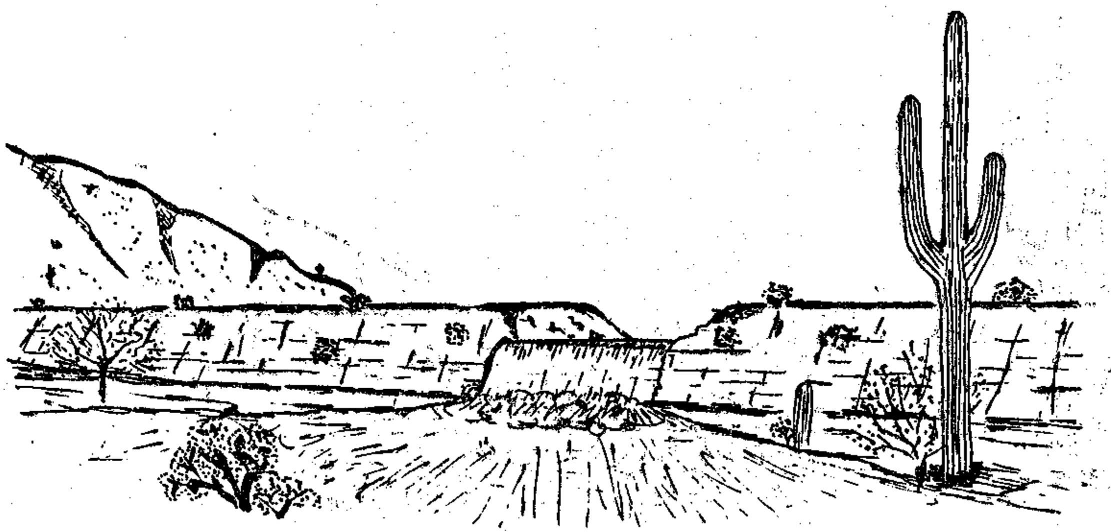
FLOOD CONTROL DISTRICT
OF
MARICOPA COUNTY
3335 WEST DURANGO STREET PHOENIX, ARIZONA 85009

SPOOK HILL FLOODWATER RETARDING STRUCTURE
FLOOD INUNDATION ANALYSIS



FCD CONTRACT NO.88-68

OCTOBER, 1990

PREPARED BY:
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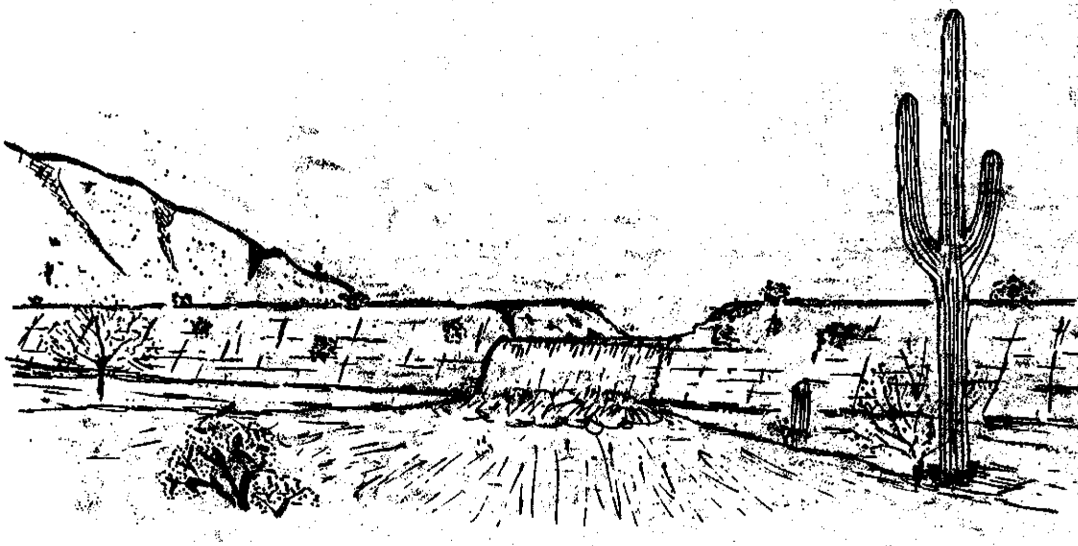

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Registered Professional Engineer (Civil)
CERTIFICATE NO. 7184
GEZA E. KMETTY
Date Signed 10-16-90
ARIZONA, U.S.A.

IN ASSOCIATION WITH:

GEORGE V. SABOL CONSULTING ENGINEERS, INC.

FLOOD INUNDATION ANALYSIS
OF THE
SPOOK HILL FRS

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SECTION I

EXECUTIVE SUMMARY

INTRODUCTION

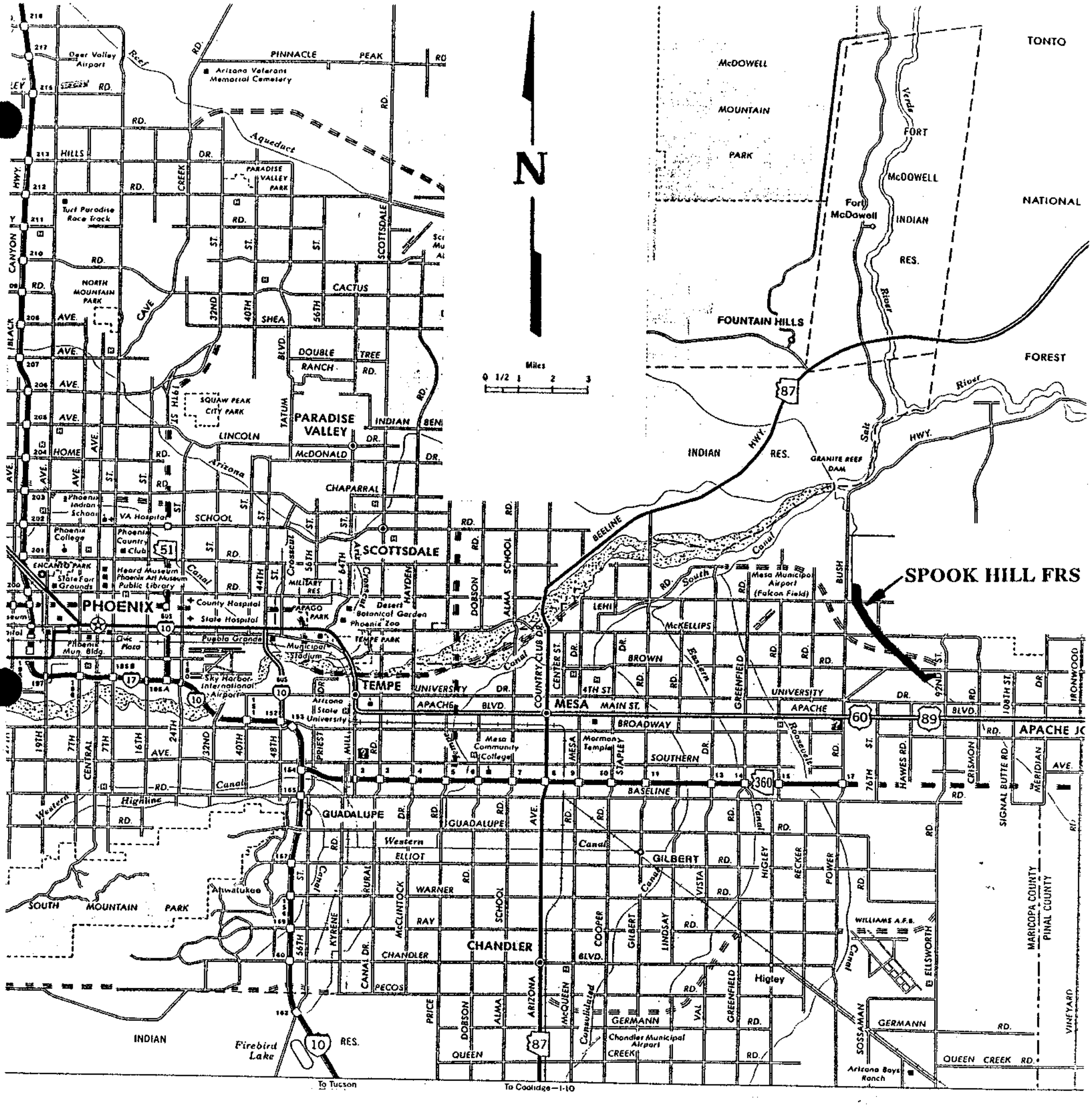
The operation of the Emergency Spillway or a breach of the dam embankment of the Spook Hill Floodwater Retarding Structure (Spook Hill FRS) would result in flood waters, possibly of severe magnitude, passing into downslope urbanizing areas of the City of Mesa, Arizona. This Flood Inundation Analysis presents the results of a study to determine the magnitude and extent of flooding downslope of the Spook Hill FRS that would occur from either the passage of the Probable Maximum Flood (PMF) through the Emergency Spillway or a breach of the earthen embankment due to piping failure.

The purpose of the Flood Inundation Analysis of the Spook Hill FRS is to complete the dam safety certification required by the Arizona Department of Water Resources (ADWR). The Flood Control District of Maricopa County (FCDMC) has legal jurisdiction over the Spook Hill FRS and is coordinating the preparation of this Flood Inundation Analysis. The Maricopa County Department of Civil Defense and Emergency Services will write an Emergency Preparedness Plan based upon the technical information supplied through the FCDMC. The basis for the Flood Inundation Analysis is summarized subsequently.

Flood Inundation Maps resulting from operation of the Emergency Spillway, and three separate piping breaches were prepared. The dam embankment is more that four miles long and it is not reasonable to estimate the flood inundation as a result of piping breach for every location along the embankment. Therefore, a Generalized Breach Inundation map was prepared for the purpose of identifying flood inundation areas and personal hazard zones for potential breach locations south of McDowell Road. Figure I-1 presents a Vicinity Map for the Spook Hill FRS. This report incorporates the findings of four previous documents issued as Progress Reports for this project.

Section II - Flood Hydrology

The Soil Conservation Service (SCS) designed and constructed the Spook Hill FRS. A review of the SCS design hydrology resulted in the need to reevaluate the inflow flood for the purpose of



VICINITY MAP

FIGURE I-1

this flood inundation study. The PMF was selected as the most critical inflow flood, and the Probable Maximum Precipitation (PMP) was determined by the procedures in Hydrometeorological Report No. 49 (Hansen and other, 1948). The alignment of the as-built Signal Butte Floodway changed from the alignment that was assumed during the design of the Spook Hill FRS and that change resulted in a reduction of drainage area from 13.57 square miles to 11.42 square miles. For the PMF, it was assumed that the Signal Butte Floodway would be contributing 2,100 cubic feet per second (cfs) into the Spook Hill FRS impoundment prior to the onset of the PMP. For that condition, the water level in the impoundment is at elevation 1,583.4 ft at the start of the PMF. These assumptions about the Signal Butte Floodway are a more severe PMF condition than was assumed by the SCS.

The HEC-1 Flood Hydrology Program was used to estimate the PMF and to route the flood through the Principal and Emergency Spillways. The PMF peak discharge is estimated as 52,150 cfs with a maximum water surface elevation of 1,590.86 ft. The PMF does not result in overtopping of the earthen embankment and therefore failure by overtopping was not deemed to be a critical flood inundation condition.

Section III - Dam Breach Analysis

Due to the length of the dam, three locations for embankment breach were selected to be representative of different downgradient flood inundation scenarios that could be expected:

1. Location A is near the embankment's northern end where the Principal Spillway conduit passes through the dam embankment.
2. Location B is near the embankment mid-length with a maximum embankment section and having extensive urbanization downslope of the dam.
3. Location C is near the embankment's southern end.

The piping breach hydrographs were estimated by application of the BOSS Breach program. Geotechnical information from the SCS dam design documentation were analyzed, and sensitivity analyses of the breach parameters were performed.

Section IV - Flood Inundation Analysis

A two dimensional computer program was used to model the PMF Emergency Spillway release and the three piping breach releases on the downslope, coalesced alluvial fan areas. The Diffusion Hydrodynamic Model (DHM), selected for the Flood Inundation Analysis, was able to model unsteady backwater effects, ponding, and channel-floodplain interfaces. Information obtained from USGS quadrangle maps, aerial photographs and field reconnaissance were input into the computer program to model the downslope topography, flood conveyance channels, and boundary conditions.

Section V - Flood Inundation Mapping

The DHM program output was analyzed and used to prepare the information appearing on the Flood Inundation Maps. These maps depict flooding resulting from the Emergency Spillway operation during the PMF and the piping breaches at Locations A, B and C. Each map set presents the magnitude and extent of flood inundation using four descriptors:

1. Arrival Times
2. Personal Hazard Zones
3. Maximum Depth Contours
4. Maximum Velocity Contours.

The Arrival Times map presents the flood wave advancement in one-hour increments. The Personal Hazard Zones Map indicates areas where evacuation would be hampered by flow depths greater that 2 ft or by a combination of flow depth (ft) and flow velocity (ft/sec) having a product greater than 7. The Maximum Depth Contours Map shows the greatest flow depth that occurs at any point in time. The maximum velocity that occurs at any point in time is presented on the Maximum Velocity Contours Map.

The Flood Inundation Maps locate Social and Economic Impact Areas, which are structures that should be evacuated first, especially those within a Personal Hazard Zone, or closely monitored during a flood event. These structures comprise five categories:

1. Hospitals
2. Emergency Medical Centers
3. Public and Private Schools
4. Nursing Homes
5. Major Shopping Centers

A brief description of the extent and magnitude of flood inundation is presented in this section of the report. A Generalized Breach Inundation Map was prepared for a potential piping breach that could occur at any location south of McDowell Road.

Appendix A contains the PMP computations from HMR-49. Appendix B - Technical Appendix, contains a hard copy of the computer output for this project, issued as a separate volume.

SECTION II

FLOOD HYDROLOGY

REVIEW OF SCS DESIGN HYDROLOGY

The flood hydrology for the design of the dam that was developed by the Soil Conservation Service (SCS) was reviewed. Specifically, that review was to determine if the flood hydrology, as computed by the SCS, was adequate for the intent of this dam break and flood inundation study. It was agreed, in discussion with FCDMC and the Arizona Department of Water Resources (ADWR), that the Probable Maximum Flood (PMF) should be used as the inflow flood for this study.

Design reports and copies of computer output were obtained from the SCS and these were reviewed. That information provided a summary of hydrologic input data for the SCS flood hydrology study. In addition to the reports that were obtained and reviewed, Mr. Harry Millsaps of the SCS Phoenix office provided an explanation of the SCS flood hydrology for this dam and he resolved some discrepancies in the information provided in the design report. The SCS produced two analyses of the flood hydrology for the purpose of dam and spillway design. The Spook Hill FRS has both an Emergency Spillway and a Principal Spillway. One of the SCS analyses was done with the aid of the TR-20 program. In the TR-20 model, the watershed (Figure II-1, Rear Folder) was divided into nine subbasins (Numbers 14 through 22) with individual subbasin input as shown in Table II-1. In that watershed delineation it was assumed that all runoff from subbasins 14 through 17 was diverted to the Spook Hill FRS by the Signal Butte Floodway. That assumption may have been based on a preliminary design concept for the floodway in regard to discharge capacity, channel freeboard, or berms along the floodway. The second analysis was performed with a separate SCS program that was used for spillway sizing, and in that analysis the watershed was modeled as a single basin. The TR-20 subbasin model of the watershed resulted in a peak discharge estimate of 47,315 cfs while the other, single basin model resulted in a peak discharge of 38,045 cfs. The SCS subsequently used the results of the single basin model analysis to size the spillway and to set the dam crest elevation. The input to these SCS watershed models is described. The majority of the SCS watershed model input was accepted. However, some of the SCS model input was not accepted for a PMF analysis because the existing conditions of the watershed are different from those that were assumed by the SCS at the time of design, or because current flood hydrology standards have changed since the SCS design hydrology was performed.

TABLE II-1

SCS FLOOD HYDROLOGY WATERSHED DATA USED IN TR-20

Subbasin No.	Area	Length	Slope	Velocity	T_c	TLAG[a] (HEC-1)	CN
	sq.mi.	feet	%	ft/sec	hours	hr	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
14	.25[b]	5,500	1.3	4.0	0.38	.23	75
15	.68[b]	5,500	1.0	4.0	0.38	.23	75
16	1.22[b]	12,200	2.0	6.0	0.56	.34	75
17	1.44	14,300	2.4	6.0	0.66	.40	75
18	.92	13,000	2.1	6.0	0.60	.36	75
19	4.08	25,800	3.6	6.0	1.19	.71	76
20	1.28	15,800	2.8	6.0	0.73	.44	75
21	1.91	18,000	3.8	6.0	0.83	.50	76
22	1.79	14,500	3.0	6.0	0.67	.40	79

13.57 Total Area

11.42 Contributing Area

a - TLAG = $0.6T_c$ used in HEC-1 model

b - noncontributing drainage area

The review of the SCS hydrology resulted in the recommendation that the following deviations from the SCS hydrology be used for estimating the PMF:

1. The rainfall time distribution should be from Hydrometeorological Report No. 49 (HMR-49) (Hansen and others, 1984).

2. PMF discharges from subbasins 14 through 16 cannot be conveyed to the Spook Hill FRS by the Signal Butte Floodway with the floodway as constructed. This results in a reduction of the effective area for the PMF that can drain directly into the Spook Hill FRS impoundment.

3. The initial condition for the PMF should include a discharge of 2,100 cfs (maximum capacity) from the Signal Butte Floodway to the Spook Hill FRS. The discharge would be for a period long enough to raise the impoundment water surface elevation so that the Spook Hill FRS spillways would have a combined outflow of 2,100 cfs. This is an equilibrium condition that could exist from runoff generated from upland watersheds that contribute to the Signal Butte Floodway from rainfall prior to the Probable Maximum Precipitation (PMP). The water surface elevation in the Spook Hill FRS would be 1,583.4 ft for that condition. The discharge capacity and alignment of the Signal Butte Floodway is discussed below.

   The revised PMF analysis was performed with the 1988 version of the HEC-1 flood hydrology program (U.S. Army Corps of Engineers, 1987). The HEC-1 program was chosen because it has more modeling capabilities than the SCS TR-20 model and it can be used to duplicate TR-20 results.

DESCRIPTION OF WATERSHED

The watershed consists of mainly undeveloped land with native vegetation typical for this semi-arid area. There is some low density residential development in the watershed, although this should have no measurable affect on the flood hydrology because the extent of urbanization is small and there is no directly connected impervious area. However, if in the future the contributing watershed is developed, then the current flood hydrology may be inappropriate, and would need to be reevaluated based on the conditions that would exist at that time.

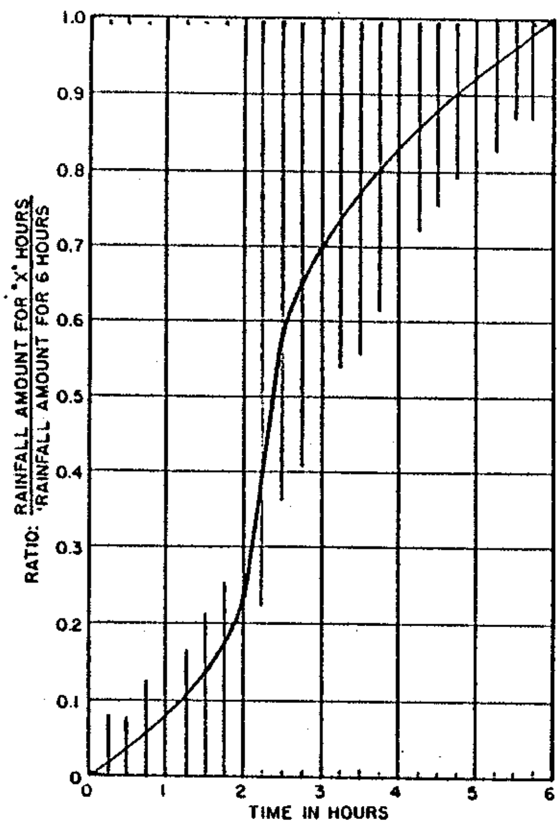
A sketch of the watershed contained in the SCS documentation was used to prepare the watershed map shown in Figure II-1 (in rear folder). The SCS used a total drainage area of 13.57 square miles (subbasins 14 through 22) as the contributing watershed area to the dam. The Signal Butte Floodway was constructed after the Spook Hill FRS and the floodway does not have the alignment as shown in the SCS design reports. The change in alignment resulted in a diminished drainage area (Figure II-1 shows the approximate location of the floodway alignment). More importantly, the floodway has a capacity for only 2,100 cfs. Discharge in excess of 2,100 cfs will overtop the floodway and will not enter the Spook Hill FRS. Therefore, subbasins 14 through 16 were not included in the drainage area for the PMF analysis, and an inflow of 2,100 cfs from the Signal Butte Floodway was added to the PMF analysis. The SCS flood hydrology was verified using a HEC-1 model with a contributing area of 13.57 square miles (subbasins 14 through 22) without other contributing flow from the Signal Butte Floodway, but the PMF for this study was estimated with direct runoff from 11.42 square miles (subbasins 17 through 22) plus a 2,100 cfs inflow from Signal Butte Floodway as described above.

The SCS report of geologic investigations at the dam site describes the soils as primarily silty sand and sandy silt. There are a few, rather isolated, hills of very highly weathered granite present in the watershed. These areas of rock outcrop are not directly connected to the impoundment area behind the dam and probably would have little effect on the runoff to the dam.

RAINFALL

Documentation could not be obtained on the development of the SCS design rainfall. However, the copies of the TR-20 output indicate that 13.0 inches of rainfall was applied according to the SCS 6-hour emergency spillway and freeboard volume adjustments and storm distribution (SCS, 1985a). The SCS distribution is shown in Figure II-2.

The Probable Maximum Precipitation (PMP) was calculated according to the procedures in HMR-49. Based on those calculations (Appendix A), the 6-hour local storm PMP is 12.8 inches. The general storm is of much lower rainfall intensity than the local storm and the local storm is the critical design storm for this dam.



(C.) SIX HOUR DESIGN STORM DISTRIBUTION

FIGURE 2-6

EMERGENCY SPILLWAY AND FREEBOARD

VOLUME ADJUSTMENTS AND STORM DISTRIBUTION

FOR AREAS WHERE NWS REFERENCES DO NOT CONTAIN AN APPLICABLE PROCEDURE

SCS rainfall distribution

(Ref. - Earth Dams and Reservoirs, SCS Technical Release No. 60, Oct. 1985)

FIGURE II-2

The calculated 12.8 inch local storm PMP agrees with the SCS design rainfall of 13.0 inches in depth, however, the SCS rainfall distribution and the HMR-49 distribution are significantly different. As shown in Table II-2, the HMR-49 rainfall is much more intense than the SCS rainfall and this will result in a larger peak discharge at the dam than the SCS inflow design flood. The HEC-1 model of the watershed was run with the SCS rainfall distribution for verification of the SCS design hydrology and with the HMR-49 rainfall distribution to develop the PMF for this study.

TABLE II-2

LOCAL STORM PMP RAINFALL FOR BOTH THE SCS DISTRIBUTION AND THE HMR-49 DISTRIBUTION

Time	Incremental Rainfall, in inches	
hours	SCS Distribution	HMR-49 Distribution
(1)	(2)	(3)
0.00	0.00	0.00
0.25	0.03	0.10
0.50	0.42	0.10
0.75	0.30	0.10
1.00	0.21	0.10
1.25	0.45	0.20
1.50	0.39	0.20
1.75	0.45	0.20
2.00	0.81	0.20
2.25	2.10	5.80
2.50	2.56	1.80
2.75	0.78	0.70
3.00	0.60	0.70
3.25	0.51	0.40
3.50	0.45	0.40
3.75	0.36	0.40
4.00	0.33	0.40
4.25	0.39	0.15
4.50	0.33	0.15
4.75	0.30	0.15
5.00	0.27	0.15
5.25	0.30	0.10
5.50	0.18	0.10
5.75	0.21	0.10
6.00	0.27	0.10
Totals	13.00	12.80

RAINFALL LOSSES

The SCS used the Curve Number (CN) method to estimate rainfall losses. The CN for each subbasin is listed in Table II-1. Supporting information on the estimate of CN was not obtained from the SCS. The selection of Curve Numbers appears reasonable for this watershed, and the CN values from the SCS have been accepted.

UNIT HYDROGRAPHS

The SCS used the SCS dimensionless unit hydrograph for each subbasin. The times of concentration (T_c) that were input into the TR-20 model by the SCS are listed in Table II-1. These times of concentration seem reasonable for the watershed slopes and assumed overland velocities, and were accepted for use in this Flood Inundation Analysis. HEC-1 requires the input of lag time rather than time of concentration. The lag time (TLAG) was estimated as $0.6T_c$ which is consistent with the computation that is performed in the execution of the TR-20 model. TLAG for each subbasin is listed in Table II-1.

SPILLWAY CAPACITY RATING TABLE

The discharge capacities of the Principal Spillway and the Emergency Spillway are shown on spillway capacity rating curves in the SCS design drawings. Those curves were digitized and the results are shown in Table II-3. The as-built elevations of the Principal and Emergency Spillways were verified in conjunction with the centerline profile survey. Figure II-3 presents the spillway elevations and centerline profile for the Spook Hill FRS. The Principal Spillway provides very little discharge capacity in relation to the capacity of the Emergency Spillway. The Emergency Spillway rating curve was checked and the SCS rating curves for both spillways were accepted.

IMPOUNDMENT ELEVATION - VOLUME DATA

The impoundment elevation-volume data was taken from a table in the SCS design notes, and these are shown in Table II-4. The volume was extrapolated to Elevation 1592.0 ft so that embankment overtopping could be modeled. The SCS elevation-volume data were accepted.

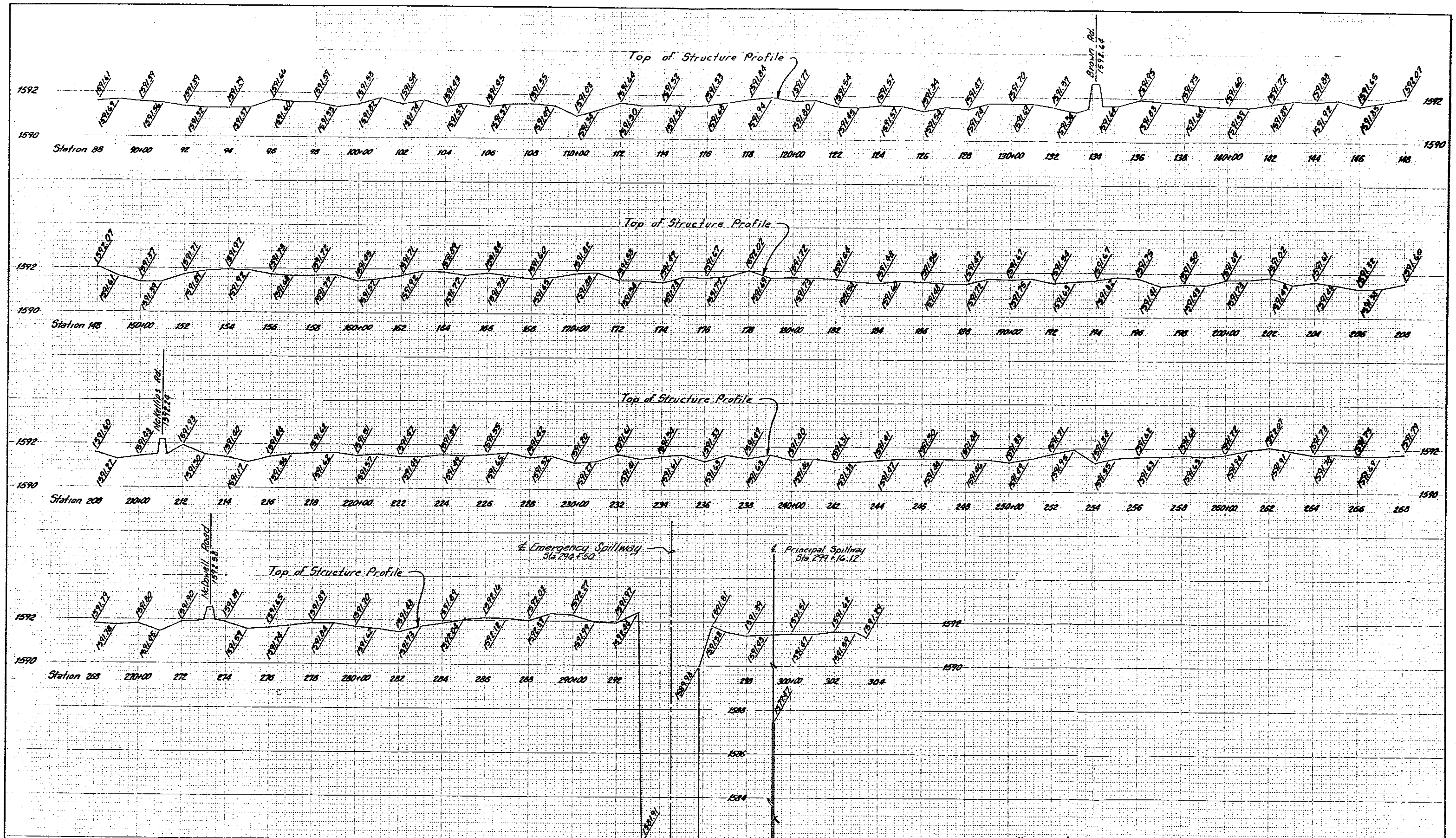
TABLE II-3

SPILLWAY RATING TABLE

Water Surface Elevation, ft (1)	Spillway Discharges, cfs Principal (2)	Emergency[a] (3)	Combined (4)
1,577.5[b]	0	0	0
1,579.01	460	0	460
1,580.0	690	0	690
1,581.0	810	0	810
1,582.0[c]	850	0	850
1,582.5	865	260	1,125
1,583.0	880	740	1,620
1,583.5	895	1,360	2,255
1,584.0	910	2,100	3,010
1,585.0	940	3,850	4,790
1,586.0	970	5,930	6,900
1,587.0	1,000	8,280	9,280
1,588.0	1,030	10,890	11,920
1,589.0	1,055	13,720	14,775
1,590.0	1,080	16,770	17,850
1,591.0	1,105	20,010	21,115
1,592.0	1,130	23,430	24,560

[a] Q = CLH**3/2, where L = 260 ft. and C = 2.85 was selected to agree with the SCS spillway capacity curve.

[b] - Principal Spillway Crest Elevation
[c] - Emergency Spillway Crest Elevation



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

DRAWN BY: R.L.H.
DESIGN BY:
CHECK BY:

Date: OCT., 1990

Project No. 89-407.01

SPOOK HILL DAM
CENTERLINE PROFILE

MKE McLAUGHLIN KMETTY ENGINEERS, LTD.
3030 N. Central Ave., Suite 402 Phoenix, AZ 85012 (602) 248-7702
Phoenix • Bullhead City • Flagstaff

FIGURE II-3

TABLE II-4

RESERVOIR ELEVATION - VOLUME TABLE
(Volumes are estimates after reservoir sedimentation has occurred.)

Water Surface Elevation ft (1)	Storage Volume acre-feet (2)
1,577.5[a]	0.0
1,578.0	149.4
1,580.0	466.4
1,582.0[b]	901.6
1,584.0	1,432.7
1,586.0	2,059.8
1,588.0	2,784.0
1,590.0	3,610.3
1,591.0[c]	4,063.4
1,592.0	4,550.0

[a] - Principal Spillway Crest Elevation
[b] - Emergency Spillway Crest Elevation
[c] - Dam Crest Elevation (without roadway crown of 0.5 ft.)

RESERVOIR INITIAL CONDITIONS

It is likely that the SCS flood hydrology was performed with no antecedent flow into the impoundment as the initial condition, that is, the impoundment was completely empty (dry). The existing condition at the dam is such that the Signal Butte Floodway outfalls directly into the impoundment at the left abutment of the dam. Signal Butte Floodway receives runoff from the Apache Junction Dam and Floodway, the Bulldog Floodway, the Signal Butte Dam, and the Pass Mountain Diversion. During the occurrence of a severe rainfall such as the local storm PMP, it would be reasonable to assume that the Signal Butte Floodway will receive runoff from the upstream structures for a period long enough to fill part of the Spook Hill FRS storage volume and the spillways will operate such that all inflow from the Signal Butte Floodway is passed through the spillways (inflow = outflow). The Signal Butte Floodway has a capacity of 2,100 cfs. At that rate of outflow from the spillways (2,100 cfs), the impoundment water level would be at Elevation 1,583.4 ft. It is assumed that discharge in excess of 2,100 cfs from upstream runoff would not be conveyed to the dam through the floodway. Therefore, the entire PMF runoff from Subbasins 14 through 16 would not drain to the Spook Hill FRS as previously discussed.

RESULTS OF FLOOD STUDY

Three cases for flood hydrology, as described and summarized in Table II-5, were modeled with the HEC-1 program. The results of the HEC-1 modeling along with the results of the two SCS flood hydrology analyses are shown in Table II-6.

Case A is a HEC-1 reproduction of the SCS TR-20 model and, in general, the HEC-1 model reproduces the TR-20 results except that the HEC-1 peak discharge (45,440 cfs) is slightly lower than the TR-20 peak discharge (47, 315 cfs).

Case B is the same as Case A except that the rainfall input is according to HMR-49. The peak discharges are significantly greater using the HMR-49 rainfall distribution as compared to the SCS rainfall distribution. The dam embankment is not overtopped for this condition.

TABLE II-5

FLOOD HYDROLOGY CONDITIONS THAT WERE ASSUMED[a]

Case (1)	Rainfall Depth inches (2)	Rainfall Distribution (3)	Watershed Area square miles (4)	Initial Condition of Impoundment (5)
A	13.0[b]	SCS	13.57	Dry
B	12.8[c]	HMR-49	13.57	Dry
C	12.8	HMR-49	11.42	2,100 cfs inflow[d] W.S. Elev. = 1,583.4 ft

a - CN and T_c from the SCS TR-20 model as shown in Table II - 1

b - SCS design rainfall

c - Local Storm PMP from HMR-49

d - from Signal Butte Floodway inflow

TABLE II-6

RESULTS OF FLOOD HYDROLOGY REVIEW USING HEC-1 AND COMPARISON WITH THE SCS TR-20 RESULTS

	SCS Flood Hydrology Analyses		HEC-1		
	Design (Single Basin) (1)	SCS TR-20 (2)	Case A (3)	Case B (4)	Case C (5)
Area, square miles	13.57	13.57	13.57	13.57	11.42
Rainfall, inches	13.00	13.0	13.0	12.8	12.8
Runoff, inches	10.28	9.84	9.67	9.58	9.68
Inflow volume, acre-feet	7,438	7,122	7,001	6,935	6,987[a]
Peak Discharge, cfs	38,045	47,315	45,440	61,010	52,150
Time to Peak, hours	3.00	3.26	2.75	2.67	2.75
Peak Spill, cfs					
Emergency Spillway	[b]	[c]	15,280	18,130	19,560
Principal Spillway	[b]	[c]	1,070	1,090	1,100
Embankment Overtopping	[b]	[c]	0	0	0
Combined	18,340[b]	[c]	16,350	19,220	20,660
Time to Peak Spill, hours	4.25	[c]	4.00	3.75	3.67
Max. Water Surface Elev., feet	1,589.69	[c]	1,589.50	1,589.37	1,590.86

a - Includes Signal Butte Floodway inflow

b - Spillway rating deviates somewhat from that shown in Table II - 3

c - Routing was for a different spillway configuration than was finally used

Case C is for the portion of the watershed (subbasins 17 through 22) that will drain directly to the impoundment behind the dam. This results in a drainage area of 11.42 square miles as compared to the 13.57 square miles that was used in the SCS flood hydrology analyses. However, for this case it was assumed that the Signal Butte Floodway was passing 2,100 cfs into the impoundment and that 2,100 cfs was passing through the spillways. For that condition the water surface elevation is 1,583.4 ft. The rainfall input was the same as Case B, that is, rainfall time distribution by HMR-49. The remainder of the input to the HEC-1 model is the same as for the SCS's TR-20 model for subbasins 17 through 22.

Case C represents a reasonable set of assumptions and model input for a PMF analysis of the watershed. The peak discharge is 52,150 cfs with a maximum reservoir water surface elevation of 1,590.86 ft. For Case C the combined spillway peak discharge is 20,660 cfs; 19,560 cfs through the Emergency Spillway and 1,100 cfs through the Principal Spillway. A copy of the HEC-1 output file for Case C (final PMF analysis) is contained in Appendix B (Separate Volume).

CONCLUSIONS OF FLOOD STUDY

1. The SCS flood hydrology and spillway routing was reviewed. The SCS TR-20 model input of the multibasin watershed was accepted. The SCS design storm rainfall time distribution is not considered adequate for a PMF and the HMR-49 rainfall should be used.

2. The HMR-49 time distribution is more intense that the SCS distribution.

3. The SCS hydrology includes area that drains to the Signal Butte floodway. That area (2.15 square miles) should not be included in a PMF model.

4. The peak discharge for the total area (13.57 square miles) and all of the model input as used by the SCS is estimated as 45,440 cfs using the HEC-1 program as compared to 47,315 cfs estimated by the SCS using the TR-20 program.

5. During a severe storm, such as a PMP, it is reasonable to assume that the Signal Butte Floodway contributes 2,100 cfs to the Spook Hill FRS impoundment and the water surface is at Elevation 1,583.4 ft at the start of the storm. This condition should be assumed for the PMF.

6. The PMF for the Spook Hill FRS is the result of the local storm PMP according to HMR-49, over 11.42 square miles plus the condition of Signal Butte Floodway inflow as described in Conclusion 5.

7. The maximum Emergency Spillway discharge during a PMF is 19,560 cfs. The PMF does not result in overtopping of the dam embankment.

EMERGENCY SPILLWAY PMF HYDROGRAPH

The SCS design flood hydrology may be a reasonable criteria for design for this dam, but it does not constitute a PMF because the rainfall intensities are not as severe as those estimated by HMR-49. Also, inflow from the Signal Butte Floodway was not included in the SCS flood hydrology. Therefore, the SCS hydrology should not be used to estimate the maximum Emergency Spillway release for this dam. Case C, as described in previous discussion, provides the greatest Emergency Spillway release and that condition provides the most serious flood inundation hazard from spillway release. The PMF, as described in this report, does not result in embankment overtopping for the Spook Hill FRS and therefore dam breach by overtopping should not be considered for a dam break flood inundation study for the Spook Hill FRS.

The Emergency Spillway, shown in a photograph in Figure II-4, is only a few hundred feet upslope from the Central Arizona Project (CAP) Canal. Discharge from the Emergency Spillway will enter a short chute below the spillway, flow perpendicular to the CAP Canal, enter the canal, and then (in essence) exit the canal over the opposite canal bank and flow downslope. Notice in the photographs of Figure II-5 that there is a concrete apron on the upslope side of the CAP Canal and that the earthen freeboard is lowered on the downslope side. Those features will aid in conveying the spillway discharges across the CAP Canal without breaking the canal. Furthermore, it is anticipated that a severe storm such as a PMP would result in operational control of the CAP Canal by the U.S. Bureau of Reclamation. The Salt-Gila Pumping Plant on the CAP Canal is about 2.1 miles upstream of the Emergency Spillway crossing, and it is reasonable to assume that that pumping plant would be shut down during such a severe storm and that the CAP Canal would not contribute to additional flood discharges beyond the discharge from the Emergency Spillway.

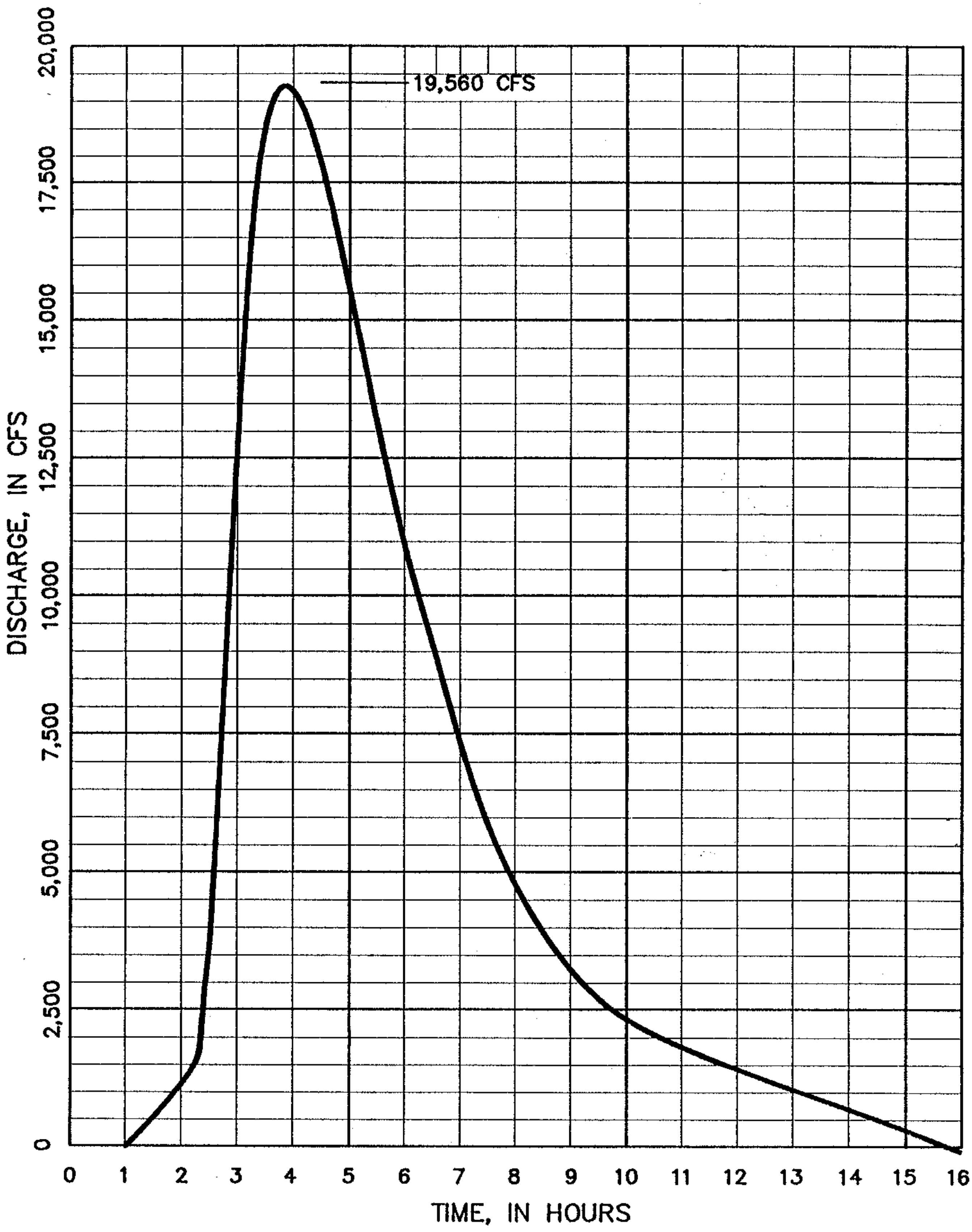
EMERGENCY SPILLWAY.
(FLOW IS FROM LEFT TO RIGHT)

FIGURE II-4

CAP CANAL DOWNSLOPE OF THE EMERGENCY SPILLWAY CHUTE. NOTE THE CONCRETE APRON UPSLOPE FROM THE CANAL AND THE DEPRESSED EARTHEN FREEBOARD ON THE DOWNSLOPE SIDE.

FIGURE II-5

For the PMF it was assumed that 2,100 cfs would be entering the impoundment from the Signal Butte Floodway prior to the onset of the PMF. For that condition, about 900 cfs would be passing through the Principal Spillway and about 1,200 cfs would be passing through the Emergency Spillway. It would not be reasonable to assume an excessively long period of 1,200 cfs release from the Emergency Spillway before the onset of the PMF, or to have a long 1,200 cfs tail after the PMF. Therefore, the Emergency Spillway release, as shown in Figure II-6, was modified on both the rising and recession limbs of the hydrograph to eliminate the effect of the sustained flow of the Signal Butte Floodway that was built into the HEC-1 model.



EMERGENCY SPILLWAY HYDROGRAPH

FIGURE II–6

SECTION III

DAM BREACH ANALYSIS

LOCATIONS OF PIPING BREACHES

Piping is the progressive erosion of leaks which develop over time through an earthen embankment or under the dam. Leakage that emerges from the downstream face of the embankment or through the foundation which continually increases may be indicative of the initiation of piping. Piping, if not controlled, can lead to erosion of a "conduit" through the embankment or foundation and subsequent breach of the dam. It is not possible to predict piping locations, however two factors can be considered when anticipating potential piping locations. First, piping potential is the greatest at locations with maximum hydraulic head. This is because forces that initiate seepage are the greatest with maximum hydraulic head and the potential exists to produce the greatest seepage velocities. Second, the potential for piping is increased along outlet pipes and other hydraulic structures that pass through the embankment. This is because of the difficulty in compacting the earthen fill along the sides of such structures. In general, piping can be initiated at any location that is conducive to seepage due to structural deficiencies. Burrowing animals can contribute to causing such deficiencies.

Since one of the objectives of this study is to determine the downslope flood potential from a breach in the dam, the location of piping breaches was selected by considering both the location of likely piping breaches and also the downslope consequences of such breaches. The centerline profile of the dam was surveyed (see Figure II-3) in order to determine if any settlement had occurred, which could affect the maximum hydraulic head on the embankment. The maximum measured variance from the design dam crest (elevation 1,591.50 ft) is about 0.5 ft at Station 110+00 (elevation 1,591.03 ft) and this is not significant, nor does the maximum reservoir water surface elevation for the PMF (elevation 1,590.86 ft) result in overtopping of the embankment at this low point.

Three critical locations for piping breach are identified as A, B, and C, as shown in Figure II-1. Location A is at the Principal Spillway where the outlet conduit passes through the embankment is a location where there is a greater potential for a piping breach. A photograph of the Principal Spillway inlet is shown in Figure III-1. The toe of the dam is at elevation 1,567 ft and this is a maximum embankment section resulting in maximum hydraulic head on the embankment. This location also defines the most northern extent of flooding from a breach.

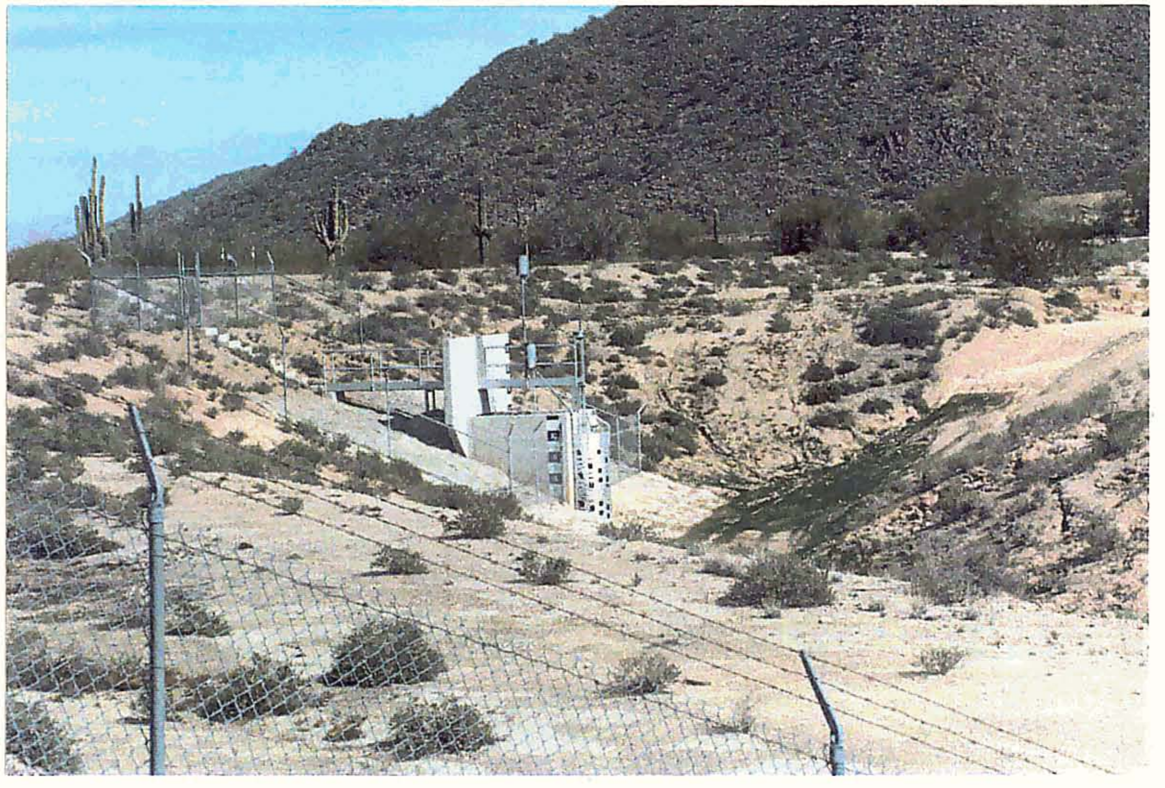


FIGURE III-1

PRINCIPAL SPILLWAY WITH OUTLET
CONDUIT THROUGH THE EMBANKMENT

Location B is just north of McKellips Road where the toe of the dam is about elevation 1,567 ft and is another maximum embankment section. It is also noted that two waterlines have been placed through the dam near McKellips Road that required excavation and backfill through the embankment and foundation. Although there is no reason to doubt the integrity of the dam at this location, however, the increased potential for a seepage path to develop does exist. Breach release waters would pass into a highly urbanized and relatively flat downstream area that is significantly different topographically than the area downstream of Location A.

Location C is near the southern end of the embankment, and breach release waters would flow in a generally southwesterly direction from such a breach. The toe of the dam at this location is about elevation 1,570 ft and the hydraulic head on the embankment is less than at Locations A and B.

ANTECEDENT CONDITIONS FOR PIPING BREACHES

The Spook Hill FRS is normally dry and will impound water only for relatively short durations during and after precipitation events in the contributing watersheds. For a piping breach to occur, impounded water must be maintained at a sufficient elevation for a duration long enough to initiate a flow path through a structural deficiency in the embankment, resulting in the formation of a flow path and piping breach. For this condition to occur, it is assumed that the water surface elevation is at or near the elevation of the Emergency Spillway (1,582.0 ft). The Principal Spillway (crest elevation 1,577.5 ft) would be operating at about 850 cfs at water surface elevation 1,582.0 ft, and therefore it is assumed that a sustained inflow to the Spook Hill FRS of 850 cfs occurs. An inflow hydrograph is also required for the execution of the BOSS DamBrk and BOSS Breach programs (used for analysis) and, therefore, this inflow is both a practical and analytically necessary condition.

METHOD OF BREACH ANALYSIS

The piping breach hydrographs at the three locations were estimated by use of the BOSS Breach program (BOSS Corporation, 1988). The BOSS DamBrk (BOSS Corporation, 1989) program was used to independently check the results from the BOSS Breach program. Both of these programs are commercially available. The BOSS DamBrk program is an enhanced version of the 1988 NWS Breach program and the BOSS Breach program is an enhanced version of the 1984 NWS DamBrk program. The analytic procedures to estimate dam breach hydrographs in these programs are different; the NWS Breach program estimates the breach based on the geometric characteristics of the embankment and the physical properties of the embankment fill materials, and the NWS DamBrk program estimates the breach based on parametric methods that are dependent mainly on the assumed time to failure and breach width.

Sensitivity analyses were performed to select the necessary input parameters. The results of the sensitivity analyses and the piping breach hydrographs are presented. In addition, the results of the analytic estimation of the piping breach was compared to the recent (1982) and well documented piping breach failure of the Lawn Lake Dam in Rocky Mountain National Park, Colorado.

The necessary characteristics of the Spook Hill FRS embankment and spillways for the piping breach analyses are shown in Table III-1.

TABLE III-1

SPOOK HILL FRS
EMBANKMENT AND SPILLWAY CHARACTERISTICS AND IMPOUNDMENT INITIAL CONDITIONS FOR PIPING BREACH

Top of Dam Elevation, ft	1,591
Bottom of Dam Elevation, ft	
at Breach Locations A and B	1,567
at Breach Location C	1,570
Principal Spillway Crest Elevation, ft	1,577.5
Dam Crest Length, ft	20,000
Dam Crest width, ft	14
Upstream Face Slope	1V:3H
Downstream Face Slope	1V:2H
Average Slopes of Inner Core	1V:1H
Initial Impoundment Water Surface Elevation, ft	1,582
Storage Volume at Initial Impoundment, ac-ft	901.6
Downstream Channel Slope, ft/mi	110

There is a certain amount of uncertainty in the values of the physical properties of the earthen materials that were used to construct the embankment. The expected range and best estimate values of the physical properties are shown in Table III-2.

TABLE III-2

PHYSICAL PROPERTIES OF SPOOK HILL FRS EMBANKMENT MATERIAL

Core and Outer Zones

Mechanical Properties	Range	Best Estimate
D_{50}, mm	0.5 - 5	2
Ratio of D_{90} to D_{30}	10 - 200	120
Porosity Ratio	0.3	0.3
Unit Weight, lb/cu. ft.	120	120
Internal Friction Angle, degrees	27 - 35	33
Cohesive Strength	0	0
Hydraulic Property		
Manning Roughness Coefficient, "n"	0.015	0.015

Outer Core Sediment Transport Parameters

	Range	Best Estimate
Average Plastic Index for Cohesive Soils, %	0 - 15	0
b′, Clay Critical Shear Stress Coefficient	0.004 - 0.02	0.01
c′, Clay Critical Shear Stress Coefficient	0.58 - 0.90	0.70

Downstream Face Mechanical Properties

	Range	Best Estimate
D_{50}, mm	0.5 - 5	2
Ratio of D_{90} to D_{30}	10 - 200	120

The best estimate value is not necessarily the most likely value but rather is a reasonable value that could be expected to occur that would result in the potential for a larger, more rapid piping breach. There are certain breach parameters that must be assumed in the execution of an analytic piping breach estimation. The expected range and best estimate values of the assumed piping breach parameters are shown in Table III-3.

TABLE III-3

ASSUMED PIPING BREACH PARAMETERS FOR THE SPOOK HILL FRS

	Range	Best Estimate
Ratio of Erosion Pipe Width to Flow Depth	1	1
Initial Piping Breach Width, ft	0.1 - 1	0.1
Initial Piping Breach Elevation, ft		
at Breach Locations A and B	1,582 - 1,567	1,567
at Breach Location C	1,582 - 1,570	1,570
Maximum Allowable Breach Bottom Width, ft	20 - 100	50
Simulation Duration, hrs	3 - 10	none
Time Step, hr	.001 - .10	none

SENSITIVITY ANALYSES

Sensitivity analyses were performed for the piping breach at Locations A and B. The embankment section and assumed properties are the same at these two locations. One of the most sensitive assumptions is the elevation at which the piping breach is to be initiated. It is known that a lower elevation for the initial piping outfall results in more rapid and larger peak discharges. Therefore, it was assumed that piping breach would initiate at the elevation of the downstream toe of the embankment and only minimal sensitivity analysis was performed of this assumption. In the BOSS Breach program, three input parameters were investigated in regard to sensitivity, namely, mean grain size (D_{50}), a descriptor of grain size gradation (D_{90}/D_{30}), and the internal friction angle in degrees. The cohesive strength of 0.0 is a conservative, but usual assumption. The results of the BOSS Breach program sensitivity analysis are shown in Table III-4.

The most sensitive parameter (assumption) is the initial piping elevation. Mean grain size is only moderately sensitive for this embankment and the ratio D_{90}/D_{30} is somewhat sensitive although an extreme range (10 to 200) was used. Internal friction angle is not significantly sensitive over the relatively small range of this property.

A parametric sensitivity analysis was also performed using the BOSS DamBrk program. The sensitivity of breach bottom width from 25 ft to 100 ft, and time to complete breach from 2 hours to 4 hours were investigated. The results are shown in Table III-5 and these are displayed in Figure III-2. As expected, shorter breach times and wider breach openings result in greater peak discharges.

TABLE III-4

SENSITIVITY OF EMBANKMENT PROPERTIES IN THE BREACH PROGRAM

Input Parameters				Results			
Initial Piping Elevation ft (1)	D_{50} mm (2)	D_{90}/D_{30} degrees (3)	Internal Friction Angle ft (4)	Breach Width Top ft (5)	Breach Width Bottom ft (6)	Breach Peak Discharge cfs (7)	Time to Peak hrs (8)
1,572	2	50	33	109	52	7,234	1.91
1,572	5	50	33	108	52	7,073	1.72
1,572	0.5	50	33	110	53	7,336	1.56
1,572	2	200	33	127	70	8,952	1.67
1,572	2	10	33	94	37	5,767	2.15
1,572	2	50	27	114	45	6,824	1.82
1,572	2	50	35	109	55	7,372	1.93
1,567	2	50	33	84	31	9,204	1.06
1,567	2	200	35	102	48	10,581	0.99
1,567[a]	2[a]	120[a]	33[a]	96	39	9,964	1.06

[a] - Input parameters that were accepted for the final breach analysis.

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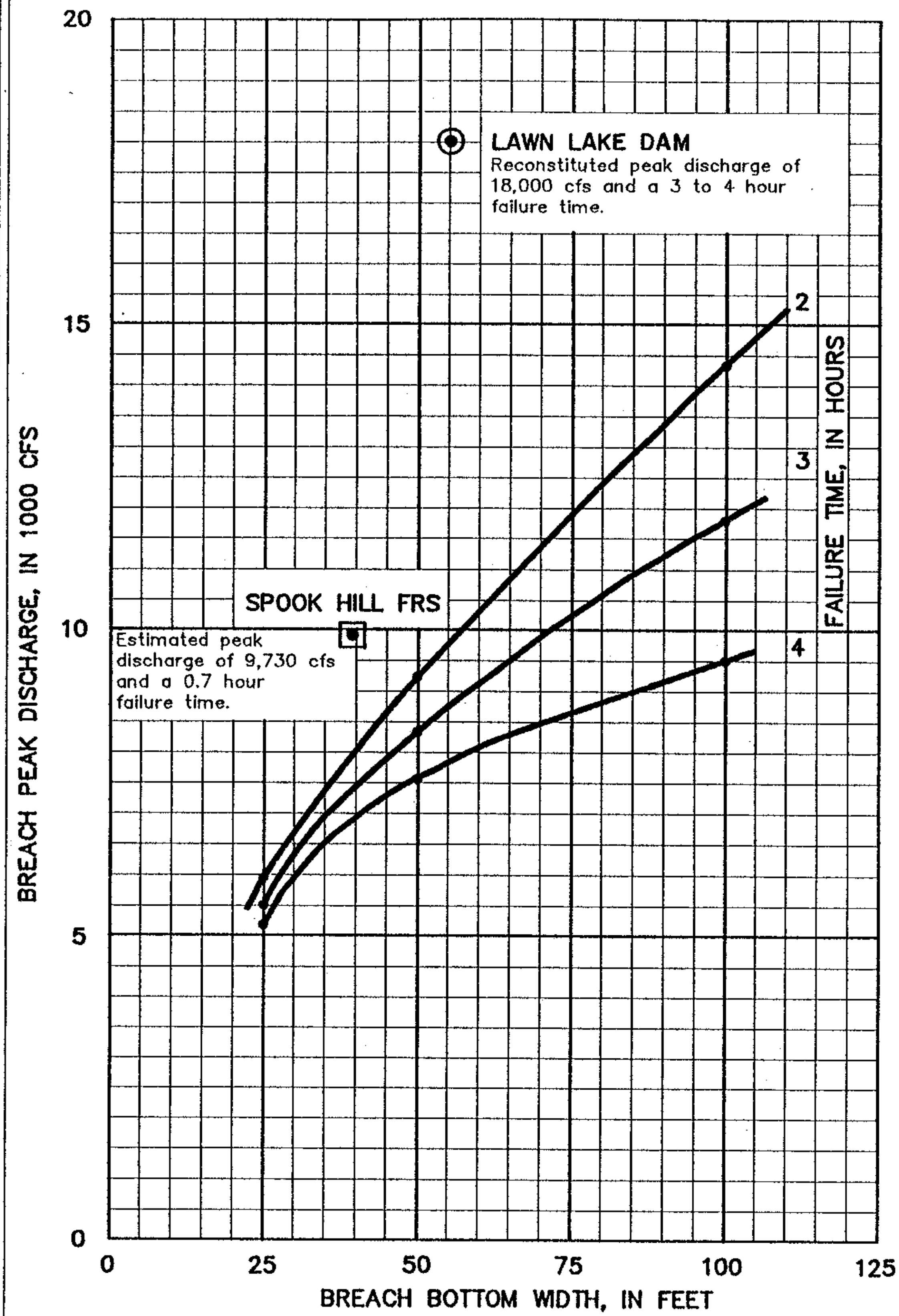
The time to peak (Table III-4) is not synonymous with time to fail (Table III-5). Time to peak with the Breach program is the time from the start of piping to the instant when maximum discharge is issuing from the breach. Time to fail with the DamBrk program is the time from the start of piping to the instant when the breach has formed to its maximum width and depth. The peak discharge is a function of both hydraulic head on the breach opening and the geometry of the breach. Both of these factors vary with time and therefore peak discharge will not necessarily correspond to largest breach opening. However, in this case the time to peak occurs closely to the time to fail because the dam has a relatively low embankment height and a small storage volume. Therefore time to fail in Table III-5 can be compared, for all practical purposes, to time to peak in Table III-4.

TABLE III-5

SENSITIVITY OF PIPING BREACH PARAMETERS IN THE DAMBRK PROGRAM

Breach Width ft (1)	Time to Fail hrs (2)	Breach Peak Discharge cfs (3)
25	2	5,924
50	2	9,268
100	2	14,422
25	3	5,590
50	3	8,407
100	3	11,915
25	4	5,272
50	4	7,610
100	4	9,624



SENSITIVITY OF BREACH BOTTOM WIDTH AND FAILURE TIME WITH THE DAMBRK PROGRAM FOR LOCATIONS A AND B

FIGURE III-2

RESULTS OF BREACH ANALYSIS

For the piping breach estimate, the best estimate properties and parameters from Tables III-2 and III-3 were used. Based on the results of the sensitivity analysis (Table III-4) the values of D_{50} = 2mm, D_{90}/D_{30} = 120, and internal friction angle = 33° were used. For Locations A and B the assumed initial piping elevation is 1,567 ft, and for Location C this elevation is 1,570 ft.

For Locations A and B, a breach peak discharge of 9,730 cfs (after adjusting for the 850 cfs inflow) and 1.0 hour failure time were estimated with the Breach program. For comparison purposes, this point is plotted in Figure III-2. The results of the DamBrk program sensitivity tests and the Breach program best estimate seem to corroborate each other, except that the time to failure (1.0 hour) seems to be very short for this embankment. However, for the intent of dam break analyses in general and this study in particular, conservative results are preferred to underestimates.

The breach outflow hydrographs for the three breach locations are shown in Table III-6 and Figure III-3. The Breach program requires an inflow hydrograph for program execution. As a result, it was felt reasonable to assume that an inflow of 850 cfs would continue for the duration of the breach. However, the tail end of the generated outflow hydrographs were adjusted to remove the effect of the 850 cfs inflow. In both of the breach hydrographs of Figure III-3 there is a dramatic increase in discharge at about time 0.7 hour. This is due to the modeling of the piping orifice collapse at that time resulting in dramatically increased discharge capacities. Furthermore, a discharge value of 2,750 cfs was added to the outflow hydrographs to account for the effect of the CAP (discussed subsequently under "Effect of CAP Canal Breach Hydrographs").

The peak discharge (6,670 cfs) for Location C is less than at Locations A and B (9,730 cfs). This is because the hydraulic head on the breach at Locations A and B is 15 feet as compared to 12 feet at Location C. The outflow hydrograph at Location C is of longer duration than at Locations A and B because the lower peak discharge is compensated by a more sustained flow. The volumes of breach release (1,077 ac-ft for Locations A and B, and 1,067 ac-ft for Location C) is comparable to the storage volume of 901.6 ac-ft at elevation 1,582 ft, (shown in Table III-1). The reason for the larger volume of breach release is because of the 850 cfs of assumed inflow which maintains the water level at elevation 1,582 ft.

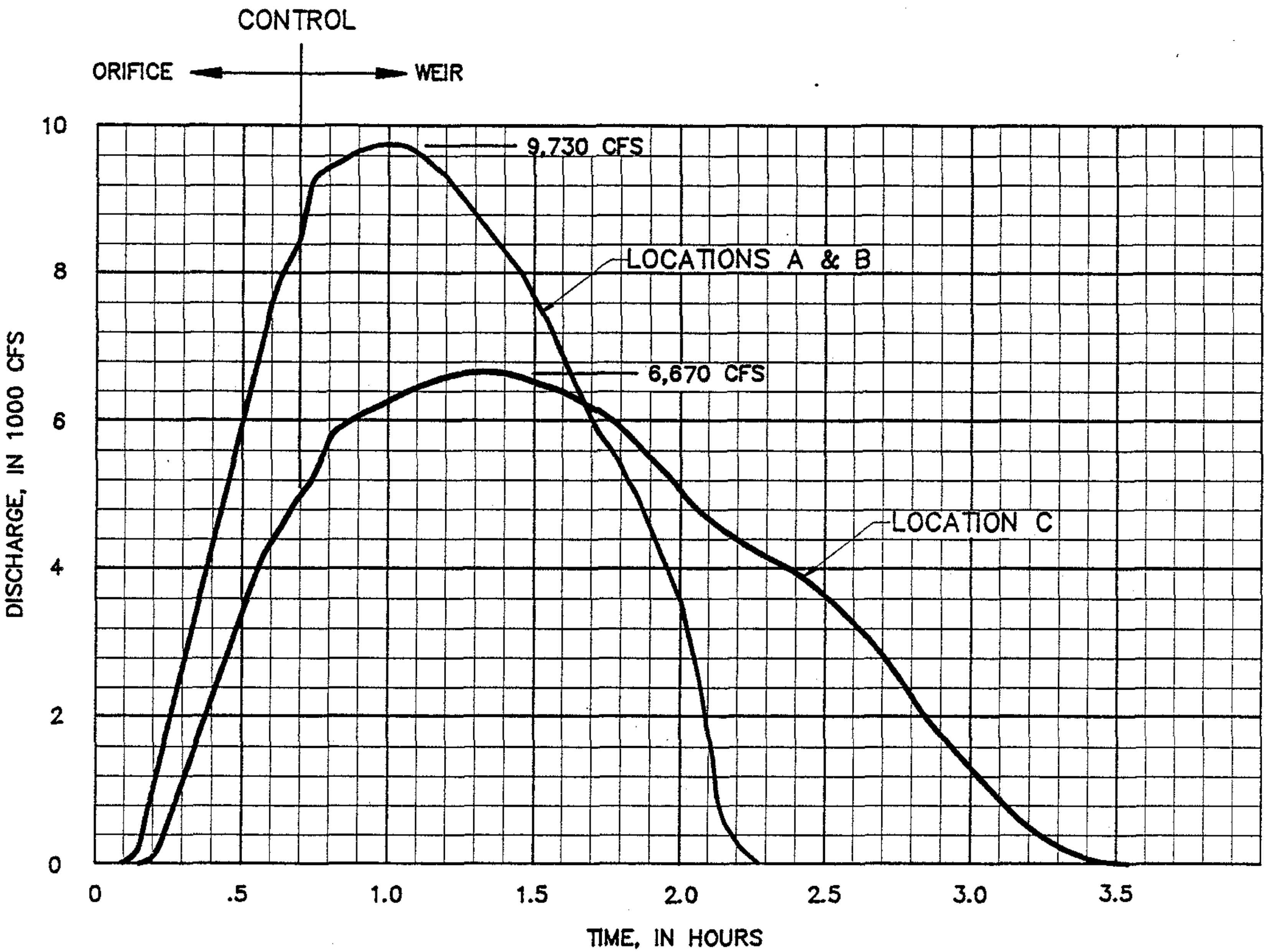


FIGURE III-3
LOCATIONS A,B & C
BREACH OUTFLOW HYDROGRAPHS

TABLE III-6

PIPING BREACH OUTFLOW HYDROGRAPHS

Time	Breach Outflow		CAP	Combined Outflow	
	Locations A & B	Location C	Breakout	Locations A & B	Location C
hrs.	cfs	cfs	cfs	cfs	cfs
(1)	(2)	(3)	(4)	(5)	(6)
0	0	0	0	0	0
.1	0	0	0	0	0
.2	810	180	0	810	180
.3	2,900	1,370	0	2,900	1,370
.4	4,800	2,570	0	4,800	2,570
.5	6,370	3,600	0	6,370	3,600
.6	7,615	4,460	0	7,615	4,460
.7	8,420	5,050	0	8,420	5,050
.8	9,430	5,820	2,750	12,180	8,570
.9	9,650	6,050	2,750	12,400	8,800
1.0	9,730	6,290	2,750	12,480	9,040
1.1	9,640	6,490	2,750	12,390	9,240
1.2	9,340	6,610	2,750	12,090	9,360
1.3	8,870	6,670	2,750	11,620	9,420
1.4	8,240	6,670	2,750	10,990	9,420
1.5	7,560	6,560	2,750	10,310	9,310
1.6	6,750	6,360	2,750	9,500	9,110
1.7	5,840	6,090	2,750	8,590	8,840
1.8	5,300	5,760	2,750	8,050	8,510
1.9	4,500	5,360	2,750	7,250	8,110
2.0	3,280	4,930	2,750	6,030	7,680
2.1	1,080	4,590	2,750	3,830	7,340
2.2	0	4,390	2,750	2,750	7,140
2.3		4,130	2,750	2,750	6,880
2.4		3,820	2,750	2,750	6,570
2.5		3,460	2,750	2,750	6,210
2.6		3,050	2,750	2,750	5,800
2.7		2,600	2,750	2,750	5,350
2.8		2,100	2,750	2,750	4,850
2.9		1,580	2,475	2,475	4,055
3.0		1,110	2,200	2,200	3,310
3.1		710	1,925	1,325	2,635
3.2		390	1,650	1,650	2,040
3.3		170	1,375	1,375	1,545
3.4		70	1,100	1,100	1,170
3.5		0	825	825	825
3.6			550	550	550
3.7			275	275	275
3.8			0	0	0

The time rate of growth of the breach bottom width is shown in Figure III-4. As shown, the breach bottom width grows quickly once the erosion pipe collapses and increased flow rates accelerate the erosion of the breach. The final breach width at Location C is greater than at Locations A and B because of the longer duration of breach flow at Location C.

COMPARISON WITH LAWN LAKE DAM PIPING BREACH

There is considerable uncertainty in estimating dam break hydrographs and, when possible, such estimates should be compared to actual dam break experience. For this purpose, the estimated piping breach for the Spook Hill FRS is compared to the piping breach failure that occurred to the Lawn Lake Dam in Rocky Mountain National Park, Colorado on 15 July 1982. The failure of the Lawn lake Dam has been described and analyzed by Jarrett and Costa (1984). A comparison of the physical characteristics of the Lawn Lake Dam and the Spook Hill FRS is shown in Table III-7.

TABLE III-7

PHYSICAL CHARACTERISTICS COMPARISON OF LAWN LAKE DAM AND SPOOK HILL FRS

	Lawn Lake Dam	Spook Hill FRS
Dam Crest Elevation, ft	11,000	1,591
Upstream Bed Elevation, ft	10,972	1,567
Height of Dam, ft	28	24
Water Surface Elevation, ft	10,996.4	1,582
Hydraulic Height of Dam, ft	24.4	15
Upstream Face Slope	1V:1.5H	1V:3H
Downstream Face Slope	1V:1.5H	1V:2H
Crest Width, ft	8	14
Impoundment Volume, ac-ft	674	901.6

A cross-sectional comparison of the embankment sections of the two dams is shown in Figure III-5. Although the structural heights (28 ft and 24 ft) of the two dams and impoundment volumes (674 ac-ft and 901.6 ac-ft) are comparable, there are some major differences.

Specifically, the hydraulic height of the dams (24.4 ft and 15 ft) and the embankment geometries are quite dissimilar. Lawn Lake Dam was probably constructed of coarser material than the Spook Hill FRS, and construction control was certainly much better at the Spook Hill FRS, and in fact the Lawn Lake Dam probably failed due to an unreported construction deviation from the outlet pipe design.

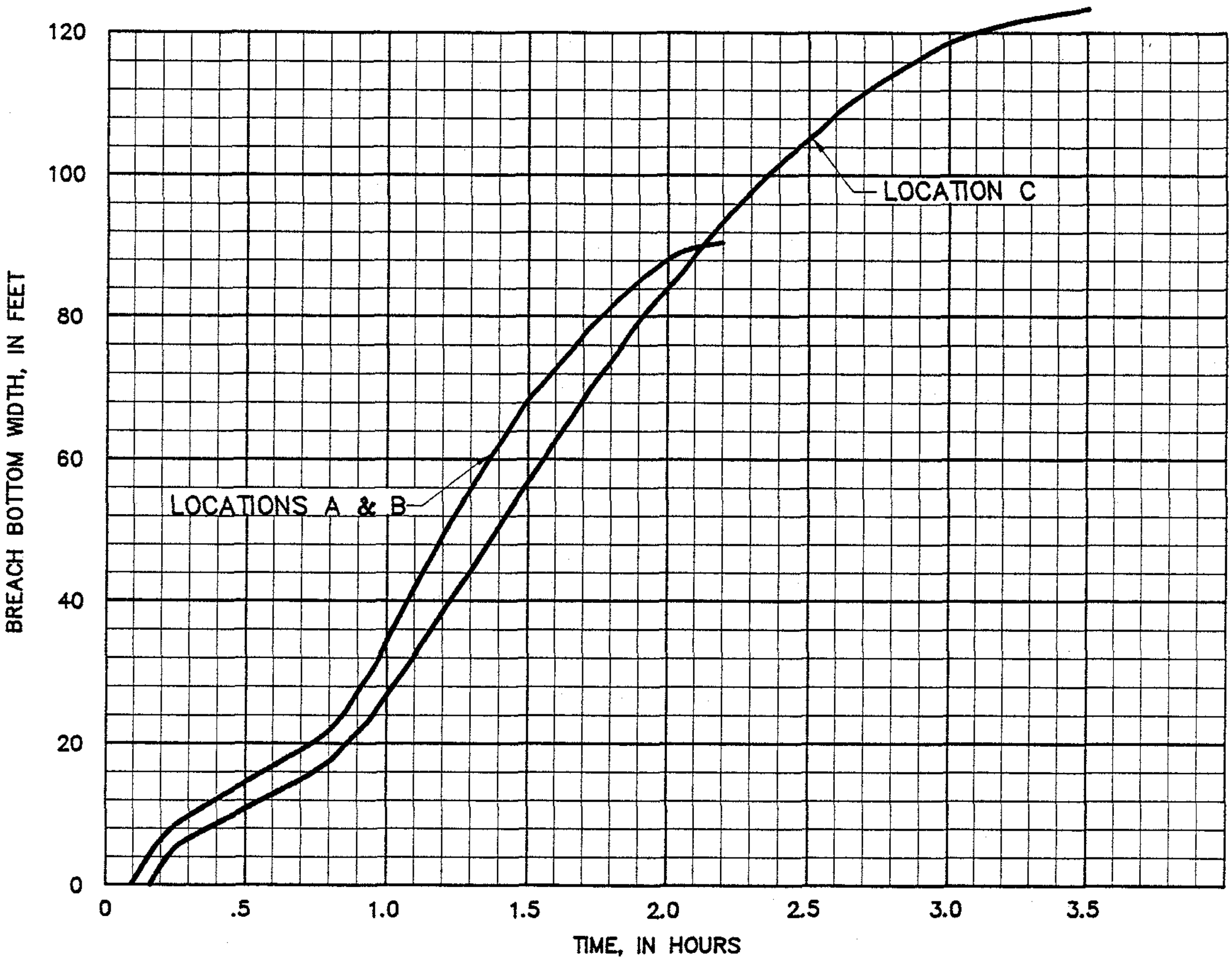


FIGURE III-4

TIME RATE OF GROWTH OF BREACH BOTTOM WIDTH

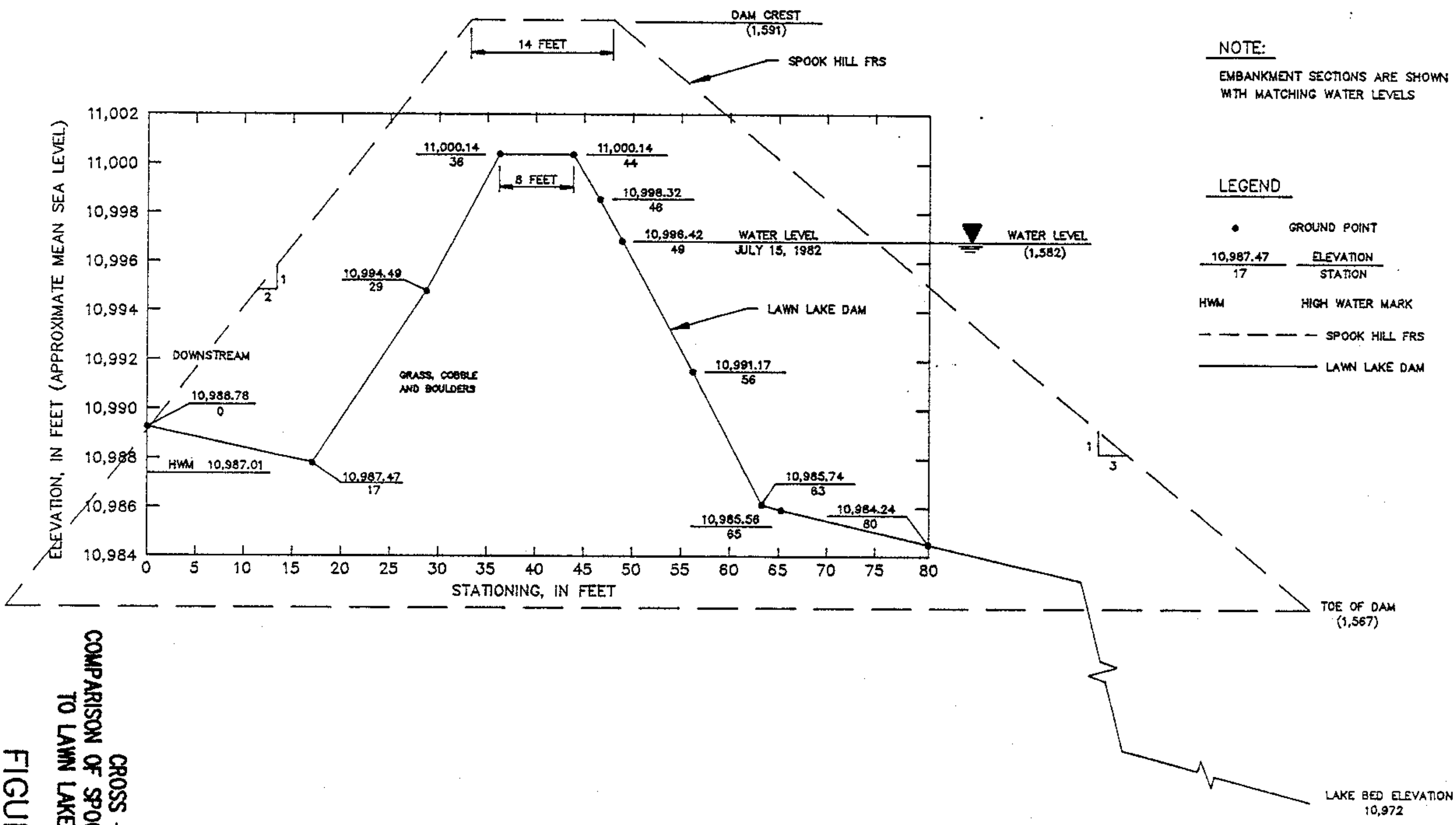


FIGURE III-5

CROSS - SECTIONAL COMPARISON OF SPOOK HILL FRS EMBANKMENT TO LAWN LAKE DAM EMBANKMENT

A comparison of the reconstituted, actual Lawn Lake Dam failure and the estimated Spook Hill FRS failure is shown in Table III-8.

TABLE III-8

COMPARISON OF PIPING BREACH CHARACTERISTICS FOR THE ACTUAL LAWN LAKE DAM FAILURE AND THE ESTIMATED SPOOK HILL FRS FAILURE AT LOCATIONS A AND B

	Lawn Lake Dam	Spook Hill FRS
Breach Depth from Dam Crest, ft	27.7	24
Breach Depth from Water Surface, ft	24.4	15
Breach Top Width, ft	97	96
Breach Bottom Width, ft	55	39
Breach Side-Slope, approximate	1V:1H	1V:1H
Time to Breach, hours	3 to 4	1
Time from Complete Breach to Peak Discharge, minutes	10	18
Peak Breach Discharge, cfs	18,000	9,730

The breach widths are very similar as are the times from complete breach (collapse of the erosion pipe through the embankment) to peak discharge. The Lawn Lake Dam failure resulted in a peak discharge of about 18,000 cfs and a time to breach of 3 to 4 hours. The hydraulic head on the breach was 24.4 ft. The estimated peak discharge for the Spook Hill FRS at Locations A and B is 9,730 cfs and a time to breach of 1 hour. Considering the lesser head on the Spook Hill FRS, the lower peak discharge is reasonable. The time to fail of the Spook Hill FRS seems too fast compared to the Lawn Lake Dam, especially when comparing the embankment geometries. The peak discharge from Lawn Lake Dam is shown in Figure III-2. The piping breach hydrographs that are shown in Figure III-3 are reasonable estimates for the purpose of this flood inundation study. This conclusion is based on the results of the various sensitivity tests that indicate that more severe flood hydrographs are not generated for any reasonable set of dam break parameters, and by comparison with the dam break hydrograph that was estimated for the actual Lawn Lake Dam failure.

EFFECT OF CAP CANAL ON BREACH HYDROGRAPHS

The CAP Canal is parallel with the embankment as shown in Figure II-1, and just downslope from the dam as shown in the photograph of Figure III-6. A Study Area Map is presented in Figure III-7. The CAP Canal crosses the Salt River via an inverted siphon and is lifted vertically at the Salt-Gila Pumping Plant in order to continue southward to Tucson. The distance from the Salt-Gila Pumping Plant to the Principal Spillway is about 2.1 miles. Between University Drive and Crismon Road, about 5 miles southeast of the Principal Spillway, there is a CAP discharge monitoring station. Canal discharge information is relayed to the main CAP Control Center. The Control Center can open or close the gates at the discharge monitoring station as well as start or stop the pumps at the Salt-Gila Pumping Station. The Study Area Map also shows the RWCD Canal, the East Maricopa Floodway and the Superstition Freeway Channel.

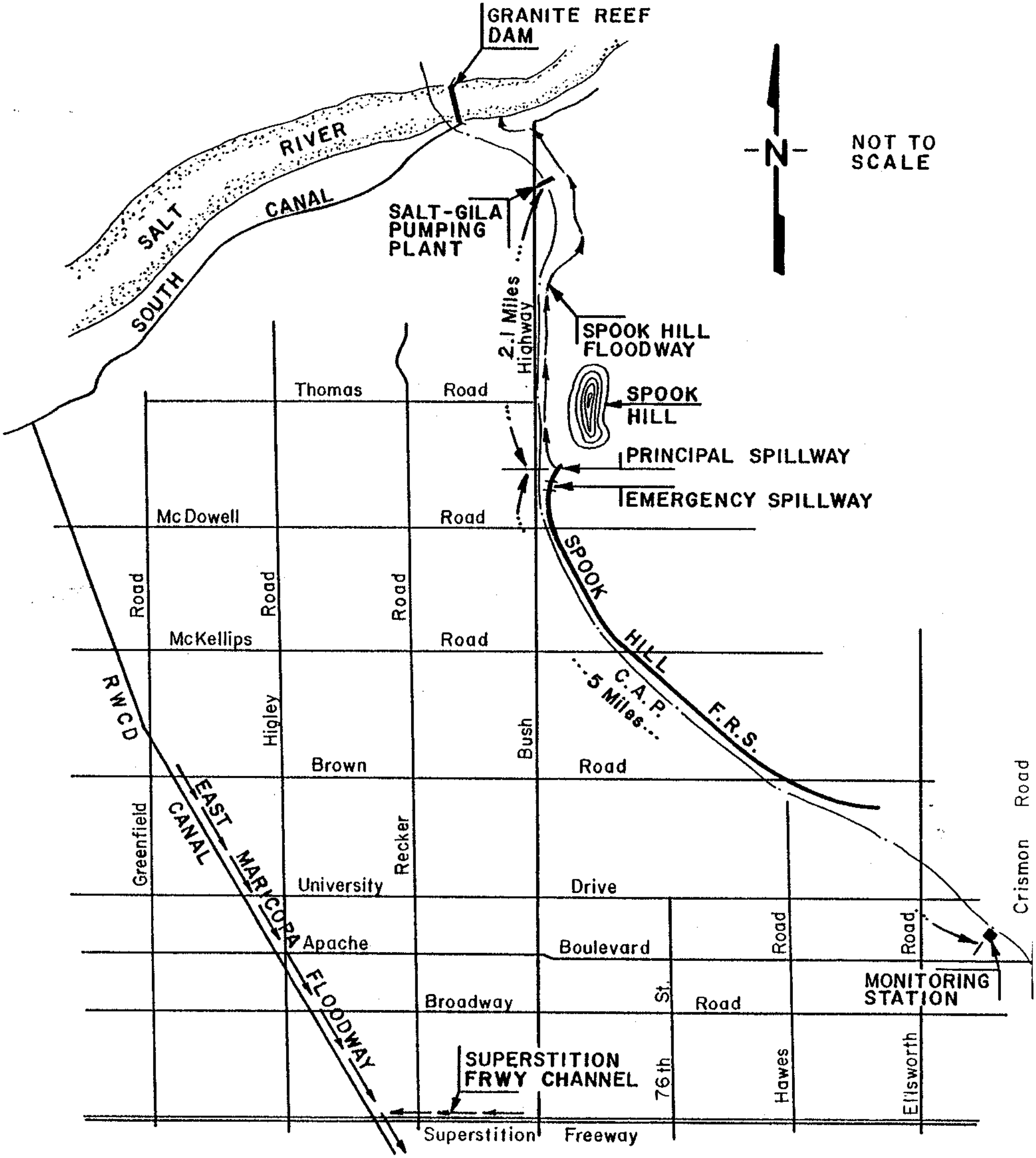
For the piping breach analysis it is assumed that the embankment breach dumps a large quantity of earthen embankment material into the CAP Canal resulting in the instantaneous plugging of the canal. This will cause the canal discharge (2,750 cfs) to break out of the canal and to join with the breach release. The CAP Canal discharge of 2,750 cfs will continue until the pumps can be stopped at the Salt-Gila Pumping Plant. Breakout discharge from the CAP Canal will then decrease until the water in the canal between the pumping plant and breach location is drained.

The CAP Canal breakout hydrograph, as qualitatively illustrated in Figure III-8, was estimated by calculating the travel time (t_1) from the Principal Spillway to the CAP Canal discharge monitoring station. At a flow velocity of about 3.7 ft/sec, the travel time from the Principal Spillway to the monitoring station is about 2 hours. The travel time from the Salt-Gila Pumping Plant to the Principal Spillway at a flow velocity of about 3.7 ft/sec is about 1 hour (t_2).

CAP CANAL SHOWING THE SPOOK HILL FRS EMBANKMENT

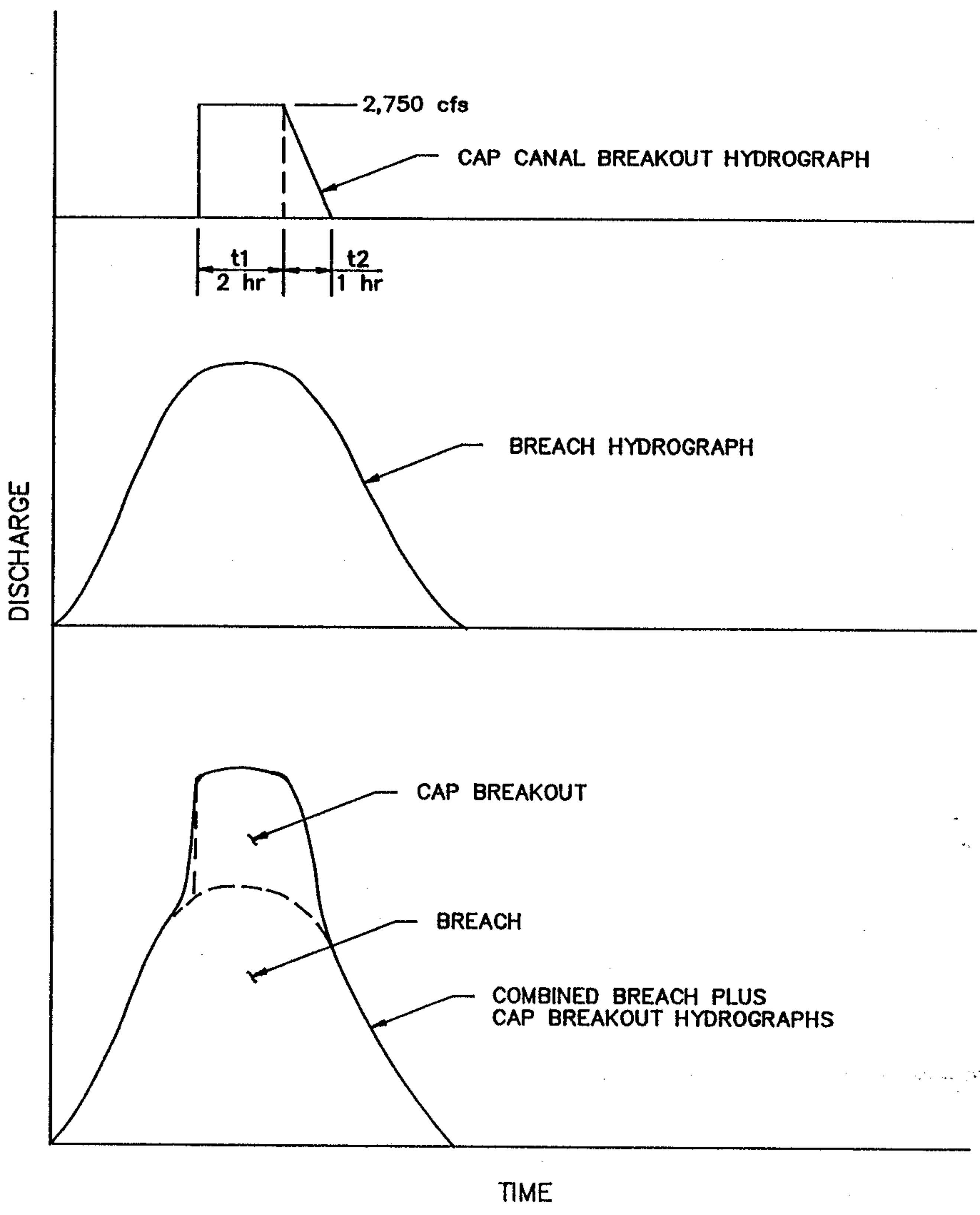
IMMEDIATELY UPSLOPE (TO THE RIGHT) OF THE CANAL.

FIGURE III-6



STUDY AREA MAP

FIGURE III - 7



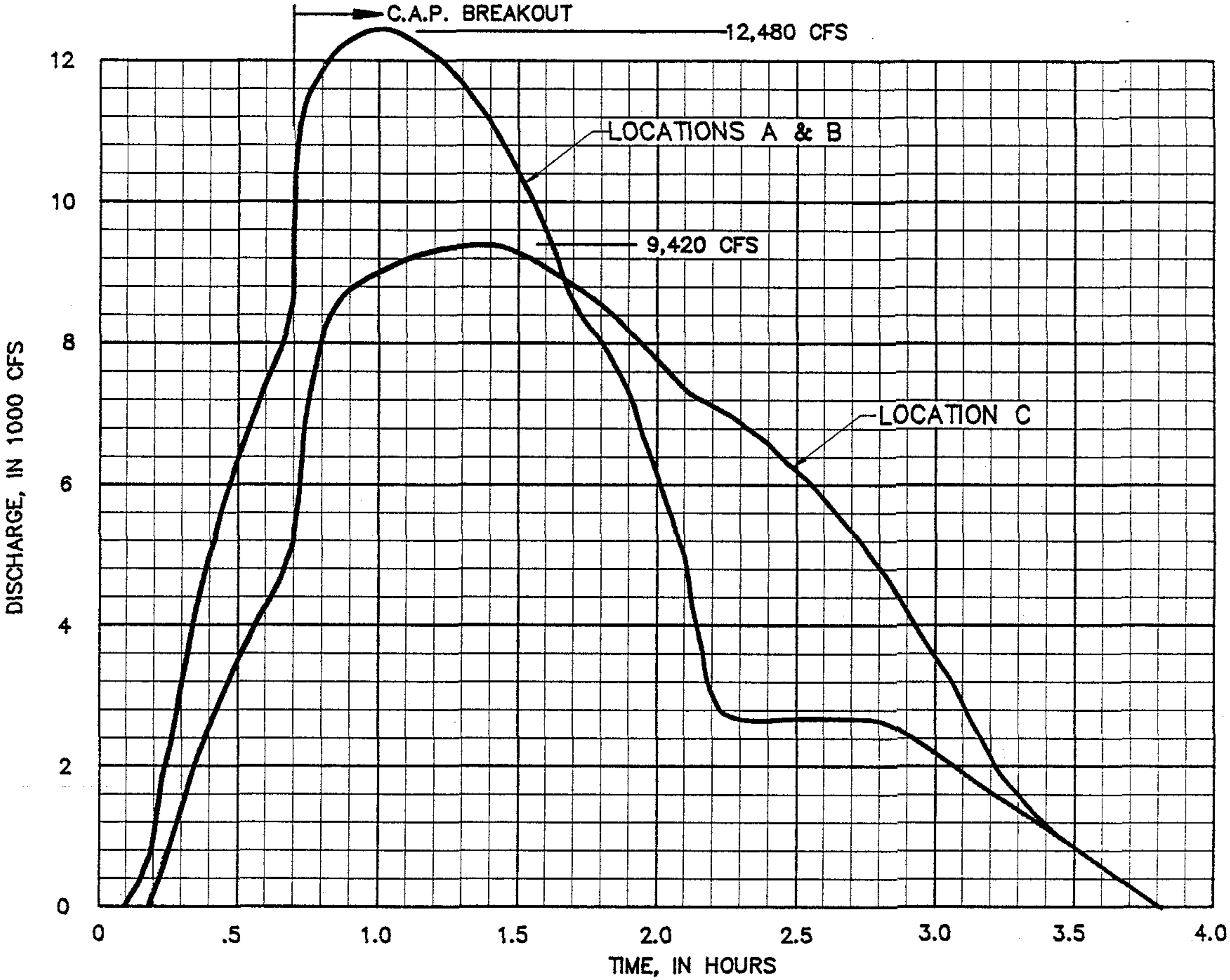
QUALITATIVE ILLUSTRATION OF COMBINATION OF BREACH HYDROGRAPH WITH CAP BREAKOUT HYDROGRAPH.

FIGURE III–8

A sketch of a typical piping breach release hydrograph is also shown in Figure III-8. It is assumed that the CAP Canal has no impact on the downslope flood hydrograph up to the time that the CAP Canal is assumed to be instantaneously and completely plugged. The time of plugging is assumed to occur near the time of peak breach outflow. The superposition of the CAP breakout hydrograph on the piping breach hydrograph is illustrated in Figure III-8. The smoothed, total flood discharge hydrograph was calculated for each breach location, and those hydrographs are presented in the next section.

BREACH HYDROGRAPHS

As described above, it is assumed that the breach of the embankment will result in the sudden influx of embankment material into the CAP Canal of sufficient volume to cause plugging of the CAP Canal, and that the ensuing breakout from the CAP Canal will be additive to the breach hydrograph. The time of plugging and breakout of the CAP Canal is assumed to occur when the erosion pipe through the embankment collapses (0.7 hour after onset of piping, see Figure III-3). The CAP Canal breakout hydrograph was added to the breach hydrographs according to the procedure illustrated in Figure III-8, and the results are shown in Table III-6 and Figure III-9. The combined breach and CAP breakout hydrographs for Locations A and B, and for Location C was input to the appropriate DHM models for the flood inundation analyses.



PIPING BREACH HYDROGRAPHS
WITH CAP CANAL BREAKOUT
FIGURE III-9

SECTION IV

FLOOD INUNDATION ANALYSIS

MODEL SELECTION

In the past, models that were selected for floodplain analysis were usually one-dimensional. In general, one-dimensional analysis can be used if there is no significant lateral variation in flow. However, in many situations the flows are highly two-dimensional, especially on alluvial fans, coalesced alluvial fans, and distributary flow systems. The use of a two-dimensional model eliminates the uncertainty in estimating flow hydraulics that may result from variations in choosing the watercourse cross-sections that are necessary in one-dimensional modeling. Two-dimensional modeling can be especially useful when modeling urban areas, where selection of appropriate cross-sections for one-dimensional models can be difficult. In addition, a two-dimensional model is a necessity where lateral flood flows are expected.

As shown on the Study Area Map, Figure III-7, the area of study, downslope from the Spook Hill FRS, is bounded on the north by the Salt River, on the west by the Roosevelt Water Conservation District (RWCD) Canal and on the east by the structure itself. The flow boundary to the south is the Superstition Freeway Channel which was constructed by the Arizona Department of Transportation. The East Maricopa Floodway (EMF) flows to the south and parallels the RWCD Canal from approximately Brown Road. The study area is approximately 31 square miles. The topography is varied and contains many different land uses; agricultural, residential and industrial, with a large portion undeveloped. A two-dimensional model allows a physical description of the hydraulic characteristics of these land surfaces.

The Diffusion Hydrodynamic Model (DHM) (Hromadka and Yen, 1987) is the two-dimensional program that was selected for this study. It is a public domain program that is readily available and has been used in studies with similar characteristics to the Spook Hill FRS study (Hromadka and others, 1985; and Hromadka and Lai, 1985). It is capable of representing unsteady backwater effects, ponding, channel overflow due to constrictions, and overbank flow on the floodplain. The DHM is easily handled by most personal computers and does not require expertise beyond that required for use of one-dimensional models. A front-end program is also available which greatly simplifies the necessary model input. Results from the DHM for a one-dimensional flow have been compared to results obtained from the USGS unsteady, one-dimension flow model (K-634 model) with favorable results (Land, 1980a; and Land, 1980b).

MODEL REQUIREMENTS

General

The DHM is based on the diffusion equation where non-inertial (gravity, friction and pressure) forces are assumed to dominate the flow equation. This is in comparison to the kinematic wave model, where the inertial and pressure terms are assumed negligible in comparison to the friction and gravity terms. In the DHM, a finite difference approach is used which equates each grid or cell-centered flow rate to a function of the four neighboring grid cells.

Input

The finite difference solution used in the DHM requires the modeling area to be divided into grids of equal length and width. Hromadka and Yen (1987), have shown that grid sizes from 1,000 ft to 5,000 ft function well within the model. The public domain model version used in the study, DHM Version 21, limits the number of grids to 250. Therefore, the selection of grid size depends, to some extent, on the size of the study area. Grids are sometimes called nodes.

Within each grid, a typical elevation, Manning's roughness value, n, and an initial water depth must be assigned. Since the current version of the DHM cannot take into account flow reduction factors such as buildings, bridges, retaining walls, etc., the Manning's "n" value is determined for each grid to account for these effects. This grid set-up is known as a Topographic Terrain Model (TTM).

Global input to the model, that is, applied to the entire model area, requires the input of minimum and maximum time step values, the simulation time, the desired computer output periods, surface detention, and minimum changes in water depth. Ranges used in this study for the minimum time step are 0.01 second to 1.0 second, 5 to 50 seconds for the maximum time step and 0.5 to 2.0 ft for the minimum changes in water depth. The time step values and the minimum change in water depth are computationally interdependent. As both values are made smaller, the model refines its calculation of inflow and outflow values for the grids and allows for smaller variations of water depth between the grids. This refining, however, greatly increases the computer run time while not necessarily providing improved accuracy.

The simulation time should be large enough to allow the inflow hydrograph to adequately pass through the model area without tying up excess computer time. Output periods should be small enough to adequately view the flood flow advance, while again realizing computer run time constraints. Surface detention values are used to simulate infiltration and storage for the model area. Each grid "holds-back" the specified depth of water as it flows past the grid. Since this is a global input, an average value must be used.

The DHM can also model channels within the floodplain. A channel is specified by the number of the grid it flows through. Width, depth, Manning's roughness value, and initial water depth are the required inputs. Inflow hydrographs can be placed at any location in the floodplain or at any place along the channel with up to ten pairs of coordinates allowed for each hydrograph. Outflow from the model area can be modeled by defining grids as no-flow boundaries, critical flow grids, or by creating "sinks" - border grids that are artificially made very deep to avoid artificial backwater due to outflow model boundaries.

Limitations
Relatively short simulation times must be used because of the time stepping method used in the DHM. Longer simulation times may cause problems with the program. This can be overcome by using a percentage of the desired simulation time in an initial run of the model and using the results as input for a succeeding run.

It has been shown, (Hromadka and DeVries, 1985), that if large velocity flow regimes are developed (greater than 25 feet per second) inaccuracies in results may develop. In typical dam break studies, such velocities are not encountered.

SPOOK HILL FLOOD INUNDATION MODELS

Input Parameters for Individual Grids
USGS quadrangle maps and areal photographs were obtained for the study area. By combining these two, it was possible to obtain information on land use and urbanization affecting overland flow, as well as the necessary topographic information. The 31 square mile study area was analyzed with three separate models: Model A contains the area downstream from the Emergency Spillway and the area downstream of the dam breach at Location A; Model B contains the area downstream

of the dam breach at Location B; and Model C contains the area downstream of the dam breach at Location C. The TTM was laid out over the expected inundation area in each model area and these are presented in the rear folder drawings entitled "Location "A" DHM Grids," (Sheet 1-1) Location "B" DHM Grids," (Sheet 2-1) and "Location "C" DHM Grids," (Sheet 3-1). TTM A and TTM C each contain 250 grids, while TTM B contains 246 grids. Each grid is 1,000 ft by 1,000 ft. It is important to note that while only three TTM's were set up, four specific models were generated, there being different hydrographs input for the PMF discharge from the Emergency Spillway and for the piping breach at Location A.

Representative elevations were taken at the center of each grid unless otherwise dictated by the local topography in the grid. The Manning's roughness value, n, was generally assigned to each grid as follows in Table IV-1:

TABLE IV-I

ASSIGNMENT OF MANNING'S ROUGHNESS VALUES

Grid containing a main street running parallel to flow	n = 0.018
Grid with mostly undeveloped areas	n = 0.025
Grid with slightly urbanized areas	n = 0.035
Grid with heavily urbanized areas	n = 0.045

Each grid was studied to determine if an adjustment in the n value from the above criteria was needed. These adjustments were made to account for flow area reduction or other factors that might affect the flow velocity across the grid. Using the aerial base maps, each grid was examined to quantify different roughness zones or obstructions that would affect velocity. Judgments were made to average the n values of these zones and arrive at an n value that would adequately describe the overland flow hydraulics in the grid. An example of this procedure was a grid where 50% of the area was single family housing with an n value of 0.045 and 50% consisted of a park with an n value of 0.025. After examining the grid for any other factors that might affect the velocity, a judgement was made that an averaged n value of 0.035 would adequately describe the roughness and therefore describe the average velocity across the grid. Other examples that necessitated adjustment were grids that contained parking lots within urbanized areas and sub-divisions with boundary fences.

Some land subsidence is reported to have occurred in the City of Mesa downslope from the Spook Hill FRS. This subsidence was investigated and it was determined that subsidence is not known to have occurred in the area of the dam, nor is future subsidence at the dam site anticipated because of the relatively shallow alluvium over bedrock in that area. Subsidence downslope from the dam is rather minimal and there is no reliable way to project future land surface elevation changes for the effects of any further subsidence. Therefore, the TTM's represent existing topography. Future subsidence in the area could require reanalysis of the downslope flood inundation. Land surface fissures and cracks due to subsidence are considered to be relatively minor. No attempt was made to model flood volume losses due to any existing or future fissures. It is anticipated that such losses would be small for normal fissures. Large fissures, if they were to develop in the future due to excessive subsidence, could redirect flow paths resulting in the need to reanalyze downslope flood inundation.

Global Input Parameters

After several preliminary runs, values of 0.5 second, 30.0 seconds, and 1 ft were ultimately used for minimum and maximum time step and the minimum change in water depth, respectively. These values yielded reasonable results without requiring a prohibitive amount of computer time. For the Emergency Spillway model, a preliminary run was made using a simulation time of 10 hours. As the peak of the inflow hydrograph passed through the Emergency Spillway at T = 2.75 hours, it was felt that a simulation time of 10 hours would adequately describe the flood inundation to the area of study. For comparison, a run was made using a simulation time of 15 hours, which is the total time of the inflow hydrograph. The differences were negligible, with only minor drainage of the grids occurring between T = 10 hours and T = 15 hours. Appendix B contains the DHM summary table, "Maximum Water Surface Values for Flood Plain," for comparison between the 10 hour and 15 hour simulations. For the models concerning the breaches at Locations A, B and C, a simulation time of 5 hours was used. As the peak flow from the breach scenarios occurs between 1 and 1.5 hours, a simulation time of 5 hours was considered adequate. To verify this, Breach Location C was simulated to 8 hours. The summary table contained in Appendix B shows negligible differences.

For all models, output periods were specified for every 15 minutes (0.25 hours). This proved to be more than adequate for following the advance of the flood flow. The surface detention was set at

0.25 ft (3 inches). This value was based on an estimate of the total infiltration of several inches and a surface ponding of an inch or more, on the average, that will occur in the area of study. The value of 0.25 ft is considered conservative and greater retention is anticipated.

Inflow-Outflow

For TTM A, which corresponds to the models for the Emergency Spillway and the breach at Location A, "sinks" were placed at the northern edge of the model area along the Salt River, to account for water flowing out of the model boundary. These sinks, which are grids with artificially low elevations, allow the water to flow off the model area and collect in the sinks without causing backwater on the upgradient model grids. Outflow rates and volumes can be estimated by calculating the amount of water that is accumulated in each grid sink during a specified time period. Along the western edge of the model, which corresponds to the RWCD Canal, 3 ft high canal banks were simulated in the border grids. A depth of water greater than 3 ft in the adjacent grids would signify that the canal banks were overtopped. No boundary conditions were specified for the southern edge of the model as no natural boundary was encountered. Sufficient grids were added, as determined necessary by subsequent runs, to allow the water to flow its course. It should be noted that any border grid where no outflow is specified acts as a no-flow boundary.

For TTM B and C, no boundary conditions were specified for the northern or southern edge of the model. As before, sufficient grids were added to allow the water to flow its course. The western edge of these two zones corresponds to the East Maricopa Floodway (EMF) which parallels the RWCD Canal in this vicinity. The floodway was specified as a channel in the DHM models. North of Broadway Road, the floodway was modeled as being 80 ft wide by 9 ft deep. These dimensions were obtained from as-built engineering plans. An n value of 0.02 was used for this section. South of Broadway Road, the floodway is less defined and flows through a golf course. A channel 300 ft wide by 9 ft deep with an n value of 0.025 was modeled for that section. The last downstream grid of the channel was defined as a "channel outflow grid," using one of the options of the DHM model. This option allows the water flowing down the channel to leave the model area and calculates the flow rate for the channel at this outflow grid.

The PMF Emergency Spillway hydrograph, Figure II-6, was used as the inflow hydrograph for the Spillway release flood inundation analysis using TTM A. The models dealing with the piping breaches at Locations A, B and C used the two breach hydrographs shown in Figure III-9. The hydrograph for Locations A and B has a time to peak of 1.0 hour and a peak flow of 12,480 cfs.

The hydrograph for Location C has a time to peak of 1.3 hours and a peak flow of 9,420 cfs. Table IV-2 summarizes the input for the four models used in the study.

TABLE IV-2

SUMMARY OF INPUT FOR THE FOUR FLOOD INUNDATION ANALYSES

Model	TTM	Inflow Hydrograph	Grid No. for Inflow
Emergency Spillway	A	Emergency Spillway Hydrograph (Figure II-5)	A7
Breach at A	A	Piping Breach A and B (Figure III-9)	A7
Breach at B	B	Piping Breach A and B (Figure III-9)	B5
Breach at C	C	Piping Breach C (Figure III-9)	C5

MODEL RESULTS

Several preliminary runs of the different models were made to check that the predicted inundation areas were reasonable. Any boundary grids that contained more than 0.25 ft of water (the detention depth) at the end of the simulation period were signaled - except those grids along the Salt River or along the RWCD Canal in TTM A. Additional grids were then added that would allow excess water to continue its course until 0.25 ft of water or less remained in the border grids. Due to the fact that the allowable limit of 250 grids was used in modeling the preliminary inundation area, the numbering of the grids which received no water was assigned to the new grids placed along the boundary as described above. This accounts for the irregular grid numbering system as it appears in Sheets 1-1, 2-1 and 3-1, rear folder. Figures IV-1 and IV-2 show examples from the output of the DHM of the Emergency Spillway operation flood inundation, and are typical to all the model outputs. Figure IV-3 contains output from the DHM of the breach at Location B and is an example of output from those models that contain channels (Breach at Location B and Breach at Location C). A hard copy of the output files for the four DHM models is provided in Appendix B, as well as floppy disks containing the output files.

A

MODEL TIME(HOURS) = 9.25 (SECONDS) = .333E+05 (TOTAL TIMESTEP NUMBER

FLOOD PLAIN RESULTS

INFLOW RATE AT NODE 7 IS EQUAL TO 2261.56 B

	C		D			F				
NODE	91	92	93	94	95	96	97	98	99	100
DEPTH	.249	.345	.249	.000	.000	11.917	.000	.000	.000	.000
ELEVATION	1425.249	1421.345	1432.249	1440.000	1363.000	211.917	1305.000	1270.000	1260.000	1430.000
VEL-N	1.476	2.463	.000	.000	.000	.000	.000	.000	.000	.000
VEL-E	.000	-2.705	-.384	.000	.000	-3.722	.000	.000	.000	.000
VEL-S	-1.826	-1.476	2.463	.000	.000	.000	.000	.000	.000	.000
VEL-W	3.101	2.956	1.131	.000	.000	-2.622	.000	.000	.000	.000

								E		
NODE	101	102	103	104	105	106	107	108	109	110
DEPTH	.248	.251	.252	.250	.249	.250	.250	.250	.115	.250
ELEVATION	1429.248	1411.251	1411.252	1414.250	1413.249	1415.250	1417.250	1423.250	1410.115	1405.250
VEL-N	.000	.000	.000	1.617	-.829	.000	.879	.000	.000	.879
VEL-E	-1.431	-2.956	-3.101	-3.416	.000	.000	-2.408	.000	.000	-1.846
VEL-S	.000	.000	-1.617	.829	.000	-.879	.000	.000	.000	.000
VEL-W	.000	1.744	2.752	3.311	3.076	.000	1.846	.000	.000	2.775

DHM OUTPUT (INTERMEDIATE) FROM EMERGENCY SPILLWAY OPERATION - LOCATION A

FIGURE IV-1

MAXIMUM WATER SURFACE VALUES FOR FLOOD PLAIN

NODE	91	92	93	94	95	96	97	98	99	100
DEPTH	.744	1.220	.402	.000	.000	12.272	.000	.000	.000	.000
TIME	2.923	3.169	3.332	10.000	10.000	10.000	10.000	10.000	10.000	10.000

NODE	101	102	103	104	105	106	107	108	109	110
DEPTH	.262	.808	1.030	.374	.342	.270	.693	.251	.130	.462
TIME	2.879	2.951	2.937	2.923	2.917	2.813	3.012	5.492	10.000	2.388

NODE	111	112	113	114	115	116	117	118	119	120
DEPTH	.278	.638	.443	.479	1.690	.259	1.056	.000	.000	.000
TIME	2.713	2.937	2.964	2.975	3.213	3.474	10.000	10.000	10.000	10.000

NODE	121	122	123	124	125	126	127	128	129	130
DEPTH	.000	.000	12.224	.000	.000	.000	.000	.000	.000	1.307
TIME	10.000	10.000	10.000	10.000	10.000	10.000	10.000	10.000	10.000	3.241

NODE	131	132	133	134	135	136	137	138	139	140
DEPTH	1.513	.287	.745	.490	.941	2.986	.000	.000	.264	.884
TIME	3.005	2.504	3.227	3.349	3.584	3.556	10.000	10.000	3.012	3.548

DHM OUTPUT (FINAL)
FROM EMERGENCY SPILLWAY
OPERATION - LOCATION A

FIGURE IV-2

MODEL TIME(HOURS) = 4.75 (SECONDS) = .171E+05 (TOTAL TIMESTEP NUMBER) = .171E+05 (TOTAL TIMESTEP NUMBER) = 1.3E+03

CHANNEL RESULTS

OUTFLOW RATE AT NODE 90 IS EQUAL TO 1001.50 G

H I

NODE	1	2	3	4	5	6	7	8	9	10
DEPTH	2.040	1.882	2.641	.000	.000	.000	.000	.000	.000	2.504
ELEVATION	1336.040	1335.882	1335.641	.000	.000	.000	.000	.000	.000	1335.504
NODE	11	12	13	14	15	16	17	18	19	20
DEPTH	2.288	.000	.000	.000	.000	.000	.000	.000	1.889	2.343
ELEVATION	1335.288	.000	.000	.000	.000	.000	.000	.000	1334.889	1334.343
NODE	21	22	23	24	25	26	27	28	29	30
DEPTH	1.825	1.195	.000	.000	.000	.000	.000	.000	.000	.000
ELEVATION	1333.825	1332.195	.000	.000	.000	.000	.000	.000	.000	.000
NODE	31	32	33	34	35	36	37	38	39	40
DEPTH	.000	.000	.000	.000	.000	.000	.000	.000	.863	2.200
ELEVATION	.000	.000	.000	.000	.000	.000	.000	.000	1330.863	1330.200

DHM OUTPUT (CHANNEL) FROM BREACH - LOCATION B

FIGURE IV-3

In Figure IV-1, the model time of 9.25 hours (see letter A on the figure) indicates the real-time in the simulation that the output represents. In this case, model output has been requested for every 0.25 hours, starting at model time = 0.25 hours and ending at model time = 10.00 hours. The inflow rate of 2,261.56 cfs (B) represents the discharge passing through grid 7, which corresponds to the Emergency Spillway release at a time of 9.25 hours after the start of the hydrograph.

Following this output is a table of the depth, surface elevation, and flow velocity in each direction for each of the grids (C) which are called nodes on the output. A negative velocity indicates an inflow to the grid, while a positive value indicates an outflow from the grid. At adjacent grids, such as grids 92 and 93, an outflow in the northerly direction of 2.463 feet per second (grid 92) corresponds to an inflow of 2.463 feet per second (grid 93) from the south (D).

Other grids of interest in Figure IV-1, include number 108 and number 96. Grid 108 (E) is an example of a grid where all inflow and outflow has ceased and only the detention depth of 0.250 ft remains. Grid 96 (F) is an example of one of the sinks. The depth of water in the grid can be compared at each output time to determine flow rate and total volume of water that has passed that point. Flow rate, in or out, of any grid can be calculated by using depth, flow velocity and the width of the grid, 1,000 feet. It should be noted that in TTM A, velocities greater than 25 fps were encountered in some grids adjacent to the sinks. These grids have high velocities because of the critical flow entering the sinks. The high velocities are not representative of actual conditions.

Figure IV-2 is an example of the final output from the DHM with the maximum depth in each grid and the time of its occurrence appearing on the printout. This part of the output facilitated the preparation of the flood inundation maps.

In Figure IV-3, the outflow rate of 1,001.50 cfs (G) represents the flow out of the channel at grid 90. This grid is the last downstream grid represented in the model, as shown in Sheet 2-1 of 3. The depth of 2.040 ft (H) represents the depth of water, and corresponding surface elevation, in the part of the channel that passes through Grid No. 1 at a model time of 4.75 hours. The nodes which contain zero values (I) have no channel elements within them, or channel flow has not begun at the model time of 4.75 hours.

This part of the output is useful in determining if the modeled channel is capable of carrying the design runoff without being overtopped. In this case, since the depth of the channel is 9 ft, any value less than 9 ft signifies that the channel is not being overtopped at that point. None of the outputs from this study contain channel element flows that overtopped their banks. Similarly, none of the outputs indicate overtopping of either the East Maricopa Floodway or the RWCD Canal.

SECTION V

FLOOD INUNDATION MAPPING

METHODOLOGY

The Flood Inundation Maps were prepared from the DHM model output. The first item to be mapped was the location and orientation of the 250 model grids (also called nodes) as described in Section IV. The Flood Inundation Maps were then prepared to present four descriptors of the magnitude and the extent of the flood inundation:

1. Arrival Time
2. Personal Hazard Zones (presented on the Arrival Time Map)
3. Maximum Depth Contours
4. Maximum Velocity Contours

Each of these map types will be discussed separately. The first two descriptors are presented on the aerial base map while the last two descriptors are presented on the aerial base map with the USGS topography superposed on it.

Arrival Time

Program output was determined at 0.25 hour increments. The time of arrival was plotted at one hour increments, starting at one hour, by examining which grids had a depth greater than 0.0 feet. A small mark was placed in the center of an inundated grid, on a drafting film overlay on the grid map. These marks represented all the grids that were inundated at the hour increment. By drawing a contour line through the middle of the outside inundated grids, the time of flood wave arrival was mapped. All grids within the boundary of the flood wave are inundated as well. The arrival time contours coincide with the zero depth contours.

Personal Hazard Zones

A personal hazard zone is defined as an area where the flow depth is 2 feet or greater, or where the product of the flow depth times the velocity (DV product) is 7 or greater. An adult would have difficulty wading in an inundated area if the DV product is greater than 9, while a product of 7 is often used in dam safety related studies. A flow depth of 2 feet or more, regardless of the velocity, presents an evacuation problem for children and adults. Flow depths greater than 2 feet were easily found in the DHM summary table entitled "Maximum Water Surface Values for Flood Plain". The depth and velocity values used in the DV product calculation were taken from the DHM output, at the concurrent time. In other words, the maximum velocity was not multiplied with the maximum depth to obtain the DV product, since the maximum depth does not necessarily occur at the same time as the maximum velocity. The DV calculation was performed for several different time values and mapped whenever the DV product first equalled or exceeded 7.0. Both the Arrival Time and Personal Hazard Zones are presented together on a map found in the rear folder of this report.

It should be understood that the Personal Hazard Zones do not necessarily occur at the first flood wave Arrival Time. They are presented together so that the Department of Civil Defense and Emergency Services may see which general areas need evacuation first (from the Arrival Time contours), and then which zones within the general areas (Personal Hazard Zones) require priority attention. It is assumed that once an area is evacuated, it will remain so until all flood waters have receded.

Maximum Depth Contours

The Maximum Depth Contours were obtained from a summary table contained at the end of the DHM output. The maximum flow depth is tabulated for any point in time and is an average for that grid, as output by the DHM model. The maximum flow depth for each grid was placed on the map at the center of each grid, and then a contour map prepared. Water surface elevations, if desired, may be obtained by adding the flow depth to the ground elevation, realizing that the depth is an average for the grid. All values were rounded to the nearest tenth of a foot.

Maximum Velocity Contours

The DHM output was scanned to determine the maximum velocity for each grid. It was discovered that the maximum velocity did not necessarily occur at the time of maximum depth. The velocity values were placed on the map at the center of each grid, similar to a spot elevation in a topographic survey, and then a contour map prepared.

SOCIAL AND ECONOMIC IMPACT AREAS

The social and economic impact areas are places that would require evacuation first or, at the least, close monitoring during a flood event. Any of these places within a Personal Hazard Zone should be evacuated immediately. They comprise five major categories of facilities:

1. Hospitals
2. Emergency Medical Centers
3. Private and Public Schools
4. Nursing Homes
5. Major Shopping Centers

Major shopping centers are those that could contain a significant number of people within them. Some of the mapped Emergency Medical Centers are not open on a 24-hour basis. Because the PMF could theoretically occur at any time of day, it was appropriate to map all Emergency Medical Centers.

The names and locations of the social and economic impact facilities have been placed on the aerial base maps. By finding these facilities on each of the three types of Flood Inundation Maps, the effect of the breach or Emergency Spillway release can be seen.

The Maricopa County Department of Civil Defense and Emergency Services will prepare the necessary evacuation plans and emergency preparedness plans, based upon the technical information supplied by the Flood Inundation Maps. To assist this end, Table V-1 has been prepared:

TABLE V-I

SOCIAL AND ECONOMIC IMPACT AREAS

Name	Address	Phone Number
1. Hospitals		
Valley Lutheran Hospital	6444 East Baywood Avenue	981-2000
2. Emergency Medical Centers		
East Mesa Family Care	6550 East Broadway Road	830-1040
Express Care	5520 East Main Street	985-6125
Velda Rose Medical Center	5801 East Main Street	396-3222

TABLE V-1 - (CONTINUED)

SOCIAL AND ECONOMIC IMPACT AREAS

Name	Address	Phone Number
3. A. Public Schools		
Falcon Hill Elementary School	1645 North Sterling	
Fremont Junior High School	1001 North Power Road	396-1701
Jefferson Elementary	120 South Jefferson Avenue	396-1740
Mendoza Elementary	5831 East McLellan Road	396-1730
O'Connor Elementary	4840 East Adobe Road	396-1735
Red Mountain High School	7301 East Brown Road	396-1800
Salk Elementary	7029 East Brown Road	396-1720
Shepherd Junior High School	1407 North Alta Mesa Drive	981-0983
3. B. Private Schools		
Desert Shadow Montessori School	6709 East University Drive	830-5887
East Mesa Seventh Day Adventist School	111 North Sunvalley Boulevard	981-1554
Holy Cross Catholic Church Private School	1244 South Power Road	981-2021
Jeanne Wright's School of Dance	6336 East Brown Road	396-8864
La Petite Academy	4909 East Brown Road	981-9196
Love of Christ Lutheran School	1525 North Power Road	981-6199
Self-Development Preschool	1721 North Greenfield Road	396-3522
4. Nursing Homes		
Chula Vista Nursing Home	60 South 58th Street	832-3903
Good Shepherd Villa	5848 East University Drive	981-0098
Hearthstone of Mesa	215 South Power Road	985-6992
Las Flores Nursing Center	6458 East Broadway Road	832-5160
Mi Casa Nursing Center	330 South Pinnacle Circle	981-0687
5. Major Shopping Centers		
Buckhorn Plaza Shopping Center	6000 to 6200 East Main Street	
Superstition Springs Shopping Center	6555 East Southern Avenue	

RESULTS

The results of the Flood Inundation Mapping are discussed briefly. Unless otherwise noted, all of the Personal Hazard Zones occur within the first hour after the breach occurs or the Emergency

Spillway begins operation. For Location A, the flooding ended at the sink near the Salt River. For Locations B and C, the downstream flooding ended when flow entered the East Maricopa Floodway. The DHM output used to prepare the Flood Inundation Maps is found in Appendix B.

Emergency Spillway Operation - Location "A", Sheets 1-1 to 1-4

The operation of the Emergency Spillway places a large area within Personal Hazard Zones. The majority of these zones are due to high velocity such that the DV product is larger than 7, and a few zones are due to depths greater than 2 feet. Some Personal Hazard Zones occur later than one hour. Flooding depth varies up to 5.7 feet. The maximum computed velocity is 13.8 feet per second. The flood wave reaches the Salt River sink between 2 and 3 hours.

Breach - Location "A", Sheets 1-1 and 1-5 to 1-7

Breach Location A contains 5 Personal Hazard Zones. The zone immediately downstream of the breach is subject to high velocities and therefore a DV product of 7 or greater. The zone near Thomas Road and Higley Road is also subject to high velocities. The other Personal Hazard Zones are in areas where the flow depth is greater than 2 feet. The sink at the Salt River is reached by the second hour of flow from the breach. The maximum calculated velocity is 11.5 feet per second and the maximum depth of 5.4 feet.

Breach - Location "B", Sheets 2-1 to 2-4

A small area immediately downstream of the breach at Location B is within a Personal Hazard Zone. The area is partly undeveloped and partly developed with single family residential dwellings. There are no identified Social and Economic Impact areas in this Personal Hazard Zone. Shortly after two hours, the flood wave reaches the East Maricopa Floodway. The maximum depth for Location B is 1.8 feet, and the maximum velocity is 7.4 feet per second.

Breach - Location "C", Sheets 3-1 to 3-4

There is a long, relatively narrow Personal Hazard Zone immediately downstream of the breach at Location C, which encompasses a mobile home community and residential dwellings. None of the identified Social and Economic Impact Areas are within the Personal Hazard Zone. The flood wave reaches the East Maricopa Floodway between 2 and 3 hours after the breach. The maximum depth of flow is 2.1 feet. The maximum velocity was determined by the DHM program to be 8.8 feet per second.

Generalized Breach Inundation Map, Sheet 1 of 1

A potential breach could occur at locations other than those modeled. The ground topography is such that potential breaches occurring north of McDowell Road would probably flow to the Salt River. Potential breaches south of McDowell Road would flow to the EMF or to the RWCD Canal and then south along the canal's 3-foot high embankment. Near Brown Road, breach flow would enter the East Maricopa Floodway. In order to identify Personal Hazard Zones and arrival times for potential breaches south of McDowell Road, a Generalized Breach Inundation Map was prepared. Instructions for use are found on this map, which is contained in the rear folder.

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APPENDIX A

PMP ESTIMATION BY HMR-49

General-storm PMP computations for the Colorado River and Great basin

Drainage Spook Hill FRS Area 13.56 mi^2 ~~(km^2)~~

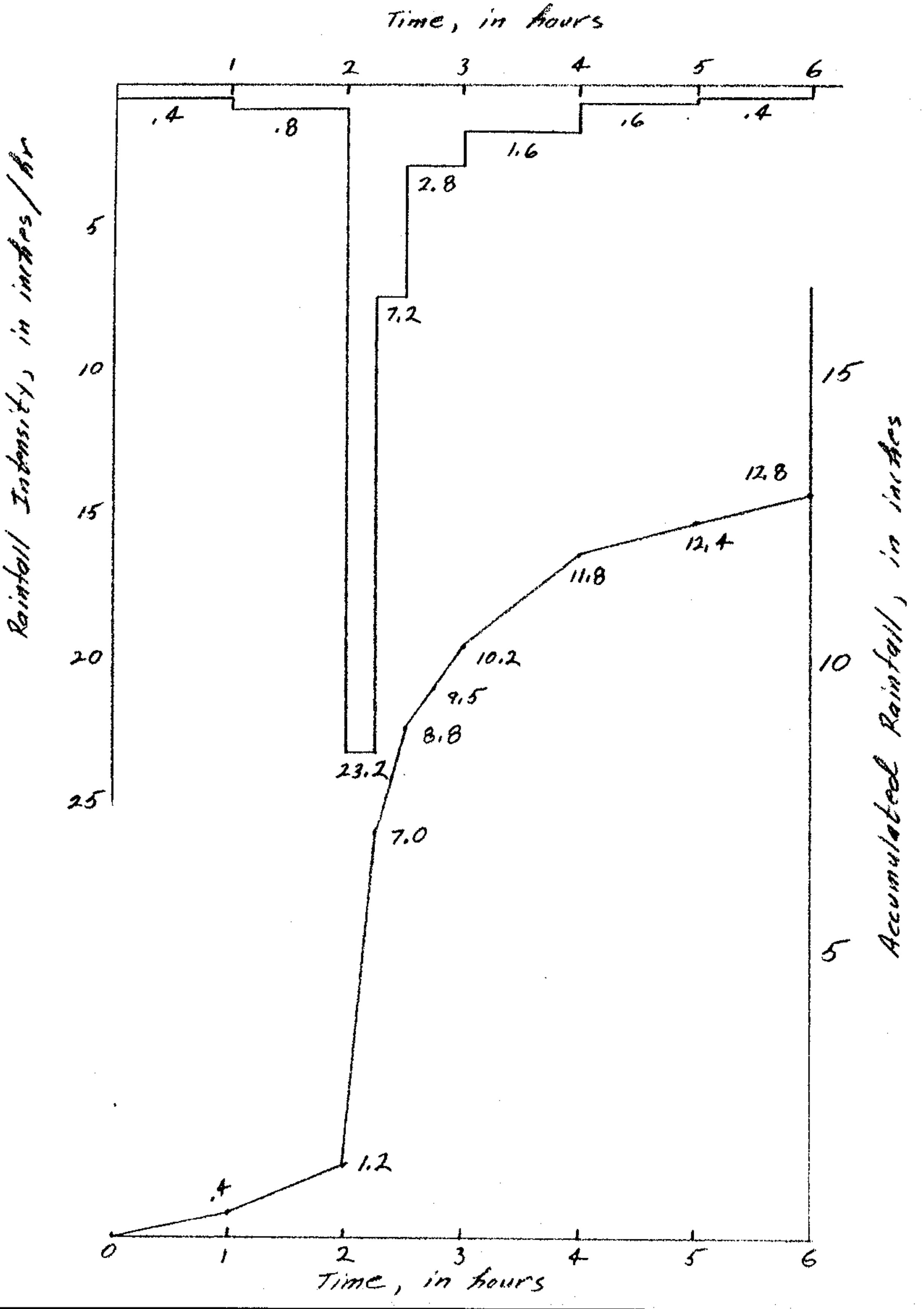
Latitude 33° 30', Longitude 111° 37' of basin center

Month August

Step		Duration (hrs) 6	12	18	24	48	72	
A.	**Convergence PMP**							
1.	Drainage average value from one of figures 2.5 to 2.16 14.1 in. ~~(mm)~~							
2.	Reduction for barrier-elevation [fig. 2.18] 64 %							
3.	Barrier-elevation reduced PMP [step 1 X step 2] 9.0 in. ~~(mm)~~							
4.	Durational variation [figs. 2.25 to 2.27 and table 2.7].	76	90	96	100	111	115	%
5.	Convergence PMP for indicated durations [steps 3 X 4]	6.8	8.1	8.6	9.0	10.0	10.4	in. ~~(mm)~~
6.	Incremental 10 mi^2 (26 km^2) PMP [successive subtraction in step 5]	6.8	1.3	.5	.4	1.0	.4	in. ~~(mm)~~
7.	Areal reduction [select from figs. 2.28 and 2.29]	100	100	100	100	100	100	%
8.	Areally reduced PMP [step 6 X step 7]	6.8	1.3	.5	.4	1.0	.4	in. ~~(mm)~~
9.	Drainage average PMP [accumulated values of step 8]	6.8	8.1	8.6	9.0	10.0	10.4	in. ~~(mm)~~
B.	**Orographic PMP**							
1.	Drainage average orographic index from figure 3.11a to d. 5.0 in. ~~(mm)~~							
2.	Areal reduction [figure 3.20] 99 %							
3.	Adjustment for month [one of figs. 3.12 to 3.17] 100 %							
4.	Areally and seasonally adjusted PMP [steps 1 X 2 X 3] 4.95 in. ~~(mm)~~							
5.	Durational variation [table 3.~~8~~9]	36	63	84	100	139	157	%
6.	Orographic PMP for given durations [steps 4 X 5]	1.8	3.1	4.2	5.0	6.9	7.8	in. (mm)
C.	**Total PMP**							
1.	Add steps A9 and B6	8.6	11.2	12.8	14.0	16.9	18.2	in. (mm)
2.	PMP for other durations from smooth curve fitted to plot of computed data.							
3.	Comparison with local-storm PMP (see sec. 6.3).							

SHEET ____ OF ____
JOB NO. ____
BY GVS
CHK. ____
DATE 22 Feb 90

JOB Spook Hill FRS
FEATURE PMP Local Storm
DETAIL Hyetograph and Mass Diagram



Local-storm PMP computation, Colorado River, Great Basin and California drainages. For drainage average depth PMP. Go to table 6.3B if areal variation is required.

Drainage Spook Hill FRS Area 13.56 mi^2 ~~(km^2)~~
Latitude 33°30' Longitude 111°37' Minimum Elevation 2,000 ft ~~(m)~~

Steps correspond to those in sec. 6.3A.

1. Average 1-hr 1-mi^2 (2.6-km^2) PMP for drainage [fig. 4.5]. 11.5 in. ~~(mm)~~

2. a. Reduction for elevation. [No adjustment for elevations up to 5,000 feet (1,524 m): 5% decrease per 1,000 feet (305 m) above 5,000 feet (1,524 m)]. 100 %

   b. Multiply step 1 by step 2a. 11.5 in. ~~(mm)~~

3. Average 6/1-hr ratio for drainage [fig. 4.7]. 1.30

	Duration (hr) 1/4	1/2	3/4	1	2	3	4	5	6	
4. Durational variation for 6/1-hr ratio of step 3 [table 4.4].	74	89	95	100	114	121	125	128	130	%
5. 1-mi^2 (2.6-km^2) PMP for indicated durations [step 2b X step 4].	8.5	10.2	10.9	11.5	13.1	13.9	14.4	14.7	15.0	in. (mm)
6. Areal reduction [fig. 4.9].	68	74	76	78	81	82	83	84	85	%
7. Areal reduced PMP [steps 5 X 6].	5.8	7.6	8.3	9.0	10.6	11.4	12.0	12.4	12.8	in. (mm)
8. Incremental PMP [successive subtraction in step 7].				9.0	1.6	.8	.6	.4	.4	in. (mm)
	5.8	1.8	.7	.7	} 15-min. increments					

9. Time sequence of incremental PMP according to:

   Hourly increments [table 4.7]. .4 .8 9.0 1.6 .6 .4 in. (mm)

   Four largest 15-min. increments [table 4.8]. 5.8 1.8 .7 .7 in. (mm)

ACKNOWLEDGEMENTS

The cooperation and effort of various individuals and agencies were key elements in the completion of this Flood Inundation Analysis.

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The input of many other individuals is appreciated.

MKE

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3030 North Central Avenue, Suite 402 Phoenix, Arizona 85012 (602) 248-7702 FAX (602) 248-7851

October 16, 1990

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Dr. Davar Khalili
Flood Control District of Maricopa County
3335 West Durango Street
Phoenix, Arizona 85009

Re: Spook Hill FRS
Flood Inundation Analysis, Contract FCD 88-68

Dear Dr. Khalili:

The following report presents the Flood Inundation Analysis for the Spook Hill Floodwater Retarding Structure (Spook Hill FRS) located in Mesa, Arizona. The magnitude and extent of downslope flooding that would occur from either a piping breach of the embankment or the passage of the Probable Maximum Flood through the Emergency Spillway is contained in this report. The Technical Appendix (Appendix B) contains the computer output used to prepare the Flood Inundation Maps, and is issued as a separate volume. Five (5) copies of the report and four (4) copies of Appendix B - Technical Appendix are submitted for the Flood Control District's use. McLaughlin Kmetty Engineers, Ltd. is pleased to have worked on this challenging project. On behalf of Dr. George Sabol and our other sub-consultants, we appreciate the opportunity to provide professional engineering consulting services to the Flood Control District.

This particular dambreak analysis is unique in its use of the Diffusion Hydrodynamic Model (DHM), a two-dimensional flow model well-suited for lateral flood flow analysis on alluvial fans. The DHM provides output for discrete geographical elements, such as maximum depth and maximum velocity, that is not available from either the NWS or the BOSS DamBrk models. In addition, the DHM model provides output used to calculate the mathematical product of depth and velocity (DV product). The DV product was used to delineate and map Personal Hazard Zones, which should greatly assist the Department of Civil Defense and Emergency Services in their preparation of an Emergency Preparedness Plan for this structure.

The Flood Inundation Mapping results from a breach at Locations B and C showed that certain similarities existed, such that a Generalized Breach Inundation Map was appropriate for potential breaches south of McDowell Road. This valuable generalized information will aid the Department of Civil Defense and Emergency Services.

After receipt of this Report, Maps and Technical Appendices, should you have any questions, please feel free to contact us.

Very truly yours,

McLAUGHLIN KMETTY ENGINEERS, LTD.

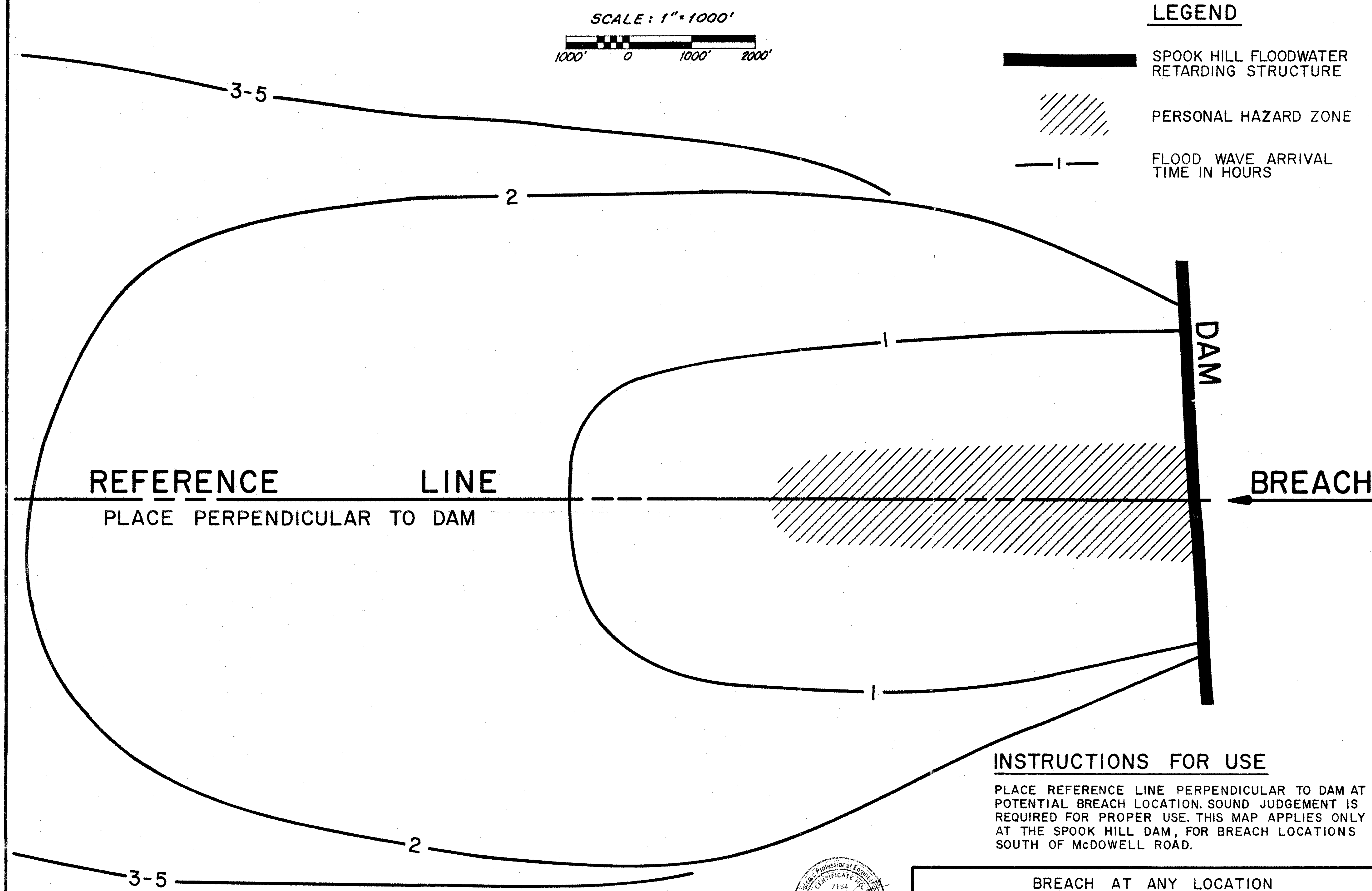
Geza E. Kmetty

Geza E. Kmetty, P.E.
Principal

Frank Edward Brown

Frank Edward Brown, P.E.
Project Engineer

FCDMC\Final.Cvr



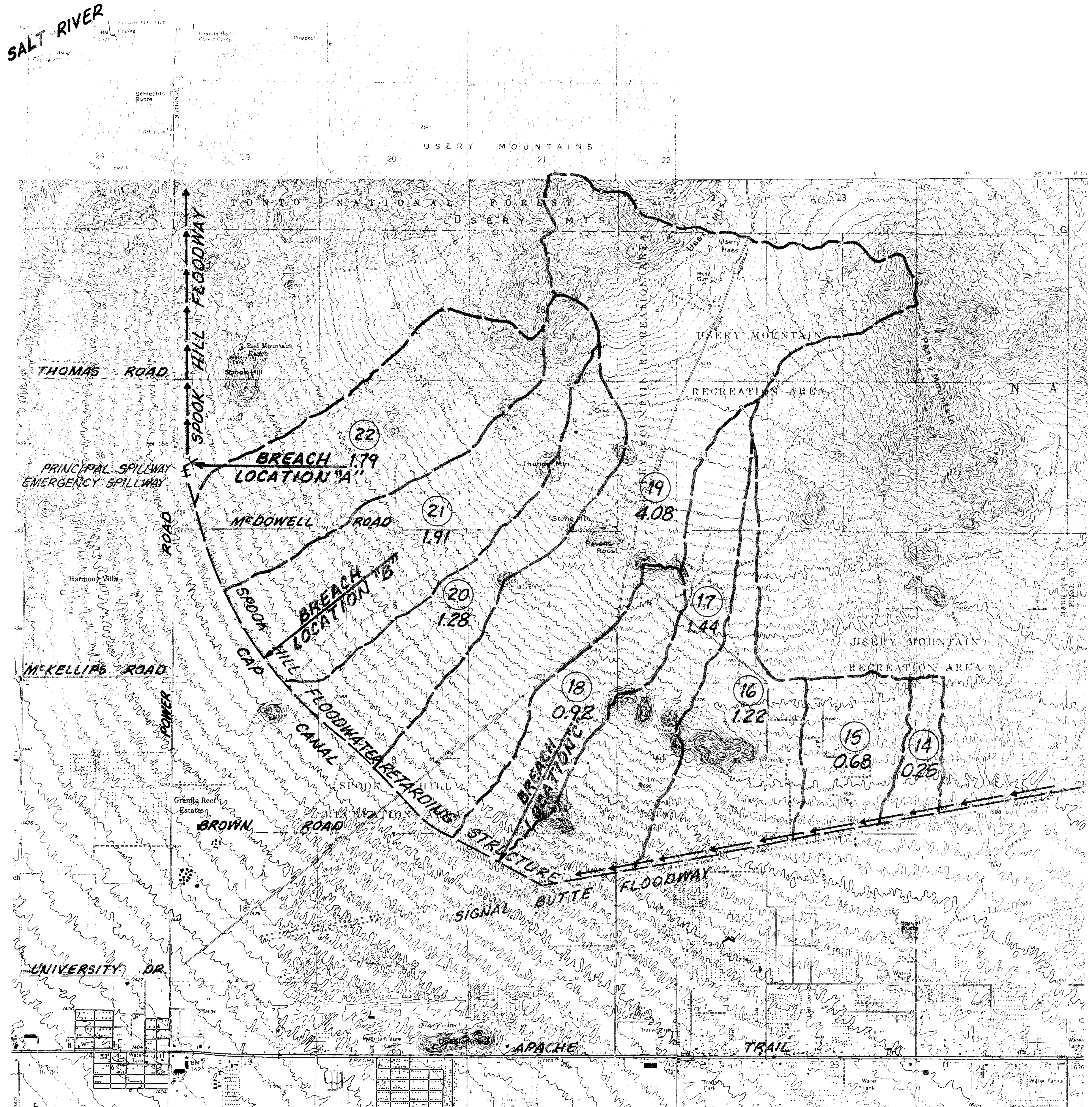
INSTRUCTIONS FOR USE

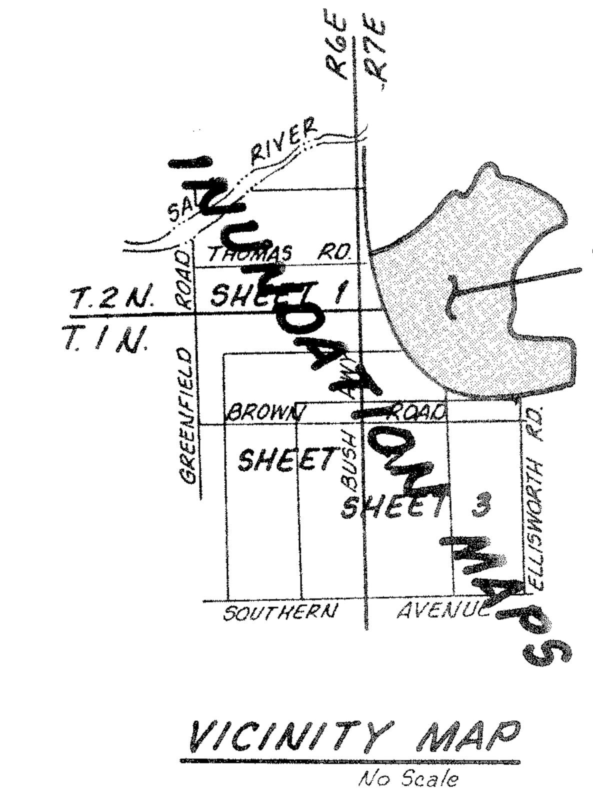
PLACE REFERENCE LINE PERPENDICULAR TO DAM AT POTENTIAL BREACH LOCATION. SOUND JUDGEMENT IS REQUIRED FOR PROPER USE. THIS MAP APPLIES ONLY AT THE SPOOK HILL DAM, FOR BREACH LOCATIONS SOUTH OF McDOWELL ROAD.

NOTE

A PERSONAL HAZARD ZONE IS AN AREA WHERE THE FLOW DEPTH IS 2 FEET OR GREATER OR THE PRODUCT OF DEPTH TIMES VELOCITY IS 7 OR GREATER.

BREACH AT ANY LOCATION PERSONAL HAZARD ZONES AND ARRIVAL TIMES		
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY		
DRAWN BY: R.G.A. DESIGN BY: G.B. CHECK BY: G.B.	SPOOK HILL DAM GENERALIZED BREACH INUNDATION MAP	SHT. 1 OF 1
Date: OCT. 1990		
Project No. 89-407.01	MKE McLAUGHLIN KMETTY ENGINEERS, LTD. 3030 N. Central Ave., Suite 402 Phoenix, AZ. 85012 (602) 248-7702	





VICINITY MAP
No Scale

-N- Scale: 1" = 2000'
0 1000' 2000'
Contour Interval 10'

LEGEND

(18) Subbasin Number
0.92 Drainage Area in Square Miles
Subbasin Boundary
Floodway Flow Direction

NOTE:
Subbasins 14, 15 and 16 are non-contributing drainage areas (See Report.)

Map Reference: Hydrology Section (Map Revised 2/19/75) of Design Report, Spook Hill FRS, PHASE III - Final Design, Soil Conservation Service, December 22, 1976.

BASE MAP: U S G S QUADRANGLE MAPS

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY		
DRAWN BY: K.G.A. DESIGN BY: S.C.S. CHECK BY:	SPOOK HILL DAM WATERSHED MAP	SHT. 1
Date: OCT. 1990 Project No. 89-407.01	MKE McLAUGHLIN KMETTY ENGINEERS, LTD. 3030 N. Central Ave., Suite 402 Phoenix, AZ. 85012 (602) 248-7702 Phoenix • Bullhead City • Flagstaff	OF 1

FIGURE II-1


BASE MAP: KENNEY AERIAL MAPPING PHOTOS (FLOWN FEB. 1990) AND USGS QUADRANGLE MAPS
SHEET INDEX
No Scale
SALT RIVER
THOMAS RD.
SHEET 1
SHEET 2
SHEET 3
T2N
T1N
R6E
R7E
BROWN
SOUTHERN AVENUE
SCALE: 1" = 1000'
1000' 0 1000' 2000'
CONTOUR INTERVAL = 10'
LEGEND
EDGE OF FLOODWAY OR SINK
SPOOK HILL FLOODWATER RETARDING STRUCTURE
FLOODWAY FLOW DIRECTION
LOCATION OF ECONOMIC AND SOCIAL IMPACT AREAS
INDEX CONTOUR
INTERMEDIATE CONTOUR
GRID KEY
GRID NO.
GRID ELEVATION
AVG. MANNINGS "N" FOR GRID
WATER SINK NEAR SALT RIVER
123 1305
A
.025 SINK
BREACH LOCATION "A"
PRINCIPAL SPILLWAY
EMERGENCY SPILLWAY
EMERGENCY SPILLWAY OPERATION LOCATION
TOP OF DAM ELEV. 1591.0
McDowell Road @ Dam Station
SPOOK HILL FLOODWATER RETARDING STRUCTURE
SELF-DEVELOPMENT PRESCHOOL
FALCON HILL ELEMENTARY SCHOOL
LOVE OF CHRIST LUTHERAN PRESCHOOL
SHEPHERD JUNIOR HIGH SCHOOL
McKELLIPS
McDOWELL
McLELLAN
VAL VISTA
DRIVE
HIGLEY
STREET
RECKER
ROAD
HERMOSA
FALCON FIELD MUNICIPAL AIRPORT
LOCATION "A" DHM GRIDS
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
SPOOK HILL DAM
FLOOD INUNDATION MAP
SHT. 1-1
OCT. 1990
89-407.01
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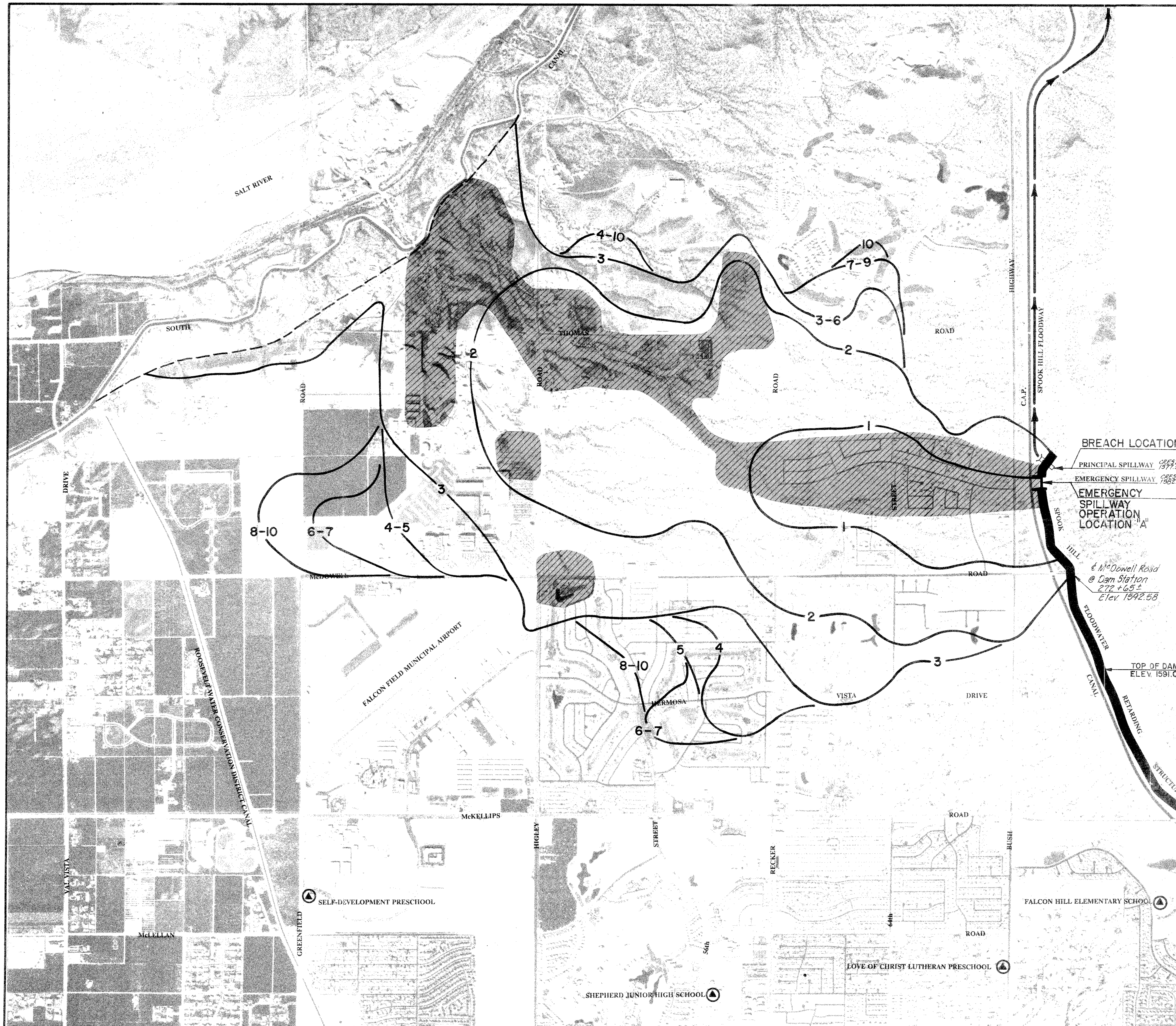
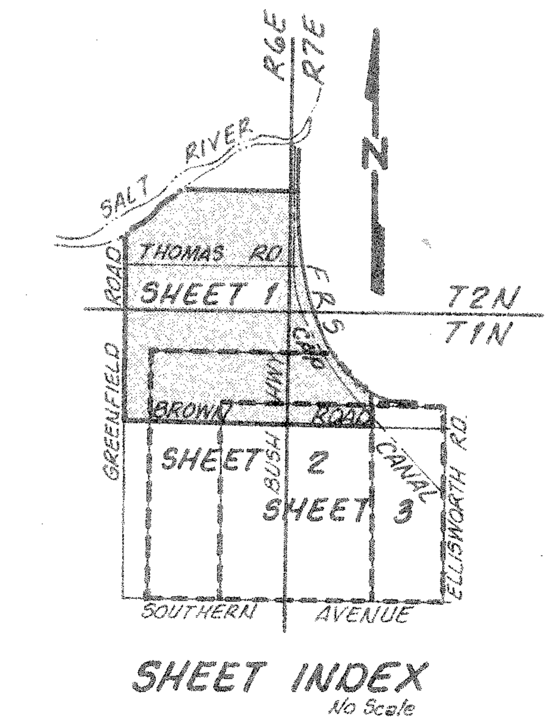


BASE MAP: KENNEY AERIAL MAPPING PHOTOS (FLOWN FEB. 1990) AND USGS QUADRANGLE MAPS



SHEET INDEX
No Scale

N

SCALE: 1" = 1000'

1000' 0 1000' 2000'

LEGEND

EDGE OF FLOODWAY OR SINK

SPOOK HILL FLOODWATER RETARDING STRUCTURE

FLOODWAY FLOW DIRECTION

LOCATION OF ECONOMIC AND SOCIAL IMPACT AREAS

PERSONAL HAZARD ZONE

FLOOD WAVE ARRIVAL TIME IN HOURS

NOTE

A PERSONAL HAZARD ZONE IS AN AREA WHERE THE FLOW DEPTH IS 2 FEET OR GREATER OR THE PRODUCT OF DEPTH TIMES VELOCITY IS 7 OR GREATER. ZONES IN THE SALT RIVER HAVE NOT BEEN SHOWN.

10/10/90

EMERGENCY SPILLWAY OPERATION-LOCATION A
PERSONAL HAZARD ZONES AND ARRIVAL TIMES

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

DRAWN BY: R.G.A. DESIGN BY: GB CHECK BY: GB	SPOOK HILL DAM FLOOD INUNDATION MAP	SHT. 1-2
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Project No. 89-407.01		

EMERGENCY SPILLWAY OPERATION-LOCATION "A"
MAXIMUM DEPTH CONTOURS

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

SPOOK HILL DAM
FLOOD INUNDATION MAP

DRAWN BY RGA
DESIGN BY PT
CHECK BY TB
Date OCT 1990
Project No. 89-407.01

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SHT 1-3

BASE MAP: KENNEY AERIAL MAPPING PHOTOS (FLOWN FEB. 1990) AND USGS QUADRANGLE MAPS
SHEET INDEX
No Scale
SALT RIVER
THOMAS RD.
SHEET 1
SHEET 2
SHEET 3
BROWN ROAD
SOUTHERN AVENUE
T2N
T1N
R6E
R7E
N
SCALE: 1" = 1000'
1000' 0 1000' 2000'
CONTOUR INTERVAL = 10'
LEGEND
EDGE OF FLOODWAY OR SINK
SPOOK HILL FLOODWATER RETARDING STRUCTURE
FLOODWAY FLOW DIRECTION
LOCATION OF ECONOMIC AND SOCIAL IMPACT AREAS
1450 INDEX CONTOUR
INTERMEDIATE CONTOUR
2 MAXIMUM VELOCITY IN FEET PER SECOND
× 1.8 LOCATION OF SPOT VELOCITY (FEET PER SECOND)
BREACH LOCATION "A"
PRINCIPAL SPILLWAY
EMERGENCY SPILLWAY
EMERGENCY SPILLWAY OPERATION LOCATION 'A'
TOP OF DAM ELEV. 1591.0
SALT RIVER
SOUTH CANAL
McDOWELL
McKELLIPS
McLELLAN
HERMOSA
VISTA
HIGLEY
RECKER
STREET
ROAD
DRIVE
VAL VISTA DRIVE
GREENFIELD
ROOSEVELT WATER CONSERVATION DISTRICT CANAL
FALCON FIELD MUNICIPAL AIRPORT
SELF-DEVELOPMENT PRESCHOOL
SHEPHERD JUNIOR HIGH SCHOOL
LOVE OF CHRIST LUTHERAN PRESCHOOL
FALCON HILL ELEMENTARY SCHOOL
SPOOK HILL RETARDING STRUCTURE
× 8.8
× 7.8
× 10.1
× 3.8
× 10.6
× 9.8
× 13.8
× 2.6
× 3.3
EMERGENCY SPILLWAY OPERATION-LOCATION "A"
MAXIMUM VELOCITY CONTOURS
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
DRAWN BY: R.G.A.
DESIGN BY: PA
CHECK BY: SB
Date: OCT. 1990
Project No 89-407.01
SPOOK HILL DAM
FLOOD INUNDATION MAP
SHT. 1-4
MKE McLAUGHLIN KMETTY ENGINEERS, LTD.
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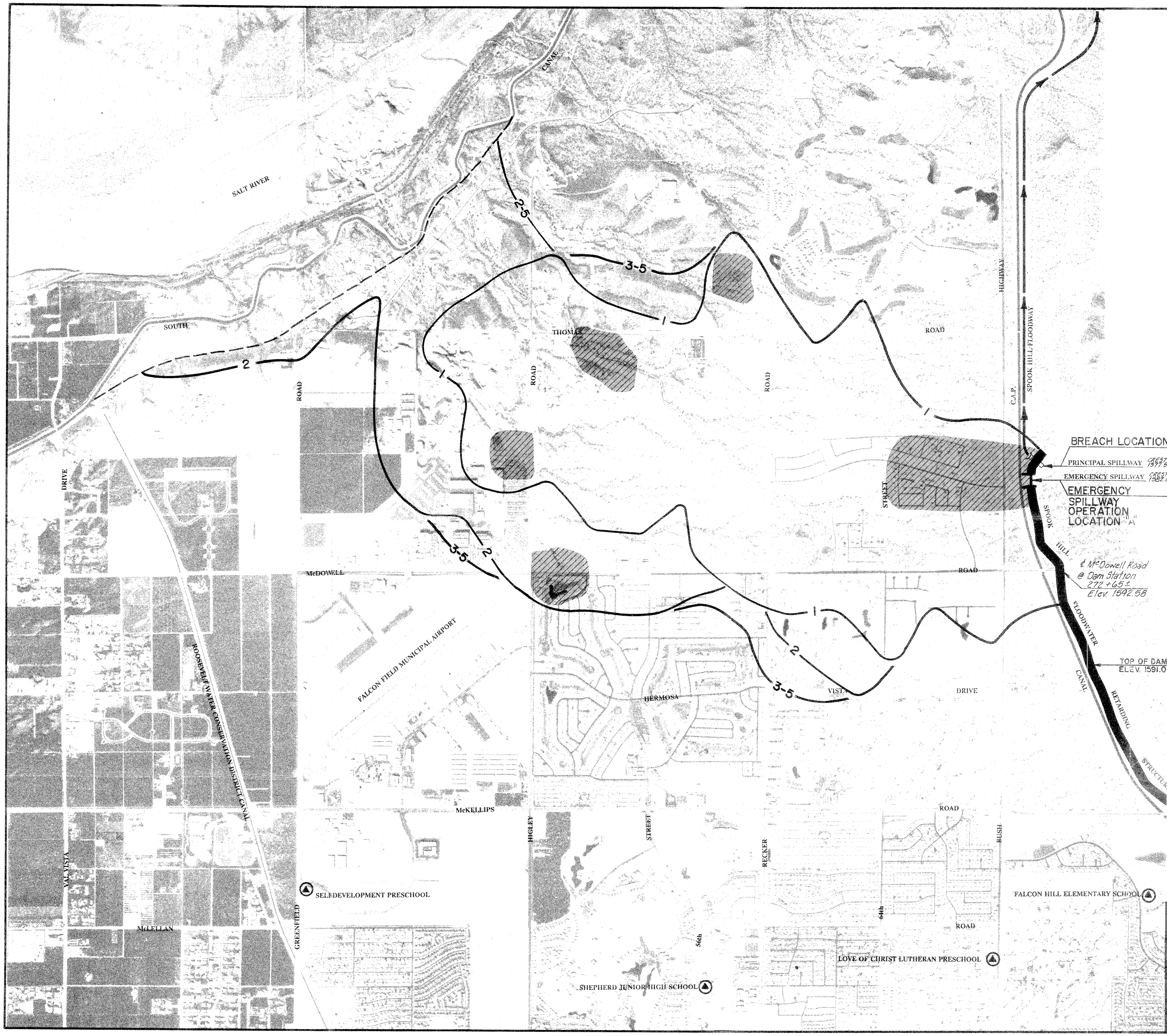
BREACH-LOCATION "A"
MAXIMUM DEPTH CONTOURS

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

SPOOK HILL DAM
FLOOD INUNDATION MAP

DRAWN BY: R.G.A. DESIGN BY: P.E. CHECK BY: P.B.		SHT.
Date OCT. 1990	MKE McLAUGHLIN KMETTY ENGINEERS, LTD.	1-6
Project No. 89-407.01	3030 N. Central Ave., Suite 400 Phoenix, AZ 85012 (602) 248-7702 Phoenix • Bullhead City • Flagstaff	



BASE MAP: KENNEY AERIAL MAPPING PHOTOS (FLOWN FEB. 1990) AND USGS QUADRANGLE MAPS

SHEET INDEX
No Scale

SCALE: 1" = 1000'

LEGEND

- EDGE OF FLOODWAY OR SINK
- SPOOK HILL FLOODWATER RETARDING STRUCTURE
- FLOODWAY FLOW DIRECTION
- LOCATION OF ECONOMIC AND SOCIAL IMPACT AREAS
- PERSONAL HAZARD ZONE
- FLOOD WAVE ARRIVAL TIME IN HOURS

NOTE

A PERSONAL HAZARD ZONE IS AN AREA WHERE THE FLOW DEPTH IS 2 FEET OR GREATER OR THE PRODUCT OF DEPTH TIMES VELOCITY IS 7 OR GREATER. ZONES IN THE SALT RIVER HAVE NOT BEEN SHOWN.

10/10/90

BREACH - LOCATION "A"
PERSONAL HAZARD ZONES AND ARRIVAL TIMES

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

DRAWN BY: R.G.A.
DESIGN BY: FB
CHECK BY: FB
Date: OCT. 1990
Project No. 89-407.01

SPOOK HILL DAM
FLOOD INUNDATION MAP

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SHT. 1-5


BASE MAP: KENNEY AERIAL MAPPING PHOTOS (FLOWN FEB. 1990) AND USGS QUADRANGLE MAPS
SHEET INDEX
No Scale
THOMAS RD.
SHEET 1
SHEET 2
SHEET 3
SOUTHERN AVENUE
T2N
T1N
R6E
R7E
SALT RIVER
SCALE: 1" = 1000'
1000' 0 1000' 2000'
CONTOUR INTERVAL = 10'
LEGEND
EDGE OF FLOODWAY OR SINK
SPOOK HILL FLOODWATER RETARDING STRUCTURE
FLOODWAY FLOW DIRECTION
LOCATION OF ECONOMIC AND SOCIAL IMPACT AREAS
INDEX CONTOUR
INTERMEDIATE CONTOUR
MAXIMUM VELOCITY IN FEET PER SECOND
× 1.8
LOCATION OF SPOT VELOCITY (FEET PER SECOND)
BREACH LOCATION "A"
PRINCIPAL SPILLWAY
EMERGENCY SPILLWAY
EMERGENCY SPILLWAY OPERATION LOCATION "A"
TOP OF DAM ELEV. 1591.0
McDOWELL
McKELLIPS
McLELLAN
HERMOSA
FALCON FIELD MUNICIPAL AIRPORT
ROOSEVELT WATER CONSERVATION DISTRICT CANAL
SELF-DEVELOPMENT PRESCHOOL
SHEPHERD JUNIOR HIGH SCHOOL
LOVE OF CHRIST LUTHERAN PRESCHOOL
FALCON HILL ELEMENTARY SCHOOL
BREACH-LOCATION "A"
MAXIMUM VELOCITY CONTOURS
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
SPOOK HILL DAM
FLOOD INUNDATION MAP
SHT 1-7
DRAWN BY: RGA
OCT. 1990
89-407.01
MKE McLAUGHLIN KMETTY ENGINEERS, LTD
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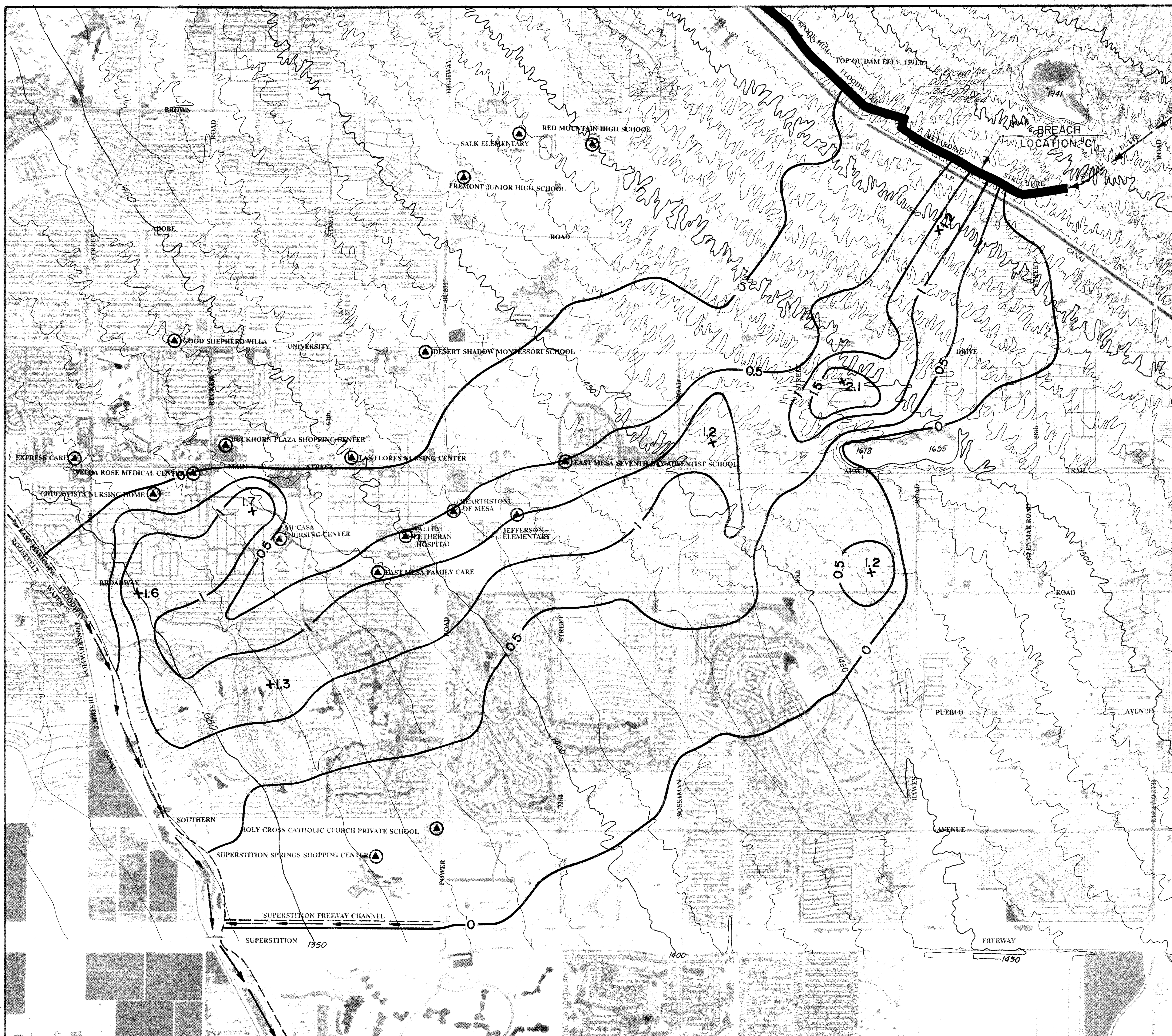

BASE MAP: KENNEY AERIAL MAPPING PHOTOS (FLOWN FEB. 1990) AND USGS QUADRANGLE MAPS.

SHEET INDEX
No Scale

SALT RIVER, THOMAS RD., SHEET 1, BROWN, ROAD, SHEET 2, SHEET 3, GREENFIELD ROAD, BUSH HWY, CANAL, ELLSWORTH RD., SOUTHERN AVENUE, R6E, R7E, T2N, T1N

N

SCALE: 1" = 1000'

1000' 0 1000' 2000'

CONTOUR INTERVAL = 10'

LEGEND

— — —	EDGE OF FLOODWAY OR SINK
▬▬	SPOOK HILL FLOODWATER RETARDING STRUCTURE
——→	FLOODWAY FLOW DIRECTION
⊙	LOCATION OF ECONOMIC AND SOCIAL IMPACT AREAS
—1450—	INDEX CONTOUR
——	INTERMEDIATE CONTOUR
—1—	MAXIMUM DEPTH IN FEET AT ANY POINT IN TIME
× 1.8	LOCATION OF SPOT DEPTH (FEET)

BREACH-LOCATION "C"
MAXIMUM DEPTH CONTOURS

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

DRAWN BY: RGA
DESIGN BY: SR
CHECK BY: 98
Date: OCT. 1990
Project No. 89-407.01

SPOOK HILL DAM
FLOOD INUNDATION MAP

SHT 3-3

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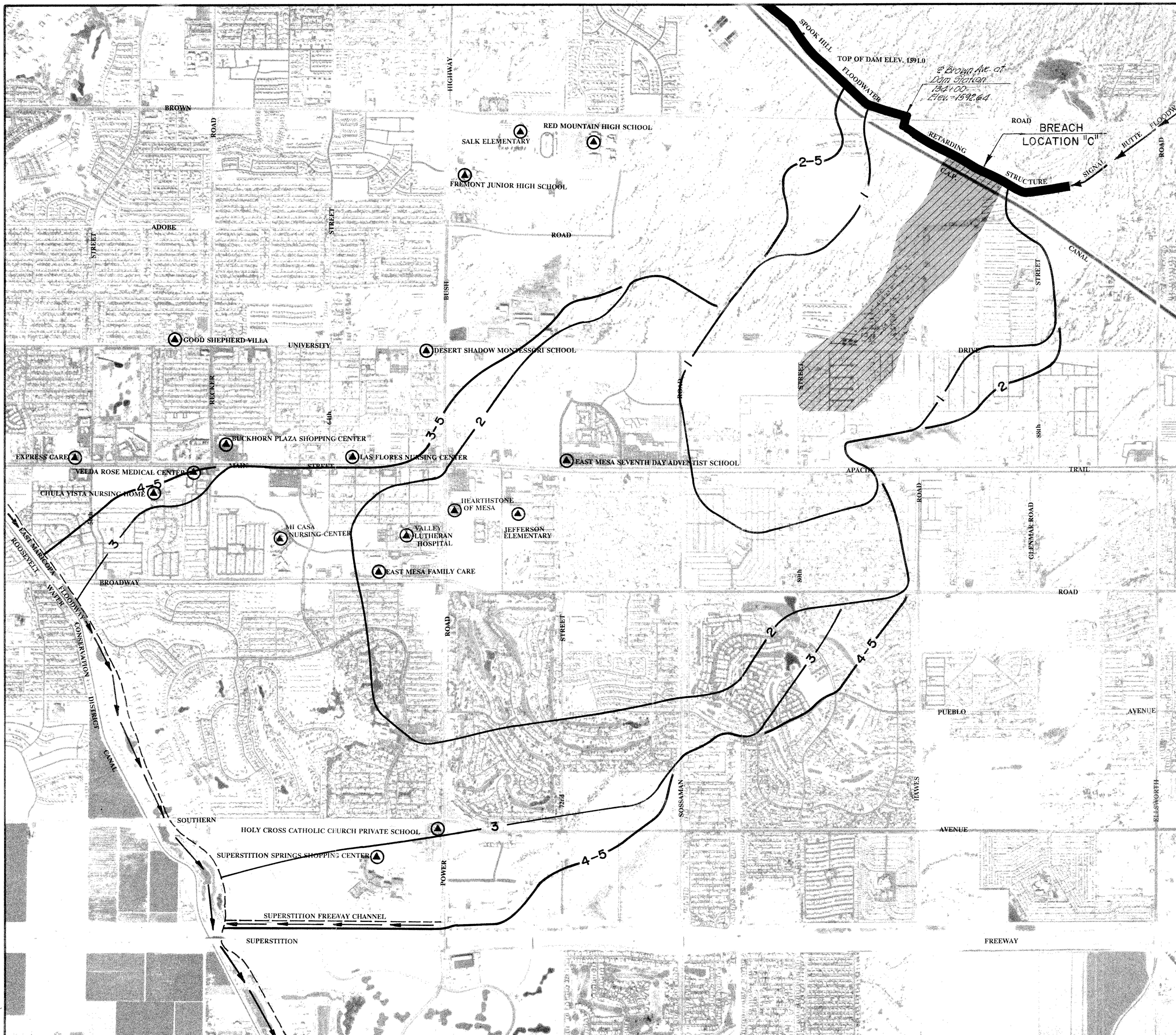
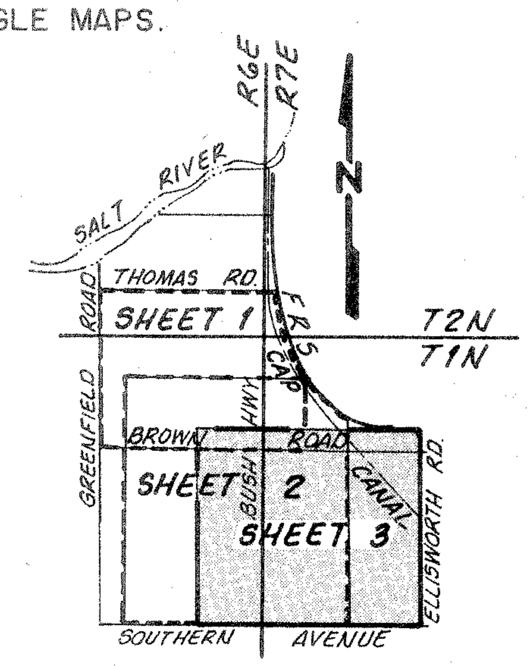


BASE MAP: KENNEY AERIAL MAPPING PHOTOS (FLOWN FEB. 1990) AND USGS QUADRANGLE MAPS.



SHEET INDEX
No Scale

N

SCALE: 1" = 1000'
1000' 0 1000' 2000'

LEGEND

EDGE OF FLOODWAY OR SINK

SPOOK HILL FLOODWATER RETARDING STRUCTURE

FLOODWAY FLOW DIRECTION

LOCATION OF ECONOMIC AND SOCIAL IMPACT AREAS

PERSONAL HAZARD ZONE

FLOOD WAVE ARRIVAL TIME IN HOURS

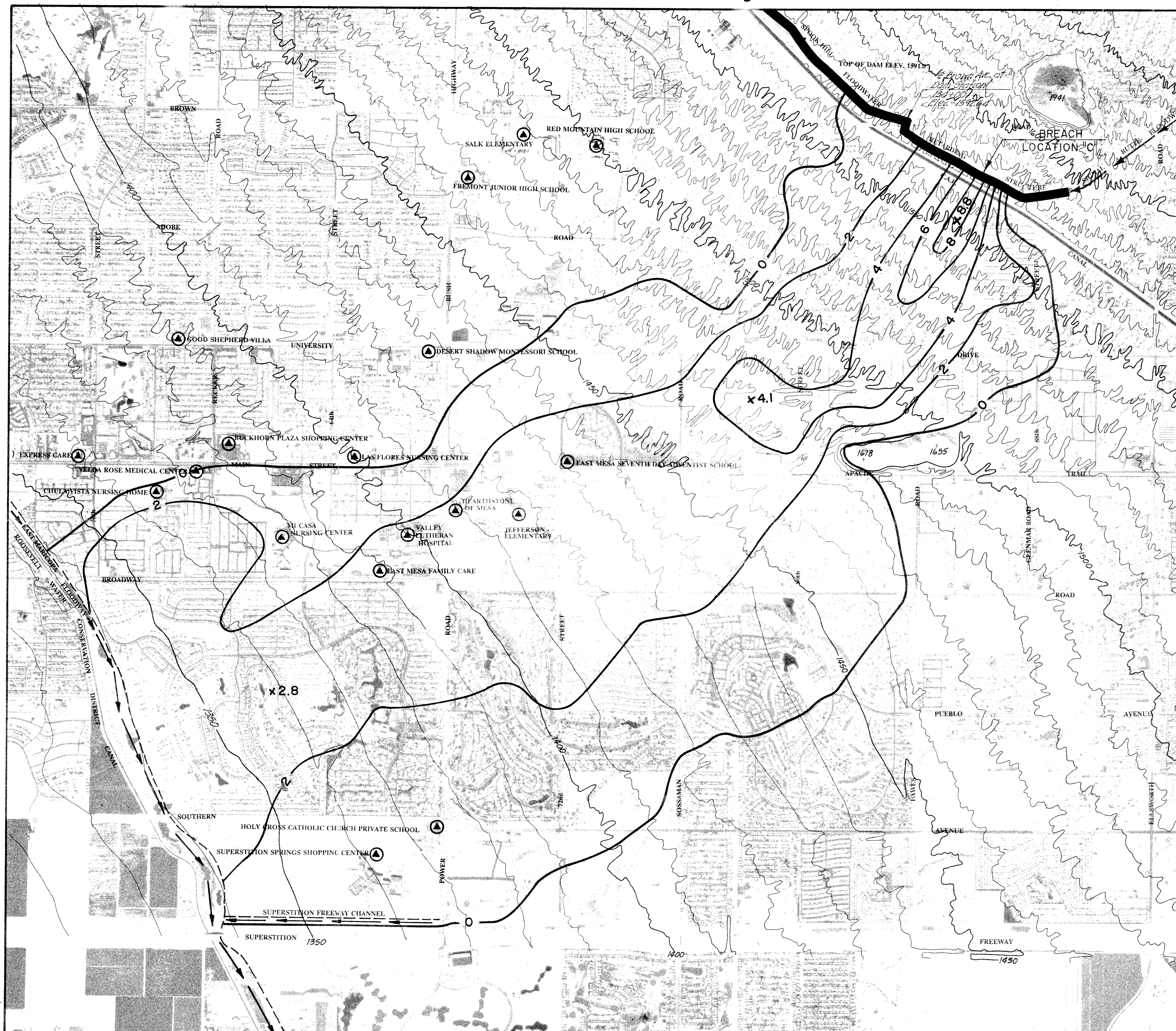
NOTE

A PERSONAL HAZARD ZONE IS AN AREA WHERE THE FLOW DEPTH IS 2 FEET OR GREATER OR THE PRODUCT OF DEPTH TIMES VELOCITY IS 7 OR GREATER.

BREACH-LOCATION "C"
PERSONAL HAZARD ZONES AND ARRIVAL TIMES

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

DRAWN BY: RGA DESIGN BY: 98 CHECK BY: 98	SPOOK HILL DAM FLOOD INUNDATION MAP	SHT. 3-2
Date OCT. 1990	MKE McLAUGHLIN KMETTY ENGINEERS, LTD. 3030 N. Central Ave., Suite 402 Phoenix, AZ. 85012 (602) 248-7702 Phoenix • Bullhead City • Flagstaff	
Project No. 89-407.01		



BASE MAP: KENNEY AERIAL MAPPING PHOTOS (FLOWN FEB. 1990) AND USGS QUADRANGLE MAPS.

SHEET INDEX
No Scale

SHEET 1, SHEET 2, SHEET 3

THOMAS RD, BROWN ROAD, SOUTHERN AVENUE, GREENFIELD ROAD, ELLSWORTH RD, SALT RIVER, T2N, T1N, R6E, R7E

N

SCALE: 1" = 1000'

1000' 0 1000' 2000'

CONTOUR INTERVAL = 10'

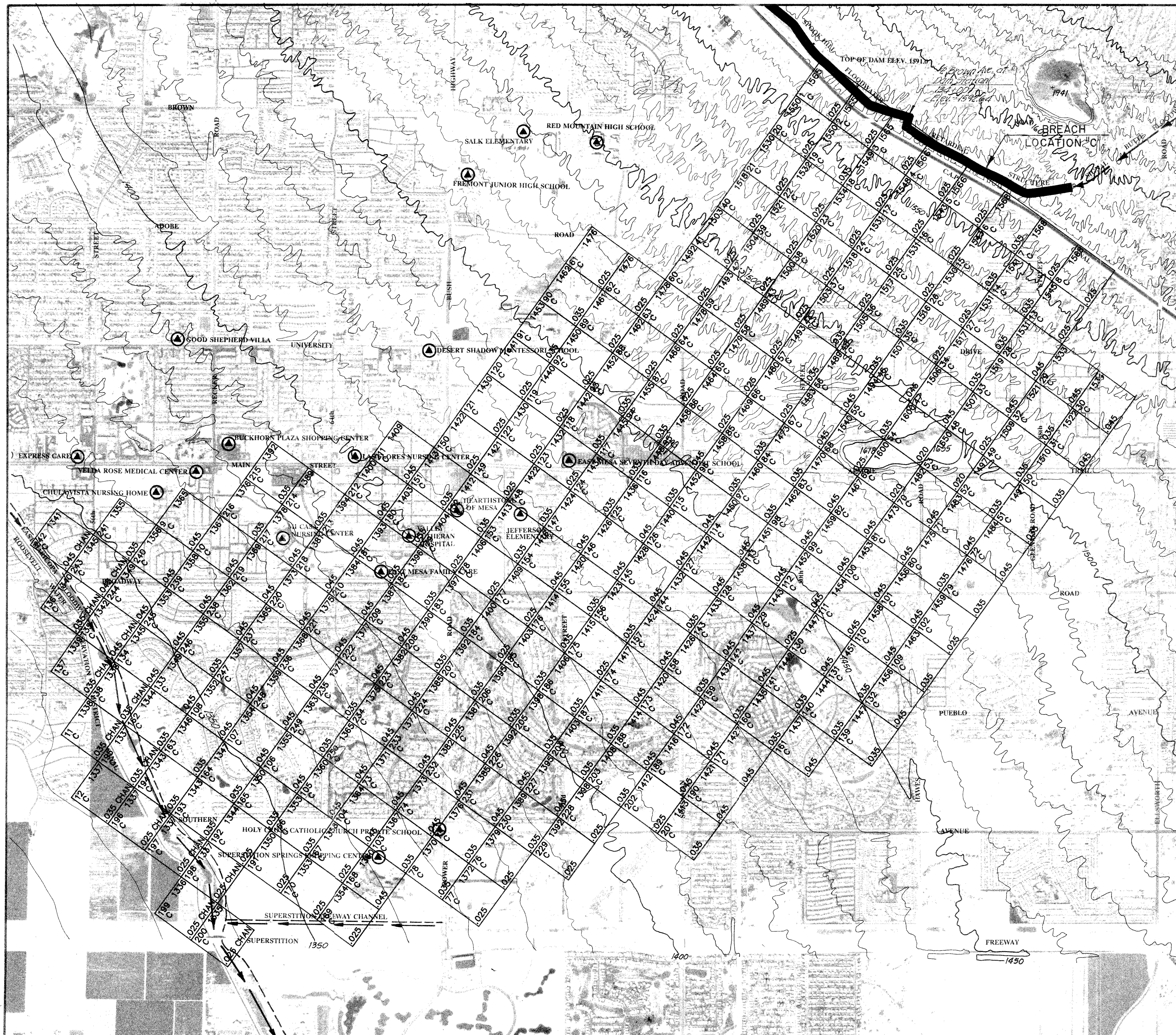
LEGEND

- EDGE OF FLOODWAY OR SINK
- SPOOK HILL FLOODWATER RETARDING STRUCTURE
- FLOODWAY FLOW DIRECTION
- LOCATION OF ECONOMIC AND SOCIAL IMPACT AREAS
- 1450 INDEX CONTOUR
- INTERMEDIATE CONTOUR
- 2 MAXIMUM VELOCITY IN FEET PER SECOND
- × 1.8 LOCATION OF SPOT VELOCITY (FEET PER SECOND)

BREACH - LOCATION "C"
MAXIMUM VELOCITY CONTOURS

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

DRAWN BY: RGA DESIGN BY: AJ CHECK BY: SG	SPOOK HILL DAM FLOOD INUNDATION MAP	SHT. 3-4
Date: OCT. 1990	MKE McLAUGHLIN KMETTY ENGINEERS, LTD. 3030 N. Central Ave., Suite 402 Phoenix, AZ 85012 (602) 248-7702 Phoenix • Bullhead City • Flagstaff	
Project No. 89-407.01		



BASE MAP: KENNEY AERIAL MAPPING PHOTOS (FLOWN FEB. 1990) AND USGS QUADRANGLE MAPS.

SHEET INDEX
No Scale

SCALE: 1" = 1000'

1000' 0 1000' 2000'

CONTOUR INTERVAL = 10'

LEGEND

EDGE OF FLOODWAY OR SINK

SPOOK HILL FLOODWATER RETARDING STRUCTURE

FLOODWAY FLOW DIRECTION

LOCATION OF ECONOMIC AND SOCIAL IMPACT AREAS

1450 INDEX CONTOUR

INTERMEDIATE CONTOUR

GRID KEY

GRID NO. — 123 1432 — GRID ELEVATION
C

AVG. MANNINGS "N" FOR GRID — .025 CHAN — OPEN CHANNEL WITHIN GRID

LOCATION "C" DHM GRIDS

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

DRAWN BY: RGA
DESIGN BY: GB
CHECK BY: GB
Date: OCT. 1990
Project No. 89-407.01

SPOOK HILL DAM
FLOOD INUNDATION MAP

SHT. 3-1

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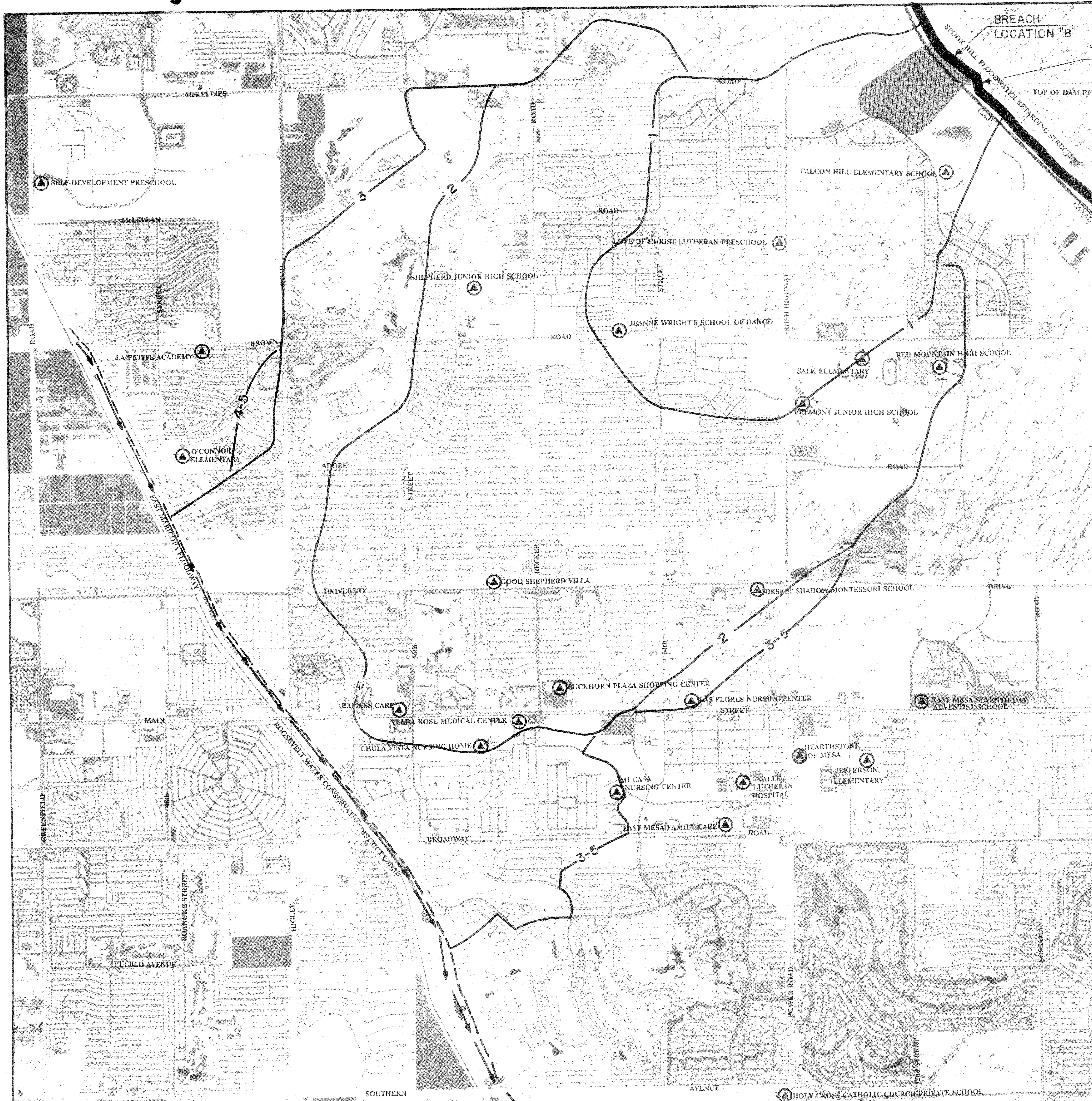
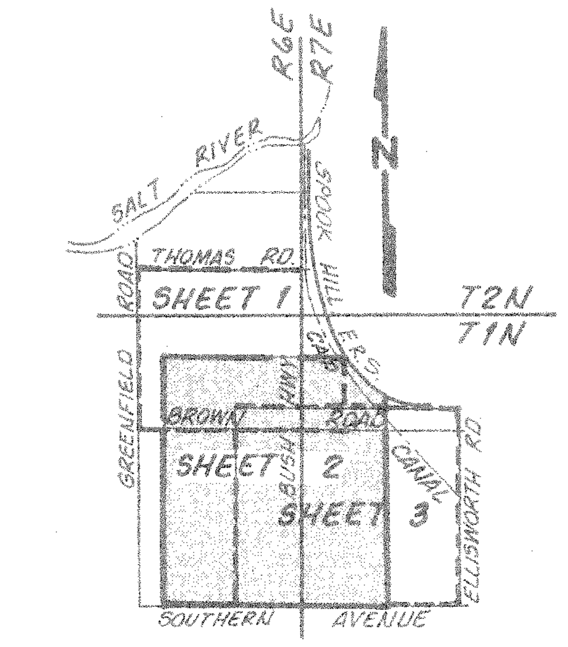


BASE MAP: KENNEY AERIAL MAPPING PHOTOS (FLOWN FEB. 1990) AND USGS QUADRANGLE MAPS.



SHEET INDEX
No Scale

SCALE: 1" = 1000'
1000' 0 1000' 2000'

LEGEND

EDGE OF FLOODWAY OR SINK

SPOOK HILL FLOODWATER RETARDING STRUCTURE

FLOODWAY FLOW DIRECTION

LOCATION OF ECONOMIC AND SOCIAL IMPACT AREAS

PERSONAL HAZARD ZONE

FLOOD WAVE ARRIVAL TIME IN HOURS

NOTE

A PERSONAL HAZARD ZONE IS AN AREA WHERE THE FLOW DEPTH IS 2 FEET OR GREATER OR THE PRODUCT OF DEPTH TIMES VELOCITY IS 7 OR GREATER.

BREACH-LOCATION "B"
PERSONAL HAZARD ZONES AND ARRIVAL TIMES

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

SPOOK HILL DAM
FLOOD INUNDATION MAP

SHT. 2-2

DRAWN BY: RGA
DESIGN BY: RG
CHECK BY: GB
Date: OCT. 1990
Project No. 89-407.01

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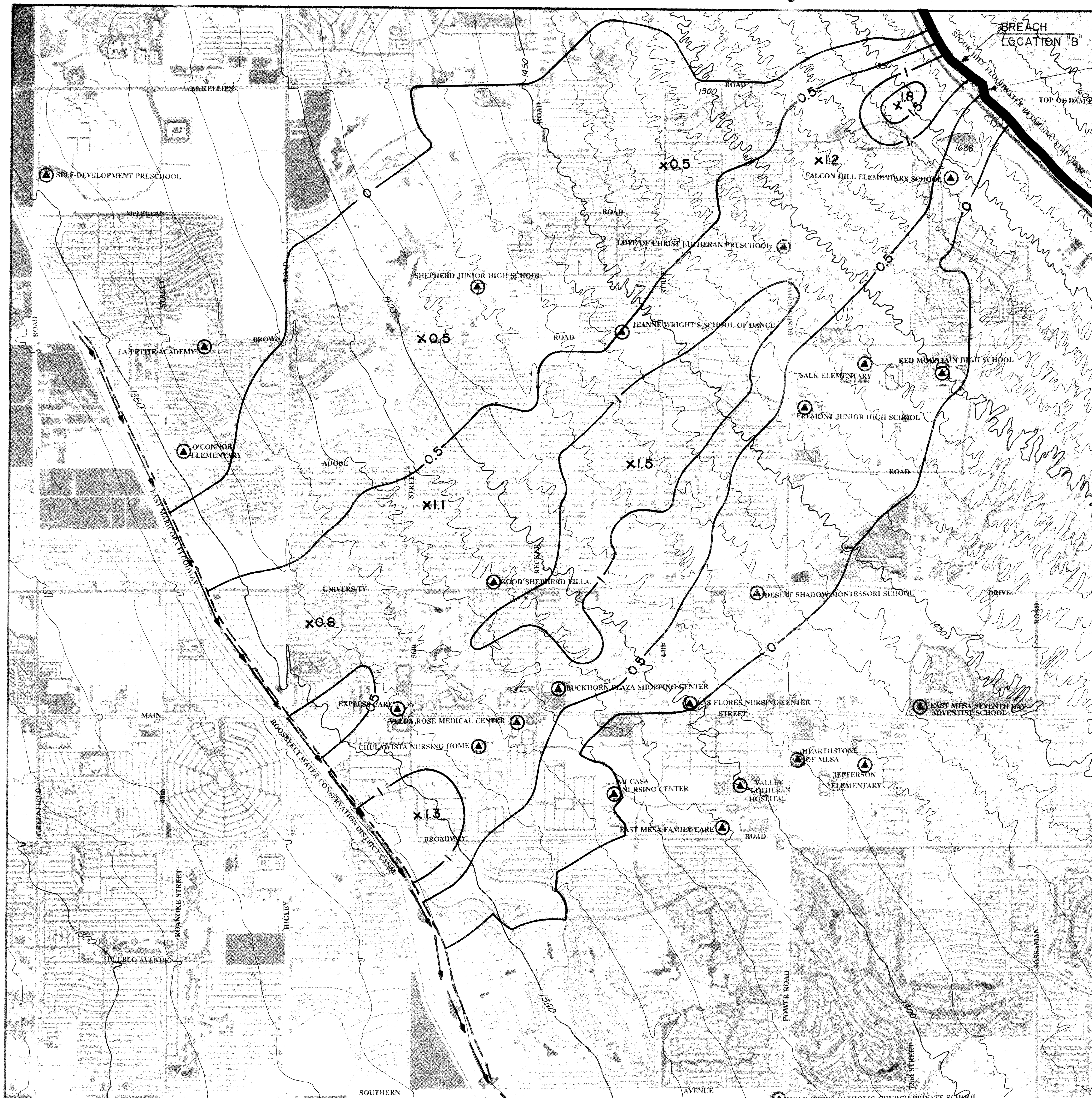
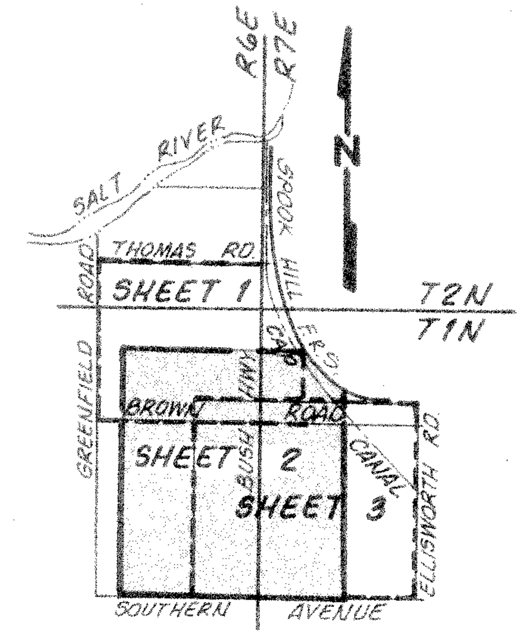


BASE MAP: KENNEY AERIAL MAPPING PHOTOS (FLOWN FEB. 1990) AND USGS QUADRANGLE MAPS.



SHEET INDEX
No Scale

N

SCALE: 1" = 1000'

1000' 0 1000' 2000'

CONTOUR INTERVAL = 10'

LEGEND

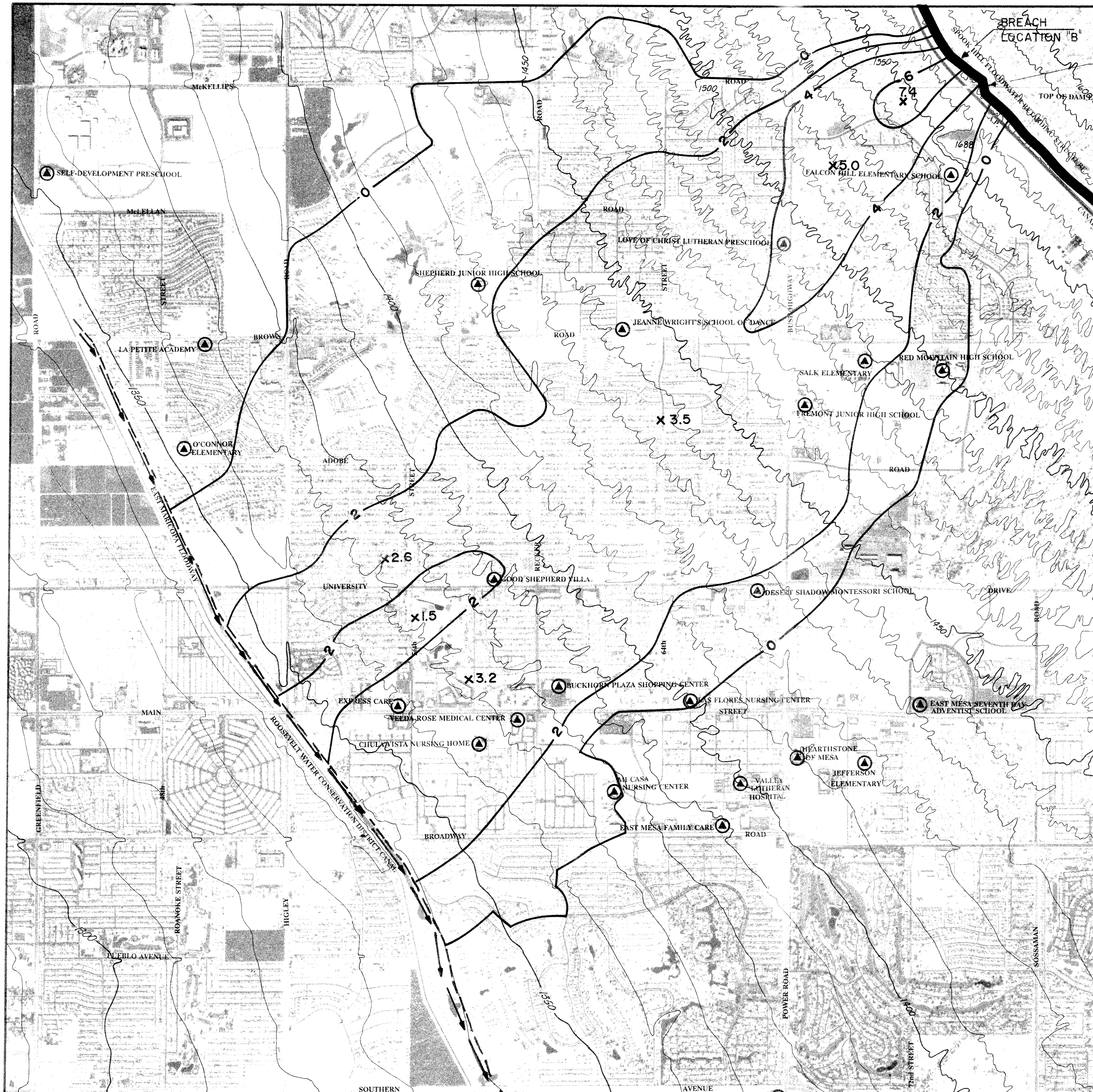
- EDGE OF FLOODWAY OR SINK
- SPOOK HILL FLOODWATER RETARDING STRUCTURE
- FLOODWAY FLOW DIRECTION
- LOCATION OF ECONOMIC AND SOCIAL IMPACT AREAS
- 1450 — INDEX CONTOUR
- INTERMEDIATE CONTOUR
- —1— MAXIMUM DEPTH IN FEET AT ANY POINT IN TIME
- × 1.8 LOCATION OF SPOT DEPTH (FEET)

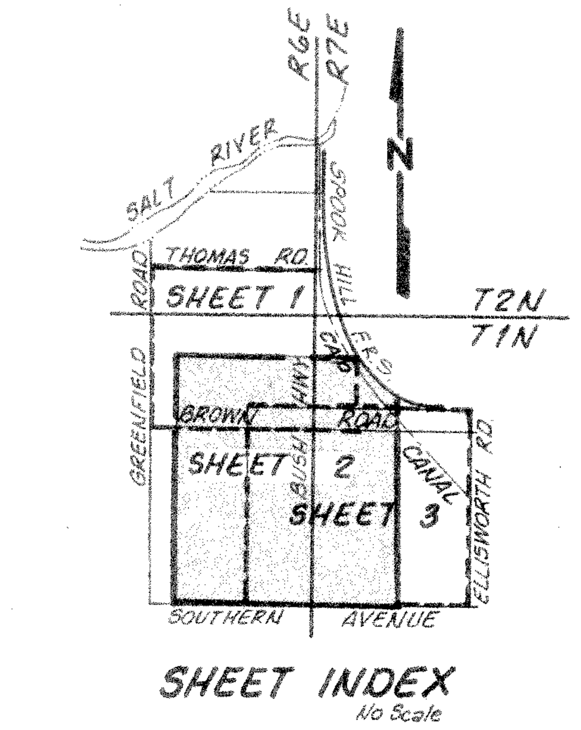
BREACH-LOCATION "B"
MAXIMUM DEPTH CONTOURS

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

DRAWN BY RGA DESIGN BY CHECK BY	SPOOK HILL DAM FLOOD INUNDATION MAP	SHT. 2-3
Date OCT 1990 Project No. 89-407.01	MKE McLAUGHLIN KMETTY ENGINEERS, LTD. 3030 N. Central Ave., Suite 402 Phoenix, AZ 85012 (602) 248-7702 Phoenix • Bullhead City • Flagstaff	

BASE MAP: KENNEY AERIAL MAPPING PHOTOS (FLOWN FEB. 1990) AND USGS QUADRANGLE MAPS.





SHEET INDEX
No Scale

N

SCALE: 1" = 1000'

1000' 0 1000' 2000'

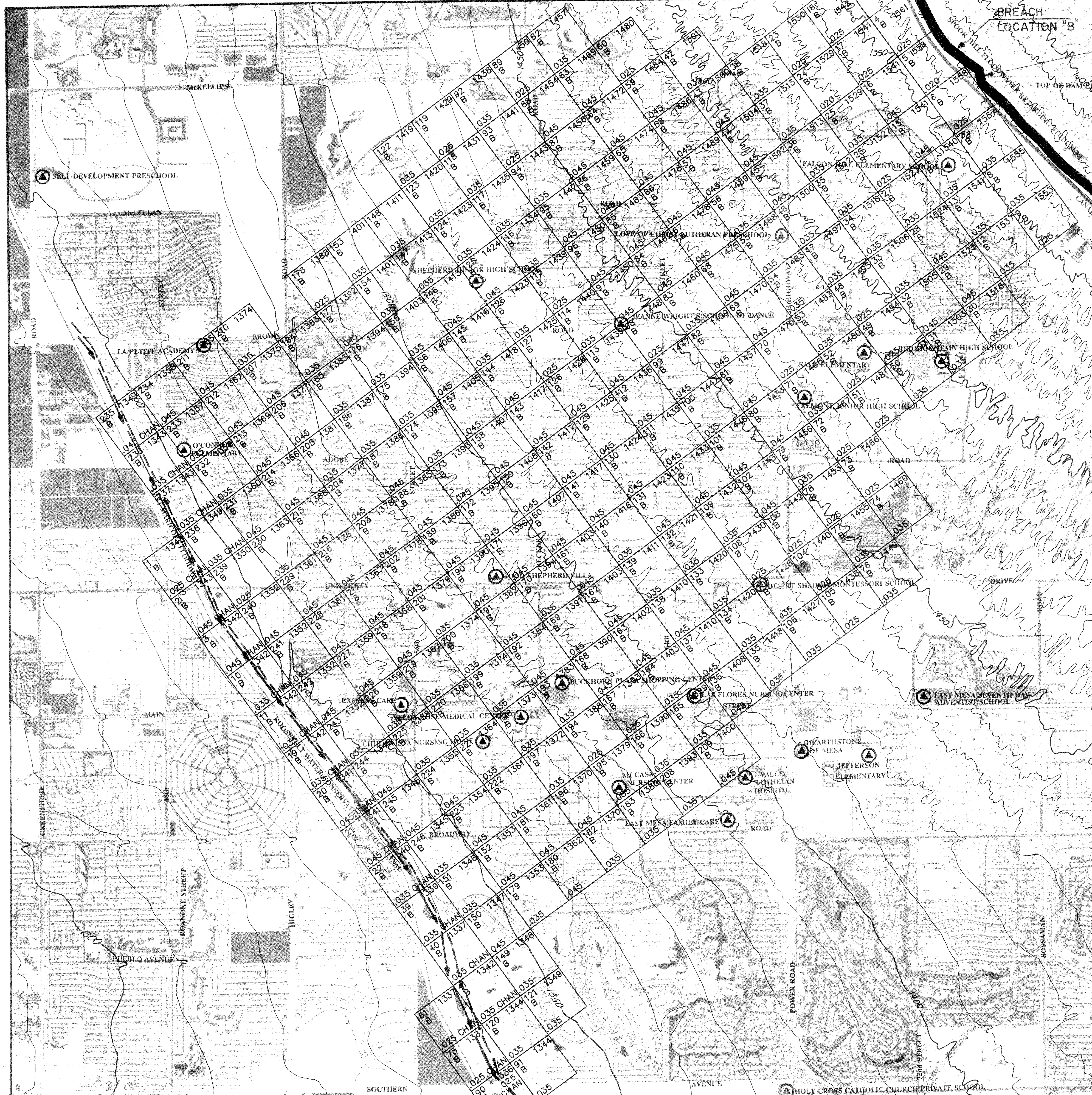
CONTOUR INTERVAL = 10'

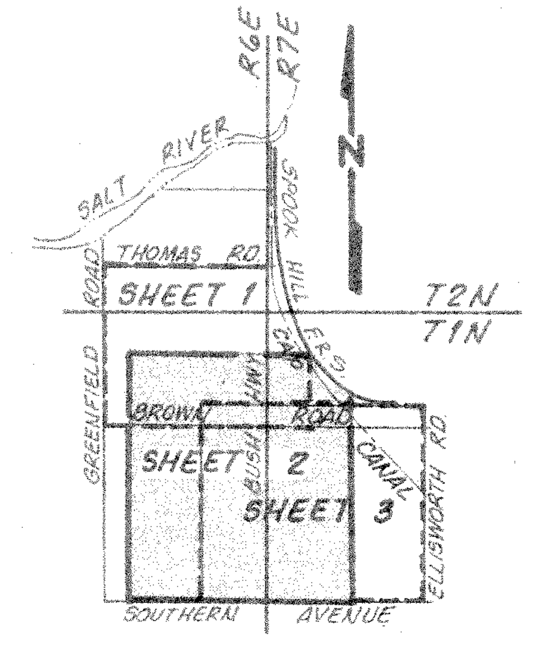
LEGEND

- EDGE OF FLOODWAY OR SINK
- SPOOK HILL FLOODWATER RETARDING STRUCTURE
- FLOODWAY FLOW DIRECTION
- LOCATION OF ECONOMIC AND SOCIAL IMPACT AREAS
- 1450 INDEX CONTOUR
- INTERMEDIATE CONTOUR
- 2 MAXIMUM VELOCITY IN FEET PER SECOND
- × 1.8 LOCATION OF SPOT VELOCITY (FEET PER SECOND)

BREACH-LOCATION "B" MAXIMUM VELOCITY CONTOURS	
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY	
DRAWN BY: RGA DESIGN BY: AS CHECK BY: GB	SPOOK HILL DAM FLOOD INUNDATION MAP — SHT 2-4
Date: OCT. 1990 Project No. 89-407.01	MKE McLAUGHLIN KMETTY ENGINEERS, LTD. 3030 N. Central Ave., Suite 402 Phoenix, AZ 85012 (602) 248-7702 Phoenix • Bullhead City • Flagstaff

BASE MAP: KENNEY AERIAL MAPPING PHOTOS (FLOWN FEB. 1990) AND USGS QUADRANGLE MAPS.





SHEET INDEX
No Scale

N

SCALE: 1" = 1000'

1000' 0 1000' 2000'

CONTOUR INTERVAL = 10'

LEGEND

EDGE OF FLOODWAY OR SINK

SPOOK HILL FLOODWATER RETARDING STRUCTURE

FLOODWAY FLOW DIRECTION

LOCATION OF ECONOMIC AND SOCIAL IMPACT AREAS

1450 INDEX CONTOUR

INTERMEDIATE CONTOUR

GRID KEY

GRID NO. — 123 B — 1420 — GRID ELEVATION

AVG. MANNINGS "N" FOR GRID — .035 CHAN — OPEN CHANNEL WITHIN GRID

LOCATION "B" DHM GRIDS

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

DRAWN BY: RGA
DESIGN BY: AE
CHECK BY: SB

Date: OCT. 1990

Project No. 89-407.01

SPOOK HILL DAM
FLOOD INUNDATION MAP

SHT. 2

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