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A Two-Dimensional Dam-Break Model of the Orange County Reservoir

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Abstract. The diffusion model of two-dimensional flood flow over a broad plain is modeled using an integrated finite-difference technique. As a case study, a hypothetical failure of the Orange County Reservoir is considered. The resulting flood plain is shown to be of broader extent than a similar study based on a one-dimensional routing technique.

INTRODUCTION

Purpose of Report

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

Project Site

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

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

Study Methodology

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

To account for these dynamic processes, a two-dimensional diffusion dam-break model is used to estimate the flood plain resulting from a hypothetical failure of the subject Orange County Reservoir site.

Orange County Reservoir

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

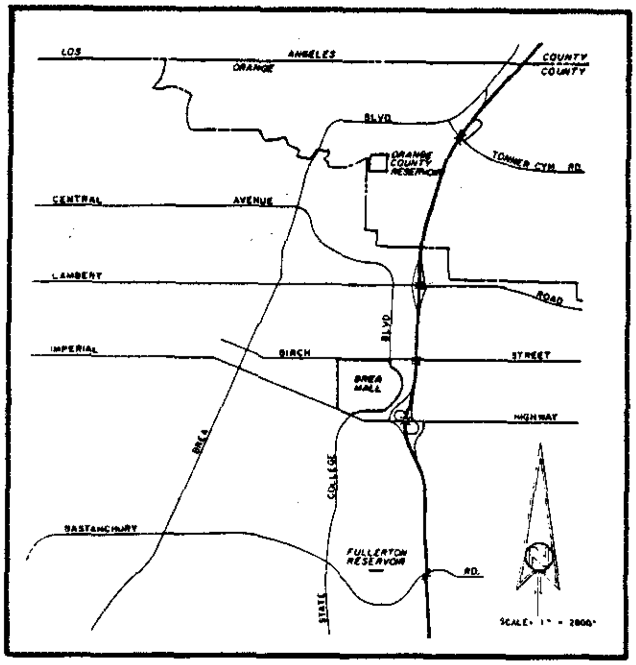

Figure 1. Location Map

then Eq. (2) can be rewritten as

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

In Eq. (4), the subscript x included in m_x indicates the directional term. The expansion of Eq. (2) to the two-dimensional case leads directly to the terms (m_x, m_y) except that now a cross-product of flow velocities are included, increasing the computational effort considerably.

Rewriting Eq. (3) and including Eqs. (4) and (5), the directional flow rate is computed by

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

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

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

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

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

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

The one-dimensional diffusion model of Akan and Yen (1981) assumes $m_x = 0$ in Eq. (7).[6] Likewise, the two-dimensional diffusion model of Xanthopoulos and Koutitas (1976) assumes $m_x = m_y = 0$.[5] Thus, the one-dimensional diffusion model is given by

$$\frac{\partial}{\partial x} K_x \frac{\partial H}{\partial x} = \frac{\partial A_x}{\partial t} \qquad (9)$$

where K_x is now simplified as

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

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

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

Model Accuracy in Prediction of Flood Depths

In order to evaluate the accuracy of the diffusion model of Eq. (11) in the prediction of flood depths, the U.S.G.S. fully dynamic flow model K-634 (Land, 1980a,b) is used to determine channel flood depths for comparison purposes.[1,2] The K-634 model solves the coupled flow equations of continuity and *momentum by an implicit finite difference approach* and is considered to be a highly accurate model for many unsteady flow problems. The study approach is to compare predicted flood depths predicted from both the K-634 and the diffusion (Eq. 11) model for various channel slopes and inflow hydrographs.

In this case study, two hydrographs are assumed; namely, peak flows of 120,000 cfs and 600,000 cfs. Both hydrographs are assumed to increase linearly from zero to the peak flow rate at time of 1-hour, and then decrease linearly to zero at time of 6-hours (see Fig. 2 inset). The study channel is assumed to be a uniform rectangular section of Manning's n equal to 0.040, and various slopes S_o in the range of $0.001 \le S_o \le 0.01$. Figure 2 shows the comparison of modeling results. From the figure, various flood depths are plotted along the channel length of up to 10-miles. Two reaches of channel lengths of up to 30-miles are also plotted in Fig. 2 *which correspond to a slope* $S_o = 0.0020$. *In all* tests, grid spacing was set at 1000-foot intervals.

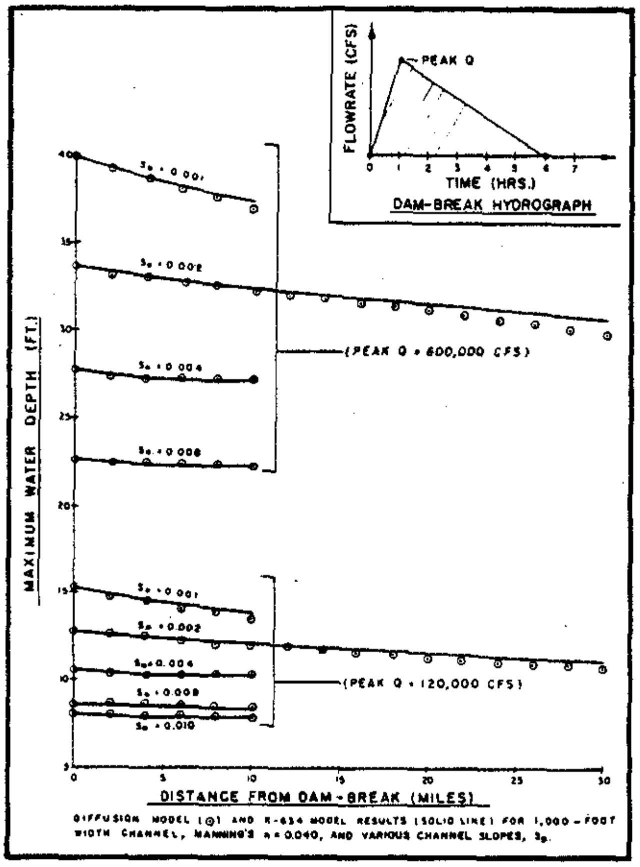

Figure 2. Comparison of K-634 Model to Diffusion Model

From Fig. 2 it is seen that the diffusion model provides estimates of flood depths that compare very well to the flood depths predicted from the K-634 model. Differences in predicted flood depths are less than 3-percent for the various channel slopes and peak flow rates considered.

Discussion of One-Dimensional Model

From the above conclusions, use of the diffusion approach of Eq. (11) in a two-dimensional dam-break model may be justifiable due to the low variation in predicted flooding depths (one-dimensional)

The local and convective acceleration momentum terms can be grouped together such that Eq. 12, 13, and 14 are rewritten as

$$m_z + \left[S_{fz} + \frac{\partial H}{\partial z} \right] = 0, \; z = x,y \tag{15}$$

where m_z represents the sum of the first three terms in Eqs. (13) and (14), and divided by gA_z. Assuming the friction slope to be approximated by steady flow conditions, the Manning's equation in inch-pound units can be used to estimate

$$Q_z = \frac{1.486}{n} A_z R_z^{2/3} S_{fz}^{1/2} \tag{16}$$

where

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

Equation 16 can be rewritten as

$$Q_z = - K_z \frac{\partial H}{\partial z} - K_z m_x \; , \; z = x,y \tag{17}$$

where

$$K_z = \frac{1.486}{n} A_z R_z^{2/3} \Big/ \left| \frac{\partial H}{\partial z} + m_z \right|^{1/2} , \; z = x,y \tag{18}$$

Hromadka (1984), Akan and Yen (1981), and Xanthopoulos and Koutitas (1976) assume m_x and m_y are both negligible, resulting in the simple diffusion model (see preceding report section) [7,6,5]

$$Q_z = - K_z \frac{\partial H}{\partial z} \tag{19}$$

The proposed two-dimensional dam-break model is formulated by substituting Eq. 19 into the continuity equation giving

$$\frac{\partial}{\partial x} K_x \frac{\partial H}{\partial x} + \frac{\partial}{\partial y} K_y \frac{\partial H}{\partial y} = \frac{\partial H}{\partial t} \tag{20}$$

Numerical Model Formulation (Grid Elements)

For uniform grid elements, the numerical modeling approach used is the integrated finite difference version of the nodal domain integration (NDI) method. For grid elements, the NDI nodal equation is based on the usual nodal system shown in Fig. 4. Flow rates along the boundary Γ are estimated using a linear trial function assumption between nodal points.

For a square grid of width δ

$$Q\Big|_{\Gamma_E} = - \left(K_x\Big|_{\Gamma_E} \right) \left(H_E - H_C \right) \Big/ \delta \tag{21}$$

where

$$K_x\Big|_{\Gamma_E} = \begin{cases} 1.486 \left(A \; R^{2/3}/n \right)_{\Gamma_E} \Big/ \left| \left(H_E - H_C \right) \big/ \delta \right|^{1/2} ; & \bar{D} > 0 \\ 0 \; ; \; \bar{D} \le 0 \;\; \text{or} \;\; |H_E - H_C| < 10^{-3} & \end{cases} \tag{22}$$

In Eq. 22, the terms A, R, and n are evaluated at the average flow depth defined by $\bar{D} = (D_E + D_C)/2$, and $n = (n_E + n_C)/2$. Additionally, the denominator of K_x is checked such that K_x is set to zero of $|H_E - H_C|$ is less than a tolerance such as 10^{-3} ft.

The model advances in time by an explicit approach

$$\underset{\sim}{H}^{i+1} = \underset{\sim}{K}^{i} \underset{\sim}{H}^{i} \tag{23}$$

where the assumed dam-break flows are added to the specified input nodes at each timestep. After each timestep, the conduction parameters of Eq. 22 are reevaluated, and the solution of Eq. 23 reinitiated. Using grid sizes with uniform lengths of one-half mile, timesteps of size 3.6 sec were found satisfactory.

Two-Dimensional Model Verification

Applying the K-634 model to computing the two-dimensional flow (Hromadka et al.,1984) was attempted by means of the one-dimensional nodal spacing shown in Fig. 3.[7]

Cross sections were obtained by field survey, and the elevation data used to construct nodal point flow-width versus stage diagrams for use in the K-634 model. A constant friction factor (Manning's) of 0.04 was assumed for study purposes. The assumed dam-break failure reached a peak flow rate of 420,000 cfs within one hour, and returned to zero flow 9.67 hours later. The resulting K-634 flood plain limits is shown in Fig. 5. As discussed in the previous section, a slight gradient was assumed for the topography perpendicular to the main channel. The motivation for specifying such a gradient was to limit the channel floodway section in order to approximately conserve the one-dimensional momentum equations. As a result of this assumption, fictitious channel sides are included in the K-634 model study which results in an artificial confinement of the flows. Thusly, a narrower flood plain is delineated (such as shown in Fig. 5) where the flood flows are falsely retained within a hypothetical channel confine. An examination of the flood depths given in Fig. 7 indicates that at the widest flood plain expanse of Fig. 5, the flood depth is about 6-feet, yet the flood plain is not delineated to expand southerly, but is modeled to terminate based on the assumed

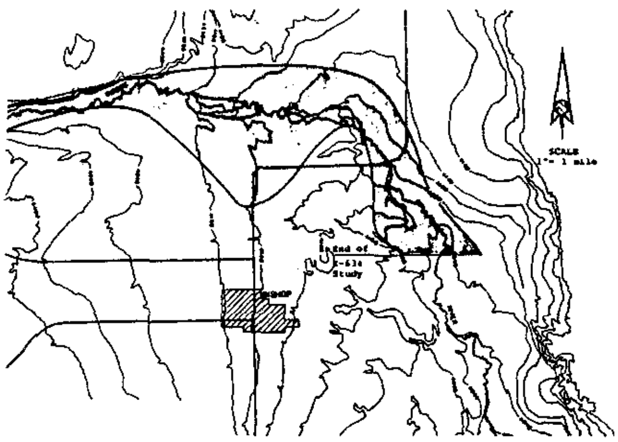

Figure 5. Floodplain Computed from K-634 Model

the low volume (200 acre-feet) retained in the reservoir, the outlet hydrograph assumed is a significant function of the remaining reservoir storage. That is, the reservoir volume is quickly depleted by low-to-moderate flows out of the reservoir.

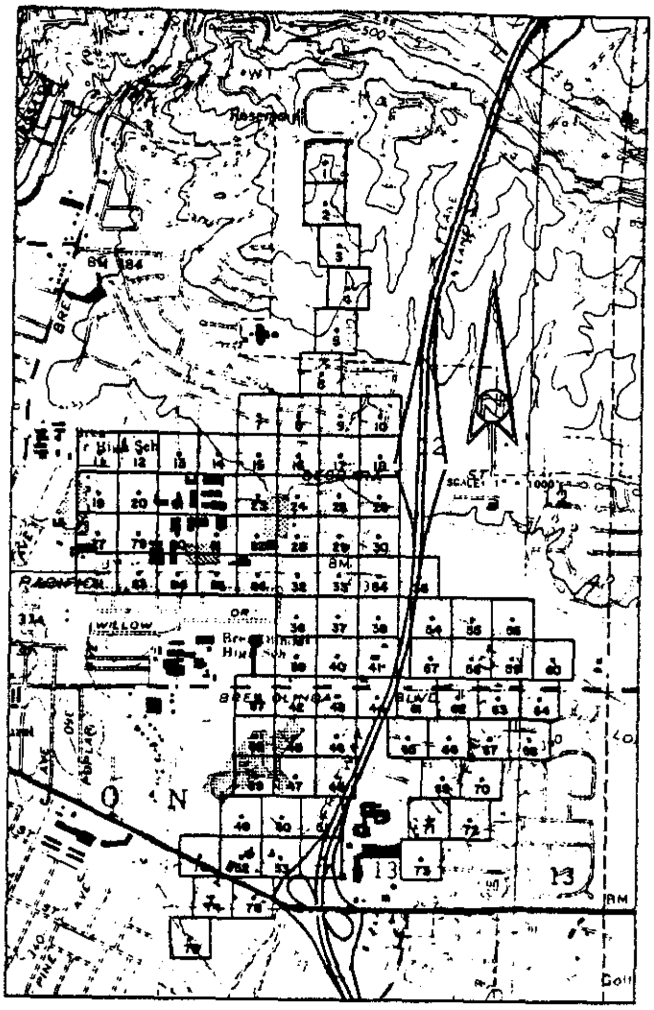

Figure 8. Domain Discretization

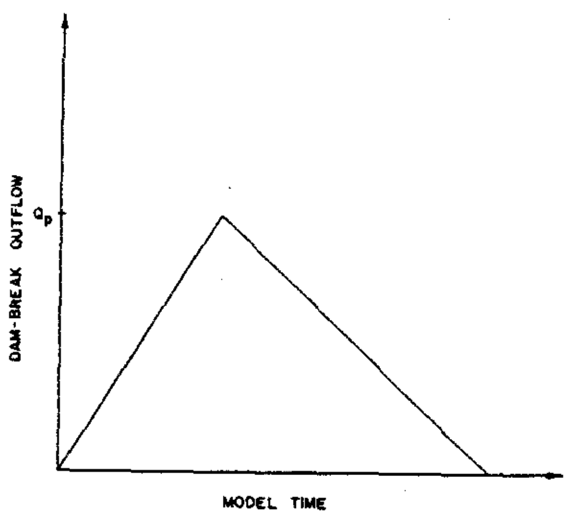

Figure 9. Example Outflow Hydrograph

To estimate a reasonable peak outlet flow, Q_p, an iteration method is used until a balance between the estimated outlet hydrograph Q_p is made to the resulting flowrate as a function of the remaining stored waters. The ultimate outlet geometry is assumed to be a V-shaped massive failure with side slopes at a 45-degree incline. Flows are then based on critical depth, with a free outlet to the steep downstream canyon reaches. Backwater effects to the dam outlet are assumed negligible due to the steep terrain, and to also assume a more conservative condition.

Based on the above assumptions, the outlet flowrate for a ponded depth H (feet) is given (for the ultimate dam-break failure geometry) by $Q_p = 2.472\ H^{2.5}$ cfs. The reservoir rating curve relating basin depth to volume is shown in Table 1.

Table 1. Orange County Reservoir Volume and Dam-Break Outflow

Depth (ft)	Volume (af)	Q_p (Dam-Break) (cfs)
2	4.6	14
4	9.5	79
6	14.8	218
8	20.3	447
10	26.4	782
12	32.7	1,233
14	39.4	1,813
16	46.5	2,530
18	54.1	3,400
20	62.0	4,420
22	70.3	5,611
24	79.2	7,000
26	88.5	8,520
28	96.2	10,300
30	108.4	12,200
32	119.1	14,300
34	130.2	16,700
36	141.9	19,200
38	154.0	22,000
40	166.7	25,000
42	179.9	28,300
44	193.8	31,800
46	211.0	35,500

For the assumed outlet hydrograph shape width Q_p occurring at 20-minutes after dam-failure, the volume drained by time 20-minutes is given by $V_d = 0.01377Q_p$ (acre-feet). The estimate of Q_p is provided by the iteration shown in Table 2.

Table 2. Estimating Dam-Break Q_p (20-Minute Peak Time)

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

Notes:

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

2: $V_d = 0.01377Q_p$AF

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

From the table, it is seen that Q_p is strongly influenced by the quick depletion of the reservoir's north of the City of Brea, California. From the study, it is concluded that the estimated flood plain is reasonable, and is based upon a modeling approach which accounts for the time-varying effects of routing, storage, and an earthen dam-failure process. The diffusion approach provides an easy-to-use two-dimensional flooding model, and develops usable estimates of flood-depths, widths, and arrival times for planning-study purposes.

REFERENCES

1. L. F. Land, "Mathematical Simulations of the Toccoa Falls, Georgia, Dam-Break Blood Waves," Wat. Resources Bulletin, (16), No. 6, 1980a.

2. L. F. Land, "Evaluation of Selected Dam-Break Flood-Wave Models by using Field Data," U.S.G.S. Wat. Res. Investigations, pp. 80-44, 1980b.

3. V. M. Ponce and A. J. Tsivoglou, "Modeling Gradual Dam Breaches," A.S.C.E., J. Hyd. Div., HY7, 1981.

4. Nikolaos Katopodes and Theodor Strelkoff, "Computing Two-Dimensional Dam-Break Flood Waves," A.S.C.E., Journal of Hydraulics Division, HY9, 1978.

5. Th. Xanthopoulos and Ch. Koutitas, "Numerical Simulation of a Two-Dimensional Flood Wave Propagation Due to Dam Failure," Journal of Hydraulic Research, 14, No. 4, 1976.

6. A. O. Akan and B. C. Yen, "Diffusion-Wave Flood Routing in Channel Networks," A.S.C.E. Hyd.Div., HY6, 1981.

7. T. V. Hromadka II et al.,"A Two-Dimensional Diffusion Dam-Break Model," Advances in Water Resources, 1984.

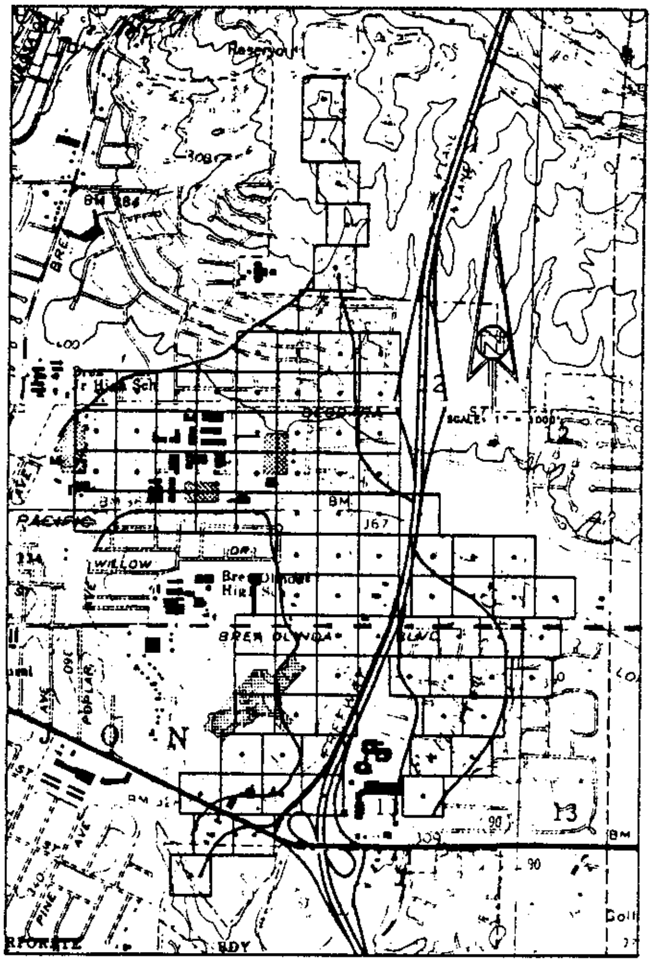

Figure 13. Flood Plain for 400 A.F. Basin Test

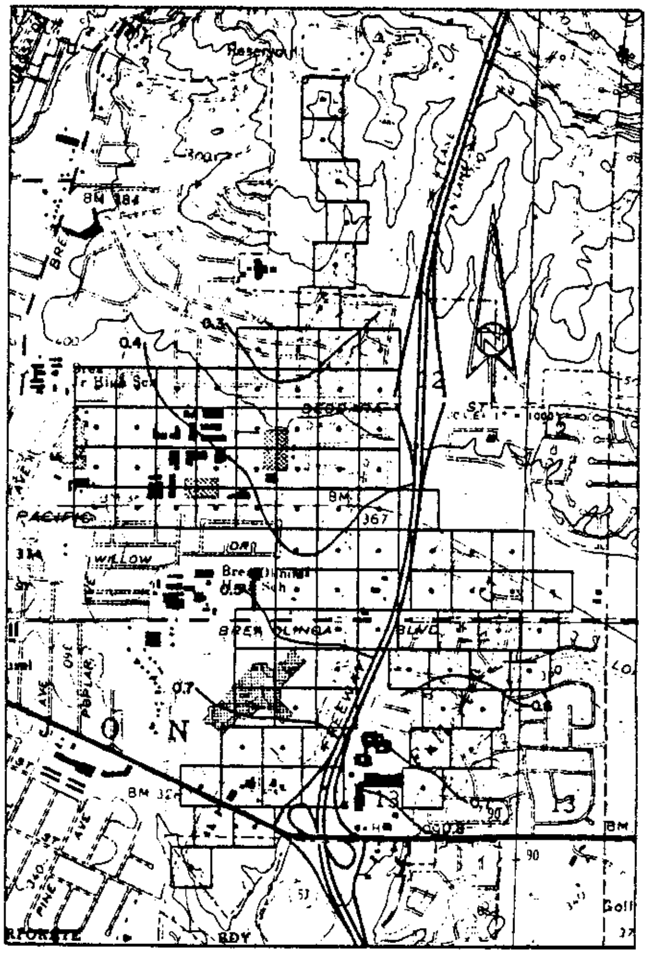

Figure 14. Maximum Flooding Depth Time, Hours (4-0 A.F. Basin Test)