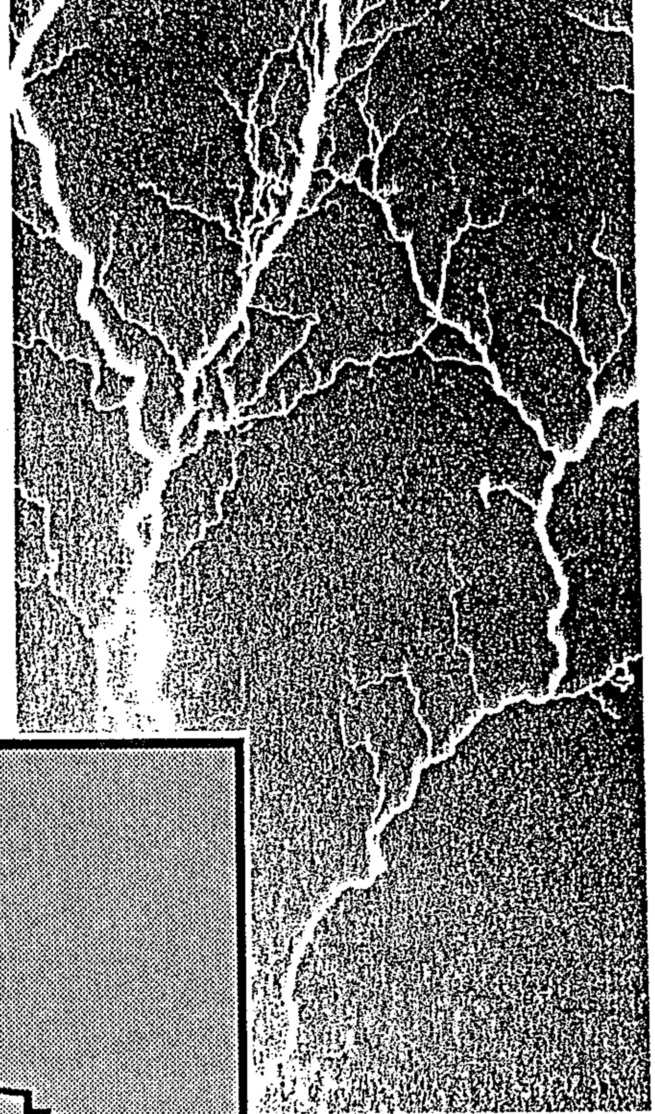
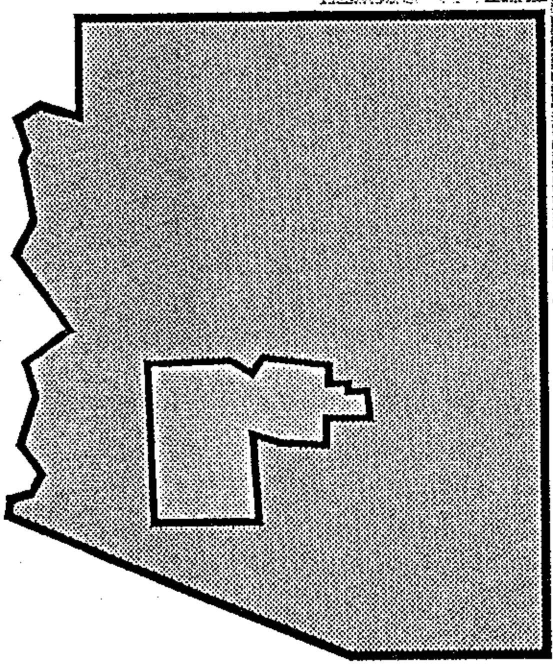
FLOOD INSURANCE STUDY



GILBERT-CHANDLER AREA
Maricopa County, Arizona

Prepared for the
Flood Control District
of
Maricopa County



DRAFT• October 1989
REVISED• July 1990
REVISED• September 1990

Registered Professional Engineer (Civil)
CERTIFICATE NO.
15872
RUSSELL L.
MIRACLE
Date Signed 10/9/91
ARIZONA, U.S.A.

Franzoy•Corey
ENGINEERING COMPANY

FLOOD INSURANCE STUDY

GILBERT-CHANDLER AREA, MARICOPA COUNTY, ARIZONA

SEPTEMBER 1990

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
CONTRACT NUMBER FCD 87-24

Prepared by

FRANZOY COREY
7776 POINTE PARKWAY WEST
SUITE 290
PHOENIX, ARIZONA 85044
602) 438-2200

PAGE

TABLE OF CONTENTS

FIGURE		REFERENCED ON PAGE NUMBER

LIST OF FIGURES

TABLE		REFERENCED ON PAGE NUMBER

LIST OF TABLES

FLOOD INSURANCE STUDY

GILBERT-CHANDLER AREA, MARICOPA COUNTY, ARIZONA

1. INTRODUCTION

1.1 Purpose of Study

This flood insurance study revises and updates a previous flood insurance study/flood insurance rate map for the Town of Gilbert, portions of the cities of Chandler and Mesa in Maricopa County, and some unincorporated areas of Maricopa County, Arizona. This information will be used by the Town of Gilbert, the cities of Chandler and Mesa and Maricopa County for floodplain management of existing and future development. Actuarial rates as a part of the National Flood Insurance Program (NFIP) will be updated. The information will also be used by the Flood Control District of Maricopa County to promote sound land use and floodplain development.

1.2 Authority and Acknowledgments

The sources of authority for this flood insurance study are the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973.

A study was performed by the U.S. Army Corps of Engineers (COE), Los Angeles District, as authorized in section 6 of the Flood Control Act of 1938 which directed the Secretary of the Army to cause surveys for flood control of the Gila River and tributaries in Arizona and New Mexico. The study was completed in September 1977.

A study to update the Gila Floodway Study for the City of Chandler was completed in January 1980. This study was prepared by Harris-Toups Associates for the Federal Insurance Administration under contract no. H-4008. The same contract also provided for a study to update the Gila floodway study for the Town of Gilbert completed in July 1979.

Further updates to the hydrologic and hydraulic analyses for the Gilbert-Chandler Flood Insurance Study area including parts of the City of Mesa and some unincorporated areas of Maricopa County were performed by Franzoy Corey Engineering Company under contract no. FCD 87-24 with the Flood Control District of Maricopa County. Intergovernmental Agreement No. IGA FCD 87040 was entered into by the Town of Gilbert and the Flood Control District on 7 December 1987. This flood insurance study was completed in October 1989.

1.3 Coordination

The results of the 1977, 1979, and 1980 studies were reviewed by representatives from the Flood Control District of Maricopa County, the Town of Gilbert, and the study contractor. Town and county officials provided information pertaining to floodplain regulations, maps of the area, flooding history, and other pertinent information.

The Gilbert-Chandler Flood Insurance Study was initiated on 27 July 1987 at an initial coordination meeting. The meeting was attended by representatives of the Flood Control District, the Town of Gilbert and the study contractor.

Review meetings were held periodically throughout the study. Representatives from the Flood Control District and the Town of Gilbert were in attendance to review the methods and assumptions. The issues raised at these meetings have been resolved to the satisfaction of those in attendance.

2. AREA STUDIED

2.1 Scope of the Study

This flood insurance study covers the incorporated areas of the Town of Gilbert, parts of the cities of Chandler and Mesa, and some unincorporated areas of Maricopa County in Maricopa County, Arizona. The area of the study is shown on the vicinity map (figure 1).

The study area is bounded by the Superstition Freeway (SR 360) on the north, by Hunt Highway (Maricopa County line) the south, the East Maricopa Floodway on the east, and the Southern Pacific Railroad (SPRR) paralleling Arizona Avenue on the west. The study included hydrologic analysis of the entire study area with mapping and delineation of the 100-year floodplain along major hydraulic barriers. Detailed methods were used to complete the analyses. The hydraulic barriers (appendix D) studied within the area are:

1. Eastern Canal from SR 360 to Riggs Road;
2. Consolidated Canal from SR 360 to Hunt Highway (Maricopa County line);
3. SPRR (Rittenhouse alignment) from the east Maricopa floodway to Baseline Road;
4. SPRR (Arizona Avenue alignment) from Baseline Road to Hunt Highway; and,

2.2 Community Description

The communities of Gilbert, Chandler, and Mesa are located in southeastern Maricopa County, Arizona and are approximately 18 miles southeast of Phoenix. The populations of these communities in 1985 were as follows: Gilbert - 12,102; Chandler - 63,855; Mesa - 239,587. The Phoenix metropolitan area has experienced a rapid growth rate. Urban development has affected the flooding conditions.

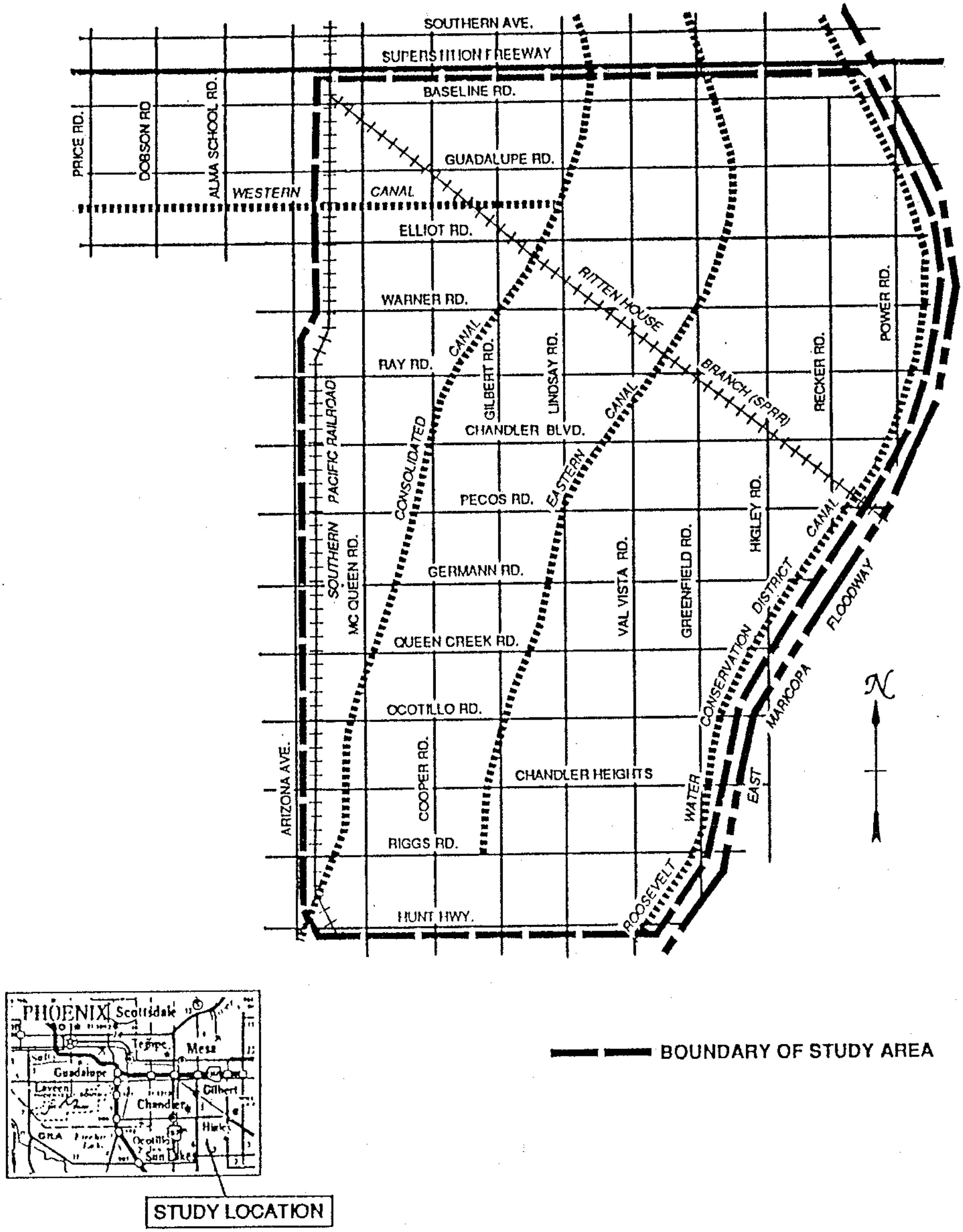


FIGURE 1
STUDY AREA SOUTH OF THE
SUPERSTITION FREEWAY
GILBERT-CHANDLER FLOODPLAIN DELINEATION STUDY

The communities are located in the Gila River watershed on an alluvial bajada formation which forms a mesa above the Salt River Valley and drains generally to the west and southwest away from the Salt River. The formation extends easterly from the Superstition Mountains (about 20 miles east of the study area). Runoff from the Superstition Mountains is intercepted by flood protection structures and does not affect the study area. There are no perennial streams or rivers in the study area. Lands within the study area generally slope to the southwest at about 0.25 percent. Soils in the area are loam, sandy loam, and clayey loam. The communities are situated in the Lower Sonoran Desert Life Zone, identified by native vegetation including mesquite, palo verde, creosote, and various cacti and scrub brush. Grasses in the desert areas generally grow in depressions where water collects from rainfall or from tailwater off nearby agricultural fields. The major part of the area investigated in this study is developed farmland. Major cultivated crops include cotton, alfalfa, and assorted citrus and fruits.

Requirements for stormwater retention were introduced in the communities about 20 years ago. In general, the communities required each development to retain a volume of stormwater equal to the runoff from a design storm designated by the individual community. The guidelines were not strictly enforced and the effectiveness of the retention requirements was diminished. All future developments are considered to retain 100 percent of the required retention volume because of better enforcement of the retention requirements for new development.

The climate in the communities is dry with an average of 7.5 inches of rain annually. Precipitation in the area is most likely to occur during the summer and winter seasons. Most of the summer precipitation results from thunderstorms originating from the flow of moist tropical air from the

Gulf of Mexico and are formed by convective air currents caused by desert heating. Most of the record summer storms occur in late August and September and are associated with deep surges of tropical air into the state from the Gulf of California and the Pacific Ocean. Winter precipitation typically results from storms originating over the Pacific Ocean which follow a path influenced by a high pressure ridge off the west coast. The summer thunderstorms tend to be more localized with higher intensities and shorter durations while the summer tropical storms and winter storms are more widespread with less intensity and longer durations. Temperatures in the communities vary from above 110 degrees Fahrenheit in the summer to below freezing in the winter. The average temperatures for the months of July and January are 90 degrees Fahrenheit and 50 degrees Fahrenheit, respectively.

2.3 Principal Flood Problems

Flooding within the study area is caused by storm runoff from lands within this area. Runoff from other areas within the drainage basin are intercepted and diverted prior to reaching the area.

Most of the historic information on flood problems in the study area was found in the Summary Report for Flood Control on the Gila Floodway prepared by the COE in 1977. The Eastern and Consolidated canals run north to south through the study area and intercept storm runoff. The SPRR has two alignments in the area which also intercept runoff. The Western Canal also crosses the study area with an east to west orientation and intercepts some storm runoff. These structures form hydraulic barriers which redirect the runoff. The Eastern and Consolidated canals redirect flows toward the southwest. The SPRR (Rittenhouse alignment) directs flows toward the northwest and the SPRR (Arizona Avenue alignment intercepts flows which are released to the west. The Western Canal conveys flows to the west. Major roads or streets, generally located at one-half mile intervals, intercept flows along the barriers. Water is ponded at these locations. When sufficient volume of runoff reaches these sites, water flows across either the barrier, the intercepting road, or both.

2.4 Flood Protection Measures

Several structures have been built within the watershed which store or divert storm runoff which would otherwise reach the developed lands in the Gilbert-Chandler area. Protection for the Central Arizona Project (CAP) canal which passes between the Superstition Mountains and the study area intercepts most of the runoff from the east and stores or diverts it to protect the CAP canal. The runoff between the CAP and the Roosevelt Water Conservation District (RWCD) canal is intercepted and directed to the south by the East Maricopa Floodway. These structures were designed to control runoff from the 100-year storm. Therefore, the area east of the RWCD canal will not contribute flows to the Gilbert-Chandler area during a 100-year event.

The Superstition Freeway is a major structure on the north side of the study area. Flood protection and drainage structures were built to protect the freeway and intercept, detain, and divert water to the west of the study area. There are three locations where the possibility of floodwater passing into the study area exists. An investigation was completed to determine the flooding impact of floodwater from the north to the Gilbert-Chandler study area. This study is detailed in appendix C to this report.

3. ENGINEERING METHODS

The flooding sources in the communities were studied using detailed methods. Standard hydrologic and hydraulic study methods were used to determine the flood hazard data required for this study. A flood event of a magnitude which is expected to be equaled or exceeded on the average of once in a 100-year period (recurrence interval) was selected as having significance for floodplain management and determining flood-prone zones for actuarial flood insurance rates. This event, commonly termed the 100-year flood, has a 1.0 percent change of being equaled or exceeded during any year. Although the recurrence interval represents the long-term average period between floods of a specific magnitude, rare floods could occur at short intervals or even within the same year. The risk of experiencing a rare flood increases when periods greater than 1 year are considered. In other words, the risk of having a flood which equals or exceeds the 100-year flood (1 percent chance of annual exceedance) in any 50-year period is approximately 40 percent (4 chances in 10), and, for any 90-year period, the risk increases to approximately 60 percent (6 chances in 10). The analyses reported herein reflect flooding potentials based on conditions existing in the communities at the time of completion of this study (October 1989). Maps and flood elevations may be amended periodically to reflect changes since completion of the study.

3.1 Hydrologic Analyses

Hydrologic analyses were carried out to establish the peak discharge and flood hydrographs for each flooding source affecting the study area. The hydrologic analyses of the study area was performed using the COE HEC-1 Flood Hydrograph package computer model modified by Haestad Methods version 3.2c.

Existing drainage policies requiring detention storage within urban developments were considered in the analysis. The study area was divided into subareas from which stormwater concentrates at a single point. Rainfall runoff was estimated using U.S. Department of Agriculture, Soil Conservation Service (SCS) curve number procedures. A synthetic storm event was used in completing the study. The

100-year 24-hour precipitation total of 3.8 inches was obtained from the National Oceanic and Atmospheric Administration Atlas 2, "Precipitation-Frequency Atlas of the Western United States, Volume VIII, Arizona," 1973. The SCS type II precipitation distribution was utilized. Hydrographs were developed using the Clark unit hydrograph method for urban areas and the SCS unit hydrograph method for other areas. At each concentration point a hydrograph of direct runoff was developed and a peak discharge and runoff volume was computed for the 100-year 24-hour storm by the HEC-1 computer model. Runoff hydrographs were computed at locations along the hydraulic barriers and were routed along the barriers.

A detailed account of methods used to determine the runoff hydrographs is presented in appendix A to this report.

3.2 Hydraulic Analyses

Analyses of the hydraulic characteristics along the barriers within the study area were completed to provide estimates of the elevations of floods for the 100-year 24-hour storm along hydraulic barriers within the study area. A detailed account of the methods used is presented in appendix B to this report.

Cross-section data for backwater and weir analyses were compiled from photogrammetrically digitized hydraulic cross sections perpendicular to the flow. Topographic mapping (1 inch = 400 ft, 2-foot contours) was utilized to determine ponding characteristics. Bridges and culverts were field surveyed to obtain elevation data and structure geometry.

All elevations are referenced to the National Geodetic Vertical Datum of 1929 (NGVD). Elevation Reference Marks (ERMs) used in this study are based on benchmarks 17, 18, 20, and 21 of the City of Chandler Municipal Control Network (CMCN) completed in 1986 by Greiner Engineering. The basis for the CMCN are the benchmarks "RYAN" and "BM-F". "RYAN" is an Arizona Department of Transportation monument established in 1973. "BM-f" was established in 1977 by the Maricopa County Highway Department.

The horizontal control is based on the Arizona State Plane Coordinate system. The coordinates of the brass cap (BC) at the quarter corner of Section 10/15, Township 1 South, Range 5 East were set by Greiner Engineering for the CMCN in 1985. This BC is point 1 (RO-21) of the survey for the Gilbert-Chandler floodplain delineation.

All of the control points, vertical and horizontal, used in this study were established by Franzoy Corey during August 1987 through November 1987 and are shown on sheet G-1 of appendix D titled "Survey Control and Sheet Index." Topographic mapping used for this study were obtained from photography taken in September 1987.

Locations of the cross sections used in the hydraulic analyses are shown on the topographic plan and profile drawings (appendix D).

Water-surface elevations at ponded locations were computed utilizing the level-pool reservoir routing routines in the COE HEC-1 computer program. Tables 1 through 4 describe the ponded areas along the Eastern Canal, Consolidated Canal, SPRR (Arizona Avenue), and the SPRR (Rittenhouse Road), respectively. The description of the location on these tables describes the concentration point where the ponding occurs. The tables also give an approximate station along the barrier and the cross sections closest to the pond. The pond elevation is also given in the table.

Water-surface elevations of flows along the hydraulic barriers between ponding sites were computed using the COE HEC-2 water-surface profiles computer program. Peak flows resulting from the 100-year 24-hour storm were used. The downstream ponded water elevation was used as the beginning water-surface elevation for the HEC-2 models. Table 5 describes the sections where floodwater flows between the ponded sites along the barriers. The location is described by both the cross sections used in the analysis and by a description of the features nearby. The flooding sources, subbasins, or subareas contributing to the flow, are listed as well as the drainage area in square miles. The peak discharge from the HEC-1 analyses is also listed in the table.

Channel roughness factors (Mannings "n") used in the hydraulic computations were determined by engineering judgment using the SCS Engineering Handbook, Supplement B on Hydraulics and field observations of the channel and floodplain areas. The values ranged from 0.025 in best maintained areas, to 0.075 in the agricultural areas. Worst case seasonal conditions were assumed for vegetative cover where applicable.

The hydraulic analyses for this study were based on existing conditions and flow. Water elevations and flood-prone area delineations can be affected by routine maintenance of the canals or reconstruction of intercepting roadways. Also, routine maintenance is required at locations where drainage channels or culverts convey flows along the canals or railroads or under the roadways. The flood elevations shown on the profiles are considered valid if hydraulic structures remain unobstructed, operate properly, and do not fail. Analyses is also based on the canal banks and railroads overtopping without structure failure.

The peak discharge at concentration points within the model are affected by hydrograph routing procedures. Typical cross sections were used to simulate routing conditions since actual cross sections were not available. Caution in the use of peak discharges for other purposes is advised and specific routing reaches should be field verified. Also,

TABLE 1 (Page 1 of 2)
Summary of Water Surface Elevations
Eastern Canal
Gilbert-Chandler Flood Insurance Study
March 1991

Location	Approximate Station Location	Cross Section	Water Surface Elevation
Riggs road north approximately three-quarter mile	0 - 58+50	5106 - 5098 (ponded)	1250.4
Chandler Heights road north approximately one-quarter mile	59+10 - 70+70	5097.1 - 5097 (ponded)	1253.6
North of Chandler Heights Road approximately one-quarter mile	76+70	5096	1253.7[1]
North of Chandler Heights Road approximately three-eighths mile	81+70	5095	1253.9[1]
North of Chandler Heights Road approximately one-half mile	86+00	5094	1254.7[1]
North of Chandler Heights Road approximately five-eighths mile	97+50	5093	1257.4[1]
North of Chandler Heights Road approximately three-quarters mile	107+50	5092	1258.4[1]
Downstream side of Ocotillo Road	118+00 BK = 125+60 AHD	5091	1258.7[1]
Ocotillo Road north approximately three-quarters mile	126+10 - 173+50	5090 - 5082 (ponded)	1257.5
South side of Queen Creek Road	184+20	5081	1260.0[1]
Centerline of Queen Creek Road	184+80	5080	1260.1[1]
Approximately one-eighth mile north of Queen Creek Road	194+00	5078	1260.4[1]
Approximately one-quarter mile north of Queen Creek Road	199+00	5076	1261.2[1]
Approximately three-eighths mile north of Queen Creek Road	206+50	5075	1260.4[1]

[1] Flowing water overtops the canal bank. The elevation of the canal maintenance road plus 6 inches is used.

TABLE 1 (Page 2 of 2)
Summary of Water Surface Elevations
Eastern Canal
Gilbert-Chandler Flood Insurance Study
March 1991

Location	Approximate Station Location	Cross Section	Water Surface Elevation
Approximately one-half mile north of Queen Creek Road	212+10	5074	1261.2[1]
Approximately one-half mile north of Queen Creek Road	212+60 BK = 217+70 AHD	5073	1261.2[1]
Ryan Road north approximately one-half mile	218+40 - 245+50	5072 - 5070 (ponded)	1259.8
Germann Road north approximately one-half mile	253+10 - 279+20	5068 - 5065.2 (ponded)	1262.5
One-quarter mile south of Pecos Road to one-half mile north of Pecos Road	293+80 - 369+00	5064 - 5055 (ponded)	1264.9
Chandler Blvd north to Val Vista Road	389+60 - 403+50	5054 - 5051 (ponded)	1265.8
Val Vista Road north to Ray Road	404+00 - 433+50	5049 - 5042 (ponded)	1266.6
Ray Road north of the SPRR	433+80 - 467+40	5041 - 5038 (ponded)	1267.7
SPRR north to Warner Road	477+70 - 509+20	5037 - 5033 (ponded)	1271.4
Warner Road north approximately one-half mile	509+40 - 538+25	5032 - 5027 (ponded)	1273.2
Approximately one-quarter mile south of Elliot Road	552+45 BK = 556+30 AHD	5026	1277.3
South side of Elliot Road	571+30	5025	1277.1[1]
Elliot Road north to Guadalupe Road	571+90 - 599+00	5024 - 5020 (ponded)	1277.1
Elliot Road north to Guadalupe Road	599+00 - 625+10	5020 - 5017 (ponded)	1277.7
Guadalupe Road north to Houston Road	625+60 - 652+90	5016 - 5012 (ponded)	1279.4
Houston Road north to Baseline Road	653+30 - 679+20	5011 - 5007.1 (ponded)	1280.8
Baseline Road north to Superstition Freeway	679+60 - 700+00	5007 - 5001 (ponded)	1282.4

[1]Flowing water overtops the canal bank. The elevation of the canal maintenance road plus 6 inches is used.

TABLE 2 (Page 1 of 2)
Summary of Water Surface Elevations
Consolidated Canal
Gilbert-Chandler Flood Insurance Study
March 1991

Location	Approximate Station Location	Cross Section	Water Surface Elevation
Chandler Heights Road south approximately one-half mile to railroad spur	64+00 - 95+25	4091 - 4086 (ponded)	1220.5
North side of Chandler Heights Road	95+70 - 100+00	4085 - 4084 (ponded)	1223.2
Approximately one-quarter mile north of Chandler Heights Road to north approximately one-half mile	105+40 - 131+70	4083 - 4081 (ponded)	1222.4
Ocotillo Road north to Queen Creek Road	165+40 - 211+00	4076.1 - 4072 (ponded)	1227.9
Queen Creek Road north to Germann Road	211+20 - 268+90	4071 - 4062 (ponded)	1229.4
Germann Road north to Pecos Road	275+00 - 328+90	4060 - 4053.1 (ponded)	1230.7
Pecos Road north to Cooper Road	328+90 - 377+00	4053.1 - 4050 (ponded)	1233.3
Cooper Road north to three-quarter mile north of Chandler Blvd	377+20 - 425+50	4049 - 4038 (ponded)	1236.0
Three-quarter mile north of Chandler Blvd to north approximately one-eighth mile	426+70 - 439+50	4038 - 4037 (ponded)	1236.8
Pecos Road north to Knox Road	442+50 - 473+70	4035 - 4030 (ponded)	1238.5
Knox Road north to Warner Road	478+00 - 510+80	4029 - 4022 (ponded)	1238.8
Warner Road north approximately one-half mile	410+80 - 561+30	4021 - 4018 (ponded)	1240.1
Southern Pacific Railroad north to Lindsay Road	561+60 - 618+50	SPRR - 4011 (ponded)	1244.8
Guadalupe Road south approximately one-quarter mile to Guadalupe Road north approximately one-half mile	618+50 - 632+40	4011 - 4006 (ponded)	1245.6

TABLE 2 (Page 2 of 2)
Summary of Water Surface Elevations
Consolidated Canal
Gilbert–Chandler Flood Insurance Study
March 1991

Location	Approximate Station Location	Cross Section	Water Surface Elevation
Approximately one-half mile north of Guadalupe north to Baseline Road	661+30 – 686+60	4006 – 4005 (ponded)	1246.9
Baseline Road north to the Superstition Freeway (Hwy 360)	687+50 – 711+10	4004 – 4001 (ponded)	1248.7

TABLE 3
Summary of Water Surface Elevations
Southern Pacific Railroad at Arizona Avenue
Gilbert-Chandler Flood Insurance Study
March 1991

Location	Approximate Station Location	Cross Section	Water Surface Elevation
Hunt Highway north to Consolidated Canal	0 - 26+50	2092 - 2090 (ponded)	1219.1
Consolidated Canal north to railroad spur line	32+80 - 94+20	2088 - 2080 (ponded)	1216.3
SPRR spur line to Chandler Heights Road	99+10 - 108+40	2079 - 2078 (ponded)	1219.0
Chandler Heights Road north to Ocotillo Road	108+80 - 150+50	2077 - 2074 (ponded)	1217.2
Ocotillo Road north to Appleby Road	159+10 - 185+60	2072 - 2070 (ponded)	1220.9
Appleby Road north to Queen Creek Road	188+30 - 212+20	2070 - 2067 (ponded)	1219.1
Queen Creek Road north to Willis Road	212+20 - 287+80	2067 - 2055 (ponded)	1218.3
Willis Road north to Pecos Road	291+50 - 319+30	2053 - 2049 (ponded)	1219.7
Pecos Road north to Ray Road	319+30 - 425+10	2049 - 2031 (ponded)	1216.5
Ray Road north to Warner Road	425+10 - 475+50	2031 - 2025 (ponded)	1214.7
Warner Road north to Elliot Road	492+70 - 533+60	2021 - 2019 (ponded)	1214.2
Elliot Road north to Western Canal	535+30 - 560+80	2017 - 2014 (ponded)	1212.8
Western Canal north to Baseline Road	560+80 - 641+00	2014 - 2004 (ponded)	1211.9

TABLE 4 (Page 1 of 2)
Summary of Water Surface Elevations
Southern Pacific Railroad at Rittenhouse Road
Gilbert-Chandler Flood Insurance Study
March 1991

Location	Approximate Station Location	Cross Section	Water Surface Elevation
Intersection with Chandler Boulevard	56+50 - 72+80	1007 - 1010 (ponded)	1301.3
Intersection with Eastern Canal	175+00 - 195+00	1022 - 1028 (ponded)	1271.4
Intersection with Consolidated Canal	295+30 - 315+00 BK = 316+60 AHD	1042 - 1044 (ponded)	1244.8
Between Consolidated Canal and Elliot Road	319+10	1044.1	1242.0
Between Consolidated Canal and Elliot Road	322+60	1044.2	1240.3
Between Consolidated Canal and Elliot Road	327+60	1044.3	1239.6
Between Consolidated Canal and Elliot road	333+85	1044.4	1239.1
Between Elliot Road and Gilbert Road	341+25	1044.5	1237.2
Between Elliot Road and Gilbert Road	345+75	1044.6	1236.4
Between Elliot Road and Gilbert Road	352+90	1044.7	1234.5
Gilbert road to Western Canal	360+05	1045	1233.0
Gilbert road to Western Canal	370+05	1046	1231.4
Gilbert Road to Western Canal	377+05 BK = 379+30 AHD	1047	1230.6
Western Canal to Guadalupe Road	380+20	1049	1229.5
Western Canal to Guadalupe Road	380+20	1049.1	1224.8
Western Canal to Guadalupe Road	388+50	1050	1223.5
Western Canal to Guadalupe Road	398+50	1051	1222.3

TABLE 4 (Page 2 of 2)
Summary of Water Surface Elevations
Southern Pacific Railroad at Rittenhouse Road
Gilbert-Chandler Flood Insurance Study
September 1990

Location	Approximate Station Location	Cross Section	Water Surface Elevation
Western Canal to Guadalupe Road	409+70	1052	1221.5
Western Canal to Guadalupe Road	421+20	1058.1	1220.1
Guadalupe Road to McQueen Road	427+95	1058.2	1219.5
Guadalupe Road to McQueen Road	436+70	1058.3	1217.6
Guadalupe Road to McQueen Road	446+70	1059	1214.6
Guadalupe Road to McQueen Road	466+90	1060	1213.1
Intersection with Superstition Freeway (Hwy 360)	446+90 - 537+70	1060 - Hwy 360	1213.1

TABLE 5
Summary of Discharges
Eastern Canal and SPRR
Gilbert-Chandler Flood Insurance Study
March 1991

Location	Flooding Source	Cross Sections	Drainage Area[1] (sq miles)	Peak Discharge (cfs)
	EASTERN CANAL			
Gilbert Road south to ponded water above Chandler Heights Road	Subarea 30 - subbasins A, B, C, D, E1, E2, F, G, H, I, J	5097 - 5093	3.88	993
Ocotillo Road south to Gilbert Road	Subarea 30 - Subbasins A, D, E1, E2, G, H	5092 - 5091	2.2	548
Queen Creek Road south to ponded water above Ocotillo Road	Subarea 29 - Subbasin D and flows overtopping Ryan Road	5082 - 5081	0.22	673
Ryan Road south to Queen Creek Road	Subarea 29 - V-flows overtopping Ryan Road	5080 - 5074	0.0	720
Elliot Road south to ponded water above Warner Road	Subarea 9 - V-flows overtopping Elliot Road	5027 - 5025	0.0	469
	SPRR - RITTENHOUSE			
Consolidated Canal to north Burk Street	Flows overtopping the Consolidated Canal	1044.3 - 1044.1	0.0	313
Burk Street to Gilbert Road at SPRR	Flows overtopping the Consolidated Canal and subarea 7 - Subbasins A and B	1044.4 - 1044.7	0.05	313
Gilbert Road at SPRR to Western Canal at SPRR	Flows overtopping the Consolidated Canal and Subarea 7 - Subbasins A and B	1048 - 1045	0.05	259
Western Canal at SPRR to Neely Street at SPRR	Flows overtopping Western Canal and Subarea 6 - Subbasins A0, A A1, B, B1, C, D, E and F	1050-1049	0.59	90
Neely Street at SPRR to Cooper/Guadalupe Road at SPRR	Flows Overtopping Western Canal and Subarea 6 - Subbasins A0, A, A1, B, B1, C, D, E, F, G and H	1058.1 - 1051	0.73	452
Cooper/Guadalupe Road at SPRR to Nevada Way at SPRR	Flows Overtopping Western Canal and Subarea 6 - Subbasins A0, A, A1, B, B1, C, D, E, F, G, and I	1059 - 1058.2	0.83	284
Nevada Way at SPRR to McQueen Road at SPRR	Flows overtopping Western Canal and Subarea 6 - Subbasins A0, A, A1, B, B1, C, D, E, F, G, I, J, and K	1060	1.01	178

[1]Drainage area for flows from upstream subareas or subbasins is not included

the Gilbert-Chandler area is prone to subsidence due to groundwater withdrawal and datum points should be verified for subsequent use.

4. FLOODPLAIN MANAGEMENT APPLICATIONS

The NFIP encourages state and local governments to adopt sound floodplain management programs. Therefore, this flood insurance study provides flood elevations and floodplain boundary delineations for the 100-year flood to assist communities in developing floodplain management measures.

4.1 Flood Boundaries

In order to establish a national standard without regional discrimination, the 100-year flood has been adopted by the Federal Emergency Management Agency (FEMA) as the base flood for the purposes of floodplain management. The 100-year floodplain boundaries have been delineated using the flood elevations determined at each cross section or ponding site. Between cross sections the 100 year floodplain boundaries were interpolated using topographic maps at a scale of 1 inch = 400 ft (appendix D).

The 100-year floodplain boundaries are to be shown on the flood insurance rate map. On this map, the 100-year floodplain boundary corresponds to the boundary of the areas of special flood hazards. Small areas within the floodplain boundaries may lie above the flood elevations but cannot be shown due to limitations on the map scale and/or lack of detailed topographic data.

4.2 Floodways

Floodways were not computed or designated within the study area. Flooding in the area is primarily caused by shallow ponding at the intersection of the canals or railroads and major streets. Encroachment upon these ponding areas would cause adverse downstream effects. This study was limited to the determination of the flood-prone areas along major hydraulic barriers.

5. INSURANCE APPLICATION

For flood insurance rating purposes, flood insurance zone designations are assigned to a community based on the results of the engineering analyses. For the Gilbert-Chandler flood insurance study the following flood insurance zone designations have been assigned:

Zone AH: Zone AH is the flood insurance rate zone that corresponds to the areas of 100-year shallow flooding (usually areas of ponding) where average depths are between 1 and 3 ft. Whole-foot base flood elevations derived from the detailed hydraulic analyses are shown at selected intervals within this zone.

Zone A: Zone A is the flood insurance rate zone that corresponds to the 100-year floodplains that are determined in the flood insurance study by approximate methods. Because detailed hydraulic analyses are not performed for such areas, no base flood elevations or depths are shown within this zone.

The Zone A designations are limited to areas between the Superstition Freeway (SR 360) and Baseline Road along the Eastern and Consolidated canals. This area is within the City of Mesa. All other areas are designated as zone AH.

This zone designation, requested by the communities involved, will aid in the management of the zones designated as flood-prone areas.

6. FLOOD INSURANCE RATE MAP

The flood insurance rate map is designed for flood insurance and floodplain management applications.

For flood insurance applications, the map designates flood insurance rate zones as described in section 5 and, in the 100-year flood plains that were studied by detailed methods, selected base flood elevations or average depths are shown. Insurance agents use the zones and base flood elevations in conjunction with information on structures and their contents to assign premium rates for flood insurance policies.

For floodplain management applications, the map shows by tints, screens, and symbols, the 100-year floodplains and the locations of selected cross sections used in the hydraulic analyses and computations.

7. OTHER STUDIES

A summary report for flood control done by the COE completed in September 1977 is the last comprehensive study done for the area. The Gila Floodway, Maricopa and Pinal Counties, Arizona is not a report for the community use in floodplain management and did not consider future development of the area.

Two reports were done by Harris-Toups Associates and completed for the communities of Gilbert and Chandler in July 1979 and January 1980, respectively. The Flood Insurance Study, Town of Gilbert, Arizona and the Flood Insurance Study, City of Chandler, Arizona are similar to this study, but only projected to the year of 1985 and was not as comprehensive.

This study is authoritative for the purposes of the flood insurance program, and the data presented here either supersedes or are compatible with previous determinations.

8. LOCATION OF DATA

Information concerning the pertinent data used in the preparation of this study will be submitted to the Federal Insurance Administration, Regional Director, Region IX Office, 450 Golden Gate Avenue, P.O. Box 36003, San Francisco, California 94102. Also, copies of the study data are on file with the Flood Control District of Maricopa County, 3335 W. Durango St., Phoenix, Arizona 85009.

9. REFERENCES

1. U.S. Army Corps of Engineers, Los Angeles District, Summary Report for Flood Control, Gila Floodway, Maricopa and Pinal Counties, Arizona. Los Angeles, California. September 1977.

2. Federal Emergency Management Agency, Federal Insurance Administration, Flood Insurance Study, City of Chandler, Arizona. San Francisco, California. January 1980.

3. Federal Emergency Management Agency, Federal Insurance Administration, Flood Insurance Study, Town of Gilbert, Arizona. San Francisco, California. July 1979.

4. Flood Control District of Maricopa County, Hydrologic Design Manual for Maricopa County, Arizona, draft April 1990.

5. The League of Arizona Cities and Towns and the Arizona Association of Counties, Local Government Directory. July 1988.

6. National Oceanic and Atmospheric Administration, Atlas 2, Volume VIII, Arizona, 1973.

7. William D. Sellers and Richard H. Hill, Arizona Climate 1931-1972, The University of Arizona Press. Tucson, Arizona. 1974.

8. U.S. Army Corps of Engineers, the Hydrologic Engineering Center, Generalized Computer Program, HEC-1 Flood Hydrograph Package. Davis, California. 1981. Revised June 1985. Modified by Haestad Methods for 900-Ordinates. Haestad Methods Incorporated, Hydrology/Hydraulics Software, HEC-1 Version 3.2c. Waterbury, Connecticut. 1989.

9. U.S. Army Corps of Engineers, The Hydrologic Engineering Center, Generalized Computer Program, HEC-2 Water Surface Profiles. Davis, California. 1982.

10. U.S. Department of Agriculture, Soil Conservation Service, Engineering Handbook, Hydraulics Section, Subsection 4.4, Supplement B. August 1956.

APPENDIX A

HYDROLOGIC ANALYSES

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER A.1

GENERAL

This Flood Insurance Study (FIS) covers the area between Roosevelt Water Conservation District Canal (RWCD) (east boundary), Hunt Highway (south boundary), the Southern Pacific Railroad (SPRR) (west boundary) and the Superstition Freeway (north boundary); and, an area bounded in the east by SPRR, in the south by Western Canal, in the west by Tempe Canal (Price Road) and in the north by the Superstition Freeway (see figure A-1). The study area is located entirely in Maricopa County, Arizona. Included in the study area is the incorporated area of the City of Gilbert; part of the incorporated area of the City of Mesa south of the Superstition Freeway; and, part of the incorporated area of the City of Chandler east of the SPRR. A portion of the area is not incorporated.

The study area has been modeled using the HEC-1 computer program, February 1981, revised June 1985 version, developed by the U.S. Army Corps of Engineers (COE), Hydrologic Engineering Center (HEC), modified by Haestad Methods, Inc. (version 3.2c) to allow simulations up to 900 time ordinates (1989).

The study area was divided into subareas using the subarea delineation of the COE study (1977) as a guide. Agricultural development and practices, and urban development forced the boundaries of subareas to follow section- or midsection-line roads. Subareas are continuously numbered from 1 through 37. Subarea 20 and 15 were further subdivided in 20A and 15A. In total there are 42 subareas. Subareas vary in size from 0.3 to 6.9 square miles.

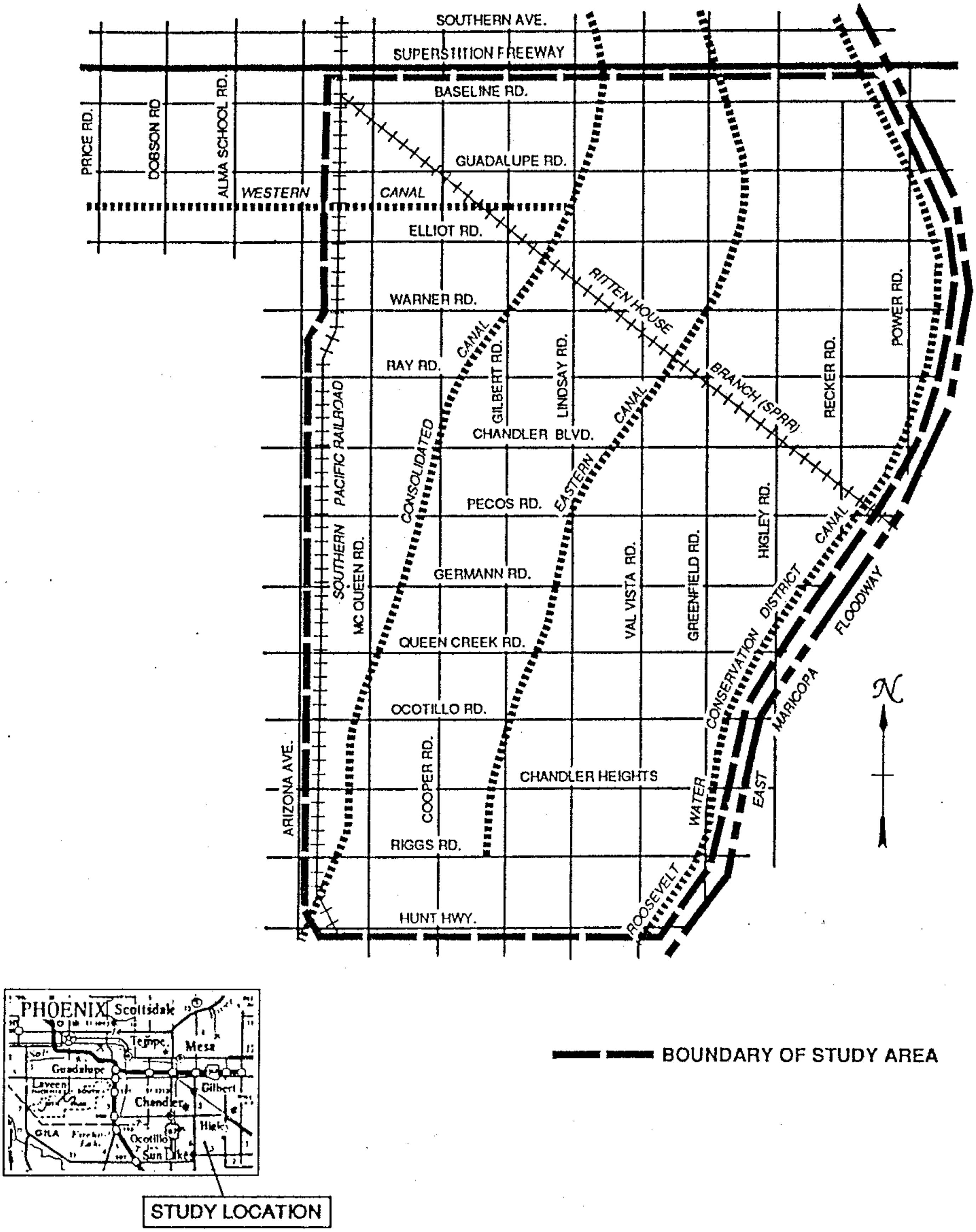


FIGURE A-1
STUDY AREA SOUTH OF THE
SUPERSTITION FREEWAY
GILBERT-CHANDLER FLOODPLAIN DELINEATION STUDY

The subareas were further subdivided into subbasins. Subbasin division was based on land-use type and time period of development. The size of a subbasin was generally equal or less than 320 acres.

Exceptions to this are urban developments with lakes, such as found between Baseline and Guadalupe roads and Eastern and Consolidated canals; between SPRR and Consolidated Canal, north of Warner Road; and, between the Superstition Freeway and Western Canal east of the SPRR. Also, some of the subbasins with agricultural land use are larger than 320 acres, if this were the predominant land-use type within a subarea. Subbasins, within each subarea, are designated by letters A through Z, AA, AB, etc.

The study area is about 98 square miles. Nearly 27.5 percent is urban development, 67 percent is agricultural development, and 5.5 percent is occupied by scrub and open space.

The subarea and subbasin division, land use, concentration points and routing direction are shown on drawings G-2 and G-3 which are included in appendix D which is bound separately from this report.

In chapter A.2 the methodology and the input to the HEC-1 model are discussed in more detail, including precipitation, subbasin characteristics, hydrograph generation, retention in urban developments, and flood routing between concentration points. The flood routing along the hydraulic barriers, split-flow handling at ponded water sites and overtopping of barriers are described in chapter A.3. Flood waters from one subarea can overflow in the next subarea downstream. Hydrograph transfer from one subarea to the next is discussed in chapter A.4.

CHAPTER A.2

METHODOLOGY AND INPUT TO HEC-1

A2.1 PRECIPITATION

The SCS' Type II precipitation time distribution for the 100-year 24-hour storm was used as input to the HEC-1 program. This distribution was provided by the Flood Control District of Maricopa County (FCDMC). The 100-year 24-hour storm precipitation total of 3.8 inches was obtained from the National Oceanic and Atmospheric Administrations', "Atlas 2, Volume VIII, Arizona," 1973.

A2.2 SUBBASIN CHARACTERISTICS

Subbasin characteristics such as area, rainfall loss rate data or unit graph data are all input to HEC-1. Hydrologic data sheets were made for each subarea with subbasins using a spreadsheet program. The hydrologic data sheets include the subarea number, area, soil types, hydrologic soil group(s), concentration point, natural ground slope, and overland slope velocities (paved and unpaved). Also included for each subbasin within the subarea are, the subbasin identification, land-use type, concentration point location, area in acres and square miles, curve number, hydraulic length, lag time and percent impervious.

The SCS unit hydrograph method was used to estimate the runoff from agricultural, desert scrub, open space and commercial/industrial subbasins; the Clark unit hydrograph method was used to estimate runoff from the residential subbasins.

A2.2.1 Land Use and Soil Types

The following land-use types were recognized in the study area: high and low density residential; commercial/industrial; agricultural (row crops or orchards); desert scrub; and, open space. High density residential includes single family homes on lots smaller than 0.25 acres, townhouses, condominiums and apartment buildings. Land-use types were determined from aerial photography at a scale of 1 inch = 1,000 ft. The cities were consulted to determine changes in land use expected to occur within a year from the beginning of the study. These future land uses were incorporated in the study.

The soil survey report "Eastern Maricopa and Northern Pinal Counties, Arizona." (SCS, 1974) was used to determine the soil types in the study area. For each subarea the percentage of the area occupied by each soil type was estimated. Soils in the study area belong generally to the Mohall-Contine association of well-drained, nearly level loams, clay loams, and sandy clay loams on old alluvial fans; and, to the Gilman-Estrella-Avondale association of well-drained, nearly level loams and clay loams on alluvial fans and floodplains (USDA/SCS, 1974).

A2.2.2 Natural Ground Slope and Area of Subbasins

The natural ground slope for each subarea was determined from USGS 7.5 minute quadrangles with 10 ft contours. Natural ground slope in the study area varied from 0.0011 ft/ft to 0.0050 ft/ft (0.11 percent to 0.5 percent). The natural ground slope determined for the subareas was also used for each subbasin within that subarea. The natural ground slope is used to determine overland flow velocities, which are used to estimate time of concentration; and for the channel routing between concentration points in the HEC-1 program.

The area of each subbasin was planimetered from overlays (with land-use types) of the 1 inch = 1,000 ft scale photo mosaic of the study area.

Areas of each subbasin are given in acres and square miles on the hydrologic data sheets. Each subarea was digitized to determine its size. The size of the subarea was compared with the sum of the areas of the subbasins to check for any discrepancies.

A2.2.3 Interception/Infiltration (Loss-Rate Analysis)

HEC-1 allows a choice of four different methods to compute the interception/infiltration part of the rainfall-runoff process. These methods simulate the interception of precipitation by vegetation, depression, etc. and the accumulation of moisture in the soil. Two important factors should be noted about the precipitation-loss computation in the HEC-1 model. First, precipitation which does not contribute to the runoff process is considered to be lost from the system. Second, the equations to compute the losses do not provide moisture or surface-storage recovery (COE/HEC, 1987).

The loss rate due to interception and infiltration for this study is expressed in the SCS curve number or as loss in initial and uniform infiltration rates. The curve number is a measure of the soil's runoff potential. The higher the curve number, the greater the runoff potential. The curve number is determined by soil type, land use and treatment, and antecedent soil moisture condition. The choice of the curve number is critical in the runoff computation. The curve number was chosen from table 2.2 in Technical Release 55 "Urban Hydrology for Small Watersheds" (USDA/SCS, 1986). This table relates the land use and treatment to the hydrologic soil groups. There are four hydrologic soil groups, A through D. Hydrologic soil group A has the lowest runoff potential, hydrologic soil group D the highest.

Hydrologic soil group A was not found in the study area. The soil-type list (TR55-Exhibit A.1) indicates the hydrologic soil group for a certain soil. In some of the subareas different soil types, and thus hydrologic soil groups were present. For each subarea the percentages of each hydrologic soil group were determined. These percentages were used to find a weighted curve number for a certain land-use type. For example, if in a subarea 60 percent of the area is classified as hydrologic soil group B and 40 percent by hydrologic soil group D. Then a weighed curve number is computed as follows (the example is for residential land use with one-quarter acre lots):

Hydrologic Soil Group	Percentage (%)	Curve Number (CN)	CN x %
B	60	75	4,500
D	40	87	3,480
	100		7,980

The weighed curve number is (7,980/100=) 79.80; which is rounded to 80.

The following curve numbers were used for the different land uses in the study area.

TABLE A-1
Curve Numbers for Each Land Use and Hydrologic Soil Group in the Gilbert-Chandler Floodplain Delineation Study Area

Land Use	Hydrologic Soil Group B	C	D
Commercial/Business	92	94	95
Industrial	88	91	93
Residential Lots			
1/8 acre of less	85	90	92
1/4 acre	75	83	87
1 acre or more	68	79	84
Agriculture			
Straight row crop (good condition)	78	85	89
Orchard	65	65	65
Open Space (poor condition)	78	86	89
Desert Scrub (poor condition)	77	85	88

Source: USDA/SCS, 1986, TR-55
Flood Control District of Maricopa County

The curve number of 65 for orchards was suggested by the FCDMC. Orchards within the study area are flood irrigated. Borders are designed within the orchards to retain water applications (typically 3 to 6 inches). Therefore, a lower curve number is warranted.

In urban areas where the Clark unit hydrograph method was used, an initial and uniform loss rate was used. The initial infiltration losses were derived from curve numbers (CN) assigned to each area and computed from the formula $IA = 0.2\ (1000\text{-}10CN)/CN$. A uniform loss rate of 0.25 inches per hour was used for the previous areas in the subbasin.

The portion of impervious surface represented by section- and midsection-line roads was not expressed in the curve number. Instead of computing the percentage of impervious areas from section- and midsection-line roads for each different land-use type, the area occupied by these roads was computed for a typical square mile (640 acres). This area of a typical section was then expressed as a percentage of the total area. For agricultural and urban developments these numbers were 2.06 and 4.11 percent, respectively. In agricultural areas, the section-line roads were assumed to be 28 ft wide and the midsection-line roads 25 ft. In urban areas the section-line roads were assumed to be 79 ft wide, including 5 ft for sidewalks and 0.5 ft curb, and the midsection-line roads 30 ft. Half of the widths of the section-line roads were used to compute the percentage impervious area for a typical square mile. If an agricultural area was totally surrounded by urban development, the percentage impervious of urban area was applied to this area and vice versa.

An average condition (soil moisture condition II) was assumed as the antecedent soil moisture condition in the study area, i.e. the index of watershed wetness before the storm.

A2.3 HYDROGRAPH GENERATION AND COMBINATION

Rainfall excesses are computed for each time interval by subtracting the interception/infiltration loss from the incoming rainfall. To convert the rainfall excess hydrograph into a runoff hydrograph the SCS dimensionless unit hydrograph method was used for agricultural, open spaces, desert scrub and commercial/industrial areas; and, the Clark unit hydrograph method for residential areas.

A2.3.1 SCS Dimensionless Unit Hydrograph

Input data for the SCS dimensionless unit hydrograph method consist of one parameter, the lag time. Theoretically, the lag time is the time in hours between the center of mass of rainfall excess and the peak of the unit hydrograph. The lag time, L, may be estimated in terms of the time of concentration, Tc, using the empirical relation:

$$L = 0.6\ T_c = 0.6\ (l_p/(3600*v))$$

where,

L = lag time in hrs

T_c = time of concentration in hrs

l_p = length of flow path (or hydraulic length, longest flow path in subbasin) in ft

v = average velocity in ft/s, and

3600 = conversion factor from seconds to hrs

The time of concentration is the time it takes for runoff to travel from the hydraulically most distant part of the subbasin to the subbasin outlet (or concentration point). The average velocity was determined from figure 3.1 in TR-55 (USDA/SCS, 1986), which relates watercourse slope in ft/ft with average velocity in ft/s. Since slope in the study area varied from 0.0011 ft/ft to 0.0050 ft/ft the graph in TR-55 was extended. The average velocities were determined for each subarea, since the available data (USGS 7.5 min. quadrangle maps) did not allow determination of ground slope for each subbasin.

A2.3.2 Clark Unit Hydrograph Method

The Clark unit hydrograph method is used to compute runoff hydrographs from urban areas. This method uses two numeric parameters, the time of concentration (Tc) and the storage coefficient (R), and a graphical parameter, the time-area relationship. These parameters were developed using the Maricopa County Flood Control District's (FCD) computer program MCUHP1.

The time of concentration is estimated from an empirical equation adopted by the FCD with some procedural modifications from Papadakis and Kazan.

$$Tc = 11.4L^{0.50}Kb^{0.52}S^{-0.31}i^{-0.38}$$

where,

- L = length of flow path for Tc in miles,
- K_b = representative watershed resistance coefficient,
- S = the watershed slope in ft per mile, and
- i = the average rainfall intensity, during the time Tc, in inches per hour

and, $K_b = -0.00625 \log(\text{basin area in sq mi}) + 0.04$
(for urban areas)

The storage coefficient (R) is estimated from the equation -

$$R = 0.37\ Tc^{1.11}A^{-0.57}L^{0.80}$$

where,

- Tc = the time of concentration,
- A = the drainage area in square miles, and
- L = the length of the flow path in miles.

The basin time-area relationship is taken from the following:

Time, as a percentage of Tc	Contributing area, as a percentage of total area
0	0
10	5
20	16
30	30
40	65
50	77
60	84
70	90
80	94
90	97
100	100

A2.3.3 Hydrograph Combination

The subbasin division in the study area was based on land-use type. Each subbasin generally has a different land use. A hydrograph was generated for each subbasin separately. This hydrograph was then routed from the subbasin concentration point to the concentration point of the next subbasin downstream. At this concentration point of the downstream subbasin the hydrographs were combined, and the combined hydrograph was routed to the concentration point of the next subbasin downstream.

In subbasins where land use was predominately agricultural and desert areas (especially between the East Maricopa Floodway and Eastern Canal) a different method of hydrograph combining was used. In those areas one subbasin may consist of agricultural, desert and/or some scattered urban development. Generally, the subbasins in the study area are not larger than 320 acres. For each of the land-use types a hydrograph was generated and the hydrographs were combined at the subbasin concentration point.

A2.4 RETENTION IN URBAN DEVELOPMENTS

Within the study area two different retention requirements are used; the drainage policies of the cities of Gilbert and Mesa require that all runoff from a 50-year 24-hour storm (3.0 inches) be retained within the subdivision boundaries (City of Mesa, 1983; Town of Gilbert, 1987). The City of Chandler's drainage policy requires that all stormwater which falls within a subdivision or site, including the respective one-half of all abutting streets, resulting from a 100-year 2-hour storm (2.5 inches) shall be retained within the boundaries of the subdivision or site (City of Chandler, 1987). Retention volume within the county is the difference between pre- and post-development runoff volume. Most of the agricultural areas in the study area are unincorporated and Maricopa County's regulations apply.

The drainage policies of the cities in the study area were in effect since the beginning and mid 1970s. However, for example Mesa did not require certification of volumes or as-built conditions of retention facilities before 1980. Also, residential, on-lot retention basins may have changed in capacity due to e.g. landscaping by the owner. For these reasons it was felt that retention facilities in subdivisions developed between 1972 and 1980 are 50 percent effective; and, retention basins in subdivisions developed after 1980 are 70 percent effective. Retention facilities in subdivisions since the start of this study (1987) were considered 100 percent effective. In the HEC-1 input files the KM card shows, if a subdivision is high or low density residential, or apartments; developed before or after 1980, if developed since 1987 the percentage of effectiveness of retention.

The runoff volume from the 50-year 24-hour and the 100-year 2-hour storm was computed using HEC-1 (PH card). The depth-duration data for these storms were computed following the procedure outlined in Arizona Department of Transportation's

(ADOT) "Hydraulic Design for Highway Drainage in Arizona," (1975). To account for retention in the subdivisions (residential and commercial), a dummy retention basin was included in the model. The maximum retention of this dummy basin was the volume of runoff from the 50-year 24-hour (Mesa and Gilbert) and 100-year 2-hour storm (Chandler) multiplied by the percentage of effectiveness. The HEC-1 diversion cards (DT, DI, and DQ cards) were used to describe the dummy retention basin. Discharge from the dummy retention basin started after the required volume of runoff was retained.

A2.5 FLOOD ROUTING BETWEEN CONCENTRATION POINTS

Flood routing is used to simulate flood-wave movement through river reaches and reservoirs. Most of the flood-routing methods in HEC-1 are based on the continuity equation, and some relationship between flow and storage or stage. Routing proceeds on an independent reach basis from upstream to downstream; neither backwater effects nor discontinuities in the water surface are considered (COE/HEC, 1987).

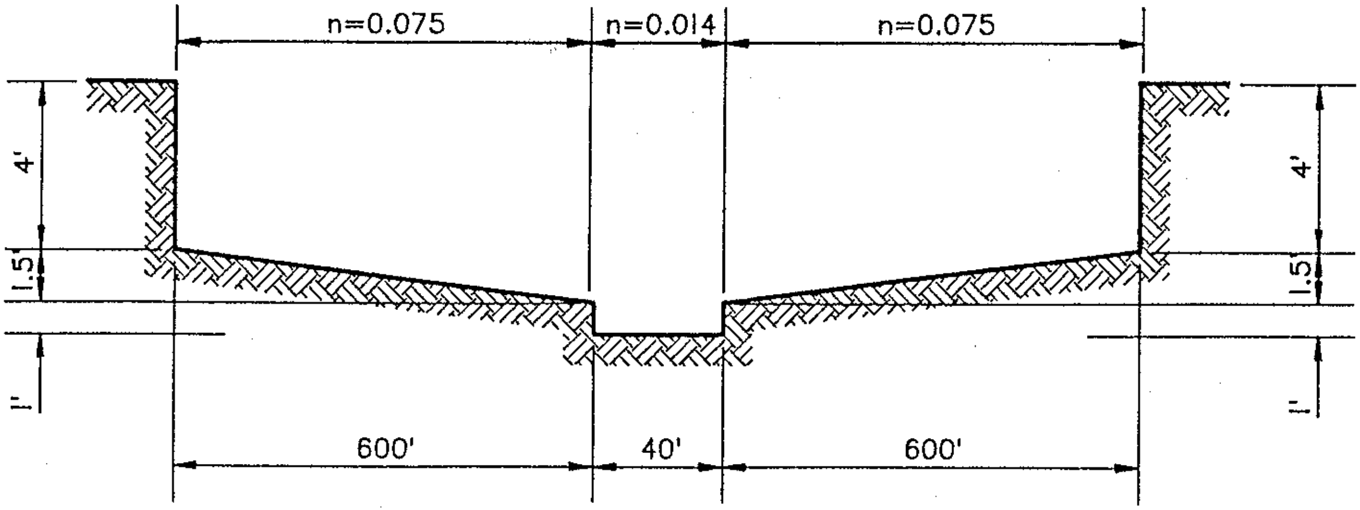
In this study, the modified puls normal depth routing method was used for channel routing. Channel routing was accomplished on a reach-to-reach basis without considering a direct continuous water-surface profile. Input parameters were channel shape, length, slope, and roughness (Mannings "n") coefficient.

The general direction of flow is towards the hydraulic barriers (irrigation canals, railroads). Flood hydrographs were routed from the concentration point of a subbasin to the concentration point of the next subbasin downstream where the hydrographs were combined. Generally flood hydrographs were routed along section- or midsection-lines.

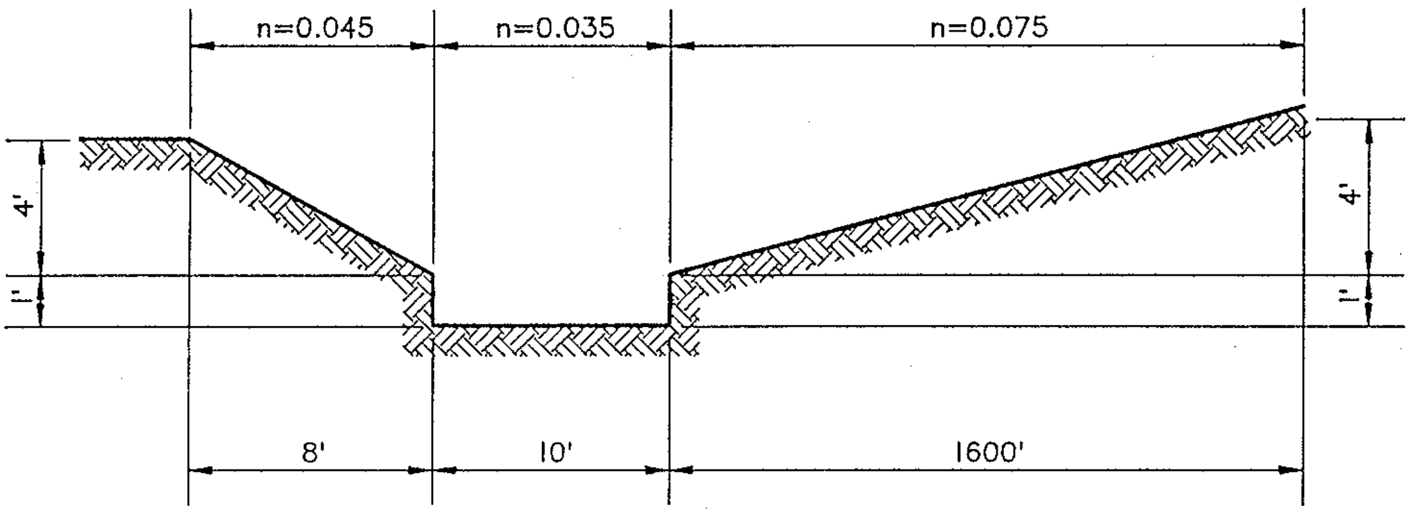
The modified puls method of channel routing was used to route hydrographs between concentration points within the subareas. The modified puls method using normal-depth storage and outflow was used. The HEC-1 program uses an 8-point cross section which is representative of the routing reach. Outflows are computed for normal depth using Manning's equation. Storage is cross-sectional area times reach length (COE/HEC, 1987).

Since detailed cross sections throughout the study area were not available, typical cross sections were established as shown on figure A-2. The cross sections represent a 40 ft wide roadway with overbank flows, as flows are generally routed along the streets and roadways throughout the basin. The typical sections are intended to simulate the attenuation of peak flows and the storage effect as flows exceed the capacity of the roadways to convey the flows.

The length of the routing reach was measured from a 1 inch = 1,000 ft scale photo mosaic of the study area. The slope of the channel was assumed to be the average slope of the subarea in which the channel was located. Since the natural ground slope of the study area varies from a 0.0011 to 0.0050 ft/ft, some of the hypothetical routing channels may actually be at supercritical or critical slope. For routing between concentration points parallel to the irrigation canals, the slope of the routing channel was set equal to the slope of the canal.



TYPICAL STREET FLOW CROSS-SECTION



TYPICAL CROSS-SECTION FOR FLOW ALONG RITTENHOUSE RAILROAD

FIGURE A-2
TYPICAL CHANNEL ROUTING CROSS SECTIONS
GILBERT-CHANDLER FLOODPLAIN DELINEATION STUDY

CHAPTER A.3

OVERTOPPING AND SPLIT-FLOW HANDLING AT PONDED WATER SITES

A3.1 GENERAL APPROACH

All runoff from the subareas flows generally westerly towards the hydrologic barriers, Eastern and Consolidated canals, Rittenhouse Southern Pacific Railroad (SPRR), and the SPRR paralleling Arizona Avenue. The flows collect behind the hydrologic barriers and are redirected generally southward. The collected flows parallel the barriers until reaching major streets which intersect the barriers. The flows pond upstream of these intersections until either the barrier or the street is overtopped. Overtopping of the canal banks may result in breach (structural failure), and release of the ponded water into downstream subbasins. However, it was assumed in the analyses that no breach would occur.

The purpose of the study is to map the area inundated behind the hydrologic barriers using the maximum (ponded) water stage. Analysis of the overtopping by assuming no breach will result in prediction of a conservative maximum water stage in the ponded area. With this approach, the model will retain as storage, the water behind the barrier below the crest of the barrier or intersecting street.

Floodflows overtopping the upstream canal bank were combined with the canal normal operating flow. Flows exceeding the canal capacity were diverted to the downstream subarea and then added to the subbasin hydrograph immediately adjacent to the canal.

A3.2 OVERTOPPING AND SPLIT-FLOW HANDLING

The procedure used to determine overtopping of hydrologic barriers and to handle split flows at ponded water sites is described in the following.

Barriers causing floodwaters to pond were identified. The potential water ponding areas were modeled with HEC-1 using the level-pool reservoir routing method. A reservoir storage volume-versus-elevation relationship is required for the level-pool reservoir routing. The relationship may be computed by HEC-1 from supplied surface area-versus-elevation data. The conic method is then used to compute reservoir volume. The volume is assumed to be zero at the lowest elevation given, even if the surface area is greater than zero at that point (COE/HEC, 1987).

The surface area-versus-elevation data were obtained from 2-foot contour mapping upstream of the hydrologic barriers. The lowest and highest ground elevations were determined and entered on a standard form. The area for even increments of elevations was planimetered and entered on this form. For each concentration point along the hydrologic boundaries where ponding occurred, the surface-area-versus elevation data were collected. These data were added to the HEC-1 input files.

Culverts, which may convey floodwater out of the ponded water area, were located and dimensions measured. A stage-discharge curve for culvert outflow was determined assuming inlet control (see appendix B, section B.4).

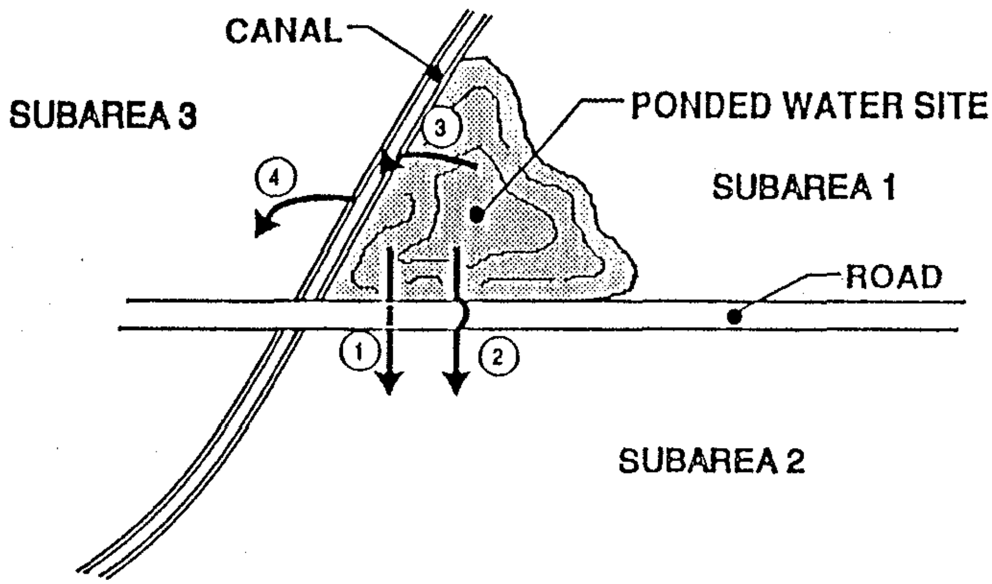
The elevations of the hydrologic barrier and cross road were determined from the digitized cross sections from 2-foot contour maps upstream of the hydrologic barriers and from field survey. The profile of the hydrologic barrier and cross road were drawn for each ponded water site. The low point, where overflow would first occur, was determined. For both the hydrologic barrier and the cross road a stage-discharge curve for overflow was computed (see appendix B, section B.3).

Culvert and weir outflow were combined into one stage-discharge curve. These data were input to the HEC-1 files to model the outflow from the ponded water sites.

After the routing through the ponded water sites the outflow can: (1) remain behind the barriers; (2) overtop the hydrologic barrier; (3) overtop the road; (4) cross the barrier in a culvert; or (4) a combination of the above (see figure A-4).

The hydrograph flowing out of the ponded water site is then split into culvert flow, hydrologic barrier overtopping, and/or road overtopping. The diversion cards of HEC-1 (DT, DI and DQ cards) are used for this operation. If the concentration point at the ponded water site is not at a subarea boundary, the flood hydrograph leaving the ponded water site over the road as weir flow and/or culvert flow, was routed downstream to the next subbasin concentration point.

If the concentration point at the ponded water site is at a subarea boundary then the outflowing hydrographs (over road and/or culvert flow) were saved in an external file (KO card), for introduction in the next subarea downstream (BI card). The cards for saving the outflowing hydrographs in an external file were placed at the end of the HEC-1 input file for each subarea.



NOTES

(1) Culvert flow under road into SUBAREA 2

(2) Weir flow over road into SUBAREA 2

(3) Weir flow over canal embankment into canal

(4) Flows exceeding canal capacity diverted to SUBAREA 3

FIGURE A-4
SPLIT-FLOW HANDLING AT
PONDED WATER SITE
GILBERT-CHANDLER FLOODPLAIN DELINEATION STUDY

The normal flow in the canals was entered in HEC-1 using IN and QI cards. Since there was already overtopping of the Eastern and Consolidated canals north of the Superstition Freeway (see appendix C) these data were input to the HEC-1 files for watershed 1 and watershed 4, respectively (north of the Superstition Freeway). The normal flow in Eastern Canal south of Southern Avenue is 250 cfs, and in Consolidated Canal 525 cfs. The canal flow hydrograph was routed downstream south of the Superstition Freeway using the Modified Puls routing routine in HEC-1 (see appendix B, section B.5).

The outflow hydrograph from the ponded water site over the canal embankment was combined with the normal flow or the routed canal flow at that point. Any flow exceeding the canal capacity at that location was diverted out. The hydrograph of the diverted flow was saved on an external file for later introduction in the downstream subarea adjacent to the west embankment of the canal (see figure A-4).

A schematic of subarea and canal routing of flood flows and the handling of split flow is presented as figure A-5.

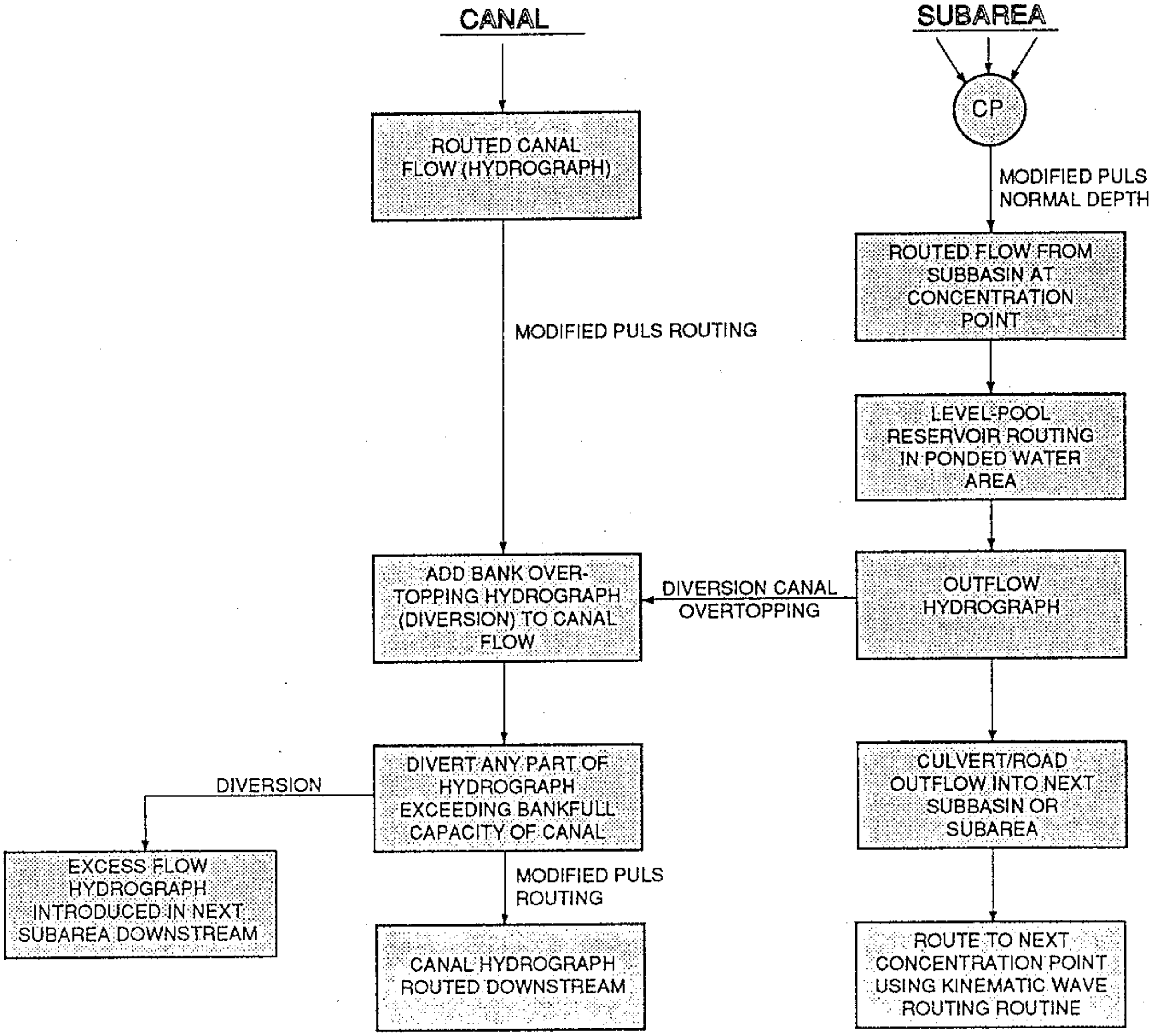


FIGURE A-5
SCHEMATIC OF SPLIT-FLOW HANDLING AT PONDED WATER SITES ALONG THE IRRIGATION CANALS

GILBERT-CHANDLER FLOODPLAIN DELINEATION STUDY

CHAPTER A.4

LINKING OF SUBAREAS

The study area was divided into 42 subareas. Each of the subareas is further divided into subbasins. A HEC-1 input file was created for each subarea separately. Flood hydrographs leaving the subarea were saved in files for retrieval during execution of subsequent subarea files. By using this approach, changes to the individual files could be made easily without long computer run times for the entire model.

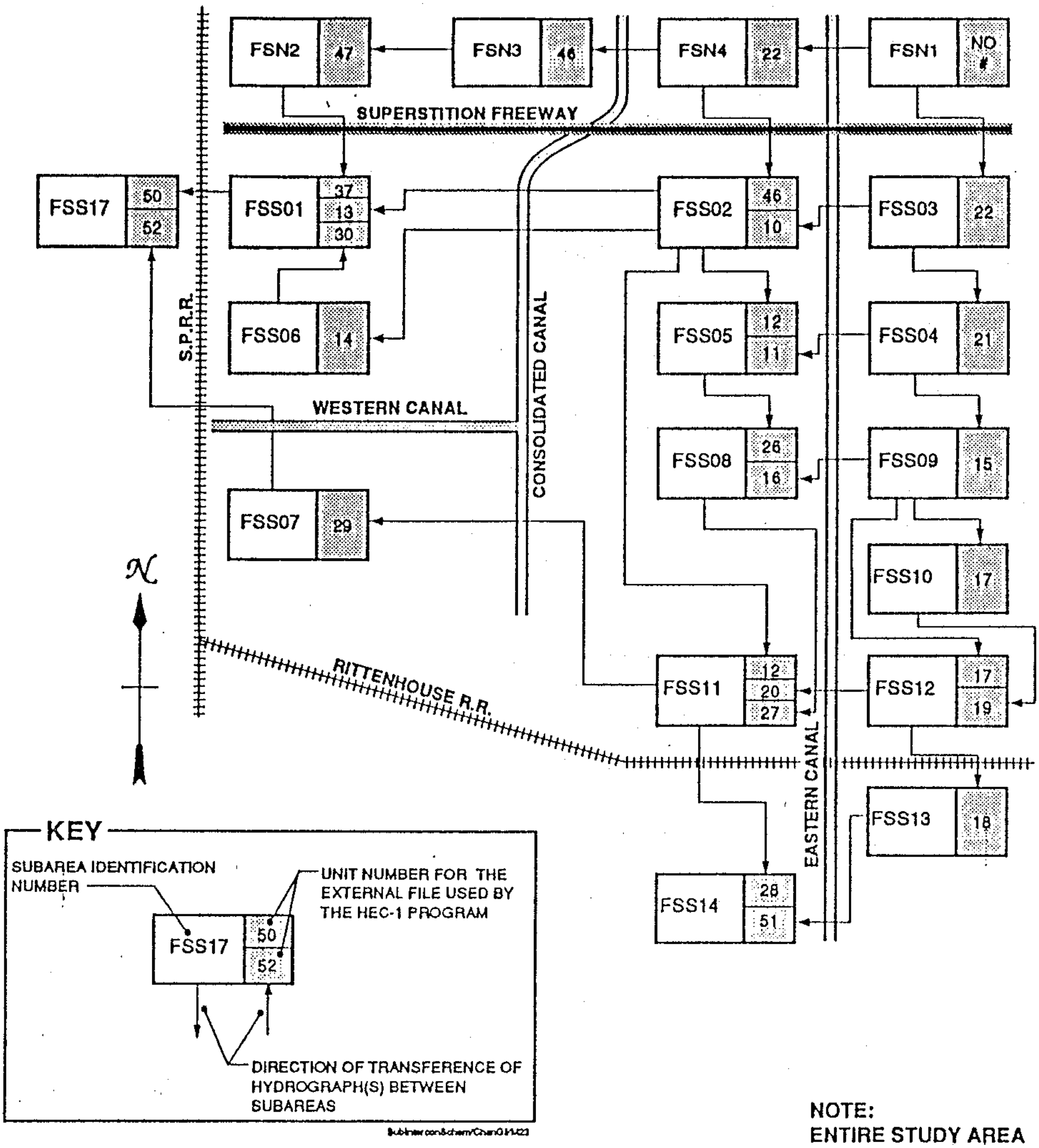
When the data for modeling of ponded water sites along the hydrologic boundaries were entered in the HEC-1 files, overflow from upstream subareas had to be accounted for. This was accomplished by using the KO and BI cards in HEC-1.

The 1988 PC version and the 900-ordinate version of the HEC-1 program (June, 1988) allow the user to save hydrographs on external output files using the KO card. These hydrographs can then be introduced in the next subarea downstream using the BI card.

HEC-1 has reserved certain output device numbers for input/output and scratch files. These device numbers are 5 through 7, 23 through 25, and 31 through 36 and 38. In principal all other two digit numbers up to 99 can be used as output device numbers.

Hydrographs were written to the external output files using unique names. More than one hydrograph could be written to a file. Generally, all hydrographs flowing out of a particular subarea were written to the same external file, if they were to be introduced in the same downstream subarea. Otherwise different device numbers were used. A typical schematic of the subareas and the external output files connecting them is shown as figure A-6.

Generally, outflow hydrographs from a subarea were diverted from the continuing flow and at the end of the HEC-1 input file retrieved and saved in an external output file. The subarea cumulative area is not transferred by saving the hydrographs this way. When the hydrograph is retrieved in the next downstream subarea, it is combined with the first subbasin hydrograph at the first concentration point within the subarea. It was not possible to route the retrieved hydrograph to the first concentration point because of the zero area. This, however, was not considered as affecting the results of the modeling significantly. Also, flood-prone areas within the study result from ponding and are only slightly affected by hydrograph peak discharges. The ponding sites are most sensitive to runoff volumes.



NOTE:
ENTIRE STUDY AREA NOT SHOWN

FIGURE A-6
TYPICAL SCHEMATIC OF SUBAREA LOCATION AND TRANSFERENCE OF HYDROGRAPHS BETWEEN SUBAREAS
GILBERT-CHANDLER FLOODPLAIN DELINEATION STUDY

CHAPTER A.5

BIBLIOGRAPHY AND REFERENCES

1. Arizona Department of Transportation, Highways Division. Hydrologic Design for Highway Drainage in Arizona. Arizona. Revised April 1975.

2. City of Chandler, Technical Design Manual Number 3 Storm Drainage System Design, Arizona, 1987.

3. City of Gilbert, Storm Drainage Policy. Arizona. Revised July 1987.

4. City of Mesa, Storm Drainage Policy. Arizona. Revised June 1983.

5. Flood Control District of Maricopa County, Hydrologic Design Manual for Maricopa County, Arizona, Draft April 1990.

6. Haestad Methods, Inc. 900-Ordinate Version 3.2c of HEC-1 Program. Waterbury, Connecticut. Revised 1989.

7. National Oceanic and Atmospheric Administration, Atlas 2, Volume VIII, Arizona, 1973.

8. U.S. Army Corps of Engineers, Hydrologic Engineering Center (COE/HEC). HEC-1 Flood Hydrograph Package, Users Manual. Davis, California. March 1987 (Rev.).

9. U.S. Army Corps of Engineers, Hydrologic Engineering Center, Introduction and Application of Kinematic Wave Routing Techniques Using HEC-1. Training Document No. 10, Davis, California. May 1979.

10. U.S. Army Corps of Engineers, Los Angeles District. Summary Report for Flood Control, Gila Floodway, Maricopa and Pinal Counties, Arizona. Los Angeles, California. September 1977.

11. U.S. Department of Agriculture, Soil Conservation Service. Soil Survey Eastern Maricopa and Northern Pinal Counties Area, Arizona. Washington, D.C. November 1974.

12. U.S. Department of Agriculture, Soil Conservation Service (USDA/SCS), Engineering Division. Urban Hydrology for Small Watersheds. Technical Release 55, June 1986 (Rev.).

APPENDIX B

HYDRAULIC ANALYSES

PAGE

TABLE OF CONTENTS

FIGURE		REFERENCED ON PAGE NUMBER

LIST OF FIGURES

TABLE		REFERENCED ON PAGE NUMBER

LIST OF TABLES

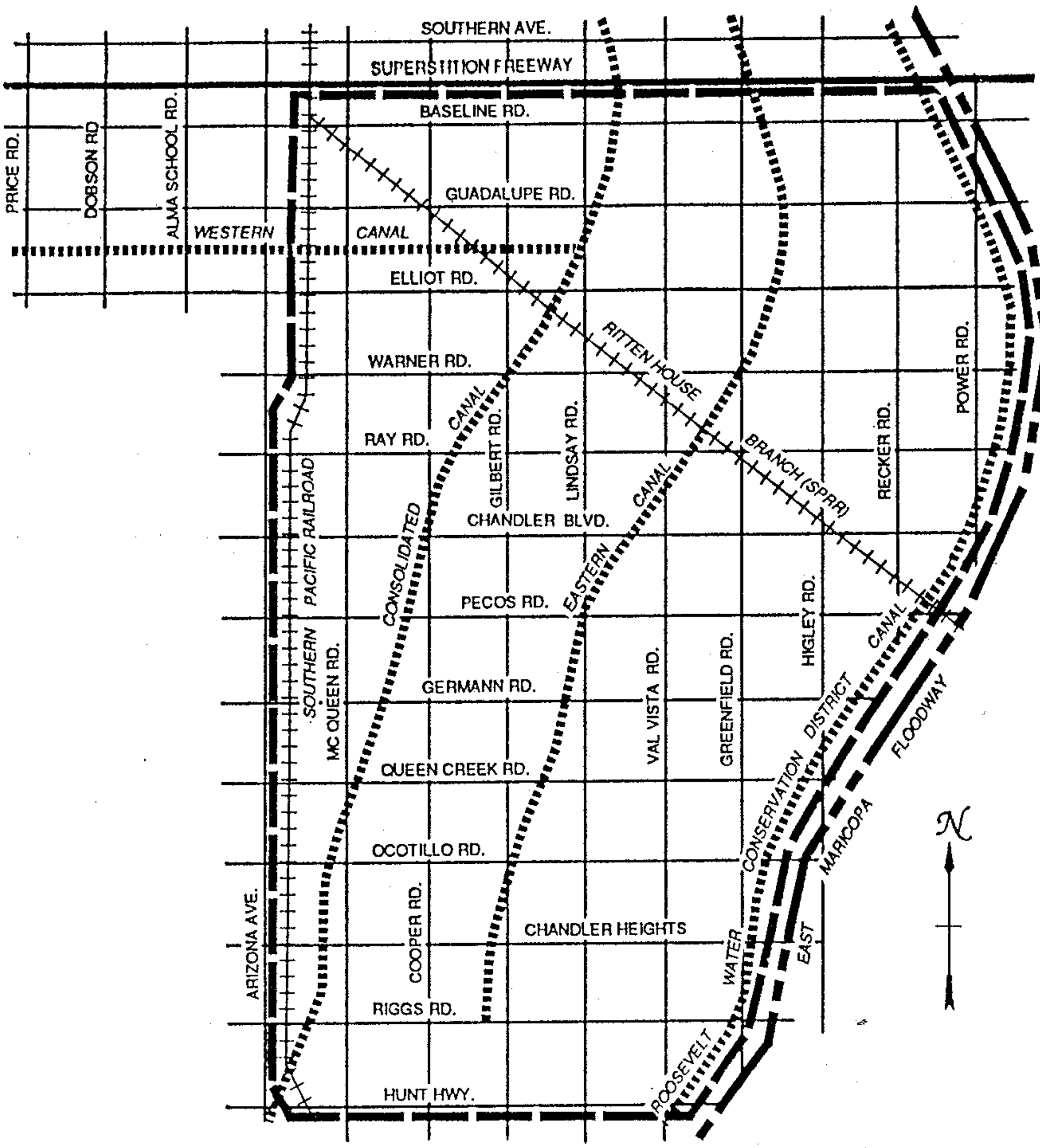
APPENDIX B

HYDRAULIC ANALYSES

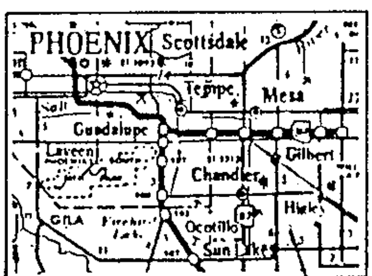
B.1 GENERAL

This flood insurance study (FIS) covers the area between the East Maricopa Floodway (EMF) (east boundary), Hunt Highway (south boundary), the Southern Pacific Railroad (SPRR) (west boundary) and the Superstition Freeway (north boundary); and, an area bounded in the east by SPRR, in the south by Western Canal, in the west by Tempe Canal (Price Road) and in the north by the Superstition Freeway (figure B.1). The study area is located entirely in Maricopa County, Arizona. Included in the study area is the incorporated area of the City of Gilbert; part of the incorporated area of the City of Mesa south of the Superstition Freeway; and, part of the incorporated area of the City of Chandler east of the SPRR. A portion of the area is not incorporated.

Floodflows originate within the study area. Drainage to the east of the EMF is intercepted and diverted to the south. Storm runoff follows the existing land form westerly. These flows are intercepted by the Eastern and Consolidated canals and by the SPRR alignments paralleling Arizona Avenue and Rittenhouse Road which form hydraulic barriers. Runoff is directed toward the southwest along the two canals. Flows along the Rittenhouse alignment are directed to the northwest and flows intercepted by the Arizona Avenue alignment of the SPRR are released to the west through existing bridges or by overtopping of the railroad. Roadways intersect these hydraulic barriers at intervals of approximately one-half mile. The water ponds at these roads until water overtops the road and continues along the barrier or overtops the canal or railroad. Flows spilling



BOUNDARY OF STUDY AREA



STUDY LOCATION

FIGURE B-1
STUDY AREA SOUTH OF THE SUPERSTITION FREEWAY
GILBERT-CHANDLER FLOODPLAIN DELINEATION STUDY

into a canal will flow down the canal until the bankfull capacity is reached. The canal capacity decreases in a downstream direction.

The study area has been modeled using the February 1981, revised June 1985 version of the HEC-1 computer program developed by the U.S. Army Corps of Engineers (COE), Hydrologic Engineering Center (HEC) modified by Haestad Methods, Inc. (1989 version 3.2c) to allow simulations up to 900 time ordinates. The detailed approach and method used in conducting the hydrologic analyses are presented in appendix A. This appendix presents the methodologies used in completing the hydraulic analyses within the flood-prone areas.

B.2 MAPPING AND SURVEY

Beginning on 13 August 1987 aerial targets were set at various locations throughout the Gilbert-Chandler Flood Insurance Study area. The targets were set at positions along the barriers for use as ground control points to obtain photogrammetric topographic data with the required accuracy. The datum for the vertical elevations used in the study is the City of Chandler Vertical Control Base List (1986), with bench marks 17, 18, 20, and 21 used as ERMs for vertical control in the Gilbert-Chandler study. The Chandler base list is founded on Arizona Department of Transportation brass cap "Ryan" and Maricopa County brass cap BM-F. These brass caps are tied to Tempe Butte, a USGS first order triangulation station which is tied to the National Geodetic Vertical Datum of 1929 (NGVD). A complete listing of the control for the FIS may be found in appendix D, sheet G-1 titled "Survey Control and Sheet Index."

Aerial photogrammetry, completed by McClain Harbers Co., Inc., was initiated with photography completed in September 1987. Mapping was completed along the Eastern, Consolidated, and Western (lateral 9.5) canals and along the SPRR-Rittenhouse Alignment and a spur line which parallels Arizona Avenue one-half mile to the east. The photo base topographic map (1"=400') included contours on a 2-foot interval. The locations of the ground control points, the major streets, and the barriers are shown.

Ground control points for the Gilbert-Chandler FIS are of three different types, horizontal control (HC), vertical control (VC), and read only points (RO) which are referenced to the Arizona State Plain Coordinate System and the NGVD. Horizontal control points (68) are identified with horizontal coordinates and elevations. Vertical control stations (31) have only elevations for reference. The read only control points (48) were used for quality control of the aerial survey. The photogrammetry subcontractor was not given elevations for the read only points but submitted the elevations obtained from the aerial survey. The photogrammetric data met criteria for third order or better.

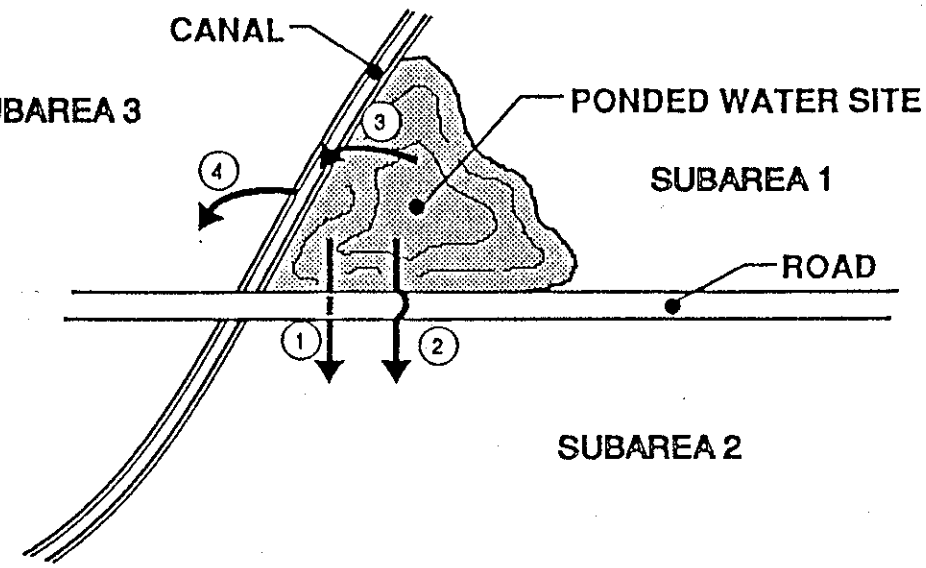
The aerial survey was supplemented with ground survey to supplement cross sections and provide site survey of hydraulic structures.

B.3 FLOW OVER BARRIERS

In general, the slope of the land in the study area is from the northeast to the southwest. Hydraulic barriers (canals and railroads) cross the area and intercept the runoff. The excess rainfall will flow to these barriers and then travel along the barriers. Major streets cross the barriers intercepting the flows along the barriers, causing water to pond at these intersections. The pond will fill gradually until it overtops either the canal railroad or the intersecting street, or both. If overtopping occurs into a canal, the flow is then added to the canal flow until the bankfull capacity is reached. The canal then spills on the downstream side adding to the runoff in the downstream subbasin. If overtopping occurs over a road or railroad, then the discharge is simply added to the downstream subbasin.

The COE HEC-1 computer model (level-pool reservoir routing routine) was used to determine the maximum ponding elevation. The required input to the model includes elevation-surface areas of the ponding site (reservoir), and elevation-outflow rate. The area of each ponding site at elevations from the lowest point to an elevation above the lowest barrier point was determined from topographical manuscript. This area-elevation data was taken at the contour interval (2 ft) and then interpolated as necessary. The area-elevation data for elevations above the low barrier point was interpolated with elevation increments of 0.1 ft. This allowed for a more exact modeling of the physical conditions.

The weir for each ponding site was determined from the survey data for cross sections or site surveys. A cross section was available along the centerline of each intersecting road. Several cross sections were used to define the top of canal/railroad elevation as the cross sections were perpendicular to these barriers. A composite weir (i.e. intersection of canal and roadway) was drawn and the low point was determined. Flow would begin at the low point and increase as the pond stage increased. Flows could overtop the canal/railroad or the intersecting street or both (figure B.2).



NOTES

(1) Culvert flow under road into **SUBAREA 2**

(2) Weir flow over road into **SUBAREA 2**

(3) Weir flow over canal embankment into canal

(4) Flows exceeding canal capacity diverted to **SUBAREA 3**

SplitFlowHandDiag/ChanGil/M23

FIGURE B-2
SPLIT-FLOW HANDLING AT PONDED WATER SITE
GILBERT-CHANDLER FLOODPLAIN DELINEATION STUDY

A stage-discharge table was created for the canal/railroad and the intersecting street. These were combined for use in the reservoir routing routine. Diversion cards were used to separate the outflow from the pond (reservoir) and the outflow which was retrieved as inflow into the appropriate downstream location. The elevation-discharge data was computed at increments of 0.1 ft for approximately 1 ft above the low point elevation of the weir. The following equation below was used to determine the discharge for each elevation increment:

$$Q = cLH^{2/3}$$

where:

Q = the discharge rate in cubic ft per second
c = the weir coefficient
L = the length of the weir
H - the available head

A weir coefficient for a broad crested weir (3.0) was used to compute flows over the street, canal bank, or railroad. The length of each weir is dependent on the depth of water above the weir. The weir-crest elevation varies gradually along the canal bank or roadway. A slight increase in height of flow over the weir increases the weir length and corresponding discharge over the weir significantly.

B.4 CULVERT FLOW UNDER AND ALONG BARRIERS

Drainage structures in the area of the Gilbert-Chandler FIS include a drainage channel along the Eastern Canal and timber bridges under the railroads. The Eastern Canal has the most extensive system for conveying excess water along the canal. The Roosevelt Water Conservation District (RWCD) has developed this system for retrieving runoff, either farm-field tailwater or rainfall runoff. This system includes an earthen ditch on the upstream (east) side of the Eastern Canal with culverts to convey flows under the intersecting roads. This collection system extends from the Superstition Freeway on the north to one-quarter mile south of Pecos Road. At Pecos Road a culvert carries the water to an inlet into the Eastern Canal Extension one-quarter mile south of Pecos. The RWCD maintains and operates the Eastern Canal south of this point. The Salt River Project (SRP) operates and maintains the Eastern Canal north of this point. The RWCD portion of the Eastern Canal is known as the Eastern Canal Extension (Extension). Further down the canal the RWCD has several storage reservoirs where excess canal flows can be stored. South of Pecos Road and along the SRP controlled Consolidated Canal there are no significant drainage structures paralleling the canal. There are however, several small tailwater ditches which drain into the canal but since the ponded areas overtop into the canal these tailwater ditches are not modeled. The SPRR drainage consists of occasional wooden culverts that allow drainage from one side of the railroad to the other. The drainage structures within the study area do not have the capacity to convey the peak runoff resulting from the design storm of 24-hour duration for 100-year frequency.

Data collected by survey crews along each of the barriers for the culvert crossings allowed an analysis of each crossing and an estimate of the discharge at various stages. The stage discharge tables were determined using charts from the U.S. Department of Transportation publication, <u>Hydraulic Design of Highway Culverts (FHWA 1985)</u>. Inlet control was used to analyze hydraulic capacities. There were four main types of culverts in the study area: concrete box, concrete pipe, corrugated metal pipe, and wooden trestle-type made of railroad timbers. Using the hydraulic charts and the geometry of the culvert, a stage-discharge table for each structure was developed to the elevation of approximately 1 ft above the low elevation of the weir. At the low weir elevation the flow will begin and in most cases quickly become the largest portion of the outflow (figure B.2).

The elevation-flow data for the culverts were combined with the weir data to model outflow from the ponded sites. The combined weir-culvert flows were then diverted and retrieved as input into the appropriate subbasin.

B.5 CANAL FLOWS AND CAPACITIES

Two of the major barriers in the Gilbert-Chandler FIS area are the Consolidated and Eastern canals operated by SRP. These canals flow generally north to south from the Salt River to the Maricopa County line at Hunt Highway. These canals deliver irrigation water to the farms and ranches in the Gilbert-Chandler area. The RWCD delivers water to the area between the Eastern Canal and the RWCD canal. RWCD also operations and maintains the Eastern Canal from one-quarter mile south of Pecos Road to the terminus at Riggs Road. Delivery ditches (laterals) generally run to the west carrying the irrigation water from the Eastern and Consolidated canals to the water users. On other canal in the study area is known as the Western Canal (lateral 9.5). This SRP canal is an earthen channel which has been abandoned by SRP and is currently used as a drainage facility. The Western Canal begins at the Consolidated Canal between Guadalupe and Elliot roads and flows westward to the Tempe Canal at Price Road. Flows are then discharged into the Tempe Canal.

The Eastern Canal is largest at the freeway and as it progresses southward and water deliveries are made it becomes smaller. The bottom width at the north end is about 20 ft and at the south end is about 4 ft. The average slope is about 0.05 percent although it varies from 0.08 percent to 0.04 percent along the way. The normal operating flow at points along the alignment are shown in table B.1.

TABLE B.1
Eastern Canal Normal Flows
October 1989

SR 369 to Warner Road	200 cfs
Warner Road to SPRR	225 cfs
SPRR to Eastern Canal Extension	188 cfs
Eastern Canal Extension to Ocotillo Road	125 cfs
Ocotillo Road to Chandler Heights Road	75 cfs

Source(s): Salt River Project, Roosevelt Water Conservation District

The Eastern Canal and Eastern Canal Extension are concrete lined the entire length.

The Consolidated Canal is larger and longer than the Eastern Canal and serves a larger area. The bottom width at the freeway is about 40 ft and the bottom width at the SPRR on the south is about 7 ft. The Consolidated Canal is concrete lined from the freeway to Gilbert Road and is earth from that point. The slope of the Consolidated Canal varies from 0.2 percent to 0.005 percent. The normal operating flows are shown in table B.2.

TABLE B.2
Consolidated Canal Normal Flow
October 1989

SR 360 to Guadalupe road	525 cfs
Guadalupe Road to Elliot Road	450 cfs
Elliot Road to Warner Road	400 cfs
Warner Road to Williams Field Road	325 cfs
Williams Field Road to Germann Road	250 cfs
Germann Road to Ocotillo Road	100 cfs
Ocotillo Road to SPRR	15 cfs

Source: Salt River Project

The canals intercept the rainfall runoff and direct it south until intercepted by intersecting streets where ponds are formed. There are many locations where ponded flows predicted by the HEC-1 model overtop the upstream bank into the canals. The emergency procedure for SRP in the case of runoff into the canals is to protect the canals from failure (breaching of the bank). The steps taken to protect the canal are: (1) cut water at the headworks (spill back in the Salt River bed), (2) fill the laterals, (3) spill on the fields if possible, and (4) carry as much of the water as possible through the canal system to existing drains.

The HEC-1 model was developed to route the overflow which enters the canal. These flows are added to the normal canal flow and using the modified PULS routing sequence carried downstream (southward) to a point where the canal capacity is reduced by channel size or another inflow is encountered, and bankfull capacity is achieved. At this point the excess flow is spilled into the downstream subarea and added to the runoff in the subbasin where the spill occurs. For the purposes of this study it was assumed that the canal bank would not breach. The bankfull capacity continues downstream to the next inflow point and the process is repeated. The canal section used in the routing portion of the HEC-1 model from survey data collected by SRP in October 1985 and October 1986. The datum used for the SRP survey is different from the one used in the Gilbert-Chandler FIS, however, the canal size and slope are the only data used in the model and this information is not affected by datum selection. The Manning's "n" value used for the canal sections lined with concrete is 0.018 and for the unlined portions 0.023. These "n" values allow for some irregularities and vegetation in the lining.

B.6 FLOW BETWEEN PONDED AREAS

There are very few places along the barriers between ponded areas where free-flowing water occurs. There are only five reaches which were analyzed using the COE HEC-2 water surface profile computer model. Three of these reaches are in agricultural areas along the Eastern Canal south of the developed area of the City of Gilbert. The two other reaches are within developed areas of the City of Gilbert along the SPRR-Rittenhouse alignment. The conveyance corridors are all similar with a well defined bank on one side (canal bank or railroad) and no bank on the other side.

The friction coefficients (Manning's "n") values for the flowing water areas were generated according to the hydraulics section of the U.S. Department of Agriculture, Soil Conservation Service (SCS) Engineering Handbook, Section 4, Supplement B. To a basic "n" the modifying values for each applicable variation in the channel are added. Variations are in five areas: (1) degree of irregularity, (2) character of variations in size and shape of channel, (3) relative effect of obstructions, (4) vegetation and flow conditions, and (5) amount of meandering of the channel. Field review and aerial photography were used to verify estimates. Worst case seasonal conditions were assumed for vegetative cover.

Hydraulic cross sections, photogrammetrically digitized, were used in the HEC-2 model. Reach lengths between cross sections were measured from the topographic mapping (1"=400'). The maximum pond elevation was used as the initial water surface elevation, at locations where the model predicted supercritical flows the critical depth was used for the water surface elevation.

B.7 *FLOOD ZONE DELINEATION*

Floodplain delineations were completed based upon runoff from the 100-year, 24-hour precipitation. Flooding caused from storms of other recurrence intervals were not investigated. Encroachments upon the floodplain would result in increased stages of water which would cause increased flow over the barriers at many locations. Encroachments may be permitted provided adequate capacity to convey flows are provided and if occupied volumes of storage are replaced such that the maximum ponding elevation is unaffected. Therefore, a floodway was not determined.

The HEC-1 reservoir routing at the ponding sites computed a maximum pool elevation at each site. This maximum stage was used to delineate the floodplain. At locations where free flowing water occurs along the barrier, the HEC-2 computer program was used to compute water surface elevations. The photo-topographic maps (2-foot contour, 1"=400') were used to determine the extent of the floodplain. Water-surface elevations between contours were interpolated. The hydraulic cross sections were used as a check. At locations where the HEC-2 program predicted flows would overtop the barrier an elevation 6 inches above the top of the barrier was used.

B.8 LIMITATIONS

The size of the Gilbert-Chandler FIS required methodologies and procedures be used which control study costs and yet provide reasonable accuracy and detail to establish continuity throughout the study. The methodologies and assumptions were discussed by and are based upon the judgment of the entities involved with the study.

The FIS is based upon the current physical information researched and analyzed in the study area. This information was modeled as accurately as possible to determine the flooded zones. This flooding information however, is limited by the lack of historical data for comparison. Historical records would allow calibration or "fine tuning" of the model and improve accuracy.

Minor changes in the configuration of the ponded sites in the study area may also cause the hydraulics of the model to change significantly. The weir lengths are generally very long and a small increase in head on the weir will cause a large increase in the outflow rate. Outflow will also be affected if a change in the weir elevation occurs. Most of the weirs modeled for the study are earthen roadways for operation and maintenance of the canals in the area. These weirs may be raised or lowered as the normal or special operations of the canal require (i.e. cleaning of the canal or adding structures to the system). The crests of the streets intersecting the barriers may also be changed over time as the asphalt streets are overlayed or reconstructed.

Canal operation during a major storm event may also affect the results predicted by the model. The steps discussed in B-5 may or may not be successful or appropriate when a particular storm occurs. Protection of the structural integrity of the canal is an important consideration for the operators. Procedures may vary depending on the circumstances.

Rapid growth has been a characteristic of the Gilbert-Chandler area in the last decade. This has caused a rapid urbanization of the agricultural area which lies within the study area. If this trend continues, the runoff will continue to decrease in the area of the FIS because of the current policies for retention. Also new and better drainage policies and retention facilities will decrease flooding problems.

B.9 REFERENCES

1. City of Chandler, Vertical Control Base List. Chandler, Arizona. 1986.

2. Salt River Project Survey Transmittal, November 15, 1988. Book Nos. 478, 479. 1985.

3. Salt River Project Survey Transmittal, November 15, 1988. Book No. 493. 1986.

4. U.S. Army Corps of Engineers, Hydrologic Engineering Center, Generalized Computer Program, HEC-1 Flood Hydrograph Package. Davis, California. 1981. Revised June 1988.

5. U.S. Army Corps of Engineers, Hydrologic Engineering Center, Generalized Computer Program, HEC-1 Flood Hydrograph Package. Davis, California. 1981. Revised June 1985. Modified by Haestad Methods for 900-Ordinates. Haestad Methods, Incorporated, Hydrology/Hydraulics Software, HEC-1 Version 3.2c. Waterbury, Connecticut. 1989.

6. U.S. Army Corps of Engineers, Hydrologic Engineering Center, Generalized Computer Program, HEC-2 Water Surface Profiles. Davis, California. 1982.

7. U.S. Department of Agriculture, Soil Conservation Service, Engineering Handbook, Hydraulics Section, Subsection 4.4, Supplement B. August 1956.

8. U.S. Department of Transportation, Federal Highway Administration, Hydraulic Design of Highway Culverts. Hydraulic Design Series No. 5. National Technical Information Service, Springfield Virginia. 1985.

APPENDIX C

NORTH OF FREEWAY

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FINAL REPORT

RUNOFF FROM AREA NORTH OF THE SUPERSTITION FREEWAY IMPACTING GILBERT-CHANDLER AREAS ARIZONA

SEPTEMBER 1990

Prepared By
Franzoy Corey
7776 Pointe Parkway West, Suite 290
Phoenix, Arizona 85044
(602) 438-2200

PAGE

TABLE OF CONTENTS

FIGURE | REFERENCED ON PAGE NUMBER

LIST OF FIGURES

(All Tables Follow Chapter)

TABLE | REFERENCED ON PAGE NUMBER

LIST OF TABLES

(All Tables Follow Chapter)

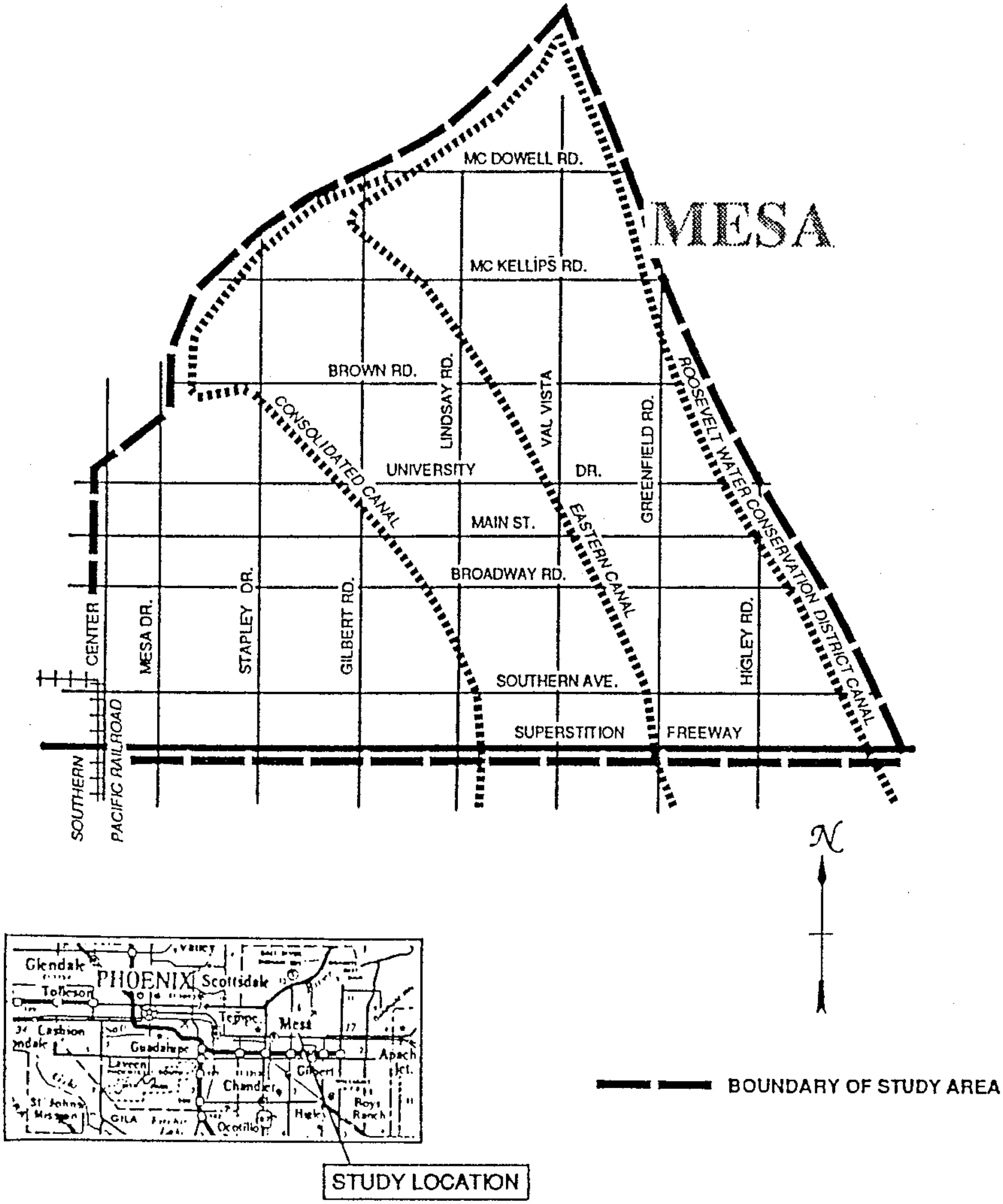


FIGURE 1.1
STUDY AREA NORTH OF THE
SUPERSTITION FREEWAY
GILBERT-CHANDLER FLOODPLAIN DELINEATION STUDY

CHAPTER 1

INTRODUCTION

1.1 PURPOSE OF THE STUDY

Five crossings of the Superstition Freeway and their contributing areas were included in the scope of work of "Aerial Mapping and Floodplain Delineation of the Gilbert Chandler Areas, Arizona" to determine their effects on the area to be mapped. These crossings are: Greenfield Road, Eastern Canal, Consolidated Canal, Lindsay Road, and Southern Pacific Railroad (SPRR) (see figure 1.1).

The purpose of this study was to determine if runoff resulting from the 100-year 24-hour storm (3.8 inches) from the area north of the Superstition Freeway would cross the freeway at the above mentioned crossings and impact the floodplain delineation and mapping of the Gilbert-Chandler areas. The area north of the Superstition Freeway is not included in the floodplain delineation and mapping of the Gilbert-Chandler areas. Therefore, information from previous studies by others was researched and no additional field information was collected.

1.2 DESCRIPTION OF AREA NORTH OF SUPERSTITION FREEWAY

The area contributing to the runoff at the Eastern Canal and Greenfield Road crossings with the Superstition Freeway is bounded on the east by the East Maricopa Floodway (EMF), on the south by the Superstition Freeway, in the west by Eastern Canal, and on the north by a ridge just south of Southern Canal, Mesa-Lehi grade break (watershed 1). The area contributing to the runoff at the Consolidated Canal crossing is bounded on the east by Eastern Canal, on the south by the Superstition Freeway, and on the west by Consolidated Canal and on the north by the Mesa-Lehi grade break (watershed 4). The area contributing to the runoff at the Lindsay Road crossing is bounded on the east by Consolidated Canal, on the west by Lindsay Road, and by the Superstition Freeway on the south (watershed 3). The area contributing to the runoff at the SPRR crossing is bounded on the east by the Consolidated Canal and Lindsay Road, on the south by the Superstition Freeway, on the west partly by the SPRR and Center Street, and on the north by Mesa and Consolidated Canals and the Mesa-Lehi grade break (watershed 2).

The Superstition Freeway is a significant factor affecting the runoff pattern. The freeway diverts flow from its north-east to south-west flow direction to travel due west. The Eastern and Consolidated canals, both irrigation canals crossing the study area from north to south, also intercept the normal drainage pattern. Generally, the berms of the canals are 2-3 ft higher than the surrounding natural ground elevation. The SPRR forms an effective hydrologic boundary at the west side of the study area. Rapid urban development since the late 1950s created numerous artificial boundaries such as high-crowned or elevated streets and highways.

The Superstition Freeway drainage system, in the study area, (designed for the 50-year 24-hour storm) consists of a system of detention basins and a drainage channel north of the freeway. The detention basins were constructed as a part of the City of Mesa's storm drainage system. Detention basins are located at the Eastern Canal/Greenfield Road and Consolidated Canal crossings with the Superstition Freeway. Two detention basins are located at the SPRR crossing with the freeway, one north and one south of the freeway. These last basins are connected by a 10 ft x 10 ft box culvert. Three more detention basins are located just north of the Superstition Freeway between Consolidated Canal and the SPRR. The detention basins are connected by a concrete-lined drainage channel, in this study referred to as the ADOT-drainage channel.

1.3 METHODOLOGY

Runoff resulting from the 100-year 24-hour storm from the area north of the Superstition Freeway was estimated using the Soil Conservation Service's (SCS) unit-hydrograph method. This storm produces a rainfall amount of 3.8 inches.

It was assumed that the Central Arizona Project Canal and the East Maricopa Floodway will intercept all runoff coming from the east and that no flow would cross any of the hydrologic boundaries (Eastern and Consolidated canals, railroad) in the study area except at the freeway crossings. Within the watersheds artificial boundaries like berms of small canals or elevated roads were not considered as barriers. The floodwaters were routed through the detention basins north of the freeway at Eastern Canal/Greenfield Road, Consolidated Canal and SPRR crossings. However, none of the other detention basins, part of the Mesa storm drainage system, were included directly in the study except the three detention basins located between Consolidated Canal and the SPRR just north of the Superstition Freeway (see chapter 3).

As part of the discussion about study methods and criteria for the "Aerial Mapping and Floodplain Delineation of the Gilbert-Chandler Areas, Arizona", Flood Control District of Maricopa County (FCDMC) provided Franzoy Corey with the precipitation time distribution Type II for the 100-year 24-hour storm. FCDMC gave the direction to use the antecedent soil moisture condition II for this study.

Discussion about the study approach for runoff determination for the area north of the Superstition Freeway resulted in the following additional direction from FCDMC:

- For agricultural land the curve number for orchards of 65 should be used and for straight row crops the curve numbers from TR55 should be used.

- Since September 1972, the City of Mesa has had a subdivision ordinance in effect establishing on-site storage requirements for storm runoff. Prior to 1972 subdivisions were not required to incorporate on-site storage. The ordinance applies to all new developments, and requires that all runoff from precipitation from a 50-year 24-hour storm (3.0 inches) must be retained within the subdivision boundaries. For developments completed between 1972 and 1980, a retention effectiveness of 50 percent should be used; and for developments after 1980 the retention effectiveness should be 70 percent.

- Time of concentration should be computed using the upland method. The lag time is 0.6 times the time of concentration. Lag times should be computed for each different land use to account for its runoff characteristics.

- The kinematic wave method of channel routing should be used to route hydrographs (subsequently revised to normal depth routing method).

CHAPTER 2

INPUT TO HEC-1

The main purpose of the HEC-1 flood hydrograph model (HEC, 1987) is to simulate the hydrologic processes during flood events. The precipitation-to-runoff process can be simulated for large complex watersheds. The model can be used as a basic tool to determine runoff from either a historical or synthetic (or design) storm in planning flood control measures. Watershed precipitation-runoff simulation is the main function of the program. The HEC-1 watershed model uses spatially and temporally lumped (or averaged) parameters to simulate the precipitation and runoff process. The time or space discretion may be changed by modifying the size of the subareas, routing reaches, or the computation interval. The HEC-1 model is a "single event" model intended to simulate discrete storm events.

2.1 SUBAREA AND SUBBASIN DELINEATION AND LAND USE

The area north of the Superstition Freeway was divided into four watersheds based on the hydrologic boundaries formed by the East Maricopa Floodway, Eastern Canal, Consolidated Canal, Lindsay Road and SPRR. The watersheds were further divided into subareas using the subarea delineation of the U.S. Department of the Army, Corps of Engineers (COE) study, (1977) as a guide. Subarea boundaries were modified to follow major roads or edges of leveled fields. Watershed 1 (Greenfield Road/Eastern Canal crossing) was divided in four subareas; watershed 2 (SPRR crossing) and watershed 3 (Lindsay Road crossing) in one subarea; and, watershed 4 (Consolidated Canal crossing) in four subareas. Each subarea was further divided in subbasins. Subbasins are generally a section (640 acres) or parts of sections, except in watershed 2 where some subbasins are larger than

640 acres. The subbasins were identified by letters (A, B, C, etc.), continuous through the watershed. The concentration point of each subbasin was the intersection of section-line or midsection-line roads, or the intersection of a road and hydrologic boundary.

Within subbasins different land uses may exist. Six types of land use were recognized in the area north of the Superstition Freeway:

- Urban development before 1972
- Urban development from 1972 to 1980
- Urban development from 1980 to present
- Agricultural development, orchards
- Agricultural development, row crops
- Open area (including parks, golf courses, etc.)

The different land uses were determined from a map in the Yost and Gardner report (1973) for urban developments before 1972; from aerial photography dated December 1981 (Landis Aerial Surveys); and, aerial photography dated May 1987 (McClain-Harbers Co., Inc.) for current land uses. Each land-use area was planimetered from USGS 7.5 min quadrangles. Areas and relevant data for each land-use type in a subbasin were entered on hydrologic data sheets. The total area per land-use type in each subarea is presented in table 2.1. The subbasins (e.g. A-G) contributing to the runoff in each subarea are indicated in the table between parentheses following the subarea labels.

A map showing the subbasins, subbasin concentration points, routing direction and the different land-use types for the area north of the Superstition Freeway is presented as appendix A. Appendix B is a floppy disk with a "Lotus" worksheet containing all hydrologic data sheets for subbasins within the area north of the Superstition Freeway.

2.2 PRECIPITATION

The precipitation time distribution USDA Soil Conservation Service's Type II distribution for the 100-year 24-hour storm was used as rainfall input to the HEC-1 program. The total rainfall amount for this storm was 3.8 inches.

2.3 INTERCEPTION/INFILTRATION (LOSS-RATE ANALYSIS)

HEC-1 has a choice of four different methods to compute the interception/infiltration part of the rainfall-runoff process. These methods simulate the interception of precipitation by vegetation, depressions, etc. and the accumulation of moisture in the soil. Two important factors should be noted about the precipitation-loss computation in the model. First, precipitation which does not contribute to the runoff process is considered to be lost from the system. Second, the equations to compute the losses do not provide moisture or surface-storage recovery (HEC, 1987).

In this study the SCS curve number method was used to estimate interception/infiltration losses. The SCS curve number is a measure of the soil's runoff potential. The choice of the curve number is critical in the runoff computation. The higher the curve number, the greater the runoff potential. The curve number is determined by the soil type, land use and land treatment, and antecedent soil moisture. The curve numbers used for this study were chosen

TABLE 2.1 (Page 1 of 2)
Area North of Superstition Freeway, Area and Curve Number for Each Land-Use Type and Slope for Each Subarea
September 1990

Area Identification	Area (ac)	Area (mi^2)	Curve Number	S (ft/ft)
1. Eastern Canal, Greenfield Road & Superstition Freeway Crossing (Watershed 1)	7193.9*	11.24*		
Subarea 1 (A-G)	2394.2	3.74		0.0050
Urban Devel. 1972-1980	129.6	0.20	75	
Urban Devel. 1980+	780.1	1.22	75	
Agric/Orchard	1239.1	1.94	65	
Open	245.4	0.38	79	
Subarea 2 (H-M)	2346.4	3.67		0.0066
Urban Devel. 1972-	73.4	0.11	75	
Urban Devel. 1972-1980	250.5	0.39	75	
Urban Devel. 1980+	714.5	1.12	75	
Agric/Orchard	1308.0	2.04	65	
Subarea 3 (N-R)	1786.1	2.79		0.0071
Urban Devel. 1972-	107.8	0.17	75	
Urban Devel. 1972-1980	676.5	1.06	75	
Urban Devel. 1980+	693.3	1.08	75	
Agric/Orchard	187.8	0.29	65	
Agric/Row Crops	17.7	0.03	78	
Open	103.0	0.16	79	
Subarea 4 (S-T)	648.1	1.01		0.0050
Urban Devel. 1980+	117.7	0.18	75	
Agric/Orchard	450.4	0.70	65	
Agric/Row Crops	80.0	0.13	78	
2. Consolidated Canal & Superstition Freeway Crossing (Watershed 4)	5900.8*	9.22		
Subarea 1 (A-C)	1098.3	1.72		0.0034
Urban Devel. 1972-	123.5	0.19	83	
Urban Devel. 1980+	810.7	1.27	83	
Agric/Row Crops	64.5	0.10	86	
Open	99.6	0.16	86	
Subarea 2 (D-I)	2678.0	4.18		0.0038
Urban Devel. 1972-	871.9	1.36	77	
Urban Devel. 1972-1980	388.2	0.61	77	
Urban Devel. 1980+	1400.7	2.19	77	
Open	17.2	0.03	80	

TABLE 2.1 (Page 2 of 2)
Area North of Superstition Freeway, Area and Curve Number for Each Land-Use Type and Slope for Each Subarea
September 1990

Area Identification	Area (ac)	Area (mi^2)	Curve Number	S (ft/ft)
Subarea 3 (J-N)	1593.6	2.49		0.0060
Urban Devel. 1972-	145.0	0.23	79	
Urban Devel. 1972-1980	836.8	1.31	79	
Urban Devel. 1980+	413.2	0.65	79	
Agric/Row Crops	63.8	0.10	82	
Open	134.8	0.21	83	
Subarea 4 (O-P)	530.9*	0.83*		0.0050
Urban Devel. 1972-1980	45.5	0.07	79	
Urban Devel. 1980+	404.4	0.63	79	
Open	71.0	0.11	83	
3. SPRR & Superstition Freeway Crossing (Watershed 2) (A-H)	5904.3*	9.23*		0.0025
Urban Devel. 1972-	4166.7	6.51	75	
Urban Devel. 1972-1980	1031.9	1.61	75	
Urban Devel. 1980+	528.5	0.83	75	
Agric/Orchard	43.6	0.07	65	
Open	109.4	0.17	79	
4. Lindsay Road & Superstition Freeway (Watershed 3) (A-B)	74.5	0.12		0.0002
Urban Devel. 1972-	21.4	0.03	75	
Open	53.1	0.08	79	
TOTAL AREA	19,073.5	29.80		

*Number includes area of detention basins

Source: Franzoy Corey

from table 2.2 in the USDA/SCS TR55, (1986). This table gives runoff curve numbers for selected agricultural, suburban, and urban land use for soil moisture condition II.

The soil survey report "Eastern Maricopa and Northern Pinal Counties Area, Arizona" (USDA/SCS, 1974) was used to determine the soil types in the study area. The soil-type list (exhibit A-1) in TR55 gives the particular hydrologic soil group to which a soil belongs. Soils are classified into four hydrologic soil groups according to their minimum infiltration rate, which is obtained for bare soil after prolonged wetting. Hydrologic soil group A consists of soils with the lowest runoff potential; soils in hydrologic soil group D have the highest runoff potential. For each subarea, the percentage of area in each hydrologic soil group was determined. These percentages were used to find a weighed curve number for a certain land-use type. For example, if in a subarea, 60 percent of the area is classified as soil group B and 40 percent as soil group D. Then a weighed curve number for urban land use is computed as follows:

Hydrologic Soil Group	Percentage	Curve Number (CN)	(CN x %)
B	60	75	4500
D	40	87	3480
	100		7980

The weighed curve number is (7980/100=) 79.80; which is rounded to 80.

Soils in the area north of the Superstition Freeway belong dominantly to hydrologic soil group B with some areas having soils in hydrologic soil groups C and D. The curve numbers used for each land-use type in each subarea are presented in table 2.1.

Urban areas consist of pervious and impervious areas. Losses are negligible for the impervious areas. The runoff curve numbers for urban and residential districts in table 2.2a (p. 2-5) in TR55 are composite curve numbers, i.e. the average percent impervious for those areas is expressed in curve numbers. Other assumptions for the use of the curve numbers for urban land use is that impervious areas are directly connected to the drainage system, impervious areas have a curve number of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.

On the LS-card (describing the rainfall losses for the SCS curve number method in HEC-1), the percent impervious specified accounts for the section-line and midsection-line roads. For a typical sq mile (640 acres) the area occupied by section-line and midsection-line roads was computed and expressed as a percentage of the total area (640 acres). The percentages were 2.06 and 4.11 for agricultural and urban development, respectively. In agricultural developments, the section-line roads were assumed to be 28 ft wide and the midsection-line roads 25 ft. In urban developments, the section-line roads were assumed to be 79 ft wide including 5 ft sidewalks, and 0.5 ft curb, the midsection-line roads 30 ft. Half of the widths of section-line roads were used to compute the percentage impervious area for a typical square mile. Instead of computing the percentage impervious from section- and midsection-line roads for each different land-use type these typical percentages were applied. If an agricultural development was totally surrounded by urban development, the percentage impervious of urban development was applied to this area and vice versa.

Land treatment encompasses agricultural practices, such as contouring or terracing and management practices, such as crop rotation and conservation tillage. Curve numbers given in SCS TR55 (USDA/SCS, 1986) for cultivated agricultural lands are mainly applicable to regions receiving more rainfall than Arizona. In those regions yield reduction can occur if the runoff cannot flow from the fields. In Arizona where irrigation is necessary to grow crops most fields are bordered such that as little as possible irrigation water is lost, especially if flood irrigation is practiced. COE (1977) suggests in their study that an average of 2 inches of rain can be stored on the field before runoff occurs. In orchards enclosed by berms little or no runoff occurs. In this study, however this retention in agricultural land was not accounted for, resulting in a more conservative runoff analysis.

The antecedent soil moisture condition is the index of watershed wetness before a storm. There are three levels of antecedent soil moisture conditions: I - lowest runoff potential; II - average condition; and, III - highest runoff potential - i.e., the watershed is practically saturated from antecedent rains. An antecedent soil moisture condition II was used in this study.

Precipitation losses were not computed for a specified percentage of the area labeled as impervious. An average precipitation loss was determined for a computation interval and subtracted from the rainfall hyetograph. The next step in HEC-1 simulation is to convert a hyetograph of rainfall excess into a runoff hydrograph from the subbasin.

2.4 RETENTION IN URBAN DEVELOPMENTS

Part of the area north of the Superstition Freeway was developed before Mesa's drainage policy came into effect in 1972. The areas developed after 1972 are required to retain all runoff from a 50-year 24-hour storm within the subdivision boundaries. This corresponds with a precipitation amount of 3 in. Retention basins in subdivisions developed between 1972 and 1980 were considered 50 percent effective; and retention basins in subdivisions developed after 1980 were considered 70 percent effective.

The runoff volume from the 50-year 24-hour storm was computed using HEC-1 for subdivisions developed between 1972 and 1980, and for subdivisions developed after 1980. The depth-duration data for the 50-year 24-hour storm were computed following the procedure outlined in Arizona Department of Transportation's (ADOT) "Hydrologic Design for Highway Drainage in Arizona," (1975). The PH-card was used to enter this data into the HEC-1 model. To account for retention in above mentioned subdivisions, a dummy retention basin was included in the model. The maximum retention of such a basin was the volume of runoff from the 50-year 24-hour storm multiplied by the percentage of effectiveness. In the model the diversion cards (DT, DI and DQ cards) were used to describe the dummy retention basin with basin outflows beginning after appropriate volumes had been retained.

2.5 SCS DIMENSIONLESS UNIT HYDROGRAPH

Rainfall excesses are computed for each time interval by subtracting the interception/infiltration loss from the incoming rainfall. To convert the rainfall excess hyetograph into a runoff hydrograph the SCS dimensionless unit hydrograph method (USDA/SCS, 1972) was used.

Input data for the SCS dimensionless unit hydrograph method consist of a single parameter, the lag time. The lag time is the time in hours between the center of mass of rainfall excess and the peak of the unit hydrograph. The SCS lag equation (USDA/SCS, 1972) was used to compute the lag time for each land-use type in a subbasin:

$$L = 0.6T_c = 0.6\ (l_p/(3600*v))$$

where,

L = lag time, h

T_c = time of concentration, h

l_p = length of flow path (or hydraulic length, longest flow path in subbasin), ft

v = average velocity, ft/s from figure 3.1 of TR55 (1986) for various surfaces

3600 = conversion factor from seconds to hours

The average subarea slope was determined from USGS 7.5 min quadrangle maps with 10-foot contours. Overland flow charts in figure 3.1 of TR55 show average velocity as function of watercourse slope and type of land surface, paved or unpaved. Slopes in the area north of the Superstition Freeway varied from 0.0025 to 0.0071 ft/ft (0.25 to 0.71 percent). The graph of figure 3.1 had to be extended because of these flat slopes, to find the average flow velocities in the study area. These average flow velocities for the subarea were assumed to be valid in each subbasin as well. With the available data (USGS 7.5 min quadrangle maps), it was not possible to determine a slope for each subbasin.

The hydraulic length (longest flow path) in each land use within a subbasin was measured from the USGS 7.5 min quadrangle maps. Then, the lag time for each land use was calculated, given the average flow velocities of the subarea in which the subbasin was located. The largest area of a particular land use within the subbasin was used to calculate the lag time for that land use.

Hydrologic data sheets were made for each subbasin in a watershed. Subbasin characteristics such as area, curve number, hydraulic length, lag time, percentage impervious for each land-use type, and subbasin concentration point were recorded. Also the subarea average natural ground slope and average overland flow velocity for paved and unpaved area were included (appendix B).

2.6 ROUTING OF FLOOD HYDROGRAPHS

Flood routing is used to simulate flood-wave movement through river reaches and reservoirs. Most of the flood-routing methods in HEC-1 are based on the continuity equation, and some relationship between flow and storage or stage. Routing proceeds on an independent reach basis from upstream to downstream; neither backwater effects nor discontinuities in the water surface are considered (HEC, 1987).

In this study, the kinematic wave routing method was used as channel routing technique. Channel routing was accomplished on reach-to-reach basis without considering a direct continuous water-surface profile. Input parameters were channel shape, length, slope, and roughness (Mannings "n") coefficient.

The flood hydrographs of each different land-use area within a subbasin were combined at the concentration point of each subbasin (appendix A). If there were for example three different land-use types within one subbasin, then for each of these land uses a hydrograph was generated. These hydrographs were simply combined at the subbasin concentration point. Any retention occurring within urban developments was subtracted (marked as "retention routing dummy basin" in HEC-1 input files) before combining the hydrographs of different land uses within the basin. Since most of the subbasin's were 640 acres or less, minimal differences were expected between routing the hydrographs within a subbasin or combining them at the subbasin concentration point. The flood hydrograph from each subbasin, however, was routed to the concentration point of the next subbasin downstream.

Flood hydrographs were routed over the section- or midsection-line roads in an urban area, and in swales adjacent to roads in an agricultural area. For flow over streets (asphalt) a Manning's n-value of 0.014 was used, and for flow in swales the n-value was 0.03. Routing lengths were measured from the USGS 7.5 min quadrangles. The slope of each routing channel was assumed to be the average slope of the subarea in which the channel was located.

The actual flow capacity in the roads and swales is probably smaller than required for the by HEC-1 generated peak flows in some areas, and more overland flow could occur.

The general direction of flow is towards the hydrologic boundary of each watershed. Data were not available to determine if somewhere along the hydrologic boundary, the floodwaters would cross this boundary. Floodwaters collected at the hydrologic boundary were thus routed to the south along this boundary to the intersection with the Superstition Freeway.

CHAPTER 3

HYDROLOGIC ANALYSIS

The peak runoff, time-of-peak (measured from the beginning of the storm) and 24-hour runoff volume for different concentration points in each watershed north of the Superstition Freeway are presented in table 3.1 The concentration points shown are the concentration points of each subarea within the watershed except for watershed 2, which had only one subarea. Peak runoff and runoff volume at a given concentration point include runoff from all upstream subareas. The runoff values given in table 3.1 for the intersection of the Superstition Freeway and Eastern Canal/Greenfield Road, Consolidated Canal, and SPRR are floodflows before routing through the detention basins or diverting any flow at those locations.

One important assumption in the modeling process was that all floodwaters collect at section- and midsection-line roads and finally at the hydrologic boundary. These floodwaters were routed southward along the hydrologic boundary. Neither actual conveyance capacity of existing swales nor elevated cross roads were verified with field survey data. In reality runoff is directed southward in the form of sheet flow by the canal or railroad levees until the conveyance capacity is exceeded or another barrier such as a cross road is encountered. Roads crossing the canals can be elevated for some distance above the surrounding ground. They form a barrier behind which ponding occurs. Greater retardation of flow may exist than was modeled. It is possible that ponding depths behind these barriers exceed the elevation of the canal embankment and some of the floodwaters may enter the canal. Also, the canals follow the contours and slopes are in reality flatter than used in the model. The USGS 7.5 min quadrangles, however, do not

TABLE 3.1
Area North of the Superstition Freeway, Peak Runoff, Time-of-Peak, and Runoff Volume for Different Concentration Points in Each Watershed (100-year 24-hour storm)
September 1990

Area Identification	Peak Runoff (ft^3/s)	Time-of-Peak (h)	24-Hour Runoff Volume (acre-feet)
1. Eastern Canal/Greenfield Road & Superstition Freeway Crossing (Watershed 1)			
Brown Rd & Eastern Canal	823	13.1	209
Main Rd & Eastern Canal	1468	13.3	394
Southern Ave & Eastern Canal	1963	13.5	549
Freeway & Eastern Canal	2129	13.6	605
2. Consolidated Canal & Superstition Freeway Crossing (Watershed 4)			
Tempe Cross Cut & Consolidated Canal	441	13.9	130
Main St & Consolidated Canal	1122	12.9	404
Southern Ave & Consolidated Canal	1884	13.0	581
Freeway & Consolidated Canal[1]	2456	13.8	700
3. Lindsay Road & Superstition Freeway Crossing (Watershed 3)[2]	660	14.0	44
4. SPRR & Superstition Freeway Crossing (Watershed 2)			
Main St. & SPRR	506	14.5	212
Southern Ave & SPRR	1293	14.9	568
Southern Ave & SPRR[4]	1195	15.3	338
Freeway & SPRR[3]	1209	15.8	449

[1] Includes overflow from watershed 1
[2] Includes overflow from water shed 1 and 4
[3] Includes overflow fromwatershed 1, 4 and 3
[4] After diversion of 237 acre-feet, total storage in Kingsborough, Emerald and Sherwood parks detention basins

Source: Franzoy Corey

give sufficient detail to determine a more exact slope. Hence, the resulting runoff peaks and volumes generated by the model are conservative.

Lag times for each land use were computed using the average slope in a subarea determined from the USGS 7.5 min quadrangles. Slopes within a subdivision or agricultural field may be flatter than used in the model. Consequently, actual lag times would be longer than used in the model. However, the accuracy of lag-time computations is consistent with available topographic data.

Methods and assumptions used for modeling the runoff for the area north of the Superstition Freeway are reasonable considering the purpose of the study: estimating runoff resulting from a 100-year 24-hour storm crossing the Superstition Freeway at certain points impacting the Gilbert-Chandler areas.

Watershed 1 (between the East Maricopa Floodway and the Eastern Canal, Mesa-Lehi grade break and the Superstition Freeway) consists of 46 percent agricultural land, mainly citrus groves. Berms enclose the groves to facilitate flood irrigation, permitting little or no runoff. The agricultural area is modeled using a curve number of 65. From the total rainfall amount of 3.8 inches, 0.97 inches was excess rainfall causing runoff. Watershed 1 is 49 percent urban development; 32 percent of the total watershed area developed after 1980. The peak runoff at the freeway crossing before routing through the detention basin was 2,129 cfs at 13.6 hours after the beginning of the storm, resulting in a 24-hour volume of 605 acre-feet.

Watershed 2 (between Consolidated Canal, Lindsay Road, Superstition Freeway, Center Street, and the Mesa-Lehi grade break) consists for 97 percent of urban development; 71

percent of the total watershed area was developed before 1972. Since Mesa's drainage policy did not exist before 1972, there was no on-site retention requirement for this urban development. However, there are three large detention basins located north of the Superstition Freeway and ADOT drainage channel between Consolidated Canal and Val Vista Road; Kingsborough Park (24th Street/freeway), Emerald Park (Harris/freeway) and Sherwood Park (Horne/freeway). The total storage capacity of these three basins is 237 acre-feet. Since these basins capture the floodflows from the area without retention, a volume equal to the storage volume of these reservoirs was diverted from the routed flood hydrograph at concentration point 5, the intersection of Southern Avenue and SPRR (see appendix A). The existing Main Street storm drain (running from Lindsay Road to Alma School Road and then turning north to discharge finally in the Salt River) was considered ineffective. All floodwaters were routed over streets described as rectangular channels, except along the SPRR, where the routing channel was defined as an earth channel.

The peak runoff at the intersection of the Superstition Freeway before diversions or detention routing was 1,209 cfs, occurring 15.8 hours after the beginning of the storm. The 24-hour runoff volume was 449 acre-feet. This includes overflow from watersheds 1, 4 and 3.

Watershed 3 (between Consolidated Canal, Lindsay Road and the Superstition Freeway) is about 0.12 square mile. The peak runoff from this watershed was 26 cfs, occurring 13.8 ours after the beginning of the storm, resulting in a 24-hour volume of 11 acre-feet. With overflow from watersheds 1 and 4, the peak runoff was 660 cfs, occurring 14.0 hours after the beginning of the storm. The 24-hour volume was 44 acre-feet. All the flow was routed to the intersection of SPRR and the Superstition Freeway in watershed 2 as overland flow.

Watershed 4 (between Eastern and Consolidated canals, the Superstition Freeway, and the Mesa-Lehi grade break) consist of 92 percent urban development; 51 percent of the total watershed area was developed after 1980, and 22 percent between 1972 and 1980. Flows from subbasins A through C (forming subarea 1) are concentrated at the intersection of Brown Road/Consolidated Canal/Tempe Cross-Cut Canal. A retention basin is located at this intersection. The flood hydrograph was not routed through this basin. Flows were routed southward along the Consolidated Canal. The peak runoff at the intersection of the Superstition Freeway and the Consolidated Canal before detention routing was 2,456 cfs, occurring 13.8 hours after the beginning of the storm. The 24-hour runoff volume was 700 acre-feet.

CHAPTER 4

FLOW AT SUPERSTITION FREEWAY CROSSINGS

Floodflows from a 100-year 24-hour storm from the area north of the Superstition Freeway at four freeway crossings were analyzed. These crossings were Eastern Canal/Greenfield Road, Consolidated Canal, Lindsay Road, and SPRR. The Eastern Canal and Greenfield Road crossings were combined into one crossing for this analysis.

4.1 METHOD OF ANALYSIS

In the following sections the method of analysis for each of the four crossings will be discussed separately starting with watershed 1, then 4, 3 and 2.

Area-capacity curves for the detention basins at the intersection of Eastern and Consolidated canals and the SPRR with the Superstition Freeway, and for Kingsborough, Emerald and Sherwood Park basins were obtained from the engineering department of the City of Mesa. Cross sections of the Eastern and Consolidated canals at the intersection of the freeway were surveyed as well as Lindsay Road and the road on the east side of SPRR crossing the freeway. Cross sections of the ADOT-drainage channel were obtained from ADOT construction plans of the crossings.

An attempt was made to model all the crossings with HEC-1 using diversion cards and transferring outflow hydrographs to external files, which were introduced in the next downstream watershed.

4.1.1 Eastern Canal/Greenfield Road Crossing (Watershed 1)

Flows from the subbasins (S and T) between the Superstition Freeway and Southern Avenue are collected in the ADOT-drainage channel at concentration point 20 (see appendix A). From there they are routed west until encountering a channel block (point 21CB). The flow then fills up the channel and spills into the detention basin north of the channel. A small portion of the flow (90 cfs) continues through a gate in the channel block and in a culvert under Eastern Canal.

The flows spilling into the detention basin from the ADOT-drainage channel and the flows from subbasins A through Q (area north of Southern Avenue) are combined. The combined hydrograph is then routed through the detention basin. The detention basin reaches maximum capacity at elevation 1284 (119 acre-feet). Flows exceeding this capacity spill into Eastern Canal. The rating curve for outflow from the detention basin was computed with the weir equation:

$$Q = c\ l\ (h)^{3/2}$$

where,

Q = flow over west maintenance road, cfs
c = discharge coefficient (in this study c=3)
l = weir length, ft
h = water depth over weir, ft

The length, l, was determined from aerial topographic mapping with 2-foot contours (McClain-Harbers, 1987) and field survey data.

The normal operating flow for Eastern Canal south of Southern Avenue is 250 cfs. This normal operating flow is combined with the overflow hydrograph from the detention

TABLE 4.1
Area North of the Superstition Freeway, Total Incoming Flows and Division of Outflows
(Runoff From 100-Year 24-Hour Storm)
September 1990

		WATERSHED IDENTIFICATION			
		(Eastern) 1	(Consolidated) 4	(Lindsay Rd) 3	(SPRR) 2
Total Incoming Flow	CFS	2129	2456	660	1209
	AF	605	700	44	449
Diversion ADOT-Drainage Channel (Culvert Under Hydrologic Barrier)	CFS	90	N/A	N/A	118
	AF	44			73
Diversion in Box Culvert	CFS	N/A	N/A	N/A	456
	AF				50
Detention Basin Storage Capacity	AF	119	170	N/A	101
Outflow From Detention Basin	CFS	2035	1859	N/A	534
	AF	443	530		218
Maximum Water-Surface Elevation	FT	1285.9	1250.8	N/A	1210.6
Spill Into Irrigation Canal	CFS	2035	1859	N/A	N/A
	AF	443	430		
Bankfull Capacity of Canal	CFS	1482	1750	N/A	N/A
Flow Crossing Superstition Freeway[1] (To Gilbert-Chandler Study Area)	CFS	1482	1750	N/A	539
	AF	914	1506		268
Flow Spilling in Next Watershed Downstream[2]	CFS	803	634	660	0
	AF	24	32	44	0

[1] Includes normal operating flow of irrigation canal, if applicable.
[2] Excluding ADOT-drainage channel flow.

Source: Franzoy Corey

basin. Flow exceeding the bankfull capacity (about 1,482 cfs) of Eastern Canal at the crossing is diverted from the canal. Included in the overflow from watershed 1 to watershed 4 is the flow exceeding the bankfull capacity of the Eastern Canal and the ADOT-drainage channel flow. The combined hydrograph of overflow continues as overland flow in watershed 4. The Eastern Canal flow (normal operating flow plus overflow from detention basin minus flow exceeding bankfull capacity of Eastern Canal) is routed south to the Gilbert-Chandler study area, south of the Superstition Freeway. The hydrograph is written to an external file (KO card) and later introduced (BI card) in subarea 3 of the Gilbert-Chandler Floodplain Delineation study. The peak flows and volumes for incoming, diverted and routed flows are presented in table 4.1.

4.1.2 Consolidated Canal Crossing (Watershed 4)

The flow overtopping the west bank of Eastern Canal at the Eastern Canal crossing with the Superstition Freeway is combined with the runoff from the subbasin adjacent to Eastern Canal in watershed 4. Flow from the subbasins (O and P) between the Superstition Freeway and Southern Avenue, and the overflow from watershed 1, is combined with all the runoff from the area north of Southern Avenue at concentration point 15 (see appendix A). This hydrograph is then routed through the detention basin at the Consolidated Canal and Superstition Freeway Crossing. This basin has a storage capacity of about 170 acre-feet. Spill out the basin into the Consolidated Canal starts at elevation 1248.7.

The normal operating flow for Consolidated Canal south of Southern Avenue is 525 cfs. This normal operating flow is combined with the overflow hydrograph from the detention basin. Flow exceeding the bankfull capacity (about 1,750 cfs) of Consolidated Canal at the crossing is diverted from the canal and continues as overland flow in watershed 3.

The continuing hydrograph in Consolidated Canal is saved on an external file (KO card) and later introduced (BI card) in subarea 2 of the Gilbert-Chandler Floodplain Delineation study. The peak flows and volumes for incoming, diverted and routed flows are presented in table 4.1.

4.1.3 Lindsay Road Crossing (Watershed 3)

The flow overtopping the west bank of Consolidated Canal at the Consolidated Canal crossing with the Superstition Freeway is combined with the runoff from the subbasin adjacent to Consolidated Canal in watershed 3. All floodflows to cross Lindsay Road. Field survey data showed that Lindsay Road is about 1-1/4 ft higher under the freeway and slopes down towards the north. Hydrographs are saved on an external file and introduced in watershed 2.

The peak flow and volume for incoming flow at the Lindsay Road crossing are presented in table 4.1.

4.1.4 SPRR Crossing (Watershed 2)

The flow overtopping Lindsay Road at the Lindsay Road crossing with the Superstition Freeway is combined with the runoff from the subbasin adjacent to Lindsay Road in watershed 2. Flows from the subbasins between the Superstition Freeway and Southern Avenue and the overflow from watershed 3 is collected in the ADOT-drainage channel at concentration point 9 (see appendix A).

The ADOT-drainage channel crosses the SPRR with a 54-inch diameter concrete culvert. Assuming inlet control, the capacity of this culvert was calculated using the nomographs of the Bureau of Public Works (U.S. Department of Transportation, 1985). Flow in the ADOT-drainage channel crossing the SPRR is diverted from the inflow hydrograph and saved in an external file.

The two detention basins at the SPRR and Superstition Freeway crossing (Heritage Park basin north and Center Street basin south of the freeway) are connected by a 10 ft x 10 ft box culvert. Weirs in the ADOT-drainage channel connecting the north with south basin via the box culvert are set at the same elevation. The width of each weir is different. The rating curves for flow of the weirs were computed with the weir equation (see section 4.1.1). Flow through the box culvert into the south basin is diverted from the inflowing hydrograph in the ADOT-drainage channel, and saved on an external file for later introduction into subarea 1 south of the Superstition Freeway.

The remaining flow into the north basin is combined with the runoff from the area north of Southern Avenue (subbasin A through E). The combined hydrograph is then routed through the north detention basin. The detention basin reaches capacity at elevation 1,208 (101 acre-feet). Flows exceeding the capacity of the basin will flow through the box culvert to the south basin. This flow is saved on an external file for later introduction into subarea 1, south of the Superstition Freeway.

The peak flows and volumes for incoming, diverted and routed flows, are presented in table 4.1.

4.2 RESULTS FOR EACH WATERSHED

In watershed 1, the detention basin of the Superstition Freeway drainage system is located west of Greenfield Road and east of Eastern Canal. The maximum storage is about 119 acre-feet. The maximum stage in the detention basin at peak outflow (2,035 cfs, 443 acre-feet) from the basin was 1,285.9 ft (table 4.1). Outflow into Eastern Canal would occur from the southwest corner of the detention basin (elevation 1,284.0). Greenfield Road and the north side of the basin are higher than the maximum stage at peak flow in the detention basin. No flow is expected in that direction. Eastern Canal will carry 1,482 cfs at bankfull capacity and 803 cfs (24 acre-feet) will flow over the west bank into a low area bounded by a block fence and the ADOT drainage channel in the north, and the west on-ramp to the Superstition Freeway in the south. The overflow over the west bank of Eastern Canal and the ADOT-drainage channel flow were combined with the runoff from watershed 4. Eastern Canal will cross the Superstition Freeway at bankfull capacity. This flow will be introduced in subarea 3 of the Gilbert-Chandler Floodplain Delineation Study.

The detention basin in watershed 4 at the northeast corner of the intersection of the Superstition Freeway and Consolidated Canal has a storage capacity of about 170 acre-feet. The area is very flat, and limited topographic data was available for analysis. The maximum stage in the detention basin at peak outflow (1,859 cfs, 530 acre-feet) was 1,250.8 ft. It was assumed that all the flow would occur over the east bank of Consolidated Canal. Consolidated Canal will carry 1,750 cfs at bankfull capacity. Bankfull capacity of Consolidated Canal is exceeded by 634 cfs (32 acre-feet). This flow spills out over the west bank of Consolidated Canal and continues as overland flow in watershed 3.

Consolidated Canal will cross the Superstition Freeway at bankfull capacity. This flow will be introduced in subarea of 2 of the Gilbert-Chandler Floodplain Delineation study.

Watershed 3 is bounded on the west by Lindsay Road. Lindsay Road is considered an effective hydrologic boundary to the south. The road is elevated under the freeway and slopes towards the north (from field survey data). Floodwaters crossing the west bank of the Consolidated Canal were combined with the runoff from watershed 3. The combined peak flow was 660 cfs, resulting in a volume of 44 acre-feet. These flows to cross Lindsay Road into watershed 2.

The Mesa drainage system at the SPRR crossing consists of two detention basins, one north of the freeway, and one south of the freeway connected by a 10 ft x 10 ft concrete box culvert. The storage capacity of the basin north of the Superstition Freeway (Heritage Park Basin) is about 101 acre-feet. The maximum stage in the detention basin at peak outflow (534 cfs, 218 acre-feet) from the basin was 1210.6 ft (table 4.1). Outflow from the basin occurs through the 10 ft x 10 ft box culvert. Except from the flow in the ADOT-drainage channel crossing the SPRR, no other flow will cross the SPRR which is 2-3-ft above the surrounding area. The floodflows of subarea 1 of the Gilbert-Chandler Floodplain Delineation study will be combined with the flow through the 10 ft x 10 ft box culvert connecting the two basins (539 cfs, 268 acre-feet).

In conclusion, floodwaters from watershed 1 will cross the Superstition Freeway in Eastern Canal; from watershed 4 in Consolidated Canal; and, from watershed 2 through the box culvert connecting the north and south detention basins. Eastern and Consolidated canals were assumed to flow at bankfull capacity of 1,482 and 1,750 cfs, respectively.

Since the canals reduce in capacity downstream of the Superstition Freeway crossings, it has to be determined at which point south of the freeway this conveyance capacity will be exceeded, and the floodflow will break out the canals. These flows will be added at this location to the runoff from the particular subarea where outbreak from the canals occurs. Floodflows exceeding the capacity of the north detention basin at the SPRR crossing will be combined with the runoff from subarea 1 in the Gilbert-Chandler Floodplain Delineation Study area.

CHAPTER 5

SUMMARY

The runoff resulting from the 100-year 24-hour storm reaching four crossings (Eastern Canal/Greenfield Road, Consolidated Canal, Lindsay Road and Southern Pacific Railroad) at the Superstition Freeway (as defined in the scope of work of "Aerial Mapping and Floodplain Delineation of the Gilbert-Chandler Areas, Arizona"), was estimated using the HEC-1 flood hydrograph model. The objective of the estimation of the runoff peaks and volumes was to determine if any floodwaters would cross the Superstition Freeway at mentioned crossings, and would have to be accounted for in the floodplain delineation study of the Gilbert-Chandler areas.

The contributing watersheds between the hydrologic boundaries (RWCD, Eastern and Consolidated canals, and SPRR) were divided in subareas. The subareas were further divided in subbasins. Subarea and subbasin parameters were determined upon USGS 7.5 min quadrangle maps. The rainfall time distribution for the type II 100-year 24-hour storm (3.8 inches) was used. Curve numbers were determined from the TR55 (1986) publication. Lag time was based upon average natural ground slope of the subareas and the overland flow velocity for paved or unpaved areas.

The SCS unit hydrograph method was used to estimate runoff. Flood hydrographs were routed between concentration points using the kinematic wave channel routing method. Runoff was generally directed towards the hydrologic boundaries (southwest) and along the hydrologic boundaries southward. Physical boundaries (like elevated roads) in the path of the floodwaters going south were not considered in the model as barriers causing retardation.

Peak runoff, runoff volume and time-of-peak (from beginning of storm) were presented for each crossing. The floodwaters were then routed through the detention basins which are part of the Mesa storm drainage system.

This study showed that flow will cross the Superstition Freeway in Eastern Canal. The canal will flow bankfull at 1,482 cfs. Floodwaters (803 cfs) overflowing the west canal bank, and the ADOT-drainage channel flow (90 cfs) were combined with the runoff from watershed 4. Floodwaters will also cross the freeway in Consolidated Canal. The bankfull capacity is 1,750 cfs. Floodwaters (634 cfs) exceeding the capacity of the canal and overflowing the west canal bank maintenance road were combined with runoff from watershed 3. Field survey showed that Lindsay Road is effective hydrologic boundary blocking flow to the south across the freeway. All floodwaters crossed Lindsay Road, and were combined with the runoff from watershed 2. The north detention basin (Heritage Park) at the SPRR will spill through the 10 x 10 ft box culvert to the south (Center Street) detention basin (534 cfs). Before spilling from the north basin, flow will cross the freeway through the box culvert connecting the north and south basins (456 cfs). Both these flows will be combined with the runoff from subarea 1 south of the Superstition Freeway in the Gilbert-Chandler Floodplain Delineation study. North of the freeway, flows (112 cfs) continue under the SPRR in the ADOT-drainage channel.

For the floodplain delineation study of the Gilbert-Chandler areas it has to be determined where the runoff in Eastern and Consolidated canals coming from the area north of the Superstition Freeway will break out the banks. These flows will be added to the runoff of the subareas where the breakouts occur. The runoff flowing out of the detention basin south of the freeway at the SPRR crossing will be added to the runoff from the subarea south of the Superstition Freeway in which this detention basin is located.

BIBLIOGRAPHY

Arizona Highway Department, Bridge Division (Rev. 1975). Hydrologic Design for Highway Drainage in Arizona.

Hromadka, II, T.V. and J.J. DeVries (1988) Kinematic Wave Routing and Computational Error. Journal of Hydraulic Engineering, ASCE 114(2): 207-217.

Landis Aerial Surveys. 1986. Aerial Survey of Phoenix Area Photodata, December 1985. Phoenix, Arizona.

McClain Harbers Co., Inc. (1987). Photomosaic Gilbert-Chandler Floodplain Delineation Scale 1"=1000'. Tucson, Arizona.

Storm Drainage Policy (Rev. 1983) City of Mesa, Arizona.

U.S. Department of Agriculture, Soil Conservation Service (USDA/SCS) (1972). National Engineering Handbook, Section 4, Washington, D.C.

__________ (1974). Soil Survey Eastern Maricopa and Northern Pinal Counties Area, Arizona. Washington, D.C.

__________. (1986). Urban Hydrology for Small Watersheds. Tech. Release 55, Washington, D.C.

U.S. Army Corps of Engineers (1977). Gila Floodway Maricopa and Pinal Counties, Arizona. Summary Report for Flood Control.

__________, Hydrologic Engineering Center (HEC) (Rev. 1987). HEC-1 Flood Hydrograph Package, Program Users Manual. California.

__________. (1982). HEC-2 Water Surface Profiles, Users Manual. California.

U.S. Department of Transportation, Federal Highway Administration (1985). Hydraulic Design of Highway Culverts. Hydraulic Design Series No. 5. NTIS. Springfield, Virginia.

Yost and Gardner Engineers (1973) Stormwater Drainage for the City of Mesa. Project No. 71-6 City of Mesa, Arizona.