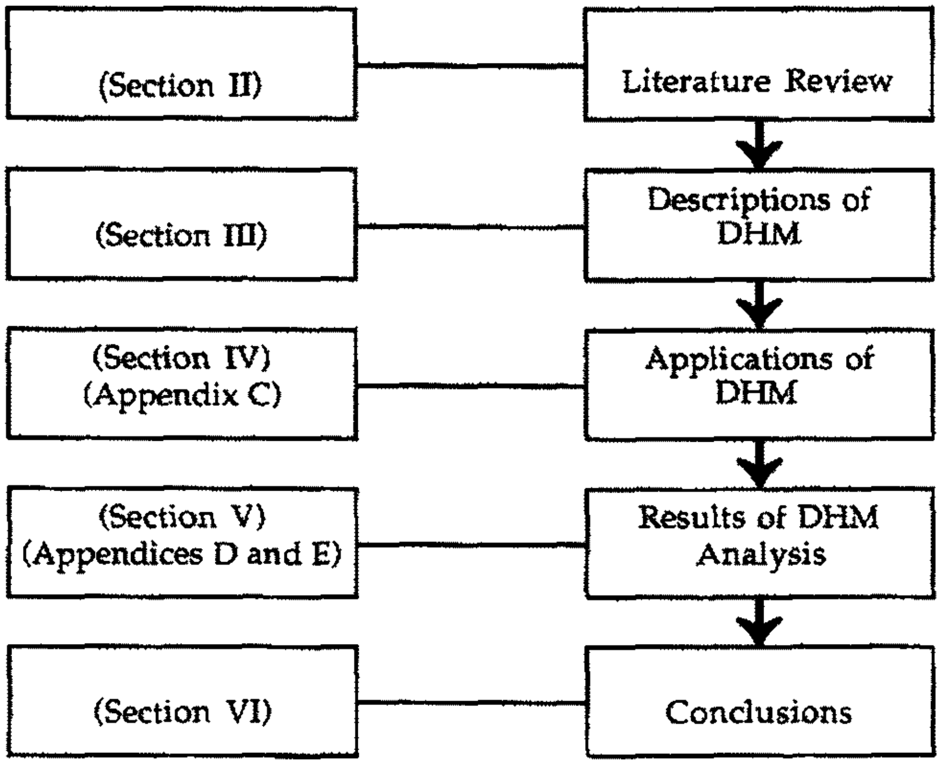

Figure 1.3 Study Flowchart

I.4. STUDY TEAM

The Williamson & Schmid engineering staff involved in the preparation of this report are as follows:

Theodore V. Hromadka II, Ph.D., Ph.D., P.H., RCE
Chung-Cheng Yen, Ph.D.
Patricia A. Bajak, RCE
Mili S. Lim, RCE

II. REVIEW OF PREVIOUS REPORTS

In this section, two key reports are reviewed. These are the Flood Control Feasibility Study of East Garden Grove-Wintersburg Channel by the U.S. Army Corps of Engineers (COE), Los Angeles District (1988), and the preliminary hydrologic study for East Garden Grove-Wintersburg Channel by the Orange County Environmental Management Agency (OCEMA, 1991). Hydrologic information and floodplain characteristics were extracted from these reports for use in this study.

II.1. COE: FLOOD CONTROL FEASIBILITY STUDY (1988), EAST GARDEN GROVE-WINTERSBURG AND OCEANVIEW CHANNELS

This report was prepared by the U.S. Army Corps of Engineers, Los Angeles District (1988) with the assumption that no breakout occurs from the Santa Ana River. This report has three major objectives: (a) to present the meteorologic, hydrologic, and physical characteristics of the East Garden Grove Wintersburg (EGGW) study area; (b) to present the methods and techniques developed and used to model the runoff process; and (c) to present discharges for the existing watershed under both present and future development conditions.

This report presents the history of flood events and the meteorologic, hydrologic, and physical characteristics of the entire watershed of EGGW Channel and the flooding problems of the EGGW watershed.

> *Documentation of flood events after the storms of December 4, 1974 and March 1, 1983 indicate that flooding problems exist on the EGGW Channel at Golden West Street and just upstream of the San Diego Freeway. The 1974 storm caused flooding on the EGGW Channel near Bushard Street. The 1974 storm also caused flooding on the Oceanview Channel just upstream of the San Diego Freeway. Officials from the City of Huntington Beach report flooding has occurred south of the Oceanview Channel between Newland Street and Beach Boulevard.*
>
> *It should be noted that there was no appreciable damage recorded for residential and commercial buildings during these storms. The 1983 storm may be viewed as approximating the 25-year rainfall event for the watershed.*

The referenced report also points out that:

> *for rare events, the Santa Ana River would overflow its banks and cause a levee breach. This could result in larger peak discharges and greater volume of water in the EGGW Channel. The Santa Ana River project will, when completed, provide approximately 200-year flood protection downstream from Prado Dam. This study assumes that the Santa Ana River project is in place.*

The following is a brief outline of the analysis.

a. Preliminary discharges were determined using the LAPRE1 computer program assuming all flow entered and stayed in the channel.

b. Channel capacities were estimated using Manning's Equation and the channel as-built plans.

c. A water surface profile was developed for the EGGW and Oceanview Channel from the tide gates to just upstream from the San Diego Freeway using HEC-2. Using a range of input discharges, the existing channel capacities and potential breakout locations were determined.

d. The LAPRE1 model was modified to divert flow exceeding the channel capacities at breakout locations.

e. The Interior Drainage Flood Routing Computer program was used to determine if side drain inflow occurs at peak channel discharge.

f. Overland flow areas along EGGW Channel were estimated using the peak discharge (from the breakout hydrographs), Manning's Equation and information from topographic maps and field investigations.

The study verified documented flooding reports that Golden West and the two San Diego Freeway undercrossings just upstream from the freeway were breakout locations. Breakouts can also occur on both channels just downstream from the freeway and also on the north levee of the EGGW Channel downstream from Gothard Street.

II.2. OCEMA: HYDROLOGY FOR EAST GARDEN GROVE-WINTERSBURG CHANNEL (1991)

This report provides 25-year and 100-year discharges for the East Garden Grove-Wintersburg Channel (Facility C05), from Bolsa Chica Bay outlet to Vermont Avenue. Twenty-five year 85-percent confidence discharges are identical to 100-year 50-percent confidence discharges. These discharges will be used in the future project report for the above channels.

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Orange County Hydrology Manual (October 1986) guidelines and hydrologic data were used in computations of the peak discharges. Peak discharges were computed by Unit Hydrograph Method at each nodal point. Lag times were calculated using 0.8Tc, where Tc is obtained from detailed Rational Method Hydrology as described in the Hydrology Manual.

The project report being prepared simultaneously with this inundation study is based on the discharges from this County hydrology study and will recommend improvements for the C05/C06 channel system.

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II.3. COMPARISON OF DHM METHODOLOGY TO PREVIOUS STUDY METHODOLOGIES

The above mentioned studies were based on an one-dimensional steady-state flow analysis. The flood depths were calculated by assuming normal depth in wide, shallow rectangular sections taken parallel to major contour lines. The overflow boundaries were determined by judgement, and inspection of overflow maps and aerial photographs of the 1916 and 1938 floods. The above procedures are strictly subjective to the analyst. The DHM computer program used in the current study is a two-dimensional, unsteady flow model which simplifies the two-dimensional St. Venant equations to eliminate local acceleration and inertial terms, and combines the simplified flow equations with the equation of continuity to form a diffusion type partial differential equation. Because the DHM provides a two-dimensional hydro-dynamic 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 floodplain analyses.

III. U.S.G.S. DIFFUSION HYDRODYNAMIC MODEL (DHM)

III.1. MODEL DESCRIPTION

The current study is based upon use of a diffusion (non-inertial) hydrodynamic model (DHM) of coupled two-dimensional overland flow and one-dimensional open-channel flow as developed by Hromadka and Yen, 1987. Because the non-inertial form of the hydrodynamic flow equations is used, several important hydraulic effects that cannot be handled by the usual kinematic routing techniques--the approach employed in most watershed models--are accommodated in this model; namely, the model is capable of treating such dynamic effects as backwater, drawdown, channel overflow, storage and ponding. Although these hydraulic effects were commonly neglected in past studies, they are important in drainage studies involving deficiencies of flood control channels or subtle grade differences between alluvial fan watershed boundaries.

The DHM can approximate all of the above hydraulic effects for channels, overland topographic surfaces, and the interfacing of these two hydraulic systems to represent channel overflow and return flow. The overland topographic flow effects are modeled by a two-dimensional unsteady flow hydraulic model based on the diffusion (non-inertial) form of the governing flow equations. Similarly, channel flow is modeled using a one-dimensional unsteady flow hydraulic model based on the diffusion type equation. The resulting models approximate both unsteady supercritical and subcritical flow (without the user predetermining hydraulic controls), backwater flooding effects, and escaping and returning flow from the two-dimensional topographic flow model to the channel system.

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

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

and two equations of motion

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

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

in which t is time, x and y (and 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 utilizes 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

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

and

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

in which n is the Manning's roughness factor; R_x, R_y are the hydraulic radii in 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). Thus Eq. (2) is simplified as

$$S_{fx} = -\frac{\partial h}{\partial x} \tag{4a}$$

and

$$S_{fy} = -\frac{\partial h}{\partial y} \tag{4b}$$

Combining Eqs. (3) and (4) yields

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

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

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

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

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

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

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

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

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

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

and

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

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).

Ample applications are cited in the references section of this report which demonstrate the utility of the DHM computer modeling approach in many drainage engineering problems which include: (1) one-dimensional unsteady flow analysis, (2) rainfall-runoff analysis, (3) dam-break flow analysis, (4) estuary analysis, and (5) channel floodplain interface analysis.

III.2. DHM MODELING COMPONENTS

The DHM model consists of a one-dimensional channel model, a two-dimensional floodplain model, and an interface sub-model. Thus the program has the capability to simulate both one and two-dimensional surface flow problems, such as one-dimensional open channel flow and two-dimensional dam-break problems, or a combination of the two. The interface model calculates the excess amount of water either from the channel element or from the floodplain element. This excess water is redistributed to the floodplain element or the channel element according to the water surface elevation.

For uniform grid elements, the integrated finite difference version of the nodal domain integration (NDI) method (Hromadka et al, 1981) is used. For grid elements, the NDI nodal equation is based on the usual nodal system shown in Figure 3.1. Flow rates across the boundary Γ are estimated by assuming a linear trial function between nodal points.

For the topographic model, grid elements are used to represent the topography. The grid elements are placed on the topography and elevations for each grid midpoint are estimated. The grids need not be placed in "floor-tile" fashion, but rather the grids are placed with emphasis on approximating the natural topographic flow patterns (i.e., the boundaries are perpendicular and parallel to the flow patterns).

Flow across grid boundaries is computed using equation (6). The flow characteristics are specified by a Manning's friction factor to be used for each boundary (unless a default value is used) of the grid.

In order to simulate the overland flow on the urban area, several enhancements have been made to the original DHM. These include the following major features:

(1) The effective flow-path: It is used to effectively block flows across some grid (floodplain element) boundaries and allow limited or full flow across the grid boundaries (see Figures 3.2).

(2) The effective area for floodplain element: It allows the available storage of a particular grid to be varied (see Figure 3.2).

(3) The stage-discharge rating curve: It can be used to simulate the channel and floodplain obstructions, such as bridges, culverts, freeway undercrossing, etc.

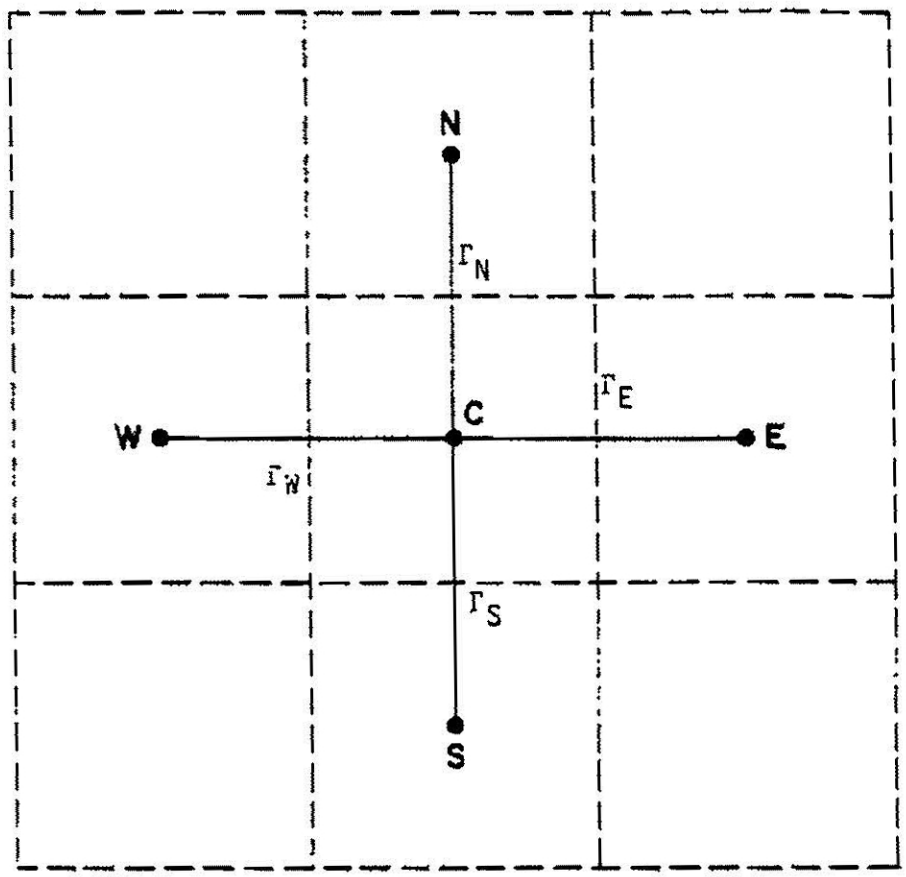

FIGURE 3.1 : DHM MODEL NODAL SYSTEM

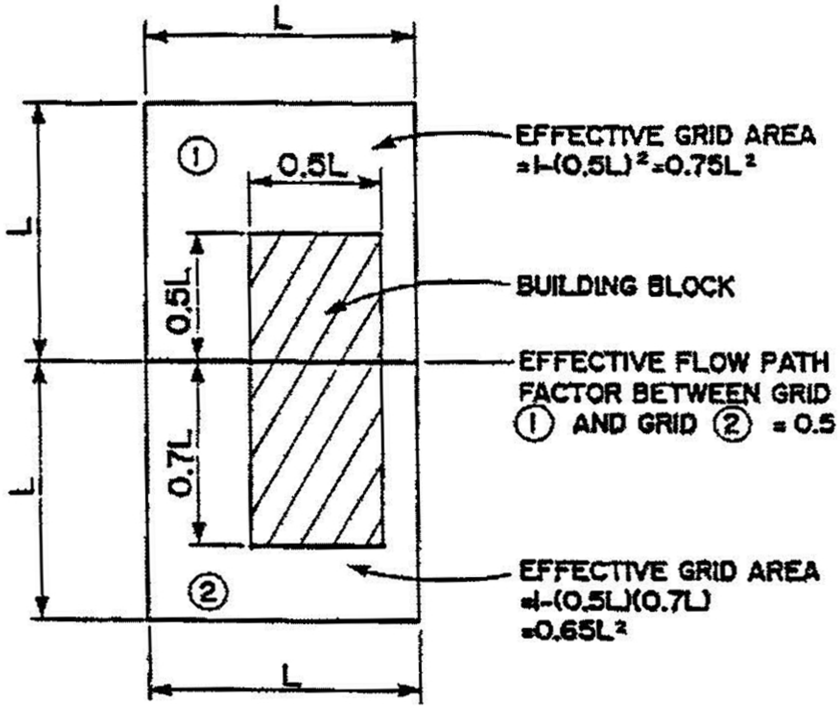

FIGURE 3.2 : DMH MODEL EFFECTIVE FLOW-PATH AND EFFECTIVE AREA ELEMENTS

(4) The storage element: A flood control retarding basin can be modeled by the storage element (depth vs. storage relationship) and stage discharge curve (depth vs. discharge relationship).

(5) The surcharged pipe element: Assumes a circular pipe system flowing under pressure (see Figure 3.3) with a hydraulic grade line which coincides with the water surface elevation of the connecting floodplain/channel element.

(6) The trapezoidal leveed channel element: The relative depth of the channel (see Figure 3.4), is defined as the difference between the representative floodplain elevation and the representative channel bottom elevation. The relative levee height is the difference between the representative top-of-levee elevation and the representative floodplain elevation.

Applications of these new DHM features are fully described in Section IV and Appendix C.

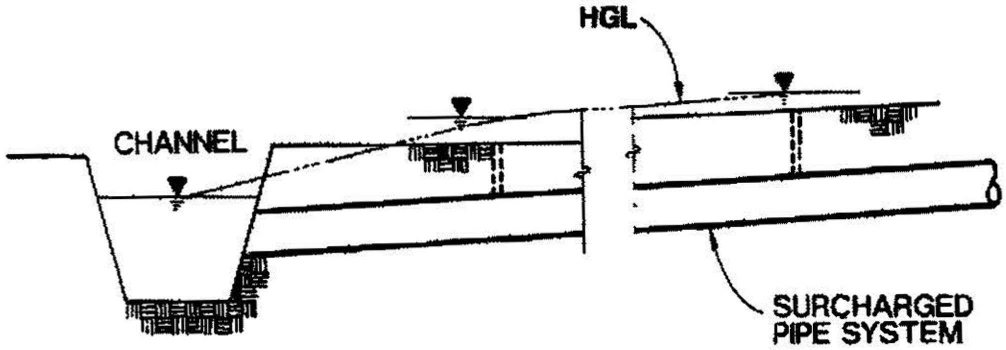

FIGURE 3.3 DHM MODEL SURCHARGED PIPE ELEMENT

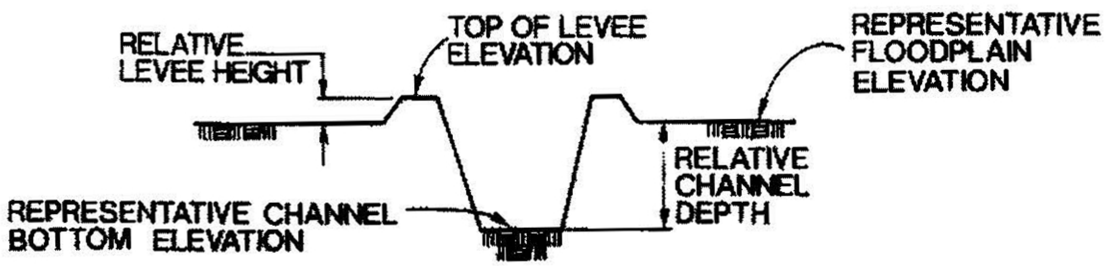

FIGURE 3.4 DHM MODEL TRAPEZOIDAL LEVEED CHANNEL ELEMENT

IV. DHM MODELING APPROACHES

For this inundation study, global and local detailed DHM models were used to analyze the existing and ultimate flood control systems for t-year events (i.e., 100-, 50-, 25-, and 10-year design storms). The global modeling area includes the entire C05/C06 watershed. The local detailed model emphasizes the neighboring area of the Haster and West Street retarding basins.

IV.1. GLOBAL MODEL

The global DHM model encompasses the entire C05/C06 watershed. The local terrain slopes southwesterly at a mild gradient (about 0.1% to 0.4%) and is fully developed with mixed residential and commercial developments. The storm runoff is collected by the local storm drain systems and then transported to the C05/C06 channel system which conveys the storm water into the Pacific Ocean.

The DHM floodplain grid schematic of the global model is shown on Figure 4.1. Using U.S.G.S. topographic 7.5 minute quadrangle maps (2000 scale), a 1000-foot grid discretization was prepared. Mean ground elevations for each grid were estimated from the maps. It was assumed that the storm runoff is confined in the street, i.e., only streets will be flooded due to the deficiency of the local storm drain system. In other words, the grid element has limiting storage capacity in the fully developed areas. An average value of 30-percent of the total grid area was used as the effective grid area for the fully developed areas. Average street section flow width within each grid element were estimated to be a total of 100 feet with a Manning's roughness coefficient of 0.02. Thus, the global Manning's roughness coefficient for each grid (1000-foot grid) is 0.20 (= 0.02 * 1000 feet/100 feet) with the exceptions of open areas where a Manning's roughness coefficient of 0.05 was used and freeway underpasses where an effective flow path factor and Manning's roughness coefficient of 0.02 were used (see section IV.1.1).

IV.1.1. Freeway Element

The Santa Ana Freeway, the Garden Grove Freeway, and the San Diego Freeway are the major landscapes on the watershed. Storm runoff may flow through freeways at various underpasses, such as streets, railroads and channel culverts.

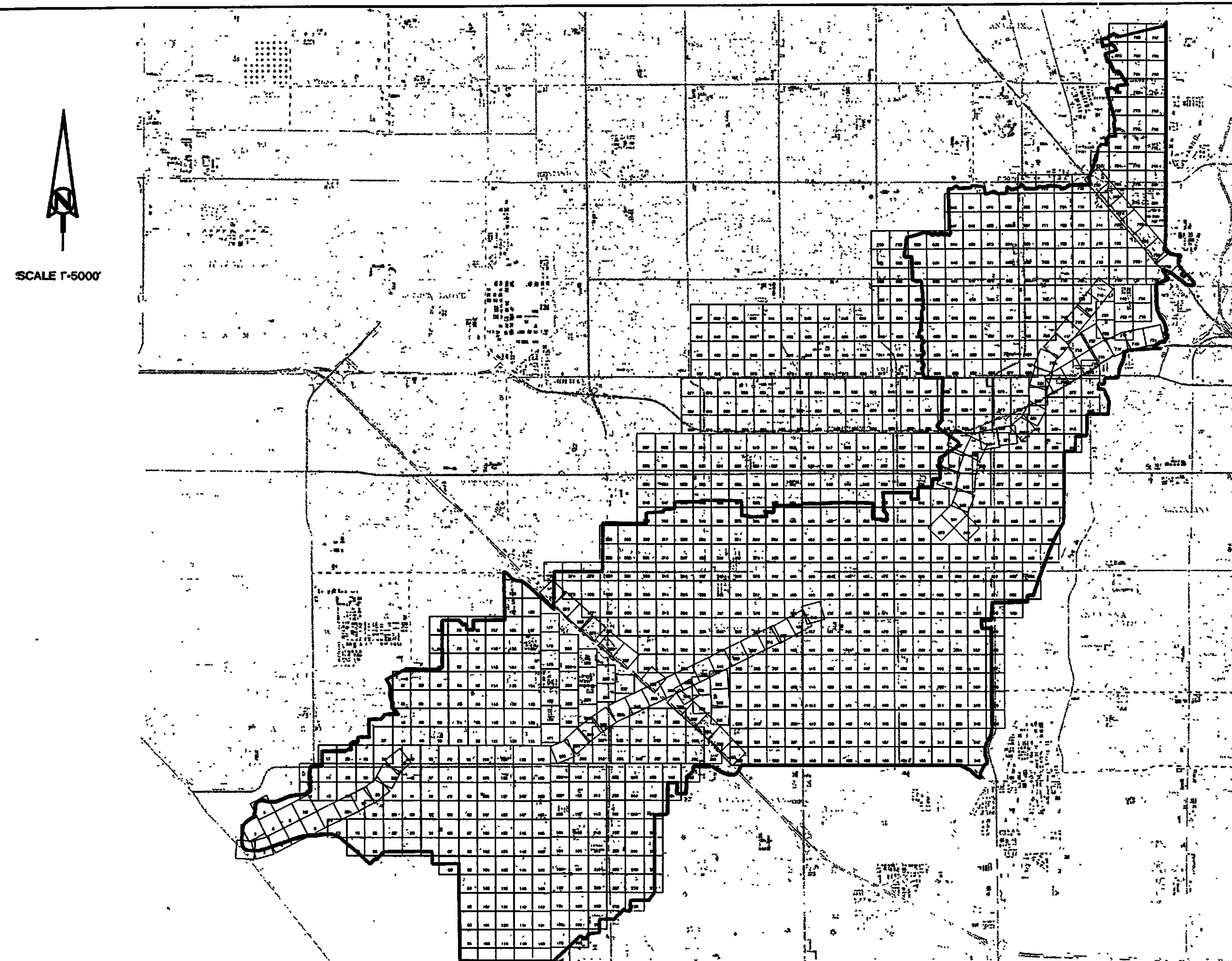

FIGURE 4.1
GLOBAL MODEL GRID
NETWORK SCHEMATIC

An effective flowpath and roughness coefficient of $n = 0.02$, was used to simulate the hydraulic characteristics for all the freeway underpasses. Table 4.1 lists the effective widths and freeway elements at each freeway underpass which conveys surface runoff under the freeway embankments.

Table 4.1.

FREEWAY UNDERPASSES AND GRID ELEMENTS

Location	Grid Element Numbers	Width of Underpass (feet)
Old Southern Pacific Railroad at I-5 Freeway	(786,769)	20
Garden Grove Boulevard at 22 Freeway	(714,618)	100
Harbor Boulevard at 22 Freeway	(675,833)	120
Trask Avenue at 22 Freeway	(622,563)	80
Newhope Street at 22 Freeway	(649,527)	200
Euclid Street at 22 Freeway	(823,827)	100
Taft Avenue at 22 Freeway	(824,919)	60
Brookhurst Street at 22 Freeway	(903,915)	100
Magnolia Street at 22 Freeway	(898,910)	100
Beach Boulevard at 405 Freeway	(280,255)	120

Culverts that convey storm runoff through the freeways are identified in Table 4.2. Estimated depth versus discharge relationships (see Appendix C) were used to represent culvert hydraulics under the freeway.

Table 4.2.

FREEWAY CULVERTS AND GRID ELEMENTS

Location	Grid Element Number
State College Boulevard at I-5 Freeway	(786,759)
Katella Avenue at I-5 Freeway	(762,745)
Orangewood Avenue at I-5 Freeway	(773,752)
C05 at 22 Freeway	(700,601)
C05 at 405 Freeway	(309,260)
C06 at 405 Freeway	(353,267)
Newland Avenue at C06	(245,246)
C05 at tide gate	(1)

IV.1.2. Storage Element

A storage element is a special floodplain element which has a specified depth versus storage relationship other than the ordinary floodplain element. For an ordinary floodplain element, the flood depth is calculated by dividing its flood flow volume by its effective area. On the other hand, the flood depth at a storage element is determined from the specified depth versus storage relationship. The outflow from the storage element is based on the specified depth versus discharge relationship. This is different from the regular floodplain element which conveys flood flow based on the two-dimensional unsteady flow equations (i.e., Eqs. 2a and 2b, section III.1).

There are two retarding basins (Haster and West Street Basins) on the north side of the Garden Grove Freeway and three storage facilities (Talbert Lake, Huntington Lake and Sand and Gravel Pit) in the City of Huntington Beach. The storage-elevation-discharge relationships were obtained from the feasibility study of the East Garden Grove-Wintersburg Channel by the Corps of Engineers (1988). Appendix C contains all the above-mentioned storage-elevation-discharge data.

In this study, the initial water surface elevations for the Haster and West Street Basins were assumed as the flowline elevations at the outlet structures. For other storage facilities (Talbert Lake, Huntington Lake and Sand and Grave Pit) which are not Orange County Flood Control District facilities, the initial water surface elevations were assumed to be at spillway elevation.

IV.1.3. Channel Element

a. Existing Channel System. A channel element can be described as a trapezoidal section which is situated on the center of the floodplain element. The relative depth of the channel is defined as the difference between the representative floodplain and channel bottom elevations as shown in Figure 3.4. The relative levee height of the channel is defined as the difference between the top-of-berm elevation and the representative floodplain elevation. It is important to realize that the relative channel depth and levee height may be different from the existing channel depth and levee height as specified on the construction plan. It is assumed in the floodplain and channel interface model that overflow from the channel is evenly distributed to the entire floodplain element and the overland flow that entered the channel is from the entire floodplain element. It should be noted that the DHM leveed channel element can prevent overland flow into the channel when flood depth is less than the representative levee height. Unfortunately, the overland flow is evenly distributed to the floodplain element, i.e., the leveed channel element can not confine overland flow to either side of the leveed channel. Thus, in order to keep overland flows from crossing elements with leveed channel sections, top of levee elevations were used as the representative flood plain elevation for the grids including channel sections from the confluence point of C05 and C06 to the C05 ocean outlet. The effective area factors were reduced to 20-percent for those floodplain/channel elements in order to simulate leveed channel sections. The effective area factors were increased to 45-percent at the adjacent floodplain elements to compensate for the losses of the effective volumes at floodplain/channel elements.

IV.1.4. Surcharged Pipe Element

The surcharged pipe element can be used to model the closed conduit drainage systems. It assumes a circular pipe system flowing under pressure with a hydraulic grade line that coincides with the water surface elevations of the connecting floodplain elements or floodplain and channel elements as shown in Figure 3.3. Non-circular pipe systems were converted into equivalent circular pipe systems for the entire study area. The surcharged pipe element can connect floodplain element to either floodplain element or channel element. Based on the water surface elevations of the two connecting elements and the Manning's equation, the flood water is transported instantaneously between these two connecting elements and results in an instantaneous change of water surface elevation on both connecting elements.

IV.1.5. Pump Stations

There are four existing pump stations in the City of Huntington Beach and one in the City of Fountain Valley. The Slater, Shields, and Marilyn Pump Stations convey storm water directly into C05 Channel System. The Heil and Sandalwood Pump Stations pump storm water into a local storm drain system which eventually connects to the C05 Channel System. Simplified channel and flood plain rating curves (see sections IV.1.7 and C.4) were used to model all pump stations. The Slater and Heil Pump Stations were modeled directly using the channel rating curve model option because these two pump stations convey storm water from open channel systems into open channel systems. The Shields and Marilyn Pump Stations, however, convey storm water from underground storm drains into an open channel, therefore, flood plain rating curves were used to simulate pumping from these stations sinse the DHM does not allow the definition of a rating curve outflow component at a storm drain pipe. The storm water at these two pump stations is assumed to bubble out before being conveyed into the C05 Channel System via the flood plain rating curve option. The Sandalwood Pump Station is modeled directly using the flood plain rating curve model option since it is located at the upstream most end of a local underground storm drain system. The rating curves simulating all five pump stations are included in Appendix C.

IV.1.6. T-Year Storm Events and Storm Centers

The temporal and spatial variabilities of a t-year storm event are simulated by applying different storm centerings over the entire C05/C06 watershed for each event. Six storm centers which progressed from the top of the watershed to the C05 Ocean outlet (see Figure 4.2) were used for each event to analyze the entire watershed response due to the uncertainties of storm locations. Table 4.3 lists the required t-year precipitation frequencies and corresponding AMC (antecedent moisture condition) designations to obtain 85-percent and 50-percent confidence level n-year runoff estimates using the Orange County Hydrology Manual (1986). Runoff estimates for the 100-, 50-, 25- and 10-year, 50% confidence frequency events were developed for the inundation study.

Data from the Hydrology Report of East Garden Grove-Wintersburg Channel by OCEMA (1991) were used to obtain effective rainfall information for the above-mentioned storm events and storm centerings. Derivations of the effective rainfall are shown in Appendix C.

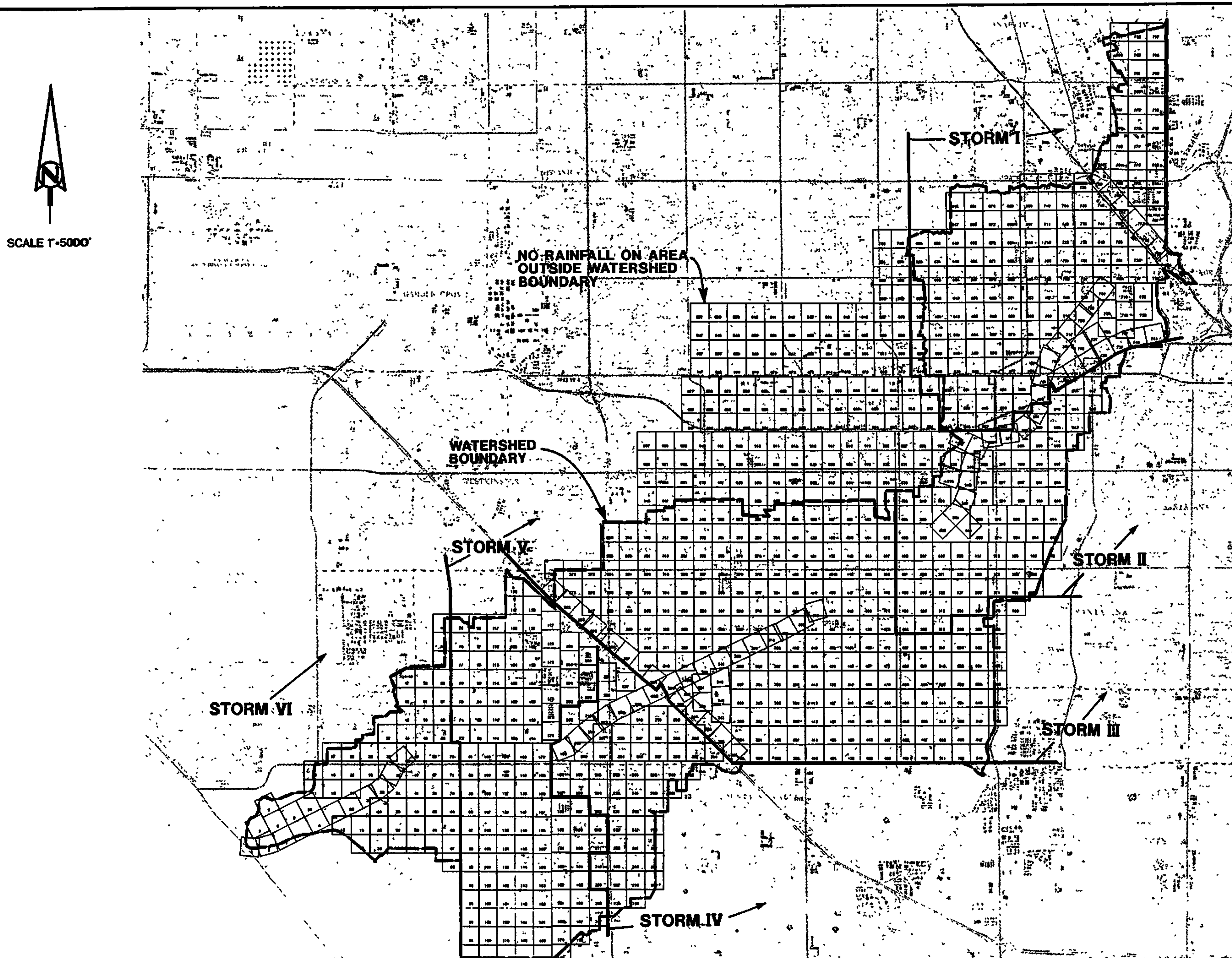

FIGURE 4.2
GLOBAL MODEL STORM
CENTER LOCATIONS

Table 4.3.

PRECIPITATION AND AMC DESIGNATIONS FOR COMPUTING 85% AND 50% CONFIDENCE LEVEL PEAK DISCHARGES, 1986 ORANGE COUNTY HYDROLOGY MANUAL

Runoff Frequency Desired	Model Input Precipitation Frequency and AMC for: 85% Confidence[1]	50% Confidence[2]
100-Year	100-Year, AMC III[3]	25-Year, AMC II
50-Year	50-Year, AMC II	15-Year, AMC II
25-Year	25-Year, AMC II	10-Year, AMC II
10-Year	10-Year, AMC II	5-Year, AMC II
5-Year	5-Year, AMC I[3]	4-Year, AMC II
2-Year	2-Year, AMC I[3]	2-Year, AMC II

1. Peak Discharges calculated using these criteria and the procedures outlined in the 1986 Orange County Hydrology Manual are considered to be 85% confidence level estimates.

2. Peak discharges calculated using these criteria and the procedures outlined in the 1986 Orange County Hydrology Manual are considered to be 50% confidence level estimates.

3. Use of AMC I and III is an OCEMA Policy Statement, whereas calibration results recommend use of AMC II.

IV.1.7. Initial Conditions and Boundary Conditions

Initial Conditions. Initially, a depth of 0.15 feet was assigned to the entire DHM modeling area. The purpose of assuming an initial depth was to avoid using a small time step (e.g., less than 1- second) in time domain advancement. The initial depth of 0.15 feet was chosen as the average gutter hike depth which allowed the minimum time step to be set at 5 seconds.

Boundary Conditions. Three types of outflow boundary conditions were used in the modeling area (see Figure 4.3). No flow boundary condition was assigned to the floodplain boundaries where no flow was allowed to cross. Surface water that migrates into neighboring watersheds without returning to the modeling area was modeled by the critical boundary condition. The third

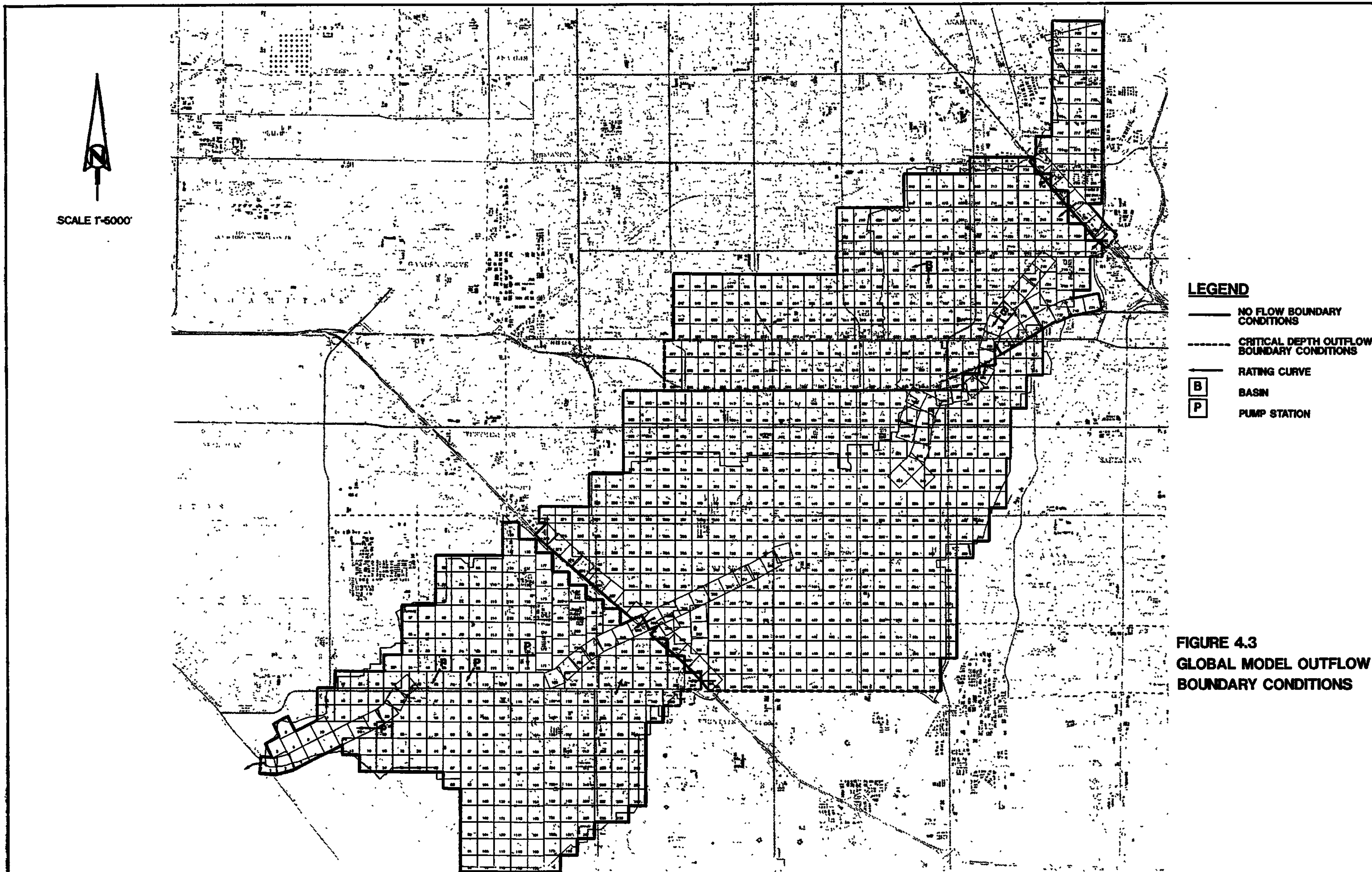

FIGURE 4.3
GLOBAL MODEL OUTFLOW BOUNDARY CONDITIONS

boundary condition uses the specified rating curve (i.e., depth versus discharge relationship) to model a control outlet structure. Rating curves were used to model the flow conditions at the C05 Ocean outlet, Haster Basin, West Street Basin, and channel undercrossings at various freeways and pump stations. This information is contained in Appendix C.

IV.2. DETAILED MODEL

A Detailed model was used to further investigate the potential flooding in the areas neighboring Haster and West Street Retarding Basins. Figure 4.4 outlines the detailed modeling area which encompassed an area of about 2,500 acres. Using a U.S.G.S. 7.5 minute topographic quadrangle map (enlarged to 500 scale), a 500-foot grid discretization was prepared (see Figure 4.5). The Detailed DHM model used the same floodplain characteristics as described in the Global model with the exceptions discussed below.

IV.2.1. Topographic Data

A U.S.G.S. 7.5 minute topographic map enlarged to 500 scale was used as a base map. Additional topographic information was compiled for the detailed model study area from the following sources:

a. Spot elevations at street intersections from street improvement plans,

b. One hundred scale aerial topographic data for a portion of the area provided by the City of Garden Grove,

c. Flow direction data provided on the County's street flow map for this area.

IV.2.2. Surcharged Pipe Elements

Special attention has been paid to the surcharged pipe elements in the Detailed model to avoid pipeflow stagnation. As illustrated in Figure 4.6, pipe stagnation will occur only when the slope of the surcharged pipe and the slope of the ground are in opposite sign. From Figure 4.6, water will stagnate at Node A until water surface elevation at Node A is greater than the ground elevation of Node B.

The Global model experienced minor effects of the pipeflow stagnation because larger elements (1000 x 1000 foot) were used. This is because the large element tended to smooth out the local topographic variabilities where smaller elements preserved some of the local topographic variabilities. Connections between surcharged pipe elements were re-established in the Detailed model in order to avoid the pipeflow stagnation.

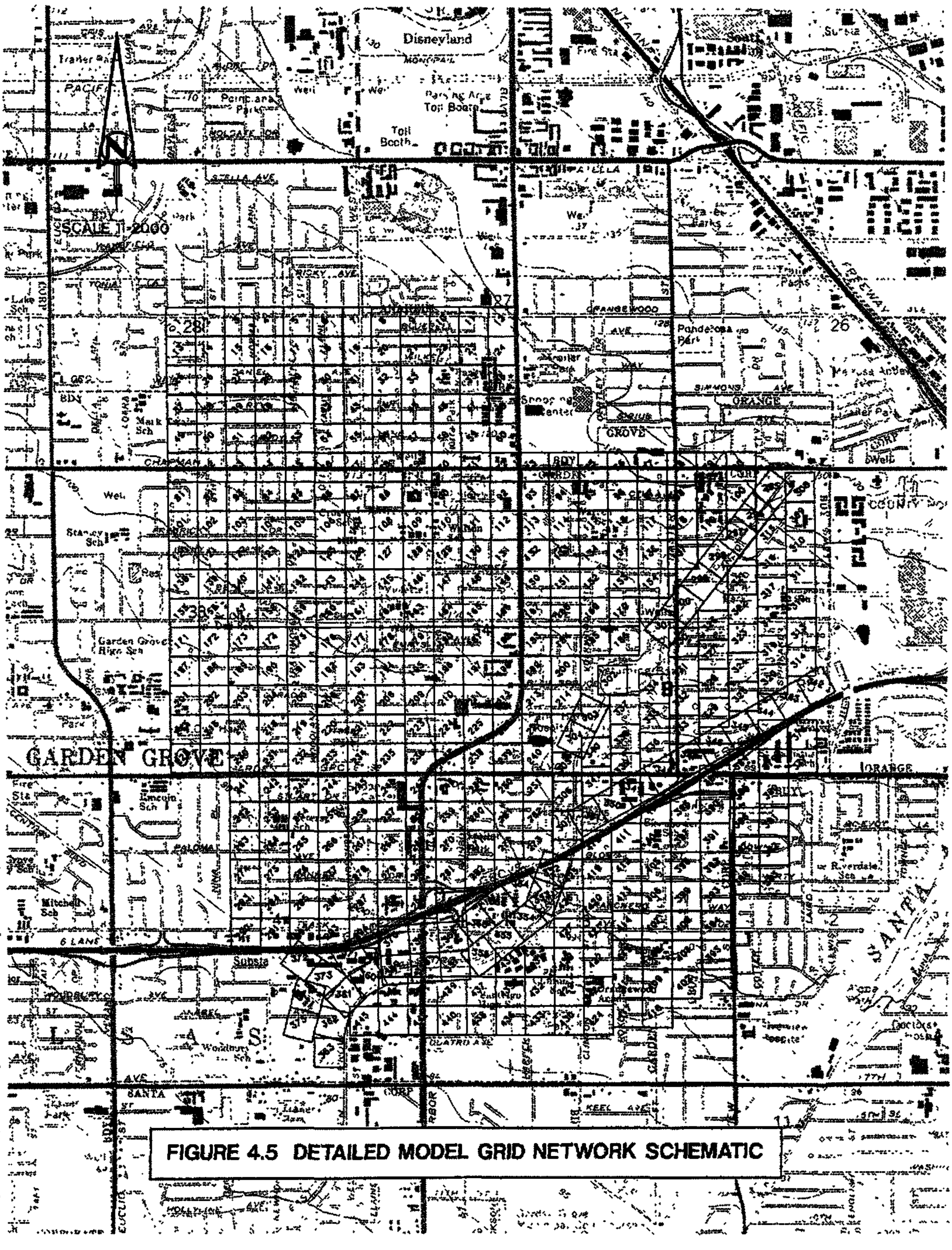

FIGURE 4.5 DETAILED MODEL GRID NETWORK SCHEMATIC

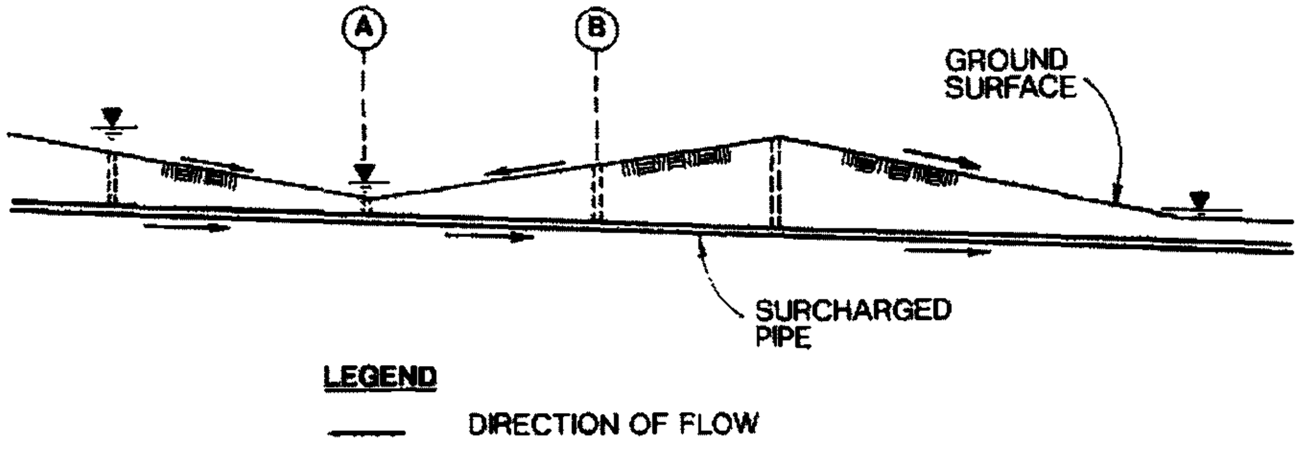

FIGURE 4.6 SURCHARGED PIPEFLOW STAGNATION

WILLIAMSON & SCHMID EXHIBIT

IV.2.3. Boundary Conditions

T-year precipitation events with Storm Center I as described in the Global model were used in the Detailed model. Cascading flow from other parts of the watershed into the Detailed modeling area were extracted from the Global model results.

Boundary conditions for the Detailed model are shown on Figure 4.7. Overland flow along Global elements 647, 660, 673, 685, 696, 695, 694, 708, 720, 731, 741, 749, 748, 747, and 737 was uniformly distributed the north and east boundaries of the Detailed model as inflow boundary conditions which represented the cascaded flows from adjacent area.

Channel flow and surcharged pipe flows that entered the Detailed model were assigned to the corresponding channel and floodplain elements as follows:

Channel flow from the upstream of the Lewis Channel was assigned to Channel element 342. Surcharged pipeflow at upstream of the Haster Basin was assigned to Floodplain element 295. Surcharged pipeflow at Harbor Boulevard and Chapman Avenue was assigned to Floodplain element 73 and surcharged pipeflow at West Street and Orangewood Avenue was assigned to Floodplain element 7.

In addition to the no flow and critical depth outflow boundary conditions (see Figure 4.7), a channel outflow rating curve was used to convey channel flow from the Detailed modeling area. No flow boundary conditions were assigned to flood plain elements adjacent to the Garden Grove Freeway where surface runoff can only flow through the freeway undercrossings and to the south-east and north-west of the Detailed model boundaries where minor overland flow was estimated from the Global model results. The critical depth outflow boundary condition was assigned to south and south-west of the Detailed model boundaries to convey overland flow away from the Detailed modeling area. A channel outflow rating curve (see Table C.9) was assigned to Channel element 353 to convey channel flow from the Detailed modeling area.

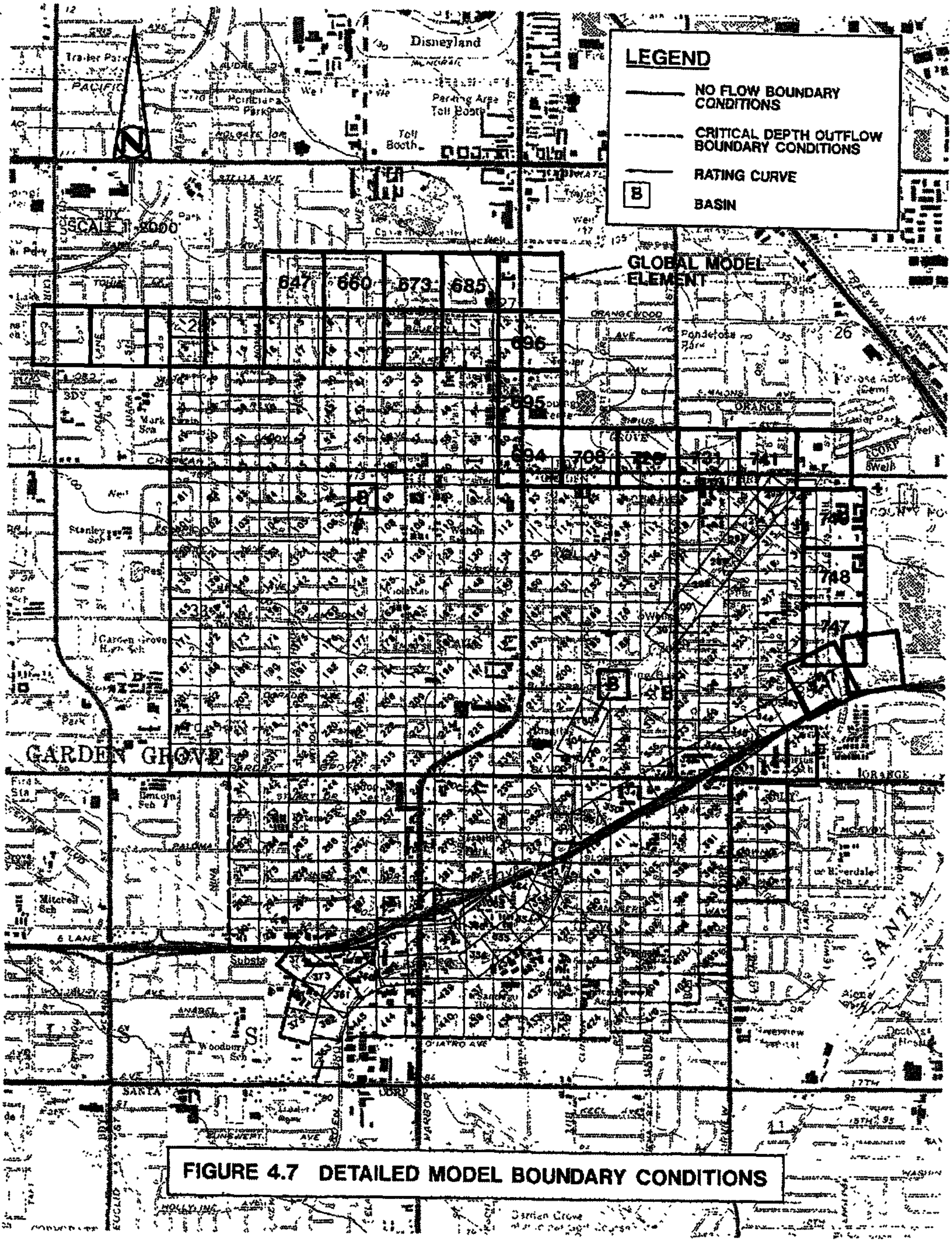

FIGURE 4.7 DETAILED MODEL BOUNDARY CONDITIONS

V. DHM MODELING RESULTS

V.1. GLOBAL MODEL RESULTS (EXISTING CHANNEL SYSTEM)

For each N-year event, the maximum flood depths from all six storm centerings were compiled to show the worst possible flooding within the entire watershed. Figures 5.1 through 5.4 depict the maximum flood depths in the study area for the 10-, 25-, 50-, and 100-year events, respectively. Summary outputs for the Global model are contained in Appendix D. Inundation maps of 1" = 2000' and 1" = 500' scale for the Global and Detailed models, respectively, are enclosed in the back of this report.

Table 5.1 summarizes the flooding areas where estimated flood depths are greater than 0.5 feet. Flooding is caused by the lower design standard of the local storm drainage systems and the existing capacity of the C05/C06 channel system. From Table 5.1, the West Street Basin is overtopped during each n-year event. There is no overtopping estimated for the Haster Basin, but emergency spillway flow occurs during the 25-, 50-, and 100-year events.

The DHM Global model was extended westerly along the Garden Grove Freeway twice (i.e., floodplain elements 799 to 958 as shown on Figure 4.1) to accommodate the overland flows that leave and re-enter the C05/C06 watershed boundary.

Maximum and averaged flood depths for those flooded areas identified by the DHM Global model are shown on Table 5.1. The estimated flooded depth at the Sand and Gravel Pit was not used in the averaging process to obtain the averaged depth for its neighboring area. This is because the adjacent floodplain elements have much higher ground elevations than the Sand and Gravel Pit element (see appendix C).

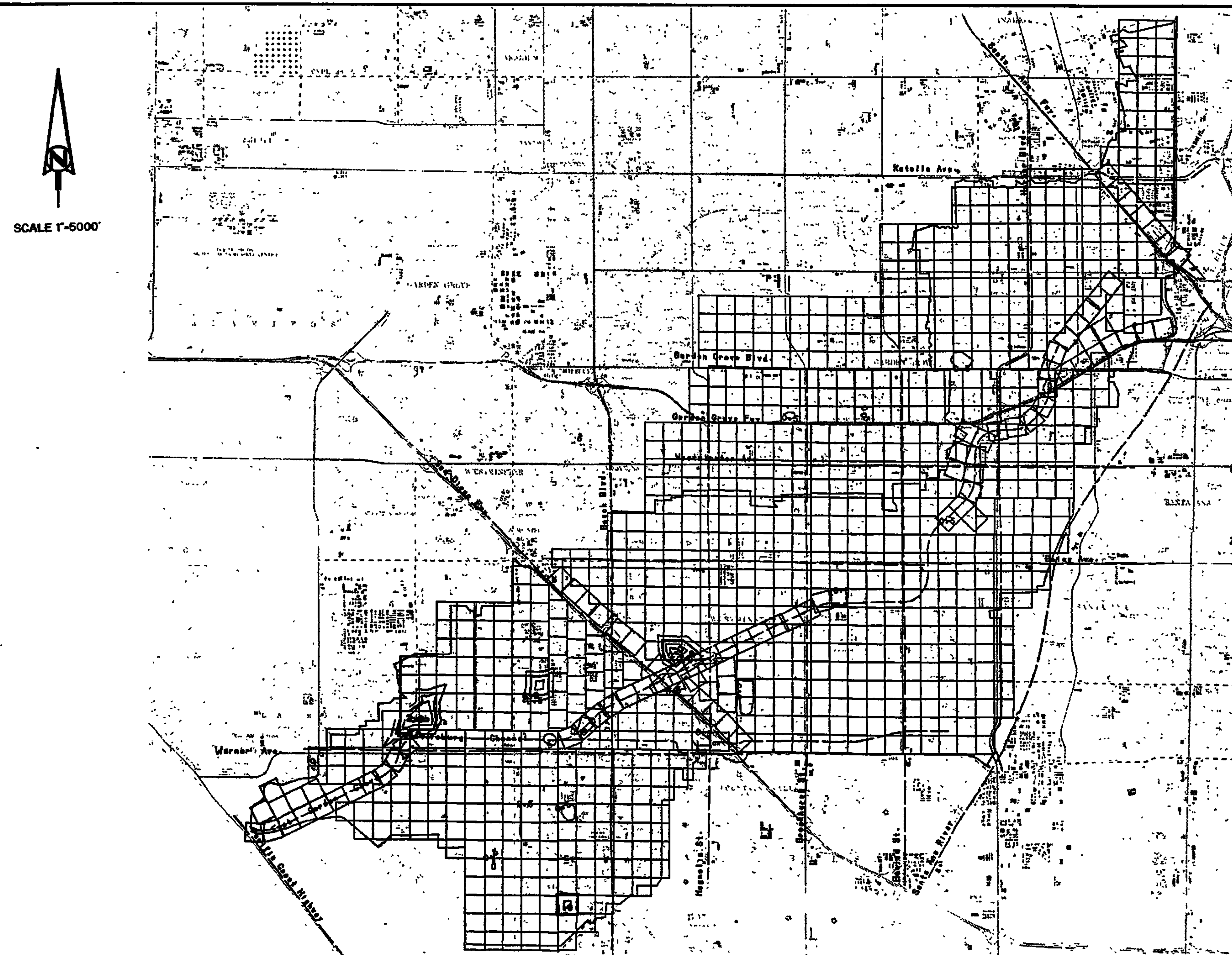

FIGURE 5.1
GLOBAL MODEL RESULTS, 10-YEAR MAXIMUM FLOODING DEPTHS

50% CONFIDENCE LEVEL

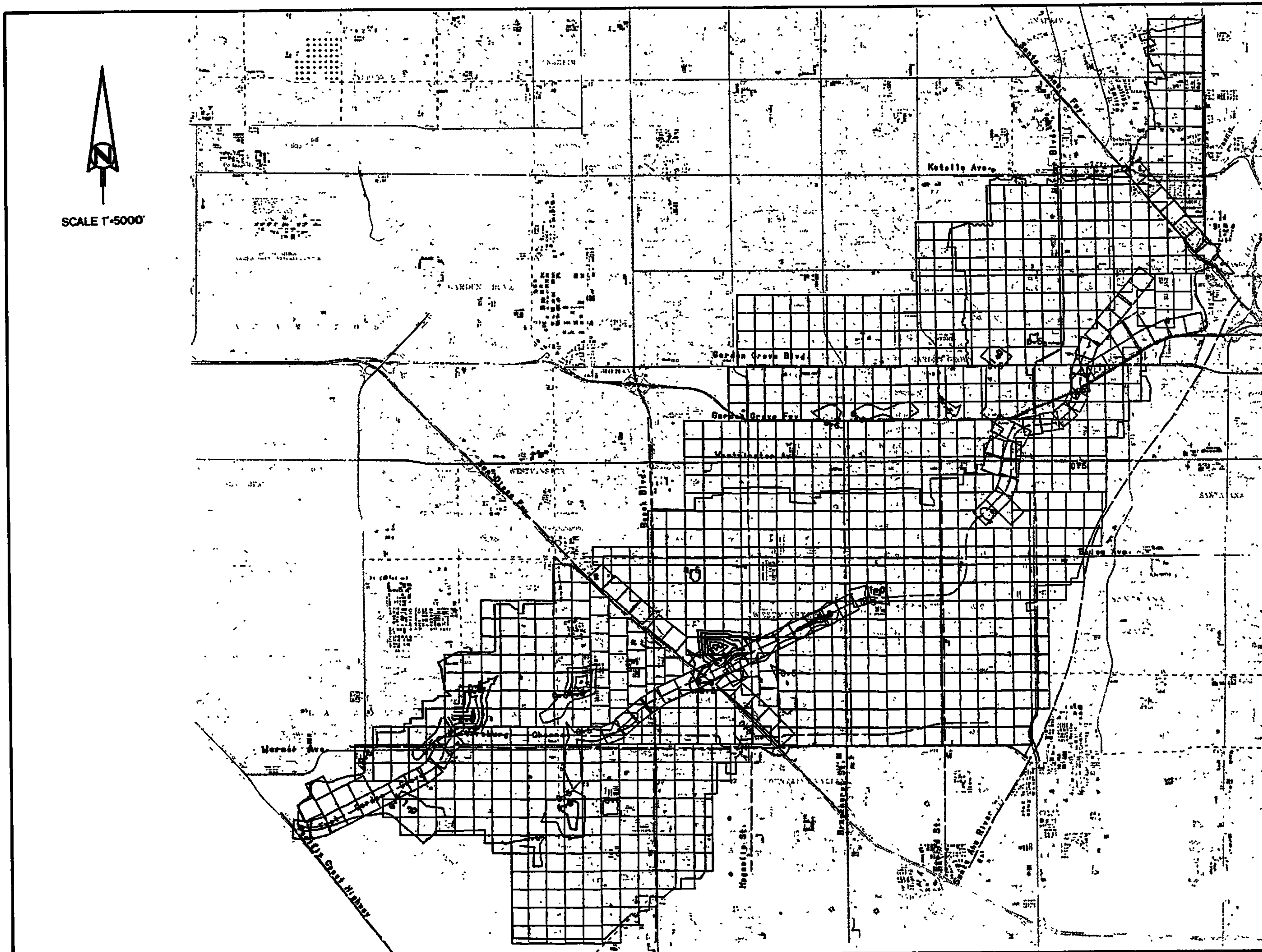

FIGURE 5.2
GLOBAL MODEL RESULTS,
25-YEAR MAXIMUM
FLOODING DEPTHS

50% CONFIDENCE LEVEL

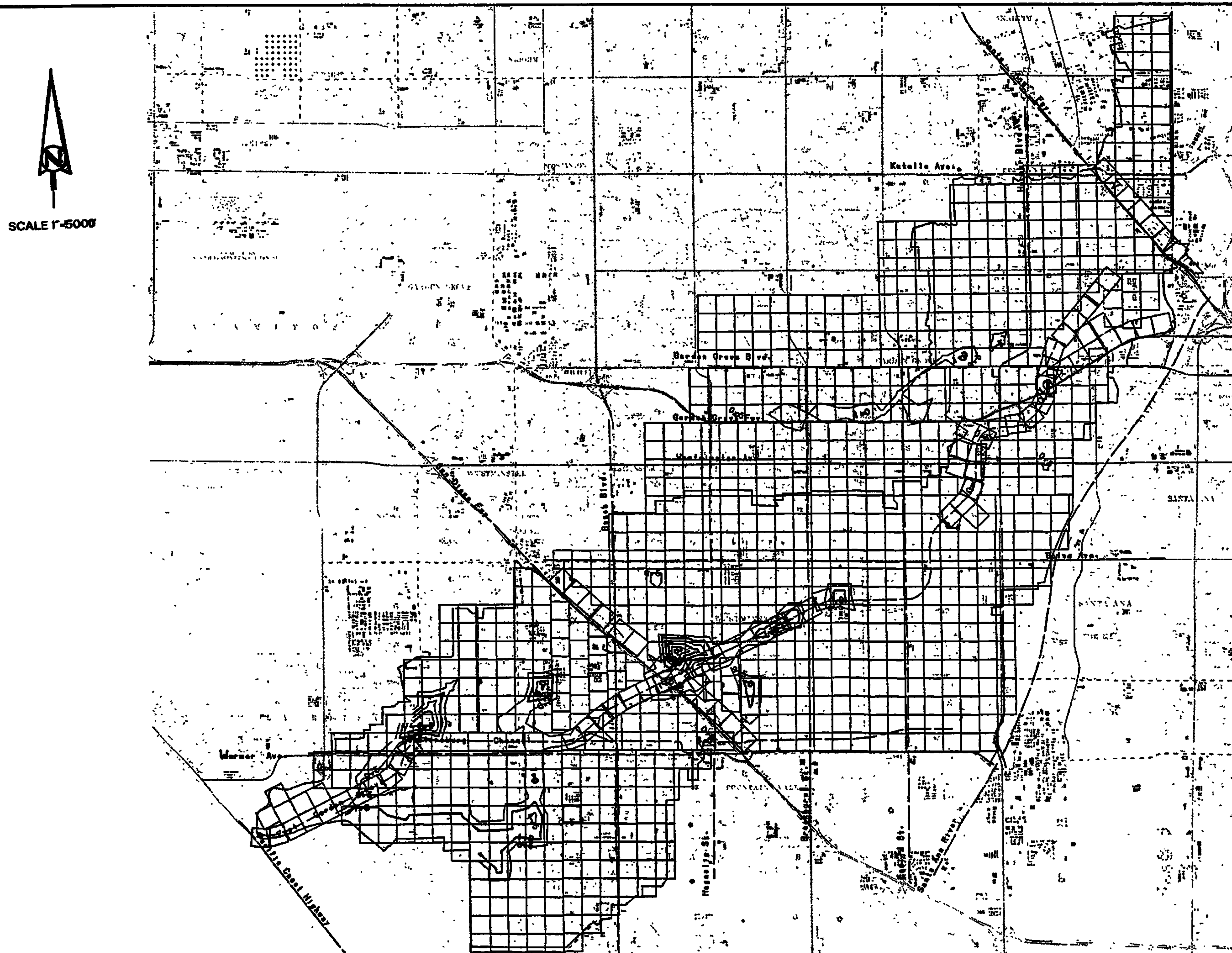

FIGURE 5.3
GLOBAL MODEL RESULTS, 50-YEAR MAXIMUM FLOODING DEPTHS

50% CONFIDENCE LEVEL

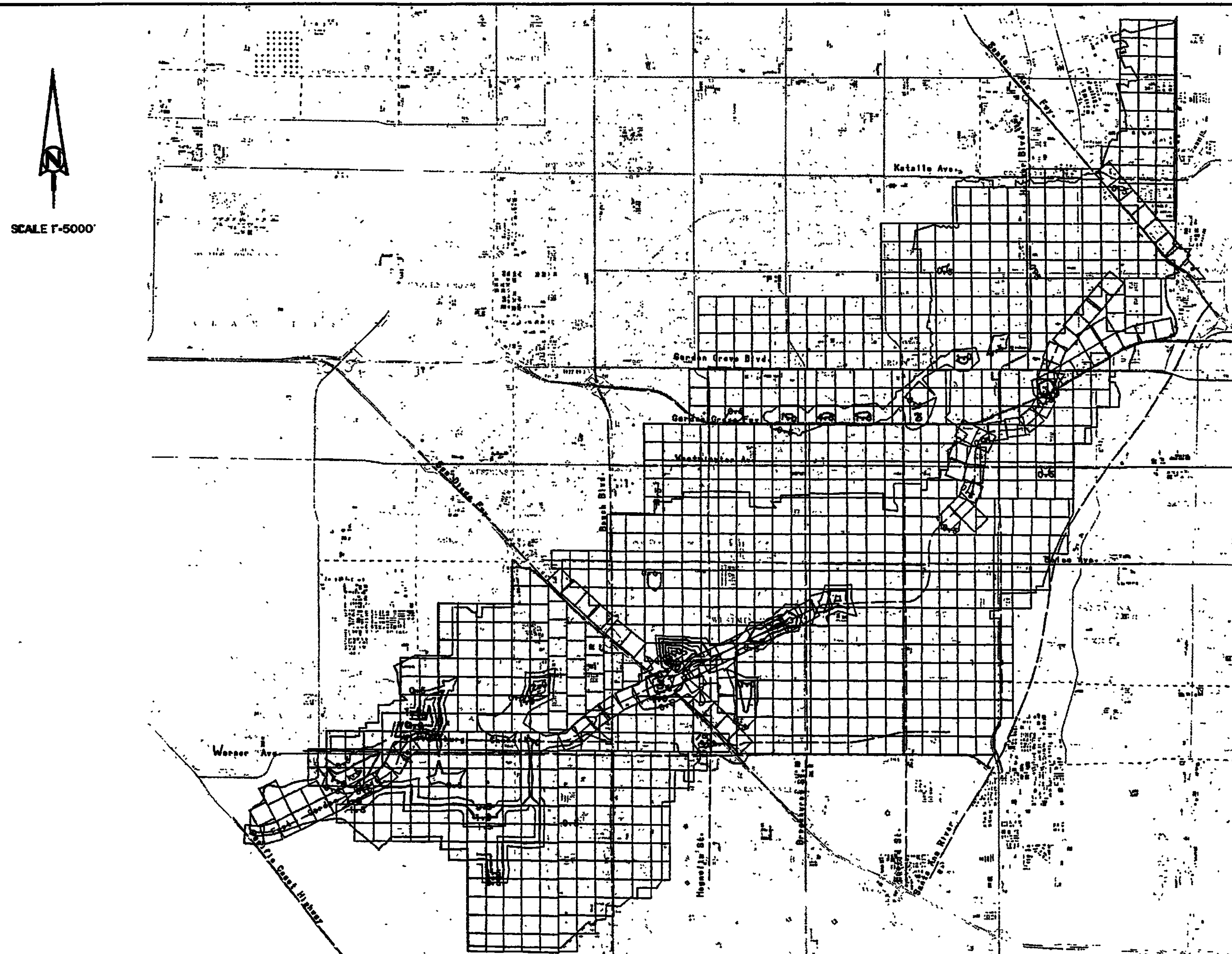

FIGURE 5.4
GLOBAL MODEL RESULTS, 100-YEAR MAXIMUM FLOODING DEPTHS

50% CONFIDENCE LEVEL

Table 5.1.

SUMMARY OF GLOBAL MODEL RESULTS

Location	n-year event	Maximum Depth (ft)	Averaged Depth (ft)	Floodplain Elements
City of Huntington Beach				
Springdale Street @ Slater Channel	10	–	–	–
	25	1.0	0.7	13,18,19,24,25, 31,32
	50	1.3	1.0	13,18,19,24,25, 26,31,32,33,38, 39,53,54
	100	1.7	1.6	13,18,19,24,25, 26,31,32,33,38, 39, 53,54
Huntington Lake Area	10	0.5	0.5	103,104,105
	25	0.6	0.5	66,85,103,104,105
	50	1.2	1.1	66,67,68,85,86, 103,104,105
	100	1.8	1.4	66,67,68,85,86,87, 103,104,105,106
Talbert Lake Area	10	1.0	1.0	145,146
	25	1.4	1.1	125,145,146,185
	50	1.6	1.2	124,125,145,146, 147,185
	100	2.0	1.6	124,125,145,146, 147,185
Sand and Gravel Pit Area	10	3.9	1.9*	141,142,161,180
	25	4.4	2.5*	141,142,161,180
	50	4.6	2.8*	141,142,161,180
	100	4.8	3.3*	141,142,161,180
Lake View School	10	–	–	
	25	–	–	
	50	–	–	–
	100	0.6	0.6	56,57,70
Shields Pump Station	10	2.1	1.6	36,44,49,50
	25	2.2	1.3	29,36,44,49,50,61
	50	2.3	1.3	21,29,36,44,49,50, 61
	100	2.4	1.3	20,21,22,29,36,44, 49,50,58,61

(Continued)

* Depth at Sand and Gravel Pit excluded.

Table 5.1.
SUMMARY OF GLOBAL MODEL RESULTS

Location	n-year event	Maximum Depth (ft)	Averaged Depth (ft)	Floodplain Elements
City of Huntington Beach (Continued)				
Marilyn Pump Station	10	–	–	–
	25	–	–	–
	50	–	–	–
	100	0.8	0.8	92
Heil Pump Station	10	1.9	1.9	153
	25	2.1	1.1	111,131,153
	50	2.1	1.0	111,130,131,153
	100	2.2	1.0	111,130,131,152, 153
Confluence of C05 and C06	10	0.9	0.8	170,201,
	25	1.0	0.8	148,149,150,170 189,201
	50	1.0	0.8	148,149,150,170 189,201,216
	100	1.1	0.9	148,149,150,170, 189,201,216
Downstream of C06 at 405 Freeway	10	1.0	1.0	267
	25	1.0	1.0	267
	50	1.1	1.1	267
	100	1.1	1.1	267
Cities of Huntington Beach/Westminster				
Downstream of C05 at 405 Freeway	10	0.9	0.9	260
	25	1.4	1.4	260
	50	1.7	1.7	260
	100	2.1	1.0	248,249,259,260
Upstream of C05, 405 Freeway	10	2.3	1.0	308,309,310,321, 332
	25	3.2	1.3	294,309,310,319, 321,332,344,359,
	50	3.2	1.3	294,307,308,309, 310,319,321,332, 344,359,366,374, 389
	100	3.2	1.3	294,307,308,309, 310,319,321,332, 344,359,366,374, 389

(Continued)

Table 5.1.

SUMMARY OF GLOBAL MODEL RESULTS

Location	n-year event	Maximum Depth (ft)	Averaged Depth (ft)	Floodplain Elements
City of Westminster/ Midway City				
Intersection of Newland/Bolsa	10	–	–	–
	25	0.7	0.7	299
	50	0.7	0.7	299
	100	0.8	0.8	299
City of Fountain Valley				
Intersection of Bushard/Heil	10	0.8	0.4	355,356
	25	0.9	0.5	355,356
	50	1.2	0.6	354,355,356
	100	1.3	0.8	354,355,356
City of Westminster				
Anthony School	10	0.9	0.9	432
	25	1.2	1.2	432
	50	1.4	1.4	432
	100	1.7	1.0	417,418,432
City of Santa Ana				
Intersection of Hazard/Newhope	10	0.6	0.6	523
	25	0.7	0.7	523
	50	0.8	0.8	523
	100	0.9	0.9	523
Intersection of Morningside/ Hastings	10	–	–	–
	25	–	–	–
	50	0.5	0.5	542
	100	0.6	0.6	542
Intersection of Westminster/ Clinton	10	–	–	–
	25	–	–	–
	50	0.6	0.6	597
	100	0.6	0.6	597
City of Garden Grove				
Confluence of C05 and C05S10	10	–	–	–
	25	–	–	–
	50	0.5	0.5	545
	100	0.6	0.6	545

(Continued)

Table 5.1.

SUMMARY OF GLOBAL MODEL RESULTS

Location	n-year event	Maximum Depth (ft)	Averaged Depth (ft)	Floodplain Elements
City of Garden Grove (Continued)				
Intersection of Lemonwood/Garden Grove Boulevard	10	0.9	0.6	628,639
	25	1.1	0.8	628,639
	50	1.1	0.9	628,639
	100	1.2	0.9	628,639
West Street Basin	10	6.9	6.9	656
	25	6.9	6.9	656
	50	6.9	6.9	656
	100	6.9	6.9	656
Intersection of Buaro/Emrys	10	--	--	--
	25	0.6	0.6	666
	50	0.7	0.7	666
	100	0.8	0.8	666
Intersection of Chapman/Candy	10	--	--	--
	25	--	--	--
	50	--	--	--
	100	0.6	0.6	633
Upstream of C05 at 22 Freeway	10	0.6	0.5	700,701
	25	0.8	0.6	700,701
	50	0.9	0.7	700,701
	100	0.9	0.7	700,701
Haster Basin	10	4.1	4.1	704
	25	5.0	5.0	704
	50	5.5	5.5	704
	100	6.0	6.0	704
Cities of Garden Grove/Anaheim				
Intersection of Chapman/Harbor	10	--	--	--
	25	--	--	--
	50	--	--	--
	100	0.5	0.5	693,694
City of Anaheim				
Intersection of Lewis/Pacific	10	--	--	--
	25	--	--	--
	50	0.6	0.6	762
	100	0.6	0.6	762

The estimated flooded areas due to the interface between channel and floodplain elements are listed in Table 5.2. Two types of channel overtopping occur. Type I overtopping indicates that the channel has limited capacity and the water surface elevations are the same for both the channel and floodplain elements. Type II overtopping indicates that water overtops the channel but stays inside the channel. Estimated type I overtopping areas along the C05 channel system are: (1) near Anthony School in the City of Westminster, (2) Between Bushard Street and Brookhurst Avenue, (3) downstream of 405 Freeway, (4) between Golden West Street and Springdale Avenue, and (5) Slater Pump Station. No channel overtopping was estimated for the C06 channel system by the DHM Global model. The Heil Street storm channel (C5-SC-2) and the Slater storm channel in the City of Huntington Beach are the other two channel systems where type I overtopping occurs. The causes of the type II overtopping may be due to either the deficiencies of the local storm drain systems (such as elements 523, 309, 155, 76, 243, 244, 245 and 287 shown on Table 5.2) or the cascaded flows from type I overtopping areas (such as elements 359, 309, 249 and 236 shown on Table 5.2), or both.

Table 5.2.

GLOBAL MODEL, LOCATIONS OF CHANNEL/FLOODPLAIN INTERFACE

			Channel Overflow		Floodplain Overflow	
Channel System	Element/ Location	n-Year Event	Maximum Depth (feet)	Levee Height*/ Channel Depth (feet)	Maximum Depth (feet)	Levee Height (feet)
C05						
	700	10	-	8.5	–	
	Upstream	25	8.5		–	
	of 22 Freeway	50	8.5		–	
		100	8.5		–	
	523	10	-		–	1.0
	@ Hazard/	25	-		–	
	Newhope	50	-		–	
		100	-		1.0	
	423	10	-	10.0	1.0	1.0
	Anthony	25	10.3		1.3	
	School	50	10.5		1.5	
		100	10.8		1.8	
	389	10	-	11.5	–	1.5
	Between	25	11.6		–	
	Bushard/	50	11.7		1.7	
	Brookhurst	100	12.0		2.0	
	374	10	11.0	11.0	–	1.0
	Between	25	11.4		1.4	
	Bushard/	50	11.7		1.7	
	Brookhurst	100	11.9		1.9	
	359	10	-		–	1.0
	South of	25	-		1.0	
	Bushard	50	-		1.0	
		100	-		1.0	
	309	10	-		–	1.5
	Upstream of	25	-		1.5	
	405 Freeway	50	-		1.5	
		100	-		1.5	
	260	10	10.6	10.0	1.1	0.5
	Downstream of	25	11.1		1.6	
	405 Freeway	50	11.3		1.8	
		100	11.7		2.2	

(Continued)

Notes:
- \- Indicates no overflow from channel
- – Indicates no overflow into channel
- * Indicates relative levee height/channel depth (see Figure 3.4)

Table 5.2.

GLOBAL MODEL, LOCATIONS OF CHANNEL/FLOODPLAIN INTERFACE
(Continued)

Channel System	Element/ Location	n-Year Event	Channel Overflow Maximum Depth (feet)	Channel Overflow Levee Height*/ Channel Depth (feet)	Floodplain Overflow Maximum Depth (feet)	Floodplain Overflow Levee Height (feet)
C05						
	249	10	-	11.7	0.5	0.5
	Downstream of	25	-		0.5	
	405 Freeway	50	-		0.5	
		100	12.2		1.0	
	236	10	-		0.5	0.5
	Downstream of	25	-		0.5	
	405 Freeway	50	-		0.5	
		100	-		0.5	
	150	10	-	11.1	–	0.
	Marilyn Pump	25	11.8		0.7	
	Station	50	12.2		1.1	
		100	12.4		1.3	
	129	10	-	11.7	–	0.
	Marilyn Pump	10	11.8		–	
	Station	50	12.1		0.4	
		100	12.3		0.6	
	110	10	-	11.6	–	0.
	Marilyn Pump	25	11.7		–	
	Station	50	12.0		0.4	
		100	12.1		0.5	
	72	10	-	12.3	–	
	Shield Pump	25	-		–	
	Station	50	-		–	
		100	12.4		–	
	58	10	-	11.3	–	0.
	Marilyn Pump	25	11.7		0.4	
	Station	50	11.9		0.6	
		100	12.0		0.7	
	20	10	-	12.9	–	0.
	Slater Pump	25	13.4		0.5	
	Station	50	13.5		0.6	
		100	13.6		0.7	

(Continued)

Notes:
- \- Indicates no overflow from channel
- – Indicates no overflow into channel
- * Indicates relative levee height/channel depth (see Figure 3.4)

Table 5.2.

GLOBAL MODEL, LOCATIONS OF CHANNEL/FLOODPLAIN INTERFACE
(Continued)

Channel System	Element/ Location	n-Year Event	Channel Overflow Maximum Depth (feet)	Channel Overflow Levee Height*/ Channel Depth (feet)	Floodplain Overflow Maximum Depth (feet)	Floodplain Overflow Levee Height (feet)
C5-SC-2						
	155	10	-	-	–	0.5
	@ Edinger	25	-		–	
	Avenue	50	-		0.5	
		100	-		0.5	
	153	10	10.1	8.5	2.1	0.5
	@ Magellan	25	10.2		2.2	
	Lane	50	10.3		2.3	
		100	10.3		2.3	
	152	10	-	7.5	–	0.5
	@ Heil Avenue	25	7.5		0.5	
		50	7.6		0.6	
		100	7.7		0.7	
Slater						
	124	10	-	8.5	0.5	0.5
	@ Golden	25	8.6		0.6	
	West	50	9.4		1.4	
		100	10.1		2.1	
	105	10	-	9.5	0.5	0.5
	Between	25	-		0.5	
	Golden West	50	10.3		1.3	
	and Edwards	100	11.0		2.0	
	86	10	-	9.5	0.5	0.5
	Between	25	-		0.5	
	Golden West	50	10.3		1.3	
	and Edwards	100	11.0		2.0	
	87	10	-	10.5	–	0.
	Between	25	-		–	
	Golden West	50	-		–	
	and Edwards	100	11.0		1.0	
	69	10	-	10.0	–	0.
	@ Slater/	25	-		–	
	Springdale	50	-		–	
		100	10.6		0.6	

(Continued)

Notes:
- \- Indicates no overflow from channel
- – Indicates no overflow into channel
- * Indicates relative levee height/channel depth (see Figure 3.4)

Table 5.2.

GLOBAL MODEL, LOCATIONS OF CHANNEL/FLOODPLAIN INTERFACE
(Continued)

Channel System	Element/ Location	n-Year Event	Channel Overflow Maximum Depth (feet)	Channel Overflow Levee Height*/ Channel Depth (feet)	Floodplain Overflow Maximum Depth (feet)	Floodplain Overflow Levee Height (feet)
Slater						
	55	10	-	10.0	–	0.
	Between	25	-		–	
	Edwards/	50	-		–	
	Springdale	100	10.4		0.4	
	40	10	-	11.0	–	0.
	@ Springdale	25	-		–	
		50	-		–	
		100	11.5		0.5	
	33	10	-	13.0	–	0.
	Between	25	-		–	
	Graham/	50	14.0		1.0	
	Springdale	100	14.9		1.9	
	26	10	-	13.0	–	0.
	Between	25	-		–	
	Graham/	50	13.9		0.9	
	Springdale	100	14.8		1.8	
C3-SC-3						
	76	10	-		–	0.5
	@ South of	25	-		–	
	Edinger	50	-		0.5	
		100	-		0.5	
C6-SC-1						
	243	10	-		0.5	0.5
	@ Slater Street	25	-		0.5	
		50	-		0.5	
		100	-		0.5	
C6-SC-1						
	244	10	-		0.5	0.5
	@ Friesland	25	-		0.5	
	Drive	50	-		0.5	
		100	-		0.5	

(Continued)

Notes:
- \- Indicates no overflow from channel
- – Indicates no overflow into channel
- * Indicates relative levee height/channel depth (see Figure 3.4)

Table 5.2.

GLOBAL MODEL, LOCATIONS OF CHANNEL/FLOODPLAIN INTERFACE
(Continued)

Channel System	Element/ Location	n-Year Event	Channel Overflow Maximum Depth (feet)	Channel Overflow Levee Height*/ Channel Depth (feet)	Floodplain Overflow Maximum Depth (feet)	Floodplain Overflow Levee Height (feet)
C6-SC-1						
	245	10	-		0.5	
	@ Warner	25	-		0.5	
	Avenue	50	-		0.5	
		100	-		0.5	
C05SC-1						
	287	10	-		0.5	0.5
	@ Northeast	25	-		0.5	
	of 405 Freeway	50	-		0.5	
	/Edinger	100	-		0.5	
Lewis Channel						
	702	10	-		0.5	0.5
	@ C05	25	-		0.5	
		50	-		0.5	
		100	-		0.5	

Notes:
- \- Indicates no overflow from channel
- – Indicates no overflow into channel
- * Indicates relative levee height/channel depth (see Figure 3.4)

V.2. DETAILED MODEL RESULTS (EXISTING CHANNEL SYSTEM)

Figures 5.5 through 5.8 show the estimated maximum flooding depths for the 10-, 25-, 50-, and 100-year events within the Detailed modeling area. Appendix E contains the DHM summary outputs for the Detailed model. Table 5.3 summarizes the estimated flooding areas where flood depths are estimated to be greater than 0.5 feet. The Detailed model results indicate no overtopping for both West Street and Haster Basins (see Table 5.3). Emergency spillway flows occur at the Haster Basin for all n-year events. These only imply that floodwater delivered to the basins will not overtop the basins but does not imply that both basins have adequate storage for all n-year events. There is more local flooding estimated by the Detailed model because more local depressions were identified by using the street intersection spot elevation information provided by the County of Orange, and the topographic data from the City of Garden Grove.

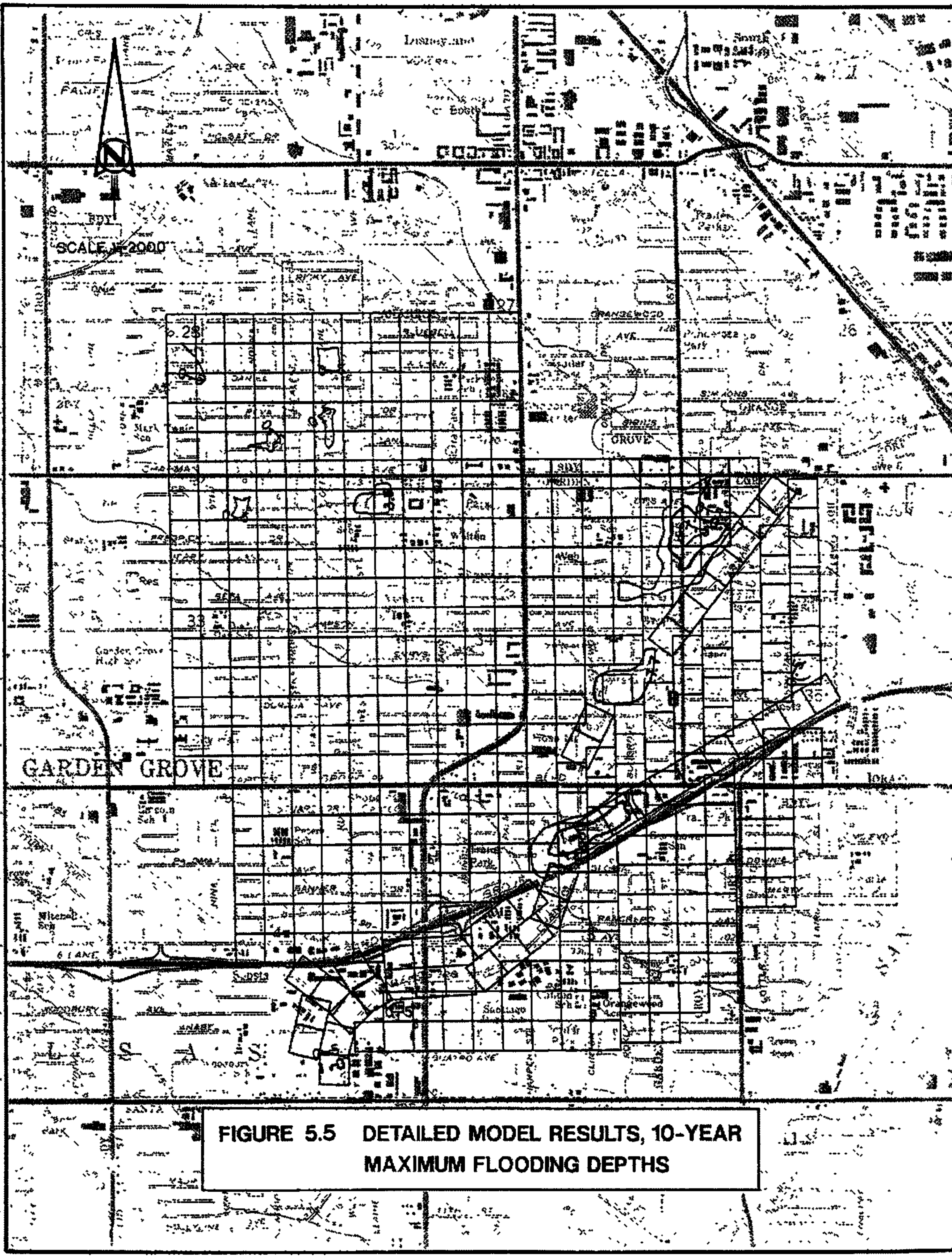

FIGURE 5.5 DETAILED MODEL RESULTS, 10-YEAR MAXIMUM FLOODING DEPTHS

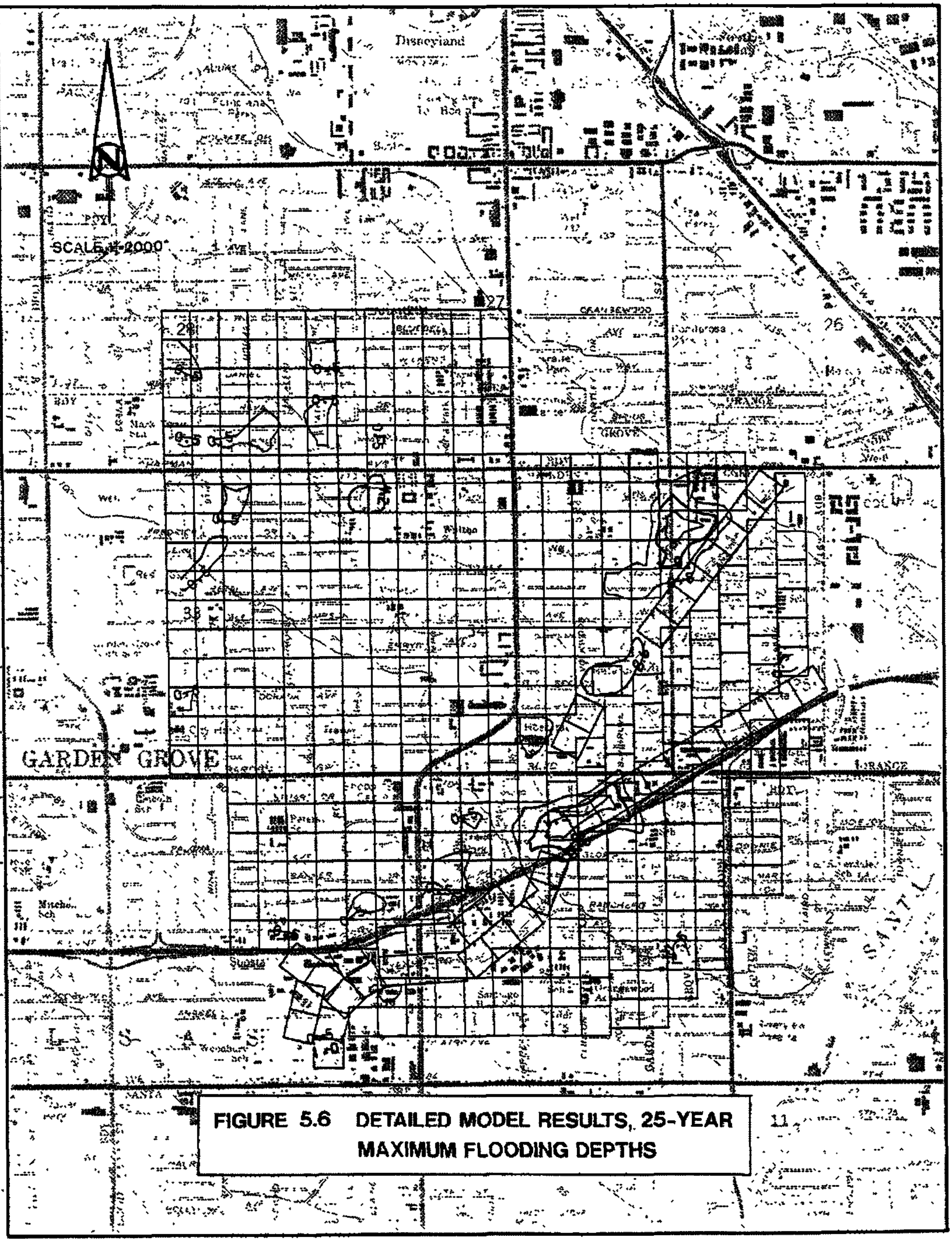

FIGURE 5.6 DETAILED MODEL RESULTS, 25-YEAR MAXIMUM FLOODING DEPTHS

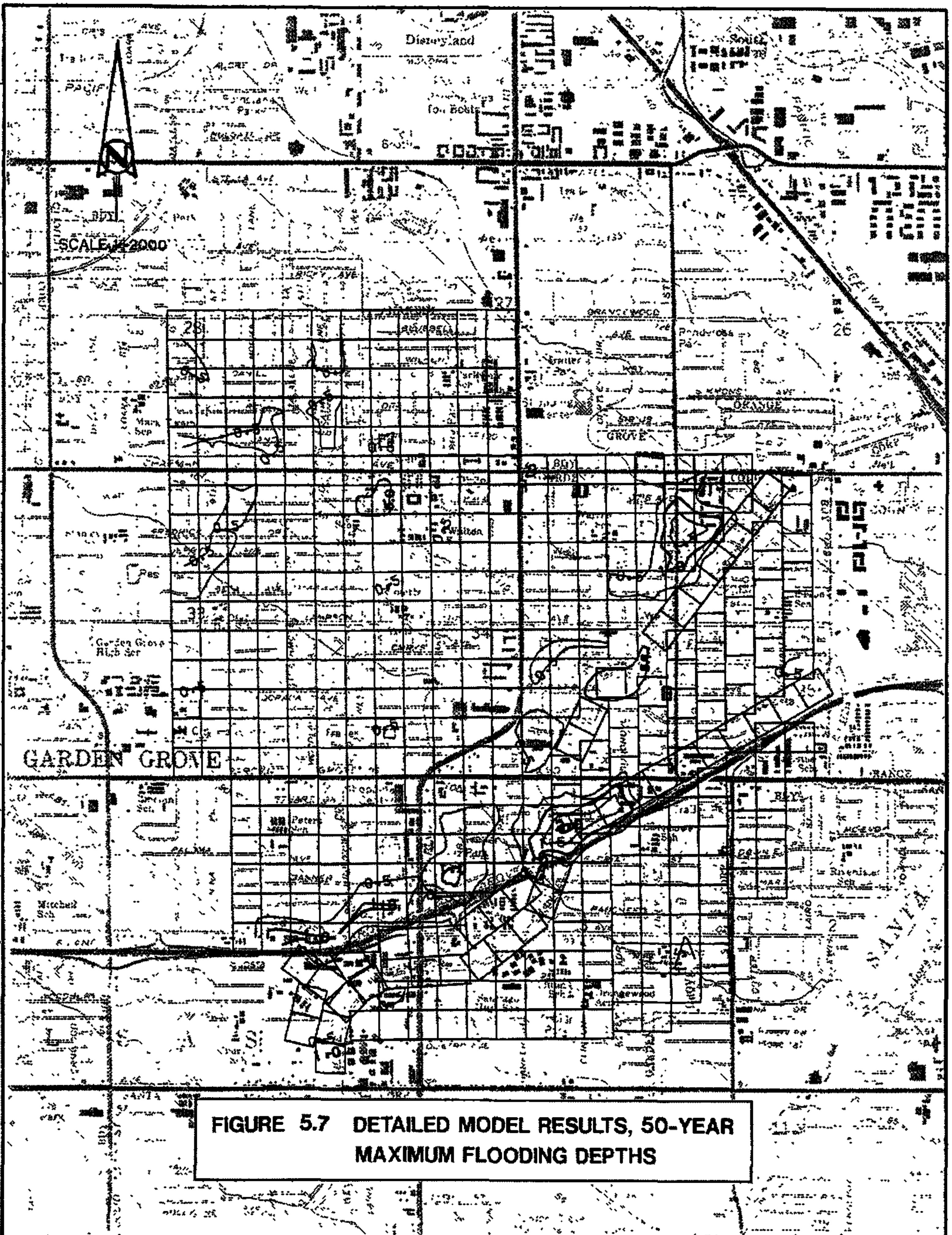

FIGURE 5.7 DETAILED MODEL RESULTS, 50-YEAR MAXIMUM FLOODING DEPTHS

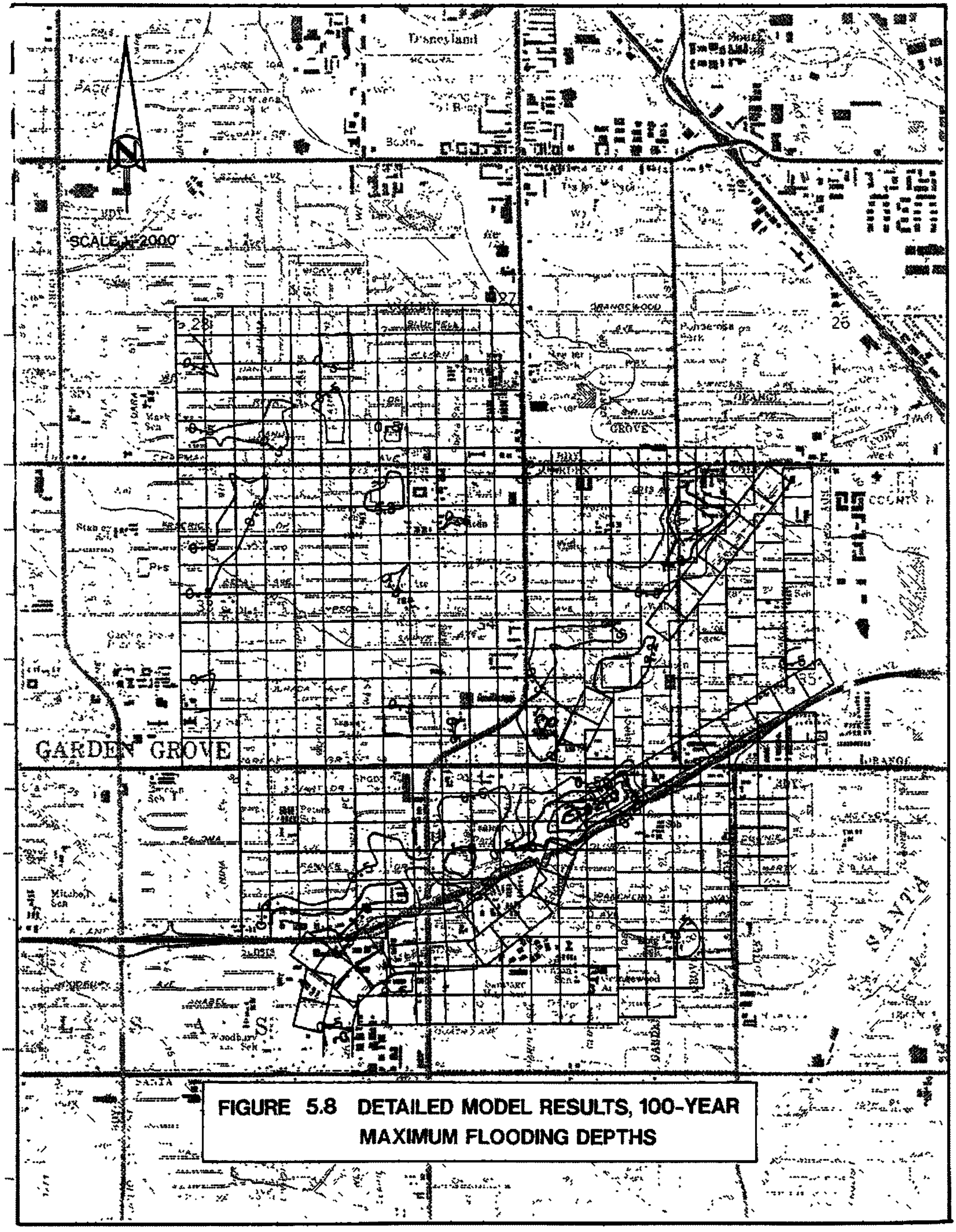

FIGURE 5.8 DETAILED MODEL RESULTS, 100-YEAR MAXIMUM FLOODING DEPTHS

Table 5.3.

SUMMARY OF DETAILED MODEL RESULTS

Location	n-year event	Maximum Depth (ft)	Averaged Depth (ft)	Floodplain Elements
City of Garden Grove				
9th Street/Waverly	10	1.0	1.0	13
	25	1.0	1.0	13
	50	1.0	1.0	13
	100	1.0	1.0	13
Daniel/Morgan	10	0.7	0.7	18
	25	0.7	0.7	18
	50	0.8	0.8	18
	100	0.8	0.8	18
West Street/Reva	10	0.6	0.6	42,54
	25	0.7	0.6	42,54
	50	0.7	0.7	42,54
	100	0.7	0.7	42,54
Candy/Chapman	10	0.6	0.6	52
	25	0.6	0.6	40,49,50,51,52
	50	0.7	0.6	40,49,50,51,52
	100	0.7	0.6	40,49,50,51,52
Candy/Debbie	10	–	–	–
	25	0.6	0.6	56
	50	0.6	0.6	56
	100	0.7	0.7	56
East of Harbor/ Chapman	10	–	–	–
	25	–	–	–
	50	0.6	0.6	73
	100	0.7	0.7	73
Chapman/Via Aloha	10	–	–	–
	25	–	–	–
	50	–	–	–
	100	0.5	0.5	76
Norma/John	10	0.6	0.6	83
	25	0.7	0.7	83
	50	0.7	0.7	83
	100	0.8	0.8	83
West Street Basin	10	3.5	3.5	87
	25	4.2	4.2	87
	50	5.0	5.0	87
	100	5.3	5.3	87

(Continued)

Table 5.3.

SUMMARY OF DETAILED MODEL RESULTS
(Continued)

Location	n-year event	Maximum Depth (ft)	Averaged Depth (ft)	Floodplain Elements
City of Garden Grove (cont'd)				
Southeast of Haster/ Chapman	10	0.9	0.9	98
	25	1.0	1.0	98
	50	1.0	1.0	98
	100	1.2	1.2	98
9th Street/Frederick	10	–	–	–
	25	–	–	–
	50	–	–	–
	100	0.6	0.5	102,103
Walton School	10	–	–	–
	25	–	–	–
	50	0.5	0.5	110
	100	0.6	0.6	110
Haster/Allard	10	1.8	1.5	118,119,137
	25	2.0	1.6	118,119,137
	50	2.0	1.6	118,119,137
	100	2.1	1.3	118,119,137,299
Jerry/Gamma/9th Street	10	0.5	0.5	121
	25	0.6	0.6	121
	50	0.6	0.6	121
	100	0.6	0.6	120,121
West of 9th Street/Beta	10	–	–	–
	25	0.6	0.6	138
	50	0.8	0.8	138
	100	1.0	1.0	138
North of Violet School	10	–	–	–
	25	–	–	–
	50	0.5	0.5	145
	100	0.6	0.6	145
Twintree/Haster	10	0.7	0.6	153,154
	25	0.8	0.7	153,154
	50	0.8	0.7	153,154
	100	0.8	0.8	153,154

(Continued)

Table 5.3.

SUMMARY OF DETAILED MODEL RESULTS
(Continued)

Location	n-year event	Maximum Depth (ft)	Averaged Depth (ft)	Floodplain Elements
City of Garden Grove (cont'd)				
Volkwood/Laux	10	–	–	–
	25	–	–	–
	50	0.6	0.6	184,185
	100	0.7	0.7	183,184,185
Emrys/Buaro	10	–	–	–
	25	–	–	–
	50	–	–	–
	100	0.5	0.5	195
Blue Spruce/ Choisser	10	–	–	–
	25	–	–	–
	50	0.6	0.6	199
	100	0.7	0.6	199,213
Sanford/9th Street	10	–	–	–
	25	0.7	0.7	201
	50	0.9	0.9	201
	100	1.0	1.0	201
Dunklee/Dungan	10	–	–	–
	25	–	–	–
	50	0.6	0.6	222
	100	0.9	0.9	222
East of Dunklee/ Buaro	10	–	–	–
	25	–	–	–
	50	–	–	–
	100	0.5	0.5	224
Palm Harbor Hospital	10	–	–	–
	25	0.8	0.8	227
	50	1.0	0.9	227,240
	100	1.1	0.9	227,240
Rainbow/Quartz/ Marble	10	–	–	–
	25	–	–	–
	50	0.8	0.7	259,260,270,271
	100	1.0	0.8	259,260,270,271

(Continued)

Table 5.3.

SUMMARY OF DETAILED MODEL RESULTS
(Continued)

Location	n-year event	Maximum Depth (ft)	Averaged Depth (ft)	Floodplain Elements
City of Garden Grove (cont'd)				
Upstream of C05/ 22-Freeway	10	1.6	1.3	273,307,350,351
	25	2.0	1.3	262,273,307,350,351
	50	2.2	1.5	262,273,306,307, 350,351
	100	2.3	1.4	262,273,306,307, 339,350,351
Upstream of Harbor/ 22-Freeway	10	–	–	–
	25	0.9	0.8	281,287
	50	1.2	1.0	281,282,287,288
	100	1.4	1.0	279,280,281,282, 287,288
Trask/22-Freeway	10	–	–	–
	25	0.6	0.6	291
	50	1.4	1.1	291,292,293
	100	1.7	1.5	291,292,293
Dapplegrey/Percheron	10	–	–	–
	25	–	–	–
	50	–	–	–
	100	0.5	0.5	267
Haster Basin	10	7.7	7.7	302
	25	8.6	8.6	302
	50	8.9	8.9	302
	100	9.2	9.2	302
Spinnaker/Heather	10	0.7	0.7	314
	25	0.7	0.7	314
	50	0.7	0.7	314
	100	0.7	0.7	314
Last Channel Element	10	1.2	1.2	363
	25	1.2	1.2	363
	50	1.2	1.2	363
	100	1.2	1.2	363
Trask/Robyn	10	0.5	0.5	401
	25	0.7	0.7	401
	50	0.7	0.7	401
	100	0.8	0.8	401

(Continued)

Table 5.3.

SUMMARY OF DETAILED MODEL RESULTS
(Continued)

Location	n-year event	Maximum Depth (ft)	Averaged Depth (ft)	Floodplain Elements
City of Garden Grove (cont'd)				
Harbor/Woodbury	10	0.6	0.6	443
	25	0.7	0.7	443
	50	0.8	0.8	443
	100	0.8	0.8	443

Table 5.4 lists the locations where channel and floodplain interface occurred in the Detailed modeling area. From Table 5.4, the neighboring area of the C05 channel and the Garden Grove Freeway was flooded for all the n-year events. Floodwater enters Lewis storm channel at the intersections of Dunklee Avenue and Spinnaker Street and at Garden Grove Boulevard and Haster Street for all the n-year events.

V.3. COMPARISON OF GLOBAL MODEL AND DETAILED MODEL RESULTS

The Detailed model shows more local flooding area in the City of Garden Grove than the Global model had estimated and different estimated depths at the West Street and Haster Basins (see Tables 5.1 and 5.3). The discrepancies are primarily due to the following: (1) more detailed topographic data used in the Detailed model, (2) difference in element size used, and (3) different boundary conditions used.

The Global model is based on U.S.G.S. 7.5 minute quadrangle maps for definition of representative ground elevations for each floodplain element. The street intersection spot elevation information provided by the County of Orange and 1" = 100' scale topographic data from the City of Garden Grove were used to define the representative ground elevations for the Detailed model. More local depressions were identified by this additional data.

Table 5.4.

DETAILED MODEL, LOCATIONS OF CHANNEL/FLOODPLAIN INTERFACE

Channel System	Element/ Location	n-Year Event	Channel Overflow Maximum Depth (feet)	Channel Overflow Levee Height*/ Channel Depth (feet)	Floodplain Overflow Maximum Depth (feet)	Floodplain Overflow Levee Height (feet)
C05						
	306	10	8.3	8.0	–	0.5
	South of	25	8.7		1.2	
	Garden Grove	50	8.9		1.4	
	Boulevard	100	9.0		1.5	
	307	10	8.2	7.5	1.7	1.0
	Upstream of	25	8.6		2.1	
	22-Freeway	50	8.8		2.3	
		100	9.0		2.5	
	363	10	-	-	1.3	1.3
	Last Channel	25	-		1.3	
	Element	50	-		1.3	
		100	-		1.3	
Lewis Channel						
	344	10	-	-	0.5	0.5
		25	-		0.5	
		50	-		0.5	
		100	-		0.5	
	345	10	-	-	0.5	0.5
		25	-		0.5	
		50	-		0.5	
		100	-		0.5	
	348	10	-	-	0.5	0.5
		25	-		0.5	
		50	-		0.5	
		100	-		0.5	
	350	10	4.5	4.5	–	1.7
		25	4.7		1.9	
		50	4.9		2.1	
		100	5.0		2.2	
	351	10	4.5	4.5	–	1.5
		25	4.8		1.8	
		50	5.0		2.0	
		100	5.1		2.1	

Notes:
- \- Indicates no overflow from channel
- – Indicates no overflow into channel
- * Indicates relative levee height/channel depth (see Figure 3.4)

A 1000 x 1000 foot element and a 500 x 500 foot element were used by the Global and Detailed models, respectively. The influenced area for the surcharged pipe element and the floodplain/channel element were reduced by a factor of 4 for the Detailed model compared to the Global model.

Because the Detailed modeling area lies inside the Global modeling area, the inflow boundary conditions for the Detailed model have to be extracted from the Global model results (see Appendix C). The no-flow and critical depth outflow boundary conditions assigned to the Detailed model boundary are in locations where the Global model had continuous floodplain elements.

Due to the above-mentioned reasons, the results from the Global model provided a general watershed response to the n-year events where the results from the Detailed model show an indepth analyses to a specific area within the watershed.

V.4. COMPARISON OF GLOBAL MODEL RESULTS TO C.O.E. FEASIBILITY STUDY RESULTS

A comparison of precipitation data used to generate n-year runoff is listed in Table 5.5 for the DHM Global model and the C.O.E. Feasibility study (1988). This study uses Orange County Hydrology Manual t-year precipitation depths to produce 50-percent confidence n-year runoff results, while the C.O.E. study used the appropriate t-year precipitation depths to produce expected probability n-year runoff results with possibly more localized rainfall data. Table 5.5 shows that the DHM uses a lower intensity rainfall for the peak 3-hour duration, but has a larger runoff volume for the entire 24-hour duration.

The potential flooded areas between the San Diego Freeway and the C05 channel outlet were estimated as: (1) area near Slater Pump Station, (2) C05 channel at downstream of 405 Freeway, (3) C06 channel at downstream of 405 Freeway, (4) area near the intersection of Edwards Street and Heil Avenue, and (5) area near intersection of Newland Street and Warner Avenue by both studies. In addition to the potential flooded areas described in the C.O.E. feasibility report, a map (see Figure 5.9) which contains all the potential flooded areas for the entire C05/C06 watershed was provided by the O.C.E.M.A. for further comparisons. From Figure 5.9, the potential flooded area above the 405 Freeway were estimated along both sides of the C05/C06 channel systems. The potential flooded areas were estimated as: (1) between Bolsa Avenue and the 405 Freeway for 25-year event and (2) between Trask Avenue and the 405 Freeway for 50- and 100-year events for the C05 channel system (see Figure 5.9). The DHM Global model estimated flooded areas between Brookhurst Street and the 405 Freeway for all the t-year events and some other isolated flooded areas as shown on Figures 5.4 and 5.8. The DHM Global model results in less flooded areas for 50- and 100-year events than Figure 5.9 indicated. For the C06 channel system, the potential flooded areas

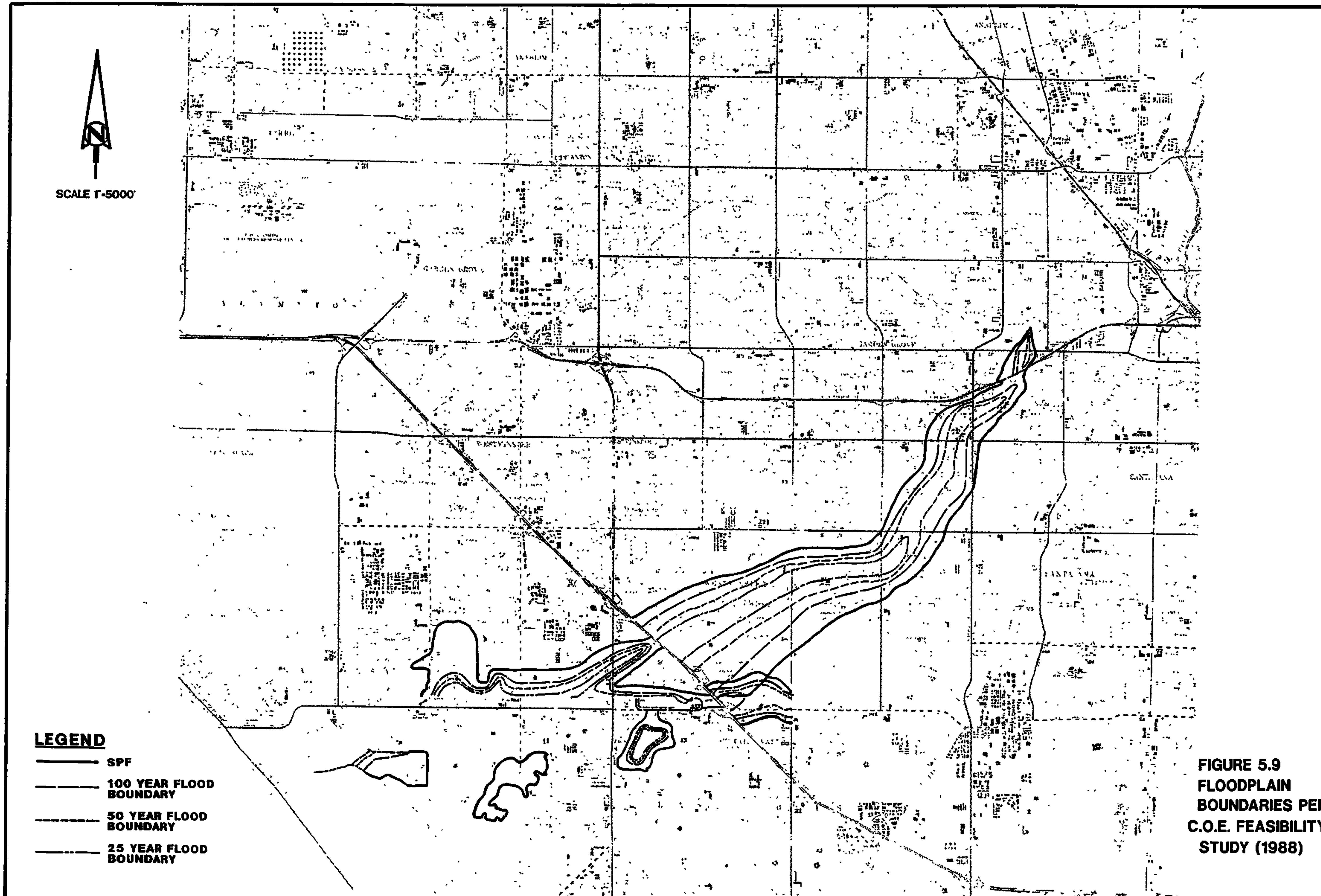

FIGURE 5.9
FLOODPLAIN
BOUNDARIES PER
C.O.E. FEASIBILITY
STUDY (1988)

were estimated from the 405 Freeway to Bushard Street for the 25-year event and to Brookhurst Street for the 50- and 100-year events of Figure 5.9. There were no flooded depths greater than 0.5 feet estimated by the DHM Global model in the above mentioned areas. Estimated flooded areas were also shown on Figure 5.9 for the areas between the Haster Basin and shown on Figure 5.9 for the areas between the Haster Basin and the Garden Grove Freeway for 50- and 100-year events. The same flooded areas were estimated by the DHM Global model for all the t-year events. Because of different methodologies employed by the two studies (see Section II.4), as expected, the resulting flooded depths and inundated areas were different. Nevertheless, the same potential areas of flooding were identified by both studies, however, the DHM Global model estimates a greater number of local flooded areas (see Table 5.1).

Table 5.5.

POINT PRECIPITATION DEPTHS (INCHES),
CURRENT STUDY VERSUS 1988 COE FEASIBILITY STUDY

	100-YEAR		50-YEAR		25-YEAR		10-YEAR	
DURATION	DHM	COE*	DHM	COE*	DHM	COE*	DHM	COE*
5-minute	0.4	0.53	0.37	0.46	0.34	0.40	0.26	0.33
30-minute	0.87	1.00	0.79	0.89	0.72	0.76	0.59	0.63
1-hour	1.15	1.31	1.05	1.16	0.95	0.99	0.78	0.82
3-hour	1.94	2.08	1.75	1.84	1.59	1.58	1.31	1.30
6-hour	2.71	2.74	2.43	2.42	2.20	2.08	1.81	1.72
24-hour	4.49	4.33	4.04	3.82	3.68	3.28	3.03	2.71

* Values obtained from Table 7 of C.O.E. Feasibility Study (1988).

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APPENDIX A

FEMA LETTER OF ACCEPTANCE

OF THE DHM MODEL

Federal Emergency Management Agency
Washington, D.C. 20472

SEP 28 1988

CERTIFIED MAIL
RETURN RECEIPT REQUESTED

Mr. Delbert Powers
City Manager, City of Garden Grove
11391 Acacia Parkway
Garden Grove, California 92642

Dear Mr. Powers:

This is in regard to a letter dated July 19, 1988, from Mr. Marshall E. Jennings, Hydrologist, U.S. Geological Survey (USGS) and a letter dated August 24, 1988, from Mr. Tory R. Walker, Assistant Engineer, Williamson and Schmid. Messrs. Jennings and Walker responded to our letter to you dated June 8, 1988, regarding the use of the Diffusion Hydrodynamic Model (DHM) to revise the 100-year floodplain delineation shown at the intersection of Haster Street and Garden Grove Avenue on the effective Flood Insurance Rate Map (FIRM) for the City of Garden Grove, California. We informed you that prior to reviewing the City's request to utilize the DHM, evidence was required that the requirements stated in Section 65.6(a)(6) of the National Flood Insurance Program (NFIP) had been met.

Mr. Jennings explained in his July 19 letter that the inclusion of the disclaimer statement on the cover and first page of USGS Water Resources Investigations Report 87-4137, submitted to us by Mr. Joseph S. Schenk, City Engineer, with a letter dated May 6, 1988, was in error. Mr. Jennings stated that the report had been reviewed within the USGS and approved for publication.

In addition, Mr. Walker stated in his August 24 letter that the DHM is available for general use without a proprietary fee and that a user's and programmer's manual are contained in the USGS Water Resources Investigations Report 87-4137. Based on the information provided by Messrs. Jennings and Walker, we agree that the requirements in Section 65.6(a)(6) of the NFIP regulations have been satisfied.

We have initiated our review of the City's request to revise the floodplain delineations based on the DHM and will notify you within 90 days of the date of this letter of our findings. Please note that, based on our findings, additional data may be required. We do not anticipate incorporating any changes that result from this request or any other revision request into the Flood Insurance Study (FIS) report and FIRM for Orange County, California and Incorporated Areas that we are now processing.

Making such changes would require that we delay the processing of the FIS report and FIRM while we complete our evaluations of the requests and would therefore further complicate the already complex process of providing a countywide FIRM for Orange County and its incorporated communities. Although the complexity of the countywide FIRM production process occasionally poses such problems, we believe that countywide FIRMs, because of their ease of use, are valuable tools for both FEMA and the mapped communities. Therefore, any changes warranted by ongoing or future revision requests will be incorporated into a subsequent revision of the FIS report and FIRM.

Should you have additional questions, please call Mr. William Judkins of my staff in Washington, D.C., at (202) 646-3458.

Sincerely,

John L. Matticks
Chief, Risk Studies Division
Federal Insurance Administration

cc: Mr. Joseph S. Schenk
City of Garden Grove

Ms. Patricia P. Importuna
City of Garden Grove

Mr. Stewart O. Miller
City of Garden Grove

Mr. Marshall E. Jennings
USGS

Mr. Tory R. Walker
Williamson & Schmid

APPENDIX C

TECHNICAL SUPPLEMENT

C.1. DHM GRID LAYOUT

The layout of the Global and Detailed DHM floodplain grid schematics were based on the following steps:

1. Align the C05/C06 channel systems to the centerline of each floodplain elements (see Figure C.1).

2. Align floodplain elements on the upstream-side of the freeway system, such that two sides of the element are parallel to the freeway embankment and the other two sides of the element are perpendicular to the freeway embankment.

3. Fill in the rest of the floodplain elements such that the orientation of the floodplain elements coincide with the orientation of the street systems.

The above steps warrant the orientation of the streamline and potential functions of the flood wave coinciding with the orientation of the floodplain element. Unfortunately, the freeway and channel systems may intercept local street systems at an angle between 0- and 90-degree. This will artificially create no-flow boundary conditions for some floodplain elements. It should be noted that floodplain elements can only be connected in north-south or east-west direction. Figure C.2 shows artificial no-flow boundary conditions for east and south boundaries of floodplain elements A and B, respectively. These artificial no-flow boundary conditions shall be considered when interpreting DHM floodplain results.

Global floodplain friction factor and effective area factor were determined by averaging values of randomly chosen floodplain elements. Table C.1 shows numbers of streets and total street lengths for randomly chosen floodplain elements. There are 76 streets that cross the boundaries of 14 randomly chosen floodplain elements. The averaged street number that cross each side of the floodplain boundary is 1.36 (=76/14/4). If the averaged street section is about 70 feet, then the averaged street width that intercepts each side of the floodplain boundary will be 95 feet. A value of 100 feet was used in the inundation study. The Manning friction factor for street is assumed to be 0.02. Therefore, the Global Manning's friction factor with a element size of 1000 x 1000 feet is 0.2 (0.02 x 1000/100) for all the developed area. It is further assumed that the surface runoff will be carried by the street section. The averaged street area on each floodplain element is estimated as 286,000 ft^2 (100 x 40,000/14). The effective area factor for the developed area is 0.286 (286,000/1000/1000), thus a rounded-up value of 0.3 was used as the effective area factor in the inundation study for residential and commercial areas. Storage elements were used to represent the retarding basins and Manning's factor of 0.05 was used for opening areas, such as Mile Square Park, Garden Grove Golf Course, etc.

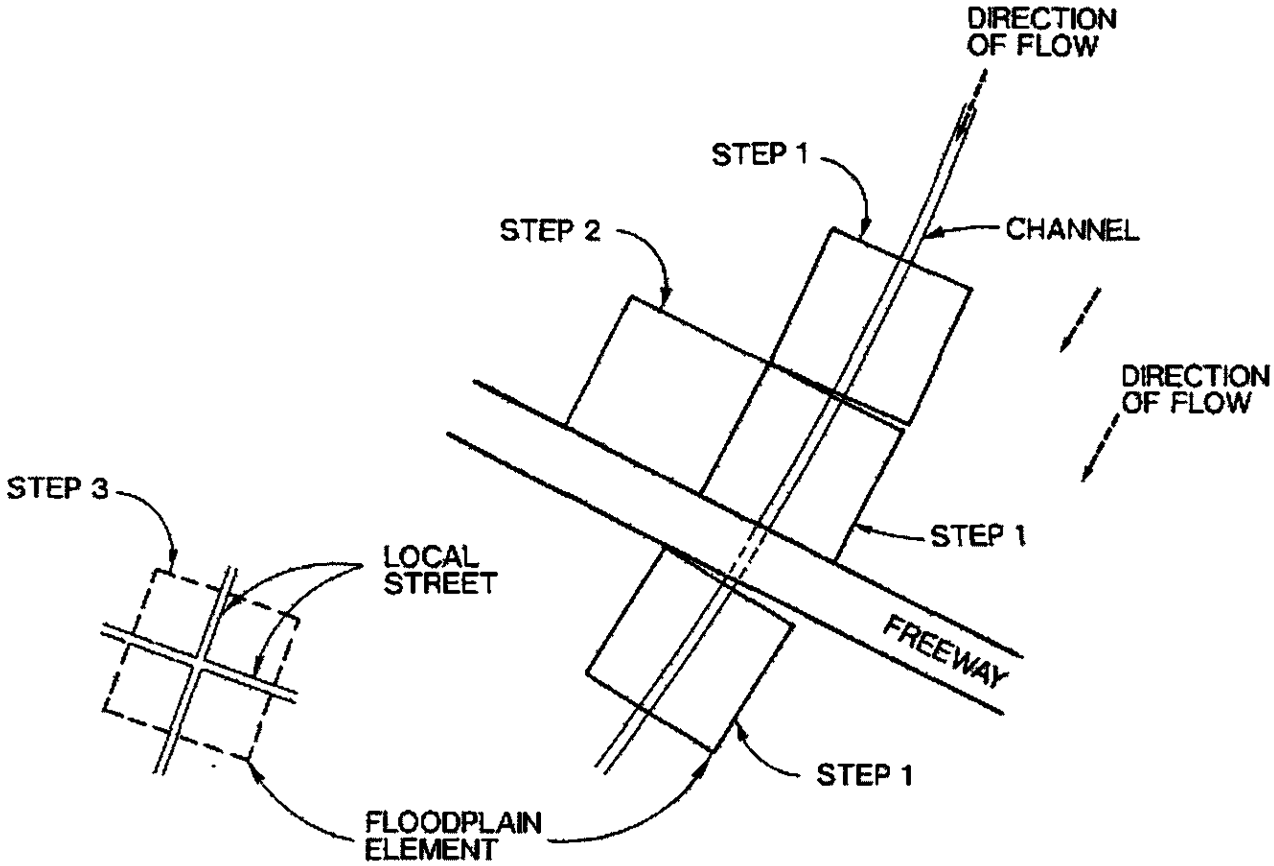

FIGURE C.1 GENERAL DHM GRID LAYOUT

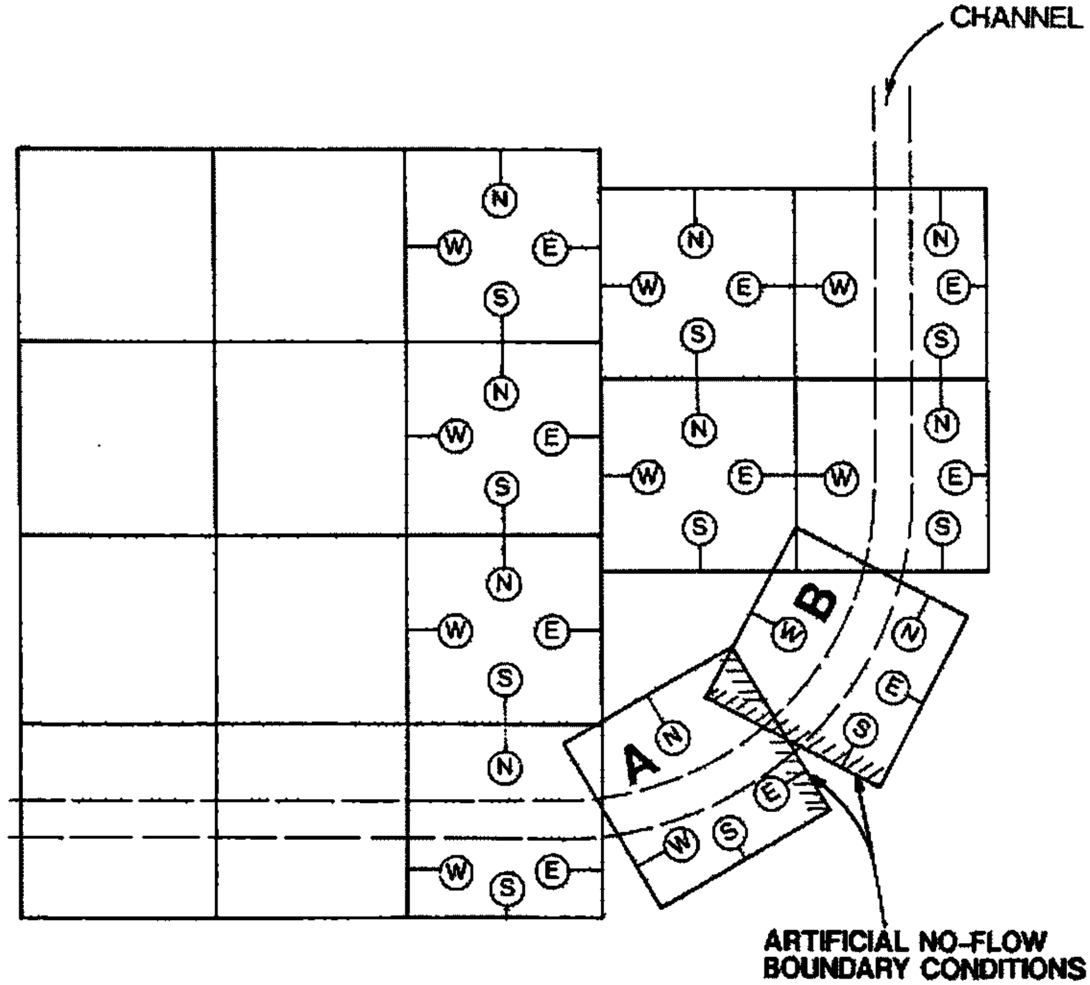

FIGURE C.2 ARTIFICIAL NO-FLOW BOUNDARY CONDITIONS

Table C.1

NUMBER OF STREET AND TOTAL STREET LENGTH FOR EACH RANDOMLY CHOSEN FLOODPLAIN ELEMENT

Floodplain Element	Number of Street Crossing Boundary North	East	South	West	Total Street Length (feet)
50	3	3	2	1	3,500
100	1	1	0	2	3,500
147	2	1	1	3	3,000
204	1	0	1	0	2,000
244	4	2	1	4	4,000
300	2	0	2	0	2,500
350	1	0	2	1	2,500
400	2	0	2	0	2,000
450	1	0	1	1	1,500
500	2	1	1	1	2,500
584	1	0	2	2	2,500
642	1	3	1	1	3,500
710	0	2	3	2	4,000
781	0	3	0	2	3,000
SUM	21	16	19	20	40,000

C.2. CHANNEL AND SURCHARGED PIPE ELEMENTS

Because the excess flow between channel and floodplain elements or the flow in a surcharged pipe element is uniformly re-distributed throughout the corresponding elements, it is advised that the channel and surcharged pipe elements be placed along the centerline of every floodplain element. This is more difficult to achieve for the surcharged pipe elements than for the channel elements because local storm drain systems are not evenly spaced in urban areas.

Channel sections are linearly interpolated between two connecting channel elements. All the channel elements and surcharged pipe elements should be linked in sequence such that all the tributary channel systems or surcharged pipe systems are connected to a confluence point before routing can proceed downstream.

C.3. STORAGE ELEMENTS

Table C.2 shows the depth versus discharge and storage relationships for all the storage facilities within the C05/C06 watershed. These data are obtained from the COE Feasibility Study (1988).

DHM uses a simple exponential formula, $Q = \alpha\ (\text{Depth})^{\beta}$, to represent all the depth versus discharge relationships, where α and β can be obtained from a least square fit analysis. Table C.3 shows α and β for Haster and West Street Retarding Basin.

C.3.1. Haster Retarding Basin

Figure C.3 depicts floodplain elements immediately downstream of the Haster Basin. Floodplain element 704 was designated as the storage element simulating the Haster Basin. It was assumed that all the overland flows will be collected by the local storm drain systems and conveyed into the basin. Thus, no-flow boundary conditions were used for the storage element.

Table C.3 shows two outflow relationships of the Haster Basin. The low flow outlet elevation and spillway elevation are 103.9 feet and 110.42 feet, respectively. The culvert outflow (low flow) relationship was assigned from storage element 704 to floodplain element 701 and the spillway outflow relationship was assigned from storage element 704 to floodplain element 703. A surcharged pipe element, which connected floodplain element 703 to channel element 701, served as the connection between Haster Basin and C05 channel system.

A culvert outflow relationship cannot be specified from the storage element to either surcharged pipe or a channel element due to limitations of the DHM. Thus, floodplain element 701 was designated to receive the culvert flow from the Haster Basin. The no-flow boundary conditions on three sides of the floodplain element 701 was to ensure that all culvert flows enter channel element 701. The open boundary between floodplain elements 703 and 701 simulates overland flow between these two elements. Both the Global and Detailed models used the same analogies to model the Haster Basin. The initial water surface elevation at the Haster Basin was 103.9 feet.

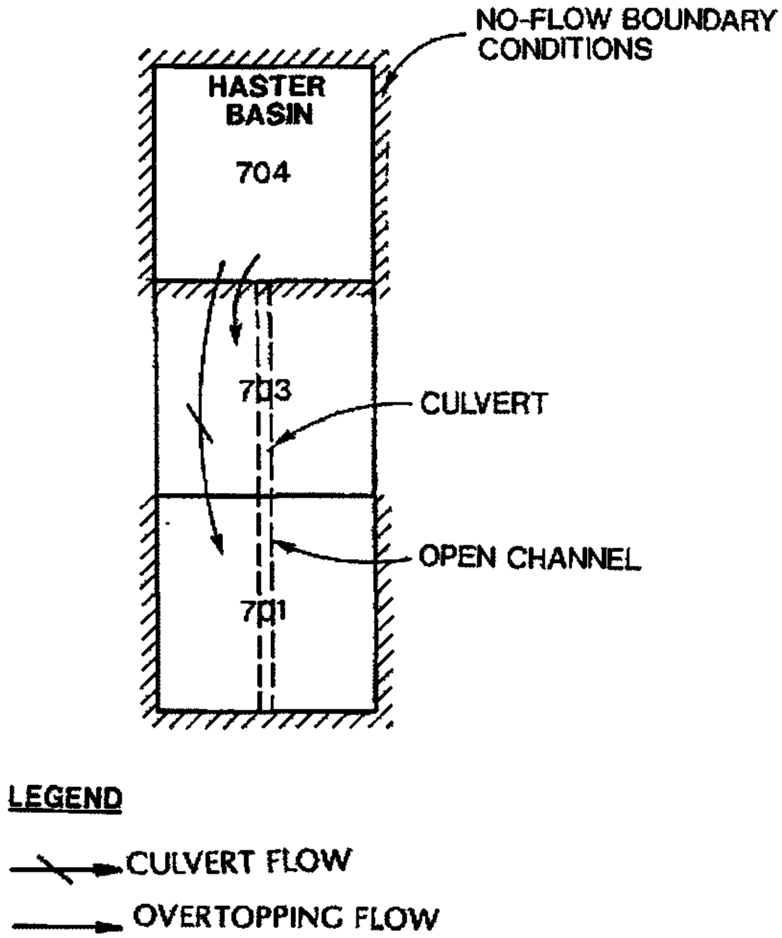

FIGURE C.3 FLOODPLAIN ELEMENTS DOWNSTREAM OF THE HASTER BASIN

Table C.2.

DEPTH-DISCHARGE-STORAGE
COE FEASIBILITY STUDY (1988)

Storage Facility	Elevation (ft)	Depth (ft)	Outflow Discharge (ft^3/sec)	Storage Volume (AF)	Remarks
Haster Basin	93	0		0	
	95	2		11.03	
	97	4		27.05	
	99	6		46.64	
	101	8		69.39	
	103	10	0	94.46	Low flow Elevations @103.9' * **
	105	12	32	121.17	
	107	14	150	149.64	
	109	16	350	179.86	
	111	18	525	211.54	Spillway Elevation @110.4'
	114	21	1135	260.97	
West Street Retarding Basin	105	0	0	0	*
	107	2	14	8	
	109	4	28	15	
	112	7	42	25	Overflow Elevations
Talbert Lake	-2.2	0	0	0	*
	-0.9	1.3	25	22	
	0.0	2.2	41	37	Overflow Elevation
Huntington Lake	-4.6	0	0	0	*
	-2.1	2.5	23	30	
	-1.6	3.0	24	36	
	-0.6	4.0	26	48	
	0.0	4.6	29	55	Overflow Elevation
Sand and Gravel Pit	-1	0	0	0	*
	4	5	48	30	
	9	10	65	60	
	19	20	92	120	
	29	30	115	177	Overflow Elevation

* Initial W.S. Elevation as per OCEMA.

** The DHM study uses 103.9' as initial depth i.e. @ 103.9 depth = 0'.

Table C.3.

COEFFICIENTS FOR DEPTH VERSUS DISCHARGE RELATIONSHIPS

Storage Facility	Elevation (ft)	Depth (ft)	Outflow** Discharge (ft^3/sec)	α	β	Remarks
Haster Basin (culvert outflow)	103.9	0	0			Low flow Elevation @ 103.9'
	105	1.1	32	29.1	1	
	106	2.1	80			
	107	3.1	150			
	108	4.1	250			
	109	5.1	350	26.3	1.57	
	110	6.1	425			
	111	7.1	525			
	112	8.1	625			
	113	9.1	725			
	114	10.1	825	39.6	1.31	
Haster Basin (spillway outflow)	103.9	0	0			
	110.42	6.52	0	0	1	Spillway Elevation @ 110.42'
	111	7.1	25	3.52	1	
	112	8.1	75			
	113	9.1	175			
	114	10.1	310	2.04×10^{-5}	7.19	
West Street Basin	105	0	0			
	107	2	14			
	109	4	28			
	112	7	42	0	1	Low flow ignored
	113	8	1956*	5.43×10^{-12}	16.12	Top of Berm @ 112'
	114	9	5532*	1.66x10-11	8.84	

* Overtopping flow is determined from the broad-crested weir formula.

** Outflow Discharge = $\alpha(\text{Depth})^{\beta}$

C.3.2. West Street Retarding Basin

Figure C.4 depicts the floodplain elements adjacent to the West Street Basin. Floodplain element 656 was designated as the storage element to emulate the West Street Basin. No overland flows were assumed to enter the Basin, i.e., no-flow boundary conditions specified. The low flow relationship from Table C.3 was ignored because a surcharged pipe element connected Storage element 656 to Floodplain element 655. The flow overtopping the berm was calculated by the broad-crested weir formula,

$$Q = 3.08 * L * (D-7)^{1.5}$$

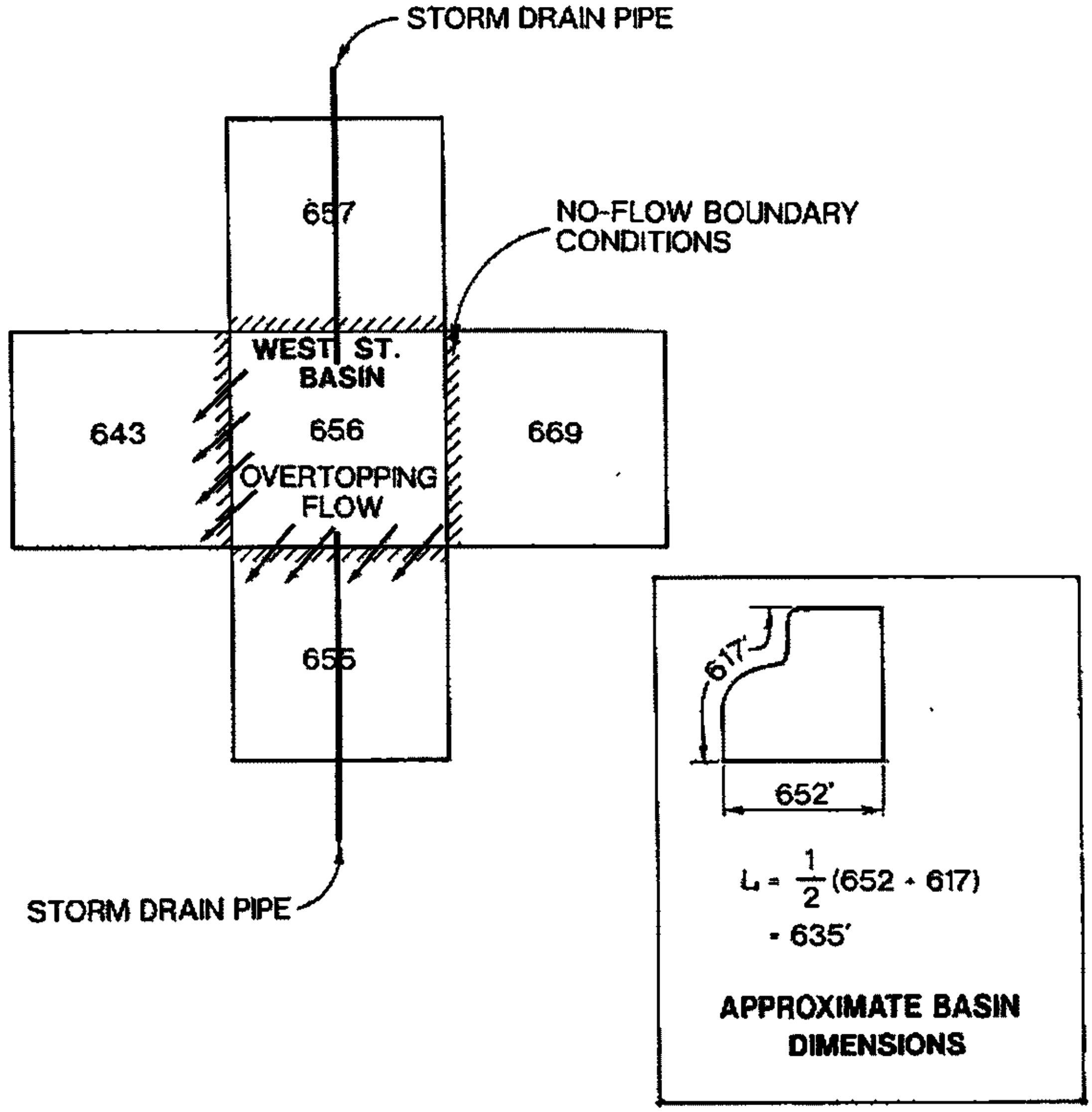

FIGURE C.4 FLOODPLAIN ELEMENTS ADJACENT TO WEST STREET BASIN

where L is the averaged length of the berm and D is the depth of water at the basin. From the construction plan of the West Street Basin, the averaged length of the Basin on each side is about 635 feet. Thus, the overflow rate is 1956 cfs when the water depth at the Basin equals to 8 feet (see Table C.3). The outflow relationship was assigned from Storage element 656 to Floodplain elements 643 and 655 to simulate the overtopping flow from the basin. Same modeling analogies were applied to both Global and Detail model. The initial water surface elevation at the West Street Basin was 105 feet.

C.3.3. Huntington Lake

Floodplain elements adjacent to the Huntington Lake are shown on Figure C.5. Floodplain element 103 was assigned to Huntington Lake. The no-flow boundary conditions were assigned to boundaries, where drastic changes in floodplain representative elevations occurred, to avoid using small time step (i.e., time step less than 1 second) in time domain approximation. The initial water surface elevation at the Huntington Lake was 0 feet i.e., mean sea level.

C.3.4. Talbert Lake

Floodplain element 145 was assigned to Talbert Lake as shown on Figure C.6. No-flow boundary conditions were assigned to two boundaries where large drop in floodplain elevations occurred. The initial water surface elevation was 0 feet i.e., mean sea level.

C.3.5. Sand and Gravel Pit

Figure C.7 depicts the floodplain elements that connect to the Sand and Gravel Pit. No-flow boundary conditions were assigned to all four boundaries where the Sand and Gravel Pit is much lower than the adjacent ground elevations. The initial water surface elevation at the Sand and Gravel Pit was 29 feet.

C.4. PUMP STATIONS

There are five pump stations located at the downstream end of the C05/C06 watershed. Slater Pump Station which conveys storm water from the Slater channel system into C05 channel system, has a maximum capacity of 750 cfs. Shields and Marilyn Pump Stations convey storm water flow from local storm drain systems into C05 channel system with a maximum capacity of 210 cfs and 120 cfs, respectively. Heil and Sandalwood Pump Stations convey storm water into local storm drain systems with a maximum capacities of 100 cfs and 95 cfs respectively.

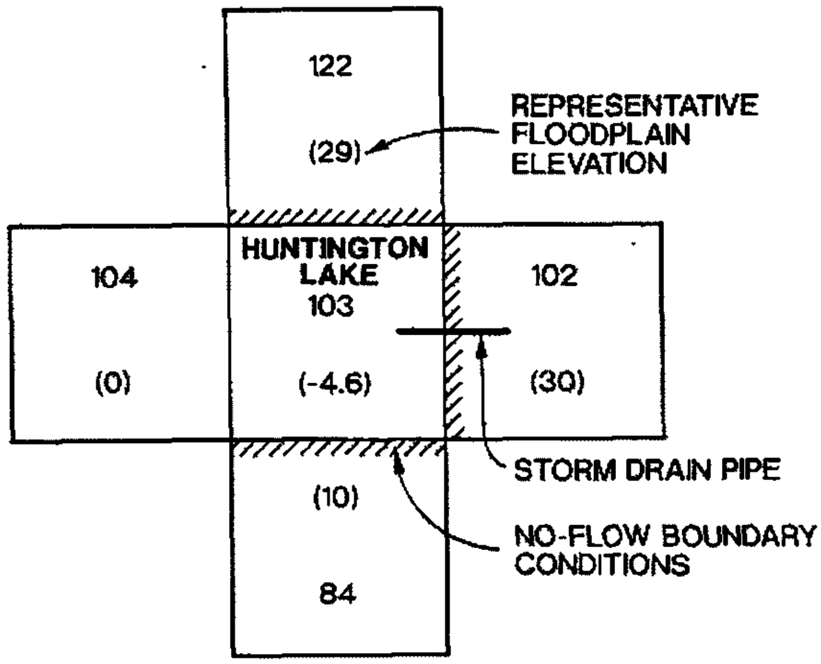

FIGURE C.5 FLOODPLAIN ELEMENTS ADJACENT TO HUNTINGTON LAKE

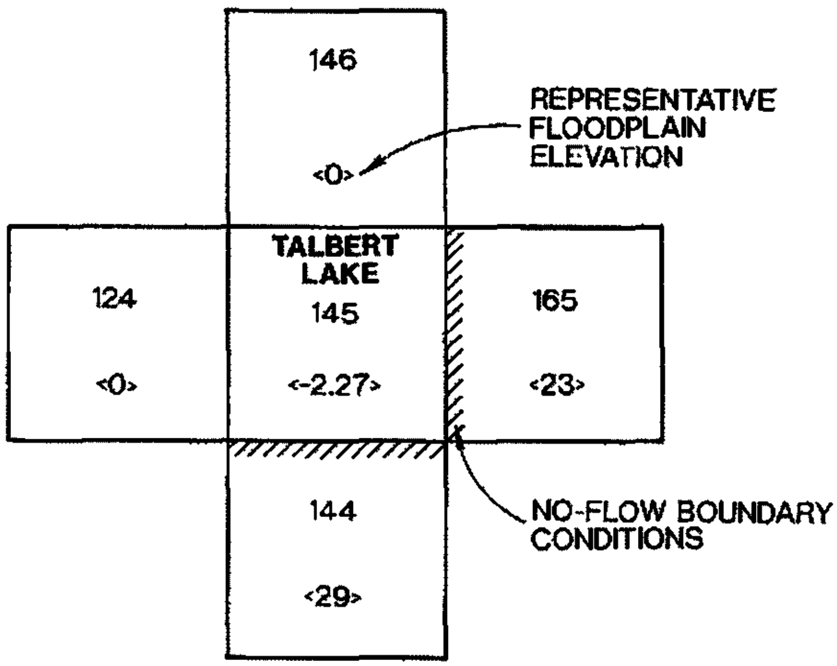

FIGURE C.6 FLOODPLAIN ELEMENTS ADJACENT TO TALBERT LAKE

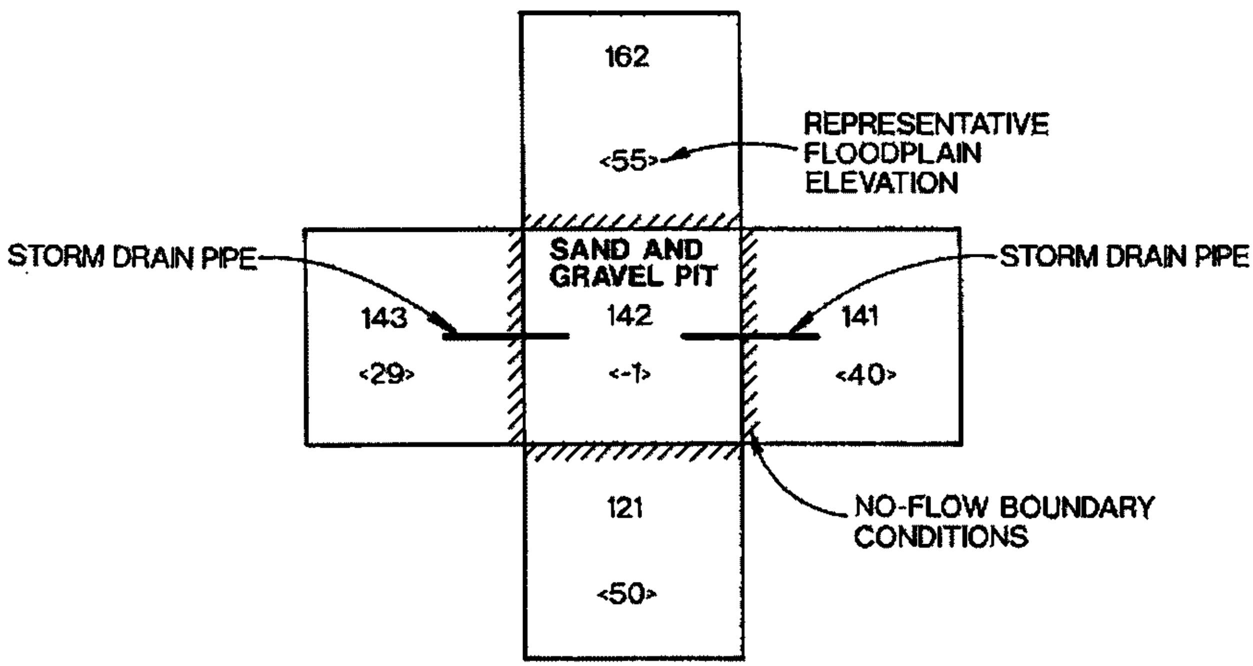

FIGURE C.7 FLOODPLAIN ELEMENTS ADJACENT TO SAND AND GRAVEL PIT

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Table C.4 lists the rating curve information for each of the pump stations. Operation information of the pump stations were obtained from the report, Storm Drain Pump Station Analysis, by L.D. King Engineering, 1979 and the City of Fountain Valley. In Table C.4, a constant discharge was used for most of the pump stations by setting β = 0. A linear outflow relationship is approximated from depth of 2.7 feet to depth of 6.0 feet for the Slater Pump Station.

Table C.4.

RATING CURVES FOR PUMP STATIONS

Pump Station	Depth (ft)	Discharge (ft^3/sec)	α	β
Slater[1]	0	0		
	2.7	15	15	0
	6.0	750	0.128	4.84
	20.0	750	750	0
Shields[2]	0	0		
	1	210	210	0
	2	210	210	0
	10	210	210	0
Marilyn[2]	0	0		
	1	120	120	0
	2	120	120	0
	10	120	120	0
Heil[1]	0	0		
	5	102	102	0
	10	102	102	0
	20	102	102	0
Sandalwood[1]	0	0	0	0
	1	95	95	0
	2	95	95	0
	10	95	95	0

Note: 1 Indicates channel rating curves.
2 Indicates floodplain rating curves.

C.5. EFFECTIVE RAINFALL MASS CURVE

Part of the total rainfall is converted into immediate runoff, usually referred to as the direct runoff, and the remainder of the rainfall is assumed to be losses. Watershed losses which consist of infiltration, depression storage, vegetation and man-made interception, and minor amount of evaporation are not incorporated in the DHM. The effective procedures of obtaining the rainfall is based on the OCEMA Hydrology Manual (1986). The relationships between the accumulated effective rainfall depths (inches) and the time was used by the DHM rainfall model. The rainfall can be assigned either to the entire study area or partial area as desired.

The 50-percent confidence level point rainfall data for the different runoff recurrence intervals are contained in Table C.5. The 50-year 50-percent confidence level point rainfall values are linearly interpolated between the 10- and 25-year rainfall values on the return frequency plot as shown on Figure C.8.

Table C.5.

50% CONFIDENCE LEVEL
POINT RAINFALL DATA (INCHES)

Runoff Recurrence Interval	Frequency of Rainfall to Produce 50% Confidence Level Runoff	5M	30M	1H	3H	6H	24H
100	25	0.4	0.87	1.15	1.94	2.71	4.49
50	15	0.37	0.79	1.05	1.75	2.43	4.04
25	10	0.34	0.72	0.95	1.59	2.20	3.68
10	5	0.26	0.59	0.78	1.31	1.81	3.03

Table C.6 shows the loss rate parameters used for each runoff recurrence interval with six storm centerings. The soil and development conditions were based on the Hydrology Report for East Garden Grove-Wintersburg Channel by OCEMA (1991). The accumulated effective rainfall depths were depicted on Figures C.9 through C.14.

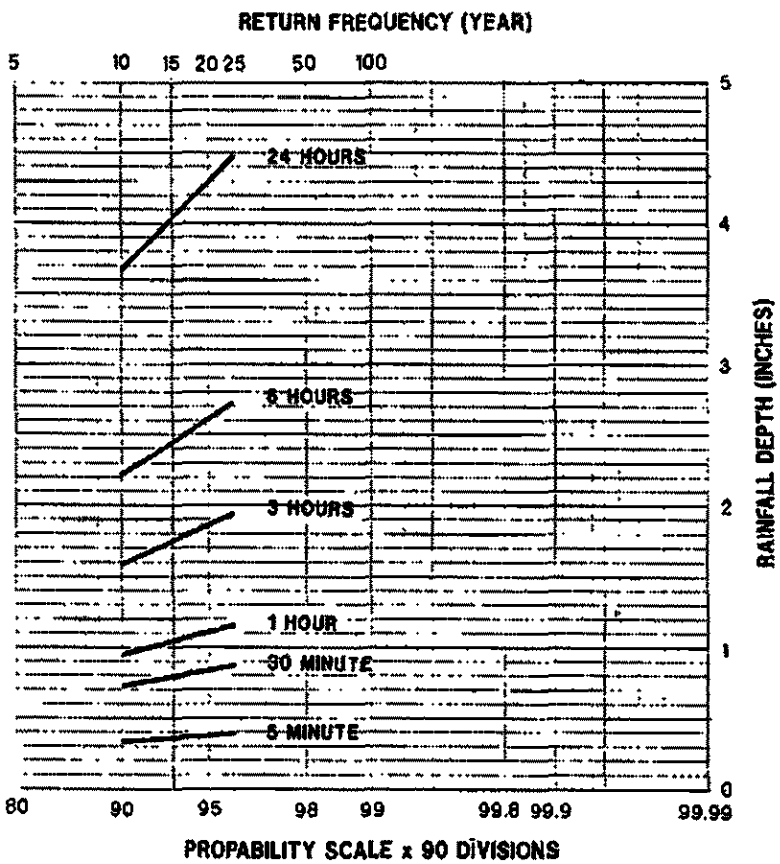

FIGURE C.8 DETERMINATION OF 15-YEAR POINT RAINFALL VALUES

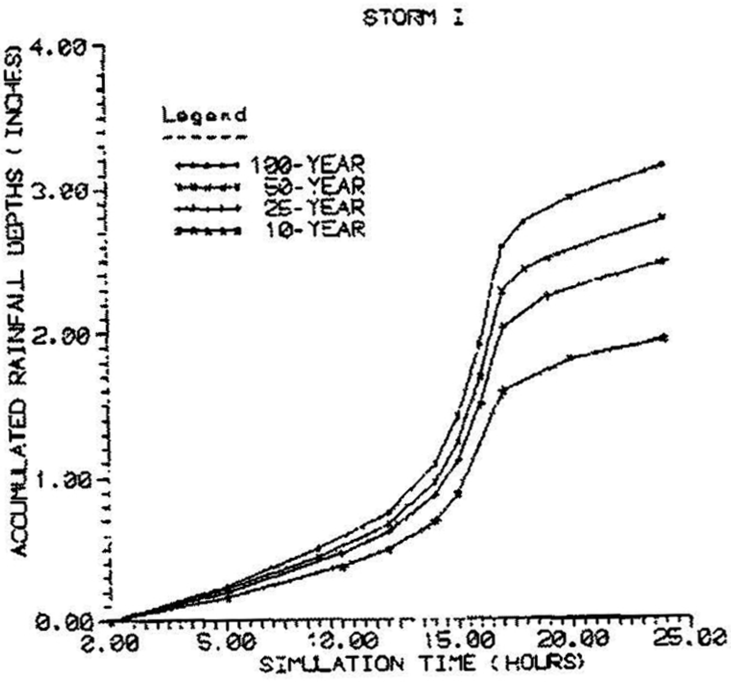

Figure C.9. Accumulated Effective Rainfall for Storm I.

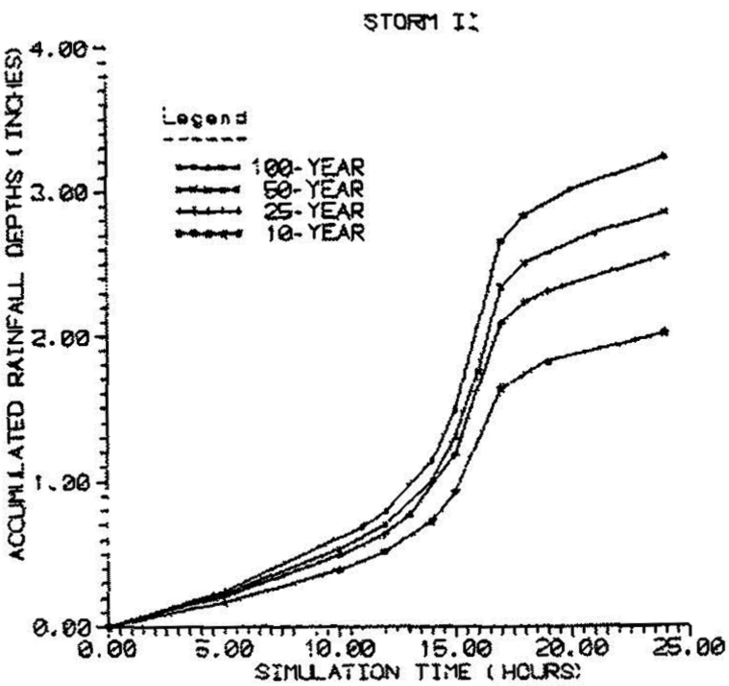

Figure C.10. Accumulated Effective Rainfall for Storm II.

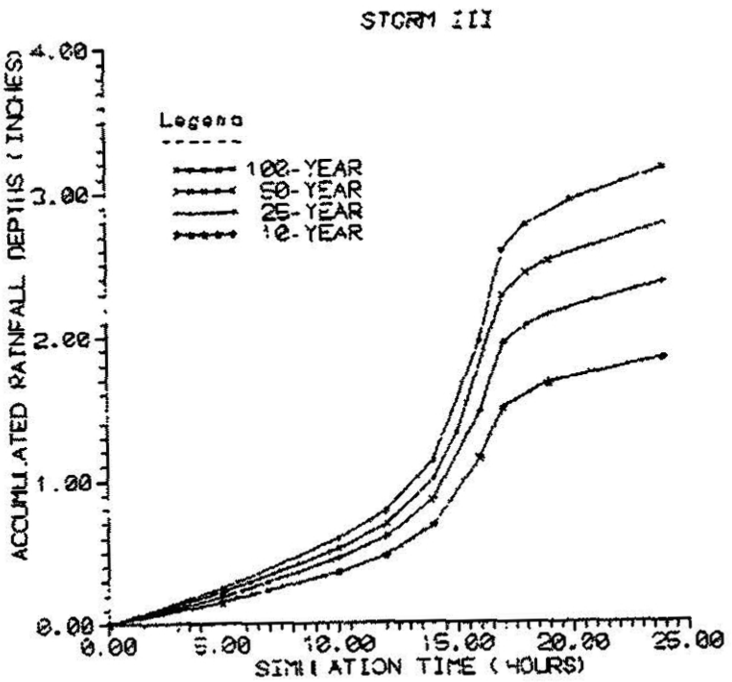

Figure C.11. Accumulated Effective Rainfall for Storm III.

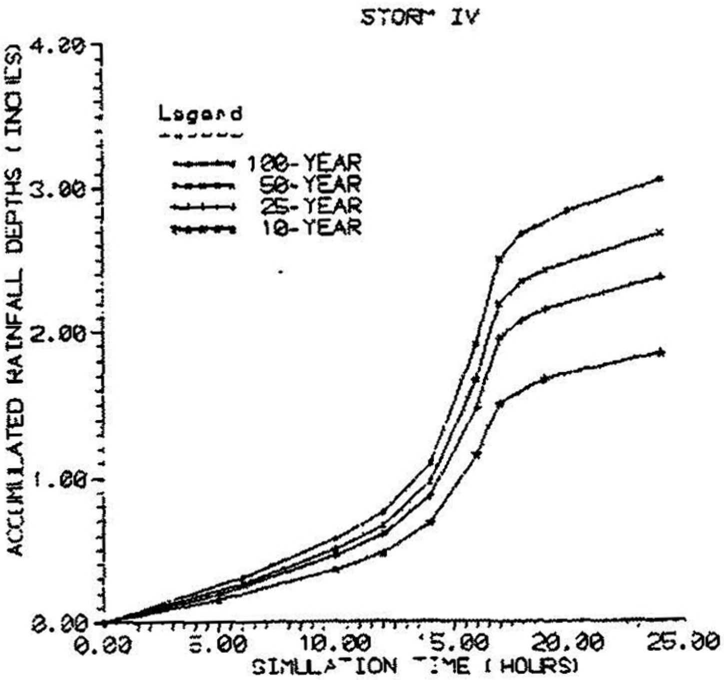

Figure C.12. Accumulated Effective Rainfall for Storm IV.

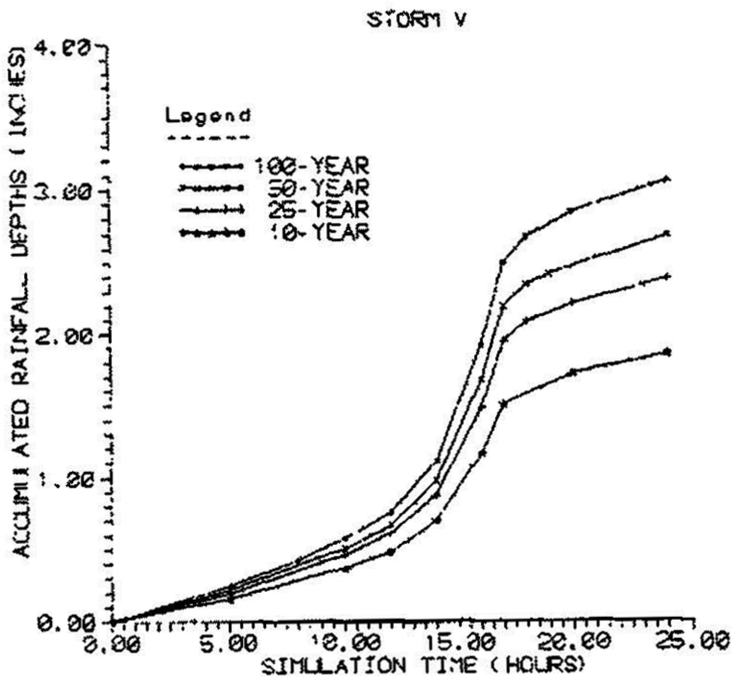

Figure C.13. Accumulated Effective Rainfall for Storm V.

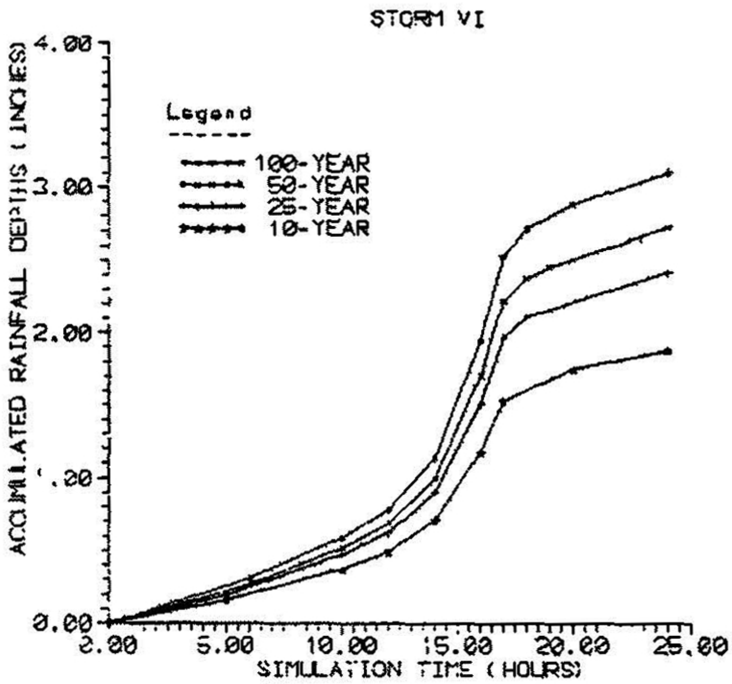

Figure C.14. Accumulated Effective Rainfall for Storm VI.

Table C.6.
LOSS RATE PARAMETERS

Storm	F_m(in/hr)	100-yr $\overline{Y}$	50-yr $\overline{Y}$	25-yr $\overline{Y}$	10-yr $\overline{Y}$
I	.126	.363*	.372	.380	.397
II	.115	.334	.342	.349	.364
III	.118	.344	.355	.365	.385
IV	.129	.369	.381	.392	.414
V	.125	.358	.371	.382	.405
VI	.120	.344	.358	.371	.396

* Non-homogeneous watershed area-averaged loss rate (F_m) and low loss fraction estimations for AMC II:

Total 24-hour Duration rainfall depth = 4.49 (inches)

Soil-cover Type	Area (Acres)	Percent of Pervious Area	SCS Curve Number	Loss Rate Fp(in./hr.)	Yield
1	373.80	10.00	32.	.400	.853
2	282.70	10.00	56.	.300	.870
3	8.20	10.00	69.	.250	.888
4	31.40	25.00	32.	.400	.711
5	31.30	25.00	56.	.300	.755
6	141.80	20.00	32.	.400	.758
7	83.00	20.00	56.	.300	.793
8	2.40	20.00	69.	.250	.829
9	2.50	35.00	56.	.300	.677
10	8.20	40.00	32.	.400	.569
11	562.30	50.00	32.	.400	.474
12	779.20	50.00	56.	.300	.562
13	71.30	60.00	32.	.400	.379
14	23.70	60.00	56.	.300	.485
15	16.20	85.00	32.	.400	.143
16	10.00	85.00	56.	.300	.292

Total Area (Acres) = 2428.00

Area-Averaged Loss Rate, $\overline{F}_m$ (in./hr.) = .126

Area-Averaged Low Loss Fraction, $\overline{Y}$ = .363

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C.6. BOUNDARY CONDITIONS

Table C.7 lists the floodplain rating curves information which were derived from the standard step of backwater curve analysis. Channel rating curves are listed in Table C.8.

Table C.7.

FLOODPLAIN RATING CURVES

Location	Conduit	Depth (ft)	Discharge (ft^3/sec)	α	β
State College Boulevard @ I-5	2-5x2 R.C.B.	0	0		
		0.9	25	28.1	1
		1.41	50		
		1.84	75		
		2.23	100	29.67	1.51
Katella Avenue @ I-5	1-12x2 R.C.B.	0	0		
		1.01	25	24.76	1
		1.61	50		
		2.11	75	24.53	1.49
Orangewood Avenue @ I-5	2-5x2 R.C.B.	0	0		
		1	25	25	1
		1.59	50		
		2.08	75	25.03	5

Table C.8.

CHANNEL RATING CURVES

Location	Depth (ft)	Discharge (ft^3/sec)	α	β	Remarks
C05 @ 22 Freeway[1]	0	0			1-12x6 R.C.B.
	2.33	106	45.49	1	
	5.67	408			
	10.81	950	32.1	1.43	
C05 @ 405 Freeway[2]	0	0			
	2	970	485	1	
	4	2070			
	6	3370			
	8	4370			
	10	5320	468.7	1.07	Channel Overflow
	11	6390			
	13	9600	26.05	2.3	Top of Freeway
C06@ 405 Freeway[2]	0	0			
	2	204	102	1	
	4	310	134.3	.6	
	6	970			
	8	1404			
	9.5	1758			
	10	2025			Berm Overtops
	11.2	2495	24.5	1.93	
Newland Avenue[1] @ C06	0	0			84" R.C.P.
	2.78	50	17.98	1	
	4.03	100			
	5.05	150			
	5.98	200			
	6.79	250			
	7.65	300	8.33	1.78	
Tide Gate[2]	9.2	0	0	1	
	10.2	3360			
	12.2	4320			
	14.2	4800			
	15.2	5040			
	15.5	5160	350.72	.98	

1. Data derived from standard step backwater curve analysis.
2. Data obtained from C.O.E. Feasibility Study (1988).

Inflow boundary conditions for Detailed model were derived from the Global model results using STORM I (storm centering I) of T-year events. Figures C.15 through C.21 show the inflow hydrographs for the Detailed model. In addition to the inflow hydrographs, the n-year effective rainfall also applied to the entire Detailed modeling area.

Table C.9 shows the channel outflow rating curve information for the Detailed model. The outflow relationships were based on the normal depth flow calculations.

Table C.9.

C05 CHANNEL OUTFLOW RATING CURVE
FOR DETAILED MODEL ELEMENT 353

Depth (ft)	Discharge (ft^3/sec)	α	β
0	0		
1	45.51	45.51	1
2	155		
4	564		
6	1273		
8	2331	41.84	1.94
10	3789		
12	5693	23.92	2.20

C.7. COMPARISON OF DHM RAINFALL-RUNOFF MODEL TO UNIT HYDROGRAPH METHOD

A portion of the DHM Global modeling area, from downstream of the Haster Retarding Basin to the intersection of Euclid Street and McFadden Avenue (i.e., C.P. #40 of the Hydrology Report for East Garden Grove-Wintersburg Channel, OCEMA, 1991) was used to generate channel outflow hydrographs for the 25- and 100-year storm events. Various combinations of Manning's friction factor and the effective area factor were used by the DHM in an attempt to match the runoff hydrographs generated by the above mentioned report. The same Manning's friction factors were used for both the Detailed and Global models. Figures C.22 and C.23 show the runoff hydrographs corresponding to 25- and 100-year storm events, respectively. The DHM results show much lower peak flow rates than OCEMA's results because the DHM model accounts for more retention in the catchment area than Unit Hydrograph model.

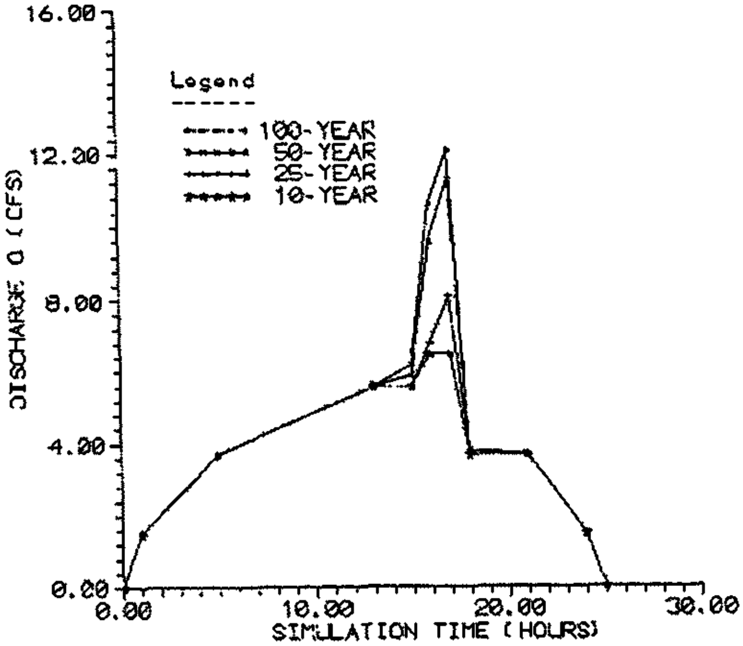

Figure C.15. Inflow Hydrographs for Floodplain Elements 5,6,7,8,9,10,11 and 12. (Detailed Model)

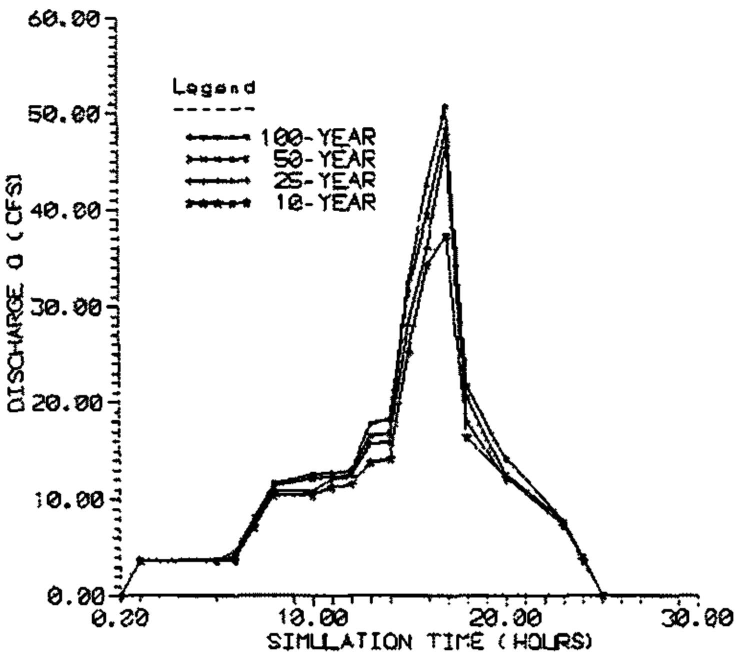

Figure C.16. Inflow Hydrographs for Floodplain Elements 73,74,75,76,77,78,79,80, 295 and 308. (Detailed Model)

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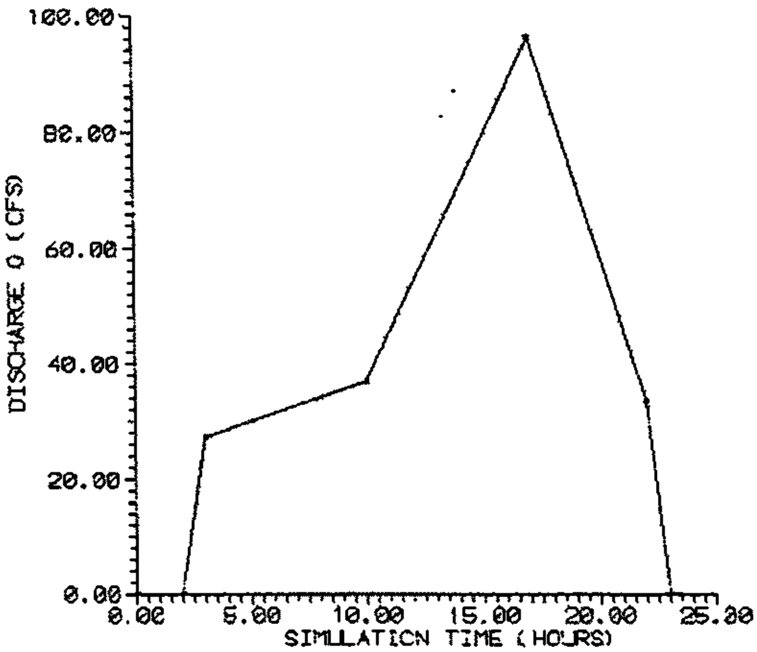

Figure C.17. Inflow Hydrograph for Floodplain Element 342. (Detailed Model)

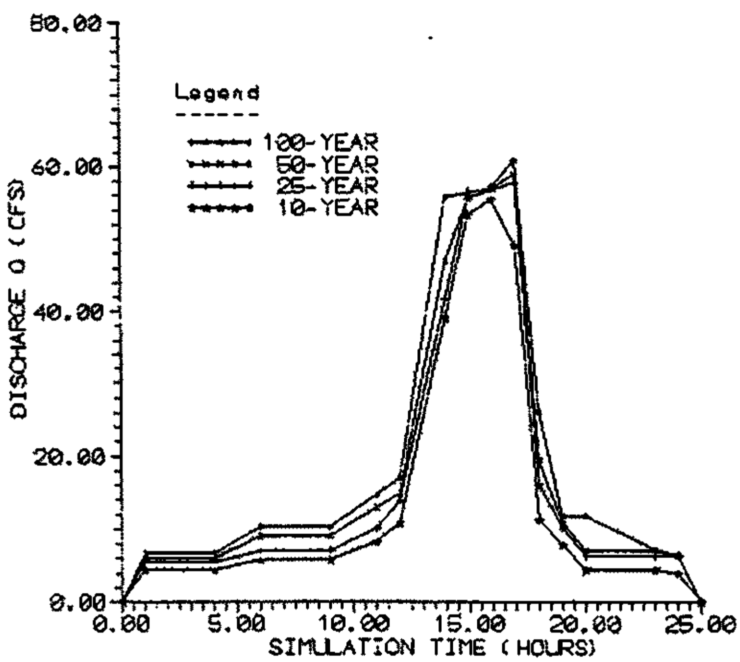

Figure C.18. Surcharged Pipeflow Assigned to Floodplain Element 7. (Detailed Model)

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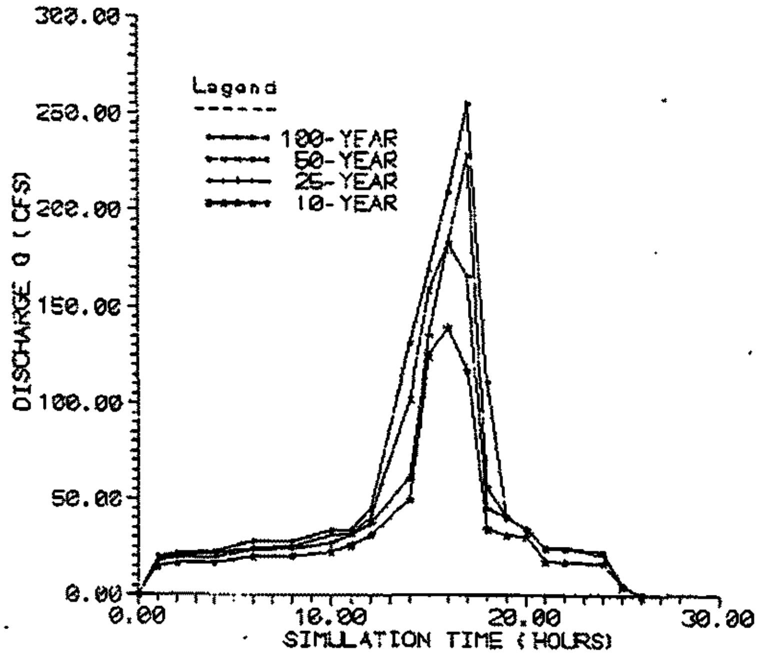


Figure C.19. Surcharged Pipeflow Assigned to Floodplain Element 73. (Detailed Model)

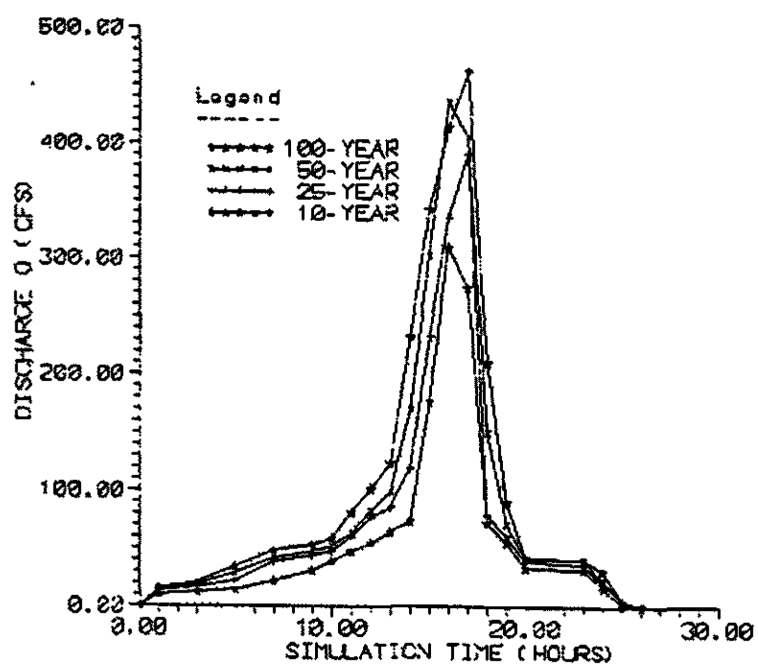


Figure C.20. Surcharged Pipeflow Assigned to Floodplain Element 295. (Detailed Model)

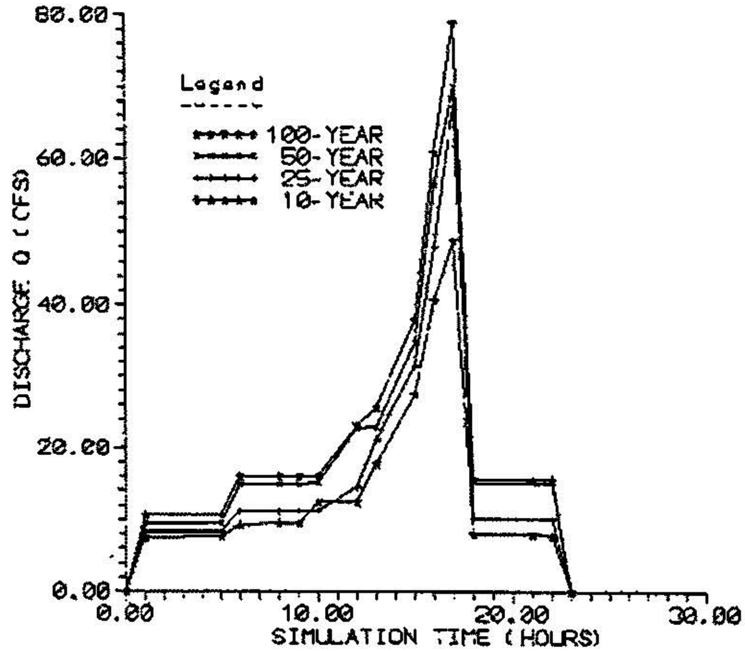


Figure C.21. Inflow Hydrographs for Channel Element 342. (Detailed Model)

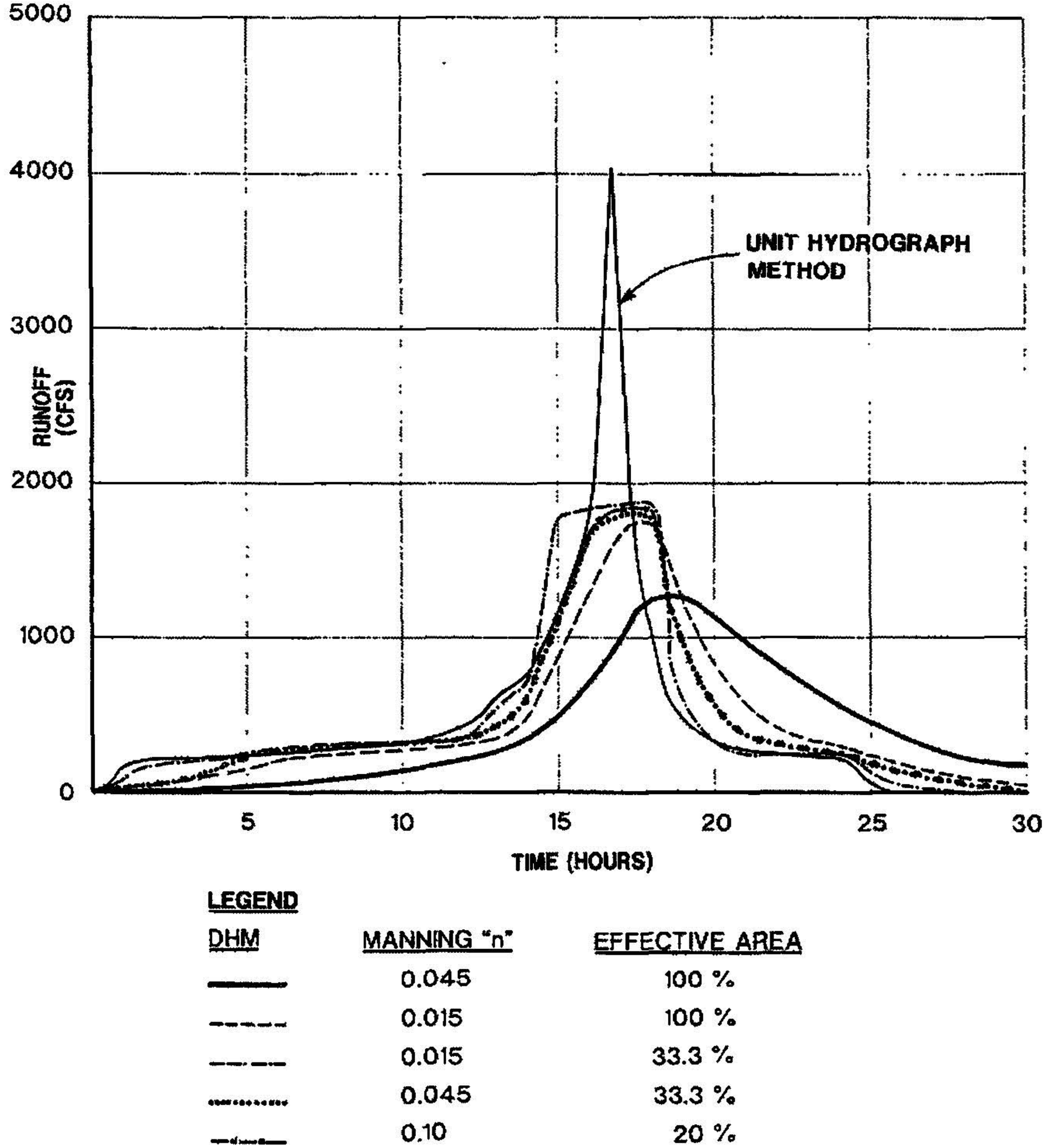
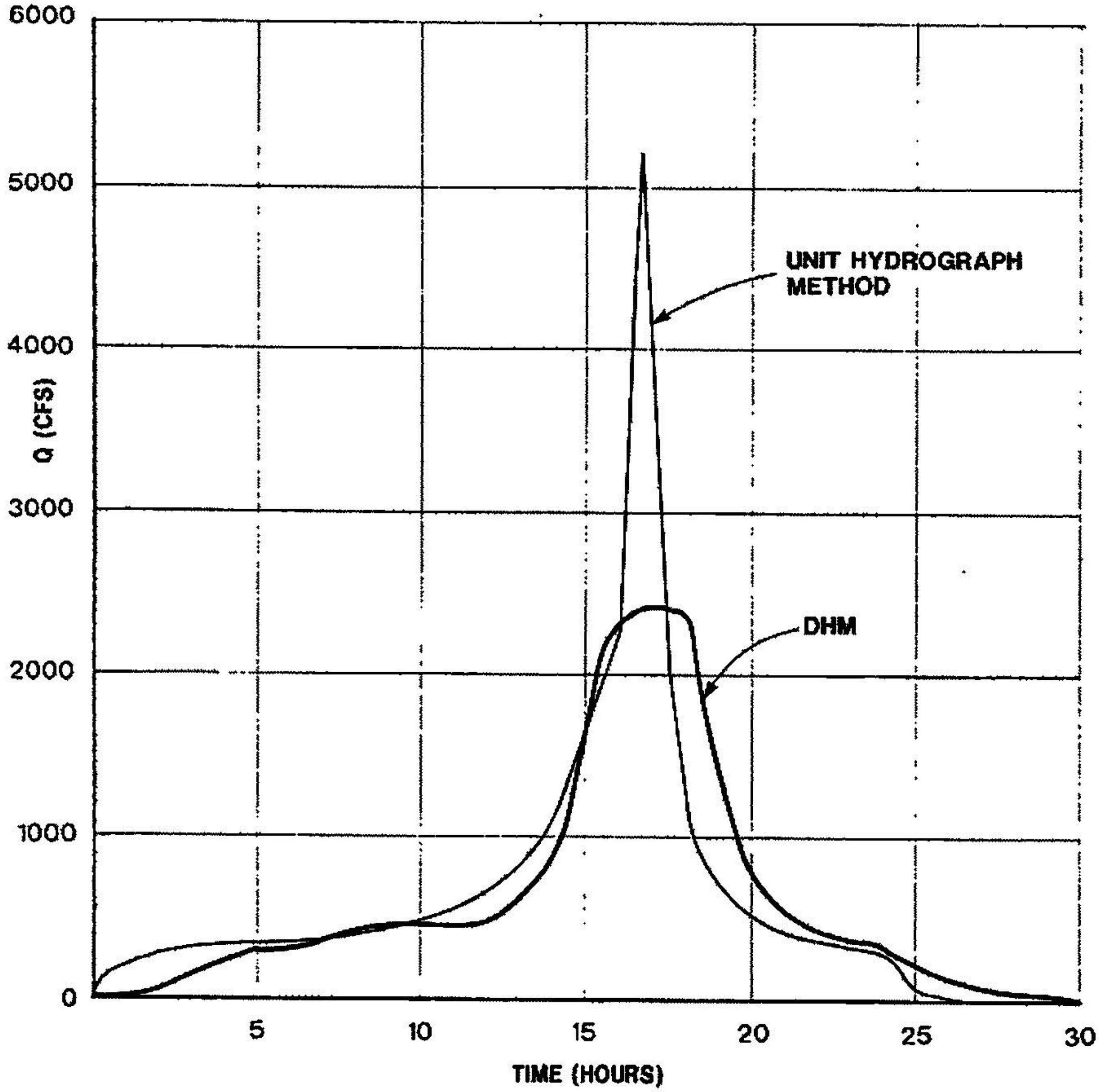


FIGURE C.22 25-YEAR HYDROGRAPHS AT C.P. # 40

WILLIAMSON & SCHMID EXHIBIT



LEGEND

DHM	MANNING "n"	EFFECTIVE AREA
———	0.045	33.3 %

FIGURE C.23 100-YEAR HYDROGRAPHS AT C.P. # 40

MAPS