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AGUA FRIA RIVER

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FINAL SEDIMENT TRANSPORT REPORT

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SEDIMENT TRANSPORT REPORT

AGUA FRIA RIVER

DRAFT

FINAL SEDIMENT TRANSPORT REPORT

Submitted to

Los Angeles District
Corps of Engineers
P.O. Box 2711
Los Angeles, California 90053

By

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Project Number PAZ-COE-03
RDN/R439

July 16, 1984

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EXECUTIVE SUMMARY

The Los Angeles District of the Corps of Engineers (COE) has contracted with Simons, Li and Associates, Inc. (SLA) to conduct a system analysis of the Agua Fria River for the proposed floodway from the confluence with the New River to the confluence with the Gila River. The system analysis includes a hydraulic and a sediment transport study to specifically address the following items:

1. Modifications necessary to the proposed floodway to adequately convey the 100-year flood with the estimated aggradation and degradation.

2. Requirements to maintain the project to design conditions over the life of the project, specifically the average annual maintenance requirements for sediment removal and bank protection.

3. The effect of the project on channel stability beyond the project reaches, such as headcutting upstream and sediment deposition downstream.

Three levels of analysis were conducted to assess the COE flood control plan: (1) a qualitative geomorphic analysis, (2) a quantitative geomorphic analysis, and (3) a mathematical model simulation. The results of the analyses are summarized.

Hydrologic studies of the project area were conducted by the COE and utilized for all hydraulic and sediment transport analyses. The hydrologic studies are documented in "Hydrology of the Agua Fria River," 1981, by the U.S. Army Corps of Engineers, Los Angeles District.

The hydraulics of the study reach were established by using the Army Corps of Engineers HEC-II backwater profile program. Main channel Manning resistance values of 0.025 were used for the sediment transport analysis, and 0.035 was used for establishing flow depths. Overbank resistance coefficients varied from 0.04 to 0.15, with most of the overbanks having a roughness coefficient of 0.045.

Backwater profiles were computed for the 10-, 25-, 50- and 100-year flows. Several flow breakouts, inundation of developed areas and overtopping of levees are prevalent during the 10-, 25-, 50-, and 100-year flows. These areas include:

1. Flow breakout on the west overbank just downstream of Broadway Road of 1,000 cfs, 3,500 cfs and 5,000 cfs for the 25-, 50-, and 100-year flows, respectively.

2. Flow breakout on the west overbank from Buckeye Road to 2,200 feet downstream of Lower Buckeye Road during the 100-year peak that inundates Avondale's wastewater treatment plant.

3. Flow breakout on the east overbank just upstream of Lower Buckeye Road that inundates a developed area at the 50- and 100-year flood peaks.

4. Flow through the urban area south of the Ball-Brosamer Development during the 100-year flood peak between Van Buren and Buckeye.

5. Flow through a trailer park directly south of McDowell Road on the east overbank during the 10-, 25-, 50-, and 100-year flood peaks.

6. Overtopping of existing levees between Indian School Road Bridge (ISRB) and the Roosevelt Irrigation District (RID) flume on both the east and west banks for the 50-, and 100-year flood peaks.

Freeboard heights of 1.5 feet, 1.0 foot, and 1.6 feet exist at the Southern Pacific Railroad (SPRR), RID flume, and ISRB crossings at the 100-year flood peak. The Camelback Road, Interstate 10 (I-10), and Buckeye Road crossings all have freeboard heights greater than three feet at the 100-year peak discharge.

Qualitative Geomorphic Analysis

The qualitative geomorphic analysis involves understanding the physical components of the watershed and river. A qualitative assessment of trends within the river, and whether the trends occurred naturally or were man-induced, are part of this level of analysis.

The Agua Fria in the study reach is an ephemeral braided stream with a wide flood plain. The general tendency of the river during the past 20 years has been to degrade. This is in part due to the numerous sand and gravel mining operations within the main channel and overbanks. Presently, there exist gravel mining operations or abandoned gravel pits at the following locations within the study reach:

- West overbank directly downstream of Camelback Road
- East and west overbanks directly below ISRB to south of the RID flume
- Abandoned pits north of McDowell Road in the main channel
- North and south of Van Buren Street in the main channel

The most severe degradation in the past 20 years has occurred between ISRB and the RID flume, where levees have encroached upon the flood plain and caused degradation.

The river bed material between Waddell Dam, which is located approximately 25 miles above the Agua Fria's confluence with the New River, and Bethany Home Road consists of gravel and small cobbles. This armor layer has formed on the surface largely as a result of Waddell Dam. In the study reach the surface bed material is largely sand, with a few patches of gravel and cobbles. The bed and bank material in the study reach is very susceptible to erosion. Subsurface samples in the study reach indicate that thin gravel and cobble layers (four to 14 inches thick) are present at varying depths (two to seven feet) below the thalweg. Thus the potential exists for an armor layer to form on the surface of the Agua Fria in the study reach.

Future upstream developments in the Agua Fria include a new proposed Waddell Dam, New River Dam, and Arizona Canal Diversion Channel. The new Waddell Dam will have the greatest impact on controlling future flood peaks, and subsequently channel morphology response, in the study reach.

Quantitative Geomorphic Analysis

A quantitative geomorphic analysis was conducted for the proposed flood control plan on the study reach. The long-term trends throughout the system showed most of the areas in equilibrium or showing a slight degradation potential, with the exception of the reach between the confluence with the New River and ISRB, and the reach between Lower Buckeye Road and Broadway Road, which exhibited slight aggradation tendencies. Should the sediment supply from the Agua Fria between Glendale Avenue and the confluence of the New River reduce dramatically as a result of the bed armoring, the study reach will continue to degrade, as has been the case historically.

Local scour was computed at all bridge piers, abutments, and utility towers within the 100-year flood plain at the 100-year flood peak. Local scour protection is recommended around ISRB, RID flume, SPRR and Buckeye Road piers. The local scour potential near the east abutment of ISRB also necessitates protection. Local scour protection of Tucson Electric Power (TEP) and Salt River Project (SRP) transmission towers within the main channel are also warranted based on scour potential. It should be noted, however, that the COE flood-control plan has not increased the degradation potential from the existing conditions.

Mathematical Model Analysis

QUASED, the SLA-developed water and sediment routing model, was executed to simulate the channel bed response to the 100-year flood on the COE floodway. Minor aggradation/degradation responses (less than one foot) resulted from the simulation except in the reach between ISRB and the RID flume, where degradation of the bed ranged from 1.4 feet to 3.9 feet. The QUASED model was previously verified by simulating the December 1978, January 1979 and February 1980 floods. The results of the verification runs are documented in the SLA "Hydraulic and Geomorphic Report," submitted to the Flood Control District of Maricopa County, September 13, 1983. Thus, the model has been tested and the results are adequate for floodway simulation.

Average annual aggradation/degradation depths throughout the study reach were computed using sediment rating equations generated from QUASED. The average annual aggradation/degradation depths throughout the study reach were all less than 0.2 feet, indicating minor maintenance requirements expected throughout the floodway.

Recommendations

Recommendations concerning the COE floodway to adequately convey the 100-year flood include:

1. Riprap blanket protection of bridge piers at ISRB, RID flume, SPRR and Buckeye Road.

2. Protection of the east abutment of ISRB.

3. Protection of TEP and SRP transmission towers that are within the main channel.

4. Backfilling of gravel pits on the east and west overbanks from ISRB to south of the RID flume to prevent headcuts initiating through the system and undermining local scour protection at ISRB and RID flume.

5. Possible relocation of the six-inch diameter natural gas line located just downstream of Thomas Road. Lower 500 feet of the 16-inch-diameter water line located just downstream of Thomas Road.

6. Occasional sediment removal of bars and islands which may form in the channel and reduce the channel water discharge capacity. The trends of aggradation/degradation as predicted from the sediment transport relations derived in QUASED will only occur if development occurs to the COE floodway. Until the channel is developed to the floodway, the aggradation/degradation response will be as predicted in the quantitative geomorphic analysis.

I. INTRODUCTION

1.1 General

The Agua Fria River originates in the mountains of central Arizona and flows southward for about 130 miles before its confluence with the Gila River approximately 15 miles west of downtown Phoenix. Figure 1.1 shows the Agua Fria River watershed in its entirety. The total drainage area is approximately 2,340 square miles, most of which lies in Yavapai County, Arizona. The course of the stream is nearly equidistant between two parallel mountain ranges, the Black Hills - New River Mountains and the Bradshaw Mountains, forming the eastern and western boundaries, respectively, of the drainage area. One thousand four hundred fifty-seven square miles of drainage area lies above Waddell Dam. The gradient of the Agua Fria is steep in the upper reaches, ranging from about 300 feet per mile in the headwaters to about 70 feet per mile at the canyon mouth. After leaving the canyon and flowing onto the alluvial valley plains, the gradient quickly decreases until it reaches a value of about ten feet per mile at the confluence with the Gila River.

Developments on and along the Agua Fria River include Waddell Dam, agriculture, sand and gravel mining, numerous road and utility crossings, and an increasing amount of urbanization. Overall, the vast majority of development occurs along the reach of the Agua Fria in the alluvial valley.

Through this area, the Agua Fria is a braided ephemeral stream. As with most braided streams in the area, the flood plain is rather wide and can shift rapidly due to the braided nature of the channel. Human development along the channel can add to the instability, if not conducted properly. Furthermore, due to topographic, vegetative, climatic and soils characteristics of the watershed, the Agua Fria is subject to high flood peaks.

Because of the wide natural flood plain, instability of the channel, and relatively frequent occurrence of floods able to inundate the flood plain, there is a need for a comprehensive flood-control plan for the lower Agua Fria River.

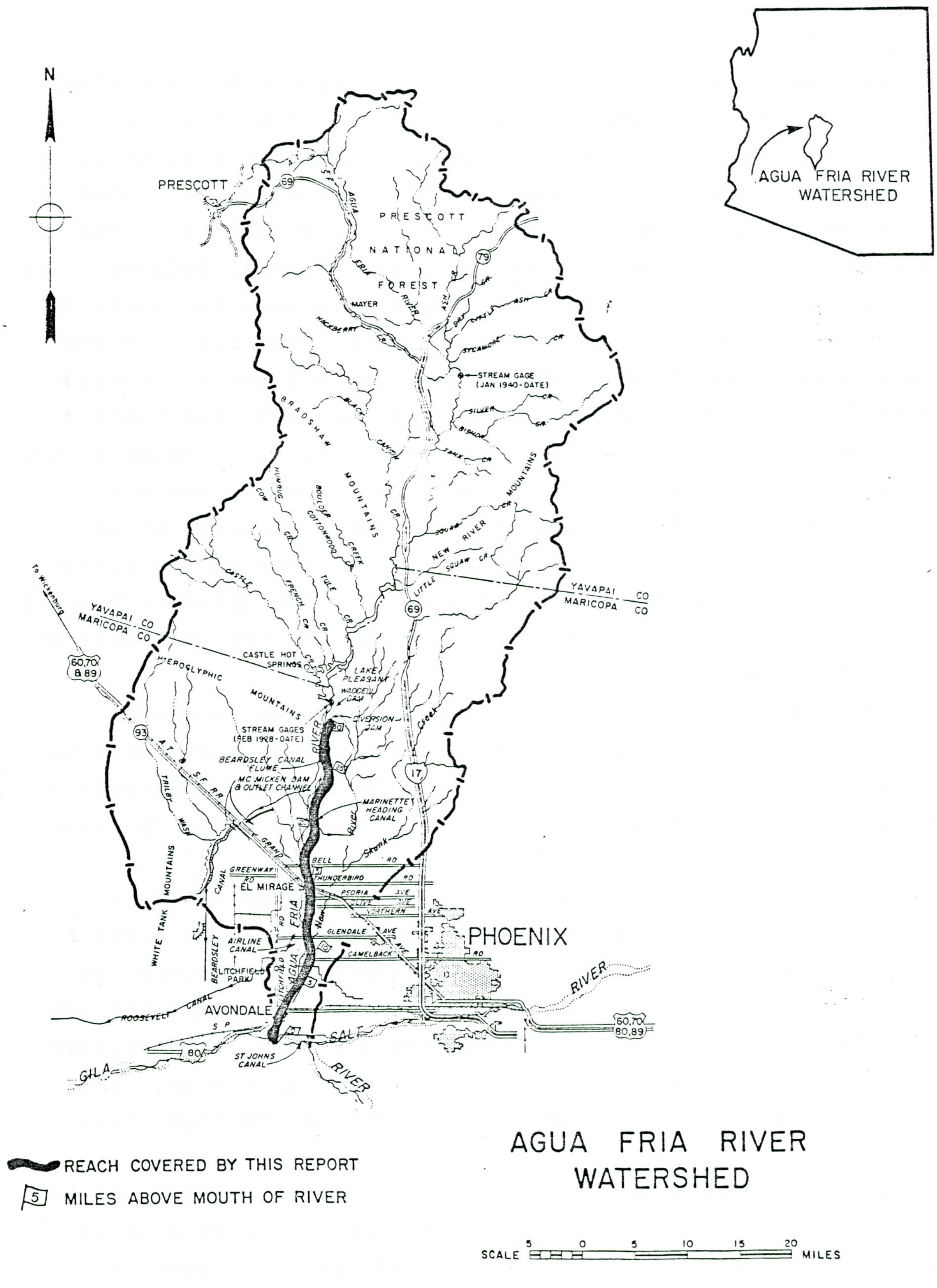


Figure 1.1. Agua Fria River Watershed.

In order to meet these needs, the Corps of Engineers (Department of the Army) has engaged Simons, Li and Associates, Inc. (SLA) to (1) study the existing flooding and channel stability problems along the lower portion of the Agua Fria River extending from the confluence with the New River, and (2) prepare a report which consists of qualitative and quantitative geomorphic analyses and sediment routing analyses for the flood control project on the lower Agua Fria as proposed by the Corps of Engineers (COE).

The material in this report presents the background, methodologies utilized, and the subsequent results of the three levels of analysis for the proposed COE flood control project on the lower Agua Fria River.

1.2 Description of Flood Control Alternative

The flood control alternative being considered for the lower Agua Fria River between its confluence with the Gila River and New River consists of purchasing flow easements for the 100-year floodway for conditions as they existed in September 1983, and providing localized floodwall protection near certain developed areas. Floodwall protection measures are provided at the following locations:

1. On the east overbank north and west of the trailer park, just downstream of McDowell Road. no.

2. On the west overbank from the Ball-Brosamer development to the Southern Pacific Railroad to protect houses south of the Ball-Brosamer project. no

3. On the west overbank from Buckeye Road extending south below Lower Buckeye Road, jogging to the west around the Avondale wastewater treatment plant, and ending approximately 2,200 feet downstream of the wastewater treatment plant.

4. Local floodwall protection upstream of Lower Buckeye Road on the east overbank protecting a subdivision near Dysart Avenue.

These floodwalls will prevent inundation of developed areas in the lower Agua Fria.

Between Indian School Road (ISR) and the Roosevelt Irrigation District (RID) flume, the levees protecting the flood plain gravel pits on both the east and west overbank are assumed to wash away during the 100-year flood. No protection of levees is considered.

River crossings considered in the flood-control plan within the study reach include the Indian School Road bridge (ISRB), RID flume, Interstate 10 (I-10), Southern Pacific Railroad (SPRR) bridge, and the Camelback Road bridge (slated for construction in the summer of 1984).

1.3 Scope of Work

To properly evaluate the COE's flood control plan, the following scope of work was performed.

1. Familiarization with COE's flood control alternative. Receipt of COE's HEC-II data.

2. Collected and assembled the pertinent data necessary to conduct a hydraulic and erosion/sedimentation analysis of the lower Agua Fria River. These data included aerial photographs, topographic information, sand and gravel mining information, channel, hydraulic, hydrologic, geologic, climatological, soil and structural data, and the data necessary for the proposed floodway.

3. Utilized the COE's hydraulic data necessary for a sediment transport analysis of the floodway for flood peaks with return intervals of 10, 25, 50, and 100 years. All bridge and flume crossings were considered in the study.

4. Grouped HEC-II cross sections into reaches made up of cross sections with similar hydraulic, geometric and sediment characteristics. Provided summaries of the average hydraulic conditions for each reach for the 10-and 100-year flood peaks.

5. Examined the adequacy of bridge and flume crossings to pass the 100-year flood with sufficient freeboard.

6. Conducted a qualitative geomorphic analysis of the river system for the proposed floodway. Compared past qualitative geomorphic studies of the Agua Fria to that of the proposed floodway and examined the effects on channel stability.

7. Conducted a quantitative geomorphic analysis of the COE flood control alternative which included the following:

    a. Determined the short-term bed response to the 10- and 100-year floods.

b. Determined the long-term bed response to the approximate bank-full discharge.

c. Used previously-derived relationships between sediment load and water discharge to determine the sediment yield from the Agua Fria and New River upstream of the study reach for the 10- and 100-year floods. Computed average annual sediment yields using an incremental probability methodology to determine sediment loading into the study reach from the New River and the Agua Fria.

d. Determined the dynamic and static equilibrium slopes that the channel bed will eventually adjust to as a result of the floodway.

e. Assessed the armoring potential of the bed to determine if it will control the eventual grade of the river.

f. Determined local scour at bridge piers, abutments and utility towers that are located in the study reach. Discussed any protection measures that may be required. Considered potential debris accumulation on bridge piers and the skew angle.

g. Determined any general regional (contraction) scour that would occur.

8. Applied the SLA-developed water and sediment routing model QUASED to the COE-developed floodway for the 100-year hydrograph. Provided plots and tables of the channel bed response at the peak discharge and at the end of the 100-year hydrograph in comparison to the pre-flood channel bed profile.

9. Established sediment rating curves throughout the study reach based upon sediment transport rates computed with QUASED. Determined the average annual sediment yield for each reach with QUASED-established sediment rating equations using an incremental probabilistic methodology.

10. Commented on average annual maintenance expected for sediment removal, and modifications to the floodway as a result of aggradation/degradation within the channel.

11. Prepared this summary report documenting all methodologies, data, assumptions and results of the sediment transport study.

II. HYDROLOGY

2.1 General

The Agua Fria River in the study area is an ephemeral stream. Runoff generally occurs only during and immediately following heavy precipitation events, because climatic and drainage characteristics in this area are not conducive to continuous runoff. Further, Waddell Dam stores a significant amount of snowmelt from the upper watershed. Significant runoff occurs in the summer months as a result of local storms and, to a lesser degree, general storms. In the winter, runoff is produced by general storms.

2.2 Flood History

Runoff records are available at five gaging stations on the Agua Fria River and three stations on the New River, which is the largest tributary of the Agua Fria. Table 2.1 shows the period of record, drainage area and maximum discharge at each of these stations.

Floods have been recorded along the Agua Fria River since 1889. The two largest reported floods on the Agua Fria occurred in January of 1916 and in November of 1919, both with estimated peak flows of 105,000 cubic feet per second (cfs). Records indicate that seven floods with flows between 50,000 cfs and 100,000 cfs, five floods with flows between 30,000 and 50,000 cfs, six floods with flows between 10,000 and 30,000 cfs, and several additional floods with unsubstantiated flows have occurred. Table 2.2 summarizes the historical floods observed in the Agua Fria River; however, a complete record of flows does not exist. The information used in formulating the flows as shown is from records of the gaging stations at Waddell Dam, the gaging station at Mayer, newspaper files, historical documents and records, and field investigations.

The most recent floods in the Agua Fria occurred in December 1978, January 1979 and February 1980. The hydrographs for these floods recorded at the USGS gaging station at Avondale are illustrated in Figure 2.1.

Table 2.1. Stream Gaging Stations Along the Agua Fria and New Rivers.

USGS Gage No.	Location	Drainage Area (sq mi)	Period of Record	Maximum Discharge Date	Maximum Discharge cfs
09512500[4]	Agua Fria River near Mayer	588	1940-80	2/19/80	34,900[1]
09512500[4]	Agua Fria River near Rock Springs	1,130	1970-80[2]	2/19/80	59,000[1]
09513650[4]	Agua Fria River at El Mirage	1,637 278[3]	1963-78	12/19/78	58,000[1]
09513970[4]	Agua Fria at Avondale	2,013 554[3]	1960-80	2/20/80	42,000[1]
09313500[5]	Lake Pleasant at Waddell Dam	1,459	1915-20[4] 1928-80[6]	2/19/80 1/28/16 to 11/27/19	66,000 (outflow) 105,000
09513780	New River near Rock Springs	67.3	1962-65[7] 1966-80	9/5/70	18,600
09513800	New River at New River	83.3	1961-80	9/5/70	19,500
0913835	New River at Bell Road, near Peoria	187	1963 1965-67[7] 1968-80	12/19/67	14,600

Notes: [1]Preliminary

[2]Historical estimates in 1891, 1915-20, 1922, 1924

[3]Below Waddell Dam

[4]Source: USGS (Watstore)

[5]Volumes only

[6]Source: MCMWD No. 1

[7]Annual maximum only

Table 2.2. Historical Floods in the Agua Fria River.

Date	Estimated Discharge (cfs)	Approximate Location
1889, March	Unknown	
1890, February 20-23	Unknown	
1891, February 19	80,000	Castle Hot Springs
1895, January	Unknown	
1905, March	Unknown	
1905, November	Unknown	
1906, March	Unknown	
1907, March 6	Unknown	
1911, February	Unknown	
1912	28,450	Above Lake Pleasant
1915, January 29	60,000	Above Lake Pleasant Site
1916, January 19	45,000	Near Lake Pleasant Site
1916, January 27	105,000	Near Lake Pleasant Site
1917, April 18	26,000	Near Lake Pleasant Site
1917, July 27	80,000	Near Lake Pleasant Site
1918, August 6	39,600	Near Lake Pleasant Site
1919, September 8	53,500	Near Lake Pleasant Site
1919, November 27	105,000	Near Lake Pleasant Site
1920, February 22	30,000	Near Lake Pleasant Site
1922, January 3	25,000	Near Lake Pleasant Site
1922, September 2	60,000	Near Lake Pleasant Site
1923,	26,300	Near Lake Pleasant Site
1923, December 27	39,000	Near Lake Pleasant Site
1925, September 19	18,600	Near Lake Pleasant Site
1927, February, Waddell Dam completed	62,000	Above Lake Pleasant

Table 2.2 (continued)

Date	Estimated Discharge (cfs)	Approximate Location
1931, February 13	Unknown	
1941, March 15	11,000	Inflow at Lake Pleasant
1943, August 3	Unknown	
1952, August 27	23,144	Inflow at Lake Pleasant
1964, July 30	1,200	Outflow at Waddell Dam
1965, April 4	460	At Avondale
1965, December 23	800	At Avondale
1967, December 12	20,000	At Avondale
1970, September 6	20,600	At Avondale
1971, August 21	8,200	At Avondale
1972, July 17	5,180	At Avondale
1972, October 7	5,000	At Avondale
1978, March 2	13,100	At Avondale
1978, December 19	60,000*	Outflow at Waddell Dam
1980, February 20	66,600*	Outflow at Waddell Dam

Source: U.S. Army Corps of Engineers report dated as follows:
1889 through 1964, except 1912 and 1923 - March, 1968
1912 and 1923 - March, 1981
1965 through 1980 - April, 1981

* Inflows to Waddell Dam were 79,500 cfs on 19 December 1978 and 73,300 cfs on 20 February 1980.

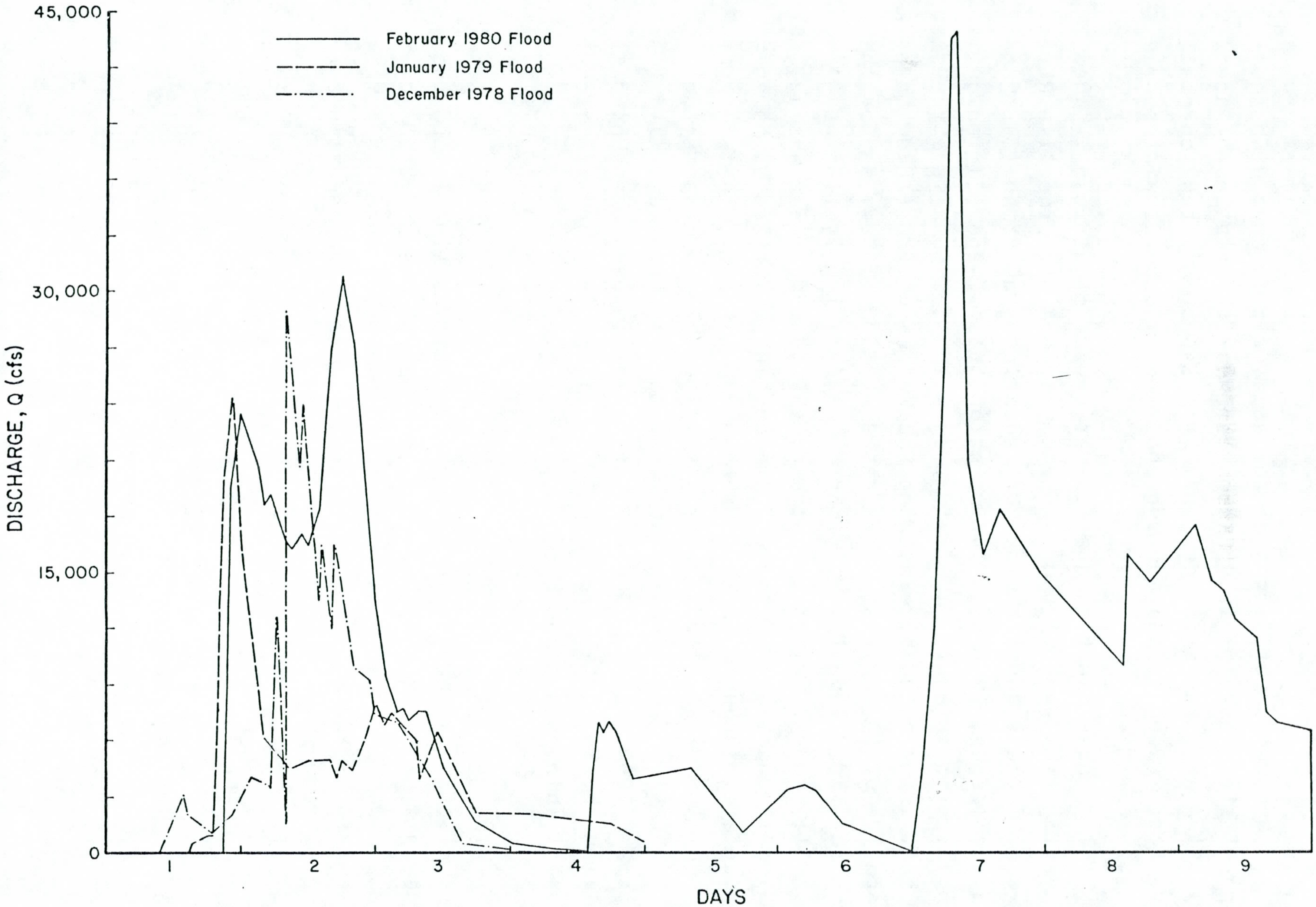


Figure 2.1. Hydrographs for 1978, 1979 and 1980 flood events on the Agua Fria.

The duration of floods on the Agua Fria depends upon the type of storm causing it. Floods can peak in a matter of hours following an intense thunderstorm, whereas it may take several days for a flood to peak during and after a general winter or summer storm.

Flood peaks in the Agua Fria attenuate significantly when traveling from Waddell Dam to the confluence with the Gila River. Several factors cause the peak attenuation, including (1) channel storage losses, (2) large infiltration losses, and (3) insignificant lateral inflows. The extent to which the peak is attenuated is best illustrated by examining the February 1980 flood. A peak discharge of 66,600 cfs was released at Waddell Dam, and by the time the flood wave traveled to the USGS gaging station at Avondale, some 30 miles downstream, the recorded peak was 42,000 cfs. Although some of the difference can be attributed to an inaccurate discharge rating curve during the flood, a large portion of the difference is surely attributable to attenuation.

Extensive flood damage occurred during the December 1978 and February 1980 floods. The COE estimated damages approaching $5.5 million for the 1978 flood and $7.6 million for the 1980 flood, most of which was done to roads and bridges.

2.3 Flood Peak Information

Flood peak information for various return flows along the Agua Fria used for the sedimentation analysis is provided in Table 2.3. The peak flows derived in the study reach were based on the following dams and drainage channels being operational:

1. Waddell Dam on the Agua Fria.
2. McMicken Dam on Tribly Wash.
3. New River Dam on New River.
4. Adobe Dam on Skunk Creek.
5. Arizona Canal Diversion Channel.
6. Cave Buttes Dam on Cave Creek.
7. Dreamy Draw Dam on Dreamy Draw Wash.
8. I-10 collection channel.

Table 2.3. Design Flood Discharge - Agua Fria River from Waddell Dam to Gila River for Existing Conditions.

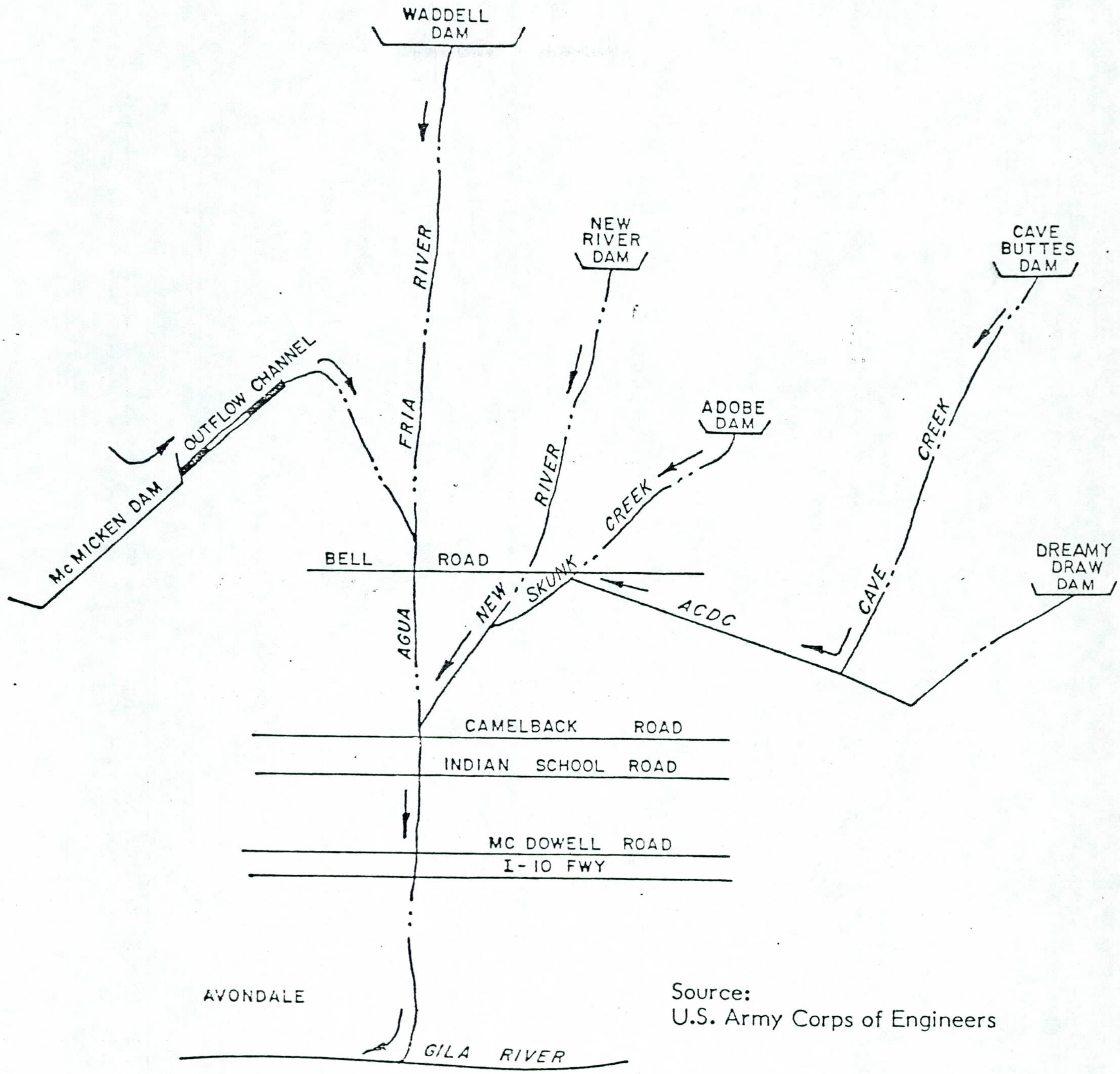
Location Along the Agua Fria River	Peak Discharge (cfs)					
	SPF	500-Year Flood	100-Year Flood	50-Year Flood	25-Year Flood	10-Year Flood
Inflow - Waddell Dam	158,000	190,000	135,000	110,000	90,000	60,000
Outflow - Waddell Dam	158,000	182,000	135,000	110,000	90,000	60,000
Bell Road	151,000	182,000	115,000	87,000	60,000	37,000
U/S New River Confluence	135,000	177,000	90,000	66,000	48,000	30,000
D/S New River Confluence	142,000	184,000	95,000	69,000	50,000	32,000
Camelback Road	142,000	184,000	95,000	69,000	50,000	31,000
Indian School Road	140,000	183,000	94,000	69,000	49,000	30,000
McDowell Road	137,000	182,000	91,000	68,000	48,000	29,000
I-10 Freeway	135,000	181,000	91,000	68,000	48,000	29,000
Avondale	131,000	179,000	90,000	67,000	47,000	28,000
Gila River	130,000	179,000	89,000	67,000	47,000	27,000

Source: U.S. Army Corps of Engineers, Los Angeles, "Hydrology of the Agua Fria River," 1981.

Figure 2.2 shows the existing and proposed dams and flood control channels that were considered in the hydrology analysis.

2.4 Flood Hydrograph

The shape of the 100-year hydrograph on the Agua Fria below the confluence with the New River was used for all sediment transport analyses throughout the study reach on the Agua Fria (see Figure 2.3). The hydrograph was constructed based on the largest general storm recorded, which occurred August 28 and 29, 1951. The flood hydrograph has a duration of four days, with the severe portion of the flood lasting just over one day.



Source:
U.S. Army Corps of Engineers

Figure 2.2. Existing and proposed dams and flood control channels.

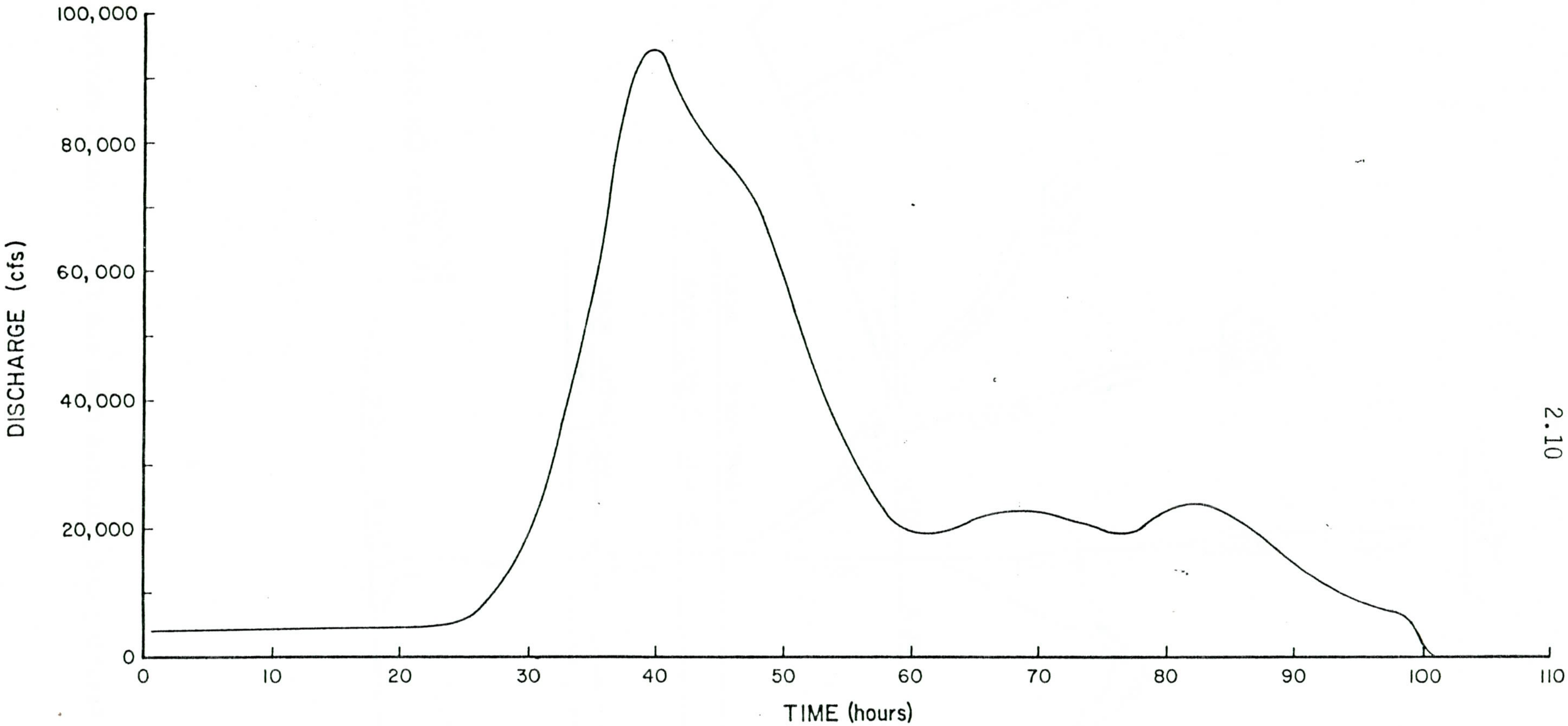


Figure 2.3. 100-year flood on the Agua Fria below the New River confluence as computed by Los Angeles Corps of Engineers, 3/2/81.

III. HYDRAULICS

3.1 Description of the Agua Fria River

The Agua Fria River is a braided stream characterized by large widths, multiple low-flow channels, and undefined banks. The channel width varies significantly along the river, ranging from 500 feet to 4,000 feet downstream of the New River confluence. The channel is generally shallow in the braided sections; however, some flood plain encroachments have caused the channel to become incised. An example of flood plain encroachment is the stretch of river between ISRB and the RID flume, where gravel mining operations have reduced the channel width to 500 feet. The qualitative geomorphic section describes in more detail the characteristics of the river.

3.2 Description of the Agua Fria ~~New River~~

Bridge crossings in the study reach of the Agua Fria River include Camelback Road, ISR, I-10, SPRR, and Buckeye Road. Table 3.1 summarizes, for each bridge, the pier diameter or width, the distance the pier extends across the bridge, the bottom elevation of the pier, the September 1983 thalweg elevation, the approximate skew angle at which the flow attacks the bridge piers, and the low-chord elevation of the bridge.

In addition to the bridge crossings of the Agua Fria River, the RID flume crosses the river approximately 2,200 feet downstream of ISRB. In 1929 the RID flume spanned a channel width of 5,959 feet. The present channel width at the flume has been reduced to 500 feet due to gravel mining operations in the area. Table 3.1 summarizes the pertinent data for the RID flume.

3.3 Hydraulic Characteristics of the Agua Fria River

Hydraulic characteristics of the Agua Fria between Glendale Avenue and the confluence with the Gila River were assessed for the 10-, 25-, 50- and 100-year flood peaks using the U.S. Army COE HEC-II backwater profile program. Hydraulic variables used to describe the flow characteristics include flow velocity, top width, hydraulic depth, and main channel and overbank discharge.

Table 3.1. Pertinent Data of Existing Bridges.

	Camelback Road	ISRB	RID Flume	I-10 Bridge	SPRR Bridge	Buckeye Road Bridge
Pier width or diameter	4'	1' 8"	4'	3'4"	6'8"	3'
Pier length	*	60'	15'	*	27'	70'
Bottom of pier footing	947.4'	983'	990.5'	945.0'	914.3' to 922.2'	947.2'
Thalweg elevation	1,017.4'	1,000'	993.6'	966.0'	952.0'	952.0'
Skew of bridge piers to flow direction	5°	30°	0°	5°	10°	10°
Low chord	1,031.7'	1,014.4'	1,008.7'	991.0'	965.7'	968.1'

*Circular piers.

The cross-sectional data used for computing backwater profiles are a mixture of the 1981 and 1983 topographies. From Buckeye Road to the confluence with the Gila River the 1981 topographic map is used. Upstream of Buckeye Road the 1983 topography is implemented in the HEC-II deck. Ninety-one cross sections were manually input to compute backwater profiles. Cross-sectional locations are shown in the plates attached to this report.

A Manning's roughness coefficient of 0.035 was used for the main channel for the backwater computations. The 0.035 value is typical of a braided sand-bed channel with sparse vegetation. Overbank roughness coefficients varied from 0.04 to 0.15, with most of the overbank area having a roughness coefficient of 0.045. The field observations and the measured stage-discharge at the Avondale gage indicate that these roughness values are reasonable.

The cross sections in the study reach were combined into 10 subreaches with similar hydraulic characteristics to provide information for sediment transport analysis. Figure 3.1 is a schematic diagram of the subreaches.

The average flow velocities, top widths and hydraulic depths for the 10-, 25-, 50- and 100-year floods in the main channel and overbanks are summarized in Tables 3.2 through 3.5, respectively.

The 100-year flood plain is plotted and shown in the 1"=500' scale topographic maps that supplement this report. Also shown in the maps are the 10-year and 100-year water-surface profiles and the thalweg profile. Average hydraulic flow depths vary from 4.6 feet to 10.2 feet, main channel velocities range from 5.5 to 9.6 feet per second, and top widths range from 2,500 feet to 5,850 feet for the 100-year flood peak.

3.4 Discussion of HEC-II Results

Within the study reach, various problems such as flow breakouts, inundation of developed areas, and overtopping of existing levees are prevalent. These problem areas are discussed in this section.

Location	Cross-Section Number	Reach Number	Reach Length (feet)
Glendale Avenue	589.25		
	581.15		
	568.70		
	558.60	1	8,337
	544.70		
	531.20		
	520.20		
	510.30		
New River	501.45		
	496.70		
	490.90		
	487.50		
Camelback Road	483.56		
Camelback Road	483.00		
	476.90	2	8,049
	473.30		
	466.60		
	459.50		
	452.60		
	444.75		
	439.45		
	433.50		
ISRB	427.75		
ISRB	426.95		
	422.30		
	417.75		
	414.95	3	2,301
	409.45		
RID Flume	403.86		
RID Flume	403.85		
RID Flume	403.71		
RID Flume	403.70		
	398.00		
	392.10		
	385.50	4	3,580
	381.40		
Thomas Road	370.50		
	358.30		
	348.60		
	344.20		
	334.20		
McDowell Road	323.20	5	7,975
	319.40		
	316.20		
	308.30		
	298.00		

Figure 3.1. Reach Definition for the Agua Fria River.

Location	Cross-Section Number	Reach Number	Reach Length (feet)
	291.80		
	288.80		
I-10	283.50		
I-10	283.40		
I-10	281.60	6	4,632
I-10	281.50		
	278.10		
	275.15		
	266.80		
Van Buren Street	254.30		
	246.20		
	240.20		
	234.00		
	227.95		
	221.40	7	5,470
	212.85		
SPRR	202.30		
SPRR	202.00		
Buckeye Road	201.00		
Buckeye Road	200.20		
	190.20		
	181.55		
	173.85		
	168.00	8	5,075
	159.00		
	151.35		
Lower Buckeye Road	146.80		
	135.40		
	130.65		
	121.45		
	117.35		
	111.00	9	6,230
	103.90		
	93.80		
Broadway Road	82.60		
	75.00		
	70.45		
	61.90		
	53.60		
	44.60	10	7,155
	35.20		
Southern Avenue	26.90		
	20.00		
	13.70		
Gila River	7.15		

Figure 3.1. Reach Definition for the Agua Fria River (continued).

Table 3.2. Average Flow Velocity, Hydraulic Depth, Effective Width, and Discharge for the 10-Year Flood Event.

	Left Flood Plain				Main Channel				Right Flood Plain			
Reach	Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Water Discharge (cfs)	Velocity (fps)	Hydraulic Depth (ft)	Effective Depth (ft)	Water Discharge (cfs)	Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Water Discharge (cfs)
1	1.09	0.65	554	396	4.63	3.29	1,905	28,981	1.03	0.75	534	410
2	0.68	0.37	367	91	4.66	3.45	1,913	30,804	0.72	0.32	224	51
3	0.33	0.16	494	27	7.06	6.28	670	29,669	0.76	0.41	1,240	390
4	1.89	1.55	455	1,327	5.75	5.99	833	28,668	0.14	0.03	930	4
5	0.53	0.40	265	56	6.35	5.34	840	28,463	1.24	0.51	1,428	907
6	0.26	0.20	123	7	5.56	4.54	1,125	28,377	0	0	0	0
7	0	0	0	0	5.63	4.78	1,041	28,000	0	0	0	0
8	0.92	0.64	527	311	5.05	4.06	1,239	25,390	2.18	1.26	759	2,085
9	1.64	1.11	1,248	2,267	5.04	3.86	1,271	24,731	0.36	0.31	15	2
10	1.16	0.77	847	756	5.26	4.07	1,225	26,244	0	0	0	0

Reach 1 Glendale Avenue to Confluence with New River.
Reach 2 Confluence with New River to Indian School Road Bridge.
Reach 3 Indian School Road Bridge to the Roosevelt Irrigation District Flume.
Reach 4 Roosevelt Irrigation District Flume to Thomas Road.
Reach 5 Thomas Road to 1,500 feet upstream of I-10.
Reach 6 1,500 feet upstream of I-10 to Van Buren.
Reach 7 Van Buren to Buckeye Road.
Reach 8 Buckeye Road to Lower Buckeye Road.
Reach 9 Lower Buckeye Road to Broadway Road.
Reach 10 Broadway Road to the Confluence with the Gila River.

Table 3.3. Average Flow Velocity, Hydraulic Depth, Effective Width, and Discharge for the 25-Year Flood Event.

	Left Flood Plain				Main Channel				Right Flood Plain			
Reach	Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Water Discharge (cfs)	Velocity (fps)	Hydraulic Depth (ft)	Effective Depth (ft)	Water Discharge (cfs)	Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Water Discharge (cfs)
1	1.30	0.82	755	803	4.78	3.37	2,123	34,199	2.29	2.33	2,433	12,998
2	1.20	0.69	560	461	5.36	4.54	2,004	48,836	1.46	0.94	433	596
3	1.13	0.70	1,043	823	7.94	8.04	684	43,672	2.00	1.62	1,418	4,591
4	2.51	1.88	1,108	5,216	6.70	7.64	835	42,744	1.06	0.58	1,677	1,040
5	1.45	1.14	446	737	6.66	6.69	905	40,328	1.92	1.50	2,551	7,361
6	1.21	1.03	676	915	6.63	6.17	1,137	46,508	0.65	0.48	86	27
7	1.15	0.56	1,477	945	6.73	6.37	1,071	45,901	0.96	0.61	263	155
8	1.44	1.34	773	1,491	6.02	5.39	1,241	40,322	2.83	2.14	856	5,188
9	2.26	1.98	1,398	6,246	6.09	5.24	1,278	40,747	0.37	0.41	47	7
10	1.92	1.31	1,482	3,728	6.24	5.47	1,239	42,268	2.31	0.59	737	1,004

Reach 1 Glendale Avenue to Confluence with New River.
Reach 2 Confluence with New River to Indian School Road Bridge.
Reach 3 Indian School Road Bridge to the Roosevelt Irrigation District Flume.
Reach 4 Roosevelt Irrigation District Flume to Thomas Road.
Reach 5 Thomas Road to 1,500 feet upstream of I-10.
Reach 6 1,500 feet upstream of I-10 to Van Buren.
Reach 7 Van Buren to Buckeye Road.
Reach 8 Buckeye Road to Lower Buckeye Road.
Reach 9 Lower Buckeye Road to Broadway Road.
Reach 10 Broadway Road to the Confluence with the Gila River.

Table 3.4. Average Flow Velocity, Hydraulic Depth, Effective Width, and Discharge for the 50-Year Flood Event.

	Left Flood Plain				Main Channel				Right Flood Plain			
Reach	Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Water Discharge (cfs)	Velocity (fps)	Hydraulic Depth (ft)	Effective Depth (ft)	Water Discharge (cfs)	Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Water Discharge (cfs)
1	1.74	1.16	855	1,722	4.98	3.90	2,134	41,404	2.97	2.86	2,696	22,874
2	2.08	1.62	878	2,963	5.77	5.39	2,018	62,665	2.15	1.59	942	3,213
3	2.04	1.47	1,343	4,024	8.62	9.19	686	54,374	2.83	2.64	1,422	10,602
4	3.01	2.20	1,527	10,100	7.36	8.56	835	52,588	1.89	1.40	2,398	6,313
5	2.12	1.98	597	2,513	6.89	7.85	908	49,141	2.40	2.24	3,128	16,771
6	2.22	2.28	1,165	5,909	7.30	7.05	1,192	61,355	1.09	0.87	127	119
7	1.90	1.51	2,128	6,087	7.23	7.78	1,072	60,306	1.44	1.19	354	607
8	2.41	2.43	996	5,822	6.69	6.40	1,242	53,200	3.18	2.93	858	7,978
9	3.03	2.79	1,653	13,959	6.88	6.00	1,284	53,026	0.42	0.47	78	15
10	2.51	1.80	2,023	9,140	6.84	6.38	1,246	54,355	2.33	0.98	1,535	3,505

Reach 1 Glendale Avenue to Confluence with New River.
Reach 2 Confluence with New River to Indian School Road Bridge.
Reach 3 Indian School Road Bridge to the Roosevelt Irrigation District Flume.
Reach 4 Roosevelt Irrigation District Flume to Thomas Road.
Reach 5 Thomas Road to 1,500 feet upstream of I-10.
Reach 6 1,500 feet upstream of I-10 to Van Buren.
Reach 7 Van Buren to Buckeye Road.
Reach 8 Buckeye Road to Lower Buckeye Road.
Reach 9 Lower Buckeye Road to Broadway Road.
Reach 10 Broadway Road to the Confluence with the Gila River.

Table 3.5. Average Flow Velocity, Hydraulic Depth, Effective Width, and Discharge for the 100-Year Flood Event.

	Left Flood Plain				Main Channel				Right Flood Plain			
Reach	Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Water Discharge (cfs)	Velocity (fps)	Hydraulic Depth (ft)	Effective Depth (ft)	Water Discharge (cfs)	Velocity (fps)	Hydraulic Depth (ft)	Effective Width (ft)	Water Discharge (cfs)
1	2.18	1.58	987	3,399	5.52	4.55	2,135	53,630	3.51	3.45	2,723	32,970
2	2.53	2.40	896	5,442	6.36	6.44	2,016	82,587	2.52	2.39	956	5,758
3	2.68	2.23	1,349	8,065	9.56	10.20	687	67,031	3.57	3.41	1,478	17,989
4	3.48	2.96	1,618	16,671	8.03	9.61	835	64,417	2.31	2.09	2,468	11,913
5	2.33	2.44	659	3,749	7.24	8.27	907	54,324	3.15	3.03	3,539	33,779
6	2.78	3.12	1,167	10,119	8.15	8.23	1,192	79,952	1.52	1.38	149	313
7	2.36	2.58	2,167	13,193	7.60	9.25	1,073	75,448	1.86	2.02	362	1,360
8	2.75	3.02	1,045	8,675	7.50	7.18	1,244	66,982	3.99	3.88	913	14,129
9	3.81	3.56	2,053	27,846	7.20	6.60	1,284	61,029	1.02	1.09	113	126
10	3.29	2.39	2,606	20,491	7.37	6.87	1,254	63,488	2.52	1.04	1,916	5,021

Reach 1 Glendale Avenue to Confluence with New River.
Reach 2 Confluence with New River to Indian School Road Bridge.
Reach 3 Indian School Road Bridge to the Roosevelt Irrigation District Flume.
Reach 4 Roosevelt Irrigation District Flume to Thomas Road.
Reach 5 Thomas Road to 1,500 feet upstream of I-10.
Reach 6 1,500 feet upstream of I-10 to Van Buren.
Reach 7 Van Buren to Buckeye Road.
Reach 8 Buckeye Road to Lower Buckeye Road.
Reach 9 Lower Buckeye Road to Broadway Road.
Reach 10 Broadway Road to the Confluence with the Gila River.

Just downstream of Broadway Road on the west overbank, breakout flow occurs during the 25-, 50-, and 100-year floods. Approximately 1,000 cfs, 3,500 cfs and 5,000 cfs leave the main channel at the 25-, 50-, and 100-year flood peaks, respectively. Once the water breaks out, it flows west along Broadway Road for approximately 1,000 feet, and then flows south over agricultural fields, eventually draining into the Gila River.

Below Buckeye Road, extending 2,200 feet downstream of Lower Buckeye Road, breakout flow occurs on the west overbank for existing conditions at the 100-year flood peak. Breakout flow would go through a developed area half a block west of Dysart Road from Buckeye Road to Harrison Drive, and inundate the City of Avondale's wastewater treatment plant during the 100-year flood. The COE's proposed levees will prevent this overflow problem.

Just upstream of Lower Buckeye Road on the east overbank, some breakout flow will occur through the developed area east of Dysart Avenue at the 50-, and 100-year flood peaks. Some houses will be inundated by three feet of water at the 100-year flood peak. The COE is proposing floodwall protection along Dysart Avenue north and west of the subdivision to prevent inundation.

Between Buckeye Road and I-10, urbanization is encroaching upon the flood plain from the west. For existing conditions, some west overbank flow occurs. Inundation of developed land south of the Ball-Brosamer Development will be prevented by COE's floodwalls. The fill imported for the Ball-Brosamer Development is high enough to prevent the 100-year flood peak from flowing through the area. The east overbank flow between Buckeye Road and I-10 is approximately 18,000 cfs. Flow breaks out 1,000 feet east of El Mirage Road and returns to the main channel at the SPRR Bridge.

Directly below McDowell Road on the east overbank, a trailer park will be inundated by the 10-, 25-, 50-, and 100-year flood peaks. ~~The COE is providing floodwall protection around this park.~~

Between ISRB and the RID flume, flow will overtop the existing levees during the 50- and 100-year floods. Local velocities, as predicted by HEC-II, approach 16.5 feet per second (fps), and the COE has assumed the levees will wash out during the course of the 50- and 100-year floods. All gravel pit stockpiles on the backsides of levees are assumed to be washed out and all flood plain gravel pits are assumed inundated with water.

Upstream of ISR to Cambelback Road the flood plain widens to 5,000 to 7,800 feet. West flood plain gravel pits will become inundated during the 50-, and 100-year flood peaks.

The flow at Cambelback Road bridge necks down considerably from existing conditions; however, water will not overtop the proposed embankments during the 100-year flood peak. Adequate freeboard (exceeding three feet) is provided for the 100-year flood at Cambelback Road.

Table 3.6 summarizes the available freeboard at all river crossings in the study reach for the 100-year flood. Less than three feet of freeboard exist at ISRB, the RID flume and the SPRR Bridge for the 100-year flood peaks.

Table 3.6. Summary of Freeboard Available at Bridges for the 100-Year Flood.

Crossing	Low Chord Elevation (ft)	100-Year Water Surface (ft)	Freeboard Height (ft)
Camelback Road	1,031.7	1,024.4	7.3
Indian School Road	1,014.4	1,012.8	1.6
RID flume	1,008.7	1,006.7	2.0
Interstate-10	991.0	981.2	9.8
Southern Pacific Railroad	965.7	964.2	1.5
Buckeye Road	968.1	962.9	5.2

IV. QUALITATIVE GEOMORPHIC ANALYSIS

4.1 General

The qualitative geomorphic analysis is used to evaluate the physical characteristics of the system. The qualitative analysis relies heavily upon examination of aerial photographs, channel and watershed data, flood reports, accounts of various instream activities, and site visits. The qualitative analysis documents the changes in the system, whether man-induced or natural, and provides the understanding of the system necessary to proceed with the quantitative engineering geomorphic, and mathematical modeling analyses.

4.2 Description of the Agua Fria River and Tributaries

The Agua Fria River begins at the south base of Mingus Mountain in Prescott National Forest and flows southward 130 miles to its confluence with the Gila River. The total drainage area is 2,340 square miles, of which 1,457 square miles are above Waddell Dam. Below Waddell Dam the Agua Fria flows through a canyon for several miles and then into a valley flood plain. In the valley the Agua Fria is a braided, generally wide river with poorly defined and unstable banks. It is characterized by a steep, shallow course with multiple channel divisions around alluvial islands. The Agua Fria flows approximately 34 miles from Waddell Dam to the confluence of the Gila River. The major tributary entering the Agua Fria in this reach is the New River.

The New River originates in the New River Mountains and flows 40 miles southward to its confluence with the Agua Fria River just upstream of Camelback Road. The drainage area of the New River at its mouth is 340 square miles, of which approximately one-third is mountainous. Stream gradients decrease from 370 feet per mile in the mountains to 10 feet per mile in the valley.

Skunk Creek, the major tributary to the New River, rises in the New River Mountains and flows generally southwestward for about 30 miles to its confluence with the New River. Only about 20 percent of the 110-square-mile watershed is mountainous.

In addition to the Agua Fria and its tributaries, several interceptor canals exist in the area, including the Arizona Canal, the Glendale-Dysart Drain, the Grand Canal, and the I-10 collector channel. The Arizona Canal Diversion Channel (ACDC), to be constructed north of and parallel to the existing Arizona Canal, will transport floodwaters from Cudia City Wash, Dreamy Draw Wash, 10th and Northern Avenue drains, and Cave Creek.

Several reservoirs exist in the watershed, including Waddell Dam (Agua Fria River), Dreamy Draw Dam (Dreamy Draw Wash), Cave Buttes Dam (Cave Creek) and Adobe Dam (Skunk Creek). Several more flood-control reservoirs are being considered for construction, including the New River Dam (New River) and new Waddell Dam (Agua Fria).

4.3 Geology and Physiography

Approximately 70 percent of the Agua Fria River basin is mountainous (above 3,000 feet in elevation) and characterized by rugged terrain and steep gradients. The remaining 30 percent consists of fairly flat valley land with regular alluvial slopes. The general geology and physiography of the Agua Fria Valley and watershed are illustrated in Figure 4.1 and described in this section. The description and interpretation of the geologic substrata within Maricopa County are based on work by Wilson et al. (1957), and on data extrapolated from a study of a similar alluvial valley adjacent to the Agua Fria (Sycamore Creek) by Anderson (1968).

The lower alluvial area is underlain by poorly consolidated alluvial deposits of Tertiary and Quaternary age. Deposits in the flood plain are unconsolidated alluvium that consists of sand, silt, gravel and some clay (unit Q_s, Figure 4.1), and contains appreciable amounts of firmly cemented fine-grained soils of low permeability. Most of the alluvium, however, is unconsolidated sand and gravel with high permeabilities.

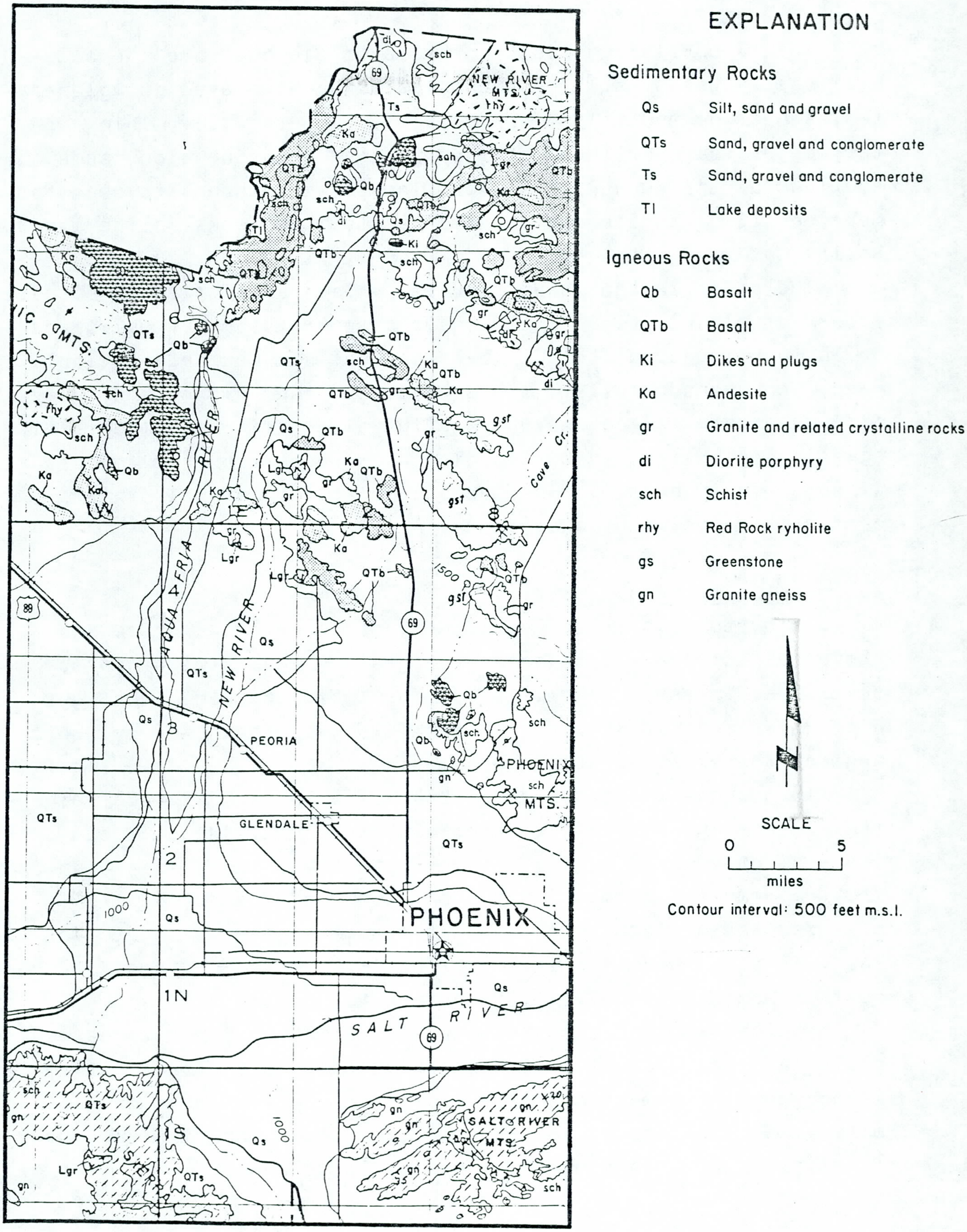


Figure 4.1. Geologic map of part of the Agua Fria River Basin, New Mexico. (From Wilson et al., 1957).

The flood plain deposits overlie or are cut into the alluvial valley deposits. These consist of sand, gravel, conglomerates, sandstone and siltstone (unit QT_s, Figure 4.1). Thin terrace gravel overlies the finer grained alluvium along some sections of the Agua Fria River. These valley deposits unconformably overlie granite and related crystalline rocks in the lower valley.

The soils in the lower alluvial valley are formed on either recent or old alluvium (Soil Conservation Service, unpublished). Soils in or adjacent to the river channel are characteristically deep, sandy and gravelly. These gravelly sandy loams and loamy fine sands are formed in recent alluvial material and moderately alkaline and slightly to strongly calcareous. Thus it appears as if no geologic controls are present to act as natural grade controls in the study area.

4.4 Sediment Characteristics

Prior to this study, sediment samples were collected and sieve tests performed to determine grain size distributions by several soil testing firms at various locations along the Agua Fria River. Additional sediment samples were gathered by SLA to augment the existing soils information.

Throughout the reach downstream of the New River confluence, the surface and subsurface materials are mainly sands with a trace of gravels. The D_{50} size (50 percent finer size) ranges from 0.7 mm to 1.3 mm and the gradation coefficient, which measures the uniformity of bed material, ranges from three to four. Typical bed-material distributions of the surface and subsurface samples are given in Figure 4.2.

While the river appears generally sandy, layers of coarse gravel and small cobbles with thicknesses ranging from four to 14 inches were observed in nearly all of the boring logs and test pits. The distance to the gravel layer below the riverbed surface varies with each of the sampling locations.

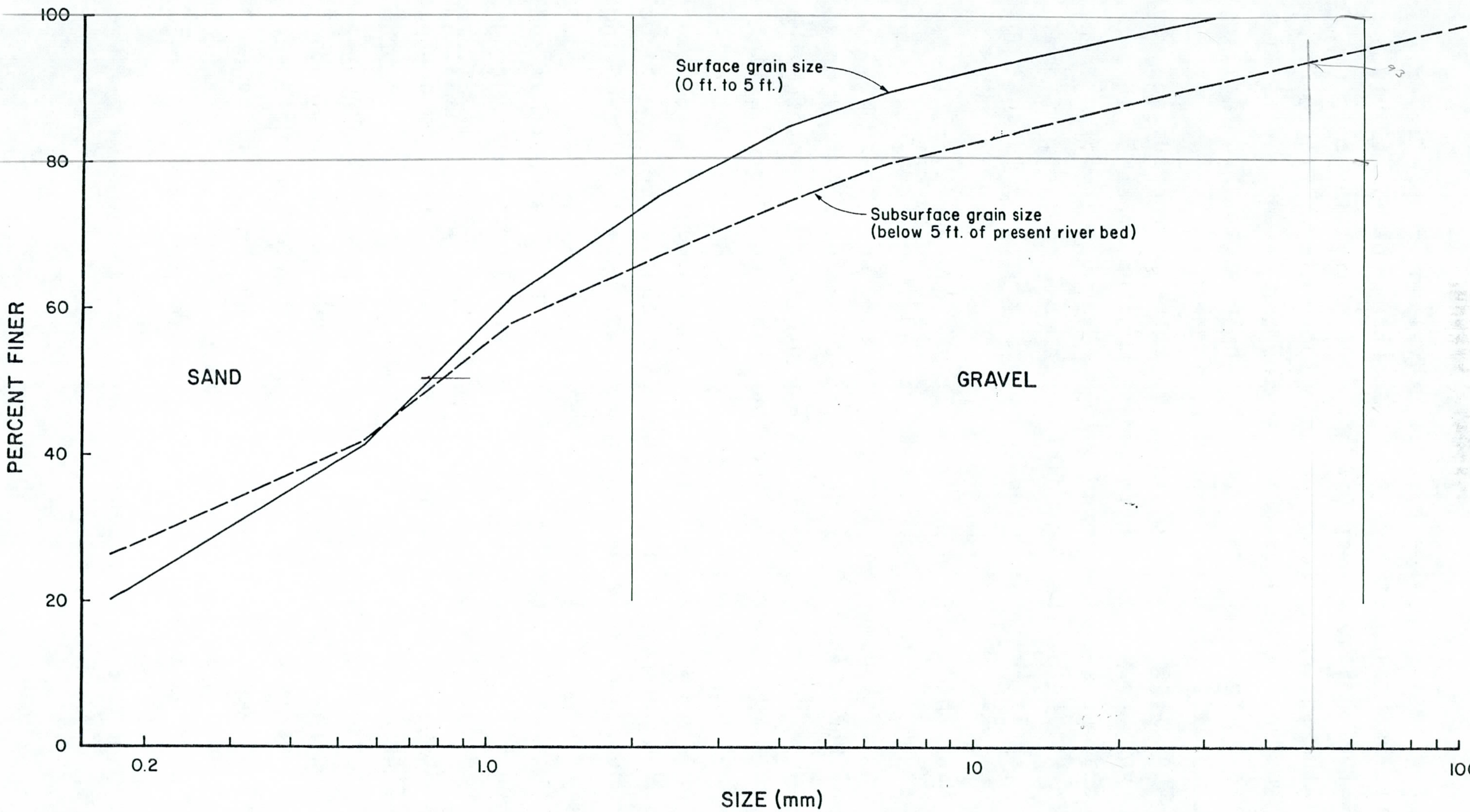


Figure 4.2. Surface and subsurface grain size distributions of the Agua Fria near Indian School Road Bridge.

Figure 4.3 shows a typical gravel and cobble layer. This picture was taken of a test pit located approximately 800 ft below Indian School Road Bridge. The gravel layer is one foot below the streambed and about one foot thick. Figure 4.4 is a close-up shot with a grid overlaying the gravel layer. The squares of the grid are two inches on a side, thus the largest particle size measures about four inches.

The distance to the gravel layer below the surface varies from two to seven feet throughout the study reach. In a few test pits clay lenses were found below the gravel layers. These clay layers will slow the degradation process; however, it doesn't appear as if there is a continuous clay stratum in the subsurface.

Near the New River confluence the gravel layer is exposed in patches on the river bed due to degradation; however, complete armoring of the bed has not taken place (see Figure 4.5). Near Bethany Home Road on the Agua Fria the sands and fine gravels have been removed from the surface gravel layer through the sediment sorting process, leaving the river bed armored by large gravels and cobbles.

Surface armoring has occurred near McDowell Road, Thomas Road, Van Buren Street, the New River confluence, and the river reach near and above Bethany Home Road. River bed armoring from Bethany Home Road to Waddell Dam is very significant. This is attributable to the trapping of sediment in Waddell Dam and the subsequent downstream channel erosion. Figures 4.6 through 4.8 show bed-material samples near Waddell Dam, Beardsley Flume and Grand Avenue, respectively. There is an increase in bed-material size from Grand Avenue to Waddell Dam.

Figure 4.9 shows the bed material found at Grand Avenue. This is typical of the upstream armored reach, which ends approximately at Bethany Home Road. From Bethany Home Road to the confluence of the New River, some patches of river armoring are evidenced in the low-flow channel. Downstream of the New River confluence, the river becomes sandy except for local gravel and cobble zones as described previously.

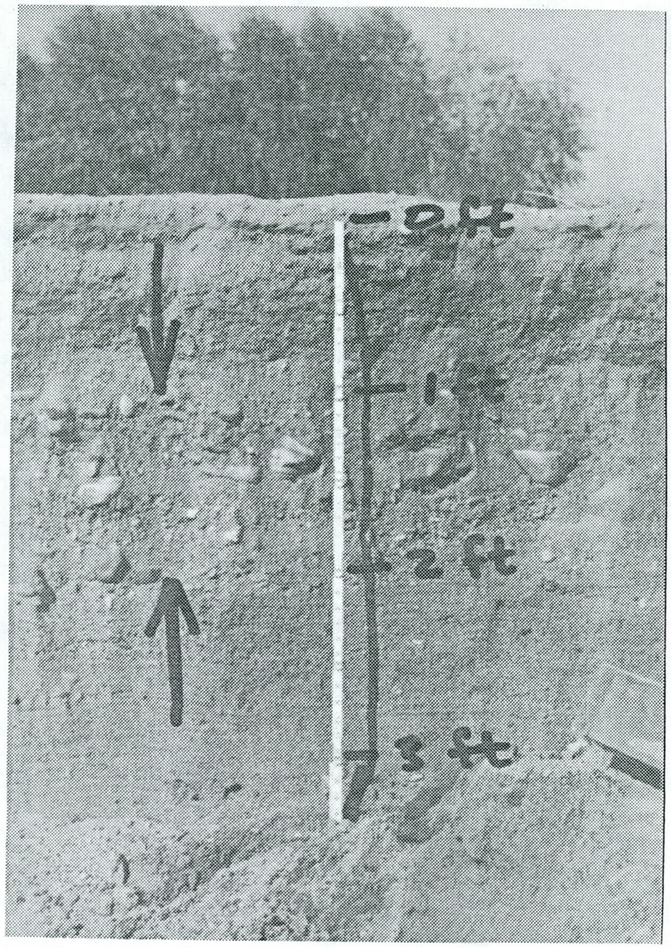


Figure 4.3. Gravel layer below the river bed of Agua Fria River approximately 800 feet down-stream of Indian School Road Bridge.

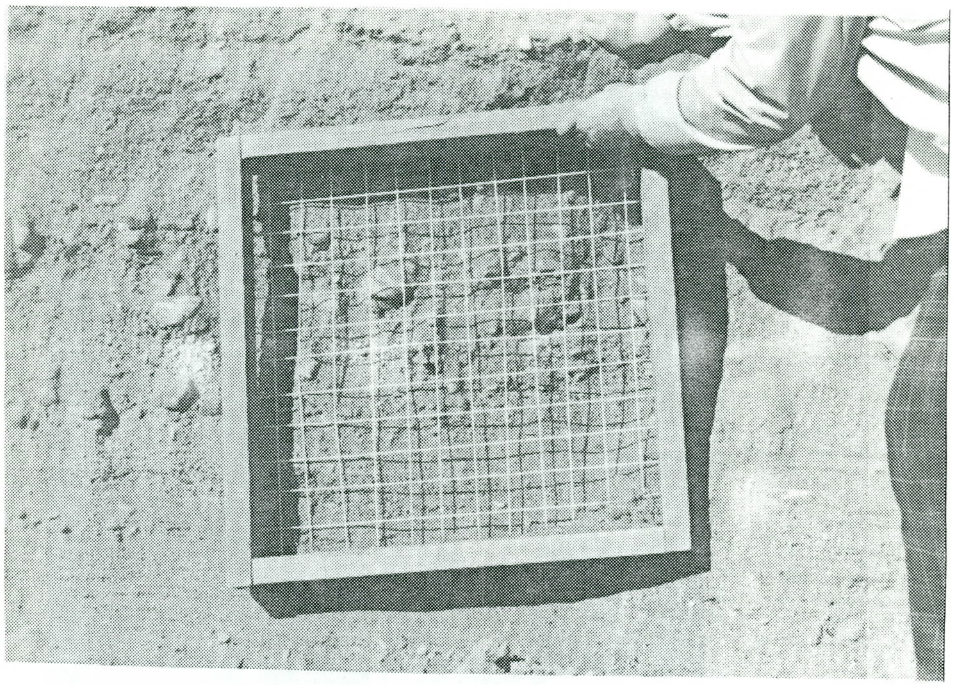


Figure 4.4. Close-up of the gravel layer below the river bed of Agua Fria River, approximately 800 feet downstream of Indian School Road Bridge.

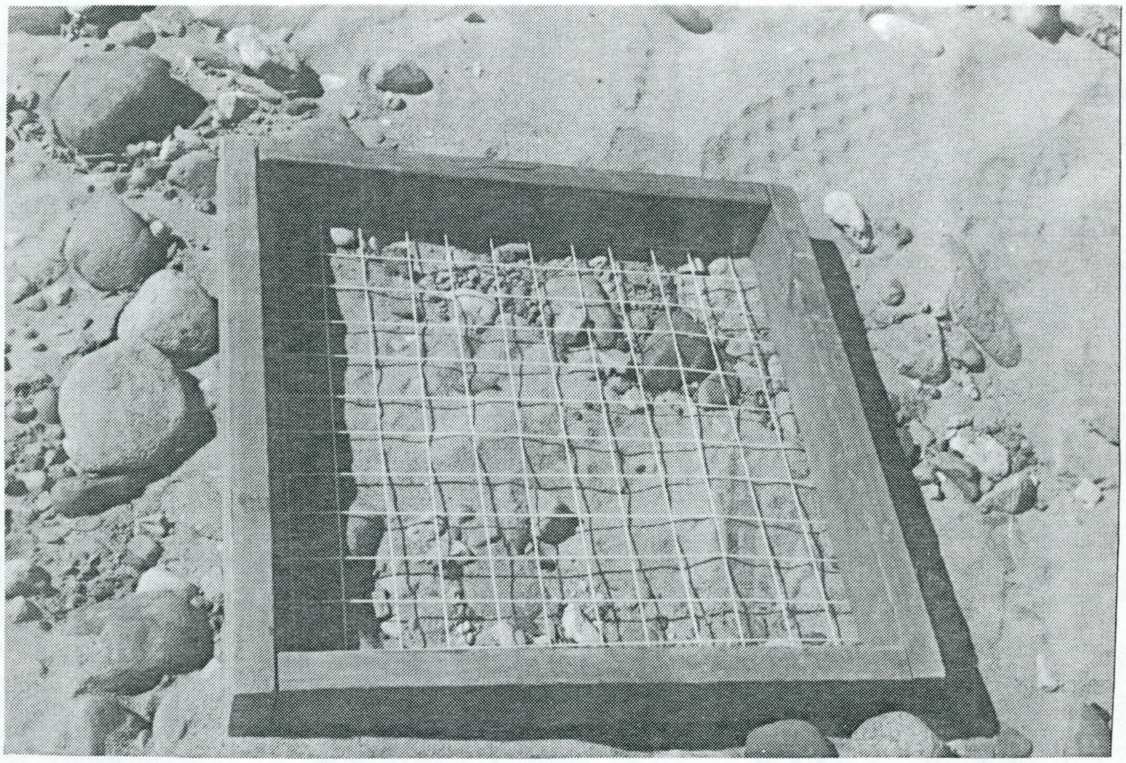
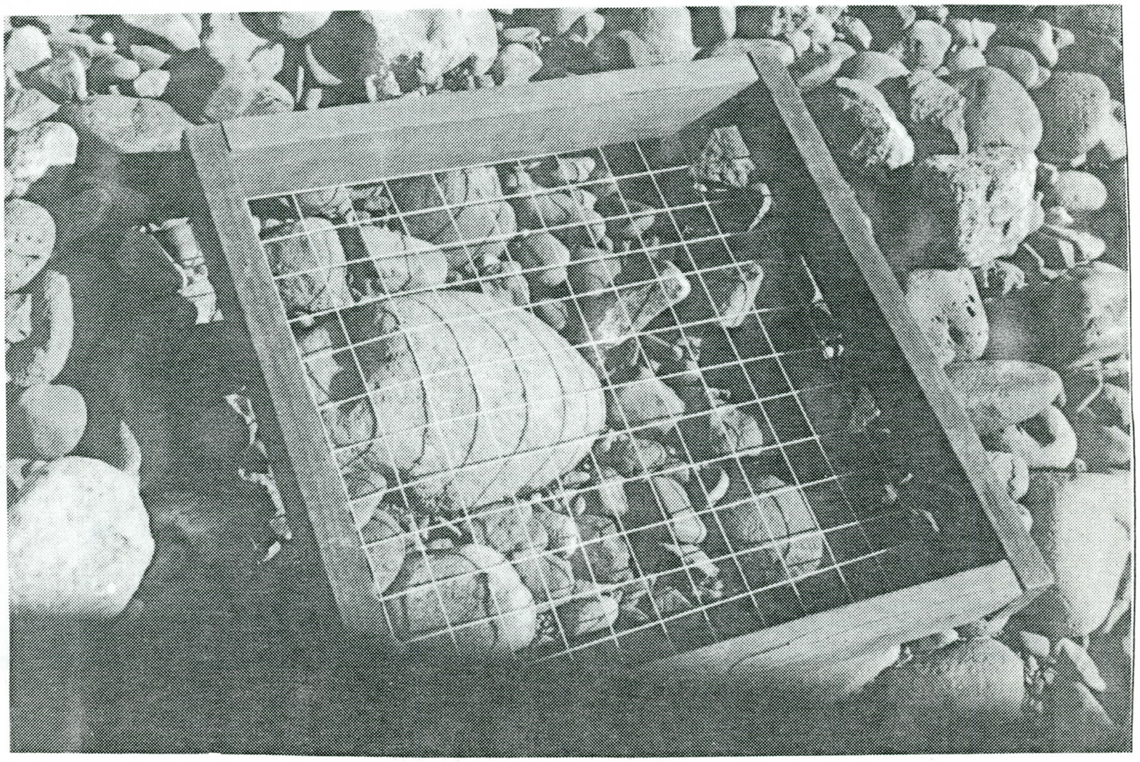


Figure 4.5. River bed materials of the Agua Fria River upstream of the confluence with New River.



* The square is two inches on each side.

Figure 4.6. Bed material of the Agua Fria River near Waddell Dam.

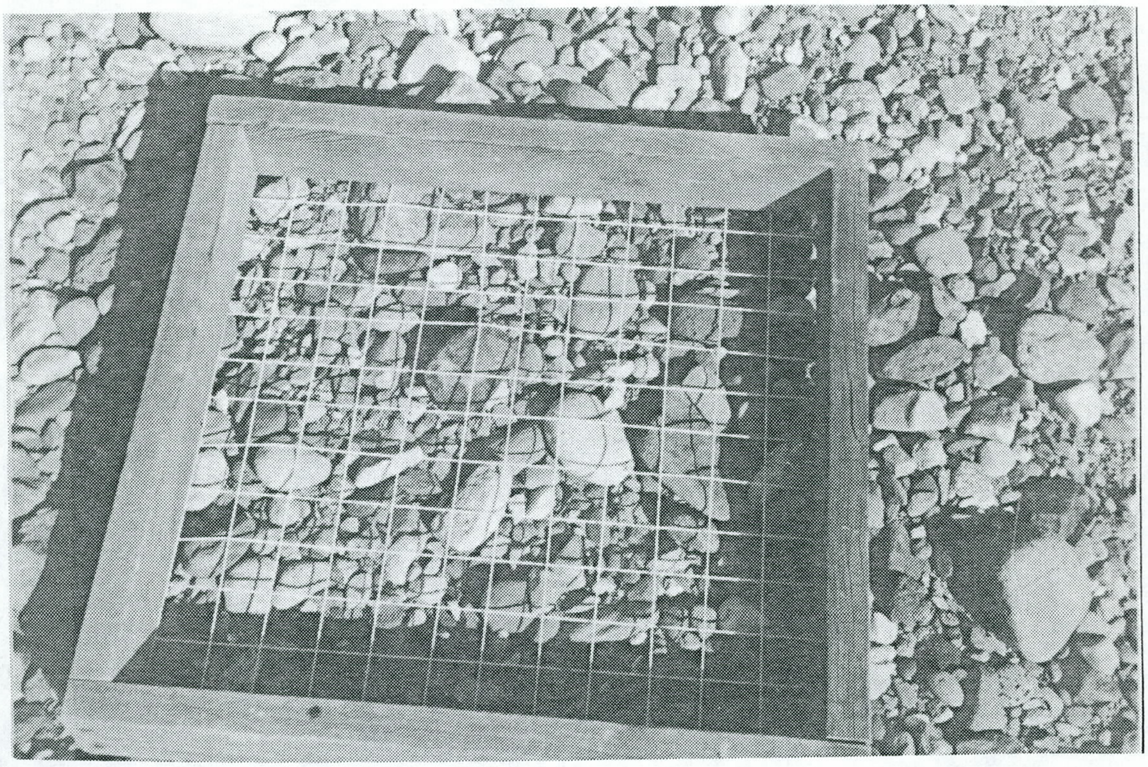


Figure 4.7. Bed material of the Agua Fria River near Beardsley flume.

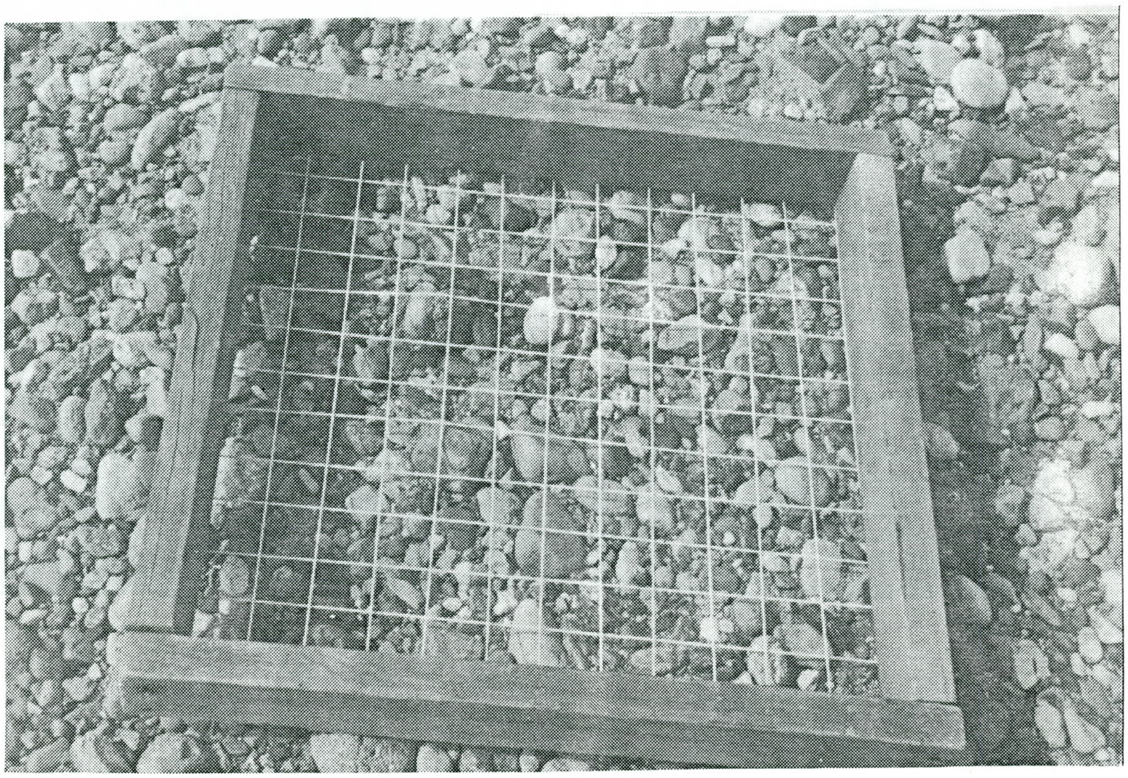


Figure 4.8. Bed material of the Agua Fria River near Grand Avenue.

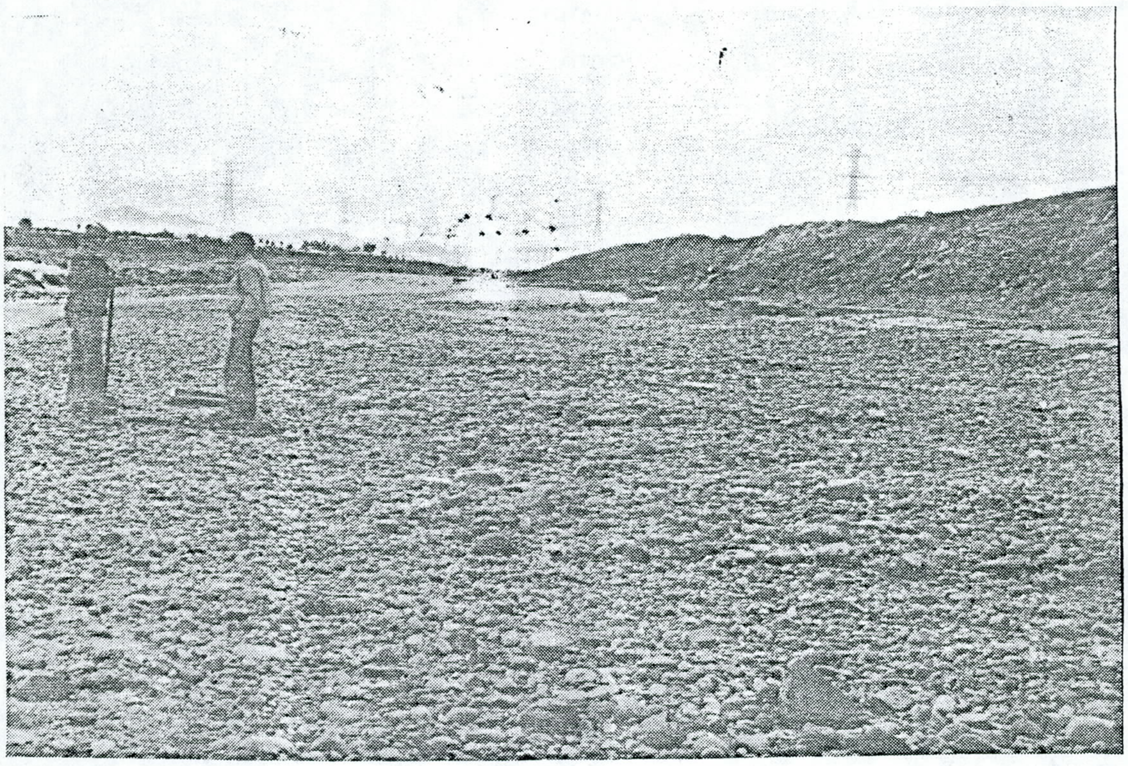


Figure 4.9. Overview of the armored river bed of the Agua Fria River at Grand Avenue, looking downstream.

In summary, the bed material in the study reach is composed of sand and fine gravels. Gravel layers which were formed from alluvial deposits are apparent in the subsurface sediment samples, and in many locations are exposed on the river bed. River armoring due to the removal of fine material from the surface gravel layer by past floods is significant from Waddell Dam to Bethany Home Road.

The maximum sediment size found in the study area is about six inches and the gravel and cobble layer varies between four and 14 inches thick. As a consequence, the armor layer developed on the river bed is relatively thin, generally less than one foot.

Since the alluvial strata of the Agua Fria River consist of distinct sand and gravel layers, the size distributions analyzed using the available sediment samples vary significantly. The typical surface and subsurface sediment distributions of the Agua Fria River shown in Figure 4.2 are used in the sediment transport computations. The potential sediment reduction due to armoring is considered in the evaluation of the long-term channel response.

4.5 Upstream Developments

Upstream developments in the Agua Fria and New Rivers have affected the hydrology and subsequently the river response. These developments include dams and reservoirs, drainage channels, urbanization, sand and gravel mining industries and agricultural practices.

The existing and proposed dams and flood control channels upstream of the study reach were shown in Figure 2.2. The existing dams include Waddell, Adobe, Cave Buttes, Dreamy Draw and McMicken (not functioning). The New River Dam is presently under construction and proposed structures include a new Waddell Dam and the Arizona Canal Diversion Channel. Descriptions of these dams and drainage canals follow.

4.5.1 Waddell Dam

Waddell Dam, located about 34 miles upstream of the Gila River confluence, was completed in 1927. About two-thirds of the Agua Fria watershed is above the dam, which is under the jurisdiction of the Maricopa County Municipal Water Conservation District No. 1.

The major impact of Waddell Dam has been the trapping of sediment in the reservoir, resulting in downstream degradation. Continuous degradation removed finer sediments from the river bed and left an armor layer of coarser particles on the surface. As stated previously, the channel downstream of Waddell Dam is armored to approximately Bethany Home Road.

4.5.2 New Waddell Dam

Large spills occurred over Waddell Dam in 1978, 1979 and 1980, initiating reinvestigation of the need to construct a new dam for flood control purposes. A flood control analysis for a new Waddell Dam was conducted by the Central Arizona Water Control Study (CAWCS). The new dam, to be located about one-fourth of a mile downstream of the existing dam, would increase the existing capacity of 157,600 acre-feet to 891,400 acre-feet and would limit the maximum release of the standard project flood to about 25,000 cfs.

The new Waddell Dam would trap more sediment than the present Waddell Dam due to the larger storage area and increased sediment detention time within the reservoir. However, flood discharge releases will be significantly reduced, so the overall effect of construction of the new Waddell Dam will be increased downstream flood control and reduced downstream sediment transporting capacity. The 100-year flood peak at Camelback Road will reduce by approximately half with construction of a new Waddell Dam.

4.5.3 ACDC and Detention Dams in the New River Watershed

The Arizona Canal Diversion Channel (ACDC) will intercept the drainage of watersheds to the north of the existing Arizona Canal from Cudia City Wash to Skunk Creek (see Figure 4.10). The existing canal diverts water for irrigation from the Salt River at the Granite Reef Reservoir. The proposed channel will run parallel to the existing channel.

Dreamy Draw and Cave Buttes Reservoirs store water upstream of the ACDC. Dreamy Draw Dam, completed in July 1973, is located 1.8 miles above the ACDC and controls about 65 percent of the Dreamy Draw watershed (1.3 square miles). Cave Buttes Dam, located about 11 miles upstream of the confluence of Cave Creek and the ACDC, controls 87 percent of the Cave Creek watershed (195 square miles). The net effect of the ACDC will be an increase in water and sediment discharge into Skunk Creek.

4.5.4 Adobe Dam

Adobe Dam was constructed on Skunk Creek, about seven miles north of Bell Road and one mile west of the Black Canyon Highway. The embankment is a compacted earthfill structure 16 feet above the streambed. Skunk Creek is the major tributary of the New River.

4.5.5 New River Dam

The New River Dam is to be constructed on the New River about eight miles upstream from the confluence with Skunk Creek. The proposed dam will regulate about 164 square miles of the existing 340 square-mile New River, and will significantly reduce peak discharges for floods.

4.5.6. McMicken Dam

McMicken Dam, which controls the runoff from about 240 square miles of Tribly Wash watershed, is located at the northeast base of the White Tank Mountains. The dam, completed in 1956, was constructed by the U.S. Army Corps of Engineers as a flood control dam. The dam was breached for safety considerations, but will be repaired.

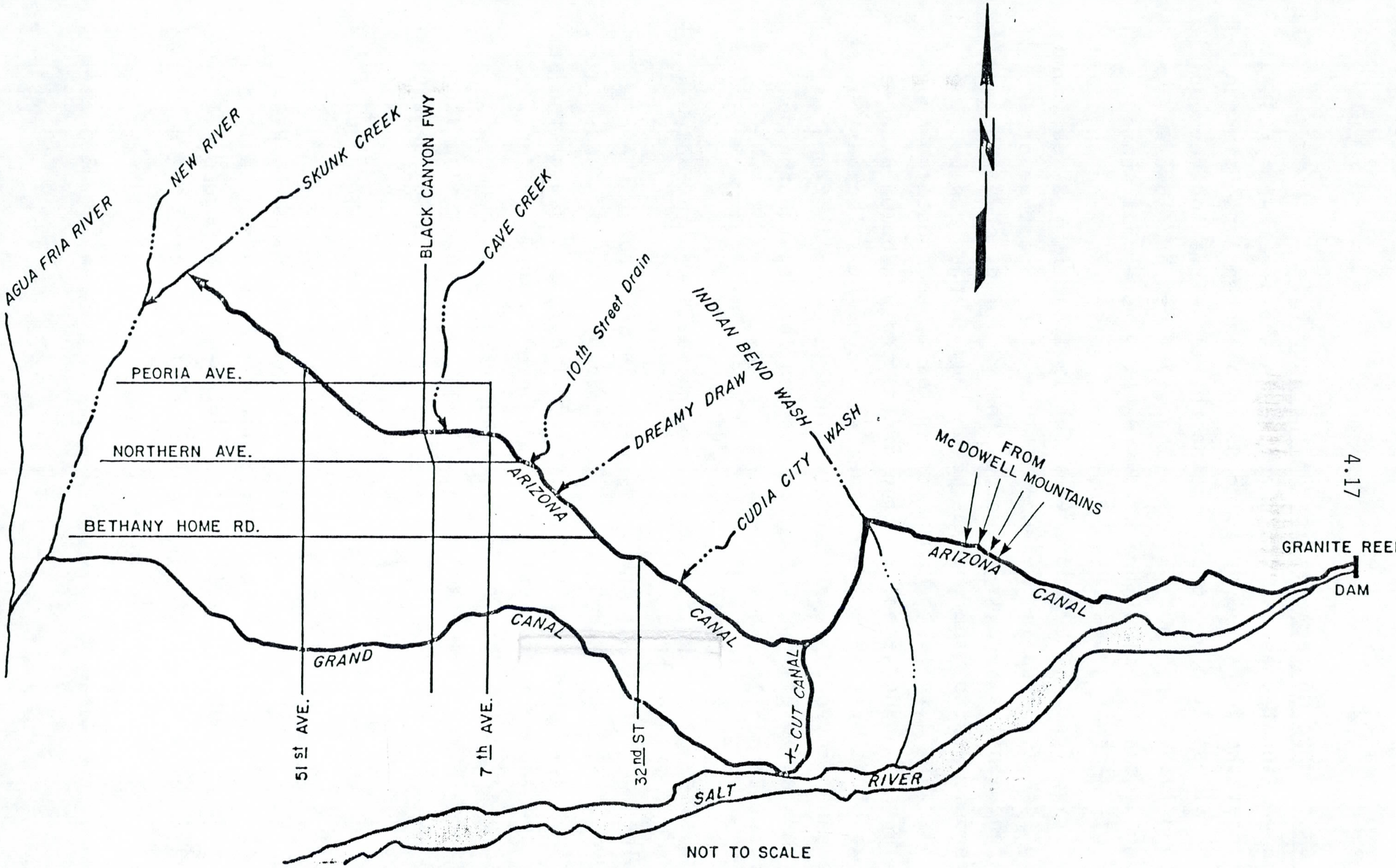


Figure 4.10. Location of the Arizona Canal and the Study Reach.

4.6 Geomorphic Characteristics of the Agua Fria

Significant changes have occurred in the lower Agua Fria River over the years. Dynamic conditions in the Agua Fria can best be illustrated by comparing the thalweg elevation between 1972 and 1983 (see Figure 4.11). The river bed has lowered throughout almost all of the study reach. Note that contour intervals on the 1972 map are four feet, while those on the 1983 map are two feet. The accuracy of the 1972 map is ± 2 feet, and that of the 1983 map is ±0.5 feet. Thus, the magnitude of the difference in thalwegs is masked by the ±2.5 feet combined map tolerance. Most of the channel morphology changes can be directly attributable to human activities in and near the Agua Fria. For instance, the lowering of the thalweg below Broadway Road is directly related to channel work done near the west bank where a new levee was constructed. The following sections describe the lower Agua Fria from its confluence with the New River to its confluence with the Gila River.

4.6.1 Agua Fria from Juncture with New River to Indian School Road

The upper limit of the study reach is located at the confluence with the New River. The river has a wide flood plain in this area. Approximately a quarter mile downstream of the New River confluence is the Camelback Road dip crossing, which will be replaced by a 1,725 foot-wide bridge with minor channelization and spur dikes to align the flow through the bridge. The flood plain will be narrowed from 3,500 feet to 1,725 feet at Camelback Road, thus an increase in flow velocity is expected at the bridge crossing, and consequently some general regional (contractual) scour.

Directly downstream of Camelback Road on the west overbank is a sand and gravel mining operation. The operation is located approximately 800 feet west of the west bank (see Figure 4.12). The sand and gravel pits intercept flow from a low-flow braid

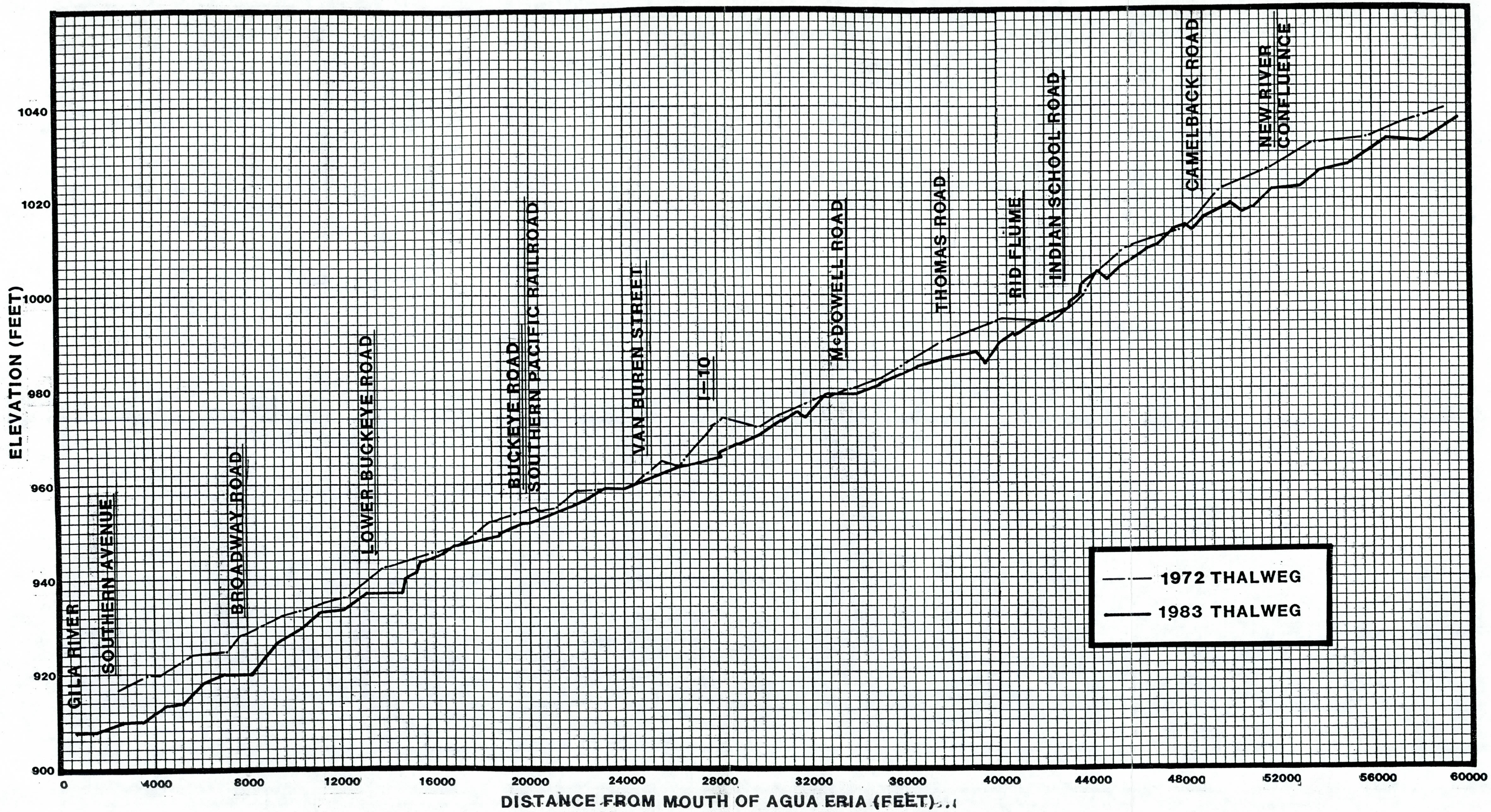


FIGURE 4.11 COMPARISON OF 1972 AND 1983 THALWEG ELEVATIONS IN THE LOWER AGUA FRIA

FIGURE 4.12 1980 AERIAL PHOTOGRAPH SHOWING LOCATION OF GRAVEL PITS SOUTH OF CAMELBACK ROAD

approximately 1,700 feet west of the proposed Camelback Road bridge. Although the west approach to the Camelback Road bridge will be high enough to prevent flow from directly entering the gravel mining operation, the dikes surrounding the sand and gravel pits are too low to prevent overtopping by the 100-year peak discharge downstream of the bridge. As a result, a potential headcut problem exists at this site.

Three hundred feet upstream of ISRB above the west abutment, a small gravel pit is being mined. The depth (20 feet) and area (50 x 100 feet) of the pit are relatively small. Should the pit dimensions remain unchanged, the potential upstream and downstream erosion impacts will be limited.

In general, gravel mining effects are not just limited to the gravel pit area. Headcuts can initiate at the upper boundary of the gravel pit and extend far upstream. A gravel pit can also act as a sink, trapping sediment, resulting in a sediment transport imbalance, and causing possible downstream degradation. The overall effect from instream mining, if the pits are deep and extend significant distances along the river, is channel entrenchment and increased channel instability. Sand and gravel mining operators frequently construct levees to protect their flood plain pits from flow in the main channel. If constriction of the river due to the levees is excessive, channel degradation can be induced. Thus, main channel and flood plain gravel mining operations can have an impact on future channel stabilization.

ISRB is a 1,620 foot-wide bridge that failed during the February 1980 flood. Several of the piers near the east bank were undermined due to excessive scour. The spread footing piers were buried approximately 25 feet below the channel bed, and the measured scour approached 29 feet during the 1980 flood. Since the 1980 flood the piers that failed have been replaced with caissons, which were drilled 50 feet below the 1980 local scour depth.

4.6.2 Indian School Road to Thomas Road

This stretch of the river has been severely encroached upon by gravel mining operations. Extensive gravel mining has occurred both on the east and west overbanks between ISRB and the RID flume crossing (see Figure 4.13). The effective channel width has been reduced to 500 feet in some areas, which results in increased flow velocities and increased sediment transport rates. Downstream of the RID flume, sand and gravel pits extend 1,200 and 2,200 feet on the west and east overbanks, respectively. A general lowering of the channel bed has resulted from gravel mining levees restricting the flow in this area.

4.6.3 Thomas Road to I-10

Between Thomas Road and I-10 the Agua Fria has been encroached upon by agricultural fields near McDowell Road (see Figure 4.14). The main channel narrows from 1,000 feet at Thomas Road to approximately 400 feet near McDowell Road, where the river makes a severe dogleg to the west. The main channel widens to 1,000 feet near the dogleg.

Near the west bank just upstream of McDowell Road there are some abandoned gravel mines that are slowly filling with sediment. McDowell Road has a dip crossing and there is no crossing of the Agua Fria at Thomas Road.

Upstream of I-10, on the east overbank, the I-10 collector channel enters the Agua Fria. A spur dike upstream of I-10 on the east overbank has been constructed to guide the east overbank flow through I-10.

With completion of the east approach to I-10, flow that previously broke out at the dogleg below McDowell Road, circumventing the I-10 bridge, will now be intercepted by the I-10 drainage channel and funneled through the bridge. Before the east approach to I-10 was completed, flow that overtopped the dogleg below McDowell Road would not return to the Agua Fria until it was intercepted by the Southern Pacific Railroad embankment (see Figure 4.15). Thus, with the completion of the east

FIGURE 4.13
SEPTEMBER, 1983 AERIAL PHOTOGRAPH SHOWING LEVEE CONSTRICTION BETWEEN ISRB AND RID FLUME

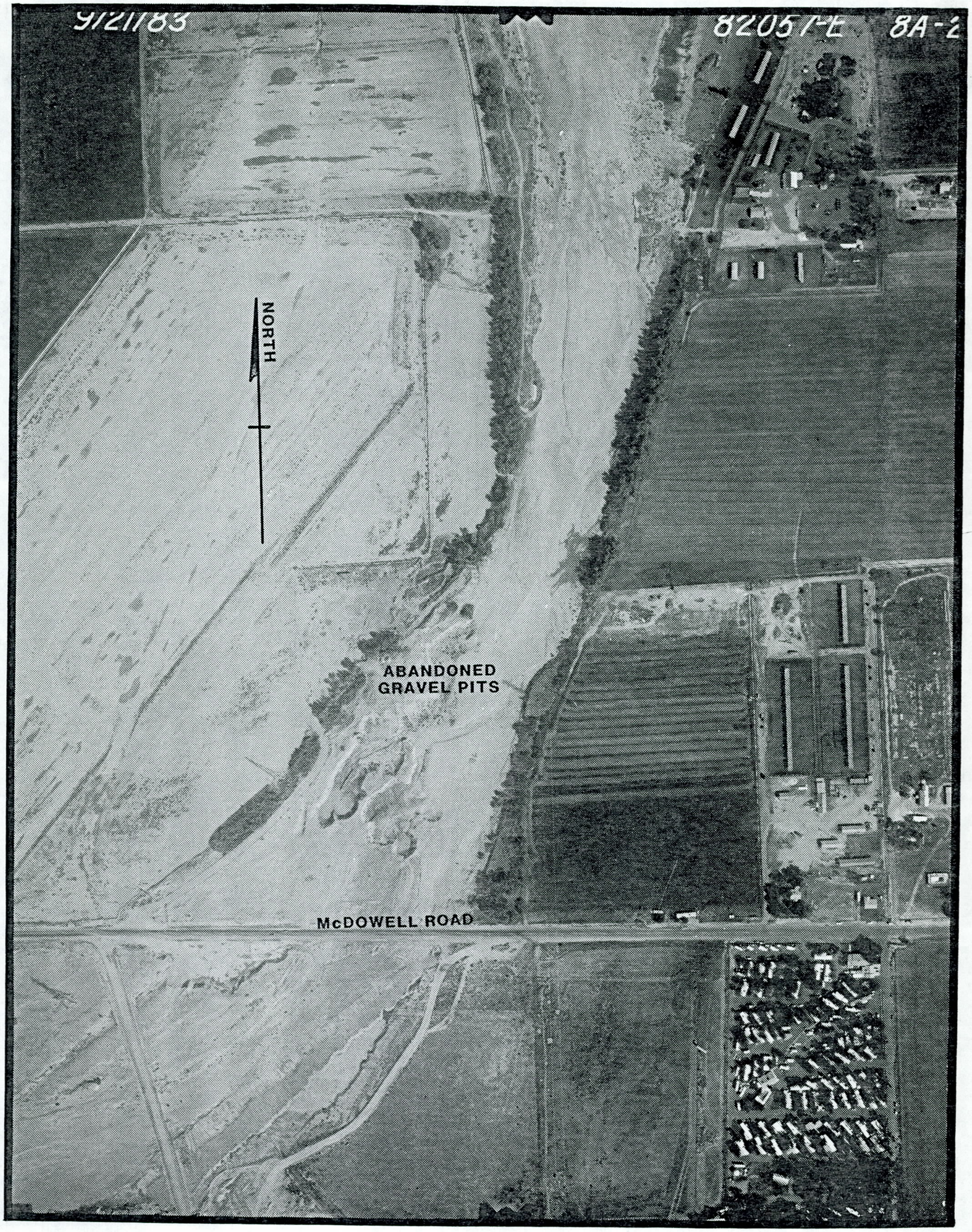


FIGURE 4.14
SEPTEMBER, 1983 AERIAL PHOTOGRAPH SHOWING AGRICULTURAL CONSTRICTION NEAR McDOWELL ROAD

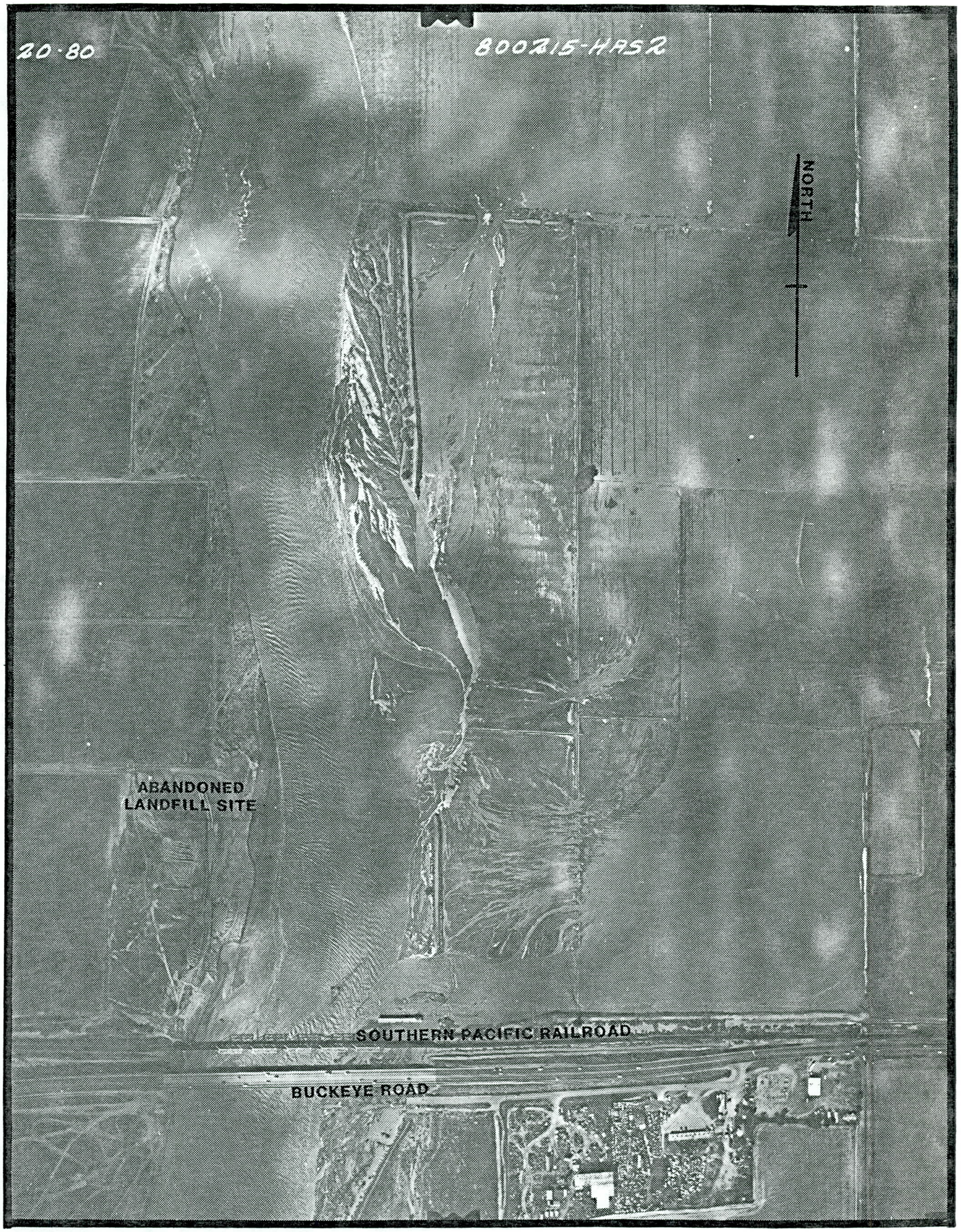


FIGURE 4.15 1980 AERIAL PHOTOGRAPH SHOWING OVERBANK FLOW DOWNSTREAM OF McDOWELL ROAD

approach to the I-10 bridge (see Figure 4.16), the flood plain widths dramatically reduce downstream of I-10.

4.6.4 I-10 to Buckeye Road

Between I-10 and Buckeye Road, several instream sand and gravel mining operations, a landfill site, and urban encroachment are narrowing the flood plain.

The instream gravel mining operation, located about 1,200 feet south of I-10 and extending 1,400 feet south to just upstream of Van Buren, has decreased the main channel width from 1,500 feet at I-10 to 600 feet near Van Buren. Five hundred feet downstream of Van Buren, near the west bank of the river, there are several gravel pits (see Figure 4.17). The gravel pits have trapped sediment from upstream, causing a sediment transport imbalance downstream and contributing to the degradation response. This is evidenced by comparing the 1972 and 1983 thalwegs.

An abandoned landfill is located 300 feet north of the SPRR bridge on the west overbank. During the 100-year flood some overbank flow will occur over the landfill, thus the need to provide floodwall protection along the west overbank. About 1,500 feet upstream of the Southern Pacific Railroad, near the east bank, a large sand bar formed after the 1980 flood. This is typical of a steep-braided channel where the sediment supply is greater than the transport capacity. It also displays the dynamic nature of the Agua Fria.

Urban sprawl from the City of Avondale on the west overbank from I-10 to Buckeye Road is slowly expanding to the Agua Fria. Evidence of the sprawl is the Ball-Brosamer Development, which will extend from Van Buren to 2,800 feet south of Van Buren and 350 feet east of 10th Street. The development will move the 100-year flood plain approximately 700 feet east of its present location. The encroachment will force more water into the east overbank and into the main channel, resulting in larger velocities within the main channel and increased sediment transport rates.

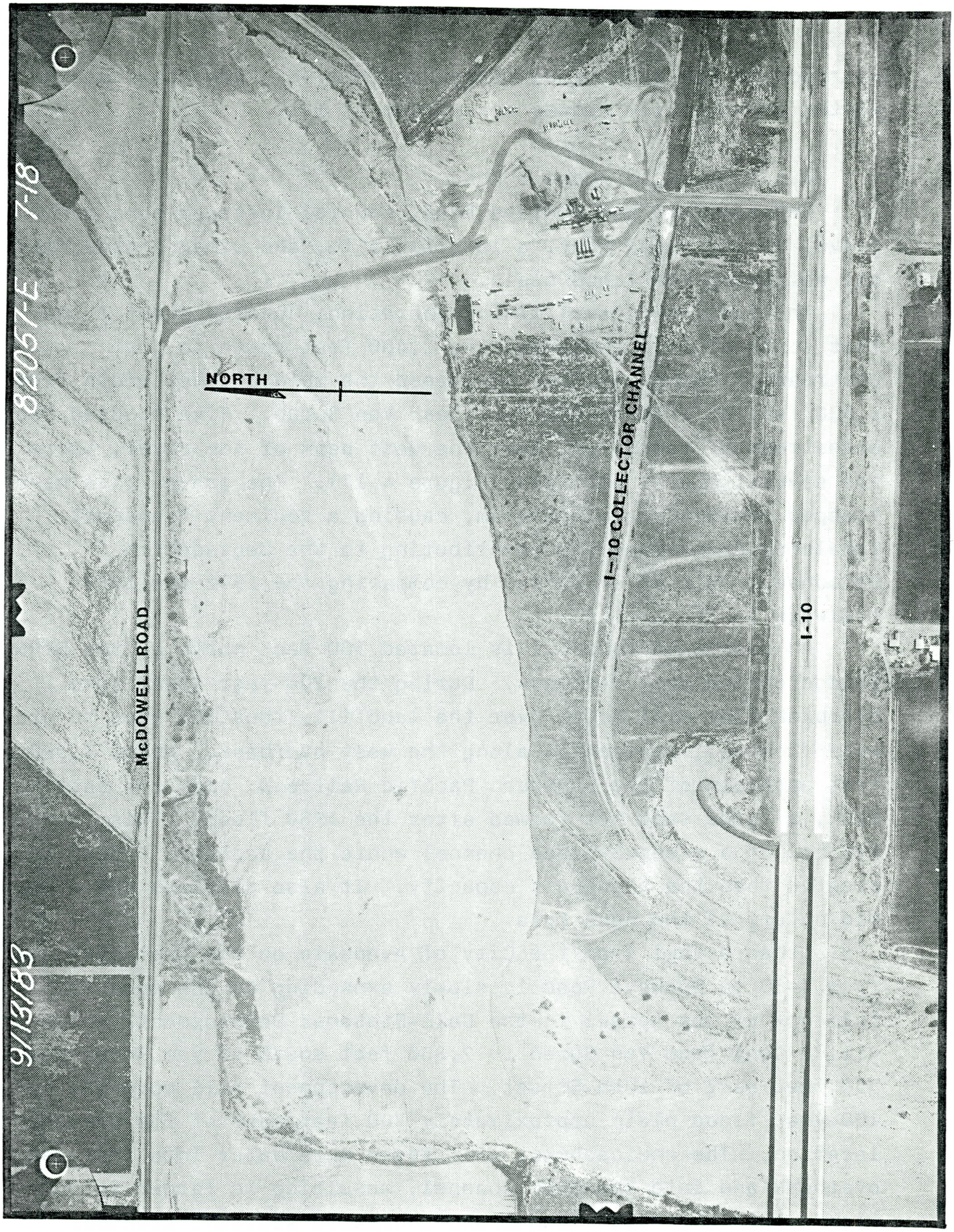


FIGURE 4.16
1983 AERIAL PHOTOGRAPH SHOWING I-10 AREA

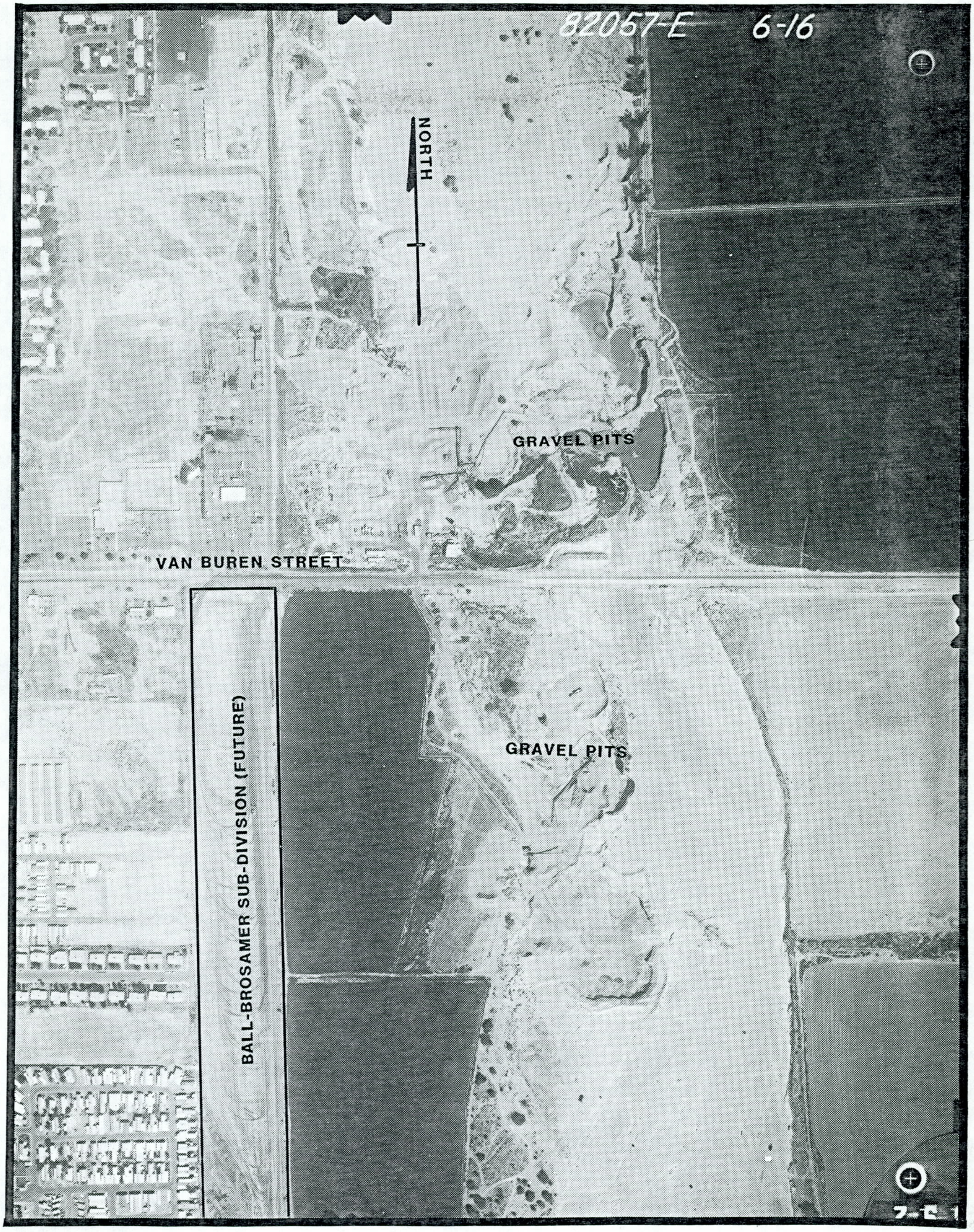


FIGURE 4.17 1983 AERIAL PHOTOGRAPH
SHOWING GRAVEL PITS NEAR VAN BUREN STREET

4.6.5 Buckeye Road to Broadway

Subdivisions border the Agua Fria on both the east and west overbanks between Buckeye Road and Broadway Road. Shallow flooding occurs on the west overbank during the 100-year flood from just below Buckeye Road to Harrison Drive about one-half block west of Dysart Road. The main channel widths vary from 600 feet to 1,400 feet in this reach. A rather severe dogleg to the west occurs just below Lower Buckeye Road.

Directly below Lower Buckeye Road on the west overbank, the City of Avondale operates a wastewater treatment plant (see Figure 4.18). The facility includes four lagoons and an infiltration basin and is located within the 100-year flood plain. The proposed floodwall below Buckeye Road on the west overbank will protect the City of Avondale's wastewater treatment plant and subdivisons.

4.6.6 Broadway to Confluence of Gila River

From Broadway Road to the confluence with the Gila River there is very little development along the Agua Fria. There are some cotton fields, but these have not significantly encroached upon the channel. Some breakout flow occurs below Broadway Road as described in Section 3.4.

4.7 Qualitative Geomorphic Summary

In general, the study reach has undergone significant man-related changes. There have been numerous sand and gravel mining operations, and urbanization and agricultural developments have encroached upon the flood plain within the study reach and contributed to the channel's degradation. This is apparent when comparing the 1972 and 1983 thalweg profiles. With the continued pressure of gravel miners and developers along the river, the degradation trend is expected to continue, thus the need to quantify the channel bed response to reasonably assess the future 100-year floodway.

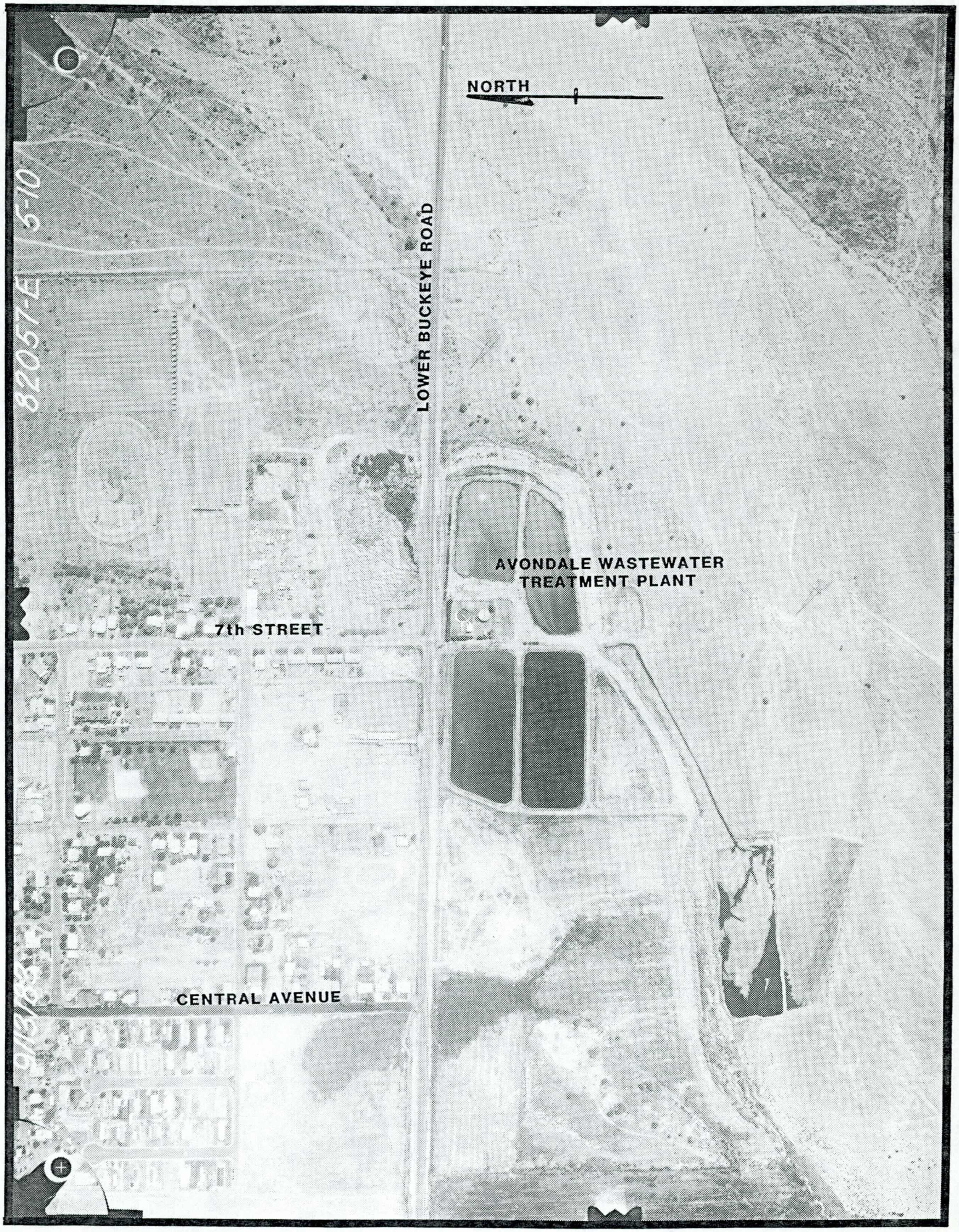


FIGURE 4.18 1983 AERIAL PHOTOGRAPH SHOWING AVONDALE'S WASTEWATER TREATMENT PLANT

V. QUANTITATIVE GEOMORPHIC ANALYSIS

5.1 General

The second level of analysis consists of identifying the channel's aggradation/degradation response considering the mechanics of sediment transport combined with the hydraulic conditions and bed-material characteristics of the Agua Fria River.

5.2 Short-Term Bed Response

The aggradation/degradation response within a channel is related to sediment transport capacity, which in turn is directly proportional to top width and proportional to velocity to approximately the fourth power. The short-term bed response is assessed by comparing the top width and velocity of the reach immediately upstream, because only the reach immediately upstream will significantly impact the downstream reach.

Table 5.1 shows the expected short-term bed responses for the 10- and 100-year flood peaks for each of the 10 reaches previously defined in Chapter III. The bed response for the reaches varies throughout the study area as expected when determining the short-term response. For instance, in Reach 3, which is between ISRB and the RID flume, the channel width narrows considerably from upstream of ISRB, resulting in increased velocities and potential degradation through the reach.

5.3 Long-Term Bed Response

Perhaps a more meaningful assessment of bed response is through long-term bed evaluation. For the long-term response, sediment transport capacities of all downstream reaches are compared with the supply reach rather than the reach immediately upstream. Over a long period, the system adjusts to meet the supply of the upstream reach that is in equilibrium.

Table 5.2 summarizes the long-term responses for the 10- and 100-year flood peaks. At the 10-year flood peak, most of the reaches exhibit a tendency to remain in equilibrium or degrade slightly, except for Reaches 2 and 9, which aggrade. At the 100-year peak, all the reaches exhibit a tendency to degrade.

Table 5.1. Expected Short-Term Bed Responses for the Agua Fria.

Reach	Change in Top Width		Change in Velocity		Overall Response	
	10-yr	100-yr	10-yr	100-yr	10-yr	100-yr
1	-	-	-	-	-	-
2	Increase	Decrease	Decrease	Increase	Slight Degrade	Degrade
3	Decrease	Decrease	Increase	Increase	Degrade	Degrade
4	Increase	Increase	Decrease	Decrease	Aggrade	Slight Degrade
5	Same	Increase	Increase	Decrease	Degrade	Aggrade
6	Increase	Increase	Decrease	Increase	Aggrade	Degrade
7	Same	Decrease	Same	Decrease	Equilibrium	Aggrade
8	Increase	Increase	Decrease	Same	Equilibrium	Degrade
9	Increase	Same	Decrease	Increase	Aggrade	Degrade
10	Decrease	Same	Increase	Decrease	Degrade	Aggrade

Reach 1: Glendale Avenue to confluence of New River
Reach 2: Confluence of New River to ISRB
Reach 3: ISRB to the RID flume
Reach 4: RID flume to Thomas Road
Reach 5: Thomas Road to 1,500 ft upstream of I-10
Reach 6: 1,500 ft upstream of I-10 to Van Buren Street
Reach 7: Van Buren Street to Buckeye Road
Reach 8: Buckeye Road to Lower Buckeye Road
Reach 9: Lower Buckeye Road to Broadway Road
Reach 10: Broadway Road to the confluence with Gila River

Table 5.2. Expected Long-Term Bed Responses For the Agua Fria.

Reach	Change in Top Width		Change in Velocity		Overall Response	
	10-yr	100-yr	10-yr	100-yr	10-yr	100-yr
1	-	-	-	-	-	-
2	Increase	Decrease	Slight Decrease	Increase	Slight Aggrade	Degrade
3	Decrease	Decrease	Increase	Increase	Degrade	Degrade
4	Decrease	Decrease	Increase	Increase	Degrade	Degrade
5	Decrease	Decrease	Increase	Increase	Degrade	Degrade
6	Decrease	Decrease	Increase	Increase	Equilibrium	Degrade
7	Decrease	Decrease	Increase	Increase	Equilibrium	Degrade
8	Decrease	Decrease	Increase	Increase	Equilibrium	Degrade
9	Decrease	Decrease	Slight Increase	Increase	Aggrade	Degrade
10	Decrease	Decrease	Increase	Increase	Equilibrium	Degrade

Reach 1: Glendale Avenue to confluence of New River
Reach 2: Confluence of New River to ISRB
Reach 3: ISRB to the RID flume
Reach 4: RID flume to Thomas Road
Reach 5: Thomas Road to 1,500 ft upstream of I-10
Reach 6: 1,500 ft upstream of I-10 to Van Buren Street
Reach 7: Van Buren Street to Buckeye Road
Reach 8: Buckeye Road to Lower Buckeye Road
Reach 9: Lower Buckeye Road to Broadway Road
Reach 10: Broadway Road to the confluence with Gila River

The overall tendency to degrade is consistent with the conclusion of the qualitative geomorphic analysis, which showed the channel bed has been degrading.

5.4 Sediment Transport Relationships

The Meyer-Peter, Mueller (MPM) bed-load equation, in combination with Einstein's integration of the suspended bed-material load, was used to determine the sediment transporting capacity of the Agua Fria River. No bed-material or suspended sediment load measurements have been made on the Agua Fria River or its tributaries to verify the accuracy of the sediment transport equations. However, the MPM and Einstein procedures have been used successfully on rivers with similar channel bed characteristics and are considered applicable for this study.

Transport of the bed-material load of a channel is divided into two zones. The sediment moving in a layer close to the bed is referred to as the bed load. The sediment carried in the remaining upper region of the flow is referred to as suspended load. The total bed-material load is the sum of the two quantities. The turbulent mixing process and the action of gravity on the sediment particles cause a continual transfer between the two zones. Although there is no distinct line between the zones, definitions are made in order to aid in the mathematical description of the process. A third type of load, the wash load, is also defined. It consists of fine particles that are not present in the bed in appreciable quantities, and will not easily settle out.

Sediment transport capacity is described as a power function of velocity, depth and top width. A regression of sediment transport capacities for a range of flow conditions and bed-material characteristics likely to occur in the Agua Fria was determined. The resultant sediment transport equation used for this study is:

$$Q_s = 8.61 \times 10^{-5} V^{3.7} H_y^{0.32} TW \qquad (1)$$

where

Q_s is the sediment-transport capacity (cfs)

V is the average flow velocity (fps)

H_y is the hydraulic depth (ft)

TW is the top width (ft).

The regression was derived for a river bed with a D_{50} of 1.0 mm and a gradation coefficient of 4.0. The gradation coefficient is a measure of the uniformity of the bed material and is defined as:

$$G = \frac{1}{2} \left(\frac{D_{84}}{D_{50}} + \frac{D_{50}}{D_{16}}\right) \qquad (2)$$

where

G is the gradation coefficient

D_{84}, D_{50}, D_{16} are the particle sizes for which the sediment mixture is finer.

The resultant sediment transport equation for the New River near the confluence with the Agua Fria is defined as:

$$Q_s = 4.55 \times 10^{-6} V^{4.13} H_y^{0.27} TW \qquad (3)$$

The D_{50} sediment size in the New River is considerably larger than that of the Agua Fria (approximately 30 mm).

Using Equation 1 in combination with the average hydraulics of the subreaches, the sediment transport capacities for the 10-, 25-, 50- and 100-year flood peaks are computed for the main channel and overbanks. Tables 5.3 through 5.6 summarize the sediment transport capacities for the main channel and left and right overbanks for the 10-, 25-, 50- and 100-year flood peaks, respectively.

The sediment transport capacity between ISRB and Thomas Road (Reaches 3 and 4) and between McDowell and Buckeye Road (Reaches 6 and 7) is significantly higher than the other subreaches of the river. The effective width of the main channel is narrower in these reaches, resulting in larger velocities and higher sediment transporting capacity.

Table 5.3. Sediment Transport Capacity for Agua Fria 10-Year Flood.

Reach	Left Overbank Water Discharge (cfs)	Left Overbank Sediment Discharge (cfs)	Main Channel Water Discharge (cfs)	Main Channel Sediment Discharge (cfs)	Right Overbank Water Discharge (cfs)	Right Overbank Sediment Discharge (cfs)
1	110	0	29,540	167	130	0
2	40	0	30,900	158	10	0
3	10	0	30,050	229	30	0
4	600	0	29,400	193	0	0
5	10	0	29,200	217	220	0
6	0	0	28,380	167	0	0
7	0	0	28,000	161	0	0
8	60	0	27,700	162	0	0
9	880	0	26,120	147	0	0
10	410	0	26,590	162	0	0

Note: Manning's n value of 0.025 used for sediment transport capacity computations.

Reach 1: Glendale Avenue to confluence of New River
Reach 2: Confluence of New River to ISRB
Reach 3: ISRB to the RID flume
Reach 4: RID flume to Thomas Road
Reach 5: Thomas Road to 1,500 ft upstream of I-10
Reach 6: 1,500 ft upstream of I-10 to Van Buren Street
Reach 7: Van Buren Street to Buckeye Road
Reach 8: Buckeye Road to Lower Buckeye Road
Reach 9: Lower Buckeye Road to Broadway Road
Reach 10: Broadway Road to the confluence with Gila River

Table 5.4. Sediment Transport Capacity for Agua Fria 25-Year Flood.

Reach	Left Overbank Water Discharge (cfs)	Left Overbank Sediment Discharge (cfs)	Main Channel Water Discharge (cfs)	Main Channel Sediment Discharge (cfs)	Right Overbank Water Discharge (cfs)	Right Overbank Sediment Discharge (cfs)
1	300	0	38,380	226	9,330	3
2	130	0	49,500	294	260	0
3	320	0	46,360	414	2,410	1
4	1,790	1	47,010	396	200	0
5	490	0	44,620	306	3,320	1
6	260	0	47,120	347	10	0
7	10	0	46,990	372	10	0
8	760	0	43,020	305	3,220	2
9	3,950	2	43,050	332	0	0
10	1,880	1	44,170	347	950	0

Note: Manning's n value of 0.025 used for sediment transport capacity computations.

Reach 1: Glendale Avenue to confluence of New River
Reach 2: Confluence of New River to ISRB
Reach 3: ISRB to the RID flume
Reach 4: RID flume to Thomas Road
Reach 5: Thomas Road to 1,500 ft upstream of I-10
Reach 6: 1,500 ft upstream of I-10 to Van Buren Street
Reach 7: Van Buren Street to Buckeye Road
Reach 8: Buckeye Road to Lower Buckeye Road
Reach 9: Lower Buckeye Road to Broadway Road
Reach 10: Broadway Road to the confluence with Gila River

Table 5.5. Sediment Transport Capacity for Agua Fria 50-Year Flood.

Reach	Left Overbank Water Discharge (cfs)	Left Overbank Sediment Discharge (cfs)	Main Channel Water Discharge (cfs)	Main Channel Sediment Discharge (cfs)	Right Overbank Water Discharge (cfs)	Right Overbank Sediment Discharge (cfs)
1	1,220	0	49,900	285	14,890	8
2	640	0	67,510	454	690	0
3	2,670	1	58,870	548	7,460	3
4	6,620	5	60,940	582	1,440	0
5	1,560	1	56,080	492	10,790	5
6	1,200	0	66,120	581	60	0
7	2,290	0	64,510	552	200	0
8	1,870	0	59,290	507	5,840	5
9	7,210	4	59,790	570	0	0
10	4,260	2	59,420	548	3,320	2

Note: Manning's n value of 0.025 used for sediment transport capacity computations.

Reach 1: Glendale Avenue to confluence of New River
Reach 2: Confluence of New River to ISRB
Reach 3: ISRB to the RID flume
Reach 4: RID flume to Thomas Road
Reach 5: Thomas Road to 1,500 ft upstream of I-10
Reach 6: 1,500 ft upstream of I-10 to Van Buren Street
Reach 7: Van Buren Street to Buckeye Road
Reach 8: Buckeye Road to Lower Buckeye Road
Reach 9: Lower Buckeye Road to Broadway Road
Reach 10: Broadway Road to the confluence with Gila River

Table 5.6. Sediment Transport Capacity for Agua Fria 100-Year Flood.

Reach	Left Overbank Water Discharge (cfs)	Left Overbank Sediment Discharge (cfs)	Main Channel Water Discharge (cfs)	Main Channel Sediment Discharge (cfs)	Right Overbank Water Discharge (cfs)	Right Overbank Sediment Discharge (cfs)
1	2,210	1	61,380	414	26,410	26
2	3,190	2	87,150	625	3,450	2
3	6,120	3	73,320	806	13,660	9
4	11,130	12	74,820	848	7,050	3
5	2,620	1	65,390	653	23,840	19
6	6,360	3	83,880	898	150	0
7	7,210	2	82,070	704	720	0
8	6,250	3	75,040	761	8,500	8
9	14,780	13	74,220	861	0	0
10	9,310	6	74,940	781	4,750	4

Note: Manning's n value of 0.025 used for sediment transport capacity computations.

Reach 1: Glendale Avenue to confluence of New River
Reach 2: Confluence of New River to ISRB
Reach 3: ISRB to the RID flume
Reach 4: RID flume to Thomas Road
Reach 5: Thomas Road to 1,500 ft upstream of I-10
Reach 6: 1,500 ft upstream of I-10 to Van Buren Street
Reach 7: Van Buren Street to Buckeye Road
Reach 8: Buckeye Road to Lower Buckeye Road
Reach 9: Lower Buckeye Road to Broadway Road
Reach 10: Broadway Road to the confluence with Gila River

5.4.1 Annual Sediment Yield from Agua Fria

The average annual sediment yield into the lower Agua Fria study reach was computed using a weighted incremental probability method. The annual sediment yield was computed for the supply reach from Glendale Avenue to the confluence with the New River.

The procedure involved establishing a sediment rating curve between water discharge and sediment discharge. This involved executing a multiple profile HEC-II run for a series of water discharges, and using the velocity, top width and hydraulic depth values for each water discharge to compute the sediment discharge (Equation 1). A regression was then computed using the water discharges and the sediment discharges, and the resultant rating curve for the supply reach on the Agua Fria is:

$$Q_s = 1.317 \times 10^{-4} \; Q^{1.36} \quad \text{for } Q < 30{,}000 \text{ cfs}$$

$$Q_s = 1.867 \times 10^{-2} \; Q^{0.879} \quad \text{for } Q > 30{,}000 \text{ cfs} \qquad (4)$$

where

Q_s is the sediment discharge (cfs)

Q is the water discharge (cfs)

The sediment volumes for the 10-, 25-, 50-, and 100-year flood hydrographs in the supply reach were computed using Equation 4. The sediment volumes were then weighted by the incremental probability of occurrence within a year, and by the average annual water volume divided by the incremental probabilistic occurrence water volume of the 10-, 25-, 50-, and 100-year flood hydrographs, to determine the average annual sediment yield. This is expressed mathematically as:

$$Q_{s_a} = \frac{Q_{w_{meas}}}{Q_{w_{inc}}} (0.01)(Q_{s_{100}}) + (0.01)(Q_{s_{50}}) + (0.02)(Q_{s_{25}}) + (0.06)(Q_{s_{10}}) \qquad (5)$$

where

Q_{s_a} is the average annual sediment volume

$Q_{w_{meas}}$ is the average annual water yield

$Q_{s_{100}}$, $Q_{s_{50}}$, $Q_{s_{25}}$, $Q_{s_{10}}$, are the sediment volumes of the 100-, 50-, 25-, and 10-year floods, respectively

$Q_{w_{inc}}$ is the weighted incremental water volume of the 100-, 50-, 25-, and 10-year floods and is computed as:

$$Q_{w_{inc}} = (.01)(Q_{100}) + (.01)(Q_{50}) + (.02)(Q_{25}) + (.06)(Q_{10}) \tag{6}$$

where

Q_{100}, Q_{50}, Q_{25}, and Q_{10}, are the water volumes of the 100-, 50-, 25-, and 10-year floods, respectively.

Table 5.7 summarizes the water and sediment yield volumes for the 10-, 25-, 50-, and 100-year floods from the supply reach. The average annual water yield measured at the Avondale gaging station (located just downstream of Buckeye Road bridge) during the years 1968 through 1972 and 1974 through 1980 was 26,810 acre-feet. The average annual water yield from the New River near Glendale for the years 1964 through 1968 was 8,110 acre-feet. Thus, subtracting the average water yield of the New River from the water yield at Avondale, the average water yield from the supply reach becomes 18,700 acre-feet. This assumes an insignificant lateral inflow to the Agua Fria between the New River and Buckeye Road. The average annual water yield value of 18,700 acre-feet is used in Equation 5 to compute the average annual sediment yield of 77.8 acre-feet from the supply reach of the Agua Fria. This translates into an average annual sediment concentration of 11,030 parts per million by weight.

Table 5.7. Water and Sediment Yields for the 10-, 25-, 50-, and 100-Year Floods for the Agua Fria Supply Reach.

Flood	Water Yield (acre-feet)	Sediment Yield (acre-feet)
10-year	69,500	271.7
25-year	112,000	476.8
50-year	154,600	674.5
100-year	212,860	938.7

Note: The shapes and durations of the 10-, 25-, and 50-year hydrographs are the same as the 100-year hydrograph of the Agua Fria near the confluence with the New River. Discharge values for the 10-, 25-, and 50-year hydrographs are scaled down according to the ratio with the peak of the 100-year flood.

5.4.2 Annual Sediment Yield of the New River

The average annual sediment yield from the New River was computed similarly to the sediment yield that enters the study reach from the Agua Fria. The supply reach of the New River starts at the confluence with the Agua Fria and extends to Glendale Avenue.

The sediment rating curve for the New River is:

$$Q_s = 1.505 \times 10^{-4} Q^{1.41} \tag{7}$$

Table 5.8 summarizes the water and sediment yields from the New River into the Agua Fria for the 10-, 25-, 50-, and 100-year floods. Using the values in Table 5.8 along with the annual water yield of 8,110 acre-feet, the average annual sediment yield from the New River becomes 5.4 acre-feet. This translates into an average annual sediment concentration of 1,760 parts per million by weight.

5.5 Equilibrium Slope

The equilibrium channel slope is defined as the slope at which the channel's sediment transport capacity is equal to the incoming sediment supply. Under this condition, the channel neither aggrades nor degrades. The equilibrium slope method is sometimes referred to as the dynamic equilibrium slope, because the gradient of the channel continually changes with upstream sediment supply.

The equilibrium slope analysis is usually determined for the dominant discharge in the river, or the discharge that most influences the cross-sectional shape. For the Agua Fria, this discharge is the bank-full discharge, which is hard to determine because of the multiple flow braids. The 10-year discharge of 31,000 cfs at Camelback Road was selected because most of the flow is contained within the banks at this discharge.

Table 5.9 summarizes, for each reach, the existing slope, the sediment transporting capacity for the 10-year flood peak, the average hydraulics, and the equilibrium slope to which the

Table 5.8. Water and Sediment Yields for the 10-, 25-, 50-, and 100-Year Floods from the New River.

Flood	Water Yield (acre-feet)	Sediment Yield (acre-feet)
10-year	5,180	2.6
25-year	8,630	5.4
50-year	12,950	9.6
100-year	18,700	16.0

Note: The shapes and durations of the 10-, 25-, and 50-year hydrographs are the same as the 100-year hydrograph of the New River near the confluence with the Agua Fria. Discharge values for the 10-, 25-, and 50-year hydrographs are scaled down according to the ratio with the peak of the 100-year flood.

Table 5.9. Summary of the Equilibrium Slope Analysis for the 10-year Peak Discharge.

Reach	Description of Reach	Top* Width (ft)	Water Discharge (cfs)	Velocity* (fps)	Sediment Discharge (cfs)	Concentration by Weight (ppm)	Slope Exist	Slope Equilibrium
1	Glendale Avenue to the confluence with New River	1,695	29,800	6.1	167	14,850	0.0022	0.0022
2	Confluence with New River to ISRB	1,825	30,900	5.9	158	13,550	0.0027	0.0029
3	ISRB to the RID flume	660	30,100	8.1	229	20,160	0.0023	0.0017
4	RID flume to Thomas Road	800	30,000	7.4	193	17,050	0.0019	0.0016
5	Thomas Road to 1,500 ft upstream of I-10	825	29,400	7.7	217	19,560	0.0021	0.0016
6	1,500 ft. upstream of I-10 to Van Buren Street	1,050	28,400	6.8	167	15,580	0.0021	0.0021
7	Van Buren Street to Buckeye Road	1,015	28,000	6.7	161	15,240	0.0017	0.0018
8	Buckeye Road to lower Buckeye Road	1,205	27,800	6.5	162	15,440	0.0023	0.0024
9	Lower Buckeye Road to Broadway Road	1,265	27,000	6.3	147	14,430	0.0025	0.0028
10	Broadway Road to the confluence with Gila River	1,200	27,000	6.6	162	15,900	0.0025	0.0026

*Main channel velocities and top widths based on Manning n = 0.025 for backwater profiles in the main channel.

reach will adjust. Most of the slopes within the reaches are near the equilibrium slope with the exception of Reaches 3, 4, and 5. These reaches exhibit a large degradation potential, which is expected because of the narrow cross-sectional width in these areas.

5.6 Armor Control Analysis

The equilibrium slopes shown in Table 5.9 were computed based on the existing sediment supply from Reach 1 (Camelback Road to Glendale Avenue). This sediment supply, however, may be reduced due to river armoring because of trapping of sediment in Waddell Dam and trapping of sediment in gravel pits. Thus, the Agua Fria upstream of the confluence with the New River should be monitored to assess if an armor layer forms on the bed, ultimately reducing the sediment supply into the study reach.

Table 5.10 shows approximate critical velocities for transporting fine to coarse gravels. This table was prepared using the Shields criterion for incipient motion of sediment particles. As can be seen from the table, to initiate incipient motion for the coarse and very coarse gravels requires velocities exceeding five and seven fps, respectively. Previous hydraulic analyses showed the main channel velocities ranged from four to six fps for the 10-year flood and from 5 to 8.5 fps for the 100-year flood in the sediment supply reach. Therefore, the armoring potential of coarse gravel and larger particles in the upstream supply reach can be significant based on the flow velocity, the critical velocity for incipient motion, and the availability of these particles.

Although armoring of the entire supply reach is unlikely, sediment can be reduced due to bed material coarsening or partial armoring. To account for the possible future sediment supply reduction, the equilibrium slopes were re-evaluated assuming a 25 percent reduction in upstream sediment supply. The resultant equilibrium slopes are shown in Table 5.11. The degradation problems become more prominent under the reduced supply condition.

Table 5.10. Critical Velocities for Incipient Motion of Sediment Particles.

Sediment Type	Size (mm)	Critical Velocity (fps)
Very fine gravel	2 - 4	1.8
Fine gravel	4 - 8	2.5
Medium gravel	8 - 16	3.5
Coarse gravel	16 - 32	5.0
Very coarse gravel	32 - 64	7.0

Table 5.11. Equilibrium Slopes Considering a 25 Percent Reduction of Sediment from the Supply Reach for the 10-Year Peak Discharge.

Reach	Slope Existing	Sediment Transport Capacity (cfs)	Slope Equilibrium
1	0.0022	125	0.0022
2	0.0027	158	0.0021
3	0.0023	229	0.0013
4	0.0019	193	0.0012
5	0.0021	217	0.0012
6	0.0021	167	0.0016
7	0.0017	161	0.0013
8	0.0023	162	0.0018
9	0.0025	147	0.0021
10	0.0025	162	0.0019

Reach 1: Glendale Avenue to confluence of New River
Reach 2: Confluence of New River to ISRB
Reach 3: ISRB to the RID flume
Reach 4: RID flume to Thomas Road
Reach 5: Thomas Road to 1,500 ft upstream of I-10
Reach 6: 1,500 ft upstream of I-10 to Van Buren Street
Reach 7: Van Buren Street to Buckeye Road
Reach 8: Buckeye Road to Lower Buckeye Road
Reach 9: Lower Buckeye Road to Broadway Road
Reach 10: Broadway Road to the confluence with Gila River

Other armor control analyses performed include computation of the static equilibrium slope and the determination of the armoring potential by particle size. These methods are discussed in the following sections.

5.6.1 Static Equilibrium Slope

The static equilibrium slope method utilizes Shields' relationship for incipient motion and assumes there is coarse material available in the subsurface to resist movement for larger flows. Approximately 10 percent of the material two inches or more in diameter is located in the subsurface layer near the ISRB. Assuming, as test pits seem to indicate, that this percentage of material is indicative of the material found in the downstream reaches, an armor layer will develop. Using Shields' relation,

$$\tau_c = 0.047\ (\gamma_s - \gamma)\ d_s \qquad (8)$$

where

τ_c is the critical shear stream stress initiating particle movement

γ_s is the unit weight of sediment

γ is the unit weight of water

d_s is the representative particle size

and assuming the two-inch material is representative of the armor material, a critical shear can be computed. Equating the shear stress in the Darcy-Weisbach resistance equation to the critical shear stress,

$$\tau = \tau_c = \frac{1}{8} \rho\ f\ V^2 \qquad (9)$$

where

τ is the bed shear stress

ρ is the density of water

f is the Darcy-Weisbach friction factor, and

V is the flow velocity

the average flow velocity can be computed. Assuming the flow in

Table 5.12. Static Equilibrium Slope Analysis, 100-year Discharge.

Reach	Existing Slope	Top Width (ft)	Water Discharge (cfs)	Unit Discharge (cfs/ft)	Static Equilibrium Slope
1	0.0022	2,135	61,380	28.8	0.0026
2	0.0027	2,020	87,150	43.1	0.0015
3	0.0023	685	73,320	107.0	0.0005
4	0.0019	835	74,820	89.6	0.0006
5	0.0021	910	65,390	71.9	0.0008
6	0.0021	1,190	83,880	70.5	0.0008
7	0.0017	1,075	82,070	76.3	0.0007
8	0.0023	1,240	75,040	60.5	0.0010
9	0.0025	1,285	74,220	57.8	0.0010
10	0.0025	1,245	74,940	60.2	0.0010

Reach 1: Glendale Avenue to confluence of New River
Reach 2: Confluence of New River to ISRB
Reach 3: ISRB to the RID flume
Reach 4: RID flume to Thomas Road
Reach 5: Thomas Road to 1,500 ft upstream of I-10
Reach 6: 1,500 ft upstream of I-10 to Van Buren Street
Reach 7: Van Buren Street to Buckeye Road
Reach 8: Buckeye Road to Lower Buckeye Road
Reach 9: Lower Buckeye Road to Broadway Road
Reach 10: Broadway Road to the confluence with Gila River

each reach can be approximated by normal depth, and the wide channel approximation is used, the slope of each reach can be computed using Manning's relation.

$$S = \left(\frac{V^{5/3}\, n}{1.48\, q^{2/3}}\right)^2 \qquad (10)$$

where

S is the bed slope

V is the flow velocity

n is the Manning flow resistance

q is the unit width discharge

Table 5.12 summarizes the static equilibrium slopes computed for the 100-year discharge for each reach assuming a two-inch armor material will form on the surface. Reaches 3 and 4 have extremely flat static equilibrium slopes due to the large unit discharge that occurs within the these reaches. It is highly probable that the banks will become unstable and start to slough before degradation of this magnitude will occur. Therefore, unless the banks are stabilized and toe-down protection is extended significantly below the present bed, static equilibrium slopes will not be achieved in these reaches.

5.6.2 Particle Armoring Method

The particle size armoring method assumes an armor layer will develop when a layer twice the diameter of the largest non-moving sediment particle forms on the channel bed. The degradation potential can be expressed mathematically as:

$$D_{sc} = \frac{2\, d_s}{P_c} \qquad (11)$$

where

D_{sc} is the depth of scour for 1 layer deep.

d_s is the size of armoring material

P_c is the percent of material coarser than the armoring size.

Using two inches as the armoring size material and the subsurface size distribution shown in Figure 4.2, the armor layer for Reaches 1 through 10 will develop at a depth of 11.6 feet. This assumes erosion of a five-foot surface layer before armoring begins to develop in the subsurface layer.

5.7 Controlling Bed Response

Table 5.13 summarizes the expected aggradation/degradation response along the study reach for the dynamic equilibrium, static equilibrium and particle armoring size methodologies. The dynamic equilibrium slope will control the grade at all reaches except the supply reach. The dynamic equilibrium slope is based on the present sediment supply into the study reach. Should armoring fully develop between Glendale and the New River, the equilibrium slopes will become flatter than those predicted in Table 5.13. Should the dynamic equilibrium slopes flatten to the values reported in Table 5.11, the channel bed will become entrenched and the banks will become unstable.

5.8 Local Scour Analysis

5.8.1 Local Scour at River Crossings

Local scour around bridge piers was evaluated using Shen's and Neil's equations. These equations were empirically developed from extensive test data on sand-bed channels and provide reasonable approximations for local scour depths. Shen's equation takes the following form:

$$d_s = k\ 0.00073\ R^{0.619} \tag{12}$$

where

d_s is the local scour depth

k is a multiplying factor to account for skew of piers (see Table 5.14)

R is the pier Reynolds number

$$R = \frac{Va}{\nu} \tag{13}$$

Table 5.13. Comparison of Bed Response Using Different Methods.

Reach	Description of Reach	Reach Length (ft)	Existing Slope	Dynamic Equilibrium Slope	Static Equilibrium Slope	Particle Armoring Method (ft)
1	Glendale Avenue to the confluence with New River	8,337	0.0022	0.0022	0.0026	11.6
2	Confluence with New River to ISRB	8,049	0.0027	0.0029	0.0015	11.6
3	ISRB to the RID flume	2,301	0.0023	0.0017	0.0005	11.6
4	RID flume to Thomas Road	3,580	0.0019	0.0016	0.0006	11.6
5	Thomas Road to 1,500 ft upstream of I-10	7,975	0.0021	0.0016	0.0008	11.6
6	1,500 ft upstream of I-10 to Van Buren Street	4,632	0.0021	0.0021	0.0008	11.6
7	Van Buren Street to Buckeye Road	5,470	0.0017	0.0018	0.0007	11.6
8	Buckeye Road to Lower Buckeye Road	5,075	0.0023	0.0024	0.0010	11.6
9	Lower Buckeye Road to Broadway Road	6,230	0.0025	0.0028	0.0010	11.6
10	Broadway Road to the confluence with Gila River	7,155	0.0025	0.0026	0.0010	11.6

Table 5.14. Multiplying Factors* for Depth of Scour d_s for Skewed Piers.

Horizontal Angle of Attack (degrees)	Length-to-Width Ratio of Pier in Flow			
	4	8	12	16
0	1.0	1.0	1.0	1.0
15	1.5	2.0	2.5	3.0
30	2.0	2.5	3.5	4.5
45	2.5	3.5	4.5	5.0
60	2.5	3.5	4.5	6.0

*Simons, D.B. and Senturk, F., Sediment Transport Technology, Water Resource Publications, 1977.

where

V is the average flow velocity upstream of the bridge pier

a is the diameter of the bridge pier

ν is the kinematic viscosity of the water

Neil's equation takes the following form:

$$d_s = d_w \, (2) \left(\frac{a}{d_w}\right)^{0.65} F_r^{0.43} \, k \qquad (14)$$

where

k is a multiplying factor to account for skew of piers (see Table 5.14)

d_w is the depth of water

a is the pier diameter

F_r is the Froude number

Results of local scour computations for the 100-year peak discharge are shown in Table 5.15. Velocity, depth and flow skew used for local scour computations are included in the table. To account for debris accumulation near bridge piers, two feet was added to either side of the piers, or equivalently a total of four feet to each bridge pier.

The largest potential local scour occurs at the SPRR bridge and at the ISRB. At the SPRR bridge, the large pier obstruction width, in combination with the angle of attack of the flow (10°), results in a local scour potential of 29.6 feet for the 100-year peak discharge. This approaches the depth of burial of the piles, and thus protection is required. At ISRB, the 100-year local scour potential is 27.9 ft. The severe angle of attack (30°), in combination with the pier length (70 ft), results in the large potential local scour at the ISRB. Since the existing thalweg elevation is 23 ft above the bottom of the bridge footing, and the local scour is 27.9 ft, protection of the ISRB piers is required for the 100-year flood. Local scour protection is also necessary at the RID flume and Buckeye Road Bridge crossings based on computed 100-year local scour depths at these locations.

Table 5.15. Approximate Local Scour Depths at Bridge Crossings for the 100-Year Flood.

River Crossing	Pier Diameter or Width (ft)	Velocity (fps)	Depth (ft)	Flow Skew (°)	Skew Factor	Local Scour Depth Shen (ft)	Local Scour Depth Neil (ft)	Average Scour Depth (ft)
Camelback Road	4	9.9	5.8	5	1.0	13.6	12.4	13.0
Indian School Road	1.67	6.7	12.6	30	3.1	26.8	29.0	27.9
Roosevelt Irrigation District Flume	4	12.3	12.6	0	1.0	15.6	15.2	15.4
Interstate-10	3.33	10.2	12.7	5	1.0	13.1	13.2	13.2
Southern Pacific Railroad	7.5	10.3	10.7	10	1.7	29.7	29.6	29.6
Buckeye Road	3	12.7	8.8	10	1.7	24.8	22.8	23.8

5.8.2 Local Scour Around Bridge Abutments

Abutment scour was evaluated for Camelback Road, ISRB, I-10, SPRR and Buckeye Road Bridges using Liu's equation, which is:

$$S = d_1 \, (1.1) \left(\frac{a}{d_1}\right)^{0.4} \quad F_{r_1}^{\;0.33} \qquad (15)$$

where S is the abutment scour depth

d_1 is the upstream depth

a is the embankment length (measured normal to the abutment)

F_{r1} is the Froude number

The results of these computations are shown in Table 5.16. Abutment scour was not computed at the RID flume because levees have narrowed the cross section, and the abutments are no longer exposed to flow. The local scour potential at the ISRB necessitates protection for the east abutment. All other bridge abutments appear to be able to withstand the 100-year flood.

5.8.3 Local Scour Around Transmission Towers

Local scour was computed using Shen's and Neil's equations for the Tucson Electric Power Company and Salt River Project transmission towers located within the 100-year flood plain. Tucson Electric Power Company has 36 transmission towers within the 100-year flood plain. The towers vary from 80 to 105 feet above the ground and have five-foot diameter pier footings. The locations of the towers are shown in Plates 1 through 4, attached to this report. Several of the towers have been reinforced with sheet pile. The obstruction width for the reinforced tower legs is 10 ft. Table 5.17 summarizes for each tower the obstruction width of each footing, the 100-year flow velocity and flow depth, the 100-year local scour depth as computed using Shen's and Neil's equations, the adopted local scour, the approximate ground elevation near the tower and the expected elevation after local scour. Depth of footings is unknown, thus comments cannot be made regarding the safety of existing towers.

Table 5.16. Local Scour at Bridge Abutments for 100-Year Flood Peak.

	Camelback Road		ISRB		I-10		SPRR		Buckeye	
	East Abutment	West Abutment	East Abutment	West Abutment	East Abutment	West Abutment	East Abutment	West Abutment	East Abutment	West Abutment
Abutment protrusion to flow (ft.)	33	33	12	16	15	22	16	13	23	22
Water Surface Elevation (ft.)	1,023.3	1,023.3	1,012.7	1,012.7	980.3	980.3	963.9	963.9	962.5	962.5
River Bed Elevation near Abutment (ft.)	1,017.4	1,017.4	1,002	1,006	975	976	960	956	958	954
Water Depth near Abutment (ft.)	5.9	5.9	10.7	6.7	5.3	4.3	3.9	7.9	4.5	8.5
Velocity (fps)	9.9	9.9	6.7	-	10.2	10.2	10.3	10.3	12.7	12.7
Froude No.	0.72	0.72	0.36	-	0.78	0.87	0.92	0.65	1.06	0.77
Local Scour (ft.)	9.0	9.0	12.1	-	7.1	6.0	5.5	9.9	6.7	11.3

[1]The flow is ineffective near the west abutment of ISRB and therefore the local scour is negligible.

Table 5.17. Local Scour Around Tucson Electric Power Company Transmission Towers for the 100-year Flood Peak.

Tower Number	Obstruction Width (ft)	Velocity (fps)	Depth (ft)	Local Scour Shen's Method (ft)	Local Scour Neil's Method (ft)	Local Scour Adopted Value (ft)	Natural Ground Elevation	Elevation After Scour
74	5	6.8	9.7	10.4	10.0	10.2	1,023.1	1,012.9
75	5	6.6	8.2	10.2	9.6	9.9	1,022.6	1,012.7
76	5	4.0	3.1	7.5	6.8	7.2	1,026.5	1,019.3
77	5	3.0	4.8	6.2	6.4	6.3	1,022.0	1,015.7
81	5	2.0	2.2	4.9	4.8	4.9	1,011.0	1,006.1
82	5	1.9	5.2	4.7	5.3	5.0	1,007.5	1,002.5
83	5	3.4	2.4	6.7	6.1	6.4	1,006.0	999.6
84	5	3.0	5.6	6.2	6.5	6.4	1,004.0	997.6
86	5	2.0	2.4	4.9	4.9	4.9	999.0	994.1
87	5	2.6	3.1	5.7	5.6	5.7	996.3	990.6
88(R)	10	3.0	1.4	9.6	8.5	9.0	995.5	986.5
89(R)	10	2.8	2.9	9.2	9.1	9.2	993.0	983.8
90	5	3.1	1.5	6.4	5.5	5.9	992.0	986.1
91(R)	10	3.5	6.3	10.5	11.1	10.8	985.0	974.2
92	5	3.2	4.7	6.5	6.5	6.5	985.0	978.5
93	5	2.6	5.4	5.7	6.1	5.9	981.0	975.1
94	5	3.3	4.5	6.6	6.6	6.6	980.0	973.4
95	5	2.2	7.5	5.2	5.9	5.6	977.0	971.4
96(R)	10	8.6	4.3	18.4	15.5	17.0	979.0	962.0
97	5	8.3	3.0	11.7	9.3	10.5	977.0	966.5
98	5	3.5	1.7	6.9	5.9	6.4	975.0	968.6
99(R)	10	2.7	3.8	9.0	9.2	9.1	970.0	960.9
100(R)	10	9.2	8.3	19.2	17.4	18.3	962.0	943.7
101(R)	10	11.9	7.0	22.5	19.0	20.7	962.0	941.3
102(R)	10	7.4	7.8	16.8	15.7	16.3	959.0	942.7
103(R)	10	9.0	7.9	18.9	17.1	18.0	956.0	938.0
104	5	11.8	8.7	14.6	12.4	13.5	951.0	937.5
105	5	6.9	8.6	10.5	9.9	10.2	948.5	938.3
106	5	7.6	6.9	11.1	10.0	10.6	948.0	937.4
107	5	7.8	9.7	11.3	10.6	11.0	942.0	931.0
108	5	8.6	9.7	12.0	11.0	11.5	937.4	925.9
109	5	9.6	9.6	12.8	11.5	12.2	934.0	921.8
110	5	9.8	7.8	13.0	11.3	12.2	931.3	919.1
111	5	7.7	10.2	11.2	10.6	10.9	926.3	915.4
112	5	9.8	14.4	13.0	12.3	12.7	920.0	907.3
113	5	6.8	13.8	10.4	10.4	10.4	920.1	909.7

Note: Velocities are increased 50% in local scour computations to account for acceleration around tower legs.

The Salt River Project has 19 transmission towers located within the 100-year flood plain. The footings of these towers are three feet in diameter. Table 5.18 summarizes for each tower the obstruction width of each footing, the 100-year flow velocity and flow depth, the 100-year local scour depth as computed using Shen's and Neil's equations, the adopted local scour, the approximate ground elevation near the tower and the expected elevation after local scour.

No local scour computations were made for the abandoned 161 kV transmission line of the Department of Energy that crosses the Agua Fria near Thomas Road.

Table 5.18. Local Scour Around Salt River Project Transmission Towers for the 100-Year Flood.

Tower Number	Obstruction Width (ft)	Velocity (fps)	Depth (ft)	Local Scour Shen's Method (ft)	Local Scour Neil's Method (ft)	Local Scour Adopted Value (ft)	Natural Ground Elevation	Elevation After Scour
43	3	6.8	9.7	7.6	7.1	7.4	1,023.1	1,015.7
44	3	6.6	8.2	7.4	6.9	7.2	1,022.6	1,015.4
45	3	4.0	3.1	5.4	4.9	5.2	1,026.5	1,021.3
46	3	3.0	4.8	4.5	4.6	4.6	1,022.0	1,017.4
49	3	1.8	1.0	3.3	3.0	3.2	1,016.5	1,013.3
51	3	2.2	0.6	3.8	3.0	3.4	1,013.0	1,009.6
52	3	1.9	3.0	3.4	3.5	3.5	1,010.0	1,006.5
54	3	3.4	2.4	4.9	4.4	4.7	1,007.5	1,002.8
55	3	3.0	5.6	4.5	4.7	4.6	1,002.0	997.4
57	3	2.2	0.4	3.8	2.9	3.4	1,002.0	998.6
58	3	2.4	2.2	4.0	3.7	3.8	998.0	994.2
59	3	2.6	3.4	4.2	4.1	4.2	996.0	991.8
60	3	9.7	12.0	9.4	8.6	9.0	990.0	981.0
61	3	10.3	11.8	9.8	8.8	9.3	990.0	980.7
62	3	12.3	12.4	10.9	9.5	10.2	990.0	979.8
63	3	9.6	11.8	9.3	8.5	8.9	992.0	983.1
64	3	2.0	2.7	3.5	3.6	3.6	986.0	982.4
65	3	2.2	8.0	3.8	4.3	4.1	980.0	975.9
66	3	2.9	8.4	4.5	4.9	4.7	979.0	970.3

Note: Velocities are increased 50% in local scour computations to account for acceleration around tower legs.

VI. MATHEMATICAL MODEL ANALYSIS

6.1 General

The third level of analysis is the mathematical model simulation of the lower Agua Fria floodway as defined by the COE. The mathematical model quantifies the channel bed response to the 100-year flood. QUASED, the SLA-developed water and sediment routing model, was used to simulate the floodway response.

In using the QUASED model, the river is subdivided into a series of computational reaches. Each of the subreaches is selected as a portion of the river where the hydraulic and geomorphic characteristics are similar. For this study, each subreach had sediment discharge input from the upstream portion of the river. Hydraulic conditions were calculated using the COE's HEC-II water-surface profile program.

6.2 General Model Concept

The amount of material transported or deposited in a channel reach is the result of the interaction of two processes. The first is the transport capacity of the reach. This is determined in part by the hydraulic conditions, which are a direct result of the water discharge, channel configuration, channel resistance and the sediment sizes present. Smaller particles can be transported at larger rates than larger particles under the same flow conditions. The second process is the supply of sediment entering the reach, which is determined by the nature of the channel and watershed above the study reach.

When sediment supply is less than sediment transport, sediment is removed from the channel bed and banks to reduce the difference. This results in degradation of the channel and possible failure of the banks. If the supply entering the reach is greater than the capacity, the excess supply is deposited, causing aggradation.

6.2.1 Sediment Routing Procedure

The sediment routing procedure is quasi-dynamic, in that the flow is assumed constant for a given time increment but varies from subreach to subreach. The flood hydrograph is broken into a number of time increments, each with a different flow, but during each increment the flow is considered steady. To account for the moveable nature of the alluvial boundary, the cross sections are recomputed at the end of each time interval. Sediment transport by size fraction is determined for the overbanks and main channel portions of the cross section, then summed to give the total transport capacity within a subreach.

The volume of aggradation or degradation within a subreach is computed as a function of the difference between the sediment inflow from upstream and the transport capacity of the subreach. This volume is translated to a change in bed elevation at each cross section which is used to generate new HEC-II data for the next time step.

6.2.2 Armoring

For this study the particle size range is large, necessitating the consideration of the armoring process for realistic determination of the river response.

The QUASED model determines the transport capacity of the channel by size fractions. This not only provides increased accuracy in determining the sediment discharge, but also allows for simulation of the variation in the particle size distribution during the degradation or aggradation process. If the channel degrades and particles too large to be transported by the flow are present in the bed material, the finer particles will be removed, leaving behind the larger particles and producing a layer of essentially non-transportable material (the armor layer). When this occurs, the amount of degradation in the channel is controlled by the quantity of large particles present.

6.3 Model Application for 100-Year Flood

The QUASED model has been verified by simulating the channel response to the December 1978, January 1979 and February 1980 floods on the Agua Fria. The results of the model verification are documented in the report "Hydraulic and Geomorphic Analysis of the Agua Fria River," submitted to the Flood Control District of Maricopa County by SLA, September 13, 1983.

The QUASED model simulated the channel trends accurately, therefore the reliability of the model to predict the floodway responses to the 100-year flood should be adequate.

The 100-year hydrograph was discretized into 17 time steps and the study reach was subdivided into the 10 subreaches as defined in Chapter III. Figure 6.1 shows the discretized hydrograph.

The sediment routing results indicate that minor degradation of the floodway is the dominant response to the 100-year flood throughout the study reach. The largest degradation occurs in Reach 3, between ISRB and the RID flume, because of the large sediment transporting capacity of the channel in comparison to the sediment supply from upstream of ISRB. Degradation in this reach ranges from 1.4 feet to 3.9 feet.

Slight aggradation occurs downstream of Reach 3, due to the large supply of sediment entering Reach 4. The aggradation tendency in Reach 4 is the result of material depositing on the recession limb of the hydrograph.

Figures 6.2 and 6.3 compare the bed response to the original channel invert elevation at the peak discharge and at the end of the flood. A tabulation of bed elevations at the flood peak and at the end of the hydrograph is provided in Table 6.1.

The simulation of the 100-year flood indicates the dynamic nature of the channel bed of the Agua Fria. Further, the degradation tendency indicates that the actual floodway may be narrower than the rigid-bed HEC-II-defined floodway.

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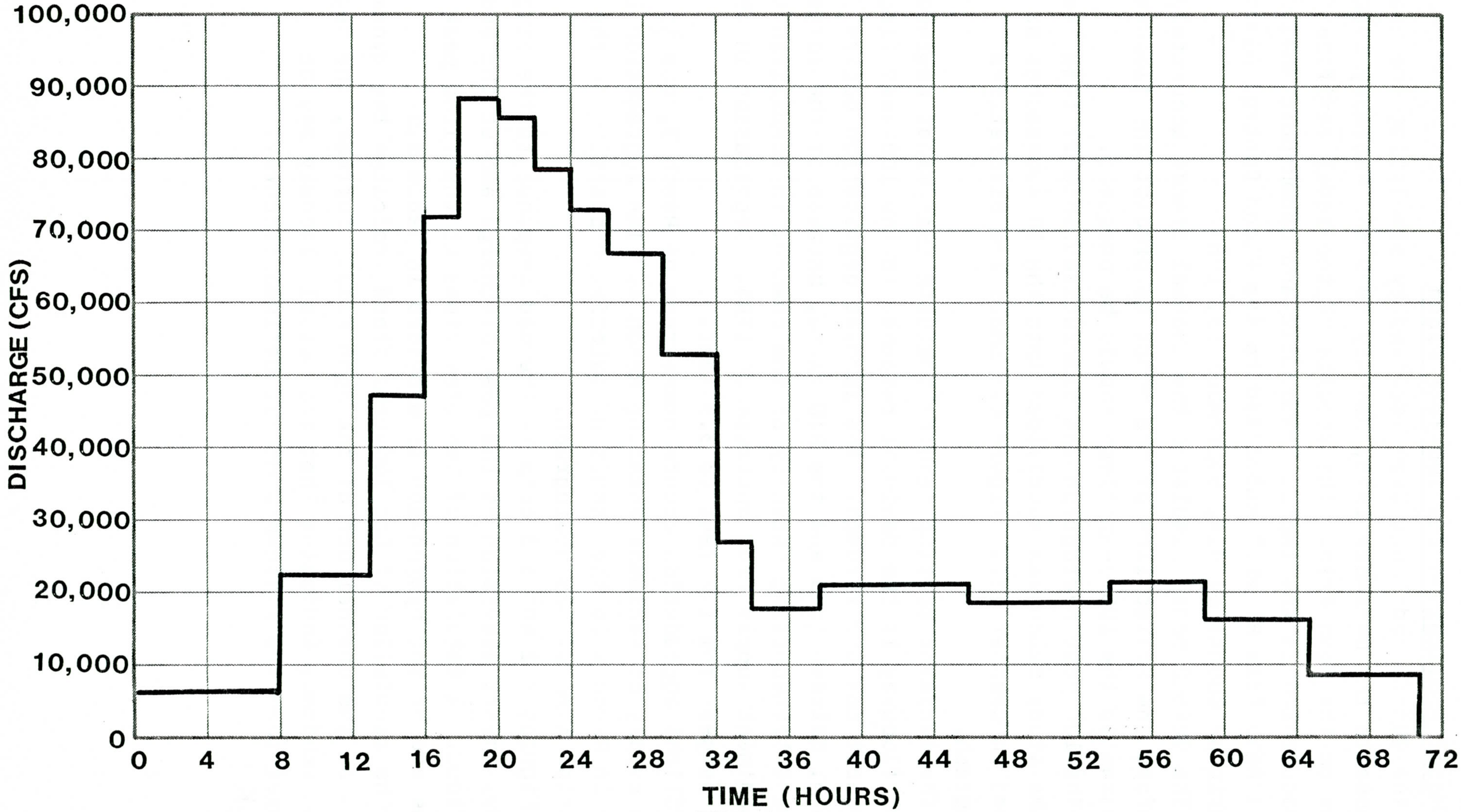


FIGURE 6.1 DISCRETIZED 100—YR HYDROGRAPH USED TO SIMULATE THE FLOODWAY RESPONSE

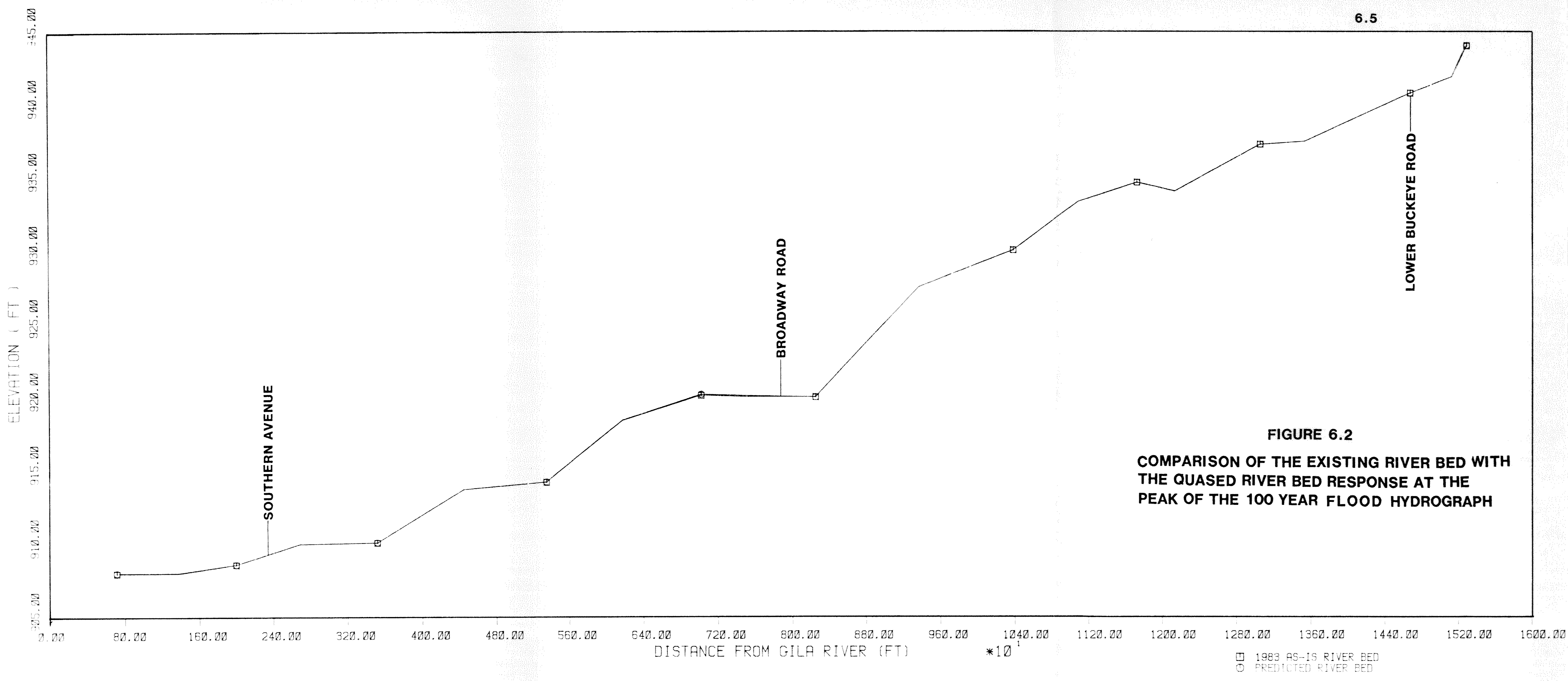


FIGURE 6.2

COMPARISON OF THE EXISTING RIVER BED WITH THE QUASED RIVER BED RESPONSE AT THE PEAK OF THE 100 YEAR FLOOD HYDROGRAPH

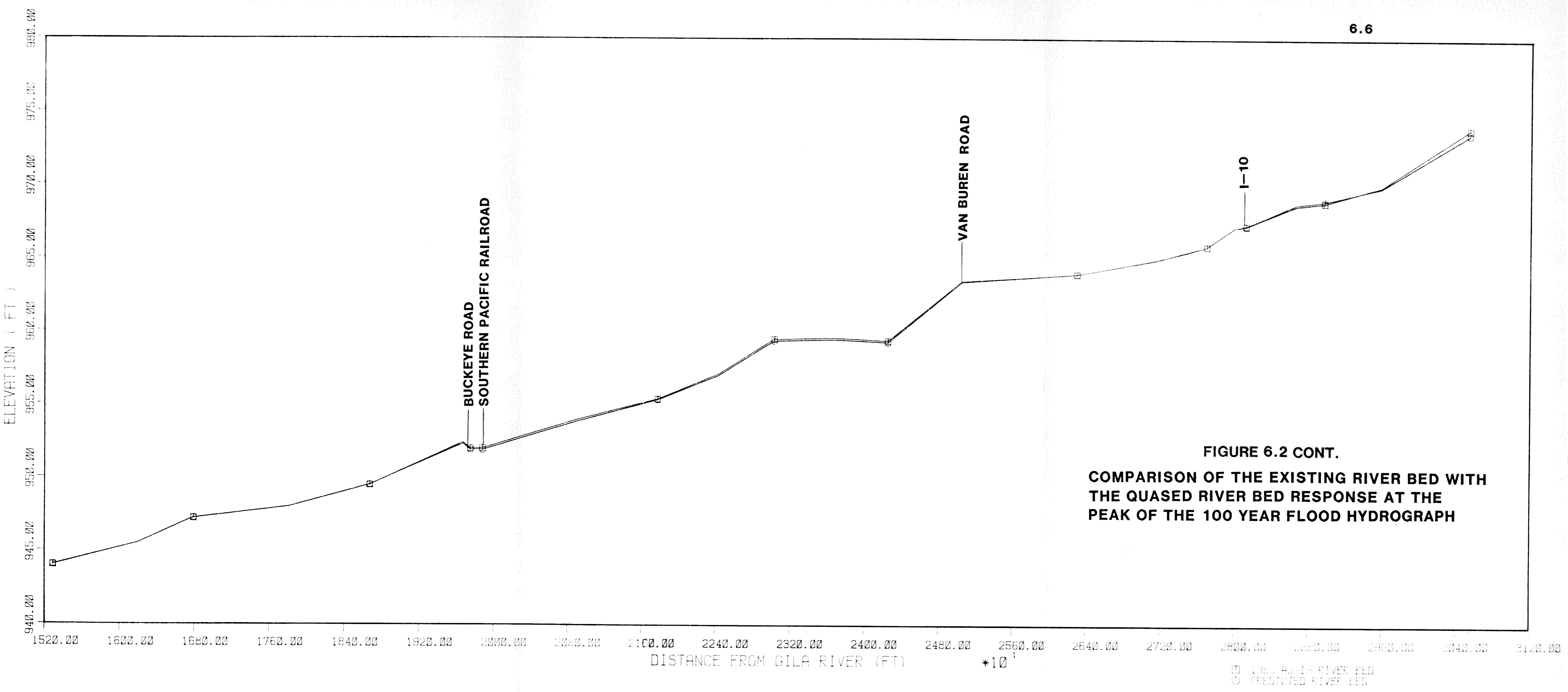


FIGURE 6.2 CONT.

COMPARISON OF THE EXISTING RIVER BED WITH THE QUASED RIVER BED RESPONSE AT THE PEAK OF THE 100 YEAR FLOOD HYDROGRAPH

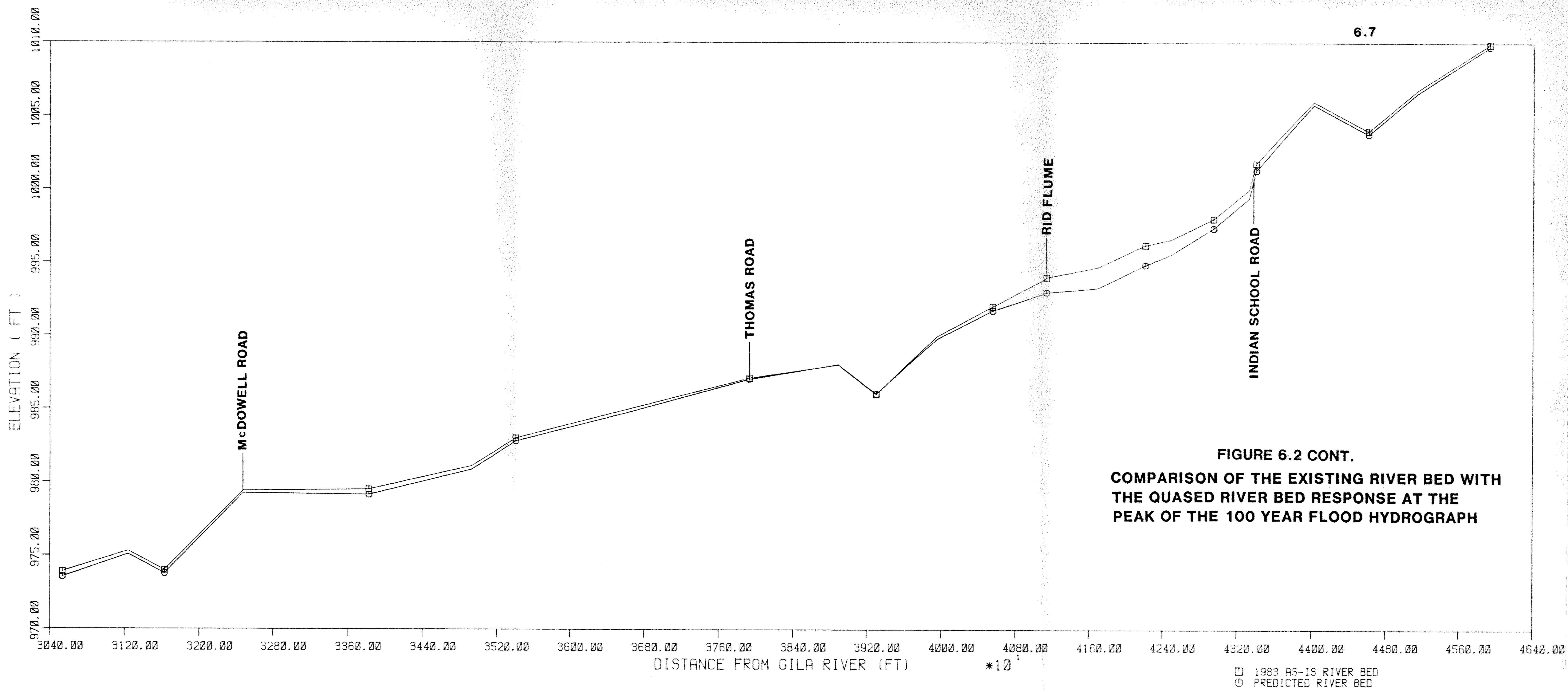


FIGURE 6.2 CONT.

COMPARISON OF THE EXISTING RIVER BED WITH THE QUASED RIVER BED RESPONSE AT THE PEAK OF THE 100 YEAR FLOOD HYDROGRAPH

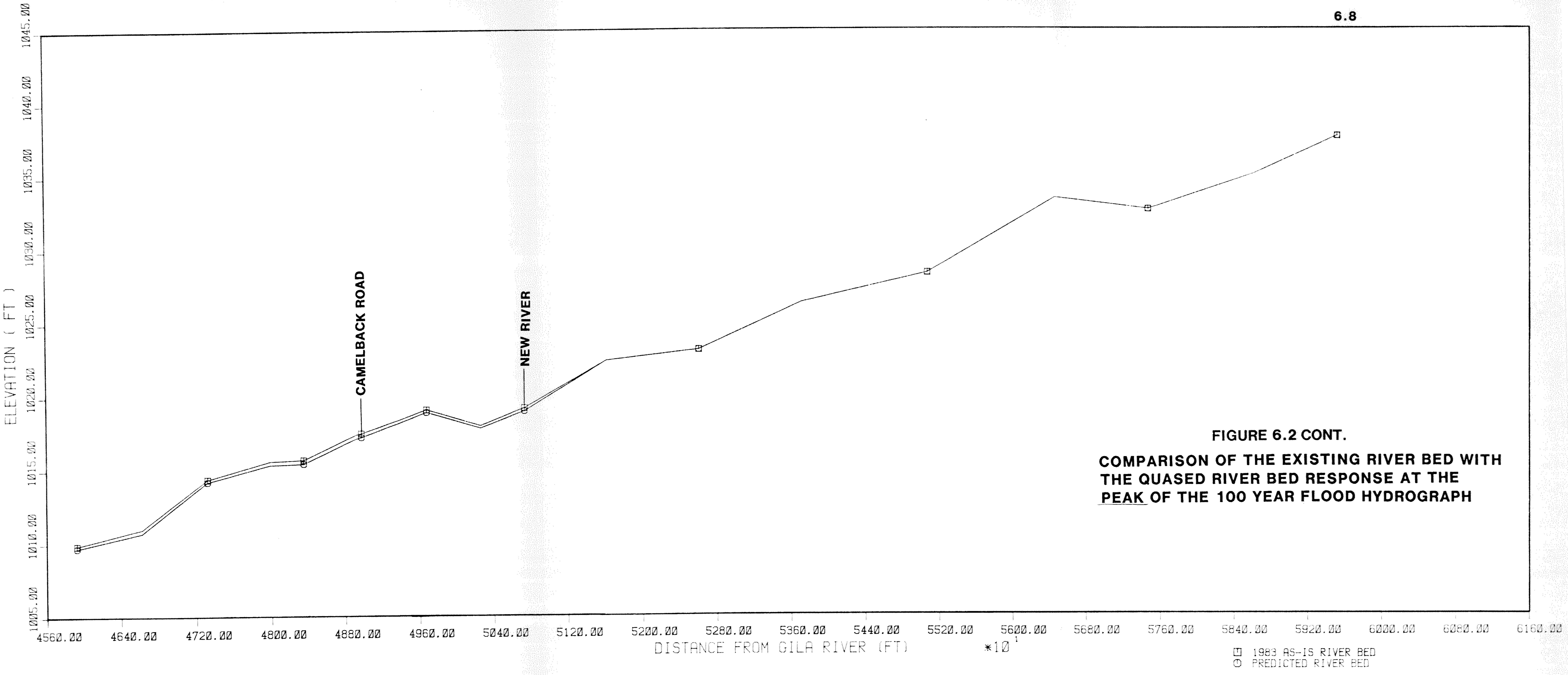


FIGURE 6.2 CONT.

COMPARISON OF THE EXISTING RIVER BED WITH THE QUASED RIVER BED RESPONSE AT THE PEAK OF THE 100 YEAR FLOOD HYDROGRAPH

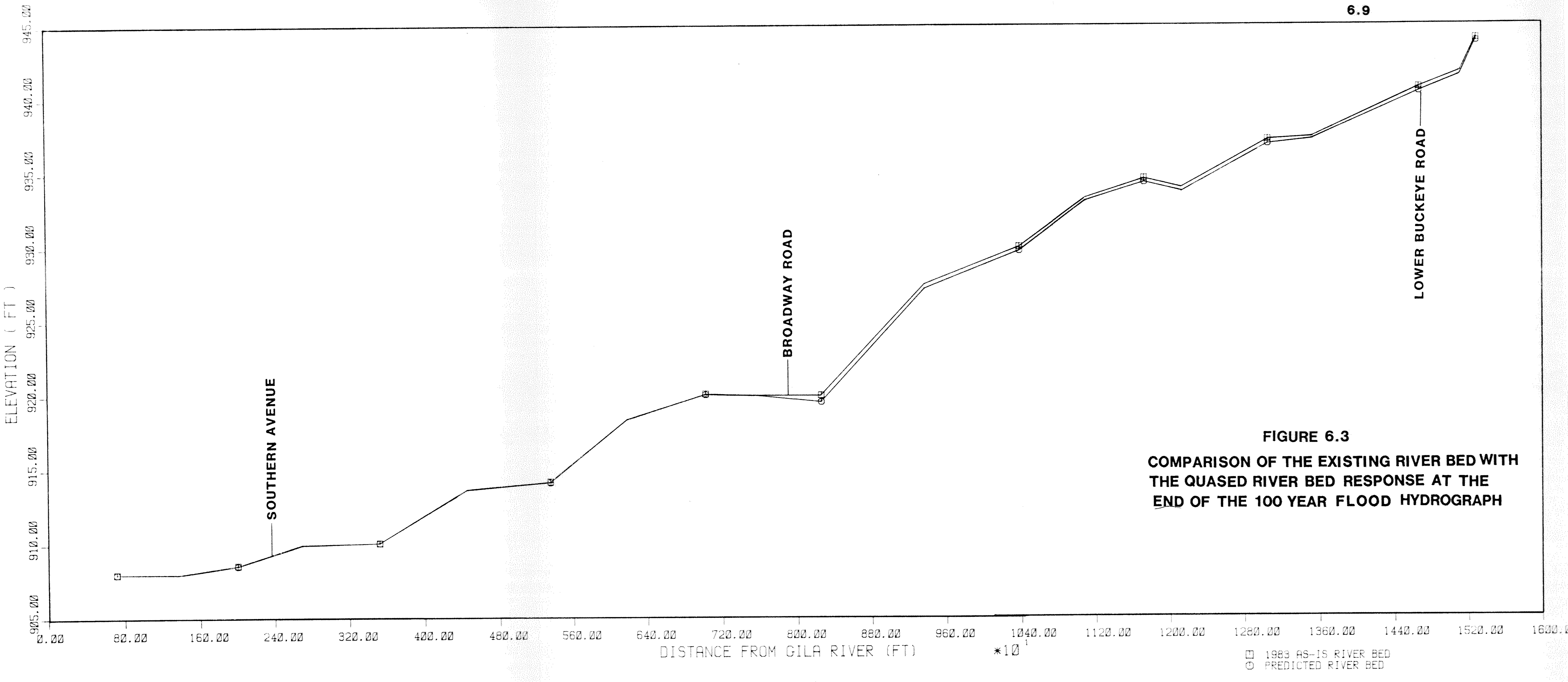


FIGURE 6.3

COMPARISON OF THE EXISTING RIVER BED WITH THE QUASED RIVER BED RESPONSE AT THE END OF THE 100 YEAR FLOOD HYDROGRAPH

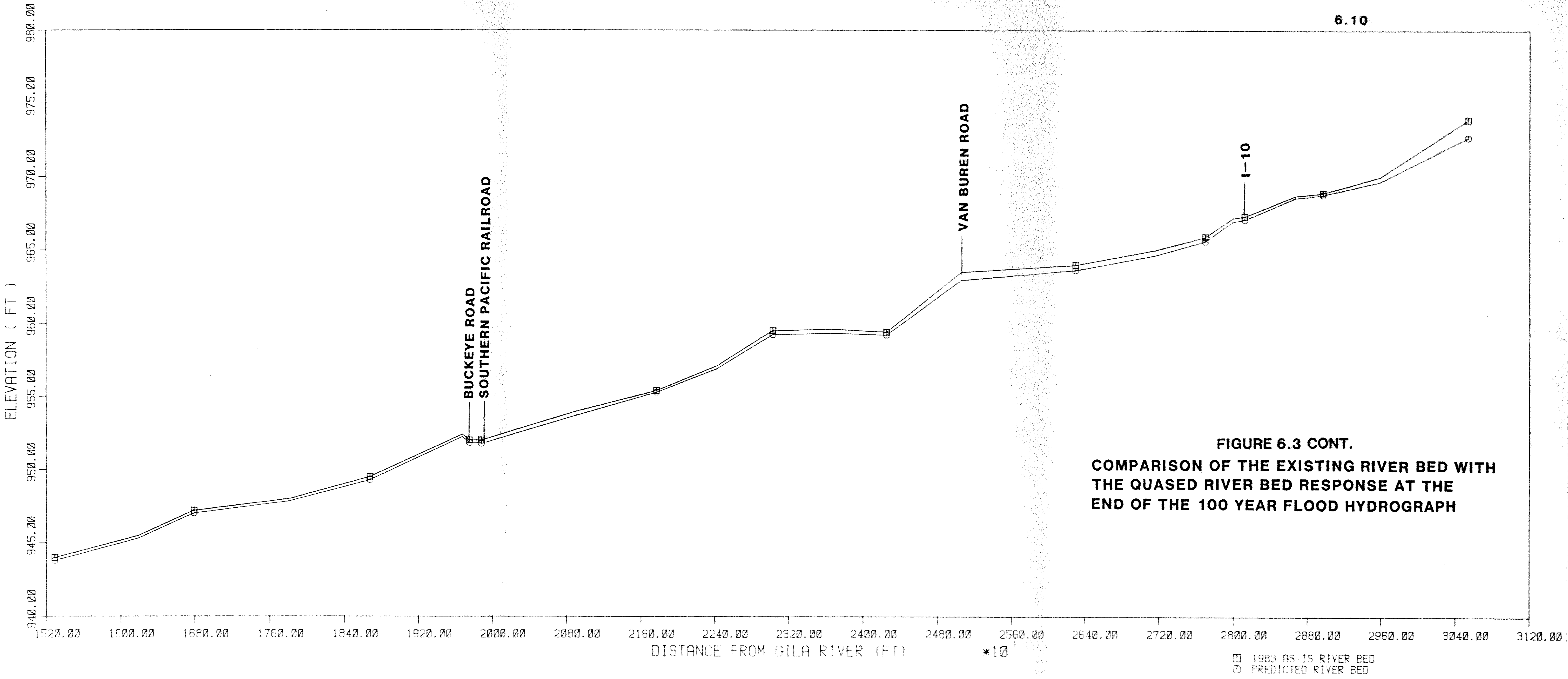


FIGURE 6.3 CONT.
COMPARISON OF THE EXISTING RIVER BED WITH THE QUASED RIVER BED RESPONSE AT THE END OF THE 100 YEAR FLOOD HYDROGRAPH

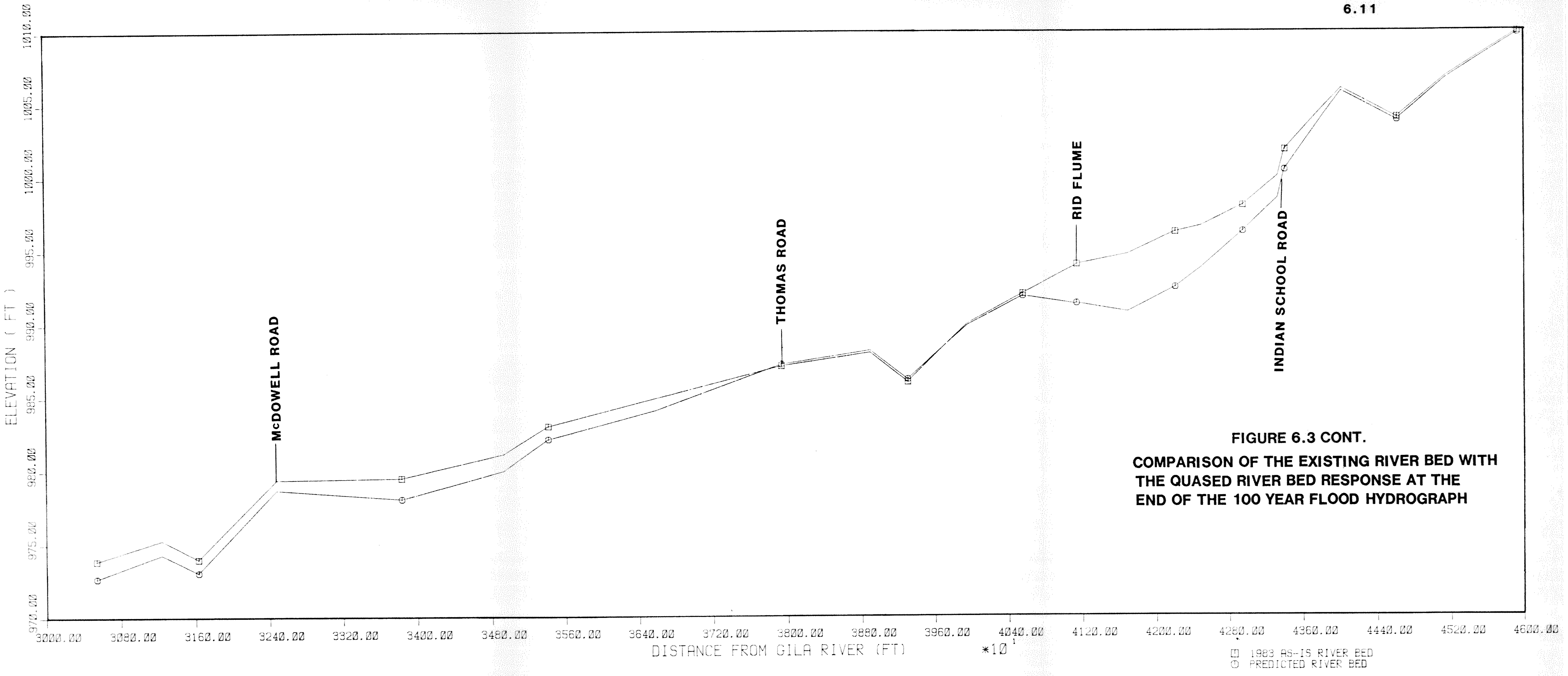


FIGURE 6.3 CONT.

COMPARISON OF THE EXISTING RIVER BED WITH THE QUASED RIVER BED RESPONSE AT THE END OF THE 100 YEAR FLOOD HYDROGRAPH

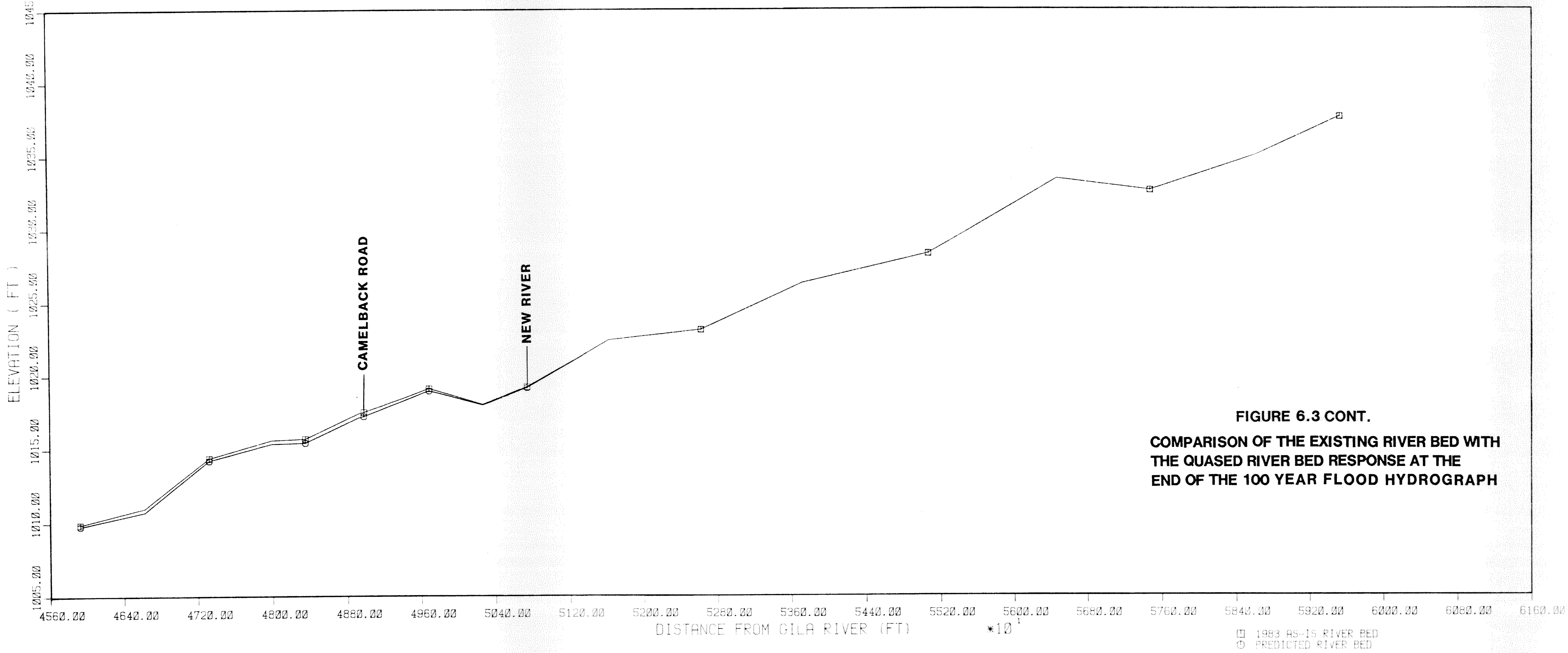


FIGURE 6.3 CONT.

COMPARISON OF THE EXISTING RIVER BED WITH THE QUASED RIVER BED RESPONSE AT THE END OF THE 100 YEAR FLOOD HYDROGRAPH

Table 6.1. Bed Response to the 100-Year Flood as Simulated by QUASED at the Peak and End of Flood.

HEC-II Cross Section Number	Original Channel Bed Elevation	Aggradation/ Degradation at Peak	Aggradation/ Degradation at End
589.25	1,037.6	0	0
580.15	1,035.0	0	0
568.70	1,032.6	0	0
558.60	1,033.4	0	0
544.70	1,028.3	0	0
531.20	1,026.3	0	0
520.20	1,023.1	0	0
510.30	1,022.4	0	0
501.45	1,019.2	-0.21	-0.07
496.70	1,018.0	-0.17	-0.04
490.90	1,019.1	-0.18	-0.15
487.50	1,018.3	-0.21	-0.20
483.56	1,017.5	-0.26	-0.28
483.00	1,017.4	-0.26	-0.28
476.90	1,015.7	-0.26	-0.27
473.30	1,015.6	-0.24	-0.24
466.60	1,014.4	-0.17	-0.16
459.50	1,011.0	-0.26	-0.26
452.60	1,009.9	-0.17	-0.12
444.75	1,006.8	-0.21	-0.14
439.45	1,004.0	-0.22	-0.20
433.50	1,006.0	-0.21	-0.20
427.75	1,001.8	-0.49	-1.38
426.95	1,000.0	-0.59	-1.51
422.30	998.0	-0.64	-1.78
417.75	996.6	-1.03	-2.88
414.95	996.2	-1.35	-3.78
409.45	994.7	-1.43	-3.94
403.85	994.0	-1.03	-2.66
403.71	993.9	-0.97	-2.60
398.00	992.0	-0.28	-0.16
392.10	990.0	-0.19	-0.11
385.50	986.0	0.04	0.18
381.40	988.0	0.03	0.15
370.50	987.1	-0.08	0.11
358.30	984.9	-0.22	-0.83
348.60	983.0	-0.20	-0.90
344.20	981.1	-0.24	-1.14
334.20	979.5	-0.36	-1.44
323.20	974.4	-0.16	-0.70
319.40	974.0	-0.22	-0.93
316.20	975.3	-0.22	-0.99
308.30	973.9	-0.36	-1.21
298.00	970.0	-0.07	-0.33

Table 6.1 continued. Bed Response to the 100-Year Flood as Simulated by QUASED at the Peak and End of Flood.

HEC-II Cross Section Number	Original Channel Bed Elevation	Aggradation/ Degradation at Peak	Aggradation/ Degradation at End
291.80	968.9	0.11	-0.13
288.80	968.7	0.10	-0.15
283.40	967.3	0	-0.23
281.60	967.2	0	-0.25
278.10	965.9	0	-0.29
275.15	965.0	0	-0.36
266.80	964.0	0	-0.36
254.30	963.5	-0.05	-0.55
246.20	959.4	-0.11	-0.21
240.20	959.6	-0.12	-0.28
234.00	959.5	-0.12	-0.28
227.95	957.1	-0.10	-0.20
221.40	955.4	-0.06	-0.12
212.85	954.0	-0.12	-0.28
202.30	952.0	-0.11	-0.24
202.00	952.0	-0.09	-0.20
201.00	952.0	-0.09	-0.18
200.20	952.4	-0.09	-0.17
190.20	949.5	0	-0.22
181.55	948.0	0	-0.16
173.85	947.2	0	-0.18
168.00	945.5	0	-0.18
159.00	944.0	0	-0.20
151.35	941.8	0	-0.25
146.80	940.7	0	-0.29
135.40	937.4	0	-0.18
130.65	937.2	0	-0.30
121.45	934.0	0	-0.27
117.35	934.6	0	-0.27
111.00	933.3	0	-0.20
103.90	930.0	0	-0.27
93.80	927.5	0	-0.31
82.60	920.0	0	-0.46
75.00	920.0	0.08	0
70.40	920.1	0.07	-0.06
61.90	918.4	0	-0.05
53.60	914.2	0	-0.07
44.60	913.7	0	-0.06
35.20	910.1	0	0
26.90	910.0	0	-0.06
20.00	908.6	0	-0.06
13.70	908.0	0	-0.05
7.15	908.0	0	0

QUASED has the limitation of not modeling local scour at river crossings. Thus, local scour at all bridge and utility towers should be added to QUASED-predicted bed responses to determine total scour potential.

6.4 Annual Aggradation/Degradation Analysis

An analysis to evaluate the average annual aggradation/degradation response of the channel bed was performed to determine if sediment deposition would reduce the flood-carrying capacity of the channel and help evaluate expected maintenance. The procedure involved:

1. Develop a sediment rating curve from the computed sediment discharges and water discharges from the QUASED run for each of the 10 reaches.

2. Use the sediment rating curves to determine the sediment yields for the 10-, 25-, 50-, and 100-year flood.

3. Using a weighted incremental probability method, determine average annual sediment transport rates for each reach.

4. Compare the average annual sediment transport rates of each reach with that of the supply reach of the New River and Agua Fria to determine the net deposition or degradation rates per year.

The developed sediment rating curves were of the following form:

$$Q_s = aQ^b \qquad (16)$$

where Q_s is the sediment transport capacity in cfs, Q is the water discharge in cfs, and a and b are the best-fit coefficient and exponent. Table 6.2 lists the coefficients and exponents a and b for each of the 10 reaches.

Sediment transport rates for the 10-, 25-, 50-, and 100-year floods were determined by applying Equation 16 to the discretized flood hydrographs. The average annual sediment yield for each reach was then computed using the weighted incremental probability of occurrence of floods.

Table 6.2. Coefficients and Exponents of Sediment Rating Curves for Each Reach of the Agua Fria River.

Reach	a	b
1	1.558×10^{-4}	1.319
2	2.35×10^{-5}	1.490
3	6.489×10^{-5}	1.406
4	4.162×10^{-6}	1.660
5	5.313×10^{-5}	1.435
6	3.088×10^{-5}	1.494
7	1.226×10^{-5}	1.577
8	1.247×10^{-5}	1.579
9	3.928×10^{-6}	1.687
10	2.548×10^{-5}	1.524

Reach 1: Glendale Avenue to confluence of New River
Reach 2: Confluence of New River to ISRB
Reach 3: ISRB to the RID flume
Reach 4: RID flume to Thomas Road
Reach 5: Thomas Road to 1,500 ft upstream of I-10
Reach 6: 1,500 ft upstream of I-10 to Van Buren Street
Reach 7: Van Buren Street to Buckeye Road
Reach 8: Buckeye Road to Lower Buckeye Road
Reach 9: Lower Buckeye Road to Broadway Road
Reach 10: Broadway Road to the confluence with Gila River

$$Q_{s_{annual}} = \frac{Q_{w_{meas}}}{Q_{w_{inc}}} (0.01)(Q_{s_{100}}) + (0.01)(Q_{s_{50}}) + (0.02)(Q_{s_{25}}) + (0.06)(Q_{s_{10}}) \quad (17)$$

where

$Q_{s_{annual}}$ is the average annual sediment yield (acre-feet)

$Q_{w_{meas}}$ is the average annual water yield

$Q_{w_{inc}}$ is the weighted probablistic water yield per year, and

the 100, 50-, 25-, and 10-year subscripts are for floods with these respective return intervals.

Table 6.3 summarizes the average annual sediment yields for each reach and compares the yields with the supply reach to determine the net aggradation/degradation response. The supply reach includes sediment inflow of the Agua Fria from Glendale Avenue to the confluence with the New River, and sediment inflow from the New River.

Several discrepancies with the quantitative geomorphic analysis result from generating sediment rating curves from QUASED. By simulating the aggradation/degradation response from the floodway instead of the entire flood plain, velocities within the floodway increase substantially, thereby increasing the sediment transport capacity. Where the floodway is significantly narrower than the flood plain, the sediment transport rates and average annual sediment yields increase substantially. Conversely, where the floodway widths remain approximately the same as the flood plain width, the increase in sediment transport rates is negligible.

The discrepancy between the QUASED and quantitative geomorphic analysis in predicting aggradation/degradation bed response is most evident in Reach 3. Located between ISRB and the RID flume, Reach 3 showed a trend toward degradation in the quantitative geomorphic analysis, and a slight aggradation trend as the result of generating sediment rating equations from QUASED. ?

Table 6.3. Average Annual Aggradation/Degradation Response for Study Reach.

Reach No.	Sediment Yield (ac-ft)	Degradation/ Aggradation (ac/ft)	Length (ft)	Average Depth * of Degradation/ Aggradation (ft)
New River	5.38	-	-	-
Agua Fria	68.20			
2	62.15	11.43	8,049	<0.1
3	70.46	3.12	2,301	0.1
4	61.60	11.98	3,580	0.2
5	76.39	-2.81	7,975	<-0.1
6	78.68	-5.10	4,632	-0.1
7	74.61	-1.03	5,470	-
8	73.97	-0.39	5,075	-
9	70.20	3.38	6,230	<0.1
10	85.04	-11.46	7,155	-0.1

Reach 1: Glendale Avenue to confluence of New River
Reach 2: Confluence of New River to ISRB
Reach 3: ISRB to the RID flume
Reach 4: RID flume to Thomas Road
Reach 5: Thomas Road to 1,500 ft upstream of I-10
Reach 6: 1,500 ft upstream of I-10 to Van Buren Street
Reach 7: Van Buren Street to Buckeye Road
Reach 8: Buckeye Road to Lower Buckeye Road
Reach 9: Lower Buckeye Road to Broadway Road
Reach 10: Broadway Road to the confluence with Gila River

*The degradation/aggradation responses are computed for initial conditions and as the bed responds toward equilibrium conditions, the net degradation/aggradation response tends toward zero. Therefore, this is just a measure of the direction in which each channel reach will respond.

The slight aggradation in Reach 3 is caused by most of the flow being contained within the main channel, so the predicted sediment yield does not change much from the floodway to the flood plain. However, the floodway of the supply reach is appreciably narrower than the flood plain, resulting in increased sediment supply for the floodway, and thus a reverse in the predicted aggradation/degradation trend.

VII. RECOMMENDATIONS FOR STUDY REACH

Based upon the three levels of analysis (qualitative geomorphic, quantitative geomorphic, and mathematical modeling) conducted for the COE proposed flood control plan, this section discusses recommended mitigative measures necessary to adequately convey the 100-year flood. Specific recommendations at river crossings, utility towers, pipeline crossings, floodwalls, and gravel pits are discussed.

7.1 River Crossings

Based upon the local scour computations, protection will be required for bridge piers at ISR, RID flume, SPRR, and Buckeye Road. The east abutment at ISR also requires protection.

It is recommended that riprap blanket protection be provided around bridge piers. The blanket should extend far enough upstream and downstream of the piers to protect against local vortices that are caused by the pier obstruction in the flow. The riprap blanket should be deep enough to prevent undermining due to downstream degradation.

With the COE flood control plan, freeboard heights of 1.5, 2.0, and 1.6 feet exist at the SPRR, RID flume, and ISRB crossing, respectively. The freeboard heights at these river crossings are below the three feet of freeboard as recommended by FEMA. Thus, some channelization near these crossings is required to achieve three feet of freeboard for the 100-year flood. The other river crossings all have freeboard heights exceeding three feet for the 100-year flood peak.

7.2 Utility Towers

The computed local scour depths for the TEP Company and SRP transmission towers were summarized in Tables 5.17 and 5.18. The computed values are relatively large, especially when the towers are within the banks of the main channel. The depth of burial of the towers was not available on the utility plans; therefore, comments regarding protection of towers cannot be made. However, the magnitude of potential local scour would indicate protection of some towers is warranted. It must be noted here, however, that the COE flood control plan is not worsening the local scour from existing conditions on the river.

7.3 Pipeline Crossings

Several pipelines cross or parallel the Agua Fria within the study reach. SLA acquired plans for pipelines at the following locations:

12-inch high-pressure gas line 100 ft downstream of Buckeye Road (Southern Pacific pipeline).

10-inch high-pressure gas line 150 ft upstream of the SPRR (El Paso Gas).

16-inch water line directly below Thomas Road (City of Avondale).

6-inch high-pressure gas line directly below Thomas Road (Southern Pacific Pipeline).

20-inch natural gas line running parallel to the east bank between ISRB and New River (El Paso Gas).

The 16-inch-diameter water line and six-inch high-pressure gas line crossings near Thomas Road are located in a potential degradation reach (Reach 5). Approximately 600 feet of the water line near the west bank is buried five feet. This portion of the line should be lowered. The burial depth of the six-inch-diameter gas line will have to be field verified before recommendations can be made regarding its adequacy of burial. The other three pipelines are located either out of the main channel or in an aggradation reach, and protection measures are not necessary.

7.4 Floodwall Protection

The floodwalls for protection of urban developments, as proposed by the COE, will be subjected to a wide range of velocities and depths during a 100-year storm.

The floodwall protecting the small subdivision just north of Lower Buckeye Road on the east overbank will be subjected to velocities that approach 3.5 fps at depths of approximately three feet during the 100-year peak. The floodwall is approximately 250 feet away from the channel bank at its nearest location, thus lateral migration of the channel should not be a significant problem. Protection and toe-downs of this floodwall will be minimal. Lower Buckeye Road will have to be raised slightly to prevent shallow flooding.

The floodwall protecting the west overbank subdivisions from Buckeye Road to Lower Buckeye Road is approximately 1,000 feet west of the existing bank. Velocities in the overbank approach 4.5 fps at depths of four feet. Thus, local scour depths may approach five to seven feet (Liu's scour equation) and some toe-down of the floodwall is needed. Below Lower Buckeye Road, the floodwall will be within the main channel. Velocities in this area average 6.3 fps, with some local velocities as high as 12 fps. Thus, floodwall and toe-down protection is required, especially as the floodwalls in this area protect the Avondale wastewater treatment plant.

The floodwall south of the Ball-Brosamer Development on the west overbank between Van Buren and the SPRR is far enough from the main channel and exposed to such minimal velocities (two fps) and depths (two feet) that little protection of floodwalls is necessary. This is also true of the floodwall protecting the trailer park south of McDowell Road on the east overbank.

no

7.5 Gravel Pits

Several gravel pits are present within the main channel and overbanks throughout the study reach. The largest sand and gravel mining operation is located between ISR and south of the RID flume on the east and west overbanks. The existing levees protecting the sand and gravel mining operations will fail during the 100-year flood. Thus, the COE assumed the levees, as well as the sand and gravel stockpiles, are non-existent for floodway and flood plain determination. Further, the COE assumed the gravel pits are inundated with water, when establishing the hydraulics.

These assumptions are reasonable for the hydraulic analysis; however, by not backfilling the flood plain gravel pits with suitable material prior to inundation of the gravel pits with water, a potential headcut problem exists. The headcut can begin as the result of water plunging into the deep gravel pits, and extend upstream of the pit, endangering nearby structures. Thus, even if local scour protection is provided around structures, the protection can be undermined from a potential headcut if the large gravel pits on the east and west overbanks are not backfilled or protected by levees. Therefore, it is recommended that the flood plain gravel pits between ISRB and south of the RID flume be either backfilled with acceptable fill or protected with properly designed levees to prevent overbank flow.

7.6 Maintenance Considerations

Only localized floodwall protection is being considered for the COE flood control plan. The effects of the floodwalls will be (1) prevention of overbank flow through developed areas, and (2) minimal sediment transport changes throughout the system.

The approximate average annual aggradation/degradation response throughout the study reach was summarized in Table 6.3. This response was based upon the sediment transport rates generated from the QUASED run for the floodway. Some of the aggradation/degradation trends for several of the reaches are opposite of those predicted in the quantitative geomorphic analysis. The quantitative geomorphic analysis was based upon predicted trends using the entire flood plain instead of the floodway. By modeling the floodway and not the entire flood plain, transport rates in reaches where the floodway is considerably narrower than the flood plain can increase dramatically.

In reaches where the floodway is similar in width to the flood plain, sediment transport rates will not increase substantially. Thus, by modeling the floodway with QUASED instead of the entire flood plain, some of the aggradation/degradation trends can change.

All of the average annual aggradation/degradation depths are less than 0.2 foot in magnitude, indicating the system is close to equilibrium and that very little maintenance will be required to keep the floodway in its original condition.

The qualitative and quantitative geomorphic analyses indicate that degradation is the dominant channel bed trend. With degradation, minor bank erosion can be expected, as the sediment imbalance between supply and transport capacity will be made up by removal of bed and bank material.

VIII. REFERENCES AND DATA SOURCES

The following references were cited in the text of this report.

1. Wilson, Moore, Pierce, 1957 Geologic Map of Maricopa County, Arizona.

2. Soil Conservation Service (unpublished), "General Soil Map with Soil Interpretation for Land Use Planning." Maricopa County, Arizona.

3. Anderson, T.W., 1968. "Electrical Analog Analysis of Groundwater Depletion in Central Arizona," U.S. Geological Survey Water Supply Paper 1860.

The following is a list of information used for the system analysis of the Agua Fria River and the New River.

<u>Aerial Photos</u>

1936 Agua Fria River channel from Camelback Road to Van Buren Street, scale 1" = 600'.

1/24/54 Eastern portion of the Agua Fria River flood plain from Glendale Avenue to Northern Avenue (3 photos).

1/16/63 Agua Fria River channel from the confluence with the New River to the confluence with the Gila River, scale 1"= 500'.

1/74 Agua Fria River channel from the confluence with the New River to the confluence with the Gila River, scale 1" = 1000'.

3/7/78 Agua Fria River channel from Northern Avenue to the confluence with the Gila River, scale 1" = 1000'.

12/20/78 Agua Fria River channel from Glendale Avenue to Northern Avenue.

2/15/80 Agua Fria River channel from Glendale Avenue to Northern Avenue.

2/20/80 Agua Fria River channel from Northern Avenue to the confluence with the Gila River, scale 1" = 600'.

Enlarged aerial photos of eastern portion of the Agua Fria River flood plain taken on 1/21/64, 1/29/70, 2/25/76, 3/78 and 12/20/78.

9/83 Agua Fria River from Northern Avenue to the confluence with the Gila River, scale: 1" = 500'.

Topographic Maps

1954 (photo revised 1969) USGS topographic map of Phoenix, Arizona.

1957 (photo revised 1971 and 1974) USGS quadrangle map of El Mirage, Arizona, 1 = 24,000.

Topographic maps of the Agua Fria River from the confluence with the New River to the confluence with the Gila River, scale 1" = 200', contour interval of 4', February/March 1972.

Topographic maps of the New River from the confluence with the Agua Fria River to approximately 5,000 feet upstream of the confluence with the Agua Fria River, scale 1" = 200', contour interval of 4', February/March 1972.

Topographic maps of the Agua Fria River from Glendale Avenue to McDowell Road, scale 1" = 200', 8/31/81.

Topographic maps of the Agua Fria River from McDowell Road to the confluence with the Gila River, scale 1" = 200', 11/81.

Topographic maps of the New River from the confluence with the Agua Fria River to Glendale Avenue, scale 1" = 100', 8/81.

Topographic maps of the Agua Fria River from Buckeye Road to the confluence with the New River, scale 1" = 200', 9/83.

Bridge Plans

1969 Plans for construction of Indian School Road Bridge. Includes boring samples at the bridge site.

1977 Plans for addition of the third and fourth lanes on the Indian School Road Bridge.

3/4/26 As-built plans of the Southern Pacific Railroad Bridge crossing.

1969 Design plans for the Buckeye Road Bridge crossing.

1980 As-built bridge plans for I-10.

1983 Design plans for the McDowell Road Bridge crossing, sheets 1-10.

1983 Preliminary bridge plans for Camelback Road Bridge, sheets 25, 29, 34-36.

Site Visits

2/4/82 Site visit of pit exposed 800 ft downstream of Indian School Road Bridge by Maricopa County Highway Department.

6/82 Site visit of excavation around one of RID flume piers.

3/8 Site visit to gather sediment samples from Waddell Dam to the confluence with the Gila River on the Agua Fria and gather several surface material samples on the New River.

Soil Reports

6/9/82 Geotechnical Investigation Report "Channelization-Agua Fria River Thomas Road, and I-10 Maricopa County, Arizona," by Sergent Hauskins and Beckwith.

4/24/81 "Geotechnical Report for Camelback Road Bridge Crossing of the Agua Fria River, Maricopa County, Arizona," by Engineers Testing Laboratory.

9/24/80 Geotechnical Investigation Report, "Indian School Road Bridge at Agua Fria River, Maricopa County, Arizona," by Sergent Hauskins and Beckwith.

10/14/80 Geotechnical Investigation Report, "Bell Road Bridge at Agua Fria River, Maricopa County, Arizona," by Sergent Hauskins and Beckwith.

4/15/80 "Pier Scour Flume Piers in the Agua Fria, Maricopa County, Arizona," by Engineers Testing Laboratories, prepared for Roosevelt Irrigation District, Buckeye, Arizona.

Reports

1981 "Hydrology of the Agua Fria River," by the L.A. Army Corps of Engineers.

10/15/82 "Hydraulic Analysis of Agua Fria Channel, McDowell Road to Thomas Road, Maricopa County, Arizona," by Lowry and Associates.

1982 "Agua Fria River Study-1982," prepared for Maricopa County Flood Control District by Willdan Associates.

10/74 "New River and Phoenix City Streams, Arizona," Design Memorandum No. 2, Hydrology, Part 1, U.S. Army Corps of Engineers, Los Angeles District.

1929 "The Agua Fria River Flume Crossing, 5959 Feet Long, An Interesting Feature," by M.E. Ready and A.V. Saph Jr.

10/82 "Litigation Support for Flooding Levels Associated with the Highway Construction of West Glendale Avenue Over the Agua Fria River," Simons, Li and Associates, Inc.

3/82 "Hydraulic, Erosion and Sedimentation Analysis of Indian School Road Bridge Over the Agua Fria River, Phoenix, Arizona," Simons, Li and Associates, Inc.

5/82 "Sediment Inflow for the Arizona Canal Diversion Channel, Final Report," Simons, Li and Associates, Inc.

3/68 "Flood Plain Information, Agua Fria River, Maricopa County, Arizona," U.S. Army Corps of Engineers, Los Angeles.

9/83 "Hydraulic and Geomorphic Analysis of the Agua Fria River," Simons, Li and Associates, Inc.

5/84 "Draft Qualitative Geomorphic Analysis," by Simons, Li and Associates, Inc.

6/84 "Draft Quantitative Geomorphic Report" by Simons, Li and Associates, Inc.

6/84 "Draft Sediment Transport Report" by Simons, Li and Associates, Inc.

<u>Hydrographs</u>

3/7/81 100-year flood event downstream of the confluence with the New River on the Agua Fria, extracted from the L.A. Corps of Engineers printout dated March 7, 1981.

DATUM IS N.G.V.D. OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

SIMONS, LI & ASSOCIATES
4030 BIRCH ST., SUITE 103
NEWPORT BEACH, CA. 92660

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

NEW RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

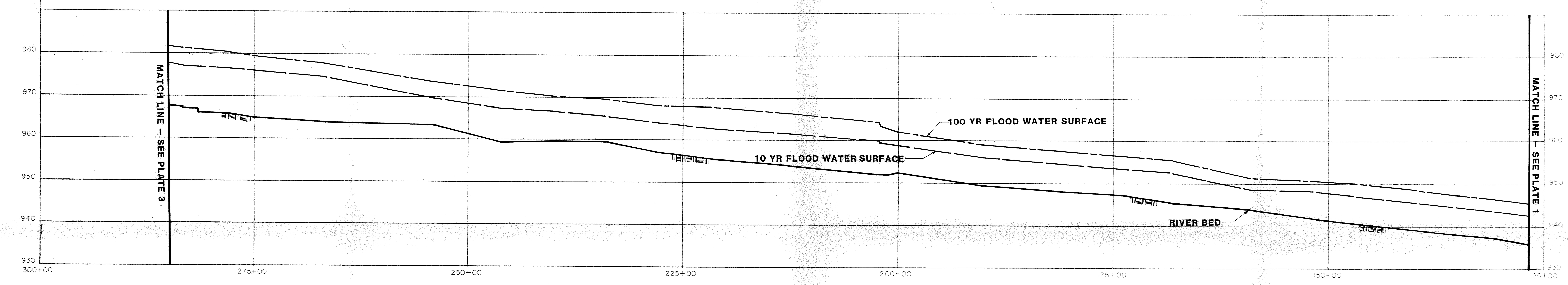
COE FLOOD CONTROL PLAN FOR LOWER AGUA FRIA RIVER

AGUA FRIA FROM GILA RIVER TO LOWER BUCKEYE ROAD

SUBMITTED BY: DATE APPROVED: SPEC. NO. DACW09-____ B-____ DISTRICT FILE NO.

PLATE 1

VALUE ENGINEERING PAYS



PROFILE
HOR. SCALE 500 0 500 1000 FEET
VERT. SCALE 10 0 10 20 FEET

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

PLAN
SCALE 500 0 500 1000 FEET

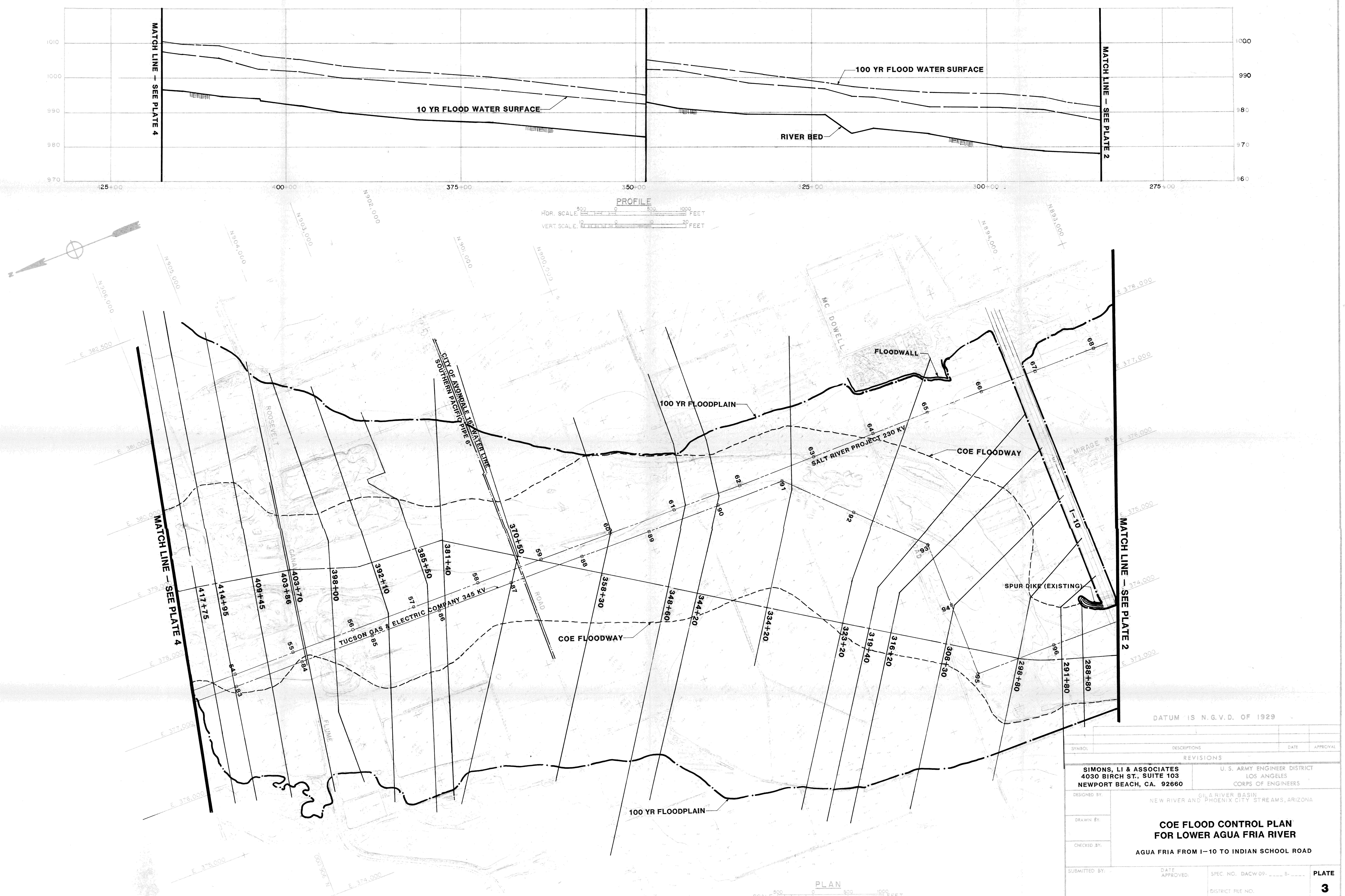
DATUM IS N.G.V.D. OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

SIMONS, LI & ASSOCIATES 4030 BIRCH ST., SUITE 103 NEWPORT BEACH, CA. 92660	U. S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS
DESIGNED BY:	NEW RIVER BASIN NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA
DRAWN BY:	**COE FLOOD CONTROL PLAN FOR LOWER AGUA FRIA RIVER**
CHECKED BY:	AGUA FRIA FROM LOWER BUCKEYE ROAD TO I-10
SUBMITTED BY: CHIEF, BRANCH	DATE APPROVED: SPEC. NO. DACW 09-____ B-____ DISTRICT FILE NO. — PLATE 2

SAFETY PAYS



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U. S. ARMY ENGINEER DISTRICT
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GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

DESIGNED BY:
DRAWN BY:
CHECKED BY:

COE FLOOD CONTROL PLAN
FOR LOWER AGUA FRIA RIVER

AGUA FRIA FROM I-10 TO INDIAN SCHOOL ROAD

SUBMITTED BY: DATE APPROVED: SPEC NO. DACW09- DISTRICT FILE NO.

PLATE 3

VALUE ENGINEERING PAYS

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

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LOS ANGELES
CORPS OF ENGINEERS

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DRAWN BY:

CHECKED BY:

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NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

COE FLOOD CONTROL PLAN
FOR LOWER AGUA FRIA RIVER

AGUA FRIA FROM INDIAN SCHOOL ROAD TO BETHANEY HOME

SUBMITTED BY: DATE APPROVED: SPEC. NO. DACW 09- ____ B- ____ PLATE 4

DISTRICT FILE NO.

CHIEF, BRANCH

SAFETY PAYS

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