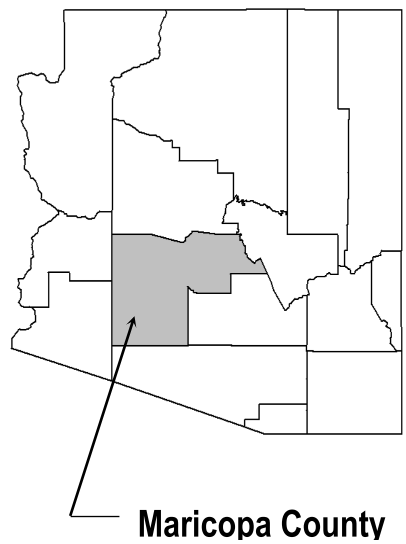
FLOOD INSURANCE STUDY

VOLUME 3 OF 31

MARICOPA COUNTY, ARIZONA
AND INCORPORATED AREAS



COMMUNITY NAME	COMMUNITY NUMBER
AVONDALE, CITY OF	040038
BUCKEYE, TOWN OF	040039
CAREFREE, TOWN OF	040126
CAVE CREEK, TOWN OF	040129
CHANDLER, CITY OF	040040
EL MIRAGE, CITY OF	040041
FOUNTAIN HILLS, TOWN OF	040135
GILA BEND, TOWN OF	040043
GILBERT, TOWN OF	040044
GLENDALE, CITY OF	040045
GOODYEAR, CITY OF	040046
GUADALUPE, TOWN OF	040111
LITCHFIELD PARK, CITY OF	040128
MARICOPA COUNTY (UNINCORPORATED AREAS)	040037
MESA, CITY OF	040048
PARADISE VALLEY, TOWN OF	040049
PEORIA, CITY OF	040050
PHOENIX, CITY OF	040051
QUEEN CREEK, TOWN OF	040132
SCOTTSDALE, CITY OF	045012
SURPRISE, CITY OF	040053
TEMPE, CITY OF	040054
TOLLESON, CITY OF	040055
WICKENBURG, TOWN OF	040056
YOUNGTOWN, TOWN OF	040057

NOTICE
This Preliminary FIS Report includes only revised flood profiles and Floodway Data Tables. See Notice to Flood Insurance Study Users page for additional details.

PRELIMINARY

FEBRUARY 26, 2016

Federal Emergency Management Agency

FLOOD INSURANCE STUDY NUMBER
04013CV003D

NOTICE TO
FLOOD INSURANCE STUDY USERS

Communities participating in the National Flood Insurance Program have established repositories of flood hazard data for floodplain management and flood insurance purposes. This Flood Insurance Study (FIS) report may not contain all data available within the Community Map Repository. Please contact the Community Map Repository for any additional data.

The Federal Emergency Management Agency (FEMA) may revise and republish part or all of this FIS report at any time. In addition, FEMA may revise part of this FIS report by Letter of Map Revision process, which does not involve republication or redistribution of the FIS report. Therefore, users should consult community officials and check the Community Map Repository to obtain the most current FIS report components.

Users should refer to Section 10.0, Revisions Description, for further information. Section 10.0 is intended to present the most up-to-date information for specific portions of this FIS report. Therefore, users of this report should be aware that the information presented in Section 10.0 supersedes information in Sections 1.0 through 9.0 of the FIS report.

Initial Countywide FIS Effective Date: April 15, 1988

Revised Countywide Dates: September 29, 1989
September 4, 1991
December 3, 1993
September 30, 1995
July 19, 2001
September 30, 2005
October 16, 2013
November 4, 2015

This preliminary FIS report does not include unrevised Floodway Data Tables or unrevised Flood Profiles, and therefore does not include any additional report volumes. These Floodway Data Tables and Flood Profiles will appear in Volumes 3-7 and 9-31 of the final FIS report.

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PUBLISHED SEPARATELY

3.2 Hydraulic Analyses

Analyses of the hydraulic characteristics of flooding from the sources studied were carried out to provide estimates of the elevations of floods of the selected recurrence intervals. Users should be aware that flood elevations shown on the Flood Insurance Rate Map (FIRM) represent rounded whole-foot elevations and may not exactly reflect the elevations shown on the Flood Profiles or in the Floodway Data tables in the Flood Insurance Study (FIS) report. Flood elevations shown on the FIRM are primarily intended for flood insurance rating purposes. For construction and/or floodplain management purposes, users are cautioned to use the flood elevation data presented in this FIS in conjunction with the data shown on the FIRM.

For areas of riverine flooding studied by detailed methods, water-surface elevations for floods of the selected recurrence intervals were computed using the U.S. Army Corps of Engineers (USACE) HEC-2 computer program (U.S. Department of the Army, 1973).

The cross section data for the Agua Fria River were taken from several sources of mapping. A 1981 USACE topographic map for the New River (U.S. Department of the Army, 1981) was used for the river section from the confluence with the Gila River to the confluence with the New River from the New River to Northern Avenue, 1982 City of Glendale mapping was used (City of Glendale, 1982). From Northern Avenue to Grand Avenue and from Beardsley Road to Jomax Road, 1983 Maricopa County maps were used (Maricopa County, 1983). The topographic maps for the reach between Grand Avenue and Bell Road (American Engineering Company, 1982) were furnished by American Engineering Company for the reach between Bell and Beardsley Roads, maps were provided by Cella, Barr, Evans and Associates (Cella Barr Evens & Associates, 1982).

Cross sections for the Gila River were digitized from 1983 topographic maps or taken from as-built data for the Bullard Avenue Bridge.

Cross sections for the Salt River between Central Avenue and 115th Avenue were based on digitized data from topographic mapping. From Central Avenue to Country Club Road in the City of Mesa, cross sections were also taken from topographic mapping (Arizona Department of Transportation, 1982; City of Phoenix, 1983).

For study purposes, Skunk Creek was divided into two sections. Lower Skunk Creek lies between Adobe Dam outlet channel and the sections were also taken from topographic mapping (Arizona Department of Transportation, 1982) Bell Road Bridge. Upper Skunk Creek is from the Central Arizona with a contour interval of 2 feet.

Project channel to Adobe Dam. Cross sections for both reaches were generated using 1974 Maricopa County topographic maps at a scale of 1:2,400, with a contour interval of 2 feet. These maps were supplemented by additional mapping from the City of Phoenix and the USACE at scales of 1:1,200 and 1:2,400, respectively, both cross sections for the Hassayampa River (below Carefree Highway) were field surveyed.

Cross-section data for the following were developed from topographic maps (Harris-Toups Associates, 1976): Skunk Creek above Carefree Highway; Cave Creek above Cave

Creek Dam; Andora Hills, Galloway, Rowe, Grapevine, Ocotillo, Willow Springs, Powder House, Mockingbird, and Little San Domingo Washes; Whitman Drainage; Aguila Farm Channel; Grass, Sand Tank, and Bender Washes; Rodeo Wash and its tributary; Airport, and Scott Avenue Washes; Lower El Mirage Wash and its tributary; Atchison, Topeka & Santa Fe Railway Channel at El Mirage; the Atchison, Topeka & Santa Fe Railway at Peoria; and the Southern Pacific Railroad and its spurs.

Cross-section data for East Branch Scatter Wash and Echo Canyon Washes were developed from topographic maps provided by the City of Phoenix (City of Phoenix, 1967).

Cross-section data for Cave Creek below Arizona Canal and for East Fork of Cave Creek were developed from aerial photographs flown in March 1980 (Aerial Mapping of Phoenix, Arizona, 1980). Cross-section data for Cave Creek between Arizona Canal and Cave Creek Dam were developed from aerial photographs flown in March 1978 (Aerial Mapping of Phoenix, Arizona, 1978).

Cross-section data for the Sols Wash backwater analyses were obtained from topographic maps, at a scale of 1:200, with a contour interval of 2 feet, prepared specifically for this project by Cooper Aerial Survey in March 1986 (Cooper Aerial Survey, 1986). Culvert and bridge data were obtained from the topographic maps and were field checked to verify structural geometry.

Cross-section data for Casandro, South Branch Casandro, Flying E, and Hospital Washes were taken from a USACE Flood Plain Information report for Wickenburg (U.S. Department of the Army, 1965) and from topographic maps (U.S. Department of the Army, 1976).

Cross-section data for Martinez Wash were digitized from topographic maps (U.S. Department of the Army, 1968).

Cross sections were located at close intervals above and below bridges in order to compute the significant backwater effects of these structures. All bridges and culverts were investigated to obtain elevation data and structural geometry.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1). For stream segments for which a floodway was computed (Section 4.2), selected cross section locations are also shown on the Flood Boundary and Floodway Map (Exhibit 2).

Starting water-surface elevations for all riverine flooding sources, except as noted below, were developed using the slope/area method.

The starting water-surface elevations for the New River were developed through the use of 1985 topographic mapping in the area of its confluence with Skunk Creek. A significant feature of the New River floodplain is the channelization in the vicinity of its confluence with Skunk Creek. This channelization has occurred from approximately 1,500 feet downstream of the Thunderbird Road Bridge upstream to the Greenway Road.

In addition, in the left over bank area above Union Hills Drive, a new wastewater treatment plant with improved channel banks is reflected in the hydraulic model.

For the upper reaches of Skunk Creek, the starting water-surface elevations were computed from the reservoir spillway elevation of 1,377 feet. For the lower reach, normal-depth and New River backwater computations were used.

Hydraulic roughness coefficients (Manning's "n") were selected on the basis of field inspection and engineering judgment. Table 5, "Range of Hydraulic Roughness Coefficients (Manning's "n")" gives the range of Manning's "n" values for each flooding source studied by detailed methods.

Table 5. Range of Hydraulic Roughness Coefficients (Manning's "n")

Flooding Source	Channel	Overbanks
191st Avenue Wash	0.012 - 0.030	0.040 - 0.070
Agua Fria River	0.022 - 0.059	0.032 - 0.070
Aguila Farm Channel	0.030	0.040 - 0.050
Airport Wash	0.025	0.035
Andora Hills Wash	0.020 - 0.045	0.020- 0.052
Apache Wash	0.045 - 0.060	0.070
Apache Wash West Fork	0.045 - 0.060	0.070
Atchison, Topeka & Santa Fe Railway Channel	0.032 - 0.037	0.032 - 0.047
Atchison, Topeka & Santa Fe (AT&SF) Railway Channel	0.035 - 0.045	0.030 - 0.080
Atchison, Topeka & Santa Fe Railway Ponding	0.035 - 0.040	0.035 - 0.040
Beardsley Canal Wash	0.024 - 0.035	0.024 - 0.070
Bedrock Wash	0.045 - 0.050	0.035 - 0.070
Bender and Sand Tank Washes	0.025	0.035
Bender Wash	0.025 – 0.080	0.030 - 0.035
Bullard Wash	0.013 - 0.070	0.030 - 0.070
Bulldozer Wash	0.035 - 0.050	0.040 - 0.070
Casandro Wash	0.030 - 0.060	0.040 - 0.060
Casandro Wash South Branch	0.030 - 0.060	0.040 - 0.060

Table 5. Range of Hydraulic Roughness Coefficients (Manning's "n") (Continued)

Flooding Source	Channel	Overbanks
Cave Creek	0.015 - 0.065	0.035 - 0.065
Cemetery Wash	0.035 - 0.100	0.040 - 0.100
Centennial Wash	0.030 - 0.070	0.030 - 0.200
Centennial Wash	0.040	0.040
Cholla Wash	0.035 - 0.070	0.030 - 0.070
Circle City Area Washes	0.030 - 0.080	0.030 - 0.080
Cline Creek and Tributaries	0.045 - 0.075	0.045 - 0.080
Consolidated Canal, Ponding	0.025 - 0.075	0.025 - 0.075
Cottonwood Creek	0.030 - 0.060	0.050 - 0.080
Cottonwood Creek Tributary 1	0.045 - 0.050	0.060 - 0.070
Cottonwood Creek Tributary 2	0.050	0.060 - 0.070
Coyote Pass Wash	0.035 - 0.06	0.025 - 0.5
Dale Creek	0.025 - 0.035	0.025 - 0.050
Delaney Wash	0.046 - 0.065	0.02 - 0.065
Delaney Wash North Split	0.046 - 0.065	0.042 - 0.065
Delaney Wash South Split	0.042 - 0.065	0.02 - 0.065
Desert Hills Wash	0.012 - 0.055	0.050 - 0.114
Desert Hills Wash - West Branch	0.050 - 0.060	0.052 - 0.065
Desert Lake Wash	0.050	0.060 - 0.065
Dickey Wash	0.05 - 0.06	0.045 - 0.06
Diversion Dike Wash	0.035	0.035 - 0.070
East Fork of Cave Creek	0.015 - 0.035	0.035 - 0.045
East Garambullo Wash	0.024 - 0.055	0.036 - 0.060
Eastern Canal, Ponding	0.032 - 0.075	0.032 - 0.075
Echo Canyon Wash	0.018 - 0.025	0.012 - 0.035
Fleming Springs Wash	0.038 - 0.060	0.055 - 0.060
Flying "E" Wash	0.030 - 0.060	0.040 - 0.060

Table 5. Range of Hydraulic Roughness Coefficients (Manning's "n") (Continued)

Flooding Source	Channel	Overbanks
Four Mile Wash	0.02 - 0.075	0.02 - 0.075
Four Mile W1	0.037	0.037
Four Mile W2	0.035 - 0.042	0.035 - 0.042
Galloway Wash	0.032 - 0.045	0.016 - 0.045
Galloway Wash	0.032 - 0.045	0.016 - 0.045
Galloway Wash-North Tributary 1	0.025 - 0.040	0.045
Gavilan Peak Wash	0.025 - 0.06	0.025 - 0.5
Gila Bend Canal	0.045	0.050
Gila River	0.030 - 0.120	0.035 - 1.000
Grapevine Wash	0.020 - 0.052	0.020 - 0.046
Grass Wash	0.025 - 0.040	0.025 - 0.045
Hassayampa River	0.030 - 0.050	0.030 - 1.000
Hospital Wash	0.030 - 0.060	0.040 - 0.060
Interstate 10	0.035	0.045 - 0.050
Jackrabbit Trail	0.012 - 0.030	0.012 - 0.060
Jackrabbit Wash	0.030 - 0.035	0.035 - 0.040
Kelley Road Wash	0.035 - 0.06	0.025 - 0.5
Lazy G Wash	0.035 - 0.055	0.055
Little San Domingo Wash	0.030	0.040
Lower El Mirage Wash	0.044	0.044
Lower El Mirage Wash	0.030 - 0.045	0.035 - 0.100
Lower El Mirage Wash Tributary	0.040 - 0.045	0.070 - 0.100
Lower El Mirage Wash Tributary	0.044	0.044
Luke Wash	0.045 - 0.075	0.045 - 0.075
Luke Wash (For concrete box culverts)	0.017	N/A
Luke Wash East Main Split	0.045	0.05
Luke Wash - East Main Tributary	0.045 - 0.06	0.050 - 0.120
Luke Wash - East Sub-Tributary	0.045 - 0.055	0.045 - 0.06

Table 5. Range of Hydraulic Roughness Coefficients (Manning's "n") (Continued)

Flooding Source	Channel	Overbanks
Luke Wash - Minor Tributary	0.045	0.05
Martinez Wash	0.025 - 0.060	0.060 - 0.100
McMicken Dam Outlet Wash	0.020 - 0.050	0.035 - 0.080
Mesquite Tank Wash	0.060	0.070
Mockingbird Wash	0.030 - 0.037	0.035 - 0.042
Morgan City Wash	0.035 - 0.100	0.055 - 0.100
New River	0.030 - 0.035	0.030 - 0.060
Centennial Wash North Branch	0.040	0.040
North Fork Bedrock Wash	0.035 - 0.045	0.070
North Fork Cholla Wash	0.070	0.070
North Fork White Granite Wash	0.035	0.070
Ocotillo Wash	0.020 - 0.045	0.020 - 0.052
Ocotillo Wash Tributary 1	0.035 - 0.040	0.045
Ocotillo Wash Tributary 1A	0.032 - 0.035	0.040 - 0.045
Ocotillo Wash Tributary 2	0.035 - 0.045	0.040 - 0.050
Ocotillo Wash Tributary 3	0.045 - 0.055	0.055
Ocotillo Wash Tributary 4	0.025 - 0.045	0.045 - 0.050
Osborn Road Wash	0.030 - 0.035	0.050 - 0.070
Paradise Wash	0.013 - 0.055	0.050 - 0.070
Paradise Wash - West Branch	0.050 - 0.055	0.053 - 0.065
Perryville Road Wash	0.022 - 0.045	0.035 - 0.080
Phillips Wash	0.04 - 0.06	0.045 - 0.06
Photo View Wash	0.038 - 0.55	0.025 - 0.06
Photo View Wash Breakout 1	0.038 - 0.055	0.055
Photo View Wash Breakout 2	0.055	0.055 - 0.5

Table 5. Range of Hydraulic Roughness Coefficients (Manning's "n") (Continued)

Flooding Source	Channel	Overbanks
Photo View Wash Breakout 3	0.055	0.055 - 0.5
Powder House Wash	0.030 - 0.060	0.040 - 0.060
Powerline Wash	0.040 - 0.045	0.050 - 0.055
Powerline Wash	0.040 - 0.041	0.043 - 0.055
Rainbow Wash	0.016 - 0.047	0.030 - 0.150
Rainbow Wash Tributary	0.013 - 0.040	0.040 - 0.050
Ranieri Wash	0.050	0.065
Rio Verde Wash 7	0.035 - 0.065	0.045 - 0.065
Rio Verde Wash 10	0.035 - 0.055	0.045 - 0.055
Rio Verde Wash 10 Tributary 1	0.035 - 0.06	0.035 - 0.06
Rio Verde Wash 10 Tributary 2	0.045 - 0.055	0.045 - 0.055
Rio Verde Wash 10 Tributary 2 Split 1	0.045 - 0.055	0.045 - 0.055
Rio Verde Wash 10 Tributary 3	0.045 - 0.055	0.045 - 0.055
Rio Verde Wash 10 Tributary 4	0.045 - 0.055	0.045 - 0.055
Rio Verde Wash A	0.04 - 0.05	0.04 - 0.05
Rio Verde Wash A Tank Spillway	0.04 - 0.05	0.04 - 0.05
Rio Verde Wash A Tributary 1	0.05 - 0.075	0.05 - 0.075
Rio Verde Wash A Tributary 2	0.04 - 0.065	0.04 - 0.065
Rio Verde Wash I	0.05 - 0.055	0.05 - 0.055
Rio Verde Wash I Tributary 3	0.05 - 0.055	0.05 - 0.055
Rio Verde Wash K	0.04 - 0.06	0.04 - 0.06
Rio Verde Wash K Split4	0.04 - 0.06	0.04 - 0.06
Rio Verde Wash K Tributary 6	0.04 - 0.055	0.04 - 0.055
Rio Verde Wash K Tributary 6D	0.05 - 0.055	0.05 - 0.055
Rio Verde Wash K Tributary 6D1	0.045 - 0.06	0.055 - 0.06
Rio Verde Wash K Tributary 6D Split 1	0.05 - 0.055	0.055
Rio Verde Wash K Tributary 11	0.045	0.045

Table 5. Range of Hydraulic Roughness Coefficients (Manning's "n") (Continued)

Flooding Source	Channel	Overbanks
Rio Verde Wash K Tributary 11A	0.045	0.045
Rio Verde Wash K Tributary 11B	0.04 - 0.045	0.045
Rio Verde Wash K Tributary 12	0.045 - 0.05	0.045 - 0.05
Rio Verde Wash K Tributary 13	0.04 - 0.06	0.04 - 0.06
Rio Verde Wash P	0.04 - 0.055	0.045 - 0.055
Rio Verde Wash P Tributary 1	0.05 - 0.055	0.05 - 0.055
Rio Verde Wash P Tributary 2	0.05 - 0.055	0.05 - 0.055
River Creek	0.038 - 0.055	0.05 - 0.055
Rodeo Wash	0.025	0.035
Rodeo Wash Tributary	0.025	0.035
Rodger Creek	0.045 - 0.080	0.055 - 0.080
Rough Rider Wash	0.03 - 0.055	0.025 - 0.5
Rowe Wash	0.020 - 0.045	0.020 - 0.052
Rowe Wash Tributary 1	0.045	0.045 - 0.055
Rowe Wash Tributary 2	0.045	0.050 - 0.055
Salt River	0.030 - 0.035	0.040 - 0.050
Sand Tank Wash	0.025 - 0.080	0.035 - 0.060
Scatter Wash, North Branch	0.020 - 0.050	0.070 - 0.150
Scatter Wash, South Branch	0.035	0.045
Scott Avenue Wash	0.025 - 0.080	0.035
Scott Avenue Wash	0.035 - 0.080	0.035 - 0.040
Sharman Wash	0.025 - 0.06	0.025 - 0.06
Skunk Creek	0.020 - 0.050	0.045 - 0.050
Soda Springs Wash	0.025 - 0.06	0.025 - 0.5
Sols Wash	0.035 - 0.065	0.025 - 0.100
South Branch of Tank Wash	0.040 - 0.050	0.050 - 0.055
Southern Pacific Railroad	0.014 - 0.050	0.014 - 0.100

Table 5. Range of Hydraulic Roughness Coefficients (Manning's "n") (Continued)

Flooding Source	Channel	Overbanks
Southern Pacific Railroad & Southern Pacific Spur,	0.025 - 0.075	0.025 - 0.075
Stagecoach Pass Wash Overflow	0.030	0.040
Star Wash	0.030 - 0.035	0.035 - 0.040
Star Wash	0.036 - 0.044	0.043 - 0.045
Sweat Canyon Wash	0.035	0.055
Table Mountain Wash	0.03 - 0.06	0.025 - 0.5
Table Mountain Wash Tributary 6	0.038 - 0.055	0.055
Tank Wash	0.040 - 0.041	0.043 - 0.055
Tank Wash	0.040 - 0.046	0.050 - 0.055
Tractor Wash	0.030 - 0.035	0.035 - 0.075
Tributary A	0.042 - 0.044	0.042
Tributary B	0.042	0.045
Tributary C	0.040	0.045
Tributary D	0.038 - 0.040	0.044 - 0.045
Tributary E	0.038 - 0.040	0.044
Trilby Wash	0.040 - 0.100	0.050 - 0.100
Tuthill Dike Wash	0.016 - 0.030	0.016 - 0.050
Twin Buttes Wash	0.024 - 0.055	0.036 - 0.060
Twin Peaks Lane Wash	0.025 - 0.055	0.025 - 0.5
Union Pacific Railroad	0.035	.035 - 0.05
Unnamed Tributary of Jackrabbit Wash	0.030 - 0.035	0.035 - 0.040
Unnamed Wash No. 1	0.025 - 0.080	0.030 - 0.035
Unnamed Wash No. 2	0.025 - 0.080	0.030 - 0.070
Wagner Wash	0.040 - 0.105	0.065 - 0.100
Wash 2 West Tributary 1	0.033 - 0.082	0.033 - 0.082
Wash T1N-R5W-S04	0.06	0.055
Wash T1N-R5W-S04Split	0.06	0.055

Table 5. Range of Hydraulic Roughness Coefficients (Manning's "n") (Continued)

Flooding Source	Channel	Overbanks
Wash T1N-R5W-S10	0.045	0.055
Wash T1N-R5W-S15	0.05	0.065
Wash T1N-R5W-S18	0.045	0.06
Wash T1N-R5W-S22	0.05-0.065	0.065
Wash T1N-R5W-S28E	0.045	0.05
Wash T1N-R5W-S32	0.045	0.05
Wash T1N-R5W-S33E	0.045	0.05
Wash T1N-R5W-S33N	0.055	0.06
Wash T1N-R5W-S33W	0.045	0.05
Wash T1N-R6W-S1	0.04 - 0.06	0.04 - 0.06
Wash T1N-R6W-S11	0.06	0.055
Wash T1N-R6W-S12	0.06	0.055
Wash T1N-R6W-S17	0.02 - 0.065	0.02 - 0.065
Wash T1N-R6W-S18	0.02 - 0.065	0.02 - 0.065
Wash T1S-R2W-S32A	0.039	0.05
Wash T1S-R5W-S17	0.045 - 0.065	0.05 - 0.06
Wash T1S-R5W-S22N	0.075	0.05 - 0.075
Wash T1S-R5W-S22S	0.075	0.05 - 0.075
Wash T1S-R5W-S29	0.04 - 0.08	0.05 - 0.075
Wash T1S-R5W-S29W	0.04	0.05
Wash T1S-R6W-S05S	0.032 - 0.065	0.032 - 0.065
Wash T1S-R6W-S08	0.035 - 0.065	0.035 - 0.065
Wash T1S-R6W-S27	0.02 - 0.06	0.02 - 0.06
Wash T2N-R5W-S27N	0.06	0.05
Wash T2N-R5W-S27S	0.065	0.05
Wash T2N-R5W-S28	0.04	0.06

Table 5. Range of Hydraulic Roughness Coefficients (Manning's "n") (Continued)

Flooding Source	Channel	Overbanks
Wash T2N-R5W-S32	0.05	0.06
Wash T2N-R5W-S33E	0.04 - 0.06	0.06
Wash T2N-R5W-S33W	0.06	0.055
Wash T2N-R6W-S02	0.02 - 0.066	0.02 - 0.066
Wash T2N-R6W-S05E	0.02 - 0.075	0.02 - 0.075
Wash T2N-R6W-S05N	0.044 - 0.072	0.044 - 0.072
Wash T2N-R6W-S05S	0.032 - 0.065	0.032 - 0.065
Wash T2N-R6W-S05W	0.044 - 0.072	0.044 - 0.072
Wash T2N-R6W-S22	0.02 - 0.06	0.02 - 0.06
Wash T2N-R6W-S22S	0.035	0.035
Wash T2N-R6W-S28N	0.02 - 0.06	0.02 - 0.06
Wash T2N-R6W-S36	0.06	0.055
Wash T2N-R6W-S36W	0.045	0.05
Wash T2N-R7W-S20W	0.042 - 0.072	0.02 - 0.072
Wash T2N-R7W-S32E	0.042 – 0.065	0.02 – 0.065
Wash T2N-R7W-S35W	0.02 – 0.065	0.02 – 0.065
Wash T3N-R6W-S27W	0.044 - 0.072	0.02 - 0.072
Wash T3N-R6W-S32	0.044 - 0.072	0.044 - 0.072
Wash T3N-R6W-S33	0.044 - 0.072	0.044 - 0.072
Wash T3N-R6W-S35	0.037 - 0.066	0.037 - 0.066
Wash T5N-R4W-S3	0.040 - 0.065	0.04 - 0.065
Wash T5N-R4W-S7A	0.050	0.065
Wash T5N-R4W-S7B	0.065	0.075
Wash T5N-R4W-S7C	0.060	0.075
Wash T5N-R4W-S19	0.045	0.045 - 0.06
Wash T5N-R4W-S20A	0.040 - 0.045	0.04 - 0.11
Wash T5N-R4W-S20B	0.040 - 0.055	0.04 - 0.06

Table 5. Range of Hydraulic Roughness Coefficients (Manning's "n") (Continued)

Flooding Source	Channel	Overbanks
Wash T5N-R4W-S21	0.040 - 0.045	0.045 - 0.065
Wash T5N-R4W-S30	0.045	0.045 - 0.05
Wash T5N-R5W-S1	0.045	0.045 - 0.07
Wash T5N-R5W-S3A	0.050	0.055
Wash T5N-R5W-S3B	0.045 - 0.050	0.04 - 0.055
Wash T5N-R5W-S10A	0.050 - 0.060	0.050 - 0.055
Wash T5N-R5W-S11	0.045 - 0.055	0.045 - 0.055
Wash T5N-R5W-S12	0.050	0.065
Wash T5N-R5W-S13A	0.050	0.065
Wash T5N-R5W-S13B	0.045	0.045 - 0.065
Wash T5N-R5W-S14	0.050 - 0.055	0.05 - 0.055
Wash T5N-R5W-S14B	0.050	0.05
Wash T5N-R5W-S22	0.065	0.050 - 0.065
Wash T5N-R5W-S23A	0.050 - 0.065	0.05 - 0.06
Wash T5N-R5W-S23B	0.045 - 0.060	0.04 - 0.06
Wash T5N-R5W-S23C	0.050	0.05 - 0.055
Wash T5N-R5W-S23D	0.050	0.05 - 0.055
Wash T5N-R5W-S23E	0.050 - 0.055	0.05
Wash T5N-R5W-S23F	0.050	0.055
Wash T5N-R5W-S25A	0.045	0.045 - 0.07
Wash T5N-R5W-S25B	0.045 - 0.06	0.045 - 0.055
Wash T5N-R5W-S25C	0.040 - 0.055	0.04 - 0.065
Wash T5N-R5W-S34C	0.050 - 0.055	0.05 - 0.055
Wash T5N-R5W-S35	0.045 - 0.055	0.04 - 0.055
Wash T6-R4W-S33	0.045 - 0.065	0.045 - 0.065
Wash T6-R4W-S33A	0.040	0.04 - 0.06
Wash T6-R5W-S36	0.045	0.045 - 0.055
Wash T6-R5W-S36A	0.045	0.045 - 0.060

Table 5. Range of Hydraulic Roughness Coefficients (Manning's "n") (Continued)

Flooding Source	Channel	Overbanks
Wash T6-R5W-S36B	0.045	0.05 - 0.06
Waterfall Wash	0.035 - 0.050	0.070 - 0.100
Waterman Wash	0.016 - 0.055	0.016 - 0.065
West Fork White Peak Wash	0.024 - 0.055	0.036 - 0.060
West Garambullo Wash	0.024 - 0.055	0.036 - 0.060
West Prong of Waterman Wash	0.036 - 0.038	0.045
White Granite Wash	0.035	0.070
White Peak Wash	0.024 - 0.055	0.036 - 0.060
White Spar Wash	0.025 - 0.06	0.025 - 0.5
White Tanks Wash No. 3	0.035 - 0.045	0.035 - 0.070
Winters Wash	0.02 - 0.065	0.02 - 0.065
Wittmann Area Washes	0.015 - 0.060	0.015 - 0.090
Wittmann Wash Tributary 1	0.033 - 0.077	0.033 - 0.077
Wittmann Wash Tributary 1 Breakout 1	0.033-0.059	0.033-0.059
Wittmann Wash Tributary 1 Breakout 1 of Breakout 3	0.035-0.069	0.035-0.069
Wittmann Wash Tributary 1 Breakout 2	0.033-0.059	0.033-0.059
Wittmann Wash Tributary 1 Breakout 3	0.035-0.069	0.035-0.069
Wittmann Wash Tributary 1 Breakout 4	0.035-0.069	0.035-0.069
Willow Springs Wash	0.020 - 0.045	0.020 - 0.080
Willow Springs Wash Tributary 1	0.030 - 0.040	0.035 - 0.055
Willow Springs Wash Tributary lA	0.028 - 0.050	0.040 - 0.060
Willow Springs Wash Tributary 2	0.030 - 0.055	0.045 - 0.060
Willow Springs Wash Tributary 2A	0.040 - 0.050	0.050 - 0.055
Willow Springs Wash Tributary 3	0.060	0.080
Willow Springs Wash Tributary 4	0.040 - 0.050	0.050
Willow Springs Wash Tributary 5	0.035 - 0.050	0.045 - 0.060
Willow Springs Wash Tributary 5A	0.040	0.045 - 0.050

Salt River photos for the 1978 and 1980 flooding events were extensively used in establishing channel parameters for bank station identification, “n” values, and flood flow conveyance patterns. Information from the current airport channelization project was also transferred to the maps. The Salt River model also includes the proposed south dike on the Salt River, which represents an extension of the airport channelization project. This dike is located between Hohokam Expressway (48th Street) and Priest Road on the southern bank of the Salt River.

Water-surface elevations computed in the HEC-2 hydraulic model were calibrated with the known floodplains of the 1978 and 1980 flooding events. This technique involved the adjustment at conveyance boundaries and “n” values.

The starting water-surface elevation for Scatter Wash was taken from Skunk Creek. Manning's “n” values were determined through field investigations and engineering judgment. Scatter Wash is a relatively flat floodplain for the majority of its reach, with a substantial amount of development in some over bank areas. In the upper Scatter Wash drainage basin, it was determined that flood flows would proceed along the many braided streamlines, until they reach I-17. At I-17, the flows will begin to concentrate in the area north of Williams Road. The 1-percent-annual-chance flows at this point will separate into a north and south branch of Scatter Wash. The Scatter Wash, North Branch, passes under I-17 through two culverts, and over I-17 via Sheet flow action. Scatter Wash, South Branch, continues to flow southerly along the eastern side of I-17, until it eventually ponds and passes under I-17 at Deer Valley Road. Both branches of Scatter Wash join in the vicinity of Rose Garden Lane and 33rd Avenue. At this location, the flows proceed downstream to their confluence with Skunk Creek.

During periods of heavy runoff, flows from Sand Tank and Bender Washes near Gila Bend are intermixed. Highway and railroad bridges traverse both washes. These structures cannot pass a 1-percent-annual-chance flood, resulting in extensive ponding at each obstruction during floods of low frequency.

Apache Creek is located on an alluvial fan near Apache Junction at the base of the Superstition Mountains. A vast network of intermingling channels exists on the fan. Flooding on alluvial fans is often erratic and unpredictable, and flow may occur on separate parts of an alluvial fan during sequent flood events. Flooding in this area was analyzed using alluvial fan methodology by FEMA.

Much of the flooding in the county is caused by sheet flow that originates from alluvial fans. Flows are intercepted by canal levees, railroad embankments, and elevated roads, causing water to pond behind the embankments. Depths of ponding depend on the elevation of the embankments. When the intercepted runoff exceeds ponding storage capacity, the flow will overtop the embankment, thus eroding the levee. Areas immediately down slope of the breakout will be affected by high water. However, flows will fan out to again become shallow sheet flow that is less than 1 foot in depth. Therefore, many areas in the county have been designated Zone X.

Approximate hydraulic analyses for Bulldog, Apache, and Goldfield Washes and the downstream reach of Weekes Wash were carried out using approximate flow velocities and normal-depth calculations. These analyses revealed that the channels have very little capacity relative to the 1-percent-annual-chance flood, and in some cases, the channels are nonexistent. Furthermore, the overbank flow is not confined to a well-defined floodplain, causing shallow flooding. The average depth of flooding for the over bank areas was determined to be less than 1 foot.

Areas of ponding on the upstream side of U.S. Highway 60/89 were also studied. Water-surface elevations for these areas were based on the elevation of the highway grade with shallow flows over the highway of less than 1 foot. This results in average shallow flooding depths behind the highway between 1 and 3 feet.

Cross sections were taken perpendicular to the canals and railroad embankments using topographic maps (Aerial Mapping Company, 1977). The top of the embankments were assumed to be the maximum ponding elevation up slope of the embankment. Flood hazard areas were then determined by projecting this elevation up slope to intersect the natural ground.

The canal levees and railroad embankments do not permanently retain storm flows, but divert them along the embankments. Most of the canal levees consist of unconsolidated material. These levees are subject to failure when runoff volumes exceed storage capacity. Potential flood hazard areas on the down slope side of the canals were analyzed for levees exceeding 2 feet in height. This analysis determined the distance required for flow through a break in a levee to spread and be reduced to an average depth of 1 foot, using Manning's equation. This analysis assumed the following:

1. A canal breach could occur at any point.

2. A broad, cresting horizontal weir equation with a head of three feet could be used to determine the length of a breach, resulting in a weir from 50 to 100 feet long.

3. Floodwaters would spread at a 45 degree angle from the breach in the levee.

4. The peak discharge at a potential levee break was the maximum canal capacity or the concentration of peak flows from runoff in the watershed, whichever was greater.

Due to the nature of flooding along the New River, Skunk Creek below Carefree Highway, Lower El Mirage Wash, Scatter Wash below Black Canyon Highway and East Branch Scatter Wash, no 0.2-percent-annual-chance flood profiles were developed. The floodplains of these streams are wide; therefore, flow could increase substantially without significantly raising the water-surface elevation or increasing the velocity of flow. Moreover, most of the area contiguous to the floodplains is subject to sheet flow during a 1-percent-annual-chance flood.

In addition, 2-percent-annual-chance flood profiles for the Agua Fria and New Rivers, Skunk Creek below Carefree Highway, Cave Creek below Cave Creek Dam, East Fork of Cave Creek, and Echo Canyon, Scatter, and East Branch Scatter Washes were not computed.

Flood profiles are not applicable for areas of shallow flooding and ponding; therefore, flood profiles are not presented for any of the canals or other areas of shallow flooding, including Sand Tank and Bender Washes, Rodeo Wash and its tributary, Lower El Mirage Wash Tributary, and Airport and Scott Avenue Washes.

For flooding sources studied by approximate methods, 1-percent-annual-chance flood elevations were computed using Manning's equation, USACE Floodplain Information reports (U.S. Department of the Army, 1967; U.S. Department of the Army, 1972; U.S. Department of the Army, 1965; U.S. Department of the Army, 1964), USGS Flood-Prone Area Maps (U.S. Department of the Interior, 1969, 1972), USGS slope maps (U.S. Department of the Interior, 1974-75), high-resolution Skylab photographs (National Aeronautics and Space Administration, Skylab Earth Terrain Camera Color Photography, 1973; National Aeronautics and Space Administration, Skylab Multispectral Camera Black and White Photography), and USGS topographic maps (U.S. Department of the Interior, 1964, et cetera).

The study was limited to the uses of fixed-bed modeling for the hydraulic analyses. However, with the occurrence of a large flood, substantial changes in the riverbed are expected to occur, particularly where the bottom slope is very non-uniform and/or where other structures, such as bridges, cause local increases in the velocity. Resultant changes in the water-surface elevations can be expected.

The hydraulic analyses for the levee failure scenario for Stagecoach Pass Wash were based on the HEC-RAS model developed by DEI Professional Services (2005). For the levee failure scenario the levee was removed from the cross section and the resulting water-surface elevations determined.

Cross-section data for the Stagecoach Pass Wash Overflow hydraulic analyses were obtained from digital topographic mapping developed from Grading Plan Sand Flower II (American Engineering Company, 1954) and North Scottsdale Floodplain Delineation Study (DEI Professional Services, 2006). The starting water-surface elevation was determined by the slope-area method. Water-surface elevations were computed using the USACE HEC-RAS step-backwater computer program (USACE, 2002).

The hydraulic analyses for this study were based on unobstructed flow. The flood elevations shown on the profiles are thus considered valid only if hydraulic structures remain unobstructed, operate properly, and do not fail.

To obtain current elevation, description, and/or location information for National Geodetic Survey bench marks shown on this map, please contact the Information Services Branch of the National Geodetic Survey at (301) 713-3242, or visit its website at http://www.ngs.noaa.gov. To obtain information about Geodetic Densification and Cadastral Survey bench marks produced by the Maricopa County Department of Transportation, please visit the Flood Control District of Maricopa County website at http://www.fcd.maricopa.gov/Maps/gismaps/apps/gdacs/application/index.cfm.

3.3 Vertical Datum

All FISs and FIRMs are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or revised FISs and FIRMs was the National Geodetic Vertical Datum of 1929 (NGVD 29). With the finalization of the North American Vertical Datum of 1988 (NAVD 88), many FIS reports and FIRMs are being prepared using NAVD 88 as the referenced vertical datum.

All flood elevations shown in this FIS report and on the FIRM are referenced to NAVD 88. Structure and ground elevations in the community must, therefore, be referenced to NAVD 88. It is important to note that adjacent communities may be referenced to NGVD 29. This may result in differences in base flood elevations across the corporate limits between the communities.

Prior versions of the FIS report and FIRM were referenced to NGVD 29. When a datum conversion is effected for an FIS report and FIRM, the Flood Profiles, base flood elevations (BFEs) and ERMs reflect the new datum values. To compare structure and ground elevations to 1-percent-annual-chance flood elevations shown in the FIS and on the FIRM, the subject structure and ground elevations must be referenced to the new datum values.

As noted above, the elevations shown in the FIS report and on the FIRM for Maricopa County are referenced to NAVD 88. Ground, structure, and flood elevations may be compared and/or referenced to NGVD 29 by applying a standard conversion factor.

Due to the statistically significant range in conversion factors, an average conversion factor could not be established for the entire community. The elevations shown in the FIS report and on the FIRM were, therefore, converted to NAVD 88 using information provided by Maricopa County. Users wishing to obtain flood elevations referenced to NGVD 29 may use the following Maricopa County website application: http://www.fcd.maricopa.gov/Maps/gismaps/apps/gdacs/application/index.cfm. This web tool allows users to obtain point-specific datum conversion values by zooming in and hovering over a Vertcon point with their mouse. The application requires that the Vertcon layer be turned on by selecting the Vertcon checkbox on the layers menu on the left side of the screen. The Vertcon grid referenced in this web application was also used to convert existing flood elevations from NGVD 29 to NAVD 88.

The BFEs shown on the FIRM represent whole-foot rounded values. For example, a BFE of 102.4 will appear as 102 on the FIRM and 102.6 will appear as 103. Therefore, users that wish to convert the elevations in this FIS to NGVD 29 should apply the stated conversion factor(s) to elevations shown on the Flood Profiles and supporting data tables in the FIS report, which are shown at a minimum to the nearest 0.1 foot.

For more information on NAVD 88, see Converting the National Flood Insurance Program to the North American Vertical Datum of 1988, FEMA Publication FIA-20/June 1992.

4.0 FLOODPLAIN MANAGEMENT APPLICATIONS

The NFIP encourages State and local governments to adopt sound floodplain management programs. To assist in this endeavor, each FIS provides 1-percent-annual-chance floodplain data, which may include a combination of the following: 10-, 2-, 1-, and 0.2-percent-annual-chance flood elevations; delineations of the 1 and 0.2-percent-annual-chance floodplains; and 1-percent-annual-chance floodway. This information is presented on the FIRM and in many components of the FIS, including Flood Profiles, Floodway Data tables, and Summary of Stillwater Elevation tables. Users should reference the data presented in the FIS as well as additional information that may be available at the local community map repository before making flood elevation and/or floodplain boundary determinations.

4.1 Flood Boundaries

To provide a national standard without regional discrimination, the 1-percent-annual-chance flood has been adopted by FEMA as the base flood for floodplain management purposes. The 0.2-percent-annual-chance flood is employed to indicate additional areas of flood risk in the community. For each stream studied by detailed methods, the 1- and 0.2-percent-annual-chance floodplain boundaries have been delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated using topographic maps at scales of 1:1,200, 1:2,400, 1:4,800, and 1:6,000, with contour intervals of 2 and 4 feet (Harris-Toups Associates, 1976; City of Phoenix, 1967; U.S. Department of the Army, 1976; and Aerial Mapping Company, 1977).

The 1- and 0.2-percent-annual-chance floodplain boundaries are shown on the FIRM (Exhibit 1). In cases where the 1- and 0.2-percent-annual-chance floodplain boundaries are close together, only the 1-percent-annual-chance floodplain boundary has been shown. Small areas within the floodplain boundaries may lie above the flood elevations but cannot be shown due to limitations of the map scale and/or lack of detailed topographic data.

Approximate flood boundaries were delineated using USGS topographic maps and Flood-Prone Areas Maps (U.S. Department of the Interior, 1969, 1972; U.S. Department of the Interior, 1964, et cetera), and high-resolution Skylab photographs (National Aeronautics and Space Administration, Skylab Earth Terrain Camera Color Photography, 1973; National Aeronautics and Space Administration, Skylab Multispectral Camera Black and White Photography).

For Stagecoach Pass Wash the 1- and 0.2-percent-annual-chance floodplain boundaries have been delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated using digital topographic mapping developed from Grading Plan Sand Flower II (American Engineering Company, 1954) and North Scottsdale Floodplain Delineation Study (DEI Professional Services, 2006).

For Stagecoach Pass Wash Overflow the 1-percent-annual-chance floodplain boundaries have been delineated using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated using digital topographic mapping developed from Grading Plan Sand Flower II (American Engineering Company, 1954) and North Scottsdale Floodplain Delineation Study (DEI Professional Services, 2006).

4.2 Floodways

Encroachment on floodplains, such as structures and fill, reduces flood-carrying capacity, increases flood heights and velocities, and increases flood hazards in areas beyond the encroachment itself. One aspect of floodplain management involves balancing the economic gain from floodplain development against the resulting increase in flood hazard. For purposes of the NFIP, a floodway is used as a tool to assist local communities in this aspect of floodplain management. Under this concept, the area of the 1-percent-annual-chance floodplain is divided into a floodway and a floodway fringe. The floodway is the channel of a stream, plus any adjacent floodplain areas, that must be kept free of encroachment so that the 1-percent-annual-chance flood can be carried without substantial increases in flood heights. Minimum Federal standards limit such increases to 1.0 foot, provided that hazardous velocities are not produced. The floodways in this study are presented to local agencies as minimum standards that can be adopted directly or that can be used as a basis for additional floodway studies.

The floodways presented in this study were computed on the basis of equal-conveyance reduction from each side of the floodplain. The results of these computations are tabulated at selected cross sections for each stream segment for which a floodway is computed (Table 6).

As shown on the FIRM (Exhibit 1), the floodway boundaries were computed at cross sections. Between cross sections, the boundaries were interpolated. In cases where the floodway and 1-percent-annual-chance floodplain boundaries are either close together or collinear, only the floodway boundary has been shown.

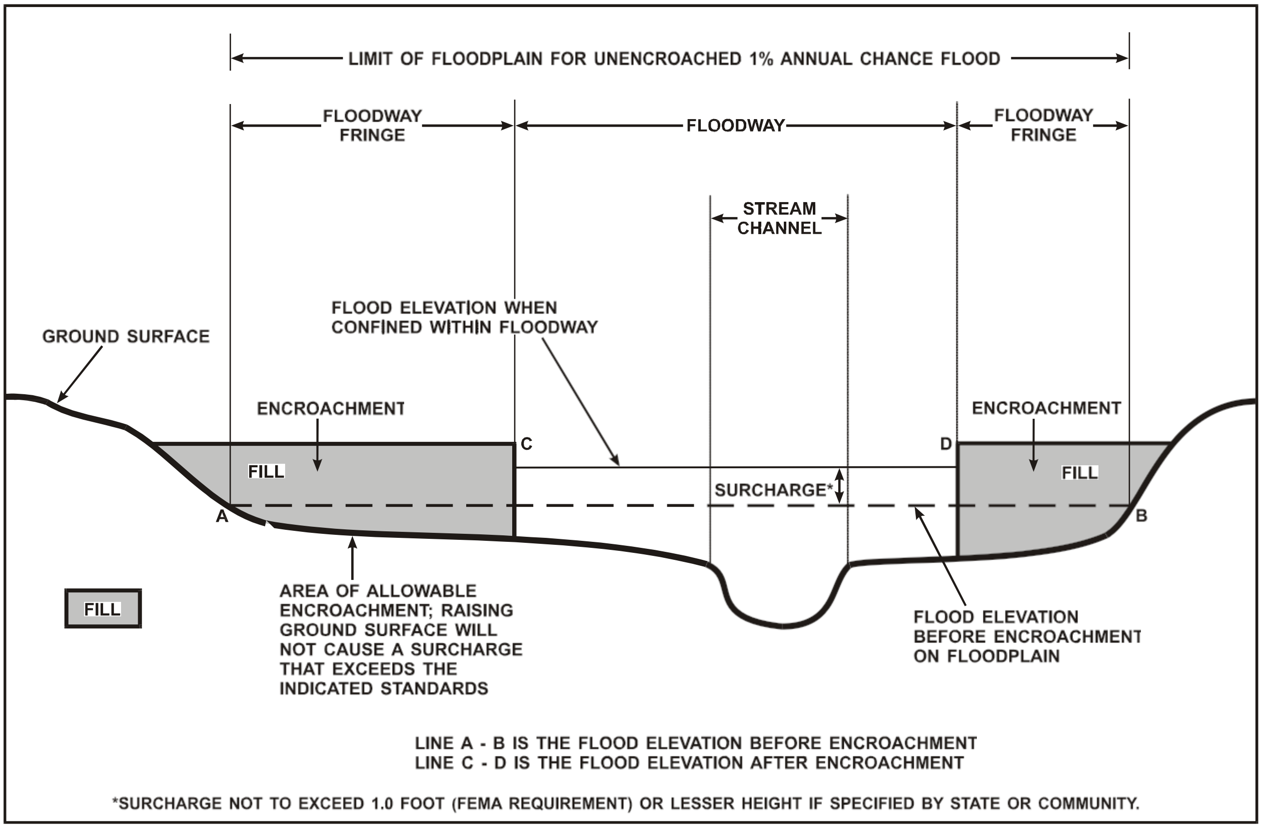
The floodways for Little San Domingo, Mockingbird, and Powder House Washes are shown coincident with the 1-percent-annual-chance floodplain boundaries because of high, hazardous velocities in their respective floodplains. No floodway was computed for Cave Creek below Arizona Canal. No floodway was computed for Wittmann Drainage due to the split flow below Center Street. Floodways for Grass Wash below the U.S. Highway 60 bridge and for Aguila Farm Channel were not computed due to excessive overbank losses.

Floodways are not applicable for areas of shallow flooding; therefore, floodways were not computed for any of the canals, railroad embankments, or for Sand Tank and Bender Washes, Rodeo Wash and its tributary, Airport and Scott Avenue Washes, Lower El Mirage Wash Tributary, and Apache Creek.

Administrative floodways have been established for some approximate SFHAs to aid local officials in regulating and managing of these areas.

The area between the floodway and 1-percent-annual-chance floodplain boundaries is termed the floodway fringe. The floodway fringe encompasses the portion of the floodplain that could be completely obstructed without increasing the water-surface elevation of the 1-percent-annual-chance flood by more than 1.0 foot at any point. Typical relationships between the floodway and the floodway fringe and their significance to floodplain development are shown in Figure 6.

Figure 6. Floodway Schematic



FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER SURFACE ELEVATION			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
					(FEET NAVD)			
Agua Fria River								
A	845	2,185	6,091	8.8	923.4	917.5[2]	917.6	0.1
B	1,827	1,113	30,454	7.7	923.4	922.0[2]	922.4	0.4
C	2,835	2,625	30,370	4.8	924.6	924.6	925.4	0.8
D	3,839	2,278	14,552	4.6	927.1	927.1	927.7	0.7
E	4,797	1,724	9,513	5.9	929.4	929.4	930.2	0.8
F	5,727	1,638	11,491	4.7	931.1	931.1	931.9	0.8
G	6,752	1,459	8,275	7.4	932.3	932.3	932.9	0.6
H	7,722	1,658	10,011	5.1	935.3	935.3	935.8	0.5
I	8,717	1,808	9,465	5.4	936.7	936.7	937.1	0.4
J	9,722	1,873	8,240	6.2	938.4	938.4	938.7	0.4
K	10,681	2,001	9,169	6.1	939.9	939.9	940.3	0.5
L	11,669	1,662	8,216	6.2	941.9	941.9	942.3	0.5
M	12,508	1,425	7,770	6.6	943.5	943.5	944.1	0.6
N	13,490	1,379	8,193	6.2	945.8	945.8	946.4	0.6
O	14,483	1,577	9,414	5.4	947.7	947.7	948.4	0.7
P	15,491	1,984	12,997	3.9	949.2	949.2	949.7	0.5
Q	16,473	1,985	11,912	4.3	950.3	950.3	950.6	0.3
R	17,487	1,818	9,986	5.1	951.7	951.7	951.9	0.2
S	18,496	1,356	9,699	5.3	953.2	953.2	953.3	0.1
T	19,182	1,179	5,813	8.8	953.8	953.8	953.9	0.1
U	19,547	1,099	5,880	8.7	955.5	955.5	955.5	0.0
V	20,418	807	5,876	8.7	958.4	958.4	958.4	0.0
W	21,173	1,116	7,448	6.8	961.5	961.5	961.5	0.0
X	21,986	1,114	7,871	6.6	963.2	963.2	963.2	0.0
Y	22,884	1,111	4,517	11.5	964.2	964.2	964.2	0.0
Z	23,924	1,111	7,809	6.7	967.8	967.8	967.8	0.0

[1]Stream Distance in Feet Above Confluence With Gila River

[2]Elevations Computed Without Consideration of Backwater Effects from Gila River

TABLE 6

FEDERAL EMERGENCY MANAGEMENT AGENCY

MARICOPA COUNTY, AZ AND INCORPORATED AREAS

FLOODWAY DATA

AGUA FRIA RIVER

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER SURFACE ELEVATION			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
					(FEET NAVD)			
Agua Fria River (Cont'd)								
AA	25,133	1,110	7,385	7.0	969.4	969.4	969.4	0.0
AB	26,062	1,110	7,755	6.7	970.9	970.9	970.9	0.0
AC	26,564	1,118	7,371	7.1	972.1	972.1	972.1	0.0
AD	27,752	1,286	9,316	5.6	974.2	974.2	974.2	0.0
AE	28,433	1,382	9,759	5.3	975.1	975.1	975.1	0.0
AF	29,082	1,363	6,862	7.6	977.9	977.9	977.9	0.0
AG	29,394	1,363	7,785	6.7	979.7	979.7	979.7	0.0
AH	30,217	1,249	8,869	6.6	981.7	981.7	981.7	0.0
AI	31,083	1,132	7,482	7.3	983.7	983.7	983.7	0.0
AJ	31,664	1,123	7,816	7.0	984.9	984.9	984.9	0.0
AK	32,615	1,114	7,581	7.2	986.4	986.4	986.4	0.0
AL	33,523	1,112	7,251	7.5	987.8	987.8	987.8	0.0
AM	34,526	1,116	6,871	7.9	990.5	990.5	990.5	0.0
AN	35,471	1,068	7,438	7.3	992.4	992.4	992.4	0.0
AO	36,633	1,113	7,543	7.2	994.7	994.7	994.7	0.0
AP	37,451	1,112	8,193	6.6	996.4	996.4	996.4	0.0
AQ	38,491	1,110	7,147	7.6	999.0	999.0	999.0	0.0
AR	39,452	1,061	7,062	7.7	1001.1	1001.1	1001.1	0.0
AS	40,445	1,005	6,722	8.1	1003.1	1003.1	1003.1	0.0
AT	41,437	984	6,854	7.9	1005.3	1005.3	1005.3	0.0
AU	42,451	1,030	7,788	7.0	1007.1	1007.1	1007.1	0.0
AV	43,417	1,316	8,183	6.7	1008.5	1008.5	1008.5	0.0
AW	44,167	1,501	9,243	5.9	1010.2	1010.2	1010.2	0.0
AX	45,329	1,562	12,867	4.2	1012.9	1012.9	1012.9	0.0
AY	46,432	2,565	23,592	2.3	1013.4	1013.4	1013.5	0.1
AZ	47,663	1,846	15,094	3.6	1013.7	1013.7	1013.7	0.0

[1]Stream Distance in Feet Above Confluence With Gila River

TABLE 6

FEDERAL EMERGENCY MANAGEMENT AGENCY

MARICOPA COUNTY, AZ AND INCORPORATED AREAS

FLOODWAY DATA

AGUA FRIA RIVER

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER SURFACE ELEVATION			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
					(FEET NAVD)			
Agua Fria River (Cont'd)								
BA	48,851	1,327	7,245	7.5	1,014.2	1,014.2	1,014.2	0.0
BB	49,827	1,662	8,906	6.1	1,023.3	1,023.3	1,023.3	0.0
BC	50,688	1,851	8,622	6.3	1,025.8	1,025.8	1,025.8	0.0
BD	51,617	1,503	7,851	6.9	1,028.2	1,028.2	1,028.2	0.0
BE	52,577	2,203	11,152	4.9	1,030.5	1,030.5	1,030.5	0.0
BF	53,574	1,394	5,092	5.9	1,032.2	1,032.2	1,032.6	0.4
BG	54,543	1,231	5,331	5.6	1,034.6	1,034.6	1,035.0	0.4
BH	55,563	740	3,634	8.3	1,036.4	1,036.4	1,036.7	0.3
BI	56,482	749	4,886	6.1	1,040.3	1,040.3	1,040.9	0.6
BJ	57,516	985	6,513	4.6	1,042.3	1,042.3	1,042.9	0.6
BK	58,648	730	4,884	6.1	1,045.1	1,045.1	1,045.3	0.2
BL	60,638	812	5,451	5.5	1,048.8	1,048.8	1,049.0	0.2
BM	61,164	620	2,998	10.0	1,050.3	1,050.3	1,050.3	0.0
BN	62,438	892	5,128	6.7	1,056.3	1,056.3	1,056.3	0.0
BO	63,532	1,738	6,858	5.0	1,060.6	1,060.6	1,061.0	0.4
BP	64,311	2,408	6,640	5.2	1,062.9	1,062.9	1,063.2	0.3
BQ	65,058	2,709	17,026	2.0	1,065.8	1,065.8	1,066.2	0.4
BR	66,132	2,586	19,624	1.8	1,067.6	1,067.6	1,067.8	0.2
BS	67,272	2,260	8,235	4.6	1,068.7	1,068.7	1,068.8	0.1
BT	68,150	850	9,133	3.8	1,069.3	1,069.3	1,069.4	0.1
BU	69,132	680	5,266	6.9	1,069.4	1,069.4	1,069.6	0.2
BV	70,130	447	4,260	8.1	1,072.9	1,072.9	1,072.9	0.0
BW	71,147	552	5,639	6.1	1,074.7	1,074.7	1,074.7	0.0
BX	72,560	1,342	7,481	4.7	1,077.4	1,077.4	1,077.4	0.0
BY	73,571	1,334	5,387	6.4	1,080.9	1,080.9	1,081.1	0.2
BZ	74,592	1,040	5,776	6.0	1,084.0	1,084.0	1,084.3	0.3

[1]Stream Distance in Feet Above Confluence With Gila River

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER SURFACE ELEVATION			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY (FEET NAVD)	WITHOUT FLOODWAY (FEET NAVD)	WITH FLOODWAY (FEET NAVD)	INCREASE (FEET NAVD)
Buchanan Wash								
A	935	255	862	2.7	1,457.6	1452.4[2]	1452.4[2]	0.0
B	1,404	516	666	3.5	1,457.6	1453.1[2]	1453.1[2]	0.0
C	2,196	705	1,859	3.6	1,457.6	1456.6[2]	1456.8[2]	0.2
D	2,855	52	735	8.3	1,461.3	1,461.3	1,461.3	0.0
E	3,686	124	439	4.3	1,465.6	1,465.6	1,466.0	0.4
F	4,677	156	502	3.1	1,468.2	1,468.2	1,468.6	0.4
G	5,333	211	531	4.3	1,473.3	1,473.3	1,473.3	0.0
H	5,834	168	440	5.2	1,476.0	1,476.0	1,476.0	0.0
I	6,362	156	501	3.2	1,479.2	1,479.2	1,479.2	0.0
J	6,832	183	475	3.4	1,481.2	1,481.2	1,481.2	0.0
K	7,482	171	385	4.2	1,484.4	1,484.4	1,484.7	0.3
L	8,316	100	370	4.3	1,489.4	1,489.4	1,489.7	0.3
M	9,324	115	337	4.8	1,493.9	1,493.9	1,494.4	0.5
N	10,217	76	195	6.7	1,498.1	1,498.1	1,498.8	0.7

1 Stream Distance in Feet Above Confluence With Skunk Creek

2 Elevations Computed without Consideration of Backwater from Skunk Creek

TABLE 6

FEDERAL EMERGENCY MANAGEMENT AGENCY

MARICOPA COUNTY, AZ

AND INCORPORATED AREAS

FLOODWAY DATA

BUCHANAN WASH

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER SURFACE ELEVATION			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
					(FEET NAVD)			
Cave Creek (Cont'd)								
AA	107,369	73	640	8.9	1,349.2	1,349.2	1,349.3	0.1
AB	107,833	114	796	7.2	1,351.4	1,351.4	1,351.5	0.1
AC	108,393	139	841	6.8	1,354.1	1,354.1	1,354.1	0.0
AD	108,916	130	1,059	5.4	1,355.1	1,355.1	1,355.1	0.0
AE	109,428	64	398	14.3	1,357.8	1,357.8	1,357.8	0.0
AF	110,141	162	1,041	5.5	1,368.8	1,368.8	1,369.3	0.5
AG	110,579	220	1,271	4.5	1,369.7	1,369.7	1,370.0	0.3
AH	111,498	151	1,041	5.5	1,371.0	1,371.0	1,371.2	0.2
AI	112,395	229	1,190	4.8	1,377.3	1,377.3	1,377.4	0.1
AJ	113,193	231	1,199	4.8	1,378.3	1,378.3	1,378.3	0.0
AK	113,895	211	919	6.2	1,380.2	1,380.2	1,380.2	0.0
AL	114,254	198	593	9.6	1,385.1	1,385.1	1,385.1	0.0
AM	114,634	198	550	10.0	1,388.8	1,388.8	1,388.8	0.0
AN	114,951	235	1,311	4.2	1,390.5	1,390.5	1,390.5	0.0
AO	115,548	222	1,321	4.2	1,393.6	1,393.6	1,393.6	0.0
AP	115,796	198	1,226	4.5	1,393.9	1,393.9	1,393.9	0.0
AQ	116,102	85	540	10.2	1,394.3	1,394.3	1,394.3	0.0
AR	116,704	406	717	7.7	1,403.6	1,403.6	1,403.9	0.3
AS	117,031	303	663	8.3	1,407.0	1,407.0	1,407.0	0.0
AT	117,601	241	880	6.2	1,410.2	1,410.2	1,410.2	0.0
AU	118,325	156	531	10.4	1,415.8	1,415.8	1,415.8	0.0
AV	118,840	205	1,334	4.1	1,418.1	1,418.1	1,418.3	0.2
AW	119,212	96	715	7.7	1,418.4	1,418.4	1,418.5	0.1
AX	119,724	198	1,406	3.9	1,419.9	1,419.9	1,420.6	0.7
AY	120,352	183	1,174	4.7	1,420.5	1,420.5	1,421.5	1.0
AZ	120,806	169	774	7.1	1,422.0	1,422.0	1,422.4	0.4

[1] Stream Distance in Feet Above Confluence With Gila River

FLOODING SOURCE		FLOODWAY			BASE FLOOD WATER SURFACE ELEVATION			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQUARE FEET)	MEAN VELOCITY (FEET PER SECOND)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
					(FEET NAVD)			
Cave Creek (Cont'd)								
BA	121,392	174	522	7.0	1,424.3	1,424.3	1,424.7	0.4
BB	121,847	117	510	6.5	1,433.1	1,433.1	1,433.1	0.0
BC	121,973	111	471	7.0	1,433.6	1,433.6	1,433.6	0.0
BD	122,454	58	270	12.2	1,437.5	1,437.5	1,437.5	0.0
BE	123,061	64	405	8.2	1,444.6	1,444.6	1,444.6	0.0
BF	123,494	62	393	8.4	1,446.9	1,446.9	1,446.9	0.0
BG	124,016	68	451	7.3	1,449.7	1,449.7	1,449.7	0.0
BH	124,434	67	427	7.7	1,451.5	1,451.5	1,451.5	0.0
BI	124,935	63	394	8.4	1,454.0	1,454.0	1,454.0	0.0
BJ	125,595	112	688	4.8	1,457.4	1,457.4	1,457.4	0.0
BK	126,102	64	420	7.9	1,459.8	1,459.8	1,459.8	0.0
BL	126,525	80	494	6.7	1,461.6	1,461.6	1,461.6	0.0
BM	127,079	72	498	6.6	1,463.8	1,463.8	1,463.8	0.0
BN	127,612	152	1,163	2.8	1,464.8	1,464.8	1,464.8	0.0
BO	128,304	261	1,589	2.1	1,465.4	1,465.4	1,465.5	0.1
BP	128,975	150	655	5.0	1,466.3	1,466.3	1,466.5	0.2
BQ	129,619	166	632	5.6	1,470.4	1,470.4	1,470.7	0.3
BR	129,877	100	308	10.0	1,472.2	1,472.2	1,472.2	0.0
BS	130,596	70	337	8.6	1,481.0	1,481.0	1,481.4	0.4
BT	131,102	40	223	13.7	1,484.8	1,484.8	1,485.0	0.2
BU	131,752	71	441	6.5	1,489.0	1,489.0	1,489.6	0.6
BV	132,206	59	263	11.0	1,491.7	1,491.7	1,492.0	0.3
BW	132,818	78	410	7.0	1,495.4	1,495.4	1,496.2	0.8
BX	133,109	195	1,042	2.8	1,496.8	1,496.8	1,497.7	0.9
BY	133,938	60	326	8.8	1,502.0	1,502.0	1,502.0	0.0
BZ	134,260	63	324	8.9	1,503.2	1,503.2	1,503.7	0.5

[1] Stream Distance in Feet Above Confluence With Salt River

TABLE 6

FEDERAL EMERGENCY MANAGEMENT AGENCY

MARICOPA COUNTY, AZ
AND INCORPORATED AREAS

FLOODWAY DATA

CAVE CREEK