GILA RIVER
HYDROLOGY ANALYSIS

FLORENCE, ARIZONA
F.I.A. COMM. NO. 040084

MAY 28, 1989

Registered Professional Engineer (Civil)
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SECTION 1

INTRODUCTION

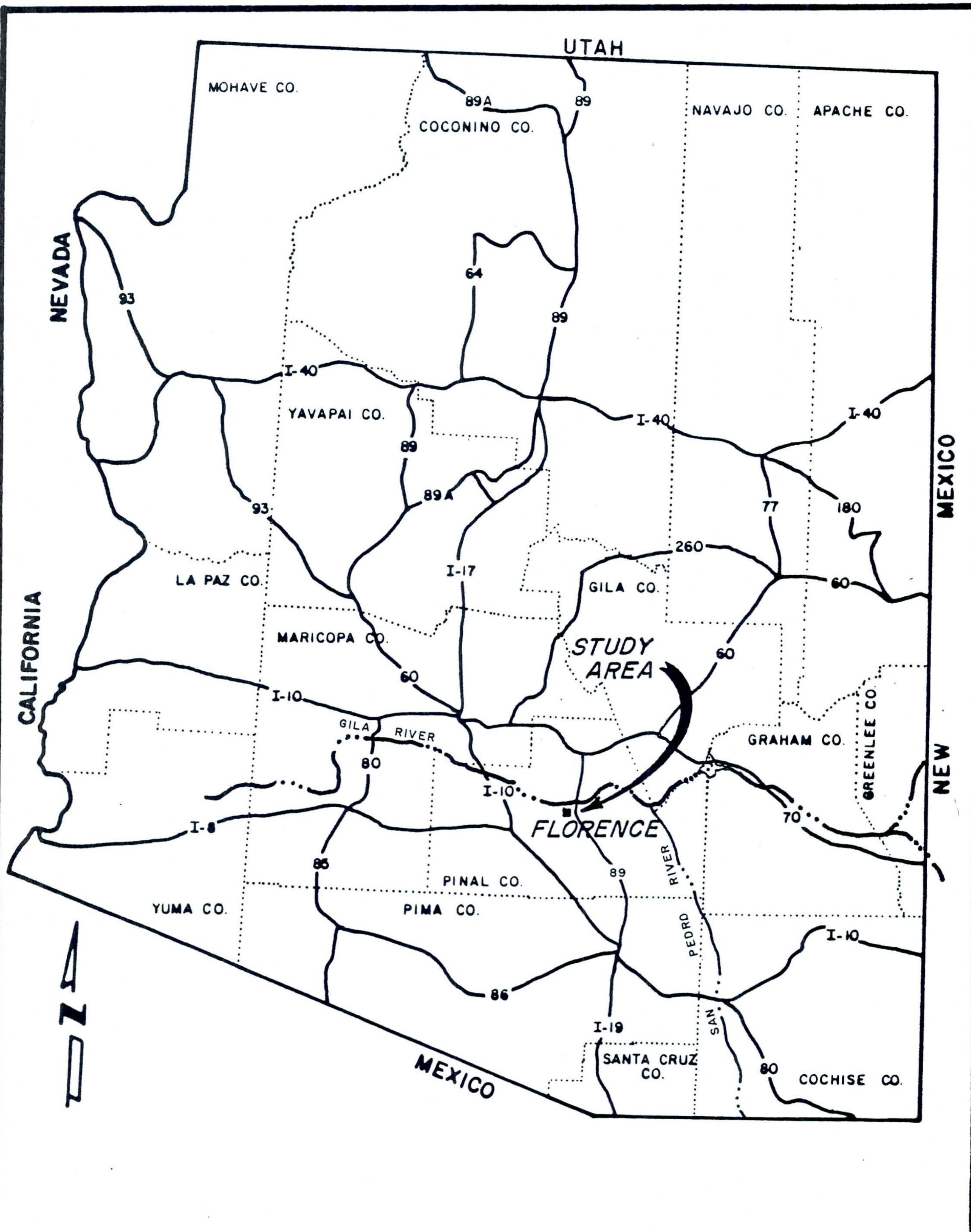
The Gila River is a water course with a drainage basin covering much of the Southeastern part of Arizona, extending into Old and New Mexico. The river bed in the study area is dry for most of the year; only transporting water from snow melt and significant thunder storms.

The Coolidge Dam, located on the border of Gila County and Pinal County, plays a major role in the regulation of the river flow. It contains the San Carlos Reservoir, one of the largest reservoirs in Arizona.

The Town of Florence, Arizona is approximately 75 miles below the Coolidge Dam. Florence is the County Seat of Pinal County. Florence has a population of approximately 4000 people. The Gila River flows southwesterly past the north edge of town. The following page is an area map showing the general vicinity of the study location.

In 1979, the Federal Insurance Administration contracted, under Number H-4607, with a contract consultant to perform a Detailed Analysis of the Gila River adjacent to the Town of Florence, Arizona. The limits of the detailed study extend from U.S. Highway 89 on the east for 1.19 miles downstream.

The results of the study area are published in two reports. One is the "Flood Insurance Study, Town of Florence, Arizona, 1981," (Reference 1). Community Number 040084. The map numbers are 040084 0002 and 0002B. The second is the "Pinal County Flood Insurance Study." Since the majority of the study is outside of the town limits, the maps showing the greatest amount of detail are Pinal County Maps 040077 0514 and 0514C. (Reference 2).



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MESA, ARIZONA

AREA MAP

The study was accepted in 1981 and was used by the local Floodplain Management Agencies to administer their ordinances. Then in October 1983 a major flood occurrence was experienced. Flow volumes on most all major tributaries feeding the Gila River were in excess of the predetermined 100-year flows. The peak flows at Florence, however, were only one half that expected during a 100-year event. Land was not inundated that the maps show should have been. The question arose at that time in the minds of City and County officials, and private land owners how discrepancies could exist between predicted outcomes and actual events.

The purpose of this study is to perform a professional analysis of the original report. A step-by-step methodical approach will be taken to verify that no errors were made during the original report nor unsubstantiated design criteria used that would yield misleading or inaccurate results.

SECTION 2

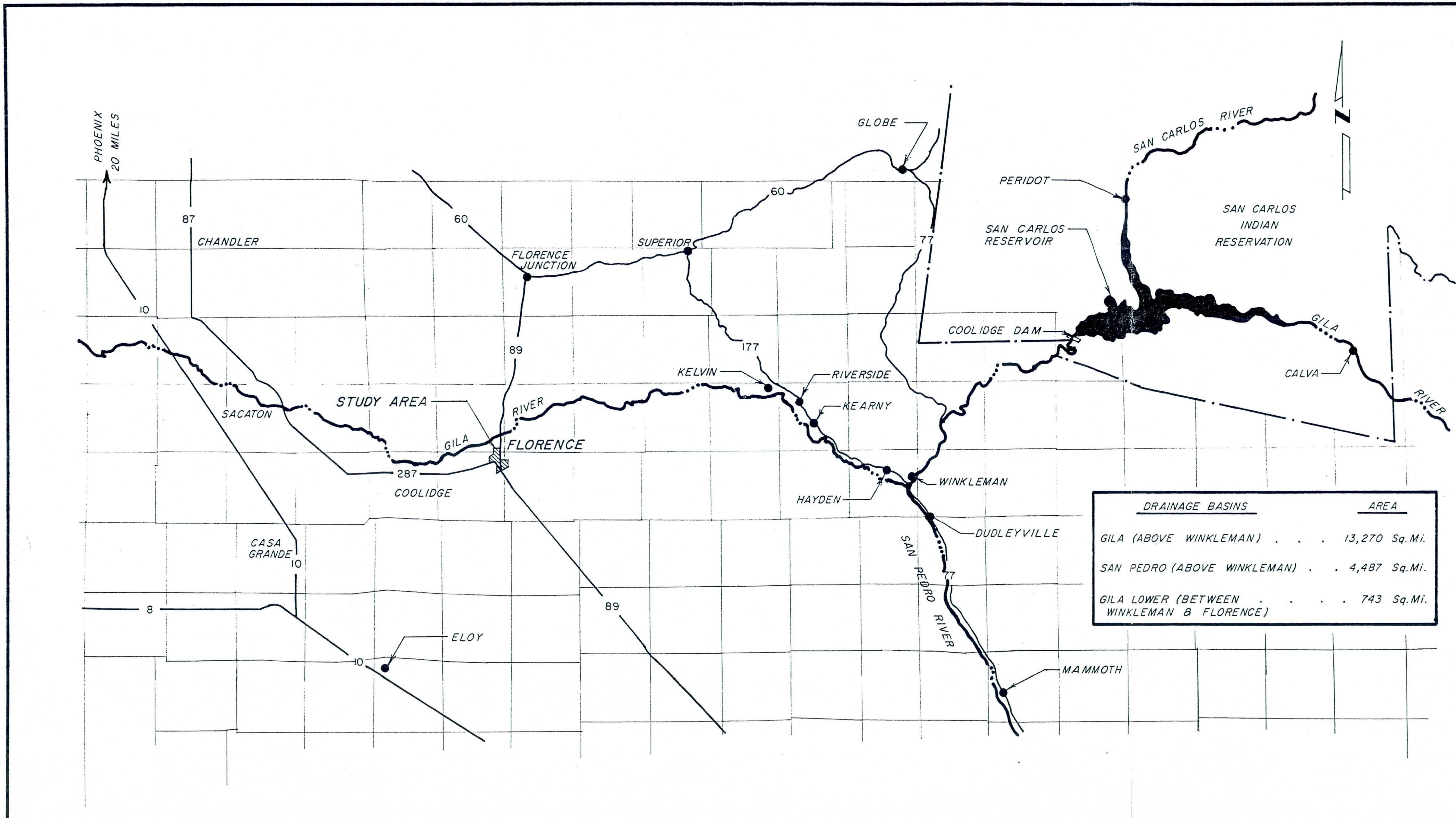
FLOOD OF OCTOBER 1983

Information that was obtained as a result of the 1983 flood on the Gila River has provided this study with a considerable amount of data that was not available to the authors of the original report in 1980. The U.S. Geological Survey performed extensive data measuring and gathering during and after the flood. Three members of that agency published their findings in Report #85-4225-A entitled "U.S.G.S. Flood of October 1983 In Southeastern AZ Along the Gila River." (Ref. 3) That report documents peak flows at several locations along the Gila River and also the San Pedro River which flows into the Gila River 35 miles below Coolidge Dam. Also provided in the report were two flood hydrographs: One at Calva (just upstream from the San Carlos Reservoir) and another at Kelvin (25 miles above Florence). The following page is a detailed map showing locations critical to this analysis.

The following table describes 100-yr peak flow comparisons for several locations along the Gila and San Pedro Rivers.

	1983 RECORDED (Reference 3)	U.S.G.S. (Reference 4)	FLOOD INSURANCE STUDY (Ref. 2)
CALVA	150,000	120,000	N/A
DUDLEYVILLE (SAN PEDRO RIVER)	135,000*	53,000	49,600
WINKELMAN (After Rivers Converge)	125,000	140,000	140,000
KELVIN/RIVERSIDE	100,000	140,000	140,000
FLORENCE	61,000	120,000	120,000

* Verbal Communication with U.S.G.S.



DRAINAGE BASINS	AREA
GILA (ABOVE WINKLEMAN) . . .	13,270 Sq. Mi.
SAN PEDRO (ABOVE WINKLEMAN) . .	4,487 Sq. Mi.
GILA LOWER (BETWEEN WINKLEMAN & FLORENCE)	743 Sq. Mi.

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GILA RIVER HYDROLOGY ANALYSIS

VICINITY MAP

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EJS	SEW	BH
DATE		DRAWING NO.
5-24-89		
SCALE		SHEET NO.
1" = 40,000'		

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The U.S.G.S. documented the 1983 flow at Calva to be a 100-year event. (Ref. 3) They did not place an event year on the flows at Dudleyville. However, the Flood Insurance Study (Ref. 2) listed a much smaller "Q" of 85,000 cfs experienced in 1926 as a 600-yr event. The San Pedro Flood had a peak flow of 59% greater than a predetermined 600-yr event.

The Coolidge Dam played a critical role in regulating the flows on the Gila River leg. Inflow to the reservoir between September 30th and October 7th was about 450,000 acre-ft. (Ref. 6) The peak flow of 150,000 cfs was experienced at Calva the morning of October 3rd. The San Carlos River, flowing into the Reservoir from the north, peaked the morning of October 2nd with a maximum discharge of 11,100 cfs. Water began running over the spillway on October 4th and reached a peak discharge of 5,020 cfs on October 6th. (Ref. 3) This was only the third time in the 54 year life of the dam that the reservoir had filled up to the spillway height. What actually happened then, was that the 100-yr flood on the Gila was practically completely absorbed by the Reservoir. The bizarre event on the San Pedro provided most of the flows that reached Florence.

These two gauging stations represent 2 of the 3 drainage areas contributing to Gila flows at Florence. The third area is the drainage area below the convergence of the two rivers, down to Florence. This is a comparatively small area of roughly 750 square miles. No data is available concerning how much this area contributed to the 1983 flood. Evidence that there likely was substantial runoff from this lower area is provided in the following precipitation data obtained at Florence and Kelvin.

PRECIPITATION (in.)(Ref. 5)

DATE (1983)	FLORENCE	KELVIN
Sept. 27	0.00	0.00
Sept. 28	1.30	0.63
Sept. 29	1.42	2.02
Sept. 30	0.25	0.96
Oct. 1	2.00	1.70
Oct. 2	0.51	1.08
Oct. 3	0.01	0.00
TOTAL	5.49	6.39

Florence's average annual precipitation is between 6 to 10 inches. In the 6 days concurrent with the flooding, the local area received 5.49 inches of rainfall. The peak flows passed by Florence in the afternoon of October 2nd. The initial 2.97 inches would certainly have saturated the soil and filled any natural retention areas. It is reasonable to assume that the following 2.52 inches received during the height of the flooding would have contributed to the impact of the flood.

SECTION 3

FLORENCE AND PINAL F.I.S. ANALYSIS

A. GENERAL

In order to complete a thorough review of the Town and County Flood Insurance Studies, it was necessary to obtain the archive files of the original reports. In determining whether the FIS's are reasonably true and accurate, three areas of question were evaluated. They are:

1. Have changes such as topography, terrain or control structures altered conditions since 1980?

2. Was the original hydraulic analysis modeled correctly through the HEC-2 Computer Program?

3. Was reasonable engineering judgment used in determining the design peak discharges?

The following sub-sections cover these questions.

B. ALTERED CONDITIONS

There appears to have been no significant changes in any of the water courses contributing to the flows at Florence. No major dikes, dams or levees have been constructed. According to elevation survey shots taken as part of this study, the 1983 flood did widen and deepen the river channel. The change was not substantial, but has moderately increased the cross-sectional area of the river.

The Central Arizona Project (CAP) has constructed their concrete canal crossing the Gila. The crossing is an inverted siphon under the river bed. This has yielded a negligible effect on the hydraulics of the Gila River.

C. HYDRAULIC EVALUATION

The original input data of the cross-sections used in this detailed study area were obtained, along with a blue line working drawing, from Cella, Barr & Associates (CBA). The author wishes to acknowledge their cooperation in extending these resources to assist this study.

The U.S.G.S. conducted field surveys upstream and downstream of the U.S. Highway 89 bridge after the 1983 flood. They recorded the high water marks left by that flood and calculated the peak flow to be 61,000 cfs. From the U.S.G.S. field notes it was possible to use the original contractor's HEC-2 model and input the 1983 flows. The output showed the water surface elevations over a foot higher than actual high water marks.

After this result was obtained, a survey crew was sent out to verify cross section elevations. Shots taken up in the overbank area fell consistently with those taken 10 years previous. However, shots taken down in the channel bottom showed the 1983 flood had lowered the channel bed between 18 to 24 inches. The new cross section elevations were then entered in the model. This lowered the profile on an average of 6 to 8 inches. This still did not quite give an accurate representation of the actual flood.

The Mannings "n" values used in the original study were 0.035, 0.030 (in the channel), and 0.035. Research concerning the accuracy of these values on the Gila River uncovered a professional paper authored by U.S.G.S. employee D. E. Burkham entitled "Precipitation, Streamflow, and Major Floods at Selected Sites in the Gila River Drainage Basin above the Coolidge Dam, Arizona." (Ref. 7) In the report, Mr. Burkham quotes a journal entry from an engineer who attempted an indirect measurement on the Gila River back in 1891. The gentleman used an "n" value of 0.025. Mr Burkham commented thusly; "The 'conservative' results based on the 'n' of 0.025 may not be conservative at all, as recent studies have shown that the 'n' for a sand channel may be as low as 0.010." While the author of this report does not concur wholly with this statement, it does support evidence that the 0.030 value could be justifiably decreased. A trial and error approach was used to determine that an "n" value of 0.026 modeled the 1983 flood the most accurate. Hydraulically, then, the model did have to be adjusted, but only in the two ways indicated which are representations of actual existing conditions. These adjustments were relatively minor.

D. HYDROLOGIC EVALUATION

The Florence F.I.S. (Ref. 1) designates the peak 100-yr discharge at Florence to be 120,000 cubic feet per second. In Section 3.1 "Hydrologic Analysis", the following comment is made; "Peak discharge values used in this study were developed by the U.S. Geological Survey using a log-Pearson Type III distribution (Reference 5)." The reference is U.S. Department of the Interior, Geological Survey, "Methods for Estimating the Magnitude and Frequency of Floods in Arizona, R.H. Roeske, Sponsored by Arizona Department of Transportation, September 1978."

This text was a widely used technical report that provided regression equations for estimating flood magnitudes at ungauged sites for various recurrence intervals in Arizona. ADOT no longer publishes the report. The main stem of the Gila River was exempted from these equations. Instead, graphs were presented for estimating the various floods on this river.

Included in the Appendix as Exhibit A are Figures 5 & 6 from that report. Figure 5 shows the various recurrence interval discharges above the Coolidge Dam. Figure 6 shows the discharges coming out of the Dam and along the subsequent miles downstream.

A close observation of these Charts show the 2, 5 and 10-year floods are considerably affected by the Dam. The 25 and 50-year floods are moderately affected by the Dam. The 100 and 500-year floods reflect a direct peak-in, peak-out relationship. These two curves act as though there is no Dam in place. On the log scale, the author is indicating that a 100-year event at Calva is 120,000 cfs. Likewise, immediately below the Dam the peak discharge is 120,000 cfs.

The U.S.G.S. technical report does not include a narrative describing the method or reasoning behind the adjustments. In the project archive files, there is correspondence between the contract consultant and U.S.G.S. concerning the graphs. Responding for U.S.G.S. was Mr. Byron N. Aldridge, Surface Water Specialist. Copies of the corresponding letters and attachments are included in the Appendix - as Exhibit B. In the Exhibit entitled "Reference to Figure 6, Report ADOT-RS-151210", Mr. Aldridge goes into detail to describe how various curves were used for the respective recurrence intervals. The archive file copy of the Florence F.I.S. quotes this document, (Exhibit C). However, the final draft of the F.I.S. converts the quote into a paraphrase. It reads "... large floods (100-year or greater) are caused by uncontrolled releases from the dam." (Ref. 1) Mr. Aldridge further clarifies in Exhibit B; "Curve A is defined by estimates of peak flows that would have reached Kelvin if San Carlos Reservoir did not exist." Mr. R.H. Roeske has confirmed this concept to the author of this report; that the 100-year and 500-year flood discharges ignore the presence and effect the Dam may have on the floods. The following section discusses the ramifications of this assumption and its subsequent effect on the design floods.

Concerning other areas of hydrology; the "Summary of Discharges - Table 2," in the Pinal County F.I.S. (Ref. 2) are relatively consistent with those reflected in the U.S.G.S. Report (Ref. 4). The following are points of observation from those data sources.

From the Coolidge Dam down to the Town of Winkelman, the peak discharge of a 100-year storm remains constant. At that point the San Pedro River drains into the Gila River. This increases the flow by 20,000 cfs. The peak 100-year discharge of the San Pedro River is called out to be 49,800 cfs. However, the San Pedro's drainage area is 4,487 square miles compared to 13,270 square miles for the Gila (above Winkelman). Therefore, stream routing shows the San Pedro peaks significantly earlier than the Gila. This was further documented by the Flood of 1983 when the San Pedro at Kelvin peaked at 0100 hours on October 2, while the Gila peaked way upstream at Calva at 0300 hours on October 3. (See Exhibits D and E)(Ref. 3) The author, therefore, concurs with the original report that a 20,000 cfs increase at convergence with the San Pedro is reasonably accurate.

From the Pinal F.I.S. (Ref. 2) "Summary of Discharges" it can be observed that at Riverside/Kelvin maximum discharges ranging from 66,000 cfs up to 200,000 cfs are projected to decrease by an estimated 20,000 cfs by the time the flows reach Florence. This, of course, is due to the regressive equation principal used for overbank storage of an otherwise dry river bed. In 1983, the measured peak discharge at Kelvin was 100,000 cfs. The measured peak discharge at Florence was 61,000 cfs. This is a difference of 39,000 cfs; nearly twice the figure estimated by U.S.G.S. (Ref. 4) and Pinal F.I.S. (Ref. 2). There is some question as to the magnitude of the contribution to the flood from the lower watershed between the convergence of the two rivers and Florence. Precipitation figures were presented earlier in this report which would support the argument that the lower area's runoff was likely substantial. There was a technical report written in 1988 by Brian M. Reich, Consulting Engineer, entitled "Flood Frequency Methods for Arizona Streams - State of the Art." (Ref. 8) The report was written for the Arizona Department of Transportation to outline how new flood estimators for Arizona could effectively be prepared. In the report Mr. Reich indicates the following:

> "Another change that should be implemented if a new FFA is undertaken concerns the second phase, involving relationships between Q100 on different watersheds... The previous studies (1) [Roeske, 1978], (4) [Eychaner, 1984] combined Q100 estimates from watersheds (WSs) over 5,000 sq. mi. with those of area less that 1/6 sq. mi. Combining point estimates of flood peaks for any design frequency (Q), from such a wide range of watershed sizes (A) into a single regression set overlooks the diverse processes that occur as desert floods propagate downstream. Floods on small headwaters result from local summer rains of high intensity and very short duration. Large watersheds flood in the autumn or winter when persistent, low-intensity rains cover wide areas. The duration of winter storm rainfalls that are casually related to large watershed Q's may be 12 hours or longer. Regressable rainfall intensities for small watersheds will probably be as short as 15 minutes, usually occur in the summer, and are unrelated to long rains. So flood magnitude will not regress against either long or short duration when large and small watersheds are pooled into one sample."

This opinion indicates that whether or not the lower watershed contributed to the flooding in 1983 is irrelevant because it would have peaked and flowed past by the time the other two watershed runoffs arrived.

So the predicted 20,000 cfs loss compared to the '83 experienced loss of 39,000 cfs is a discrepancy. The difference, however, is subject to personal opinion and interpretation.

All other areas associated with peak discharges within the study area appear, in this author's mind, to be consistent and reasonable.

SECTION 4

COOLIDGE DAM REGULATION

A. GENERAL

Approximately 76 miles upstream from the town of Florence is the Coolidge Dam which impounds the San Carlos Reservation. The Bureau of Indian Affairs (BIA) owns the dam and the San Carlos Irrigation Project (SCIP) operates it. The Dam was constructed in 1929. It delivers water downstream to irrigated lands of the Gila River Indian Community and to the San Carlos Irrigation and Drainage District.

The reservoir has been subject to wide fluctuations of volume since its construction. The reservoir has been emptied 18 times. It has only experienced spills over the spillway four times. The reservoir capacity is large; with a surface area at the spillway crest elevation of 17,212 acres. This correlates to a volume capacity of 917,000 acre-feet. (Ref. 9)

The reservoir has had a history of low volumes and inadequate irrigation storage. The table on the following page reflects the average monthly data of the reservoir over its 60 year life.

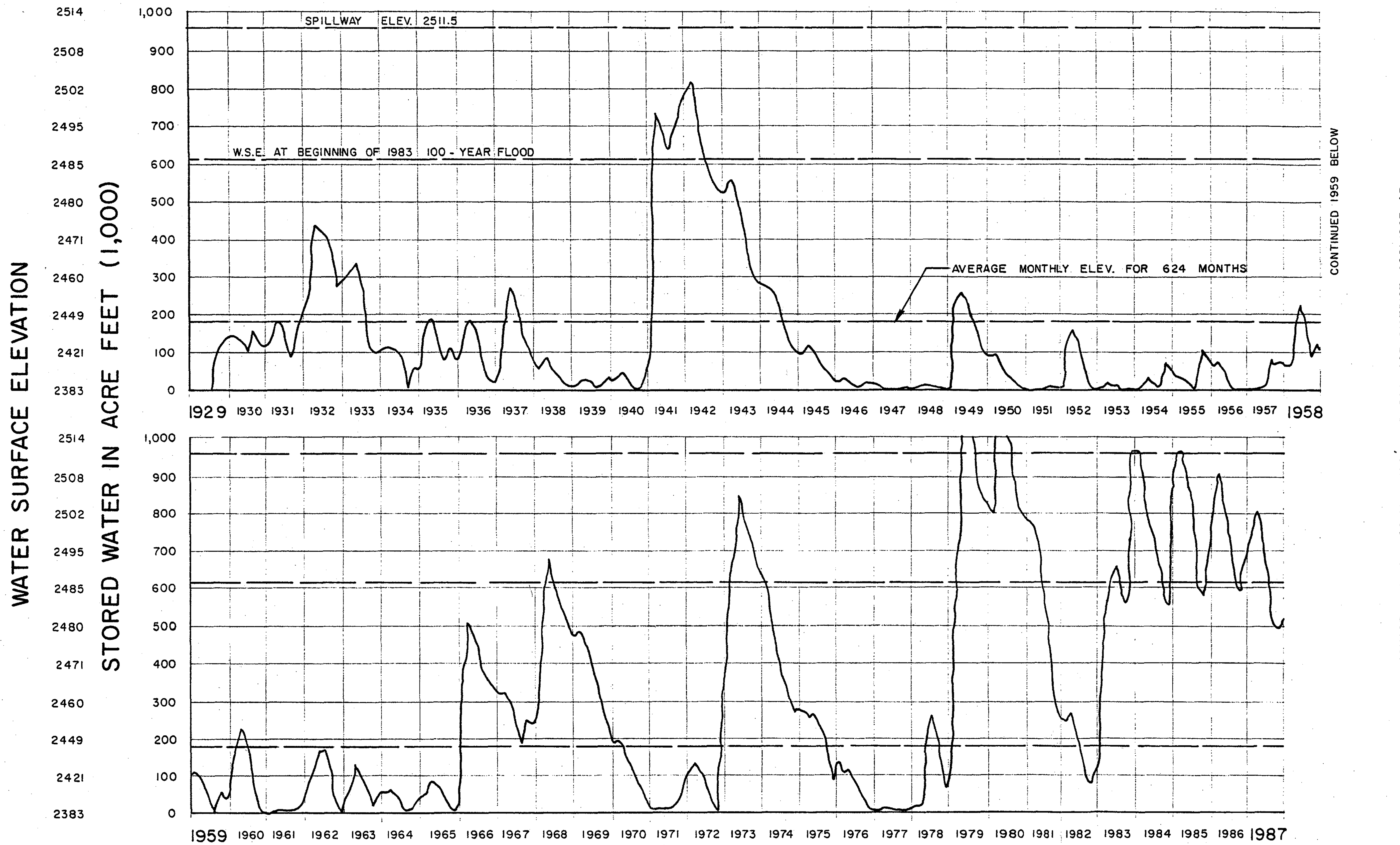
The year-to-year reservoir volume from 1929 to 1987 is graphically presented on page 16. Included on that graph are three additional lines. The line near the top represents the spillway crest elevation. The next line down represents the water surface elevation on September 28, 1983. This line is noteworthy because it was here that the 100-year flood began filling the reservoir with an ultimate negligible outfall of 5023 cfs.

SAN CARLOS RESERVOIR

Monthly Historical Gaged Volume

MONTH*	AVG. ELEV. (FT.)	STORED WATER (ACRE-FEET)	AVAIL. FLOOD CAPACITY (ACRE-FEET)
JAN.	2432	190,655	726,345
FEB.	2438	215,960	701,040
MAR.	2442	245,779	671,221
APR.	2444	264,695	652,305
MAY	2442	245,551	671,449
JUN.	2439	229,686	687,314
JULY	2433	201,112	715,888
AUG.	2430	190,655	726,345
SEPT.	2429	179,067	737,933
OCT.	2428	172,055	744,945
NOV.	2429	182,762	734,238
DEC.	2430	191,349	725,651
TOTAL AVG'S	2435	209,110	707,890

* DATA FROM THE FIRST DAY OF EACH MONTH UNDER RECORD, IS USED FOR LIFETIME AVERAGE.



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GILA RIVER HYDROLOGY ANALYSIS

SAN CARLOS RESERVOIR VOLUME RECORDINGS

DESIGNED	CHECKED	DRAWN
EJS	SEW	BH
DATE	DRAWING NO.	
5-10-89		
SCALE	SHEET NO.	

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The third line on the graph is the average water surface elevation of the reservoir. This was obtained by averaging the measured height of the reservoir on the first day of 624 recorded months.

It can be seen that the average elevation line provides more than enough remaining capacity to totally absorb an entire 100-year event.

B. MODELING ALTERNATIVES

There are three possible ways to logically evaluate the effect of a major flood event as it travels through the San Carlos Reservoir.

1. Assume the Dam presents no net effect on the maximum Q values. (i.e. Dam does not exist)

2. ~~The Dam does exist, but is filled to the spillway crest as a flood approaches.~~

3. The Dam exists and is maintaining a water surface elevation equal to the historic average.

Method #1 is what the current Flood Insurance Studies have used for Q-100 and Q-500.

Method #2 is a very conservative approach considering the reservoir has reached or exceeded this elevation only a few times in its 60 year existence. This method would allow a simple reservoir routing procedure to be run. Because the reservoir is so large, the peak discharge would likely be substantially reduced as the wave travels across the lake.

Method #3 would allow a 100-yr, and possibly a 500-yr flood event to be completely absorbed by the reservoir with only an insignificant amount of spillage over the spillway. Method #3 would technically be the most correct. The reason lies with mathematics. Statistically speaking, any event that had a one percent chance of happening, which approaches an obstacle that has an 80 to 90 percent chance of reducing the effect of that event to zero, will not maintain the same outcome at those odds.

It can be seen how, in 1980, Method #1 could have been viewed as a conservative acceptable alternative because the reservoir had never seen a major flood event. Only approximations could have been used to predict how the Dam would affect such an event. It is now, however, apparent that Method #1 is not a viable, legitimate alternative. It is physically impossible for the peak of a flood to travel through the reservoir and not be significantly affected. Method #2 will produce the highest Q values that could possibly occur with the existing Dam intact.

Since the reservoir has been in somewhat of a wet cycle the last 10 years, Method #3 would probably be difficult, politically, to gain approval. Perhaps a fourth method should be considered which would be a compromise between Method #2 and Method #3.

After considerable effort discussing the above concepts with professional peers, it is the author's recommendations that the reservoir be allowed to be full when a 100-year event such as that experienced in 1983 enters the mouth of the reservoir. In FEMA's "Guidelines and Specifications for Study Contractors", this is the suggested approach for reservoirs having no dedicated flood storage capacity. This report will show that the adjustment from Method #1 to Method #2, even remaining conservative, makes a considerable difference in the outcome.

C. RESERVOIR ROUTING

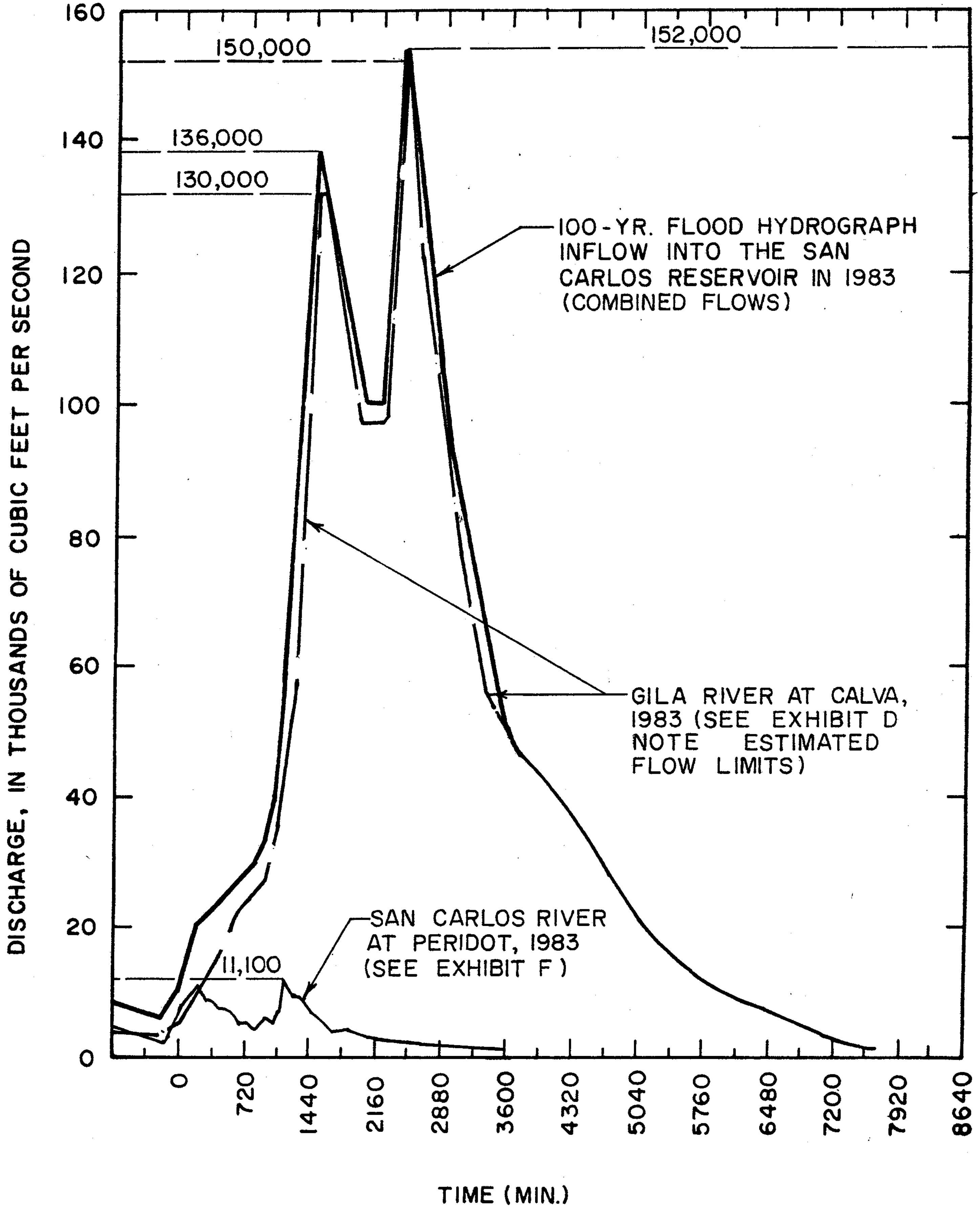
Computer programs used in this section were devised by software manufacturers, CAW, Inc., entitled "CIVILTOOLS", available commercially. The Reservoir Routing program requires four sets of data; the inflow hydrograph, the area capacity of the reservoir, the outflow rate, and the time increment used in routing the storm.

There are basically two tributaries that flow into the San Carlos Reservoir. The largest is the Gila River through Calva gauging station. The other is the San Carlos River flowing through Peridot gauging station. The reservoir inflow hydrograph is constructed by combining the two hydrographs developed at these locations following the 1983 flood. These gauging station hydrographs were prepared by the U.S.G.S.and are included as Exhibit D and Exhibit F. The graph on the following page shows these hydrographs superimposed. The resultant hydrograph reflects a peak of 152,000 cfs. The original design 100-yr peak discharge was 120,000 cfs. So a higher peak Q will be modeled.

The area capacity tables for the reservoir were gathered from the Bureau of Reclamation, the San Carlos Irrigation District and the Bureau of Indian Affairs. All the figures varied slightly based on sedimentation deposit estimates. Where this routing model deals with the upper elevations of the reservoir, sedimentation does not play a factor. The table on the subsequent page provides the area capacity data of the reservoir. The Spillway crest at elevation 2511.5 is established to have 917,000 acre-feet of storage capacity. The figures needed to be converted from acre-feet into cubic feet for the computer program.

The outflow rate is established beginning with 4000 cfs which is the flow capacity through the penstocks, typically open to satisfy irrigation needs. The spillway capacity is then calculated. Weir equations are used with an opening coefficient of 4.03 to match the spillway crest shape. The total spillway width is 300 feet. The Bureau of Reclamation is currently designing safety improvements to the Coolidge Dam. The proposed

FLOOD HYDROGRAPHS



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SAN CARLOS RESERVOIR

AREA CAPACITY TABLE

	WATER SURFACE ELEV (Ft.)	STORAGE CAPACITY (ACRE - FEET)	STORAGE (ADJUSTED -14,000 Ac. Ft. FOR SILTING)	REMAINING FLOOD AVAILABLE STORAGE UP TO CREST	AVAILABLE FLOOD STORAGE ON SEPT. 28, 1983
	2383	0	0	917,000	0
	2400	22,100	8,100	908,900	0
	2420	79,900	65,900	851,100	0
	2440	180,000	166,000	751,000	0
	2460	328,000	314,000	603,000	0
	2480	536,500	522,500	394,500	0
	2486.5	533,000	519,000	398,000	0
	2500	770,000	756,000	161,000	237,000
	2510	905,000	891,000	26,000	372,000
SPILLWAY CREST	2511.5	931,000	917,000	0	398,000
	2512	939,000	925,000	+ 8,000	406,000
	2514	976,000	962,000	+ 45,000	443,000
	2516	1,010,000	996,000	+ 79,000	477,000
	2519	1,070,000	1,056,000	+ 139,000	537,000
AUTHORIZED W.S.E.	2523	1,147,000	1,133,000	+ 216,000	614,000
TOP OF DAM	2535	1,384,000	1,370,000	+ 453,000	851,000

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improvements include modifications to the spillway chutes and extensive abutment strengthening. There are no plans to widen the spillway width or provide increased outflow capacity. The following chart represents the outflow capacity at the various heights of head for the spillway.

SPILLWAYS AND WEIRS

Enter up to 10 weirs.
Enter ±Return° only for flowrate and length to end.

SOLUTION FLOWRATE (CFS)	LENGTH (FT)	COEFF (-)	HEAD (FT)	CORRESPONDING ELEV. (FT)
0.00	300.0	3.400	0.00	2511.5
360.62	300.0	3.400	0.50	2512.0
1873.86	300.0	3.400	1.50	2513.0
4031.90	300.0	3.400	2.50	2514.0
6678.86	300.0	3.400	3.50	2515.0
9736.86	300.0	3.400	4.50	2516.0
%16903.25	300.0	3.400	6.50	2518.0
%20950.39	300.0	3.400	7.50	2519.0
%39778.37	300.0	3.400	11.50	2523.0
%116198.89	300.0	3.400	23.50	2535.0

Note: Add 4000 cfs for Penstock Capacity.

±Shift° ±Prt Sc° print ±Return° repeat ±Space Bar°

These two outflows combined provide the outflow verses elevation curve.

The fourth input item required by the Reservoir Routing program is "time increment." This establishes the interval of time for output results. This is arbitrarily selected at two and one half hours. Since the flood event extends over 5 days, this will yield 50 output flow values.

Prior to routing the 100-year flood through the reservoir filled. The model's input data was first calibrated using the 1983 Flood Event. The area/capacity of the reservoir is set to reflect the September 28th water surface elevation of 2486.5. The U.S.G.S. technical report (Ref. 3) includes the Calva inflow hydrograph extending from September 30 to October 5. (See Exhibit D) This is used as the input data. The report indicates that at 1330 hours on October 4th, water began to flow over the spillway and reached a peak of 5,020 cfs on October 6th. The data sheets on the following pages show the programs input data along with the results. In this program 1330 hours on October 4 is very nearly 4350 minutes on our scale. It can be seen that the outflow exceeds that of the penstock and begins to increase at this time. The output reflects a maximum outflow is experienced at 6150 minutes. This coincides with the evening of October 5th. The outflow Q can be seen as 8609 cfs. If the penstock outflow is subtracted, the model is within a few hundred cfs of the actual outflow. This is very reasonably accurate.

24 hrs. diff.

Though reservoir routing is not an extremely complicated process, the above procedure was performed so that the reviewing agencies can gain confidence in the computer software. "Civil Tools" is a widely used civil engineering program. The above exercise certifies that it's application to our specific project is valid.

1983 FLOOD MODEL

INPUT

R E S E R V O I R   R O U T I N G

INFLOW HYDROGRAPH		CAPACITY/OUTFLOW RATING		
TIME-min	FLOW-cfs	STAGE-ft	VOLUME-cf	OUTFLOW-cfs
0	5770	0	0	4000
920	32580	13.5	10323720000	4002
1440	136010	23.5	16204320000	4004
1900	99000	25	17336880000	4006
2160	99000	25.5	17685360000	4360
2360	152000	27.5	19297080000	8032
3400	56000	29.5	20778120000	13736
5020	23000	32.5	23391720000	24950
5900	10000	36.5	26745840000	43778
7600	2100	48.5	37069560000	120199

TIME INCREMENT (min) = 150

TIME (MIN)	INFLOW (CFS)	STAGE (FT)	VOLUME (CF)	OUTFLOW (CFS)	
7------					------7.3.09
6150.0		27.7	% 19448118D+03	8613.7<	MAXIMUM VALUES

<Shift> <Prt Sc> print   <P> hydrograph   <Ret> repeat   <Space> back tc

1983 FLOOD MODEL
OUTPUT HYDROGRAPH

TIME (MIN)	INFLOW (CFS)	STAGE (FT)	VOLUME (CF)	OUTFLOW (CFS)	
0.0	5770.0	0.0	0	4000.0	
150.0	%10141.2	0.0	%35.600.349	4000.0	
300.0	%14512.4	0.1	%110.541.363	4000.0	
450.0	%18883.6	0.3	%224.822.973	4000.0	
600.0	%23254.8	0.5	%378.445.110	4000.1	
750.0	%27626.0	0.7	%571.407.706	4000.1	
900.0	%31997.2	1.1	%803.710.692	4000.2	
1050.0	%58437.5	1.5	%1.174.665.000	4000.2	
1200.0	%88273.1	2.4	%1.798.860.004	4000.3	
1350.0	%118108.7	3.5	%2.691.573.878	4000.5	
1500.0	%131182.6	4.9	%3.777.378.920	4000.7	
1650.0	%119114.1	6.4	%4.867.706.709	4000.9	
1800.0	%107045.7	7.6	%5.849.416.388	4001.1	
1950.0	%99000.0	8.8	%6.740.610.847	4001.3	
2100.0	%99000.0	9.9	%7.595.598.349	4001.5	
2250.0	%122850.0	11.2	%8.557.909.267	4001.7	
2400.0	%148307.7	12.7	%9.742.102.928	4001.9	
2550.0	%134461.5	14.6	% 10978546D+03	4002.2	
2700.0	%120615.4	16.5	% 12090370D+03	4002.6	
2850.0	%106769.2	18.2	% 13077576D+03	4002.9	
3000.0	%92923.1	19.6	% 13940164D+03	4003.2	
3150.0	%79076.9	20.9	% 14678134D+03	4003.5	
3300.0	%65230.8	21.9	% 15291486D+03	4003.7	
3450.0	%54981.5	22.8	% 15796407D+03	4003.9	
3600.0	%51925.9	23.5	% 16241455D+03	4004.1	
3750.0	%48870.4	24.1	% 16658998D+03	4004.8	
3900.0	%45814.8	24.6	% 17049035D+03	4005.5	
4050.0	%42759.3	25.1	% 17411227D+03	4081.5	
4200.0	%39703.7	25.6	% 17743725D+03	4493.0	
4350.0	%36648.1	25.9	% 18043795D+03	5176.6	
4500.0	%33592.6	26.3	% 18310554D+03	5784.4	
4650.0	%30537.0	26.6	% 18544678D+03	6317.8	
4800.0	%27481.5	26.8	% 18746828D+03	6778.4	
4950.0	%24425.9	27.0	% 18917655D+03	7167.6	
5100.0	%21818.2	27.2	% 19059788D+03	7491.4	
5250.0	%19602.3	27.4	% 19177551D+03	7759.7	
5400.0	%17386.4	27.5	% 19273182D+03	7977.6	
5550.0	%15170.5	27.6	% 19346783D+03	8223.4	
5700.0	%12954.5	27.6	% 19398440D+03	8422.4	
5850.0	%10738.6	27.7	% 19428733D+03	8539.0	
6000.0	9535.3	27.7	% 19442869D+03	8593.5	
6150.0	8838.2	27.7	% 19448118D+03	8613.7	PEAK
6300.0	8141.2	27.7	% 19447021D+03	8609.5	
6450.0	7444.1	27.7	% 19439794D+03	8581.6	
6600.0	6747.1	27.7	% 19426648D+03	8531.0	
6750.0	6050.0	27.6	% 19407782D+03	8458.4	
6900.0	5352.9	27.6	% 19383393D+03	8364.4	
7050.0	4655.9	27.6	% 19353668D+03	8249.9	
7200.0	3958.8	27.5	% 19318789D+03	8115.6	
7350.0	3261.8	27.5	% 19278805D+03	7990.4	
7500.0	2564.7	27.4	% 19233575D+03	7887.3	

Now the program is calibrated, the actual Reservoir Routing to determine the new outflow maximum discharge is begun. The inflow that will be used is that of the 1983 flood. The substantiating support for this is that an actual measured event certainly has more credibility than an estimated or predicted flood whose parameters may vary.

Consideration was given to establishing a hydrograph from the Gila River Flood of 1916. While it had a lower peak discharge, the total volume of runoff was larger than the 1983 Flood. Reference 7 provides a narrative that describes this event. It was actually a 2 stage type of flood. The first was the most severe, occurring January 18-20, 1916. It had a peak discharge estimated at 100,000 cfs at the head of Safford Valley and 130,000 cfs at San Carlos. This was, of course, before the Dam was constructed. On January 29th another peak discharge was experienced, estimated at 30,000 cfs. In analyzing the recorded average mean daily flows for the first flood stage, it reflects roughly a 5 day flooding event. This is about one additional day longer than the 1983 flood. All the data figures surrounding this event were estimated. All staff gauges were taken out early in the flood. The second stage of flooding continued to yield heavy volumes. However, this extra flow would be experienced by the reservoir after the spillway outflow had peaked and subsided. A greater spillway discharge peak than the first would not have been possible with only a 30,000 cfs inflow peak. Based on the uncertainty of the accuracy of the 1916 data, as well as the fact that a lot of the volume of the flood was stretched out over many days, thus not effecting a reservoir routing model's peak discharge, it was determined that data obtained from the 1983 Flood is the best representation of a 100-yr event.

The reservoir capacity remains the same for this model, although it is now adjusted to have standing water at the crest of the spillway preceding the flood.

The outflow rating data also remains the same as the calibration model. This is a combination of the spillway capacity plus the penstock capacity of 4000 cfs.

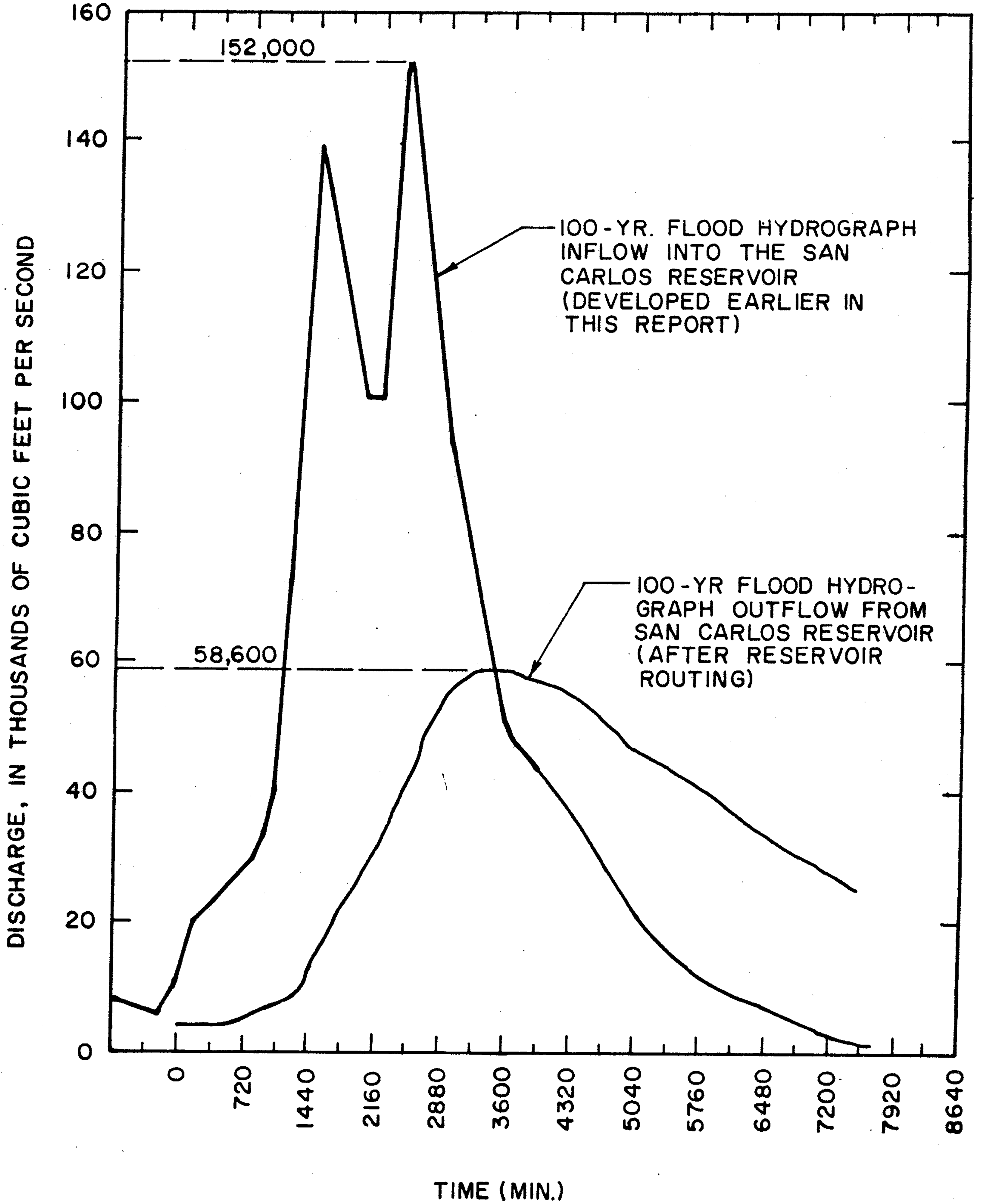
The following pages show the input data and the output results. It can be seen that the extreme size of the reservoir significantly impacts the peak of a flood. The new 100-year Q value at the base of the Dam should be reduced from 120,000 cfs to 58,600 cfs. Another important point to observe is that the damaging high floodwaters are delayed by over a day and a half. The graph on the following page shows these two flood hydrographs and graphically depicts the impact the full reservoir has on a major flood.

D. REVISED PEAK DISCHARGES

In Sub-section 3D of this report, it was discussed how the San Pedro River would likely peak more than a full day before the Gila River would peak. The Flood Hydrograph from the previous section shows how the Dam will delay the Gila River peak an additional 17 hours.

The Pinal County F.I.S. indicates that the San Pedro will contribute 20,000 cfs to the Gila peak. The aforementioned 17 hour delay could reduce this figure by 10,000 cfs. However, the nature and length of a San Pedro flood may vary and could produce an unusually late peaking flood that would contribute 20,000 cfs to the Gila Peak. For this reason, the 20,000 cfs is left as originally dictated.

FLOOD HYDROGRAPHS



SUNRISE ENGINEERING INC.

100-YEAR EVENT

INPUT

R E S E R V O I R   R O U T I N G

INFLOW HYDROGRAPH

TIME-min	FLOW-cfs
0	5770
920	32580
1440	136010
1900	99000
2160	99000
2360	152000
3400	56000
5020	23000
5900	10000
7600	2100

CAPACITY/OUTFLOW RATING

STAGE-ft	VOLUME-cf	OUTFLOW-cfs
0	0	4000
0.5	348480000	4360
2.5	1960200000	8032
4.5	3441240000	13736
7.5	6054840000	24950
11.5	9408960000	43778
23.5	19732680000	120199

TIME INCREMENT (min) = 150

TIME (MIN)	INFLOW (CFS)	STAGE (FT)	VOLUME (CF)	OUTFLOW (CFS)	
7					207.8.23
3450.0		13.8	% 11415965D+03	%58634.8	MAXIMUM VALUES

<Shift> <Prt Sc> print <P> hvdrooraph <Ret> reoeat <Soace> back tc

100-YEAR EVENT

OUTPUT HYDROGRAPH

TIME (MIN)	INFLOW (CFS)	STAGE (FT)	VOLUME (CF)	OUTFLOW (CFS)	
0.0	5770.0	0.0	0	4000.0	
150.0	%10141.2	0.1	%35.435.649	4036.6	
300.0	%14512.4	0.2	%109.702.079	4113.3	
450.0	%18883.6	0.3	%222.439.933	4229.8	
600.0	%23254.8	0.5	%373.155.542	4416.2	
750.0	%27626.0	0.8	%560.452.753	4842.9	
900.0	%31997.2	1.0	%782.889.963	5349.7	
1050.0	%58437.5	1.5	%1.138.057.188	6158.9	
1200.0	%88273.1	2.2	%1.736.687.265	7522.8	
1350.0	%118108.7	3.3	%2.584.587.299	%10436.7	
1500.0	%131182.6	4.7	%3.594.658.502	%14394.3	
1650.0	%119114.1	5.8	%4.572.564.207	%18590.1	
1800.0	%107045.7	6.8	%5.406.863.817	%22169.8	
1950.0	%99000.0	7.6	%6.120.374.851	%25317.9	
2100.0	%99000.0	8.3	%6.767.175.627	%28948.6	
2250.0	%122850.0	9.2	%7.486.785.492	%32988.1	
2400.0	%148307.7	10.3	%8.387.353.823	%38043.3	
2550.0	%134461.5	11.4	%9.294.510.444	%43135.5	
2700.0	%120615.4	12.2	% 10030540D+03	%48379.2	
2850.0	%106769.2	12.9	% 10599408D+03	%52590.3	
3000.0	%92923.1	13.4	% 11011001D+03	%55637.1	
3150.0	%79076.9	13.7	% 11275458D+03	%57594.7	
3300.0	%65230.8	13.8	% 11402266D+03	%58533.4	
3450.0	%54981.5	13.8	% 11415965D+03	%58634.8	PEAK
3600.0	%51925.9	13.8	% 11370838D+03	%58300.7	
3750.0	%48870.4	13.7	% 11302008D+03	%57791.2	
3900.0	%45814.8	13.6	% 11211001D+03	%57117.6	
4050.0	%42759.3	13.5	% 11099249D+03	%56290.3	
4200.0	%39703.7	13.3	% 10968089D+03	%55319.4	
4350.0	%36648.1	13.1	% 10818772D+03	%54214.1	
4500.0	%33592.6	12.9	% 10652468D+03	%52983.0	
4650.0	%30537.0	12.7	% 10470273D+03	%51634.3	
4800.0	%27481.5	12.5	% 10273212D+03	%50175.6	
4950.0	%24425.9	12.3	% 10062242D+03	%48613.9	
5100.0	%21818.2	12.0	%9.840.211.817	%46970.3	
5250.0	%19602.3	11.7	%9.611.489.894	%45277.2	
5400.0	%17386.4	11.5	%9.377.973.004	%43604.1	
5550.0	%15170.5	11.2	%9.138.101.389	%42257.6	
5700.0	%12954.5	10.9	%8.890.597.825	%40868.2	
5850.0	%10738.6	10.6	%8.635.838.381	%39438.2	
6000.0	9535.3	10.3	%8.378.624.885	%37994.3	
6150.0	8838.2	10.0	%8.125.744.692	%36574.8	
6300.0	8141.2	9.7	%7.879.206.418	%35190.9	
6450.0	7444.1	9.4	%7.638.697.560	%33840.8	
6600.0	6747.1	9.1	%7.403.921.013	%32522.9	
6750.0	6050.0	8.8	%7.174.594.314	%31235.6	
6900.0	5352.9	8.6	%6.950.448.916	%29977.4	
7050.0	4655.9	8.3	%6.731.229.507	%28746.8	
7200.0	3958.8	8.1	%6.516.693.353	%27542.6	
7350.0	3261.8	7.8	%6.306.609.685	%26363.3	
7500.0	2564.7	7.6	%6.100.759.101	%25207.8	

Tracking the peak flow, then, there is a 100-yr discharge from the dam of 58,600 cfs. It remains relatively constant down to the San Pedro convergence. There it is increased by 20,000 cfs. The F.I.S. (Ref. 2) indicates that the peak is to remain constant until it reaches Kelvin/Riverside. (The 1983 Flood lost 25,000 cfs in this stretch.) At this point it begins reducing until the flows at Florence have lost 20,000 cfs off the peak. Earlier in Sub-section 3D, it was discussed how the actual experienced 1983 peak decreased by 39,000 cfs between Kelvin and Florence. Precipitation data was presented to support the fact that the lower drainage area did substantially contribute to the flooding.

It is the opinion of this report that with the flows indicated herein, from Kearny to Florence, a straight declining regression of 30,000 cfs is a good representation of what actually will occur during a 100-year event. The one possibility that this does not account for is a bizarre storm dropping excessive rainfall along the river during a flood event between Kelvin and Florence. Once again, the author of this report elects to take the conservative stand and stay with the prescribed losses of only 20,000, cfs. Using this figure then, the design 100-year peak discharge at Florence, Arizona should be revised to be 58,600 cfs.

In establishing this new 100-yr peak discharge, then, reservoir routing was the only variation from the original F.I.S and U.S.G.S. report. All other inflows and regression variables have been used as originally specified.

The 10-year design peak discharge at Florence was originally handled appropriately, being a product of flooding caused below the dam. Therefore, this Q is accurate in the F.I.S.

The 50 and 500-year peak discharges are calculated using the same techniques as the 100-yr. The 50-year and 500-year reservoir routing input and output are on the subsequent pages. The input was determined by using the 1983 Flood hydrograph at Calva as a unit hydrograph. It was adjusted in height to match the F.I.S. peak discharges. The duration was kept the same for all three floods. The swale in the middle of the hydrograph was removed for the 500-year flood to pose a worst case for that severe of a flood.

The results of these hydrographs were then added to the San Pedro flow of 20,000 cfs and then diminished by 20,000 cfs as described above to produce the design flows at Florence, Arizona. This is consistent with the Pinal County F.I.S.

The following chart shows the established new discharges. This should replace the "Table 1, Summary of Discharges", in the Florence F.I.S. (Ref. 1).

TABLE 1. SUMMARY OF DISCHARGES

Flooding Source and Location	Peak Discharges (Cubic Feet per Second) 10-year	50-year	100-year	500-year
Gila River at Florence	19,000	25,000	58,600	110,000

50-YR EVENT

R E S E R V O I R   R O U T I N G

INFLOW HYDROGRAPH		CAPACITY/OUTFLOW RATING		
TIME-min	FLOW-cfs	STAGE-ft	VOLUME-cf	OUTFLOW-cfs
0	2900	0	0	4000
920	16300	0.5	348480000	4360
1440	68000	2.5	1960200000	8032
1900	49500	4.5	3441240000	13736
2360	76000	7.5	6054840000	24950
3400	28000	11.5	9408960000	43778
5020	11500	23.5	19732680000	120199
5900	5000			
7600	1050			

TIME INCREMENT (min) = 150

TIME (MIN)	INFLOW (CFS)	STAGE (FT)	VOLUME (CF)	OUTFLOW (CFS)
7				-218.5.0
3750.0		7.5	%6.060.720.567	24983.0MAXIMUM VALUES

<Shift> <Prt Sc> print <P> hydrograph <Ret> repeat <Space> back t

500-YR EVENT

R E S E R V O I R   R O U T I N G

INFLOW HYDROGRAPH		CAPACITY/OUTFLOW RATING		
TIME-min	FLOW-cfs	STAGE-ft	VOLUME-cf	OUTFLOW-cfs
0	8940	0	0	4000
920	50500	0.5	348480000	4360
1440	210800	2.5	1960200000	8032
2360	235600	4.5	3441240000	13736
3400	86800	7.5	6054840000	24950
5020	35650	11.5	9408960000	43778
5900	15500	23.5	19732680000	120199
7600	3250			

TIME INCREMENT (min) = 150

TIME (MIN)	INFLOW (CFS)	STAGE (FT)	VOLUME (CF)	OUTFLOW (CFS)
7				-894.0.840
3300.0		21.9	% 18358133D+03	110024.0AXIMUM VALUES

<Shift> <Prt Sc> print <P> hydrograph <Ret> repeat <Space> back tc

SECTION 5

CONCLUSIONS

The purpose of this study has been to perform a professional analysis of the original Flood Insurance Study to verify that no errors were made nor unsubstantiated design criteria used. Three basic areas were researched: Alterations, hydraulics and hydrology.

It has been concluded that no significant channel modifications have taken place. No control structures have been constructed, nor major topographic alterations developed.

Extensive review has uncovered no apparent errors in the hydraulic modeling of the design floods through the use of the Corps of Engineers' HEC-2 Computer Program. Elevations and reference benchmarks have also shown to be accurate.

There was actually no original hydrologic work done by the F.I.S. contract consultant. The peak discharge values were obtained from a technical report entitled "Methods for Estimating the Magnitude and Frequency of Floods in Arizona." That report was prepared by Mr. R. H. Roeske of the Water Resources Division of the U.S. Geological Survey. It provided regression equations that could be used in various regions throughout the State. It more specifically included a graph of projected river flows on the Gila River. This U.S.G.S. Report documented well the projected flows extending clear from the New Mexico border. Basically all the storm runoff values appear reasonably accurate. However, there was one design criteria used that has made a profound discrepancy between theoretical and actual. The author viewed the 100-year and greater floods to be such major events that the Coolidge Dam would have little or no impact on the peak discharge. The U.S.G.S. report indicates that during a 100-yr event, 120,000 cfs should enter the reservoir and 120,000 cfs should come over the spillway and continue down the river. In 1983, an actual 100-yr flood was experienced. That flood provided actual data that disputes the assumptions made in the U.S.G.S. report. Not only did the Dam reduce the peak flows, it contained nearly the entire flood. In 1983, the peak inflow to the reservoir was 152,000 cfs. The resultant outflow over the spillway was 5020 cfs. To handle projected flood flows properly, they must be calculated using reservoir routing techniques.

Documentation has been submitted herein which verifies that the Dam does affect peak flood discharges - even when the reservoir is full. The author of this report has been liberal in allowing several "worst case" scenarios to occur in establishing the revised design flood flows.

In October of 1983, 72 percent of the drainage area above Florence yielded a 100-yr flood. Twenty-four percent of its drainage area yielded an unprecedented volume far in excess of a previously defined 600-yr flood. The remaining 4 percent is undocumented, but received heavy rains at the time. The Dam played a major role in controlling the flooding. The 61,000 cfs experienced at Florence came principally from the San Pedro River. Even if the reservoir would have been full, the flooding would not have been worse. Florence would have only experienced a second peak that would have occurred 2 days later, equivalent to 58,600 cfs.

The peak discharge from a 100-yr event should still come from the Gila River, through the Dam. The new discharges, however, should give credit to the Dam's ability to buffer the flood peak. The Dam was there in 1983. It is projected to remain there. Flood Insurance Studies should accept this flood control structure and the benefit it gives to the communities downstream.

The Appendix of this report includes a copy of the Florence, Arizona Flood Insurance Study. The conclusions of this report have been presented to the Florence Town Council. A copy of correspondence from the Town Manager is included in the Appendix.

The findings of this report were also presented to Mr. Rod Roeske of the U.S.G.S. He concurred conceptually that the approach presented in this report is a reasonable approach. He verified that reservoir routing was not performed in his original study. He declined a request to perform a thorough review of this report, stating that he did not have funds allocated is his department to perform such work and deemed the report a private concern and not his problem. He indicated that an inter-governmental agreement would have to be entered into between U.S.G.S. and F.E.M.A. to authorize such a review.

Pinal County and Arizona Department of Water Resources will perform reviews of this report.

SECTION 6

NEW EFFECTIVE WATER SURFACE ELEVATIONS

The new revised peak discharges for Florence, Arizona are substantially different than those presently used in the community's Flood Insurance Study. Therefore, a new hydraulic analysis must be performed to determine the actual water surface profile in the detailed study area.

The original model, with its same cross-section locations will be used with the new discharge values. The Pinal County Maps 040077 0514 and 0514C, along with Florence Town Maps 040084 0002 and 0002B, can be amended with minimal effort because the length of the study area has been kept the same. Only the water surface elevations and corresponding flood boundaries will change.

The original GR grade cards have been adjusted only slightly to reflect actual existing river bottom elevations. Under a separate cover, the HEC-2 modelling input and output results are prepared for FEMA review. A Map, along with a brief narrative is included with that submittal. This data provides the necessary information to complete the Flood Insurance Map Revisions in accordance with Federal Emergency Management Agency's guidelines in "A Guide for Community Officials".

SECTION 7

BIBLIOGRAPHY AND REFERENCES

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2. Federal Emergency Management Agency, Flood Insurance Study, Pinal County, Arizona, Unincorporated Areas, 1987

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4. U.S. Department of the Interior, Geological Survey, Methods for Estimating the Magnitude and Frequency of Floods in Arizona, R.H. Roeske, Sponsored by Arizona Department of Transportation, September 1978

5. U.S. Weather Bureau, Phoenix, Arizona

6. San Carlos Irrigation and Drainage District, Coolidge, Arizona

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8. Brian M. Reich, Consulting Engineer, Flood Frequency Methods for Arizona Streams, State of the Art, Prepared for Arizona Department of Transportation, FHWA-AZ88-801, October 1988

9. U.S. Bureau of Reclamation, Draft Environmental Assessment, Modification of Coolidge Dam, Arizona Projects Office, Phoenix, Arizona, August 1988

APPENDIX

APPENDIX

EXHIBIT A

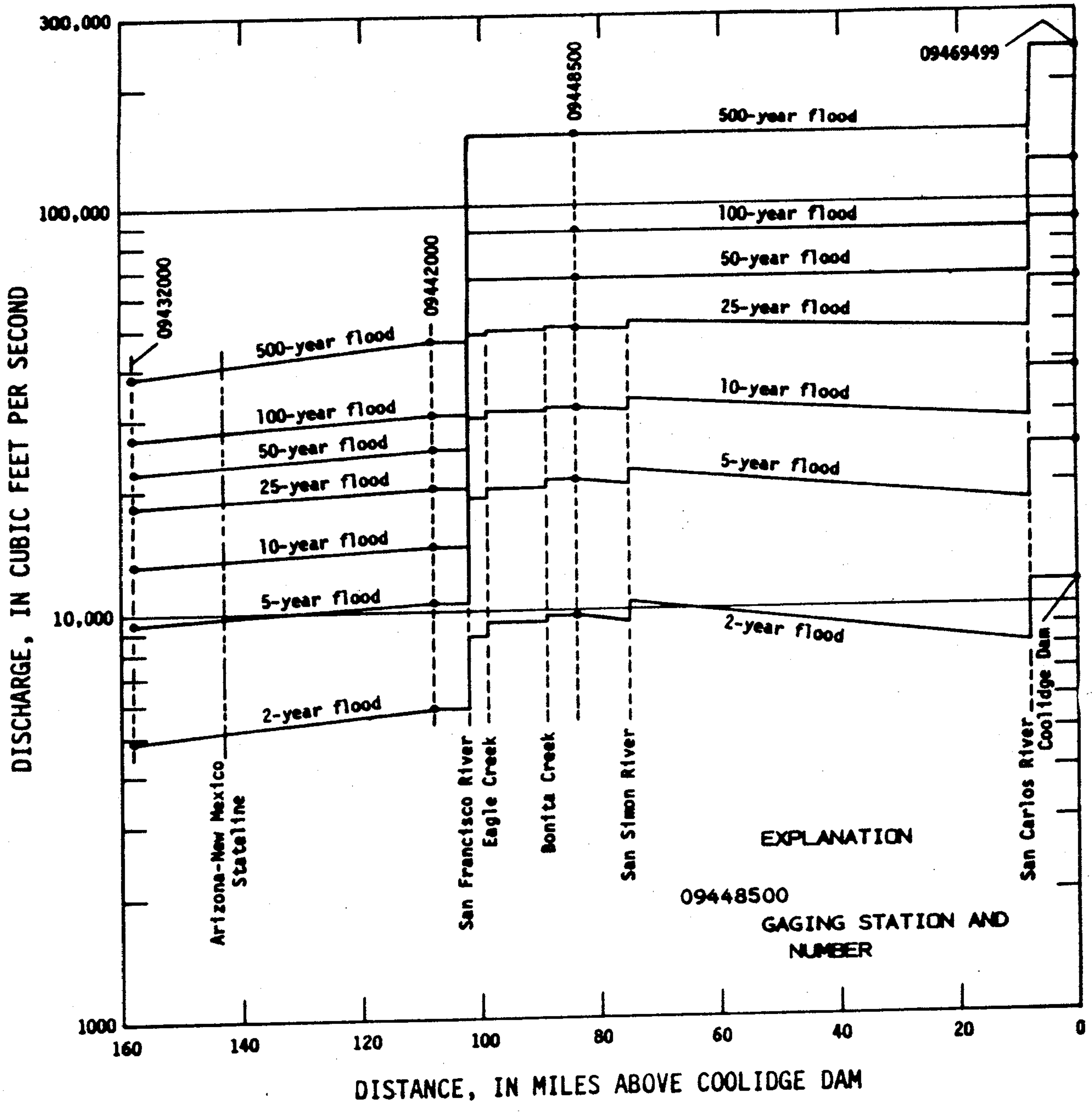


Figure 5.--Relation of discharge to miles above Coolidge Dam, Gila River main stem, for selected recurrence intervals.

EXCERPT FROM REFERENCE 4

EXHIBIT A

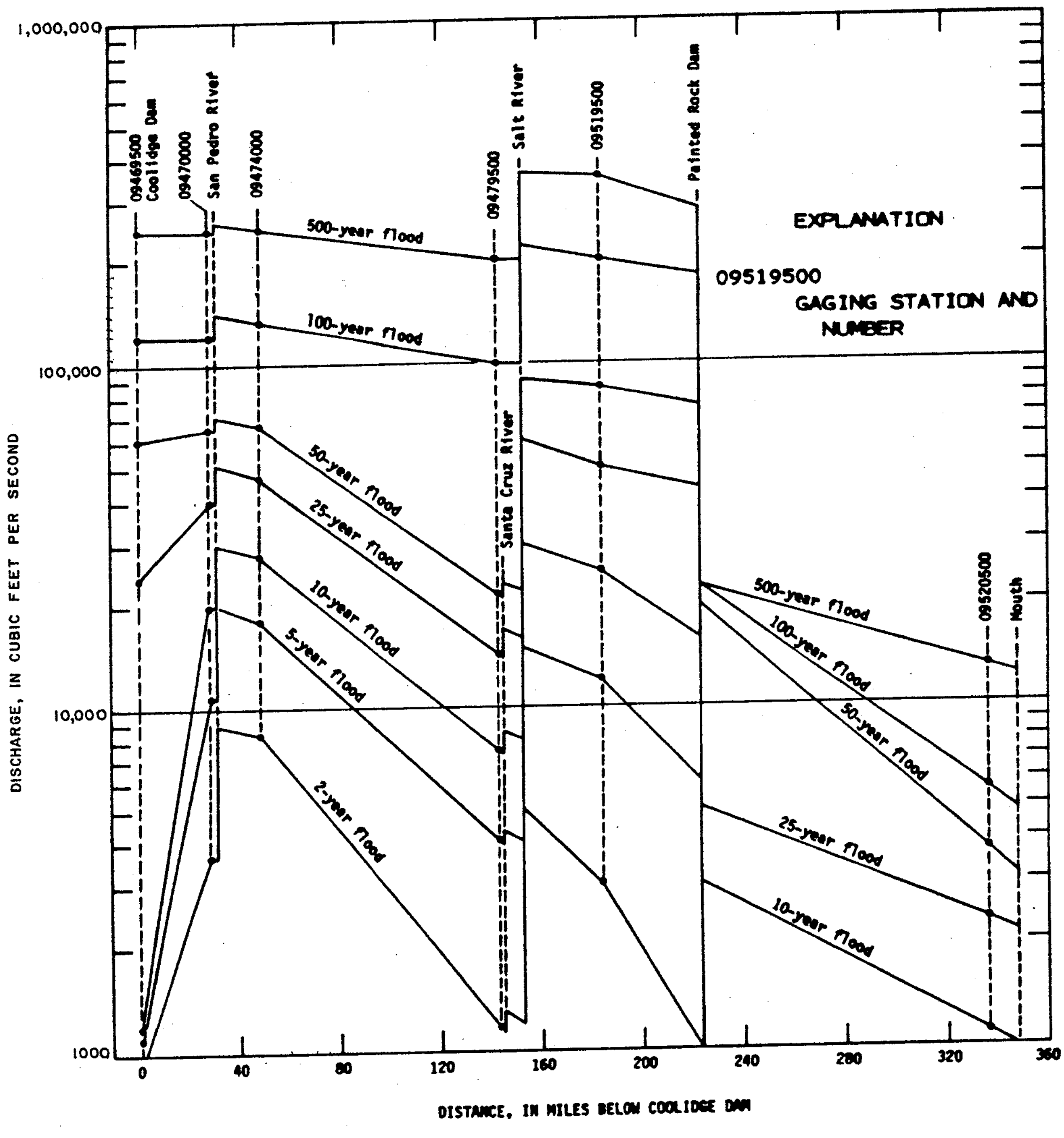


Figure 6.--Relation of discharge to miles below Coolidge Dam, Gila River main stem, for selected recurrence intervals.

EXCERPT FROM REFERENCE 4

EXHIBIT B

EXCERPTS FROM ARCHIVE FILE (3 PAGES)

Cella, Barr, Evans and Associates
5132 North Seventh Street • Phoenix, Arizona 85014 • (602) 274 3596

Ms. Laura Murphy
Dames and Moore
7101 Wisconsin Ave.
Suite 700
Bethesda, Maryland 20014

Dear Laura:

Regarding our telephone conversation of September 26, there are approximately 75 river miles between Florence and Coolidge Dam. Peak discharges mentioned on page 9 of the FIS text may be referenced to:

> Magnitude and Frequency of Floods in the United States, Part 9, Colorado River Basin, Geological Survey Water-Supply Paper 1683, Patterson and Somers.

Enclosed you will also find the original letter sent to me by Byron Aldridge concerning the hydrologic study of the Gila River, and therefore Florence. That letter is referenced to the Arizona Department of Transportation Report # 151210. Applicable excerpts from that report are also enclosed.

Please let me know if further information is required.

Sincerely,

Myron S Tanner

Myron S. Tanner

MST nn
Encl

Reference to Figure 6, Report ADOT-RS-151210

The report provides only a limited explanation of this illustration and fails to point out that the curves represent the peak flows under the existing degree of regulation. The curves could not be derived from any set of published data because long-term records for the Gila River have been collected under varying degrees of regulation. When deriving the curves, it was assumed that (1) low recurrence interval floods occurred entirely as a result of inflow below the reservoirs, (2) intermediate floods resulted from combination of inflow below the reservoirs plus controlled releases from the reservoirs and (3) floods having a recurrence interval of 100 years or greater would result from uncontrolled releases from the reservoirs. In order to derive the curves for these three assumptions, it was necessary to have three complete sets of data to represent (1) unregulated flow, (2) controlled releases, and (3) inflow below the reservoirs. It was particularly important that each set of data include the high runoff years that occurred early in the twentieth century, before the reservoirs were constructed. The existing reservoir systems do not have adequate capacity to completely contain floods similar to those that occurred during the wet years, and frequency curves developed only from post-reservoir data show discharges that are too low. In general, the data sets were reconstructed for the period 1913-1975 or 1917-1975 The moderate to high floods of 1977-79 were not included.

The procedure is illustrated by figure 1 which shows frequency curves for Gila River at Kelvin. Curve A is defined by estimates of peak flows that would have reached Kelvin if San Carlos Reservoir did not exist. Unless the dam failed or releases from the dam greatly exceeded inflow through human error, curve A represents the upper limit of discharge for a specified frequency. Curve B is defined by peaks that originated entirely downstream from Coolidge Dam, and represents the lower limit for discharges. The data set for curve B was relatively easy to compile because the highest peaks originating downstream from the reservoir seldom occur at the same time as those from upstream of the reservoir.

The same data set was used for curve B was used for curve C except that the annual peaks for 1916, 1917 and 1920 were replaced with estimates of peak discharges that would have resulted from controled releases from San Carlos Reservoir if the reservoir had existed at the time. This curve should not be extended beyond the length of record. Curve B represents the curve selected for use.

The curves shown in figure 1 of this discussion and figure 4 of the reference report are based on station skew. WRD skew was not used for these long-term records for main stem drainages.

B. N. Aldridge
2/15/79

EXHIBIT C EXCERPTS FROM ARCHIVE FILE

in addition to the controlled releases from the dams, and (3) large floods (100-year or greater) resulted from uncontrolled releases from the dams. The following quote, taken from Aldridge, 1979, further clarifies the methodology used.

> "In order to derive the curves for these three assumptions, it is necessary to have three complete sets of data to represent (1) unregulated flow, (2) controlled releases, and (3) inflow below the reservoirs. It was particularly important that each set of data include the high runoff years that occurred early in the twentieth century, before the reservoirs were constructed. The existing reservoir systems do not have adequate capacity to completely contain floods similar to those that occurred during the wet years, and frequency curves developed only from post-reservoir data show discharges that are too low. In general, the data sets were reconstructed for the period 1913-1975 or 1917-1975". (Reference 9).

A summary of the peak discharges used in this study are shown in Table 1.

Table 1 - Summary of Discharges

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (CFS) 10-year	50-year	100-year	500-year
Gila River at Florence	18,500	19,000	46,000	120,000	230,000

EXHIBIT D

GRAPH OBTAINED FROM REFERENCE 3

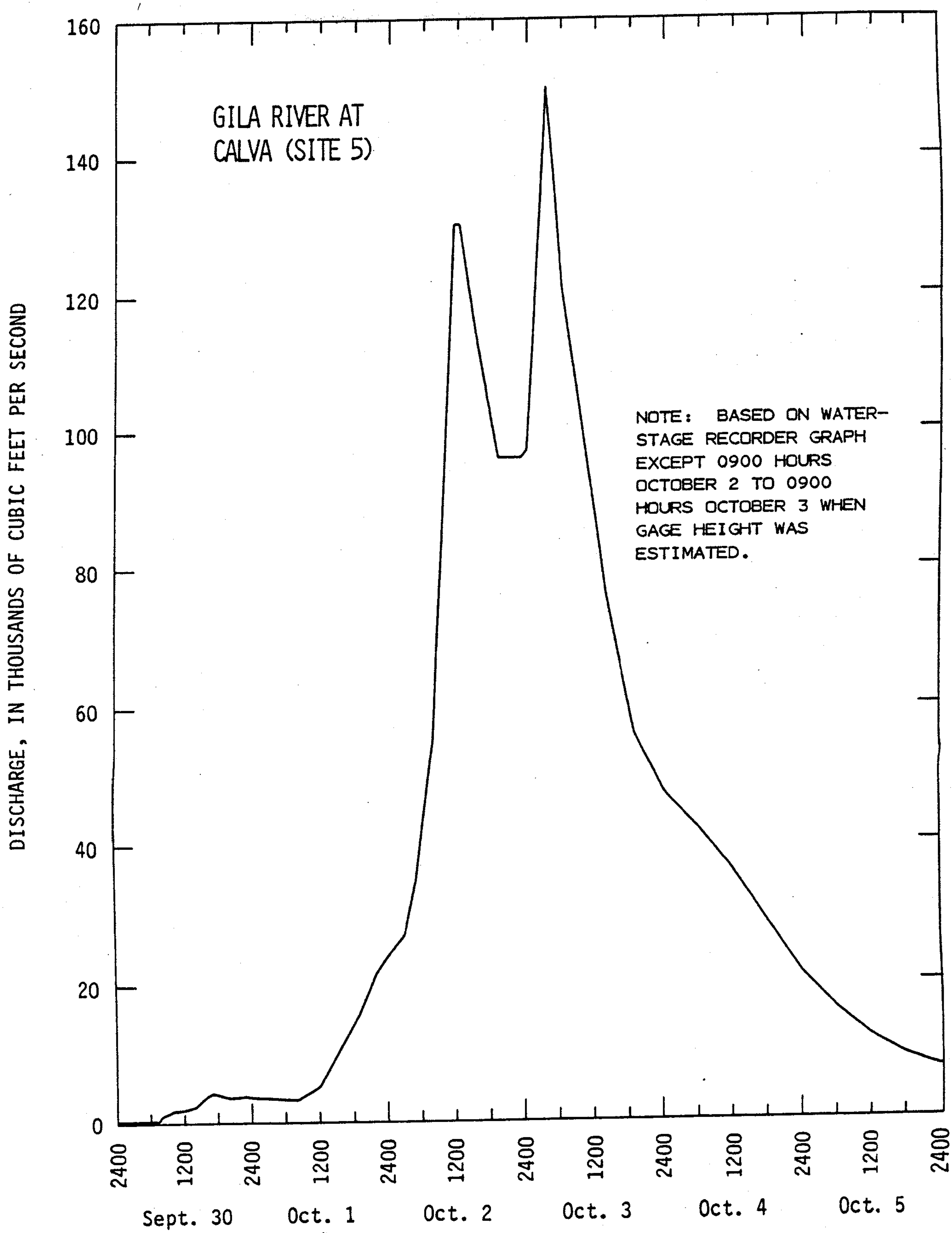
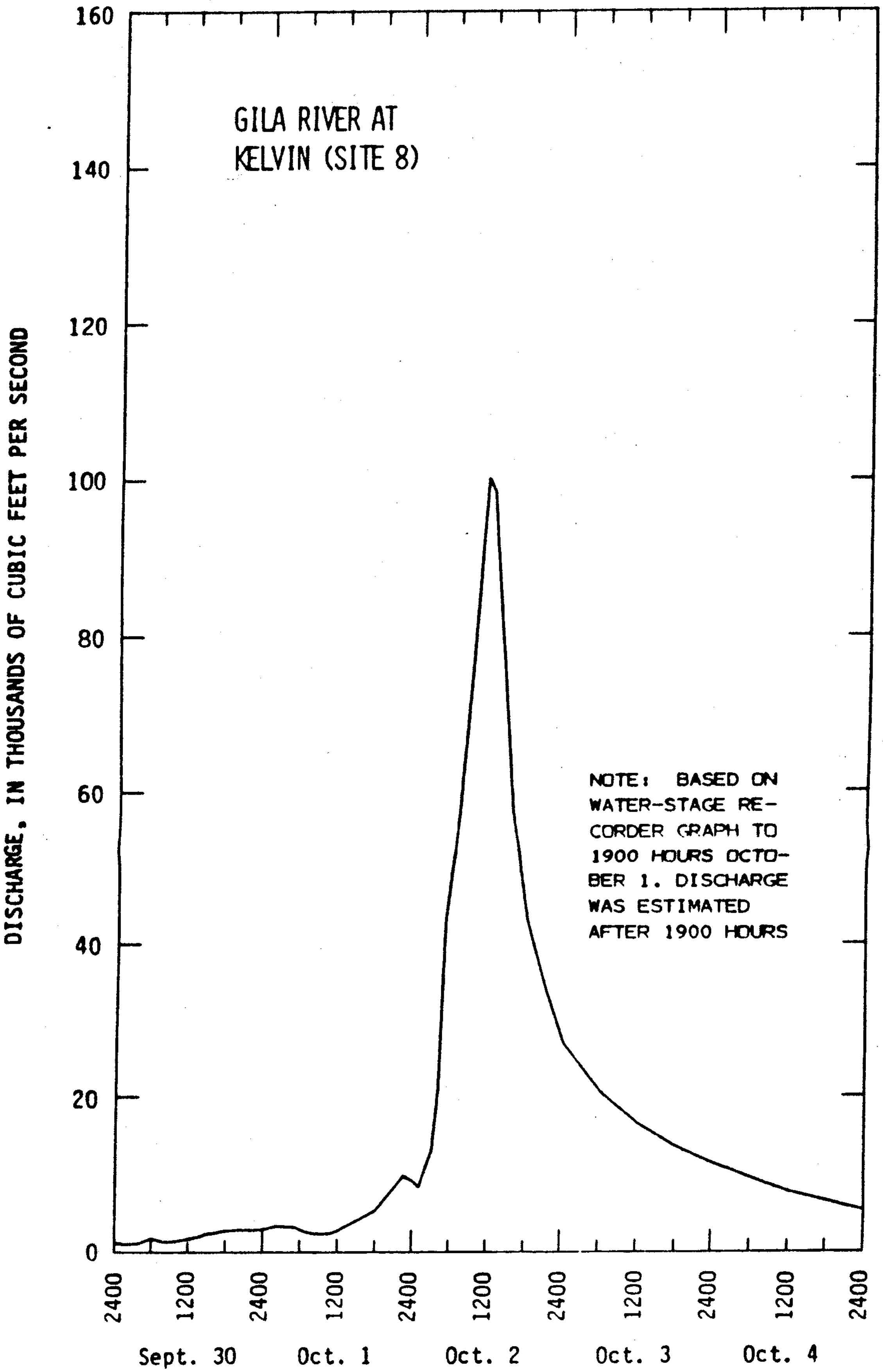


Figure 5.--Discharge hydrographs for flood of Oct

EXHIBIT E

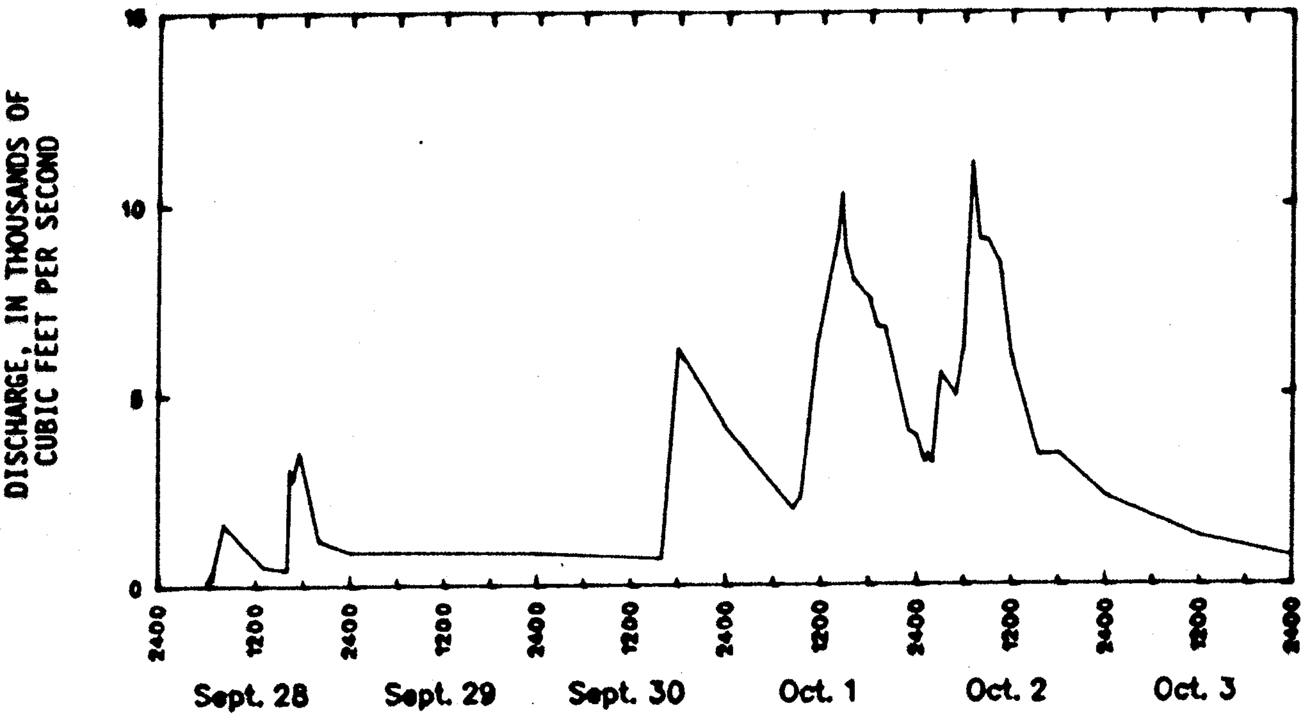


October 1983, at two gaging stations on the Gila River.

GRAPH OBTAINED FROM
REFERENCE 4

EXHIBIT F

(32) 09468500 San Carlos River near Peridot, Ariz.—Continued

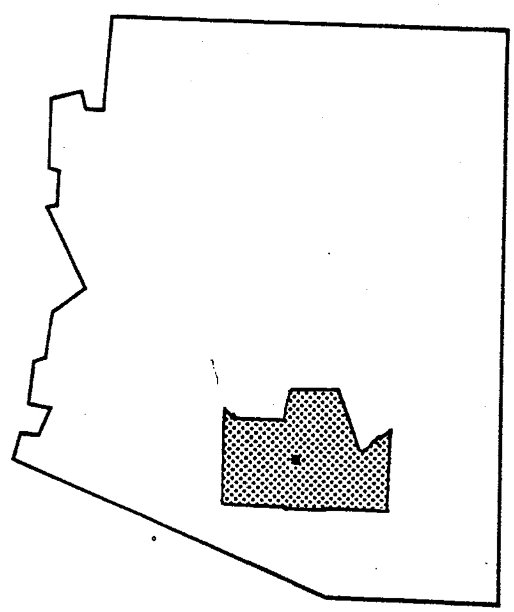


(32) 09468500 San Carlos River near Peridot, Ariz.

09-28	0630	4.35	190	10-01	1800	8.35	7,550
09-28	0800	5.80	1,620	10-01	1900	8.10	6,800
09-28	1300	4.80	497	10-01	2000	8.09	6,770
09-28	1600	4.70	416	10-01	2300	6.95	4,000
09-28	1615	6.56	3,040	10-01	2400	6.89	3,880
09-28	1630	6.42	2,740	10-02	0100	6.60	3,200
09-28	1730	6.80	3,500	10-02	0130	6.70	3,390
09-28	2000	5.44	1,160	10-02	0200	6.58	3,160
09-28	2400	5.18	861	10-02	0300	7.63	5,580
09-29	2400	5.15	830	10-02	0500	7.35	4,920
09-30	1600	5.0	680	10-02	0600	7.90	6,260
09-30	1800	7.9	6,260	10-02	0700	9.45	11,100
09-30	2400	7.0	4,110	10-02	0800	8.85	9,050
10-01	0830	6.03	1,970	10-02	0900	8.85	9,050
10-01	0930	6.21	2,300	10-02	1030	8.65	8,450
10-01	1030	7.10	4,250	10-02	1200	7.80	6,010
10-01	1130	7.95	6,400	10-02	1530	6.65	3,360
10-01	1400	8.88	9,140	10-02	1800	6.69	3,400
10-01	1430	9.24	10,300	10-02	2400	6.20	2,280
10-01	1500	8.75	8,750	10-03	1200	5.55	1,240
10-01	1600	8.50	8,000	10-03	2400	5.05	708

FLOOD INSURANCE STUDY

TOWN OF FLORENCE, ARIZONA
PINAL COUNTY



FEBRUARY 17, 1981

federal emergency management agency
federal insurance administration

COMMUNITY NUMBER - 040084

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PUBLISHED SEPARATELY:

1.0 INTRODUCTION

1.1 Purpose of Study

This Flood Insurance Study investigates the existence and severity of flood hazards in the Town of Florence, Pinal County, Arizona, and aids in the administration of the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973. This study will be used to convert Florence to the regular program of flood insurance by the Federal Insurance Administration. Local and regional planners will use this study in their efforts to promote sound flood plain management.

In some states or communities, flood plain management criteria or regulations may exist that are more restrictive or comprehensive than those on which these federally supported studies are based. These criteria take precedence over the minimum Federal criteria for purposes of regulating development in the flood plain, as set forth in the Code of Federal Regulations at 24 CFR, 1910.1(d). In such cases, however, it shall be understood that the State (or other jurisdictional agency) shall be able to explain these requirements and criteria.

1.2 Authority and Acknowledgments

The source of authority for this Flood Insurance Study is the National Flood Insurance Act of 1968, as amended.

The hydrologic and hydraulic analyses for this study were performed by Cella, Barr, Evans and Associates, for the Federal Insurance Administration, under Contract No. H-4607. This work, which was completed in July 1980, covered all significant flooding sources affecting Florence.

1.3 Coordination

The Gila River detailed study miles were identified at a meeting attended by representatives of the study contractor, the Federal Insurance Administration, and officials of the Town of Florence in August 1977.

Results of the hydrological analysis were coordinated with the U.S. Geological Survey, the U.S. Soil Conservation Service, and the U.S. Army Corps of Engineers. Additional information was obtained from the Pinal County Flood Control Board.

On December 5, 1979, the results of the study were reviewed at the intermediate/final meeting which was attended by representatives of the study contractor, the Federal Insurance Administration, and community officials. The study was acceptable to the community.

2.0 AREA STUDIED

2.1 Scope of Study

This Flood Insurance Study covers the incorporated areas of the Town of Florence, Pinal County, Arizona. The area of study is shown on the Vicinity Map (Figure 1).

The limits of detailed study in Florence were determined by the Federal Insurance Administration with community and study contractor consultation at the meeting in August 1977.

The detailed study area consists of 1.19 miles of Gila River, extending from U.S. Highway 80/89 on the east, to the extreme western corporate limits.

Those areas studied by detailed methods were chosen with consideration given to all proposed construction and forecasted development through 1985.

Small washes southeast of Florence were not studied due to the existence of a U.S. Soil Conservation Service flood control structure in this area. This structure was designed to protect against the 1 percent frequency (100-year) flood (Reference 1).

2.2 Community Description

Florence, the county seat, is centrally located in Pinal County, in south-central Arizona. Unincorporated areas of Pinal County surround Florence. Coolidge is the nearest city, located approximately 8 miles to the west-southwest, while the Town of Superior is approximately 28 miles to the northeast.

Florence is approximately 2.25 square miles in area (Reference 2), with a population of approximately 3200. Gila River, flowing southwesterly, divides Florence into northern and southern areas, with the population concentrated in the southern portion of the community. A thin strip of the town runs across the Gila River flood plain. Very little development has taken place in this strip, and it is mainly used for agricultural purposes.

Gila River at Florence has developed a gently sloping flood plain consisting of alluvium from mixed sources. An average annual rainfall of 6 to 10 inches supports annual grasses, creosotebush, mesquite, paloverde, and cactus as the natural vegetation. Mean

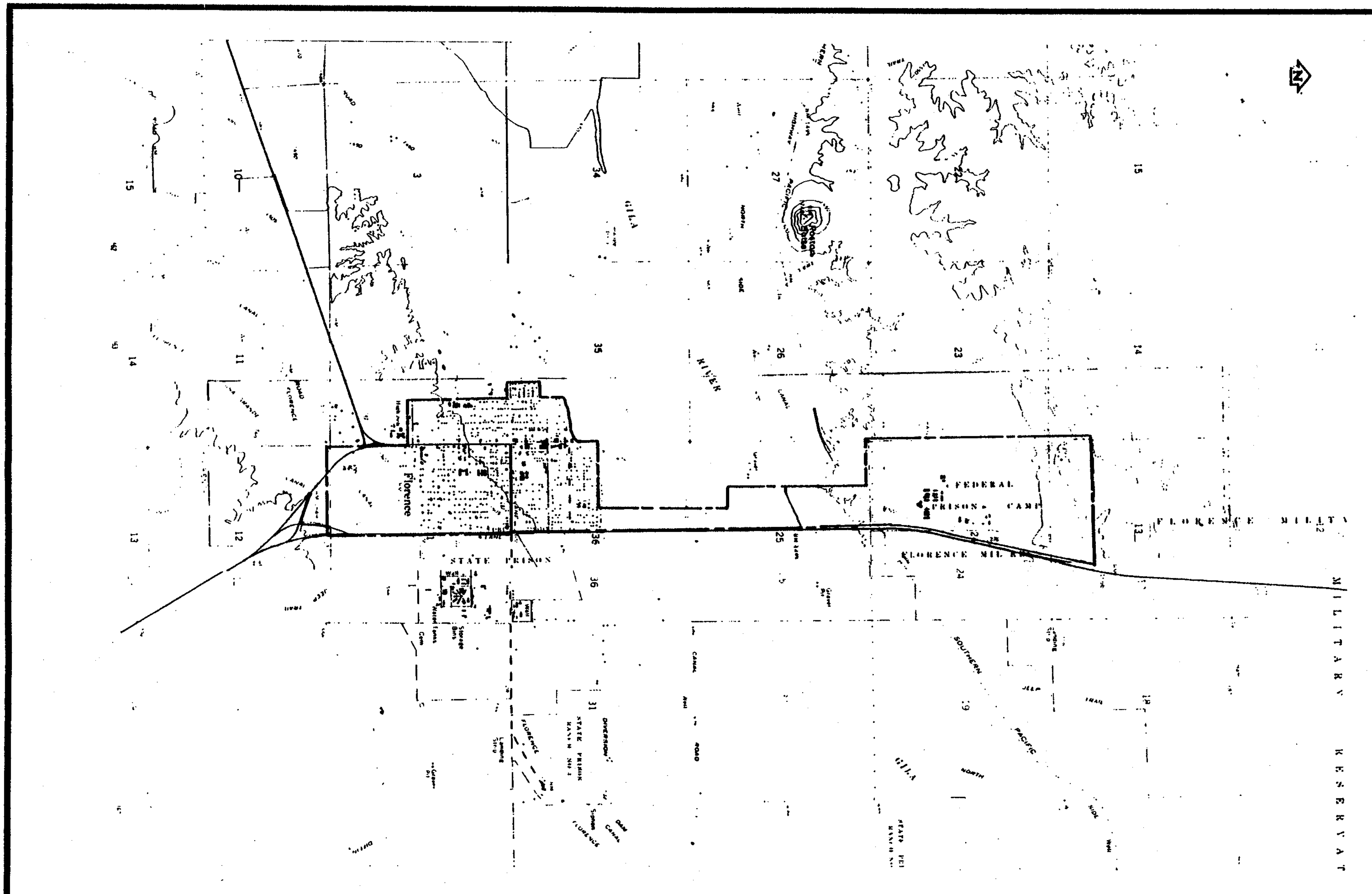

FIGURE 1

FEDERAL EMERGENCY MANAGEMENT AGENCY
Federal Insurance Administration

TOWN OF FLORENCE, AZ
(PINAL CO.)

APPROXIMATE SCALE

2000 0 2000 4000 6000 FEET

VICINITY MAP

annual air temperature is 68°F to 71°F, and the frostfree period is approximately 275 days per year.

The population of Florence was approximately 2200 in 1970 and 3200 in 1978, making the average annual growth rate approximately 4 percent.

2.3 Principal Flood Problems

In 1929, Coolidge Dam was constructed approximately 75 miles upstream of Florence, forming the San Carlos Reservoir. Of the 18,500-square-mile area contributing to the flood hazard at Florence, approximately 12,900 square miles are controlled by Coolidge Dam. Thus, Coolidge Dam plays an important role in the flooding problems of Florence. Assuming the reservoir to be at capacity, there are three types of events which could lead to severe flooding on Gila River: a widespread frontal type storm of large magnitude and long duration, a warm airmass moving in on a large snow accumulation, or a frontal type storm falling on snow (Reference 3).

An examination of Gila River discharge records collected at Kelvin, gage No. 94740, (approximately 25 miles upstream from Florence) show that the annual peak discharge occurs most often during the months of August through January (Reference 4).

The estimated maximum discharge at Kelvin is 190,000 cubic feet per second (cfs) and occurred on November 28, 1905. Based on newspaper accounts describing floods of similar magnitude, Gila River has been known to swell to 1 mile in width, cutting Florence off from communication with other communities and washing out three railroad bridges between Florence and Kelvin. According to the current discharge-frequency relationships, a flood of this magnitude has a chance of occurring at Florence on the average of once every 285 years.

The maximum recorded discharge at Kelvin is 132,000 cfs, which occurred on January 20, 1916. According to the January 22, 1916 edition of the Arizona Blade-Tribune, both the north and south approach to a bridge in the vicinity of the existing U.S. Highway 80/89 bridge was washed away, and the river cut a new channel to the south of the bridge. According to the current discharge-frequency relationships, a flood of this magnitude has a chance of occurring at Florence on the average of once every 120 years.

A more recent flood occurred on December 20, 1967. The peak discharge of this flood was 27,700 cfs, recorded at Kelvin. A flood of this magnitude has a chance of occurring at Florence on the average of once every 21 years.

The bridge on U.S. Highway 80/89 is the only structure in the vicinity of Florence which affects the flow of floodwater. Both the bridge and the south approach ramp to the bridge create a backwater condition.

2.4 Flood Protection Measures

There are no flood protection structures for Gila River at Florence.

As mentioned earlier, a flood retarding structure is located to the southeast of Florence. This structure protects Florence against discharges emanating from the washes in that area.

3.0 ENGINEERING METHODS

For the flooding sources studied in detail in the community, standard hydrologic and hydraulic study methods were used to determine the flood hazard data required for this study. Flood events of a magnitude which are expected to be equalled or exceeded once on the average during any 10-, 50-, 100-, or 500-year period (recurrence interval) have been selected as having special significance for flood plain management and for flood insurance premium rates. These events, commonly termed the 10-, 50-, 100-, and 500-year floods, have a 10, 2, 1, and 0.2 percent chance, respectively, of being equalled 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. For example, the risk of having a flood which equals or exceeds the 100-year flood (1 percent chance of annual occurrence) in any 50-year period is approximately 40 percent (4 in 10), and, for any 90-year period, the risk increases to approximately 60 percent (6 in 10). The analyses reported here reflect flooding potentials based on conditions existing in the community at the time of completion of this study. Maps and flood elevations will be amended periodically to reflect future changes.

3.1 Hydrologic Analyses

Hydrologic analyses were carried out to establish the peak discharge-frequency relationships for floods of the selected recurrence intervals for each flooding source studied in detail affecting the community.

Peak discharge values used in this study were developed by the U.S. Geological Survey using a log-Pearson Type III distribution (Reference 5). Frequency-discharge data were based on records from several U.S. Geological Survey gaging stations on Gila River. The gaged information was adjusted to reflect the regulating effect of Coolidge Dam by assuming low recurrence interval floods to be a result of inflow below the dam, assuming intermediate floods to be the sum of inflow below the dam and controlled releases from the dam, and assuming large floods (100-year or greater) are caused by uncontrolled releases from the dam.

Peak discharge-drainage area relationships for Gila River are shown in Table 1.

3.2 Hydraulic Analyses

Analyses of the hydraulic characteristics of the flooding sources studied in the community were carried out to provide estimates of the elevations of floods of the selected recurrence intervals along each of these flooding sources.

Cross sections for the backwater analysis of Gila River were obtained using topographic maps at a scale of 1:2400, with a contour interval of 2 feet (Reference 6). These topographic maps were developed from aerial photographs flown in January 1979. Flight altitude was 4200 feet, and the aerial photographs were taken at a scale of 1:8400 (Reference 7). Vertical control was adjusted (+1.5 feet) to coincide with the National Geodetic Vertical Datum (NGVD) of 1929.

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 is computed (Section 4.2), selected cross section locations are also shown on the Flood Boundary and Floodway Map (Exhibit 2).

Roughness factors (Manning's "n") used in the hydraulic computations were chosen by engineering judgment, based on field observations of the river and flood plain area. The roughness value used for the main channel of Gila River was 0.030 with an overbank roughness value of 0.035 for all floods.

Water-surface elevations of floods of the selected recurrence intervals were computed through use of the U.S. Army Corps of Engineers HEC-2 step-backwater computer program (Reference 8). Flood profiles were drawn showing computed water-surface elevations for floods of the selected recurrence interevals. The starting water-surface elevation for Gila River was calculated using the slope-area method.

Flood profiles were drawn showing computed water-surface elevations to an accuracy of 0.5 foot for floods of the selected recurrence intervals (Exhibit 1).

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.

All elevations are referenced to the National Geodetic Vertical Datum of 1929 (NGVD). Elevation reference marks used in the study are shown on the maps.

Table 1. Summary of Discharges

Flooding Source and Location	Drainage Area (Square Miles)	Peak Discharges (Cubic Feet per Second) 10-Year	50-Year	100-Year	500-Year
Gila River					
At Florence	18,500	19,000	46,000	120,000	230,000

4.0 FLOOD PLAIN MANAGEMENT APPLICATIONS

The National Flood Insurance Program encourages State and local governments to adopt sound flood plain management programs. Therefore, each Flood Insurance Study includes a flood boundary map designed to assist communities in developing sound flood plain management measures.

4.1 Flood Boundaries

In order to provide a national standard without regional discrimination, the 100-year flood has been adopted by the Federal Insurance Administration as the base flood for purposes of flood plain management measures. The 500-year flood is employed to indicate additional areas of flood risk in the community. For each stream studied in detail, the boundaries of the 100- and 500-year floods have been delineated using the flood elevations determined at each cross section; between cross sections, the boundaries were interpolated using topographic maps at a scale of 1:2400, with a contour interval of 2 feet (Reference 6).

Flood boundaries for the 100- and 500-year floods are shown on the Flood Boundary and Floodway Map (Exhibit 1). In cases where the 100- and 500-year flood boundaries are close together, only the 100-year flood boundary has been shown. Small areas within the flood boundaries may lie above the flood elevations and, therefore, not be subject to flooding; owing to limitations of the map scale, such areas are not shown.

4.2 Floodways

Encroachment on flood plains, such as artificial fill, reduces the flood-carrying capacity, increases the flood heights of streams, and increases flood hazards in areas beyond the encroachment itself. One aspect of flood plain management involves balancing the economic gain from flood plain development against the resulting increase in flood hazard. For purposes of the National Flood Insurance Program, the concept of a floodway is used as a tool to assist local communities in this aspect of flood plain management. Under this concept,the area of the 100-year flood is divided into a floodway and a floodway fringe. The floodway is the channel of a stream plus any adjacent flood plain areas that must be kept free of encroachment in order that the 100-year flood may be carried without substantial increases in flood heights. Minimum standards of the Federal Insurance Administration limit such increases in flood heights to 1.0 foot, provided that hazardous velocities are not produced. The floodway in this report is presented to local agencies as a minimum standard that can be adopted or that can be used as a basis for additional studies.

The floodway presented in this study was computed on the basis of equal-conveyance reduction from each side of the flood plain. The results of these computations were tabulated at selected cross sections for each stream segment for which a floodway was computed (Table 2).

As shown on the Flood Boundary and Floodway Map (Exhibit 2), the floodway widths were determined at cross sections; between cross sections, the boundaries were interpolated. In cases where the boundaries of the floodway and the 100-year flood are either close together or collinear, only the floodway boundary has been shown.

The area between the floodway and the boundary of the 100-year flood is termed the floodway fringe. The floodway fringe thus encompasses the portion of the flood plain that could be completely obstructed without increasing the water-surface elevation of the 100-year flood more than 1.0 foot at any point. Typical relationships between the floodway and the floodway fringe and their significance to flood plain development are shown in Figure 2.

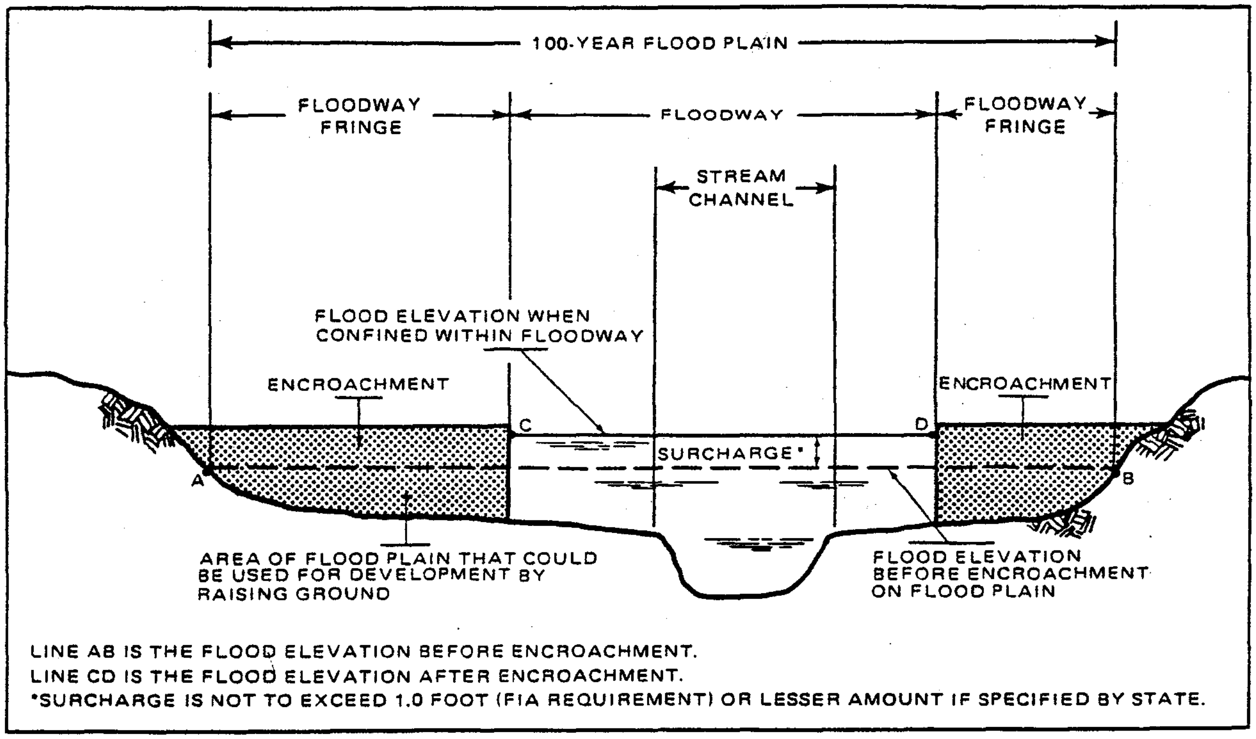

Figure 2. Floodway Schematic

5.0 INSURANCE APPLICATION

In order to establish actuarial insurance rates, the Federal Insurance Administration has developed a process to transform the data from the engineering study into flood insurance criteria. This process includes the determination of reaches, Flood Hazard Factors (FHFs), and flood insurance zone designations for each flooding source studied in detail affecting Florence.

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 (FEET NGVD)	WITH FLOODWAY (FEET NGVD)	INCREASE
Gila River								
A	156.87	3,059[2]	18,872	6.4	1,468.2	1,468.2	1,469.1	0.9
B	156.95	3,230[2]	19,257	6.2	1,469.0	1,469.0	1,469.9	0.9
C	157.01	3,300[2]	19,778	6.1	1,469.6	1,469.6	1,470.3	0.7
D	157.16	3,183[2]	17,867	6.7	1,470.9	1,470.9	1,471.7	0.8
E	157.37	3,352[2]	14,813	8.1	1,473.4	1,473.4	1,474.3	0.9
F	157.58	4,209[2]	22,483	5.3	1,476.5	1,476.5	1,477.4	0.9
G	157.74	4,373/970[3]	25,642	4.7	1,477.7	1,477.7	1,478.5	0.8
H	157.90	4,803	27,762	4.3	1,478.7	1,478.7	1,479.3	0.6
I	157.99	4,964	11,386	10.5	1,480.1	1,480.1	1,480.1	0.0

[1] Miles Above Painted Rock Dam [2] Floodway Lies Entirely Outside Corporate Limits
[3] Width/Width Within Corporate Limits

TABLE

FEDERAL EMERGENCY MANAGEMENT AGENCY
Federal Insurance Administration

TOWN OF FLORENCE, AZ
(PINAL CO.)

FLOODWAY DATA

GILA RIVER

5.1 Reach Determinations

Reaches are defined as lengths of watercourses having relatively the same flood hazard, based on the average weighted difference in water-surface elevations between the 10- and 100-year floods. This difference does not have a variation greater than that indicated in the following table for more than 20 percent of the reach:

Average Difference Between 10- and 100-Year Floods	Variation
Less than 2 feet	0.5 foot
2 to 7 feet	1.0 foot
7.1 to 12 feet	2.0 feet
More than 12 feet	3.0 feet

The locations of the reaches determined for the flooding sources of Florence are shown on the Flood Profiles (Exhibit 1) and summarized in Table 3.

5.2 Flood Hazard Factors

The FHF is the Federal Insurance Administration device used to correlate flood information with insurance rate tables. Correlations between property damage from floods and their FHF are used to set actuarial insurance premium rate tables based on FHFs from 005 to 200.

The FHF for a reach is the average weighted difference between the 10- and 100-year flood water-surface elevations expressed to the nearest one-half foot, and shown as a three-digit code. For example, if the difference between water-surface elevations of the 10- and 100-year floods is 0.7 foot, the FHF is 005; if the difference is 1.4 feet, the FHF is 015; if the difference is 5.0 feet, the FHF is 050. When the difference between the 10- and 100-year water-surface elevations is greater than 10.0 feet, accuracy for the FHF is to the nearest foot.

5.3 Flood Insurance Zones

After the determination of reaches and their respective FHFs, the entire incorporated area of Florence was divided into zones, each having a specific flood potential or hazard. Each zone was assigned one of the following flood insurance zone designations:

Zone A8: Special Flood Hazard Areas inundated by the 100-year flood, determined by detailed methods; base flood elevations shown, and zones subdivided according to FHFs.

FLOODING SOURCE	PANEL[1]	ELEVATION DIFFERENCE[2] BETWEEN 1% (100-YEAR) FLOOD AND			FLOOD HAZARD FACTOR	ZONE	BASE FLOOD ELEVATION (FEET NGVD)[3]
		10% (10-YEAR)	2% (50-YEAR)	0.2% (500-YEAR)			
Gila River Reach 1	0001	-4.1	-2.5	2.2	040	A8	Varies - See Map

[1]Flood Insurance Rate Map Panel [2]Weighted Average [3]Rounded to Nearest Foot

TABLE 3

FEDERAL EMERGENCY MANAGEMENT AGENCY
Federal Insurance Administration

TOWN OF FLORENCE, AZ
(PINAL CO.)

FLOOD INSURANCE ZONE DATA

GILA RIVER

Zone B: Areas between the Special Flood Hazard Areas and the limits of the 500-year flood, including areas of the 500-year flood plain that are protected from the 100-year flood by dike, levee, or other water control structure; also areas subject to certain types of 100-year shallow flooding where depths are less than 1.0 foot; and areas subject to 100-year flooding from sources with drainage areas less than 1 square mile. Zone B is not subdivided.

Zone C: Areas of minimal flooding.

The flood elevation differences, FHFs, flood insurance zones, and base flood elevations for each flooding source studied in detail in the community are summarized in Table 3.

5.4 Flood Insurance Rate Map Description

The Flood Insurance Rate Map for Florence is, for insurance purposes, the principal result of the Flood Insurance Study. This map (published separately) contains the official delineation of flood insurance zones and base flood elevation lines. Base flood elevation lines show the locations of the expected whole-foot water-surface elevations of the base (100-year) flood. This map is developed in accordance with the latest flood insurance map preparation guidelines published by the Federal Insurance Administration.

6.0 OTHER STUDIES

Flood Hazard Boundary Maps have been published for the Town of Florence, Arizona, and Pinal County, Arizona (References 9 and 10). However, this study represents a more detailed analysis.

This study is authoritative for the purposes of the National Flood Insurance Program; data presented herein either supersede or are compatible with all previous determinations.

7.0 LOCATION OF DATA

Survey, hydrologic, hydraulic, and other pertinent data used in this study can be obtained by contacting the Insurance and Mitigation Division, Federal Emergency Management Agency, 211 Main Street, Room 220, San Francisco, California 94105.

8.0 BIBLIOGRAPHY AND REFERENCES

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2. The Arizona Association of Counties and The League of Arizona Cities and Towns, Local Government Directory, Phoenix, Arizona, January 1977

3. U.S. Department of the Interior, Geological Survey, Professional Paper 655-B, Precipitation, Streamflow, and Major Floods at Selected Sites in the Gila River Drainage Basin Above Coolidge Dam, Arizona, D.E. Burkham, 1970

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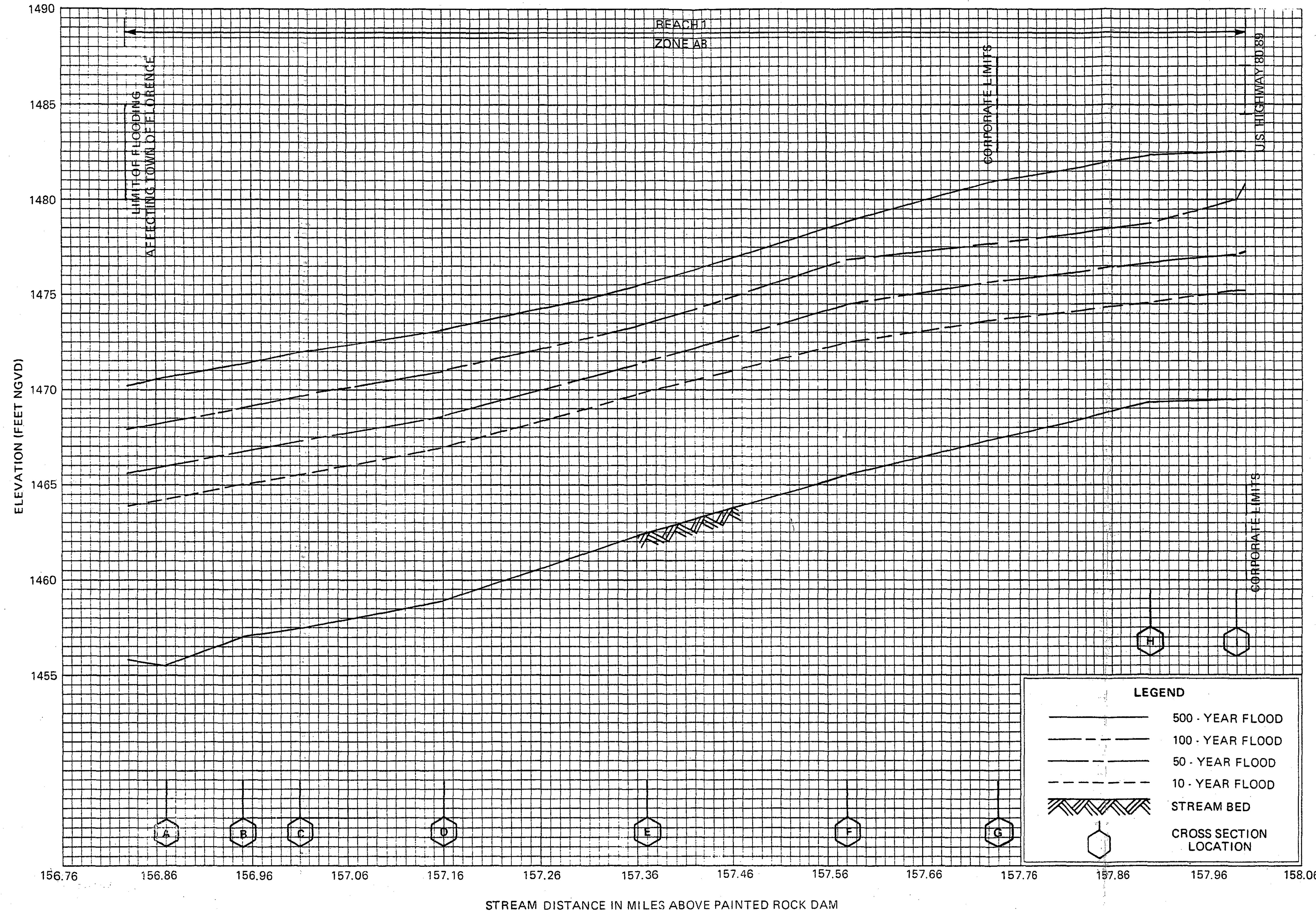

FLOOD PROFILES

GILA RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY
Federal Insurance Administration

TOWN OF FLORENCE, AZ
(PINAL CO.)

01P

Town of FLORENCE

June 21, 1989

Mr. Allan Johnson
Federal Insurance Administration
Federal Emergency Management Agency
Federal Center Plaza
500 C Street
Washington, D.C. 20472

Dear Mr. Johnson:

I am writing this letter bringing to your attention what I believe are inconsistencies in the Florence Flood Plain boundaries.

In 1983, considerable flooding along the Gila River occurred which was estimated to be well over a 100 year event. It was noticed that the areas in question were not affected by the flood, only inadequate drainage in the local areas.

Subsequent to that event, property owners in the affected areas have questioned the flood plain status to me as well as to other government officials and have gone to an extent of providing an independent engineering analysis of the boundaries which appear on the surface to contradict existing floodplain policy.

The firm of Sunrise Engineering gave a presentation of their analysis to the Town Council of the Town of Florence at its regular meeting June 19, 1989 which acknowledged receipt of the report. The Town Council gave direction to investigate the discrepancies.

I request that you look into this matter and take action on a decision to finalize these discrepancies.

Should you be in need of further information or action on my part, do not hesitate to call upon me for assistance. Thank you.

Very Truly Yours,

Ken Buchanan

Ken Buchanan
Town Manager
Town of Florence

KWB/kb

Box 490, Florence, Arizona 85232 ★ (602) 868-5889