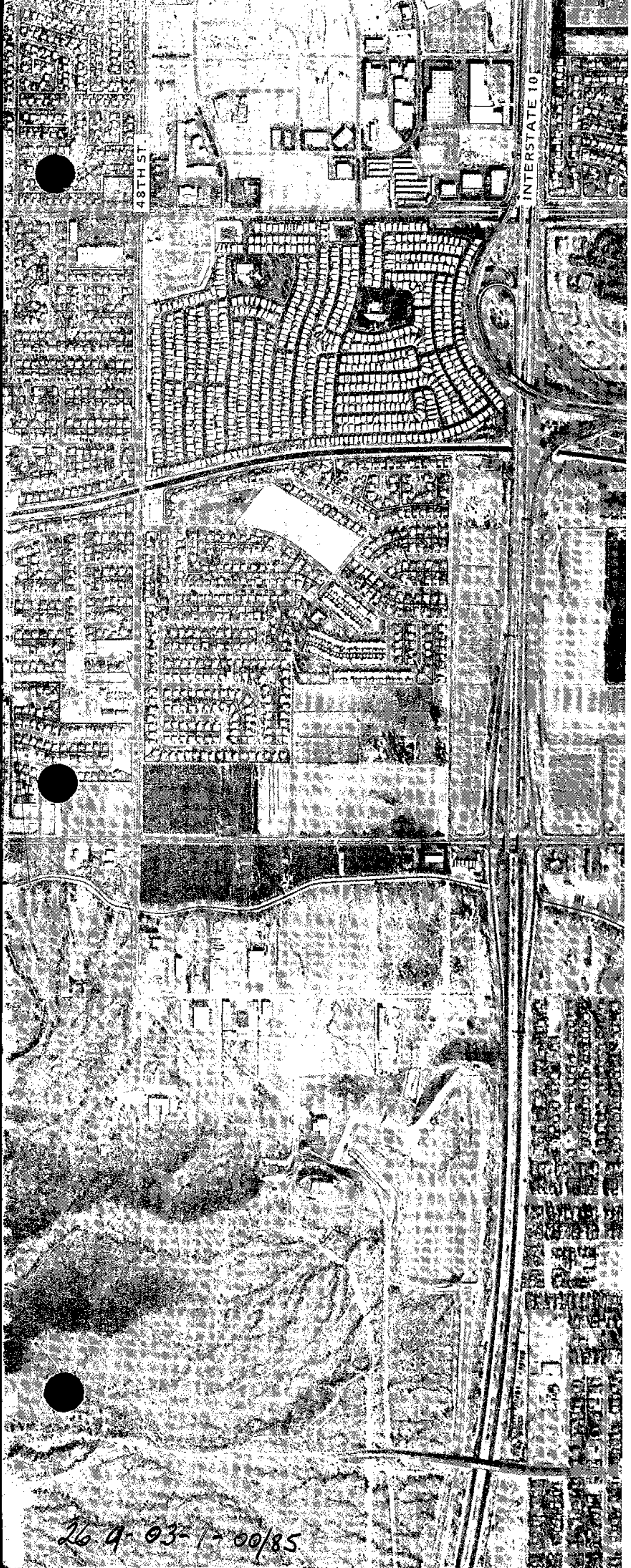

FLOOD CONTROL DISTRICT of MARICOPA COUNTY

1985

GUADALUPE FLOOD RETARDING STRUCTURE

SPILLWAY INUNDATION AREA STUDY

PREPARED BY

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By

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August, 1985

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GUADALUPE FLOOD RETARDING STRUCTURE SPILLWAY INUNDATION AREA STUDY

ABSTRACT

This report identifies expected conditions following a 100-year flood event overflow over the spillway of the Guadalupe Flood Retarding Structure.

The study is based on 100-year storm inflow hydrographs developed by the U.S. Soil Conservation Service for the design of Guadalupe Flood Retarding Structure. Outflow hydrographs were developed using the modified Puls method to route flow through a reservoir assumed full before the 100-year event. Spillway overflows to the downstream flood plain were then simulated with a two-dimensional model that also incorporates model uncertainty to determine the following:

1. Maximum expected flooding depths

2. Maximum expected runoff velocities

3. Maximum expected Froude numbers.

The results are summarized in map form in this report. A technical appendix containing computer printouts has also been prepared.

CHAPTER I

INTRODUCTION

OBJECTIVE

This report on the Guadalupe Flood Retarding Structure Spillway Inundation Area has been prepared for the Flood Control District of Maricopa County to assist in delineating the hydraulic effects and extent of inundation downstream of the emergency spillway of the Guadalupe Flood Retarding Structure (Guadalupe Dam).

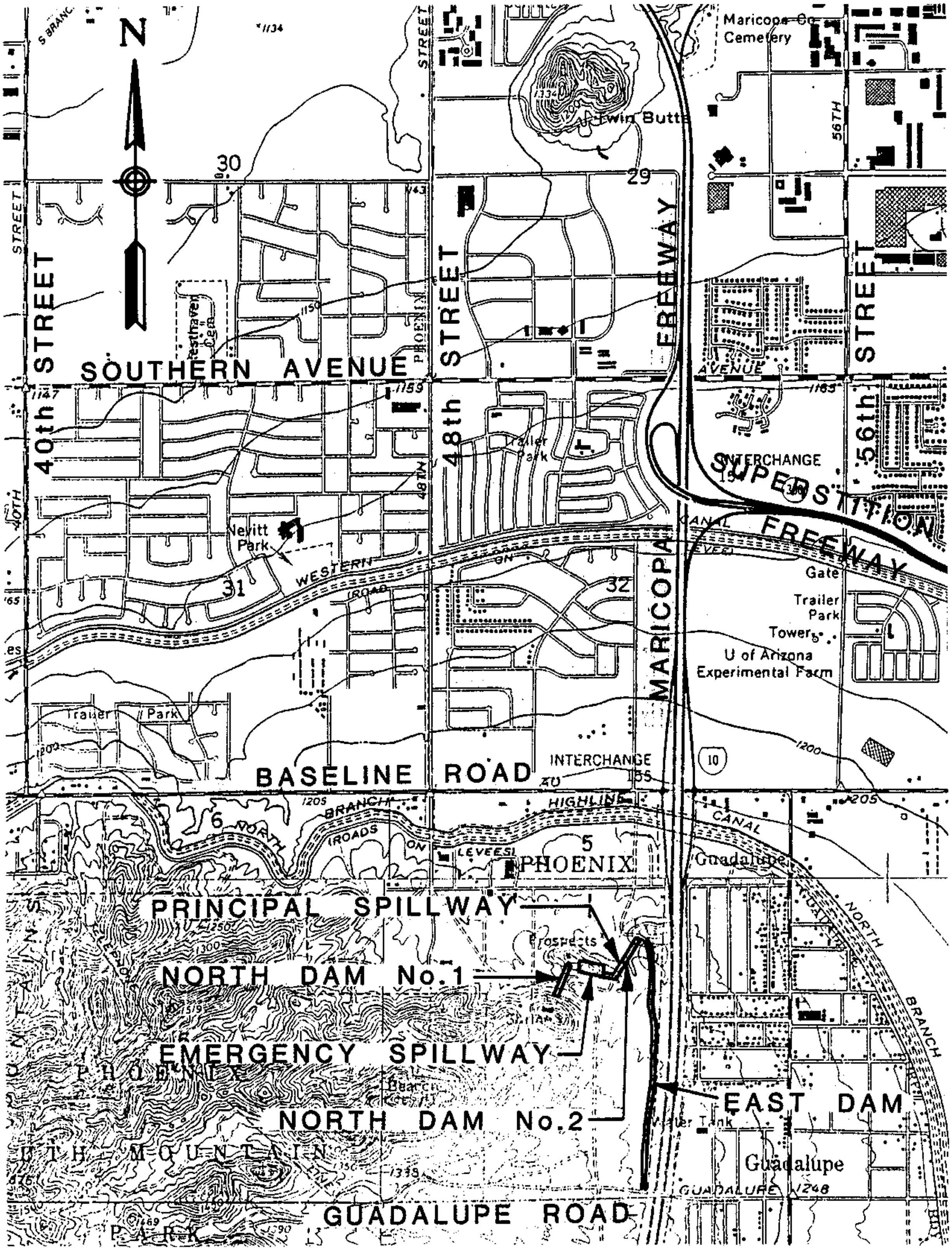
LOCATION

Figure I-1 shows the general location of Guadalupe Dam.

STUDY AREA

The study area shown on Figure I-2 has been selected to include those areas where the effects of a spillway overflow of Guadalupe Dam would be most severe. The study area extends to the north of Southern Avenue, to the east of Interstate 10, to the west of 48th street, and to the south, upstream of the Guadalupe Dam spillway.

For computational purposes, this study area has been partitioned into 252 400-foot square elements. Results were computed for each element under varying hydraulic conditions. Plate C in the back flap of this report shows the location of each 400-foot square element within the study area.

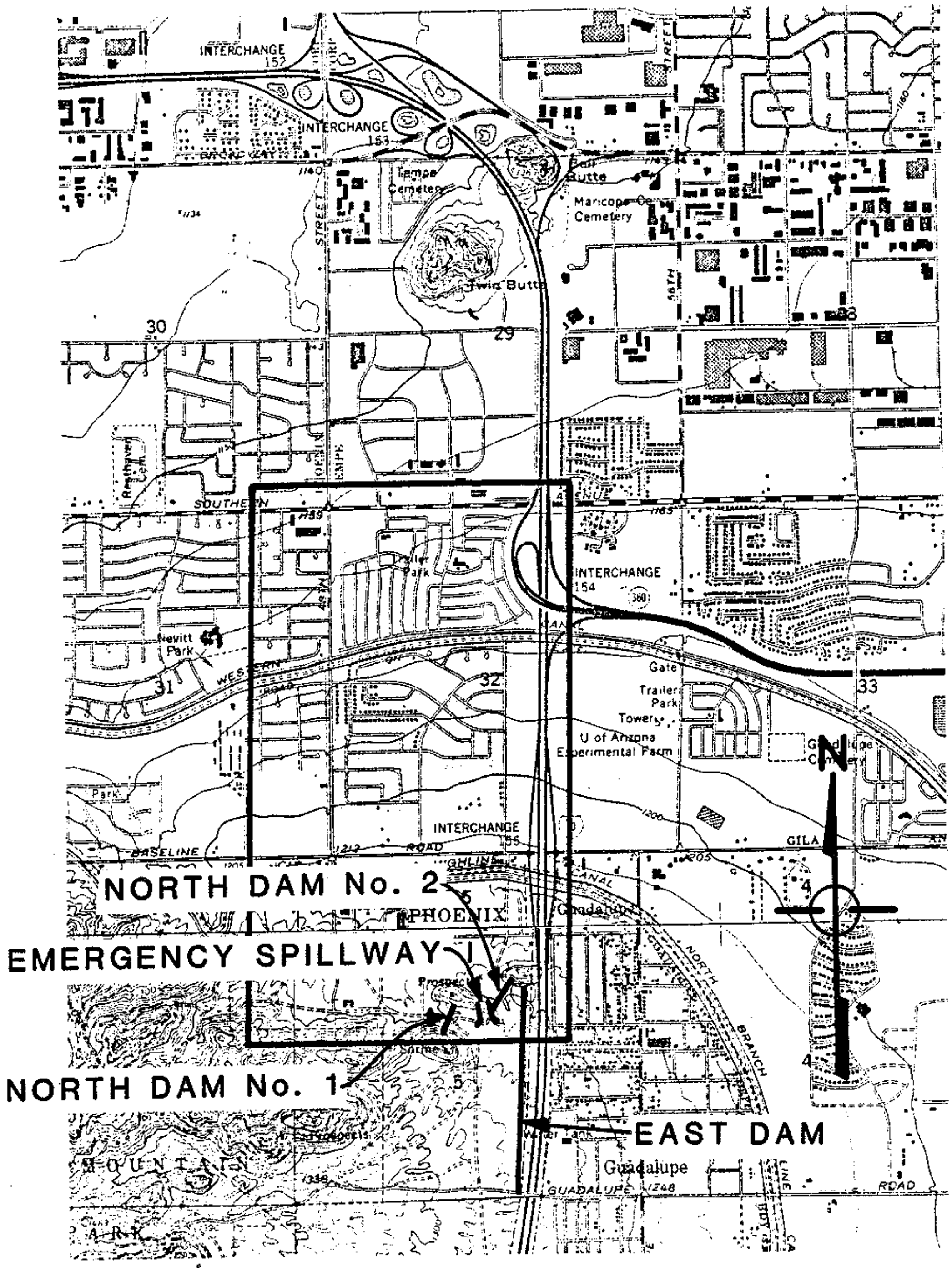


GUADALUPE FLOOD RETARDING STRUCTURE

VICINITY MAP

Not to Scale

FIGURE I-1



NOT TO SCALE

GUADALUPE DAM SPILLWAY INUNDATION
STUDY AREA BOUNDARY MAP

FIGURE I-2

LAND USE

Approximately half of the study area is now either developed or under construction. Existing development includes a mobile home park, a single family housing subdivision, a multi-family housing development and a restaurant. Industrial and commercial development is now under construction along the west side of Interstate 10.

Proposed development includes the completion of the Pointe at South Mountain planned unit development and miscellaneous commercial and residential development along Baseline Road and 48th Street. The Pointe development is expected to include a resort hotel and some multi-family housing within the study area.

CHAPTER II

ENGINEERING AND PLANNING CRITERIA

PROJECT FLOOD

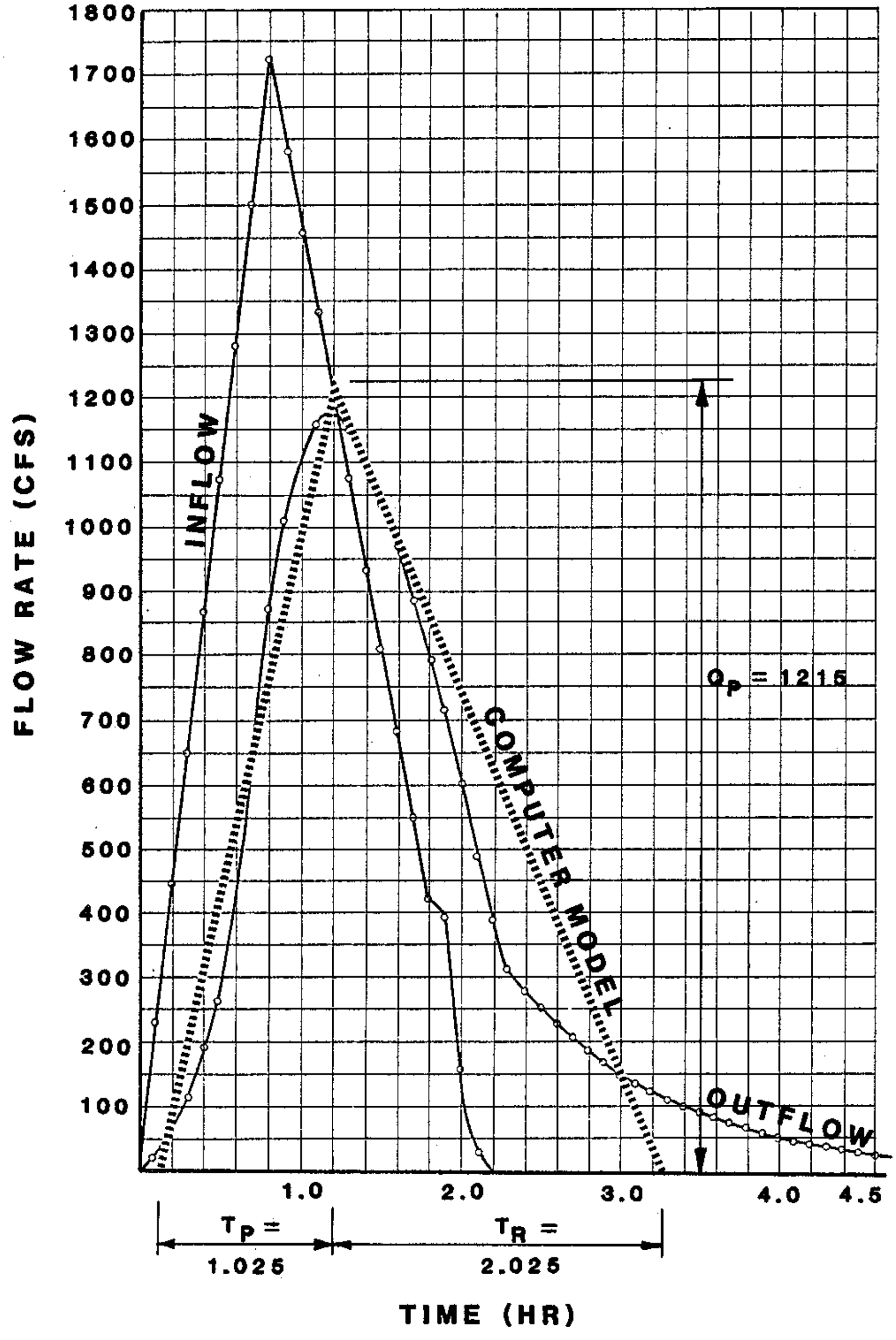
All computations used in this study are based on expected runoff from a 100-year storm event over the 1.87 square-mile watershed tributary to Guadalupe Dam. Flood retarding basin inflow hydrographs were developed prior to this study by the Soil Conservation Service and were used to design the spillway structure.

To route the inflow through the flood retarding structure, the water surface was assumed to be at the spillway crest immediately before runoff began. The modified Puls method was then used to develop the spillway overflow hydrograph.

To model the effects of spill on the downstream area, the outflow hydrograph has been transformed into triangular form as shown on Figure II-1.

FLOODPLAIN CHARACTERISTICS

Once the spillway overflow hydrograph has been computed, the next step is to simulate the flow of the spillway overflow across the downstream floodplain. To do this, a two dimensional mathematical model of the downstream floodplain was developed by superimposing a grid of 400-foot square elements over the study area. For each grid element, an



GUADALUPE DAM 100 YEAR FLOOD HYDROGRAPHS

FIGURE II-1

elevation (from USGS topographical maps) and a Manning's n value has been determined and input into the floodplain model.

A mean n value of 0.035 has been used throughout the floodplain area.

To model the floodplain overland flow, each 400-foot square element in the floodplain grid is modeled as a separate diffusion type equation. A diffusion type equation is arrived at by considering the complete hydraulic equation for two dimensional flow and assuming inertial terms are negligable. Solving the mathematics then requires solution of as many simultaneous equations as the sum of the number of grid squares and the number of grid boundaries, and repeating this process for each 0.001 hour time increment. A small simulation time step is required because an explicit method is used to solve the dynamic equations of fluid motion.

UNCERTAINTY MODELING

Hydrology is an inexact science. Development of reliable values for peak spillway discharges and times to peak require much human interpretation of watershed and hydraulic structure characteristics and their interrelationships. Estimates of Manning's n values over a floodplain may also vary. Answers then vary significantly depending on estimates of input parameters.

One traditional approach to uncertainty modeling has been to apply factors of safety to key input variables as a hedge against errors in judgement in data input. In applying factors of safety, a hydrologist might wish to increase his estimate of peak flow by 50%, increase his estimate of Manning's n by 50% and decrease his estimate of time to reach peak flow by 50% to ensure that flood depths computed downstream from a spillway are "conservative".

The main drawback to the factor of safety approach is that while it is plausible that an expert's judgement might be off by 50% on any one of several variables, the chances that all of his judgements are off by 50% in the wrong direction are very slim. Since conservative estimates ultimately end up costing the public more than accurate estimates the factor of safety approach has been criticized as being "too" conservative. The factor of safety approach also requires caution since factoring input parameters such as Manning's n may result in conservative depth estimates but also cause corresponding velocities to be underestimated. Flood damage not only depends on flood depths but flow velocities as well.

This has led to the development of probabilistic models which quantify the risk that computed results based on one or more poorly estimated input variables may be exceeded. The Monte Carlo method is the most familiar of the probabilistic methods. Here, each of several input parameters is allowed to vary normally about an estimated mean. A random number generator is used to supply the "error" to be used for each parameter for each of several hundred or even several thousand simulations. Following all the iterations, the mean and standard deviation of all the "results" can be computed. Confidence levels can then be obtained using the computed standard deviations for each output parameter.

Incorporation of Monte Carlo techniques into hydrologic simulations is not practical since it is still relatively time consuming to solve complex hydrologic computations several hundred times before obtaining results. A statistical technique known as Rosenblueth's method, however, can be used to obtain improved estimates of standard deviations and confidence levels of computed results besides reducing computing time needed. (hours as opposed to weeks). This technique has been incorporated into the computer simulations used in this study, using estimated coefficients of variation of 50% for peak flow, time to peak and Manning's n. This allows for quantifying the sensitivity of results to varying estimates for these three parameters.

CHAPTER III

SUMMARY OF COMPUTER MODEL METHODOLOGY

INTRODUCTION

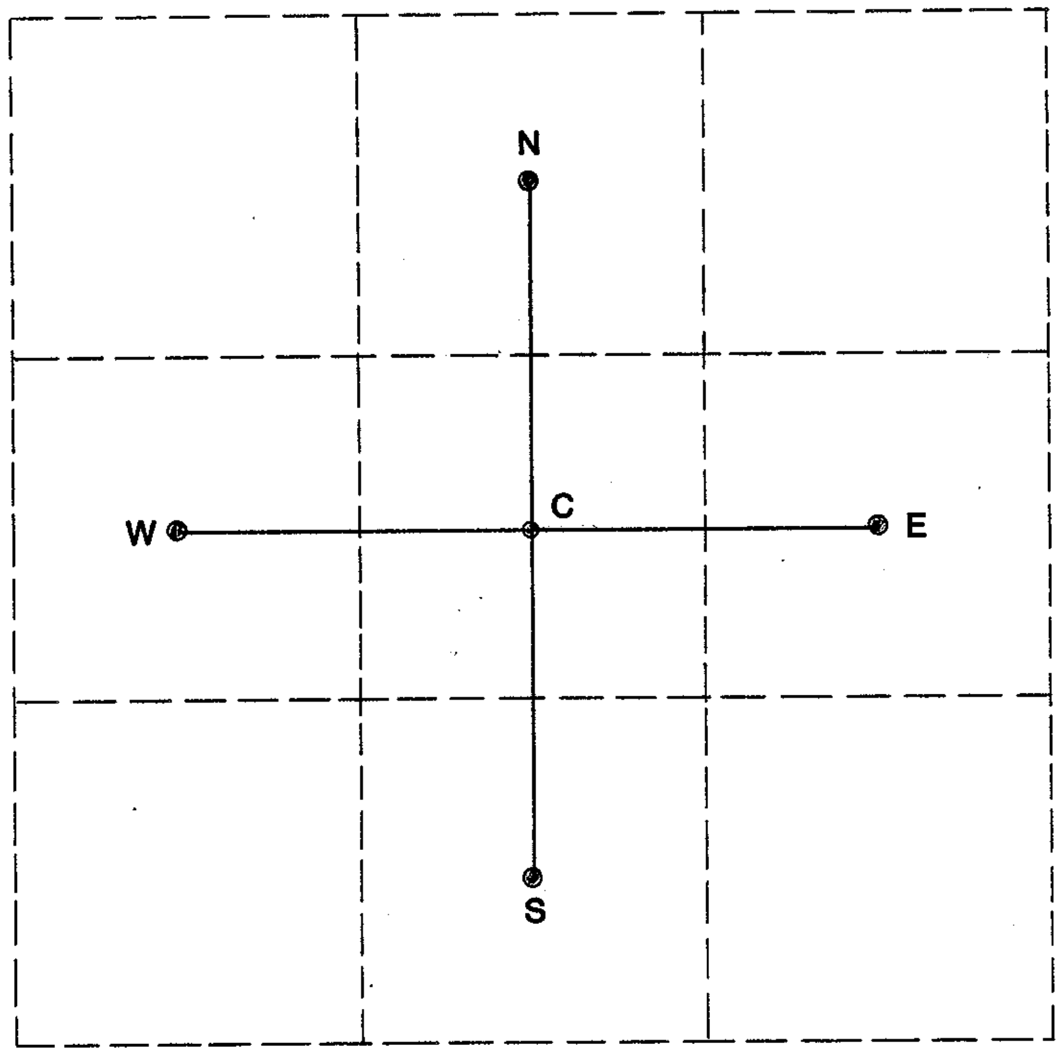
The purpose of this chapter is to summarize the mathematical techniques used in the two-dimensional diffusion-probabilistic computer model applied to this study.

Discussion in this chapter is broken into two parts since the computer program contains two successive routines. The first routine computes overland flow depths, velocities and Froude numbers based on input elevation data, Manning's n and hydrograph data. The second routine estimates the probability that the real world system might differ from computed results due to errors in estimating input data.

TWO-DIMENSIONAL FLOOD WAVE MODEL

This routine computes overland flow depths, velocities and Froude numbers based on input elevation data, Manning's n and hydrograph data.

A single cell hydraulic model such as that shown on Figure III-1, can be described using three partial differential equations. These equations are:



TWO-DIMENSIONAL FLOOD WAVE SINGLE CELL ELEMENT

FIGURE III-1

A) The continuity Equation.

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

B) The "Saint Venant" equations which describe the conservation of momentum in both the x and y directions.

$$\frac{\partial Qx}{\partial t} + \frac{\partial Qx^2/Ax}{\partial x} + \frac{\partial QxQy/Ax}{\partial y} + gAx\left(\frac{\partial h}{\partial x} + Sfx\right) = 0 \qquad (2)$$

$$\frac{\partial Qy}{\partial t} + \frac{\partial Qy^2/Ay}{\partial y} + \frac{\partial QxQy/Ay}{\partial x} + gAy\left(\frac{\partial h}{\partial y} + Sfy\right) = 0 \qquad (3)$$

where:

A = area of cell cross section on subscripted side (sf)

g = gravitational acceleration (ft/s2)

h = depth of water (ft)

Q = discharge across cell boundary on subscripted side (cfs)

q = discharge per foot of cell width (cfs/ft)

s = friction slope

t = time (sec)

The Saint Venant equations can be simplified if it is assumed that the inertial terms can be neglected when velocities are less than 25 feet per second. This simplifies the Saint Venant equations to

4

$$\frac{\partial h}{\partial x} = -Sf_x \qquad (2.1)$$

and

$$\frac{\partial h}{\partial y} = -Sf_y \qquad (3.1)$$

Flow across each boundary is described by the Manning Equation as follows:

$$Q = \left[\frac{Cm\ Ax\ Rx^{2/3}}{n\ |Sf_x|^{1/2}}\right] Sf_x \qquad (4)$$

where:

C = 1.486 for British units

n = Manning's n

Q = Flow across boundary (cfs)

R = Hydraulic Radius (ft)

and all other terms are as defined previously

The Manning equation can also be written in simplified form.

$$Q = K_x\ Sf_x \qquad (4.1)$$

where:

$$K = \frac{Cm\,Ax\,Rx^{2/3}}{n\,|Sf_x|^{1/2}} \qquad (4.2)$$

It is possible to substitute the values from equations (2.1) and (3.1) into equation (4.2) to obtain expressions for q in both the x direction and the y direction. These values can then be substituted back into equation (1) yielding:

$$\frac{\partial}{\partial x}\left[Kx\frac{\partial h}{\partial x}\right] + \frac{\partial}{\partial y}\left[Ky\frac{\partial h}{\partial y}\right] = \frac{\partial h}{\partial t}(\text{grid width}) \qquad (5)$$

This is a nonlinear partial differential equation which is very difficult to solve using numerical methods. It can be simplified by integration over x and y to the following finite difference equation which can be solved very easily.

$$Qx - (Qx + \Delta x) + Qy - (Qy + \Delta y) = \frac{\Delta h}{\Delta t}(A\ \text{cell}) \qquad (6)$$

The two dimensional flood wave model enables the computer to solve equation (6) written for each cell and equation (4.2) written for each cell boundary simultaneously. Successive solutions are computed after each 0.001-hour interval to provide a continuous simulation of expected depths, velocities, and flow rates over each cell as the flood wave progresses across the study area.

PROBABILISTIC MODEL

This routine estimates the probability that the real world system might differ from computed results due to errors in estimating input parameters. This is done using a statistical technique known as Rosenblueth's method.

The Rosenblueth algorithm proceeds as follows:

1. Estimates of the standard deviation for Manning's n, time to peak flow, and peak flow are made. From this, a high and low estimate of each of these parameters is made by adding the standard deviation to the mean to obtain the high estimate, and subtracting the standard deviation from the mean to obtain the low estimate.

2. Complete depth results are computed for each of the eight permutations of these estimates. (n-high, Q-high, t-high); (n-high, Q-high, t-low); (n-high, Q-low, t-high); etc.

3. The second moment (variance) of the depths computed at each location is then computed using the following formulas:

$$E\left[(h)^2\right] = \frac{1}{2^3}\left[(h_{+++})^2 + (h_{++-})^2 + (h_{+-+})^2 + \ldots + (h_{---})^2\right] \qquad (7.1)$$

$$V_h = E\left[(h)^2\right] - \left[E(h)\right]^2 \qquad (7.2)$$

4. The standard deviation is simply the square root of the computed variance.

CHAPTER IV

CONCLUSIONS

INTRODUCTION

This chapter summarizes overland flow depths, velocities of flow and Froude numbers computed using techniques outlined in the previous chapter. Results are reduced to graphical form for the readers convenience. Detailed computer printouts of results are contained in a separate appendix.

OVERLAND FLOW DEPTHS

Plates A and B show computed floodplain depth contours within the study area following a 100-year storm overflow of Guadalupe Dam.

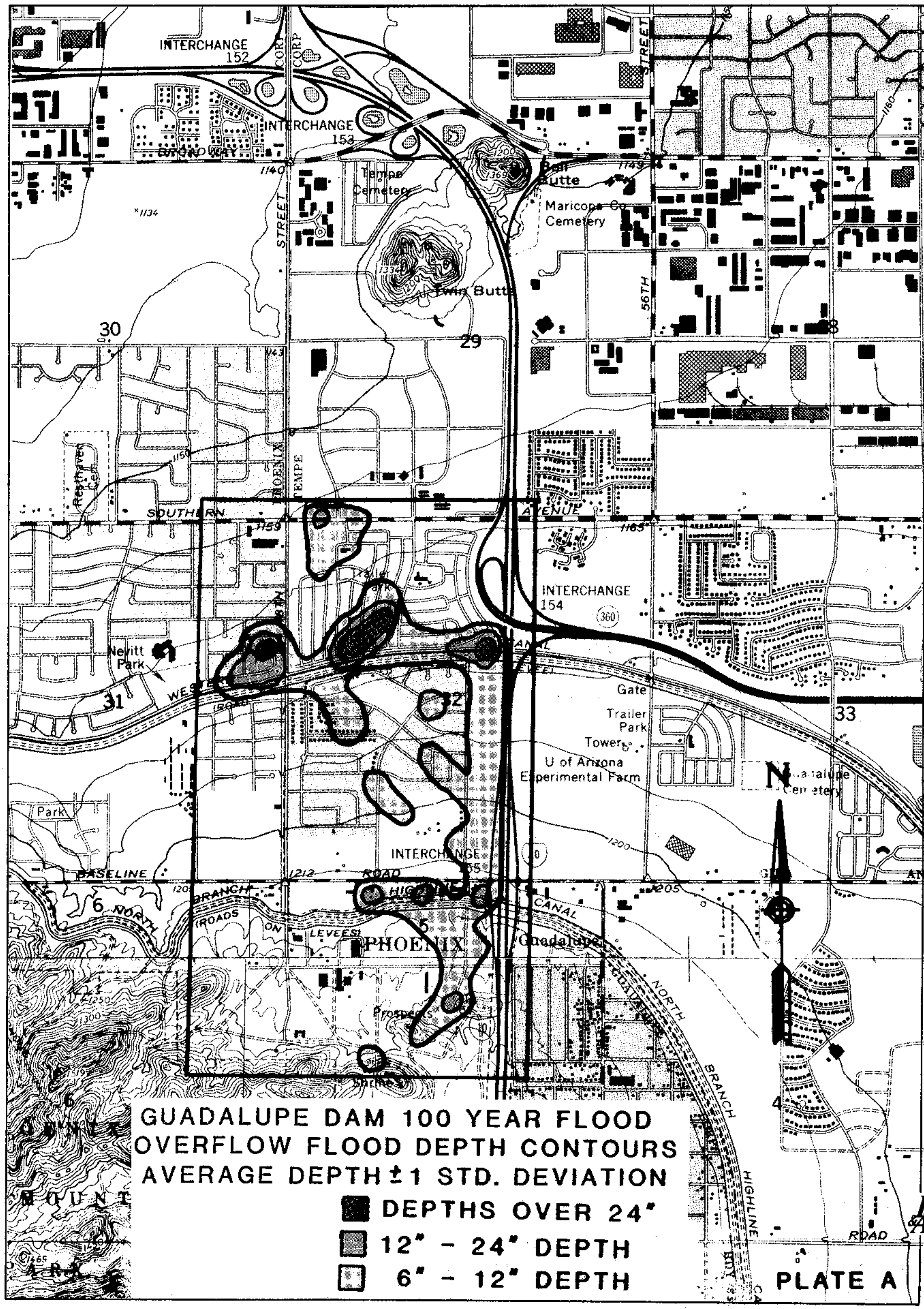
Plate A shows maximum computed water depths for all simulations run. Typically these depths occurred when the following input parameters were used:

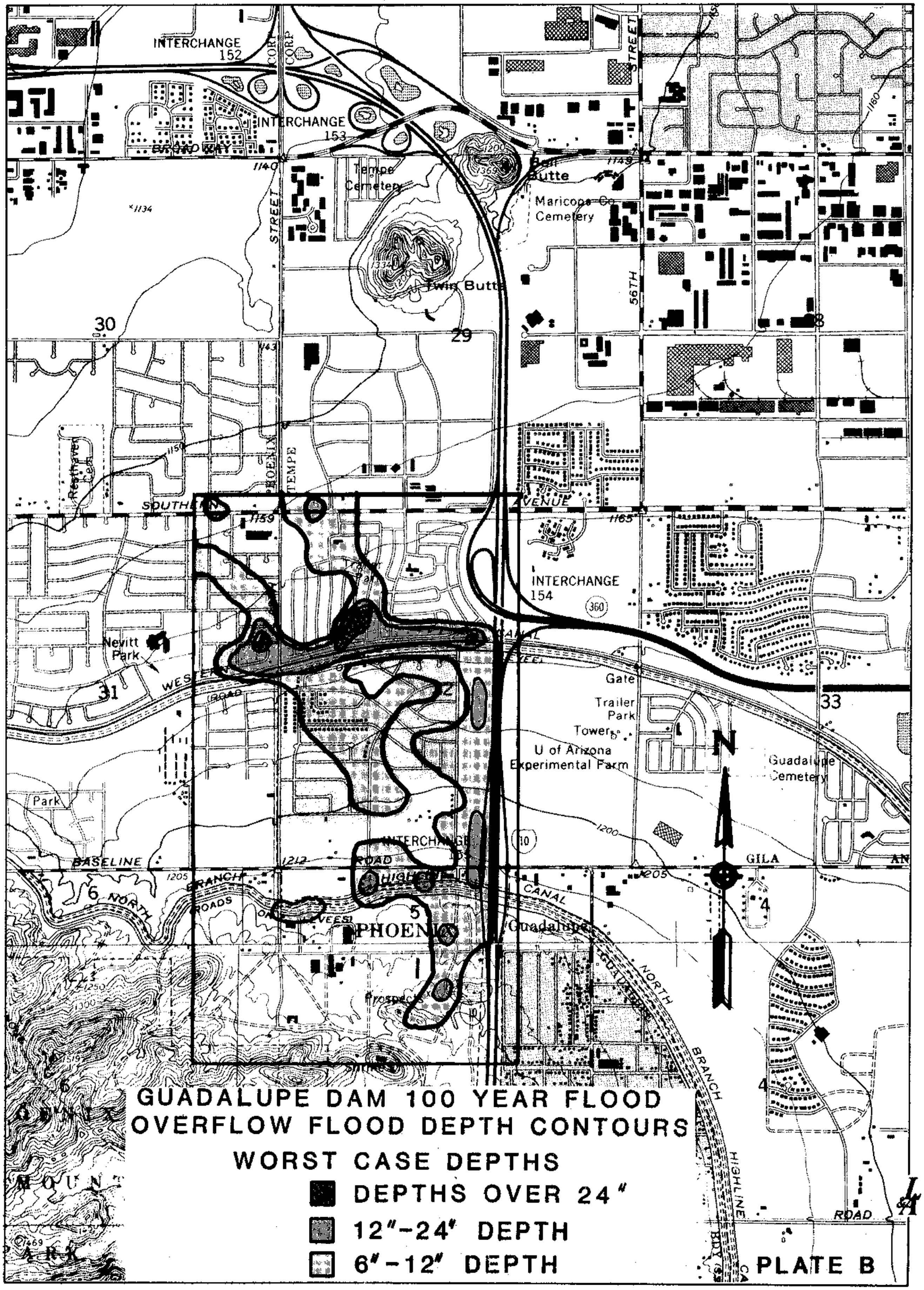
Peak flow 50% higher than mean of 1215 cfs

Time to peak 50% lower than mean of 1.025 hours

Manning's n 50% higher than mean of 0.035

In those few cases where other input parameter combinations produced greater depths in local areas, the larger depths were used to prepare Plate A.


GUADALUPE DAM 100 YEAR FLOOD
OVERFLOW FLOOD DEPTH CONTOURS
AVERAGE DEPTH ±1 STD. DEVIATION
DEPTHS OVER 24"
12" - 24" DEPTH
6" - 12" DEPTH
PLATE A
INTERCHANGE 152
INTERCHANGE 153
INTERCHANGE 154
Bell Butte
Tempe Cemetery
Maricopa Co. Cemetery
Twin Butte
SOUTHERN AVENUE
Nevitt Park
U of Arizona Experimental Farm
Trailer Park
PHOENIX
Guadalupe
BASELINE ROAD
HIGHLINE CANAL
NORTH BRANCH



GUADALUPE DAM 100 YEAR FLOOD
OVERFLOW FLOOD DEPTH CONTOURS
WORST CASE DEPTHS
DEPTHS OVER 24"
12"-24" DEPTH
6"-12" DEPTH
PLATE B
INTERCHANGE 152
INTERCHANGE 153
INTERCHANGE 154
INTERCHANGE 155
BROADWAY
Tempe Cemetery
Bell Butte
Maricopa Co Cemetery
Twin Butte
56TH STREET
SOUTHERN AVENUE
Nevitt Park
WESTERN CANAL
Trailer Park
U of Arizona Experimental Farm
Guadalupe Cemetery
BASELINE ROAD
GILA
HIGHLINE CANAL
PHOENIX
Guadalupe
Prospect
Park
N

Plate B was obtained by plotting average depths plus one standard deviation. The probability that these average depths plus one standard deviation would be exceeded because of errors in judgement in selecting input parameters is less than 14%.

OVERLAND FLOW VELOCITIES

Figure IV-1 shows worst-case overland flow velocity contours within the study area following a 100-year storm overflow of Guadalupe Dam. In most cases these velocities occurred when the following input parameters were used:

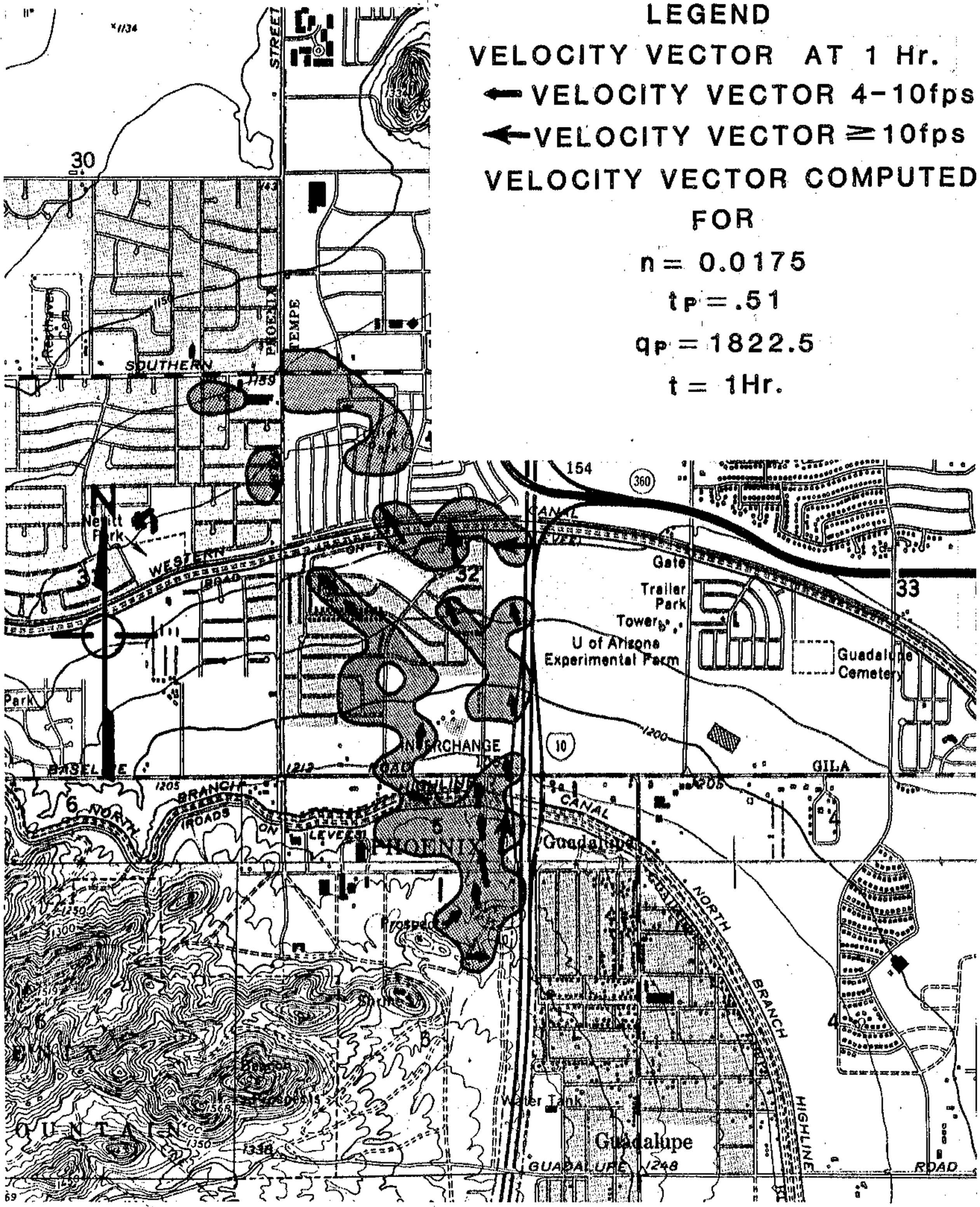
Peak flow 50% higher than mean of 1215 cfs

Time to peak 50% lower than mean of 1.025 hours

Manning's n 50% lower than mean of 0.035

In those cases where other input parameter combinations produced the highest velocities in local areas, the higher velocities were used.

Figure IV-1 also shows the general direction of flow computed one hour after the spillway overflow begins. These vectors are valid only for a time of one hour after overflow and will be different at other times. They are included to present a general concept of expected flow patterns within the study area.

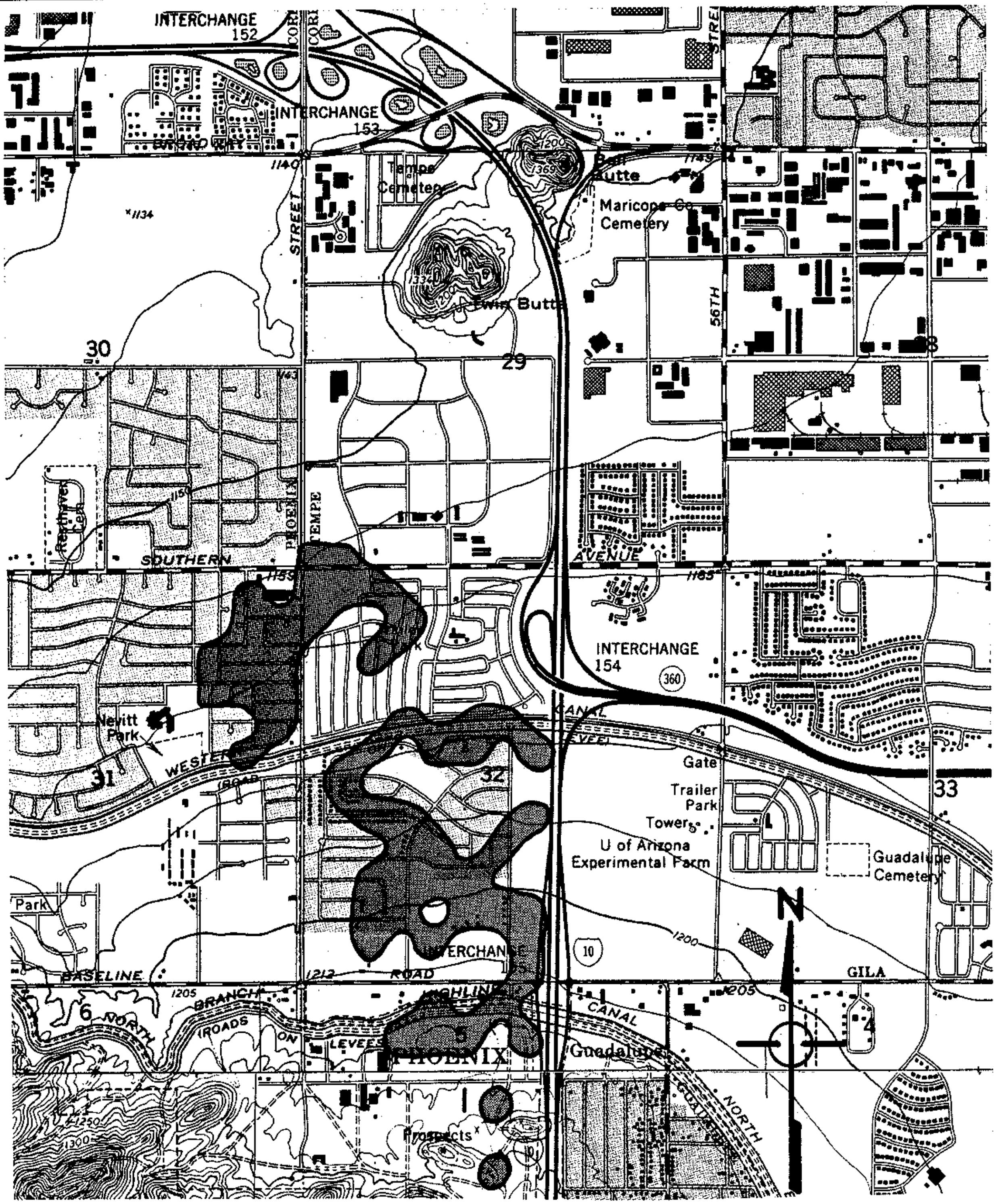


GUADALUPE DAM 100 YEAR FLOOD
OVERFLOW AREAS HAVING V≥4 fps

FIGURE IV-1

FROUDE NUMBERS

Figure IV-2 shows those portions of the study area where under any condition, Froude numbers were found to exceed one. In come cases where depths are very shallow, Froude numbers over one may be tolerable. Detailed results contained in the computer printout appendix to this report may assist in making this determination.



LEGEND

AREAS HAVING Fr ≥ 1

GUADALUPE DAM 100 YEAR FLOOD
OVERFLOW AREAS HAVING Fr > 1

FIGURE IV-2

REFERENCES

Guymon, G.L. et al., 1971, A Probabilistic-Deterministic Analysis of One-Dimensional Ice Segregation in a Freezing Soil Column, Cold Regions Science and Technology, 5, 127-140, Elsevier Scientific Publishing Company, Amsterdam.

Guymon, G.L. and Hromadka, T.V. II., Two-Dimensional Diffusion-Probabilistic Model of a Slow Dam Break, submitted to Water Resources Bulletins

Hromadka, T.V. II, Berenbrock, C.E., Freckleton, J.R., Guymon, G.L., 1985, A two-dimensional dam-break flood plain model, Advances in Water Resources, Vol. 8, pp. 7-14.

United States Soil Conservation Service, Design Calculations Summary for Guadalupe Watershed Arizona Guadalupe Flood Retaining Structure, 1983.

United States Soil Conservation Service, Guadalupe Watershed Arizona, Guadalupe Flood Retarding Structure Hydrology, (file containing miscellaneous hydrologic calculations), 1980.

-Spookhill Study-

1- Upon implementation and completion of Spook Hill Area Drainage Master Study the design hydrology that should be utilized should be that which was reviewed and approved in the ADMS and not the SCS hydrology which was questioned in the past -- In essence the hydrology in the report is antiquated and upon completion of the ADMS the Structure will have more than the initial design detention due to detention in the upstream watershed.

2 In lieu of buying flowage easements downstream of the the structure from the cost standpoint to handle the excess runoff, it may be more advantagous to enlarge the existing channel and outlet structures under Bush highway instead of buying easements through Hughes Helicopter Plant, Major sand and gravel companies, residentual and commercial developments proposed in that area in addition to SRP and Mesa land holdings.

-Guadalupe Study-

1- 1822 cfs just doesn't disappear at the study area boundaries but should be routed down to a major discharge point instead of stopping at Southern.

2- Question some of the depth areas on Figure IV-2 in that they are in subdivisions with constant slopes. Also question what happens at both the Western and Highland Canals with respect to lateral movement.

3- Gosnell and other developments have constructed fences perpendicular to the flows some 1/4 to 1/2 mile in length. This would tend to concentrate the flows into the major north/south streets resulting in higher flow rates, froude numbers etc. which are not indicated in the figures.

-General Comment on both studies-

Question the overall results when 5 foot topography is used to depict 6 inch differentials in flows while ignoring 6 foot walls and structures.

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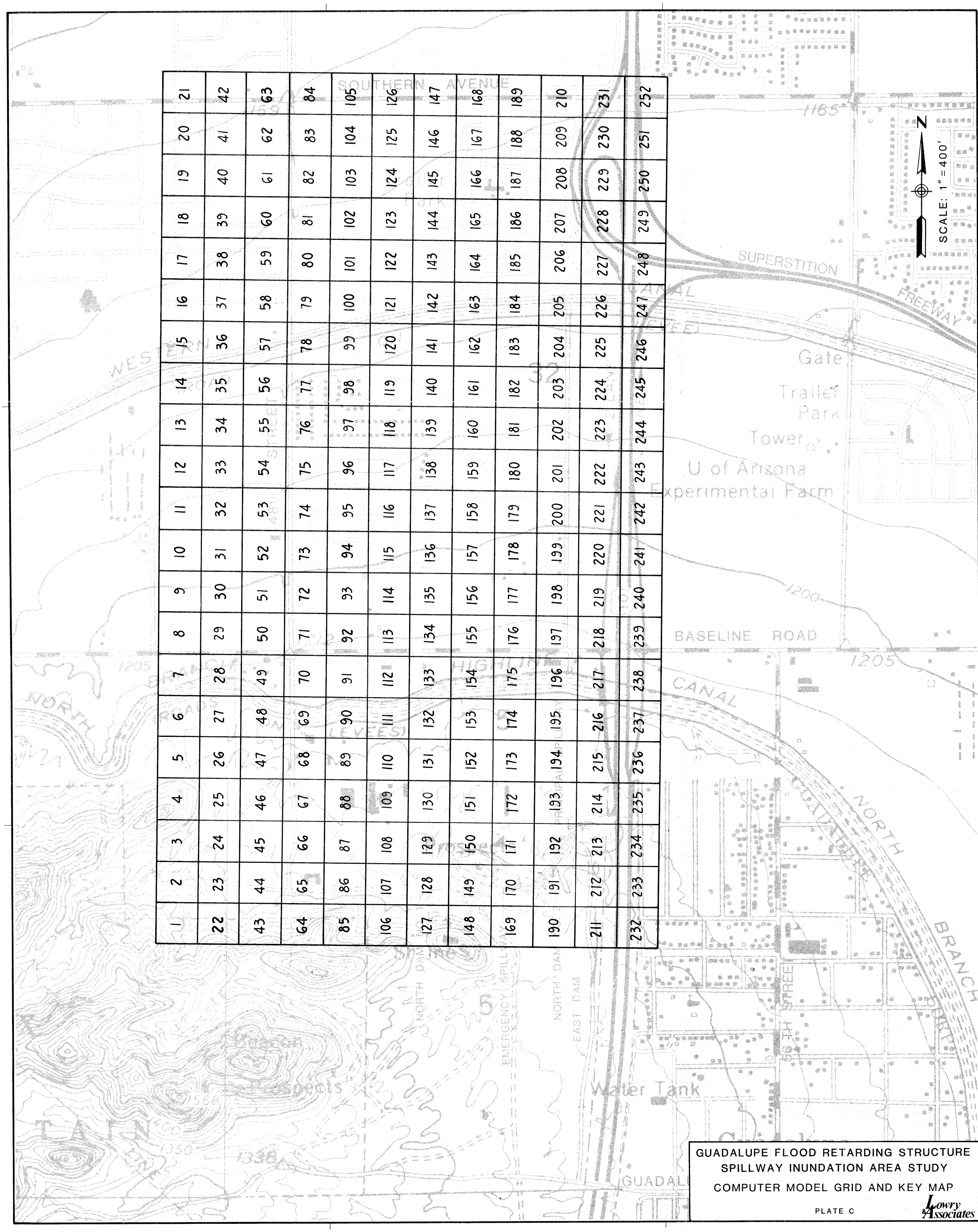

GUADALUPE FLOOD RETARDING STRUCTURE
SPILLWAY INUNDATION AREA STUDY

COMPUTER MODEL GRID AND KEY MAP

PLATE C

Lowry Associates