GEOMORPHIC ANALYSIS

FINAL REPORT

HYDRAULIC AND GEOMORPHIC ANALYSIS

OF THE AGUA FRIA RIVER

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FINAL REPORT

HYDRAULIC AND GEOMORPHIC ANALYSIS OF THE AGUA FRIA RIVER

Submitted to

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EXECUTIVE SUMMARY

The Flood Control District of Maricopa County has contracted Simons, Li & Associates, Inc. (SLA) to conduct a system analysis of the Agua Fria River from its confluence with the New River to its confluence with the Gila River. The system analysis includes an assessment of existing conditions in the study reach, and an assessment of proposed flood control projects between Camelback Road and Buckeye Road. SLA was also contracted to provide design plans and specifications for a flood control project between Camelback Road and Thomas Road.

Included in this report is the comprehensive hydraulic and geomorphic analysis of existing conditions in the Agua Fria River. The primary objective of this report is to provide baseline information on hydraulic and sediment transport characteristics of the Agua Fria for future flood control projects.

Three levels of analysis were conducted to assess existing conditions in the Agua Fria which included (1) a qualitative geomorphic analysis, (2) an engineering geomorphic analysis, and (3) a mathematical model simulation. The major results for each level of analysis are summarized.

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 qualitative assessment relies heavily upon aerial photographs, site visits, previous flood plain reports, and descriptions and accounts of previous floods in the river.

The Agua Fria River in the study reach is an ephemeral braided stream with a wide flood plain. The general trend of the bed has been to degrade in the past 20 years. This is due in part to the numerous sand and gravel mining operations within the study reach which have extracted large quantities of material from the channel or narrowed the channel by construction of levees to protect their operations. Presently two large sand and gravel mining operations are just upstream of Glendale Avenue and downstream of Indian School Road Bridge (ISRB).

The most severe degradation of the channel bed occurred between ISRB and the Roosevelt Irrigation District (RID) flume. The flood plain has been severely encroached by the construction of levees in this reach, causing the 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 (4 to 14 inches thick) are present at varying depths (2 to 7 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, Arizona Canal Diversion Channel and an I-10 collector channel. The new Waddell Dam will have the greatest impact on controlling future flood peaks and subsequently channel morphology response in the study reach.

Engineering Geomorphic Analysis

The engineering geomorphic analysis quantifies the aggradation/degradation response of the system. Determining the hydraulics is necessary for assessing the sediment transport characteristics of the system.

The hydraulics of the Agua Fria River between Glendale Avenue and the confluence with the Gila River were established using the Army Corps of Engineers HEC-II backwater profile program. Hydraulic characteristics were determined for floods with return intervals of 10, 25, 50 and 100 years.

The 10-year flood peak of approximately 31,000 cfs will be contained in the main channel for most portions of the study reach. For floods with larger return intervals, overbank flow becomes large. The 100-year flood plain is 8000 feet wide near the New River, decreases to 7,000 feet near Camelback Road, reduces to 5,000 feet at ISRB, and varies between 3,000 and 5,000 feet from ISRB to the confluence with the Gila River.

The main channel velocities generally range from 5 fps to 7 fps for the 10-year peak discharge and from 7 fps to 10 fps for the 100-year peak discharge. The main channel velocities upstream of ISRB are slightly lower than the velocities downstream of ISRB.

The potential long-term bed response of the Agua Fria in the study reach is summarized below:

- Remains relatively stable between the confluence with the New River and ISRB.
- Slight degradation between ISRB and Buckeye Road.
- Slight aggradation between Buckeye Road and Broadway Road.
- Remains relatively stable between Broadway Road and the confluence with the Gila River.

The largest degradation potential exists between ISRB and the RID flume, and between McDowell Road and Thomas Road.

The present flow capacity of ISRB and the RID flume is not large enough to pass the 100-year flood peak. The I-10, Southern Pacific Railroad and Buckeye Road bridge crossings do have adequate capacities to pass the 100-year flood peak.

Local scour depths around the bridge piers at ISRB are greater than the present pier foundation burial depth for the 100-year discharge. Local scour depths at the RID flume and the Southern Pacific Railroad crossing are approaching the pier foundation burial depths. Some form of protection at these three crossings will be required to withstand the 100-year flood.

Mathematical Model Analysis

The third level of analysis involves executing the SLA developed sediment routing model to determine the Agua Fria River response to the 100-year flood. The SLA model considers sediment routing by size fraction, and therefore can simulate the armoring process of a river bed.

The model was verified by simulating the channel response of the Agua Fria River to the December 1978, January 1979 and February 1980 floods. The 1973 Los Angeles District of the Corps of Engineers cross sections were used to simulate pre-flood conditions and the 1981 cross sections were used to approximate post-flood conditions. The SLA model simulated the bed response reasonably well for the three floods and proved itself reliable for future simulations.

The SLA model was executed for the 100-year flood using the 1981 river cross sections. The predicted bed response of the model verified the results of the engineering geomorphic analysis. The model showed that degradation occurred in the constricted sections of the Agua Fria between ISRB and Buckeye Road and that deposition occurred downstream of Buckeye Road. The degradation due to the 100-year flood averages less than one foot for most of the channel reaches except between ISRB and the RID flume where the degradation averages 3 feet and the entrenched channel section between Thomas and McDowell Roads where the degradation averages 1.6 feet.

Recommendations Regarding Flood Control Projects

1. To reduce the flood plain width for future developments in the study reach, channelization to contain the 100-year flood is needed.

2. Main channel velocities for the 100-year flood peak range from 7 to 10 fps. Considering the available parent bed and bank material, some form of protection will be required on levees to insure a stable channel.

3. The long-term bed response of the Agua Fria between ISRB and Buckeye Road is a slight degradation. With channel encroachment in this section of the river, the degradation potential will increase and some grade control structures will be necessary to prevent large headcuts progressing up the river.

4. No instream gravel mining should be allowed in the channelized section, except for removal of any bars or islands that may form.

5. Protection of bridge piers at ISRB, RID flume and Southern Pacific Railroad to withstand the 100-year peak discharge is required.

I. INTRODUCTION

1.1 General

The Agua Fria River originates in the mountains of central Arizona and flows southward for about 130 miles before emptying into the Gila River 15 miles west of downtown Phoenix. Figure 1.1 shows the entire watershed of the Agua Fria River. 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, that form the eastern and western boundaries respectively, of the drainage area. Waddell Dam controls 1,457 square miles of drainage. The gradient of the Agua River is steep in the upper reaches and ranges 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.

Development 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. Man's development along the channel can add to the instability, if not conducted properly. In addition, 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 capable of inundating the flood plain, there is a need for a comprehensive flood control plan for the Agua Fria River in order to insure the safety of existing developments and to accommodate the increasing pressure for future development along its course. In order to meet these needs, the Flood Control District of Maricopa County has contracted Simons, Li & Associates, Inc. (SLA) to 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 downstream to the confluence with the Gila River, recommend a conceptual plan for flood control

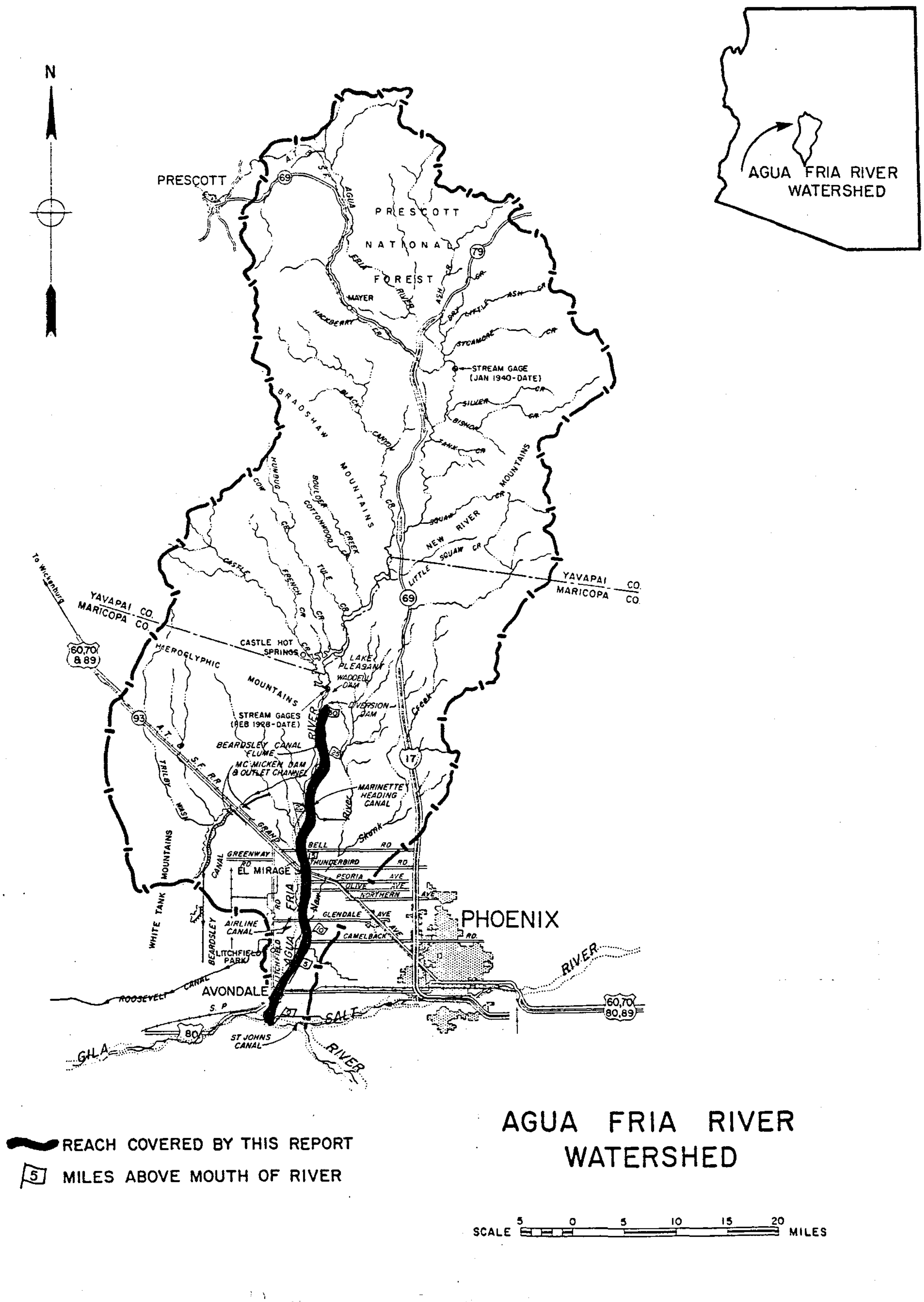


Figure 1.1. Agua Fria River Watershed.

between McDowell Road and Buckeye Road, review proposed construction plans for a channelization between Thomas Road and McDowell Road and to develop construction plans for channel improvement work between Camelback Road and Thomas Road. Figure 1.2 illustrates the study area.

The material in this report presents the background, methodologies used and results of the analysis of the existing conditions on the Agua Fria River. This material was used in the later phases of the study to define the needs for and guide the design of the proposed channel improvements.

1.2 Study Description

Three levels of analysis were conducted to assess the existing hydraulic, geomorphic, erosion and sedimentation conditions of the Agua Fria River. The first level was a qualitative investigation of the system considering the basic physical characteristics of the watershed, data identifying changes in the river system, and principles of fluvial geomorphology. This level of analysis provided an understanding of the most important factors contributing to the current condition of the Agua Fria River. An engineering geomorphic analysis provided the second level of assessment. It consisted of identifying channel aggradation and degradation response considering the mechanics of sediment transport combined with the hydraulic conditions and bed material characteristics of the Agua Fria River. The third level of analysis further quantified the river response using a continuous computer simulation of the channel's response to the 100-year flood hydrograph.

Prior to performing the three-level analysis, existing hydrologic information was obtained and reviewed. A hydraulic analysis was also performed using the U.S. Army Corps of Engineers' HEC-2 program. The hydrologic and hydraulic information provided a large portion of the information used in the three-level analysis.

The specific scope of work for the study follows.

1. Conduct site investigation by SLA engineers.

2. Collect, collate, and assemble watershed and channel data including aerial photographs; topographic maps; sand and gravel mining information; and channel, hydraulic, hydrologic, geological, climatological, soil and structural data.

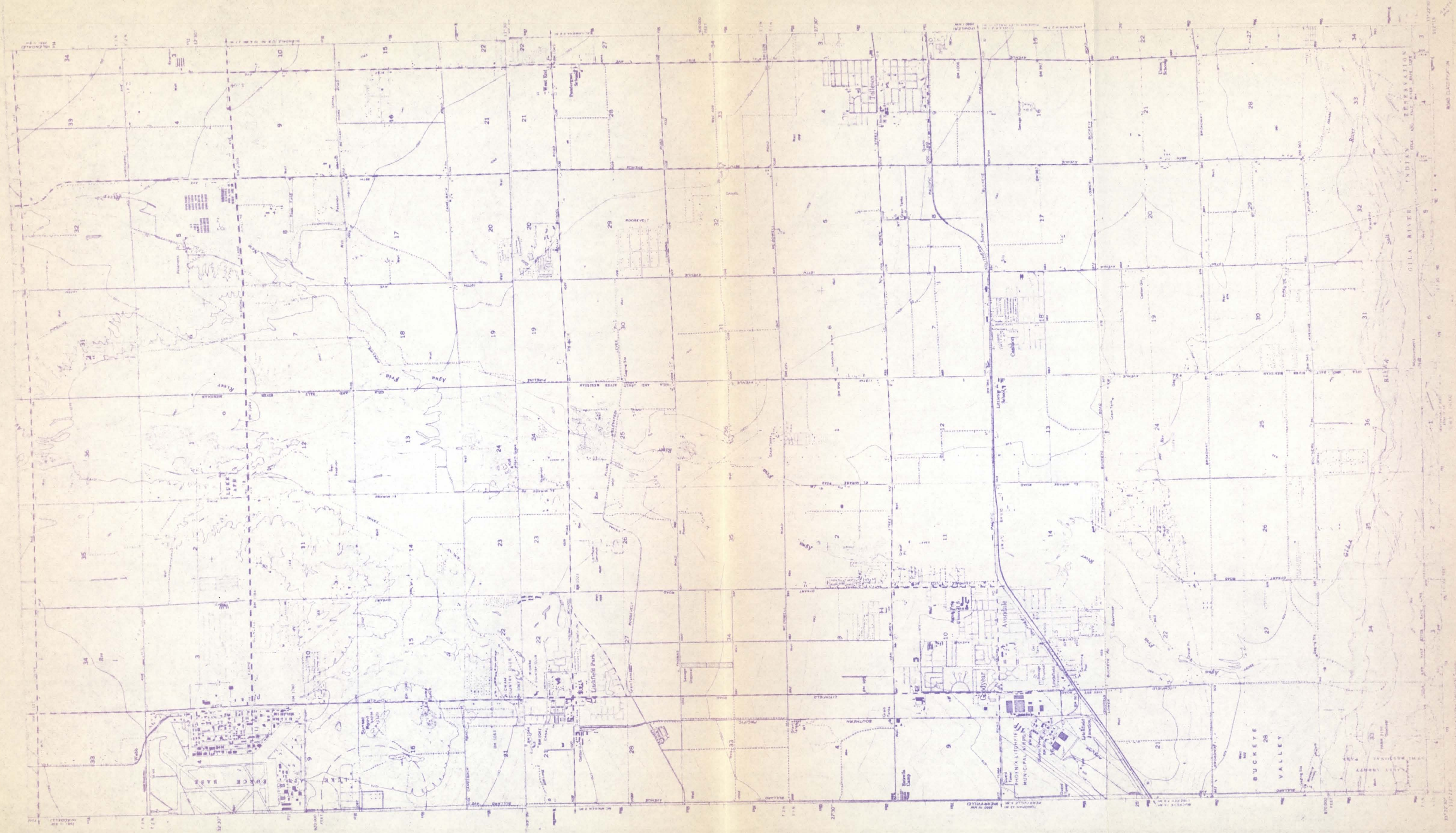


FIGURE 1.2 U.S.G.S. MAP OF STUDY AREA ON THE AGUA FRIA RIVER

3. Review previous reports regarding flooding, urban developments, existing and proposed hydraulic structures, and litigation materials for court cases associated with sand and gravel mining impacts.

4. Conduct a qualitative geomorphic analysis of the watershed and river system.

   a. Investigate historical channel changes using aerial photographs.

   b. Evaluate the effect of sand and gravel mining and other activities of man on the channel morphology.

   c. Investigate the existing channel characteristics.

   d. Evaluate the potential impact from the proposed structures such as New River Dam and the new Waddell Dam.

5. Conduct a quantitative engineering analysis.

   a. Compute backwater profiles using HEC-2 for the 10-, 25-, 50- and 100-year floods.

   b. Analyze hydraulic characteristics along the river.

   c. Analyze sediment characteristics along the river.

   d. Develop sediment transport relations for the study reach and divide the entire study reach into subreaches that have similar sediment, geometry and hydraulic characteristics.

   e. Compute sediment transport capacities for each subreach for the 10-, 25-, 50- and 100-year flood peaks.

   f. Estimate the channel degradation/aggradation response of each subreach for the 100-year flood discharge.

   g. Determine the long-term channel response for the existing and future development condition.

   e. Determine bridge capacities for the 100-year flood peak and evaluate the local scour potential around bridge piers and abutments.

6. Perform water and sediment routing.

   a. Verify the SLA model using the historical channel changes.

   b. Simulate the river response to the 100-year flood.

7. Make suggestions regarding channelization schemes.

8. Prepare a draft report and the supporting documents for the study of existing channel conditions.

II. HYDROLOGY

The Agua Fria River in the study area is an ephemeral stream. Generally runoff occurs only during and immediately following the heavier precipitation because climatic and drainage area characteristics are not conducive to continuous runoff. Significant runoff occurs in the summer months as a result of local storms and to a lesser degree general storms. In the winter months runoff is produced by general storms.

This chapter presents a discussion of the climatology of the Phoenix area, a description of the flood history of the Agua Fria River, and the design discharges that have been adopted for varying return period storms on the Agua Fria River. The latter is detailed for both the existing condition and planned future watershed modifications.

2.1 Climatology

The climate of Phoenix and vicinity ranges from warm and arid over the desert floor to cool and moderately humid in the mountainous portions of the basin. Mean maximum/minimum January temperatures range from approximately 65 to 35 degrees Fahrenheit in the valleys (64.0 to 35.3 at Phoenix Weather Bureau Airport) to about 50 to 25 in the mountains. Mean maximum/minimum daily temperatures during July vary from 105 to 75 in the lower portions of the region (104.6 to 75.0 at Phoenix Weather Bureau Airport) to 90 to 60 in the higher mountains. The extreme temperature experienced in the region ranges from 120 degrees in portions of the lower desert to near zero in some of the more remote mountain areas.

Mean annual precipitation in the basin ranges from around 7 inches in the area south and west of Phoenix to more than 22 inches in the New River Mountains. The average annual precipitation for the entire drainage area is 11.4 inches, of which 4.4 inches (38 percent) falls during the summer months of June through September, and the greatest portion of the remainder falls during the period of December through March. Much of the winter precipitation falls as snow at elevations above 6,000 feet, and snow can occur at times over the entire basin, although snow below 2,000 feet is rare. There is considerable year-to-year variability in the individual monthly, seasonal and annual precipitation amounts which fall in the vicinity of Phoenix, Arizona. Some of the drier months of the year have at times experienced more than ten times the normal precipitation, and each month has passed at least once during the 20th century with no measurable precipitation reported at some stations.

There are three basic types of storms--general winter storms, general summer storms, and summer thunderstorms--which can affect the Phoenix area, although some individual storms may consist of a combination of types. A brief description of each storm type appears below.

1. General winter storms. General winter storms are usually most prevalent and most intense during the months of December through March, although they can occur any time from October through May, and occasionally in combination with other types of storms during the summer months. This type of storm is characterized most typically by cool, stable air masses with widespread overcast stratiform cloudiness and steady, light rain or snow. A few locally heavy showers and occasional isolated thunderstorms may occur. Despite the relatively low intensities of rainfall, the large areal extent and the relatively long durations of this type of storm can produce significant volumes of runoff and even peak flows on the larger rivers of the region, such as the Agua Fria.

2. General summer storms. These storms usually consist of general rains of a convergence, frontal and/or orographic nature, with moderate to heavy thunderstorms often superimposed. General summer storms occur primarily during the months of July, August and September, although it is possible for this type of storm to occur any time from May through October (often with some of the characteristics of general winter storms, especially during the latter portions of the greater summer season). Cloudiness in this type of storm is dominated by the convective types: cumulus and cumulonimbus, although considerable stratiform cloudiness is often present as well. Rainfall normally consists of a mixture of general steady rain and numerous convective showers with locally heavy precipitation associated with convective activity. The convective type usually accounts for the bulk of a general summer storm's total rainfall. The general summer storms are capable of producing peak discharges for large rivers such as the Agua Fria.

3. Summer thunderstorms. Thunderstorms can occur in the vicinity of Phoenix, Arizona, at any time of the year, but most common and the most significant thunderstorms occur during the summer months, usually between late June and late September. Summer thunderstorms are normally scattered or isolated phenomena, and are more than twice as common over the higher mountain peaks as they are in the desert valleys. The most severe of these thunderstorms, however, appear to have little preference for either high or low-elevation areas. Heavy thunderstorms, sometimes referred to as "cloudbursts," can produce severe flash floods over small drainage basins, resulting in serious local damage and sometimes loss of life. They generally do not produce peak discharges on the larger rivers.

2.2 Flood History

Runoff records are available at five gaging stations on the Agua Fria River and three stations on the New River. 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 greatest 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 seven floods with flows between 50,000 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. 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 are from records of the gaging stations at Waddell Dam, the gaging station at Mayer, newspaper files, historical documents and records and field investigations.

Most recently three floods have occurred in the Agua Fria in December 1978, January 1979 and February 1980. The hydrographs for these floods are illustrated in Figure 2.1.

The duration of a flood depends upon the type of storm causing it. Floods can peak in a matter of hours following an intense thunderstorm on the Agua Fria River, 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 and include (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 could be attributed to an inaccurate discharge rating curve during the flood, a large portion of the difference was surely attributable to attenuation.

Table 2.1. Stream Gaging Stations Along Agua Fria River and New River.

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 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
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

Table 2.2 (continued)

Source: U.S. Army Corps of Engineers reports dated as follows:
1889 thorugh 1964, except 1912 and 1923 - March, 1968
1912 and 1923 - March, 1981
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 February 20, 1980.

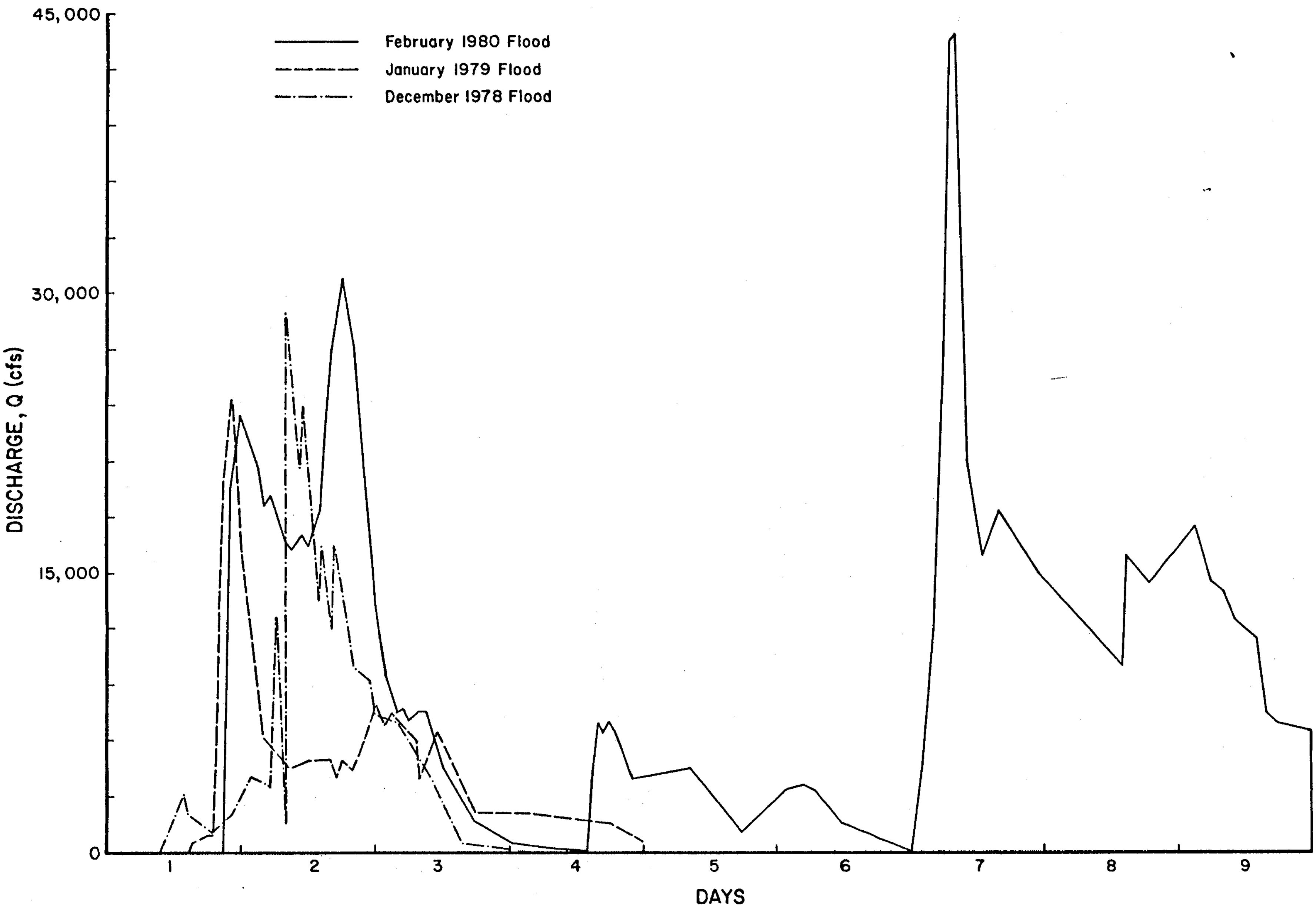


Figure 2.1. Hydrographs for 1978, 1979 and 1980 flood events on the Agua Fria.

Severe flood damages occurred in the December 1978 and February 1980 floods. Damages estimated by the U.S. Army Corps of Engineers approached $5.5 million for the 1978 flood and $7.6 million for the 1980 flood. The bulk of the damage was done to roads and bridges.

2.3 Design Floods

Design floods for the Agua Fria River have been developed by the Los Angeles District of the U.S. Army Corps of Engineers. Floods have been determined for both the existing condition and with the alterations the proposed Arizona Canal Diversion Channel project would have on the hydrologic system. This section presents both sets of design discharges.

2.3.1 Design Discharges for Existing Conditions

The Army Corps of Engineers, Los Angeles District, has conducted extensive hydrology studies in conjunction with the New River and Phoenix City Streams Project and has documented the design flood discharge at various locations in the Agua Fria watershed in the General Design Memorandum, Hydrology Part 1. Table 2.3 lists the discharges for design floods with return periods ranging from the 10-year to the 500-year along the Agua Fria River for the existing condition. In addition, the 100-year discharge of the New River (at the confluence of the Agua Fria River), as derived by the Corps of Engineers, is approximately 53,000 cfs.

The major hydraulic structure that was considered in calculating the design floods was the existing Waddell Dam (Lake Pleasant).

2.3.2 Design Discharge for Future Condition

Plans are underway to construct the Arizona Canal Diversion Channel (ACDC). This project would intercept flood flows before they entered the Arizona Canal and convey them to Skunk Creek. The diverted flows would then pass down the New River to the Agua Fria River. The ACDC, along with Adobe Dam and the New River Dam, was considered in the hydrologic analysis of the future condition. Figure 2.2 shows the existing and proposed dams and the ACDC.

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.

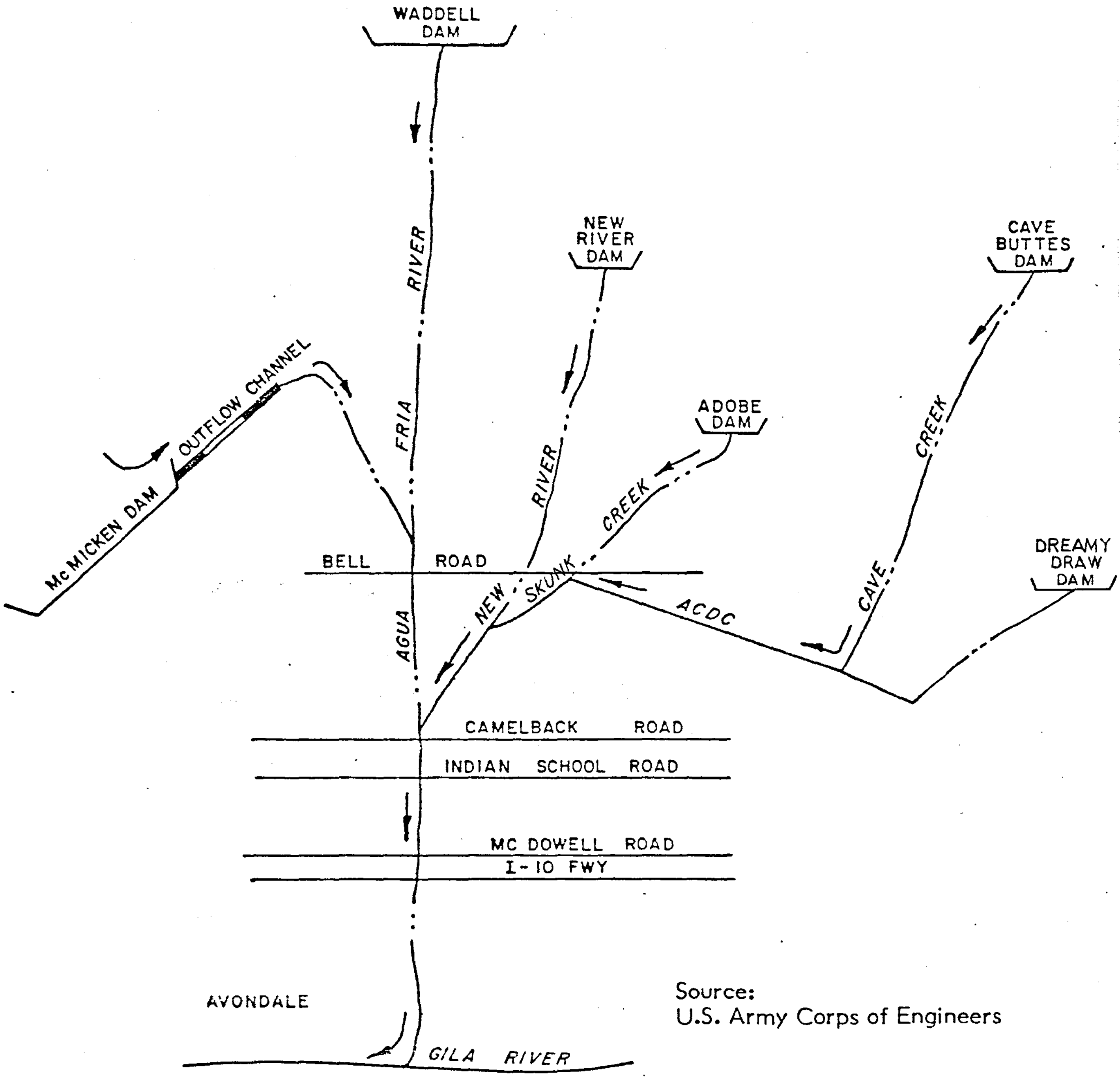


Figure 2.2. Existing and proposed dams and flood control channels.

There was actually very little change in the hydrology caused by these projects since the peak flows on the New River and the Agua Fria River do not coincide in time. When the Agua Fria is flowing at its peak for the 100-year flood, it was estimated that the New River would only be contributing 5,000 cfs. Table 2.4 provides a comparison for the existing and future design discharges on the Agua Fria River, New River and Skunk Creek. From the table, it is apparent that future projects would not significantly alter the hydrology on the Agua Fria River.

2.3.3 Flood Hydrographs

The U.S. Army Corps of Engineers computed a 100-year flood hydrograph for the Agua Fria River. The resulting hydrograph is shown in Figure 2.3. It was constructed based on the largest general summer storm recorded, which occurred August 28-29, 1951. The flood lasts approximately four days with the severe portion of the flood lasting just over one day. This hydrograph was used for the 100-year flood in the sediment routing analysis.

Table 2.4. Comparison of Flood Peak Discharges on Skunk Creek, New River and the Agua Fria River for Existing and Future Watershed Conditions.

Return Period	Skunk Creek Below ACDC and Above New River		New River				Agua Fria River			
			Below Skunk Creek		Above Agua Fria		Below New River		At Avondale	
	With Project	Existing	With Project	Existing	With Project	Existing	With Project	Existing	With Project	Existing
SPF	55	60	68	86	69	84	142	142	131	131
100	35	37	41	58	39	53	95	95	90	90
50	25	27	29	44	27	39	69	69	67	67
25	17	20	19	31	18	28	50	50	47	47
10	9.2	11	10.5	17.0	10.8	15	18[1]	23[1]	17[1]	22[1]
5	5.3	5.6	5.9	8.5	6.8	7.4	8	8.1	8	8
2	1.8	1.2	2.0	1.7	2.7	1.5	3.2	1.6	3.2	1.5

*Data provided by Flood Control District, Maricopa County.

[1]Values conflict with Table 2.3. These values were sent to the Flood Control District of Maricopa County in a letter addressed to Daniel Sagramoso on January 14, 1983, from the Los Angeles District of the Corps of Engineers.

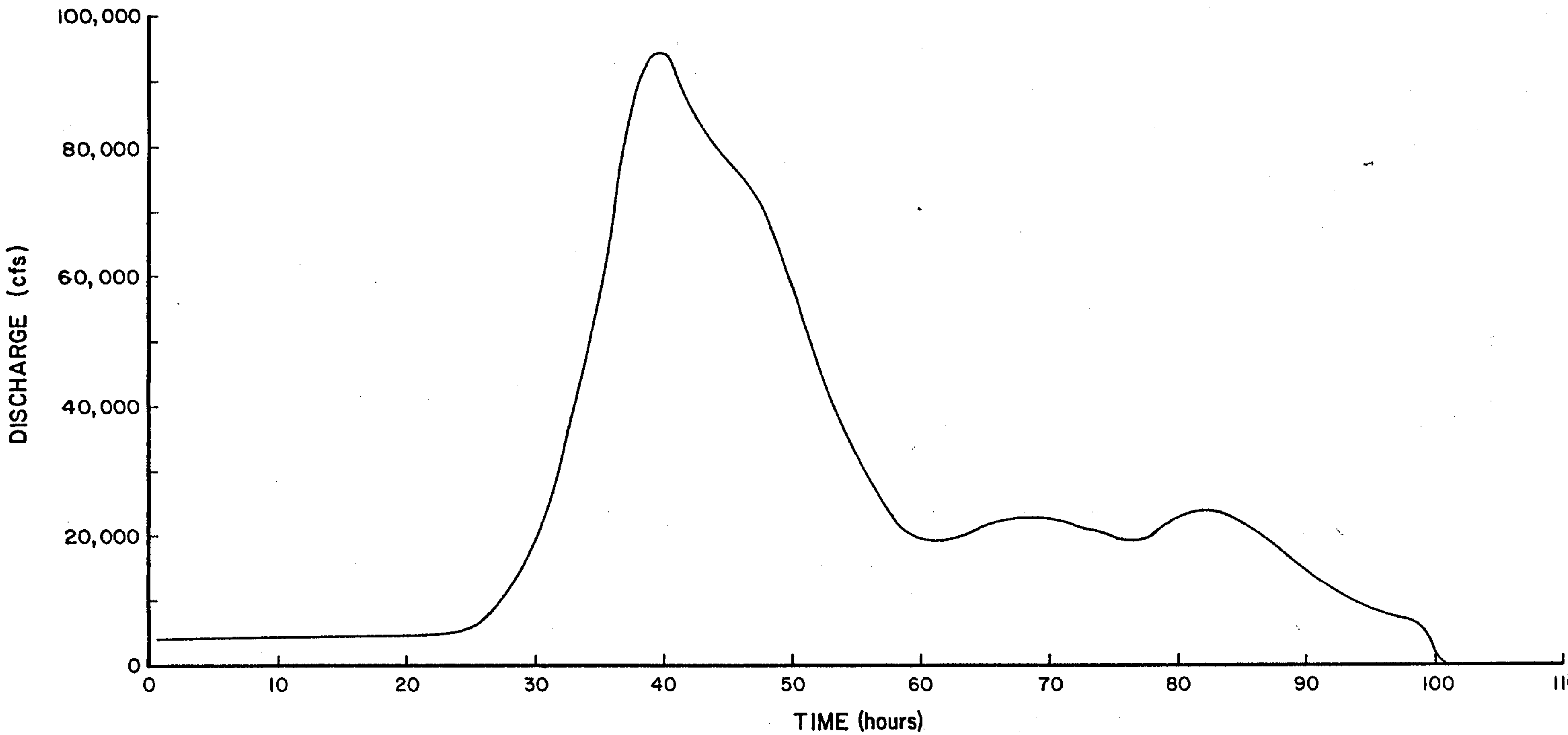


Figure 2.3. 100-year flood on the Agua Fria below the New River confluence as computed by Los Angeles Corps of Engineers, 3/2/81.

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 Indian School Road Bridge and the Roosevelt Irrigation District (RID) flume, where gravel mining operations have reduced the channel width to 500 feet.

3.2 Description of the New River

The New River is the largest tributary of the Agua Fria River. The New River empties into the Agua Fria approximately 1500 feet upstream of Camelback Road. Near the confluence with the Agua Fria, the New River channel width ranges from 300 feet to 700 feet. The New River channel has a much more defined cross section than the Agua Fria with banks approximately eight feet high.

3.3 Bridge Crossings in the Agua Fria

Bridge crossings in the study reach of the Agua Fria River include Indian School Road, I-10, Southern Pacific Railroad (SPRR) and Buckeye Road. Table 3.1 summarizes for each bridge the pier diameter or width, the length the pier extends across the bridge, the bottom elevation of the piers, the current thalweg elevation, the approximate skew angle the flow hits 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 5,959 feet across the channel. 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.

Table 3.1. Pertinent Data of Existing Bridges.

	ISRB	RID Flume	I-10 Bridge	SPRR Bridge	Buckeye Road Bridge
Pier width or diameter	1' 8"	4'	3'4"	6'8"	3'
Pier length	60'	15'	---	27'	70'
Bottom of pier footing	983'	990.5'	945.0'	914.3' to 922.2'	947.2'
Thalweg elevation	1000'	993.5'	967.3'	952.9'	952.6'
Skew of bridge piers to flow direction	30°	---	5°	10°	10°
Low chord	1015.4'	1010.0'	988.5'	966.2'	968.1'

3.4 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 Army Corps of Engineers 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.

The cross-sectional data used for computing backwater profiles were digitized from the August 1981 topographic maps supplied by the Los Angeles District of the Corps of Engineers and modified by SLA. A total of 73 cross sections was used in the study. Summaries of cross section stations and distance between cross sections are included in Appendix A. Also included as a supplement to this report are the cross section locations drawn on the 1981 topographic maps with locations of utility crossings in the Agua Fria.

A Manning's roughness coefficient of 0.035 was used for the main channel and varied from 0.045 to 0.1 on the overbank depending on the location in the flood plain. The Manning's n values were determined from field observations and from the HEC-II input data of the L.A. Corps of Engineers. Flood stages were verified at Avondale from measured stage-discharge data; however, no other gaging stations exist on the Agua Fria between Glendale and the Gila River to verify computed profiles.

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.

Figure 3.2 compares the 10- and 100-year water-surface profiles from Glendale Avenue to the Gila River. The 1981 thalweg profile is also shown in Figure 3.2. The 100-year water-surface profile is approximately three feet higher than the 10-year water surface. The largest depths are in the confined channel reach between ISRB and the RID flume.

The 10-year flood remains in the main channel throughout most of the study reach. The 100-year flood plain is much wider than the 10-year flood, and the has significant overbank bank flow. The 100-year flood plain width ranges from 3,000 feet to 5,000 feet downstream of Thomas Road, and from 5,000 feet to 8,000 feet upstream of Thomas Road to Camelback Road (see Figure 3.3).

Cross-Section Number	Reach Number	Length (feet)	
568.70			
558.60			
544.70	1	6855	
525.20			
520.20			
510.30			
501.45			New River
496.70			
490.90			
483.30			Camelback Road
476.90			
473.30	2	7545	
466.60			
459.50			
452.60			
444.75			
439.45			
433.50			
427.35			ISRB
426.95			
417.75	3	2660	
409.45			
403.90			RID flume
403.75			
398.00			
392.10	4	3933	
385.50			
381.40			
370.50			Thomas Road
358.30			
348.60			
344.20			
334.20	5	7520	
323.20			McDowell Road
316.20			
308.30			
303.00			
294.00			
283.50			
283.40			I-10 Bridge
281.60			
281.50	6	5397	
275.15			
266.80			
254.30			Van Buren Street

Figure 3.1. Reach definition for the Agua Fria River.

Cross-Section Number	Reach Number	Length (feet)	
240.20			
227.95			
212.85	7	4632	
202.30			SPRR
202.00			
201.00			Buckeye Road
200.20			
190.20			
181.55	8	6755	
171.35			
151.35			
135.40			Lower Buckeye Road
130.65			
121.45			
117.35	9	5423	
103.90			
93.80			
82.60			
75.00			Broadway Road
70.40	10	2970	
61.90			
53.60			

Figure 3.1 (continued)

Table 3.2. Average Flow Velocity (V), Hydraulic Depth (D) and Effective Width (EW) for the 10-Year Flood Event.

	LEFT FLOODPLAIN			MAIN CHANNEL			RIGHT FLOODPLAIN		
REACH	V	D	EW	V	D	EW	V	D	EW
1	0.47	0.08	545.	5.24	3.26	1675.	1.05	0.47	2794.
2	0.46	0.49	561.	5.06	2.69	2345.	0.32	0.22	414.
3	0.00	0.00	0.	7.35	5.13	796.	0.00	0.00	0.
4	0.62	0.50	515.	6.39	5.47	853.	0.00	0.00	0.
5	1.02	0.58	822.	6.91	4.66	868.	1.61	0.64	1509.
6	0.95	0.57	1201.	6.30	4.31	1042.	0.81	0.38	153.
7	0.54	0.56	19.	6.86	4.33	975.	0.00	0.00	0.
8	1.27	0.64	584.	4.86	2.89	1915.	1.31	0.65	786.
9	0.10	0.04	159.	4.61	2.50	2422.	1.08	0.62	98.
10	0.08	0.03	196.	5.32	2.95	1782.	0.00	0.00	0.

Table 3.3. Average Flow Velocity (V), Hydraulic Depth (D) and Effective Width (EW) for the 25-Year Flood Event.

	LEFT FLOODPLAIN			MAIN CHANNEL			RIGHT FLOODPLAIN		
REACH	V	D	EW	V	D	EW	V	D	EW
1	1.09	0.52	751.	5.95	3.92	1691.	1.56	1.64	3172.
2	0.81	1.29	1241.	5.25	3.61	2395.	1.10	2.12	1423.
3	0.15	0.21	231.	8.45	7.08	819.	0.13	0.14	995.
4	0.96	2.02	769.	7.68	6.85	873.	0.86	1.20	1536.
5	1.70	0.90	1043.	7.85	6.13	872.	2.27	0.85	2809.
6	1.81	0.26	2698.	7.17	5.79	1123.	1.28	0.36	226.
7	0.77	0.11	1050.	8.08	5.84	1006.	0.92	0.91	577.
8	1.96	1.35	669.	5.77	3.87	1915.	2.09	1.45	805.
9	0.24	0.18	161.	5.33	3.27	2691.	1.16	0.82	123.
10	0.22	0.15	199.	6.32	4.07	1825.	0.00	0.00	0.

Table 3.4. Average Flow Velocity (V), Hydraulic Depth (D) and Effective Width (EW) for the 50-Year Flood Event.

	LEFT FLOODPLAIN			MAIN CHANNEL			RIGHT FLOODPLAIN		
REACH	V	D	EW	V	D	EW	V	D	EW
1	1.69	0.95	1228.	6.54	4.55	1700.	1.81	2.25	3303.
2	1.33	1.31	2161.	5.51	4.60	2426.	1.42	1.51	1750.
3	0.34	0.39	2370.	8.73	9.45	822.	0.56	1.22	1257.
4	1.67	2.17	1288.	8.58	7.71	874.	1.50	1.78	2444.
5	2.16	1.48	1092.	8.63	7.03	872.	2.59	1.51	3229.
6	2.12	0.99	3563.	7.88	6.78	1123.	1.66	0.94	344.
7	1.00	1.17	1884.	8.81	7.15	1012.	1.26	2.57	632.
8	2.46	1.41	709.	6.55	4.88	1915.	2.62	1.55	822.
9	0.43	0.30	163.	5.95	3.89	2888.	1.21	1.03	139.
10	1.20	0.91	953.	6.97	5.10	1855.	0.00	0.00	0.

Table 3.5. Average Flow Velocity (V), Hydraulic Depth (D) and Effective Width (EW) for the 100-Year Flood Event.

	LEFT FLOODPLAIN			MAIN CHANNEL			RIGHT FLOODPLAIN		
REACH	V	D	EW	V	D	EW	V	D	EW
1	2.16	1.18	1508.	7.17	5.22	1704.	2.27	2.67	3697.
2	1.74	1.77	2679.	5.84	5.59	2426.	1.80	2.02	2071.
3	0.98	1.37	3270.	8.47	12.05	825.	1.17	2.37	1945.
4	2.10	3.19	1448.	9.09	8.71	874.	1.95	3.08	2519.
5	2.54	2.10	1146.	9.27	7.84	872.	3.03	2.35	3438.
6	2.54	1.69	3798.	8.55	7.66	1123.	1.96	1.44	402.
7	1.35	2.81	1955.	9.31	8.28	1028.	2.00	2.80	681.
8	2.84	1.94	900.	7.31	5.67	1916.	3.09	2.61	840.
9	1.13	2.18	228.	6.57	4.50	2996.	1.82	3.07	154.
10	2.77	1.22	2590.	7.43	5.81	1882.	0.00	0.00	0.

FIGURE 3.2

COMPARISON OF 10 YEAR AND 100 YEAR WATER SURFACE PROFILES

WATER SURFACE PROFILES FOR 10YR AND 100YR FLOOD EVENTS - AGUA FRIA RIVER, AS-IS

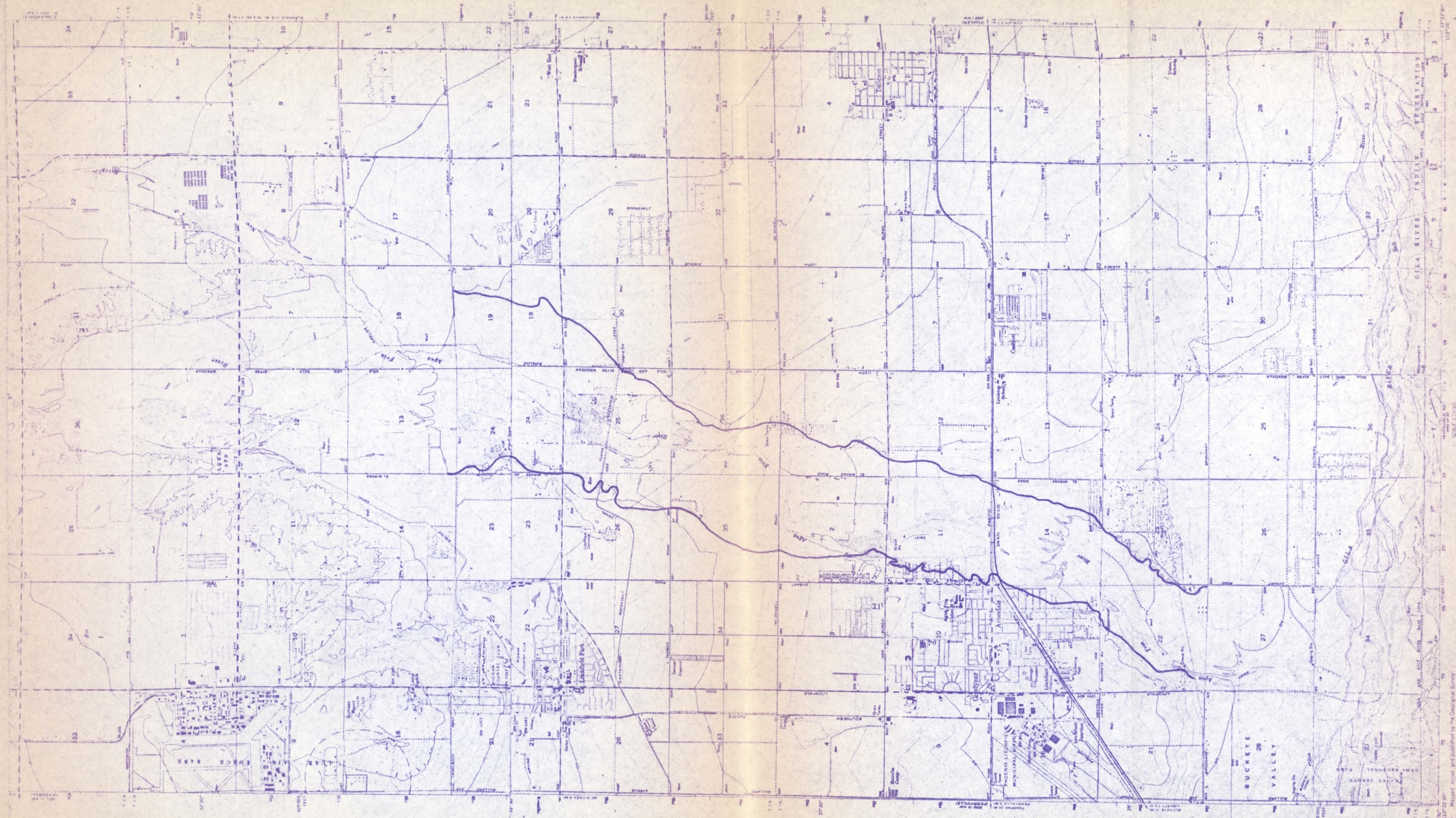


Figure 3.3 Approximate 100 year flood plain from Camelback Road to Broadway

FIGURE 4.10

COMPARISON OF 1973 AND 1981 THALWEG ELEVATIONS

CHANGE IN THALWEG ELEVATION FROM 1973 TO 1981 - AGUA FRIA RIVER

A summary of velocities for the 10- and 100-year floods is presented in Appendix A. Main channel velocities generally range from 5 to 7 feet per second (fps) for the 10-year peak discharge and 7 to 10 fps for the 100-year peak discharge. Overbank velocities are considerably less than than the main channel and generally range from 1 to 3 fps for the 100-year peak discharge.

3.5 Hydraulic Characteristics of the New River

A backwater profile was determined for the New River extending 5,500 feet upstream of the confluence with the Agua Fria for the 100-year flood. The 100-year peak discharge of 39,000 cfs, which considers the New River Dam and ACDC in place, was used for analysis. The backwater profile was computed to provide baseline information on the flood plain width, overbank flow, and main channel and overbank velocities in the vicinity of the confluence with the Agua Fria. This information will be particularly useful for future channelization design.

The main channel of the New River does not contain the 100-year discharge. Approximately 3,000 ft upstream of the confluence with the Agua Fria, flow starts to overtop the south bank. This flow will inundate the fields in the south overbank. Approximately 14,000 cfs will flow over the south bank. Velocities in the main channel range from 8 to 10 fps, while the velocities in the overbanks range from 2 to 4 fps.

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 in-stream activities and site visits. The qualitative analysis documents the changes in the system, whether man-induced or natural, and provides the necessary understanding of the system to proceed with the quantitative engineering geomorphic and mathematical modeling analyses. A qualitative assessment of the Agua Fria was made from the confluence with the Gila River to Waddell Dam.

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 controlled by 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 river generally wide 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, ~~Glendale~~ AGUA FRIA - Dysart Drain, Grand Canal, and I-10 collector channel. The Arizona Canal Diversion Channel (ACDC) will be constructed parallel to the existing Arizona Canal for transporting floodwaters from Cudia City Wash, Dreamy Draw Wash, 10th and Northern Avenue drains and Cave Creek.

Several reservoirs exist in the watershed and include 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 currently being considered for construction and include the New River Dam (New River) and new Waddell Dam (Agua Fria).

4.3 Vegetation and Land Use

In general, the vegetation is sparse in the Agua Fria watershed below Waddell Dam. Cacti grow throughout the area along with other desert shrubs on the level areas at the lower elevations. A few stunted trees including Juniper, Paloverde, Mesquite, Iron Wood and Scrub Oak exist among the shrubs. The vegetation tends to be thicker along and adjacent to the stream courses. Perennial grasses form a very small portion of the vegetation, but a good cover of annual grasses occur after the winter rains.

Agriculture represents the predominant land use in the study area. Cotton is the major commercial crop.

Urban development is rapidly increasing in several areas adjacent to the river, particularly Avondale, Litchfield Park and Sun City West. Communities near the river that are encroaching upon the flood plain include Surprise, El Mirage, Sun City, Youngtown, Peoria, Glendale, Goodyear and Avondale.

Sand and gravel miners have actively extracted material from the flood plain and channel bed of the Agua Fria. From examination of the 1981 aerial photographs, there are 8 large in-stream and flood-plain operations in existence. Table 4.1 summarizes the locations of these operations. The United Metro operation located near Glendale Avenue and the Allied Concrete and Phoenix Sand and Rock operations near Indian School Road are the largest operations in the study reach.

Table 4.1. Gravel Pit Locations.

Mining Location	Remarks
Beardsley Road	On east channel
Grand Avenue	2,200 ft above Grand Avenue on west channel
Olive Avenue	On east and west bank
Northern Avenue to Glendale Avenue	Large operation across the flood plain extending to 2,000 ft downstream of Glendale Avenue
Camelback Road	Extends from Camelback Road to 1,000 ft downstream of the roadway on west bank flood plain
Indian School Road to RID	Significant east and west overbank operations covering 4,000 ft
McDowell Road	Upstream, in main channel
Van Buren Street	Approximately 500 ft upstream of Van Buren extending to 1,000 ft downstream

*Data from 1981 aerial photographs

4.4 Geology and Physiography

Approximately 70 percent of the Agua Fria River basin is mountainous and the remaining 30 percent is valley. The mountain areas above 3,000 feet in elevation are characterized by rugged terrain and steep gradients. The lower areas consist 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 both are described. 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, i.e. 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). This alluvium contains appreciable amounts of firmly cemented fine-grained soils of low permeability. However, most of the alluvium is unconsolidated sand and gravel with high permeabilities.

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 soils. 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.

4.5 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.

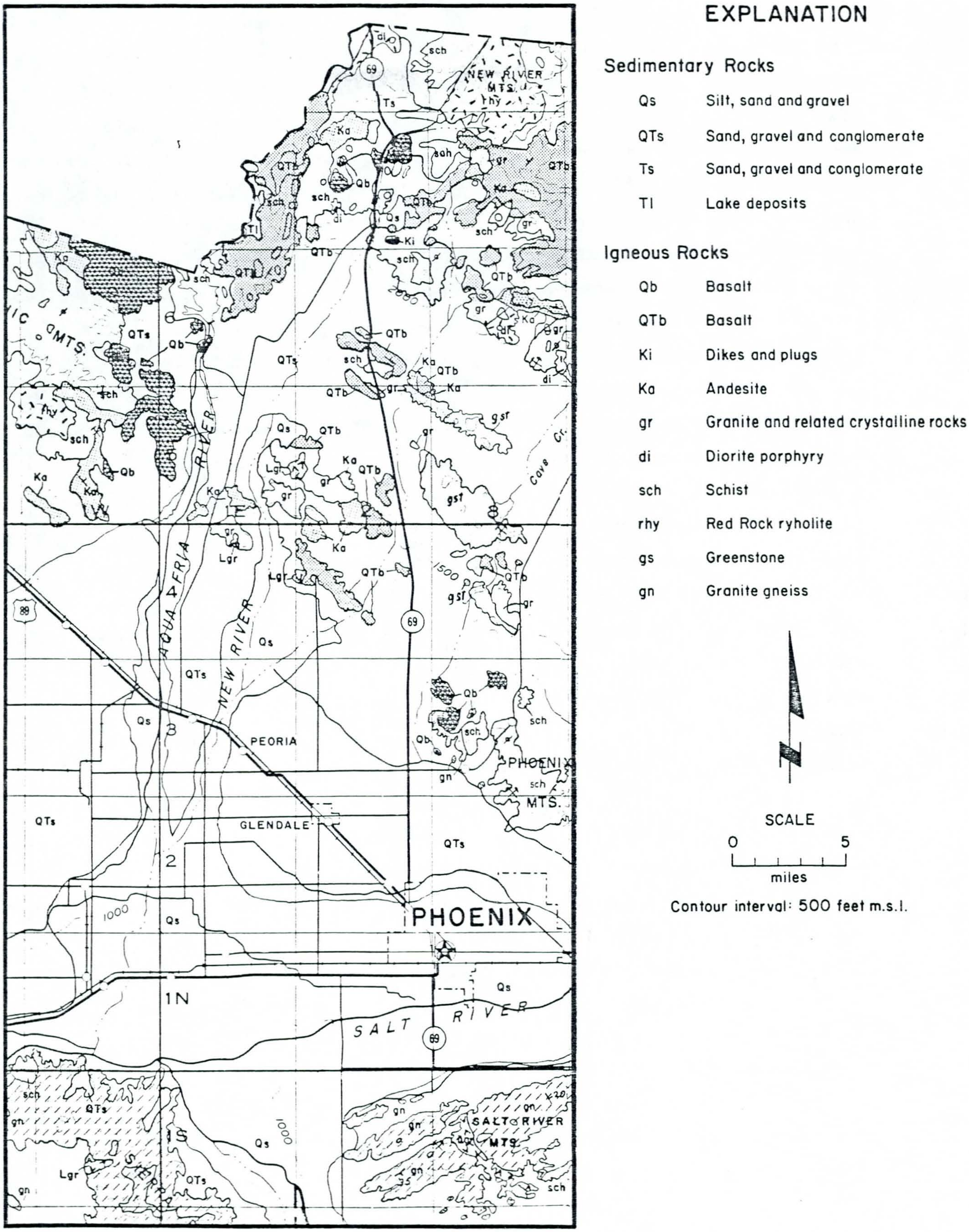


Figure 4.1. Geologic map of part of the Agua Fria River Basin, ~~New Mexico~~. (From Wilson et al., 1957).
ARIZONA

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 cobble with thicknesses ranging from 4 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 4.3 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 the thickness is about one foot. A close-up shot with a grid overlaying the gravel layer is in Figure 4.4. The squares of the grid are two inches on a side. The largest particle size measures about four inches.

The distance to the gravel layer below the surface varies from 2 to 7 feet throughout the study reach. In a few test pits clay layers were found below the gravel layers. These clay layers will slow down 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.

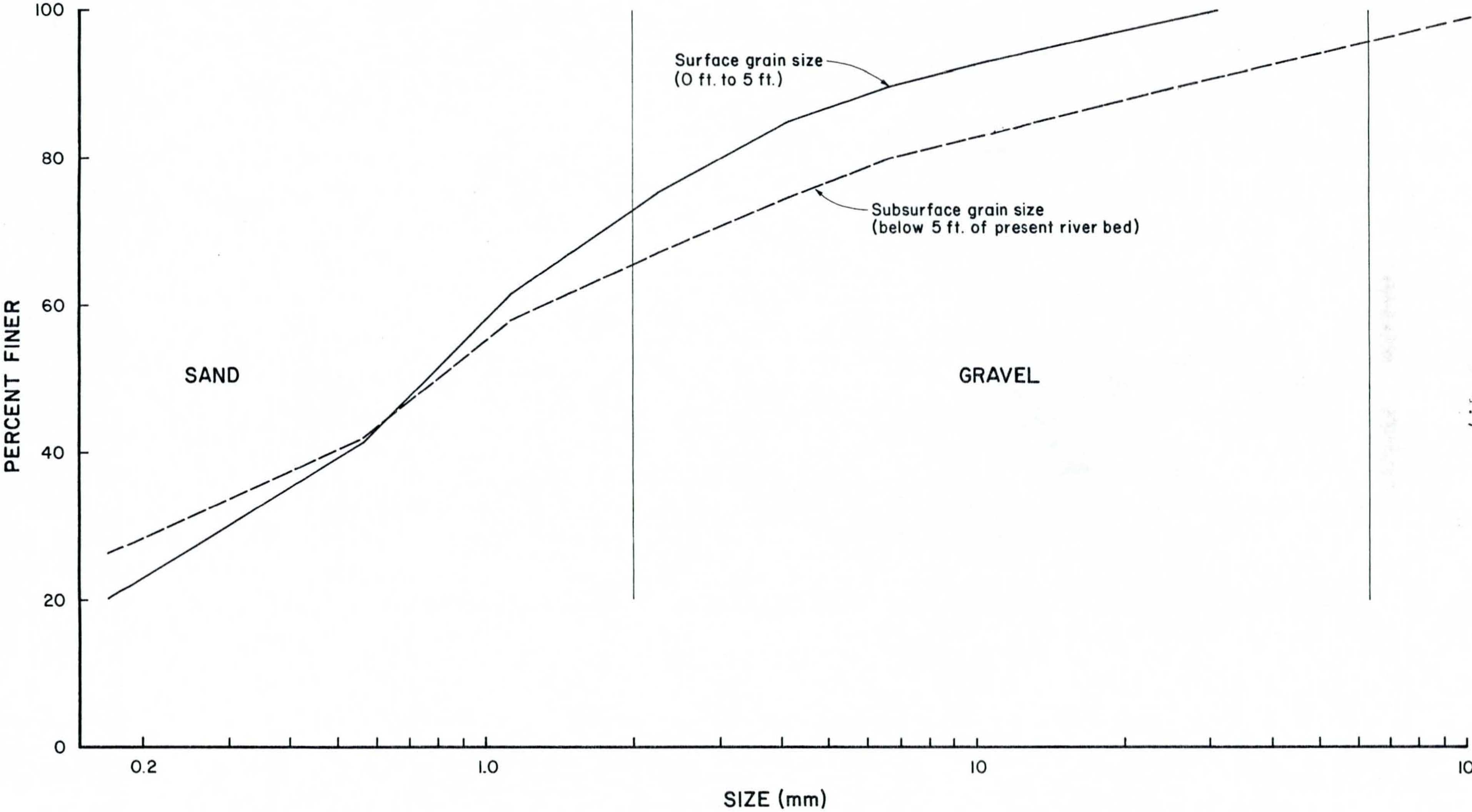


Figure 4.2. Surface and subsurface grain size distributions of the Agua Fria near Indian School Road Bridge.

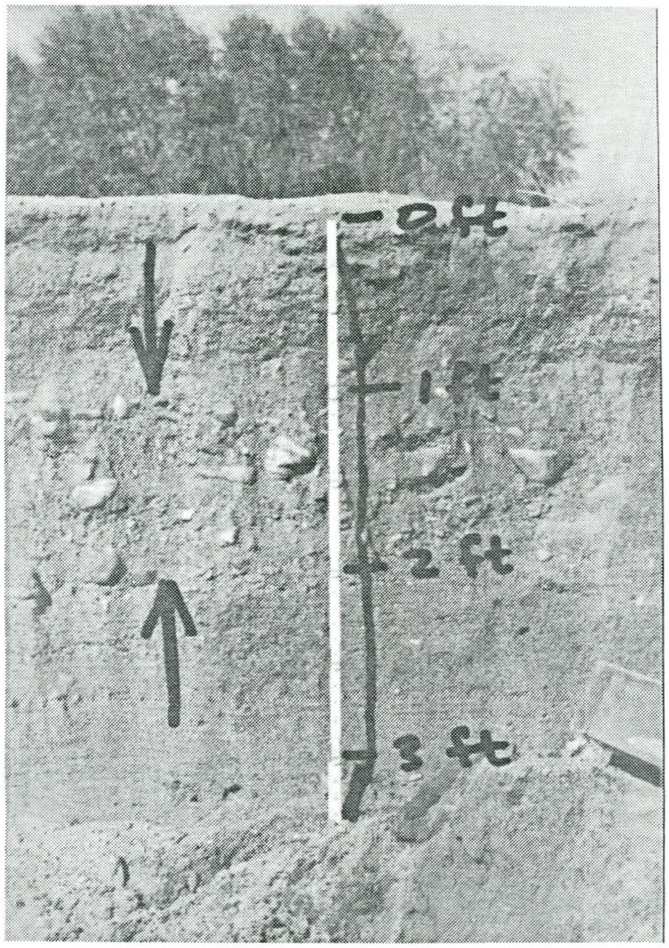


Figure 4.3. Gravel layer below the river bed of Agua Fria River approximately 800 feet downstream of Indian School Road Bridge.

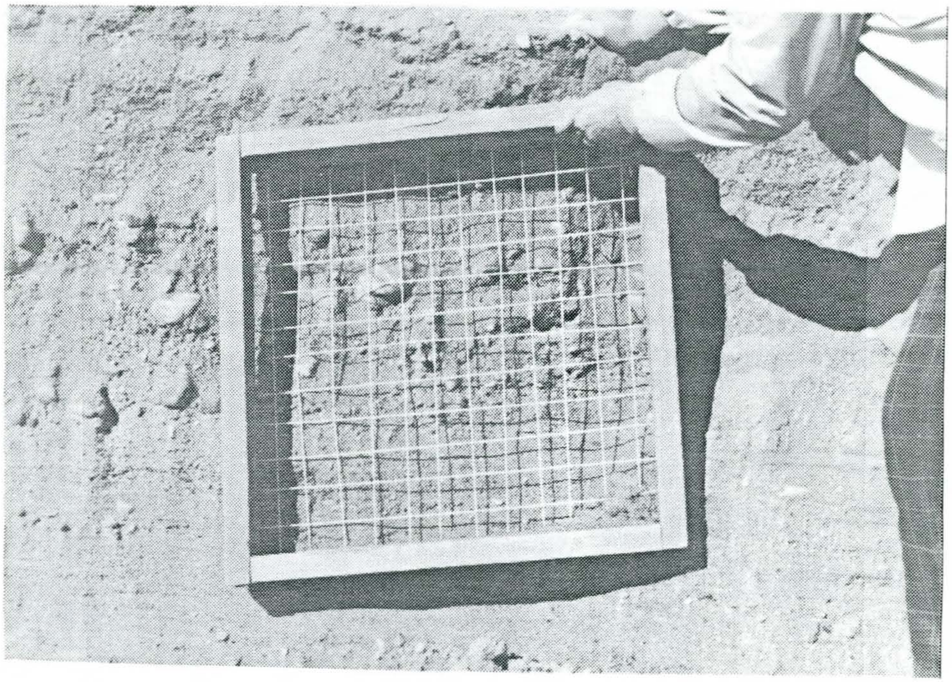


Figure 4.4. Close-up of the gravel layer below the river bed of Agua Fria River, approximately 800 feet downstream of Indian School Road Bridge.

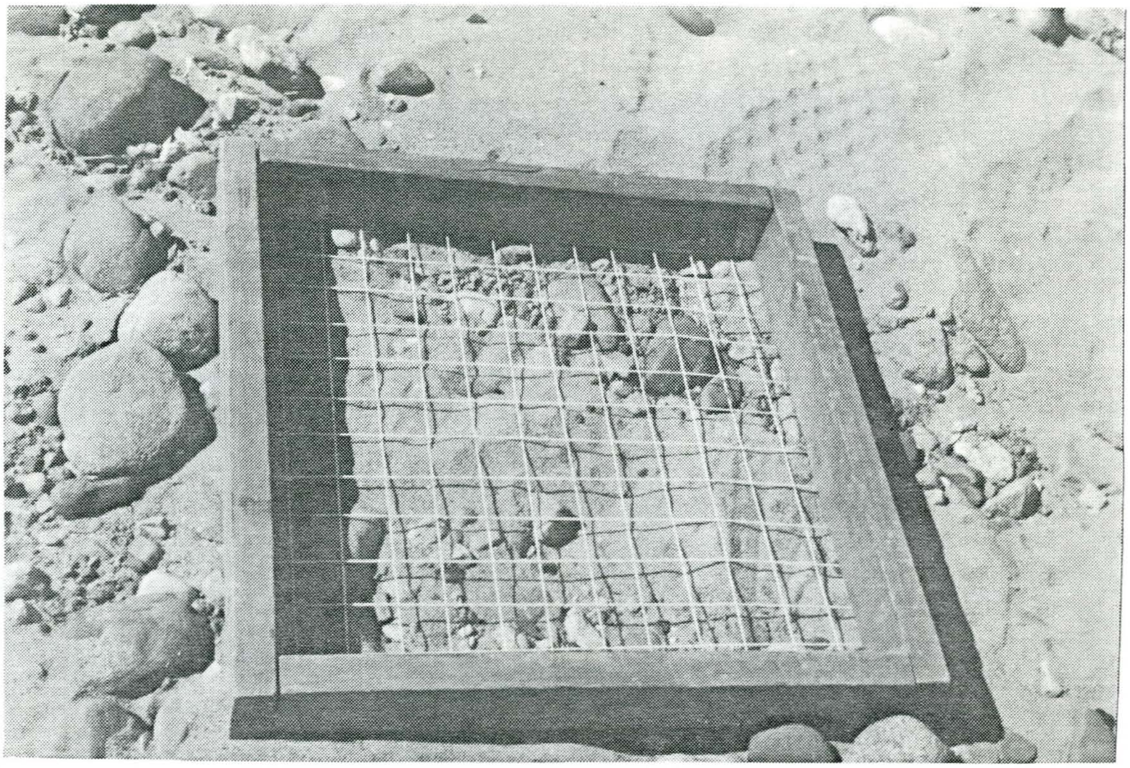
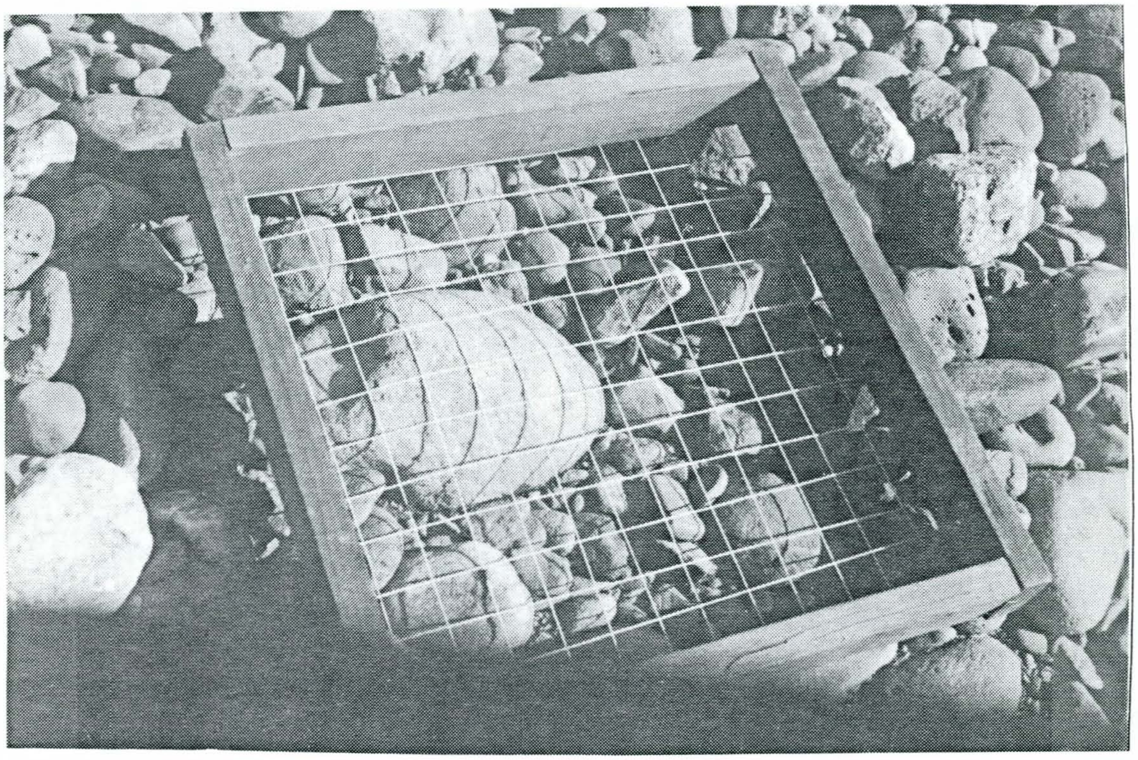


Figure 4.5. River bed materials of the Agua Fria River upstream of the confluence with New River.



* The square is two inches on each side.

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

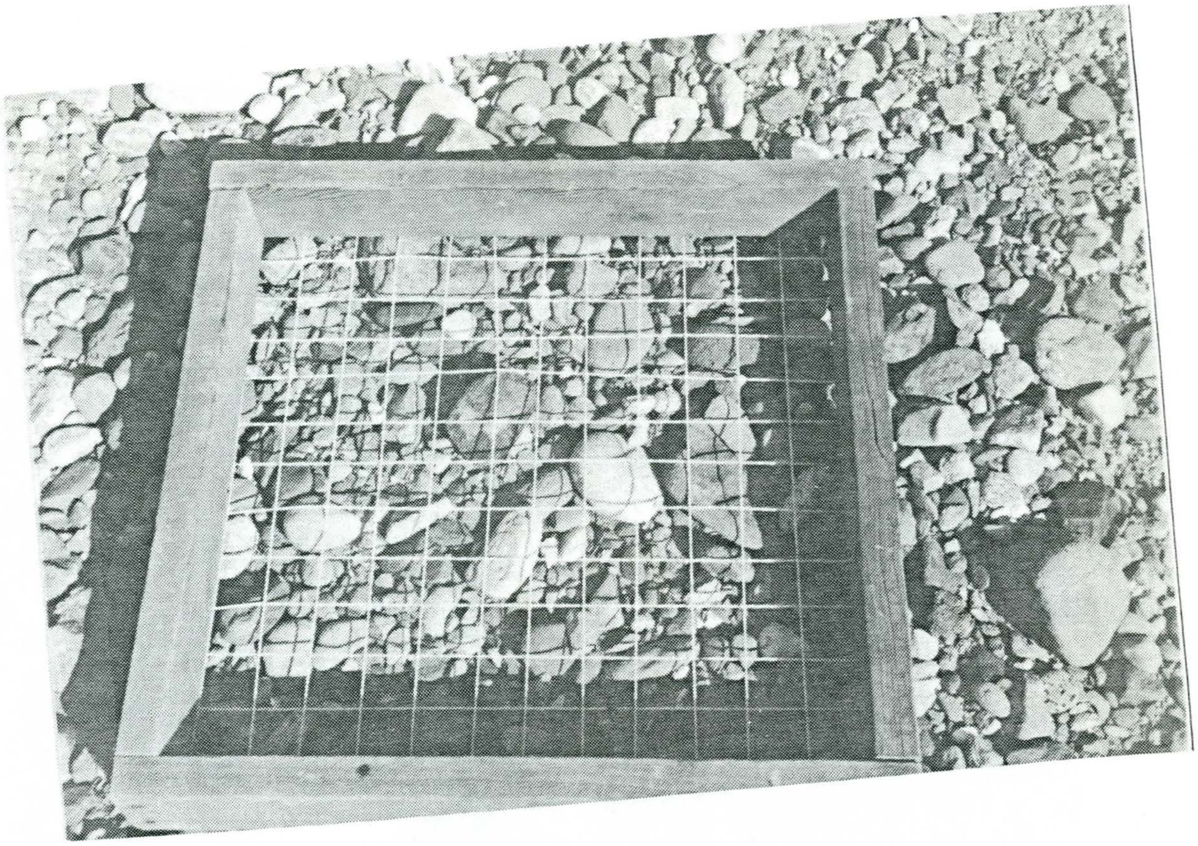


Figure 4.7. Bed-material of the Agua Fria River near Beardsley flume.

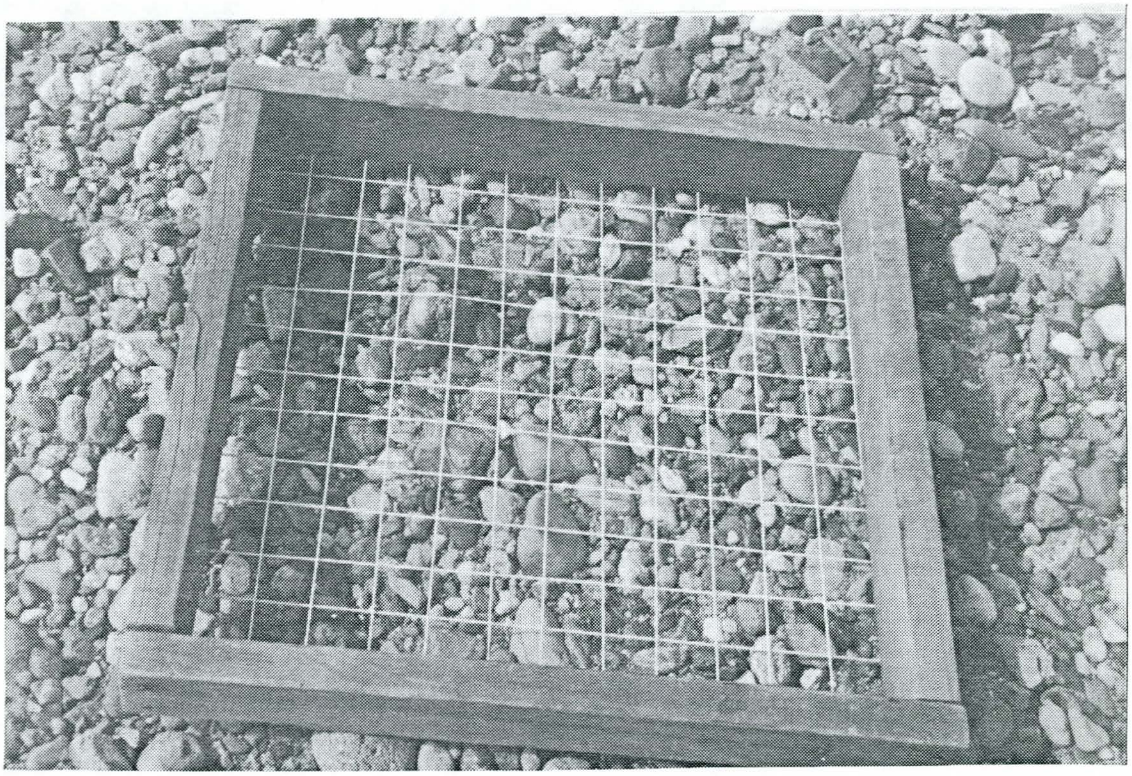


Figure 4.8. Bed-material of the Agua Fria River near Grand Avenue.

Figure 4.9 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.

In summary, the bed material in the study reach is composed of sand and fine gravels. Gravel layers which were formed from alluvial deposits were apparent in the subsurface sediment samples. In many locations gravel layers 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 6 inches and the gravel and cobble layer varies between 4 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 size distributions of the sediment samples collected for this study are given in Appendix B. The typical surface and subsurface sediment distributions of the Agua Fria River shown in Figure 4.2 were used in the sediment transport computations. The potential sediment reduction due to armoring was considered in the evaluation of the long-term channel response.

4.6 Changes in Channel Morphology

Significant changes have occurred in the Agua Fria River over the years. Dynamic conditions in the Agua Fria can be best illustrated by comparing the thalweg elevation between 1973 and 1981 (see Figure 4.10). The river bed has lowered throughout almost all of the study reach. It should be noted that contour intervals on the 1972 map are 4 feet and on the 1981 map the contour intervals are 2 feet. The accuracy of the 1972 map is $\pm$ 2 feet and the 1981 map is $\pm$ 5 feet. Thus, the magnitude of the difference in thalwegs is masked by the $\pm$ 2.5 feet combined map tolerance. Most of the channel morphology changes can be directly attributable to man's activities in and near the Agua Fria.

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

Upstream developments in the Agua Fria and New River have affected the hydrology and subsequently the river response. In some areas urban and agricultural encroachments have caused entrenchment of the main channel. Sand and gravel mining operations have extracted material from the main channel and overbanks altering the shape of the river. These activities and their subsequent results on channel morphology are discussed below.

4.6.1 Upstream Developments

The existing and proposed dams as well as 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 proposed structures include New River Dam, a new Waddell Dam, and the Arizona Canal Diversion Channel.

4.6.1.1 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 controlled by the dam. The dam 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. Through continuous degradation, finer sediments were removed from the river bed leaving coarser particles on the surface forming an armor layer. As stated previously, the channel downstream of Waddell Dam has armored to approximately Bethany Home Road.

4.6.1.2 New Waddell Dam

Large spills have occurred over Waddell Dam in 1978, 1979 and 1980, initiating reinvestigation of the need to construct a new dam for flood control purposes. The flood control analysis for a new Waddell Dam has been conducted by the Central Arizona Water Control Study (CAWCS). The new dam would be located about one-fourth of a mile downstream of the existing dam. The new 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 the increased detention time sediment would have within the reservoir. However, flood discharge releases will be significantly reduced, so the overall effect of construction of new Waddell Dam will be increased downstream flood control and a reduction in downstream sediment transporting capacity. The 100-year flood peak at Camelback Road is assumed to be reduced approximately in half with construction of a new Waddell Dam.

4.6.1.3 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 4.11). 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 ACDC. Dreamy Draw Dam, completed in July 1973, is located 1.8 miles above ACDC and controls about 65 percent of the Dreamy Draw watershed (1.3 square miles). Cave Buttes Dam is located about 11 miles upstream of the confluence of Cave Creek and ACDC and controls 87 percent of the Cave Creek watershed (195 square miles). The net effect of ACDC will be an increase in water and sediment discharge into the New River. However, the construction of the New River Dam will more than offset the increase caused by the ACDC flows.

4.6.1.4 Adobe Dam

Adobe Dam was constructed on Skunk Creek, about 7 miles north of Bell Road and 1 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 to the New River.

4.6.1.5 New River Dam

The New River Dam is to be constructed on the New River about 8 miles upstream from the confluence with Skunk Creek. With the proposed dam, about 164 square miles of the existing 340 square-mile New River basin will be regulated. Peak discharges for floods will be significantly reduced as explained in Chapter II.

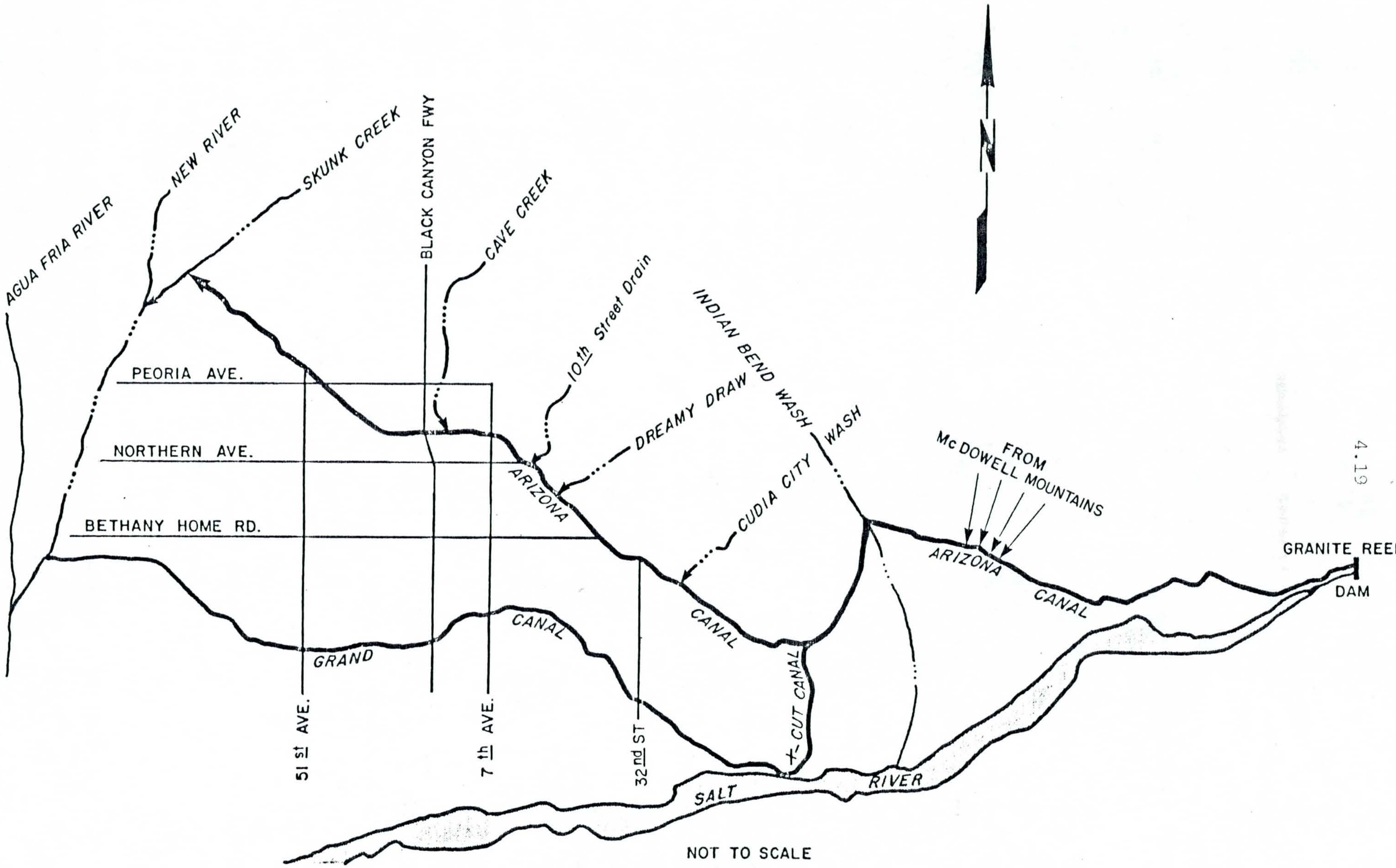


Figure 4.11. Location of the Arizona Canal and the Study Reach.

4.6.1.6 McMicken Dam

McMicken Dam, which controls the runoff from about 240 square miles of Trilby 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.

4.6.2 Agricultural and Urban Development

Agricultural and urban developments have led to some encroachment on the Agua Fria flood plain. An example of agricultural related encroachment is the river section just upstream of McDowell Road. Figure 4.12 shows the bankfull width has been reduced to 500 feet due to farming on the east and west overbanks.

Urbanization has taken place at several locations along the Agua Fria River. Population growth at Avondale, Sun City, Goodyear and Litchfield Park have necessitated the construction of several bridges. Since the flood plain is generally wide, it has not been economically feasible to construct the bridges across the entire flood plain. Thus bridges have been constructed that have encroached the flood plain and consequently entrenched the main channel upstream of the bridge crossing.

In general, agricultural and urban developments have not severely encroached upon the natural flood plain of the Agua Fria. However, if future population predictions are correct, there will be a demand for further urban developments along the river and consequently more encroachments upon the existing flood plain.

4.6.3 Sand and Gravel Mining

Sand and gravel mining has been the most active industry in the study area. A significant amount of sand and gravel has been removed from the river bed and overbank areas, resulting in degradation problems.

Gravel mining effects are not just limited to the gravel pit area. Headcuts can initiate at the upstream boundary of the gravel pit and extend large distances. 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

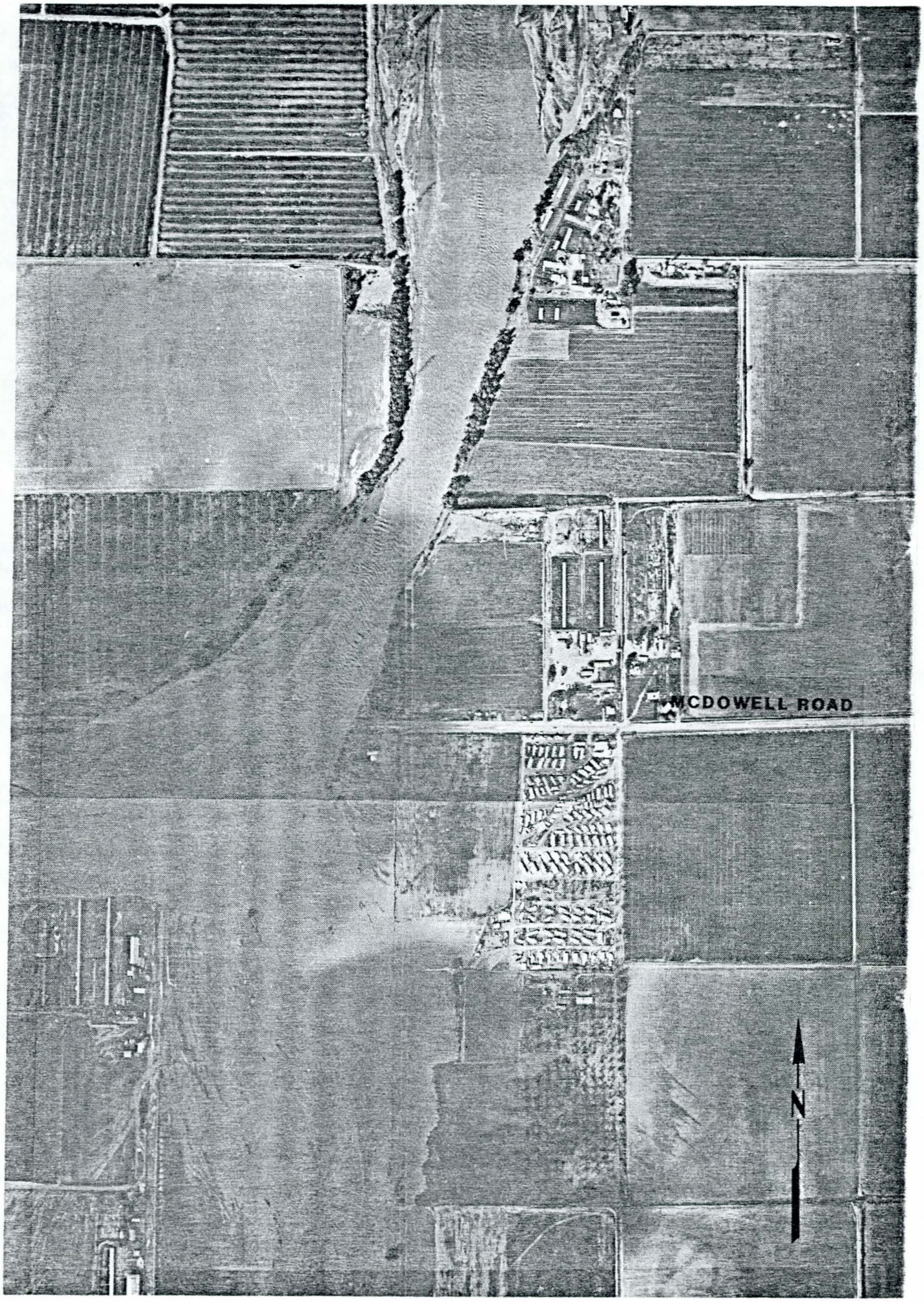


Figure 4.12. 1980 aerial photo showing confinement of the river channel due to agricultural development in the Agua Fria River upstream of McDowell Road.

increased channel instability. In addition, sand and gravel mining operations often construct levees to protect their pits from the river. If constriction of the river due to the levees is excessive, channel degradation can be induced.

4.6.3.1 Sand and Gravel Mining Near Indian School Road

Instream sand and gravel mining near Indian School Road (ISR) began in the late 1950's. Figure 4.13 shows the channel near Indian School Road as it existed in 1964. Mining was concentrated in the west branch of the low-flow channel above ISR and near the east bank, halfway between ISR and the RID flume.

Prior to 1964, the east branch of the low-flow channel was more defined than the west branch. By 1964, the west low-flow channel had deepened and widened due to the extraction of sand and gravel. The west low-flow branch became the dominant low-flow channel prior to the construction of Indian School Road Bridge in 1970. However, the river started migrating gradually eastward after construction of the bridge. Examination of the 1980 aerial photograph reveals the channel upstream of the bridge has shifted 700 feet east of the east abutment. The migration of the channel to the east (see Figure 4.14) upstream of the bridge resulted in the flow attacking the bridge piers at a severe angle during the 1980 flood.

Downstream of the bridge, mining is readily apparent on both overbanks. Dikes have been constructed to protect the gravel pits from the flow (see Figure 4.14). This causes a further entrenchment of the channel.

4.6.3.2 Sand and Gravel Mining Near Glendale Avenue

Sand and gravel pits near Glendale Avenue were observed first in the 1964 aerial photographs. The pit in 1964 was halfway between Northern Avenue and Glendale Avenue east of the low-flow channel.

In 1970 the sand and gravel pits covered the entire flood plain midway between Northern Avenue and Glendale Avenue extending to Glendale Avenue. Sand and gravel pits were also observed immediately downstream of Glendale Avenue.

Figure 4.13. 1964 aerial photo of the Agua Fria River near Indian School Road.

Figure 4.14. 1980 aerial photo of the Agua Fria River near Indian School Road.

Major operations north and northeast of the 1964 gravel pit locations were observed in the 1976 aerial photograph (see Figure 4.15). Although there were dikes to isolate the mining area from the low-flow channel, massive excavations within the river have caused significant impacts on the hydraulic and geomorphic characteristics.

Figure 4.16 is an aerial photograph taken during the December 1978 flood. The flow broke out of the main channel and inundated the large pit immediately downstream of Northern Avenue. The low-flow channel constructed by United Metro for transporting low flows up to 30,000 cfs was not operating at full capacity during this flood.

The flow pattern shown in Figure 4.16 is the result of mining and bridge construction. Without these modifications, the natural flow pattern would be that of a typical braided river as shown in Figure 4.17.

4.7 Conclusions

1. The Agua Fria is a braided ephemeral stream, and is quite unstable. The river flows in a canyon reach for several miles below Waddell Dam before it enters the valley and exhibits its braided characteristics.

2. The thalweg of the Agua Fria has dropped between 0.5 feet and 3 feet throughout most of the study reach between 1973 and 1981. Not only has the thalweg dropped, but the entire cross section has lowered. It should be noted that contour intervals on the 1972 maps are 4 feet and contour intervals on the 1981 maps are 2 feet. The magnitude of the degradation may be masked according to accuracy of contour intervals.

3. Sediment samples obtained during the site visit show significant armoring of the river bed from Waddell Dam to Bethany Home Road. Between Bethany Home Road and Camelback Road the river bed has patches of coarse material; however, large amounts of sand are evident in the bed and banks. Below Camelback Road the Agua Fria is composed of sand and fine gravels except near Thomas Road, McDowell Road and Van Buren Street crossings.

4. Subsurface samples show thin layers of gravel and cobble exist. The layers vary in thickness between 4 and 14 inches and range in depth below the present river bed from 2 to 7 feet. Thin armor layers can develop on the river bed surface to control the eventual grade; however, the armor layer will be very susceptible to mechanical disruption.

5. There is no evidence of geologic controls present in the lower Agua Fria Valley to control the bed elevation of the Agua Fria.

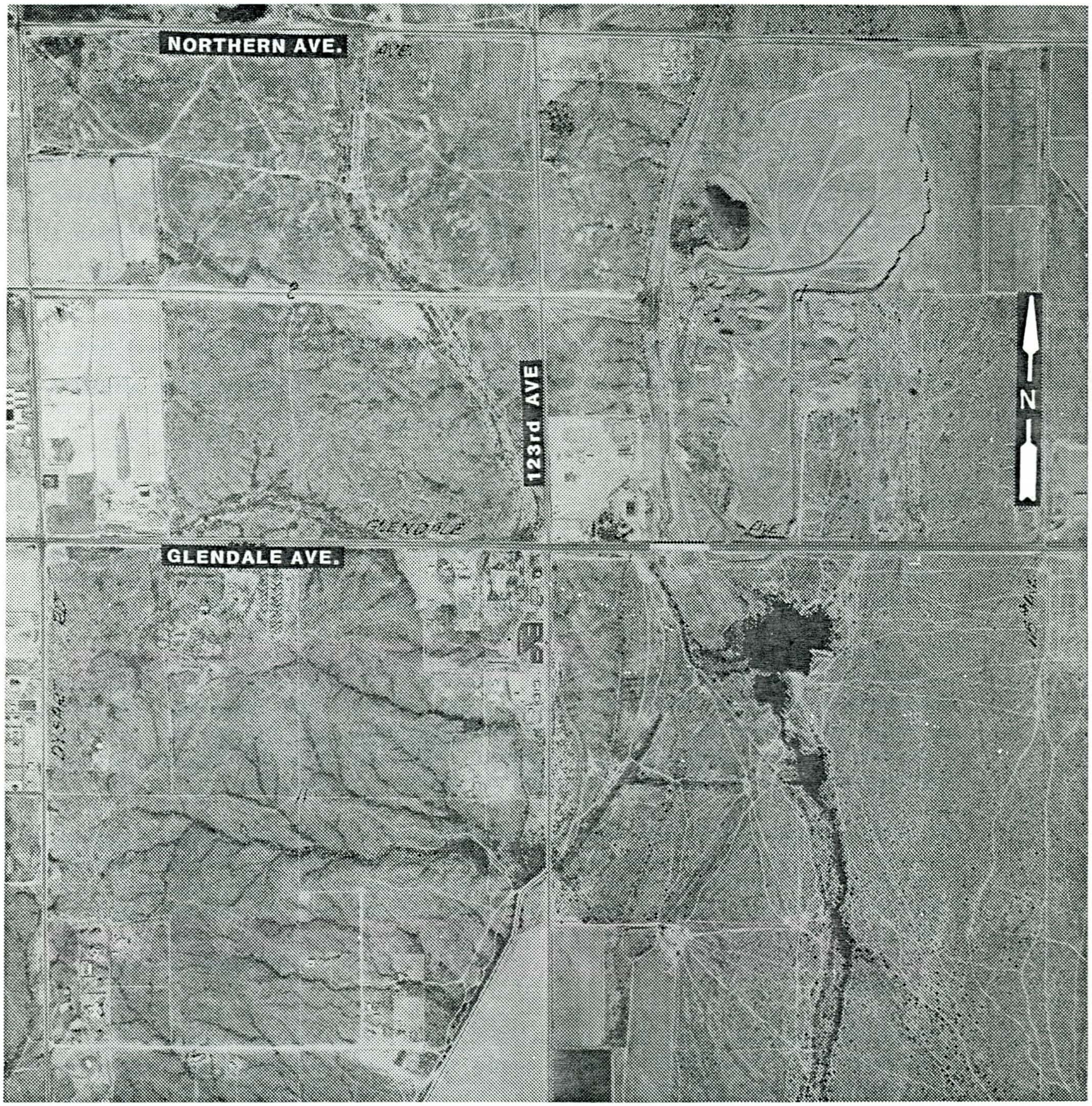


Figure 4.15. 1976 aerial photo of the Agua Fria River showing sand and gravel mining between Northern Avenue and Glendale Avenue.

Figure 4.16. Aerial photo showing flow pattern between Northern Avenue and Glendale Avenue during December 1978 flood.

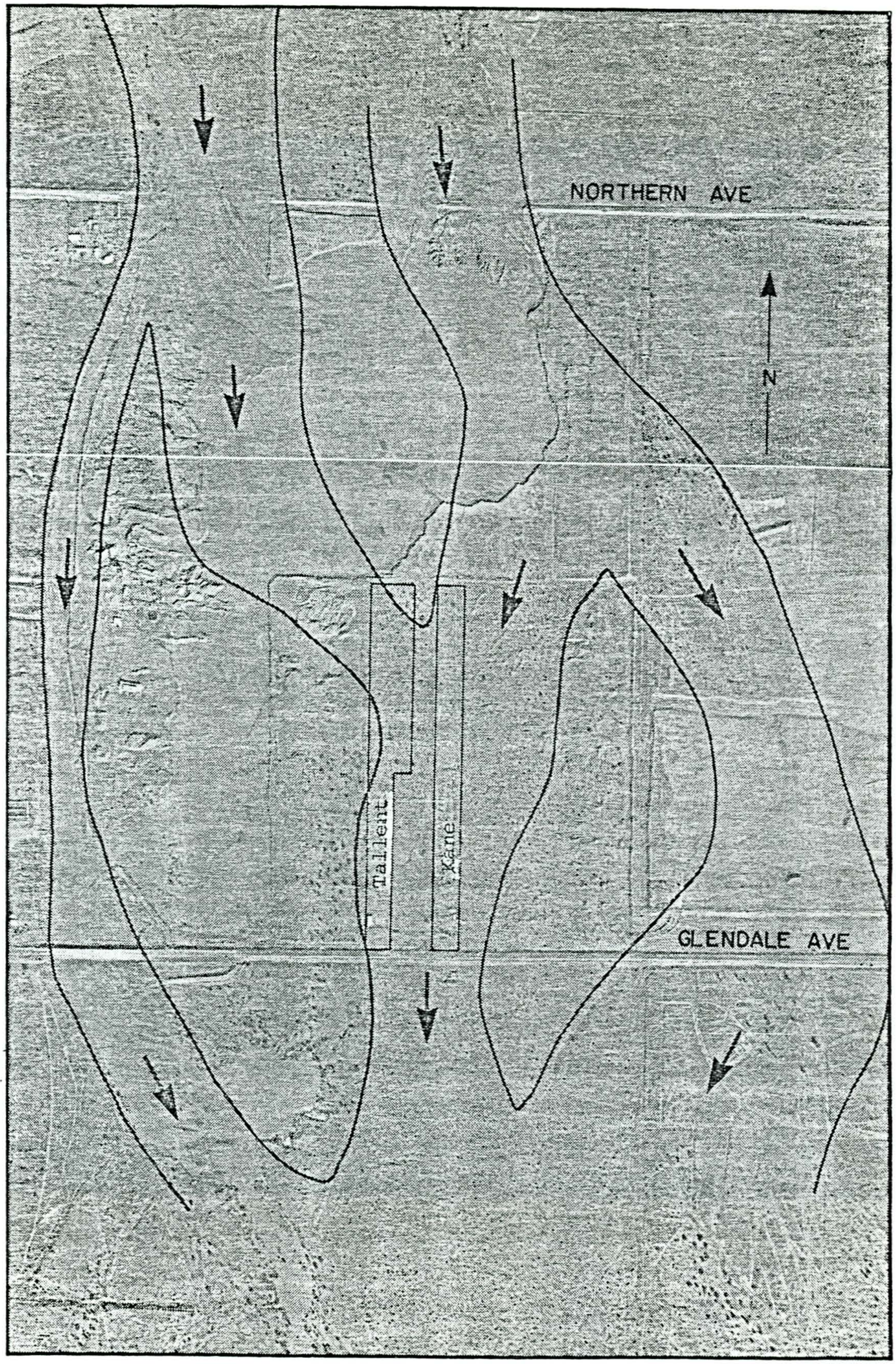


Figure 4.17. Aerial photo taken 12/20/78 with estimated flow patterns for pre-bridge, pre-mining condition.

6. Existing developments upstream of the study reach include Waddell Dam (Agua Fria), Dreamy Draw Dam (Dream Draw Wash), Cave Creek Dam (Cave Creek), Adobe Dam (Skunk Creek), McMicken Dam (Trilby Wash), and several collector canals. Proposed developments upstream and along the study reach include new Waddell Dam (Agua Fria), New River Dam (New River), and the Arizona Canal Diversion Channel (ACDC).

   Of the upstream developments, Waddell Dam has had the biggest impact on river morphology by trapping upstream sediments, causing downstream degradation resulting in downstream armoring of the river bed. A new proposed Waddell Dam will have the greatest impact on the future developments because of the increased flood control and reduction of downstream peak discharges. The reduction in flood peaks will also reduce downstream sediment transportation capacities and slow the degradation response of the bed. ACDC will increase sediment and water inflows into the study reach, however the New River Dam will more than offset the increased flows and sediment discharges into the Agua Fria River.

7. Projected growths of cities along the Agua Fria will increase the demand for land along the flood plain, thus increasing the need for a flood control project. Urbanization has caused encroachment of the flood plain at Bell Road, Grand Avenue, Glendale Avenue, Indian School Road and Buckeye Road bridge crossings.

8. Sand and gravel mining has been the most active industry in the area. The two largest gravel mining operations along the river include the United Metro operation near Glendale Avenue and the Allied Concrete and Phoenix Sand and Rock operation downstream of Indian School Road Bridge. Gravel mining has significantly altered the shape of the Agua Fria and is one of the causes of degradation throughout the study reach.

V. ENGINEERING 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. Specifically, the analysis consists of (1) identifying applicable sediment transport equations describing sediment movement in the channel bed, (2) determining the short-term and long-term response of the bed using an equilibrium slope analysis, (3) assess the armoring potential of the bed to control the long-term response, and (4) assess the present bridge crossing's adequacy for passing the 100-year discharge and withstanding the local scour around bridge piers and abutments.

5.2 Sediment Transport Analysis

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 and 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 should be 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, the 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:

$$Q_s = 8.61 \times 10^{-5} V^{3.7} H_Y^{0.32} T_w \quad (5.1)$$

where Q_s is the sediment-transport capacity (cfs)
V is the average flow velocity (ft/sec)
H_Y is the hydraulic depth (ft)
T_w is the top width.

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:

$$G = \frac{1}{2}\left(\frac{D_{84}}{D_{50}} + \frac{D_{50}}{D_{16}}\right) \quad (5.2)$$

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

Using Equation 5.1, in combination with the average hydraulics of the subreaches summarized in Table 3.2, the sediment transport capacity for the 10-, 25-, 50- and 100-year flood peaks is computed for main channel and overbanks. Tables 5.1 to 5.4 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 Buckeye Road (reaches 3 through 7) is significantly higher than the other subreaches of the river. The effective width of the main channel is narrower in reaches 3 to 7, resulting in larger velocities and higher sediment transporting capacity.

5.3 Short-Term Channel Response

The short-term response of the channel is assessed by comparing the sediment transport capacity of the subreach directly upstream of the subreach being considered. If the upstream subreach has a larger sediment transport capacity, then the downstream subreach will aggrade, and if the upstream subreach has a smaller sediment transport capacity, the downstream subreach will degrade.

Table 5.1. Sediment Transport Capacity for the 10-Year Flood Peak.

REACH	LEFT FLOODPLAIN (CFS)	MAIN CHANNEL (CFS)	RIGHT FLOODPLAIN (CFS)	TOTAL (CFS)
1	0.	105.	0.	105.
2	0.	112.	0.	112.
3	0.	187.	0.	187.
4	0.	122.	0.	122.
5	0.	157.	1.	158.
6	0.	131.	0.	131.
7	0.	168.	0.	168.
8	0.	81.	0.	81.
9	0.	80.	0.	80.
10	0.	106.	0.	106.

Table 5.2. Sediment Transport Capacity for the 25-Year Flood Peak.

REACH	LEFT FLOODPLAIN (CFS)	MAIN CHANNEL (CFS)	RIGHT FLOODPLAIN (CFS)	TOTAL (CFS)
1	0.	167.	2.	169.
2	0.	145.	0.	145.
3	0.	357.	0.	357.
4	0.	265.	0.	265.
5	1.	277.	5.	283.
6	1.	249.	0.	250.
7	0.	349.	0.	349.
8	1.	167.	1.	169.
9	0.	166.	0.	166.
10	0.	227.	0.	227.

Table 5.3. Sediment Transport Capacity for the 50-Year Flood Peak.

REACH	LEFT FLOODPLAIN (CFS)	MAIN CHANNEL (CFS)	RIGHT FLOODPLAIN (CFS)	TOTAL (CFS)
1	1.	249.	3.	253.
2	1.	189.	1.	190.
3	0.	444.	0.	444.
4	1.	415.	1.	417.
5	2.	410.	11.	423.
6	5.	373.	0.	378.
7	0.	517.	0.	518.
8	2.	289.	3.	295.
9	0.	283.	0.	283.
10	0.	357.	0.	357.

Table 5.4. Sediment Transport Capacity for the 100-Year Flood Peak.

REACH	LEFT FLOODPLAIN (CFS)	MAIN CHANNEL (CFS)	RIGHT FLOODPLAIN (CFS)	TOTAL (CFS)
1	2.	377.	9.	388.
2	2.	250.	2.	254.
3	0.	439.	0.	439.
4	3.	535.	3.	541.
5	4.	554.	23.	581.
6	13.	525.	0.	538.
7	1.	675.	0.	676.
8	5.	454.	6.	465.
9	0.	446.	0.	446.
10	10.	480.	0.	490.

The analysis for the short-term response of the channel is based on the following assumptions:

- The hydraulics are based on a rigid or fixed bed geometry and do not adjust as the aggradation/degradation proceeds.
- The river bed material characteristics do not change throughout the analysis. Armoring is not considered.
- The degradation/aggradation response will be through bed slope changes.

Thus the short-term response sometimes does not agree with the mathematical model simulation which considers the erosion and sedimentation processes on a higher level of complexity.

Table 5.5 summarizes the short-term response to the 10-year and 100-year flood peaks. Reach 3, between ISRB and the RID flume, has the greatest potential degradation response in the study reach.

5.4 Long-Term Channel Response

The long-term channel response was assessed using the equilibrium slope method. Equilibrium 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 bankfull 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.

The subreach upstream of Camelback Road, Subreach 1, was used to determine the upstream sediment supply into downstream subreaches.

Table 5.6 summarizes for each subreach the existing slope, the sediment transporting capacity, and the equilibrium slope to which the subreach will adjust. Most of the subreaches show a tendency to degrade slightly, except below Buckeye Road, where a slight aggradation response is shown.

Table 5.5. Short-Term Aggradation/Degradation Response.

Reach	10-Year			100-Year		
	Q_s (cfs)	ΔQ_s (cfs)	Response	Q_s (cfs)	ΔQ_s (cfs)	Response
1	105			379		
2	112	-7	Equilibrium	254	+125	Aggradation
3	187	-75	Degradation	446	-192	Degradation
4	122	+65	Aggradation	541	-95	Degradation
5	158	-36	Degradation	581	-40	Degradation
6	131	+27	Aggradation	538	+43	Aggradation
7	168	-37	Degradation	676	-138	Degradation
8	81	+87	Aggradation	465	+211	Aggradation
9	80	+1	Equilibrium	446	+19	Near Equilibrium
10	106	-26	Degradation	492	-46	Degradation

Notes: (1) Q_s is sediment transport rate, ΔQ_s is degradation (-)/aggradation (+) rates of the flood peak.

(2) Reach locations:

1 Upstream of Camelback
2 Camelback to ISRB
3 ISRB to RID
4 RID to Thomas Road
5 Thomas Road to 3000 ft upstream of I-10
6 3000 ft upstream of I-10 to Van Buren
7 Van Buren to Buckeye
8 Buckeye to Lower Buckeye
9 Lower Buckeye to Broadway
10 Broadway to 4000 ft upstream of Gila River

Table 5.6. Agua Fria River Equilibrium Slope - As-Is Condition.

Reach	S_{ex}	10-Year Flood	
		Q_s	S_{eq}
1		105	
2	0.0033	112	0.0031
3	0.0024	187	0.0014
4	0.0023	122	0.0020
5	0.0021	158	0.0014
6	0.0023	131	0.0018
7	0.0016	168	0.0010
8	0.0023	81	0.0030
9	0.0025	80	0.0032
10	0.0025	106	0.0025

S_{ex} = existing channel slope (average river bed, ft/ft)

Q_s = bed material transport (cfs)

S_{eq} = equilibrium slope (ft/ft)

Figure 5.1 compares the existing slope for subreaches 2 through 7 with the long-term equilibrium slope. The equilibrium slopes were pivoted about the downstream end of each subreach to determine the potential aggradation/ degradation in each subreach. Instead of a stepwise equilibrium profile, the actual aggradation/degradation process will be more of a smooth transition between reaches as conceptually shown in Figure 5.1.

5.5 Armor Control Analysis

The equilibrium slopes shown in Table 5.6 were computed based on the existing supply from Subreach 1 (Camelback Road to Glendale Avenue). This sediment supply, however, may be reduced due to river armoring.

Table 5.7 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 greater than 7 and 9 fps, respectively. Previous hydraulic analysis showed the main channel velocities ranged from 4 to 6 fps for 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, critical velocity for incipient motion and the availability of these particles (see sediment distributions in Appendix B).

Although armoring of the entire subreach is not likely to occur, sediment supply can be reduced due to bed material coarsening or partial armoring. To account for the possible future sediment supply reduction, the equilibrium slopes were reevaluated assuming a 25 percent reduction in upstream sediment supply. The resultant equilibrium slopes are shown in Table 5.8. The degradation problems become more prominent under the reduced supply condition.

5.6 Bridge Analysis

5.6.1 General

Several bridge crossings exist in the study reach including ISRB, RID flume, I-10, Southern Pacific Railroad (SPRR), and Buckeye Road. Bridge data for these crossings were presented earlier in Section 3.3. This section summarizes the adequacy of the bridge crossings to pass the 100-year flood and

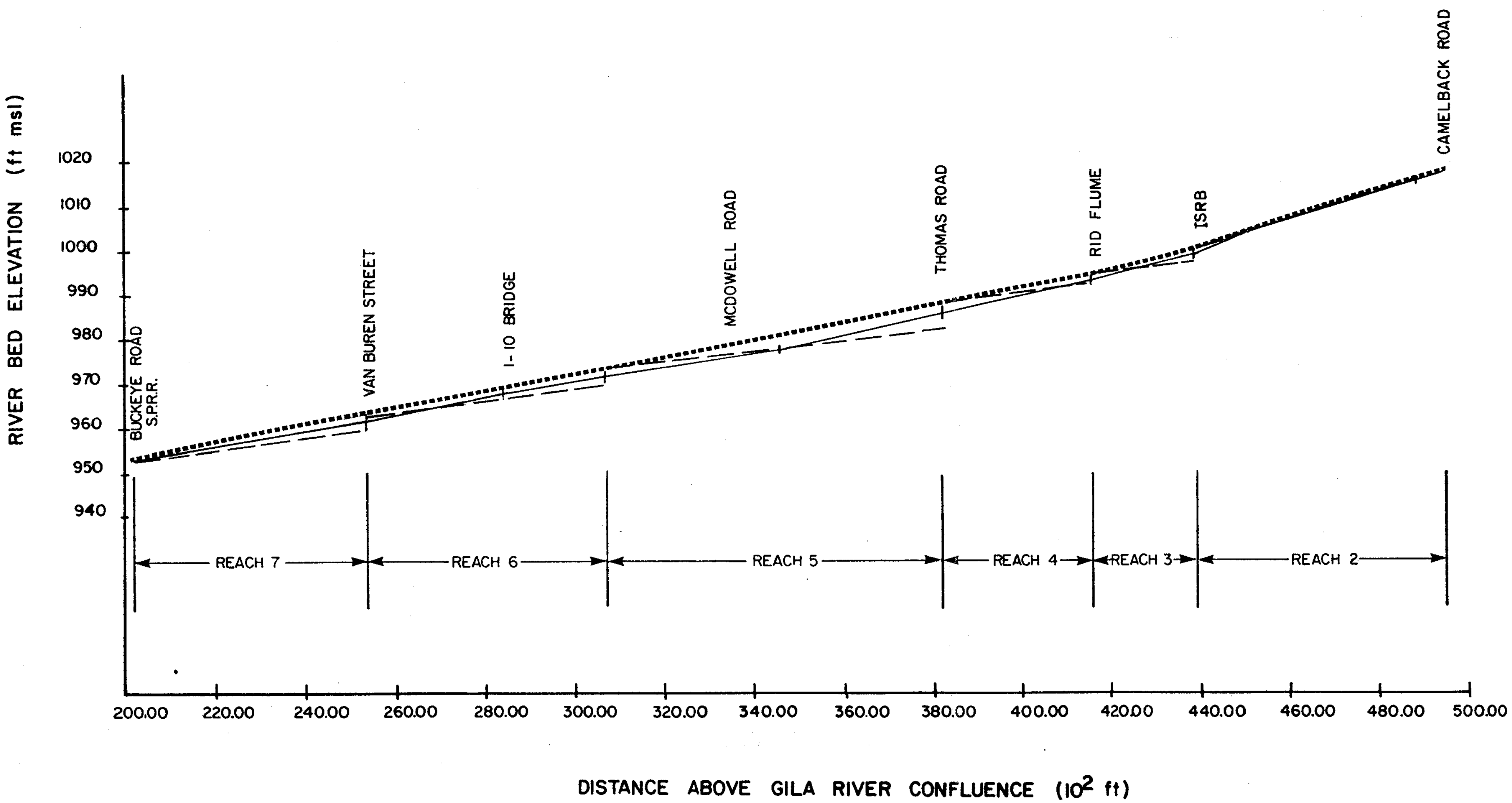


Figure 5.1

Comparison of existing and equilibrium slope profiles between Buckeye Road and Camelback Road

Table 5.7. Critical Velocities for Incipient Motion of Sediment Particles.

Sediment Type	Size (mm)	Critical Velocity (fps)
Very fine gravel	2 ~ 4	2.3
Fine gravel	4 ~ 8	3.3
Medium gravel	8 ~ 16	4.6
Coarse gravel	16 ~ 32	6.5
Very coarse gravel	32 ~ 64	9.2

Table 5.8. Agua Fria River Equilibrium Slope with 25% Reduction of Sediment Supply.

Reach	S_{ex}	10-Year Flood Q_s	S_{eq}
1		79	
2	0.0033	112	0.0023
3	0.0024	204	0.0009
4	0.0023	123	0.0015
5	0.0021	162	0.0010
6	0.0023	133	0.0014
7	0.0016	170	0.0008
8	0.0023	83	0.0022
9	0.0025	82	0.0024
10	0.0025	108	0.0018

S_{ex} = existing channel slope (average river bed, ft/ft)

Q_s = bed material transport (cfs)

S_{eq} = equilibrium slope (ft/ft)

evaluates the local scour around bridge piers and abutments for existing channel conditions.

5.6.2 Bridge Capacity

The 100-year water-surface elevation at the bridge crossings in the study reach were determined considering aggradation (if any), wave height, superelevation around bends and potential blockage due to debris. Table 5.9 summarizes the low-chord elevations, the 100-year water-surface elevation considering 0, 10 and 20 percent debris blockage, the sum of half the antidune height, half the surface wave height, superelevation and aggradation, and the available freeboard.

The low chord of ISRB is not high enough to pass the 100-year peak discharge. The constriction downstream of the bridge has significantly reduced the effective flow capacity of the bridge. To pass the 100-year flood, the channel downstream of the bridge will have to be widened or the bed lowered.

The low chord of the RID flume will be approximately at the 100-year water-surface elevation when wave heights and antidune heights are considered. No aggradation of the bed or superelevation occurs in this area. As with ISRB the channel will have to be widened or the bed lowered for the flume to be able to pass the 100-year flood peak.

I-10, SPRR and Buckeye Road have freeboard heights of 6.7, 0.6 and 3.4 feet, respectively. The freeboard heights consider 20 percent debris blockage of the bridge, and therefore the bridges possess adequate capacity to pass the 100-year flood peak.

5.6.3 Local Scour Around Bridge Piers

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 will provide reasonable approximations for local scour depths. Shen's equation takes the following form:

$$d_s = k\ 0.00073\ R^{0.619} \qquad (5.3)$$

where d_s is the local scour depth

Table 5.9. Comparison of 100-Year Water-Surface Elevation and Low-Chord Elevation of Bridges in Study Reach.

Bridge Location	Low Chord (ft, MSL)	100-Year Flood Water Surface (ft MSL) Debris Blockage None	10%	20%	Sum of Antidune Height, Wave Height, Aggradation & Super-elevation (ft)	Available Freeboard With 20% Debris Blockage & Sum of Waves & Aggradation (ft)
ISRB	1015.4	1015.8	1016.0	1016.4	0.8	---
RID	1010.0	1007.9	1009.0	1012.0	2.9	---
I-10	988.5	979.4	979.5	979.7	2.0	6.8
SPRR	966.2	962.2	963.1	964.1	1.5	0.6
Buckeye Road	968.1	961.5	962.0	963.0	1.8	3.3

k is a multiplying factor to account for skewness of piers (see Table 5.10)

R is the pier Reynolds number

$$R = \frac{Va}{\nu} \tag{5.4}$$

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:

$$d_s = d_w \,(2)\, \left(\frac{a}{d_w}\right)^{0.65} F_r^{0.43}\, k_1 \tag{5.5}$$

what about shape of the pier

where k is a multiplying factor to account for skewness of piers (see Table 5.10)

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 5.11. Velocity, depth and flow skewness used for local scour computations are included in the table. To account for debris accumulation near bridge piers, the actual pier width was expanded by adding two feet to each side of the pier, or equivalently a total of four feet to each bridge pier.

The largest local scour depth occurred at ISRB because of the pier length across the bridge and the severe angle of attack of flow on the bridge piers. Since the existing thalweg elevation is only 17 feet above the bottom of bridge footing, and the local scour is estimated to be more than 30 feet, the present ISRB piers will not withstand a 100-year flood.

The RID flume and SPRR both have concrete footings buried approximately 3 feet below the present thalweg. Below the footings concrete piles are driven another 20 feet. Thus local scour depths for the 100-year flood peak will be significantly below the footings, and approaching the depth of the piles, therefore some form of pier protection will be necessary for these structures to withstand the 100-year flood for present conditions.

Table 5.10. Multiplying Factors* for Depth of Scour d_s for Skewed Piers.

Horizontal Angle of Attack	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 F. Sentürk, Sediment Transport Technology, Water Resource Publications, 1977.

Table 5.11. Approximate Local Scour Depths Around Bridge Piers for the 100-Year Peak Discharge.

Bridge Location	Flow Velocity (fps)	Flow Depth (ft)	Flow Skewness (°)	Local Scour at Bridge Pier* Shen (ft)	Local Scour at Bridge Pier* Neil (ft)	Distance Above Footing to Thalweg (ft)
ISRB	6.21	15.7	30	20.8	32.1	17
RID	11.59	13.7	--	12.1	16.5	3[1]
I-10	9.23	12.5	5	10.0	13.9	22.3
SPRR	8.33	9.4	10	21.1	29.2	5[1]
Buckeye Road	9.20	8.9	10	16.5	21.9	25

*Assume 2 feet debris blockage on each side of pier.

[1]Have concrete piles driven approximately 20 feet below the concrete footings.

Both I-10 and Buckeye Road bridge piers are buried deeper than the 100-year local scour depths. However, should the thalweg drop significantly near these bridge crossings, the bridge piers could be in jeopardy of being undermined.

5.6.4 Local Scour Around Bridge Abutments

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

$$S = d_1 \, (1.1) \left(\frac{a}{d_1}\right)^{0.4} F_{r_1}^{0.33} \qquad (5.6)$$

where S is the abutment scour depth

d_1 is the upstream depth

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

F_{r_1} is the Froude number

The results of computations are shown in Table 5.12. The abutment scour ranges from 9 to 13 feet.

5.7 Analysis of Utility Crossings

5.7.1 General

The following agencies were contacted in regards to utility crossings in the Agua Fria from Camelback Road to Buckeye Road:

1. Tucson Electric Power Company
2. Salt River Project
3. El Paso Gas Company
4. Arizona Public Service
5. Mountain Bell
6. Roosevelt Irrigation District
7. Southern Pacific Pipeline Incorporated
8. City of Avondale
9. City of Phoenix
10. Town of Goodyear
11. Department of Energy

Table 5.12. Approximate Local Scour Depths Around Bridge Abutments for the 100-Year Peak Discharge.

	ISRB		I-10		SPRR		Buckeye Road	
	East	West	East	West	East	West	East	West
Abutment Length (ft)	12	16	15	22	16	13	23	22
Flow depth (ft)	15.7		12.5		9.4		8.9	
Velocity (fps)	6.2		9.2		8.3		9.2	
Froude Number	0.28		0.46		0.48		0.54	
Local Scour (ft)	10.1	11.4	11.4	13.3	10.0	9.2	11.7	11.5

Of these agencies, the following did not have utilities in or near the river between Camelback and Buckeye Roads: Arizona Public Service, Mountain Bell, City of Phoenix and the Town of Goodyear.

Tucson Electric Power Company has a 345 kV transmission line, the location of which is shown in the topographic maps attached with this report. The Salt River Project has a 230 kV transmission line that is shown in the topographic maps attached with this report. The Department of Energy has an abandoned 161 kV transmission line that crosses the Agua Fria River just downstream of Thomas Road.

El Paso Gas Company has two pipelines near the river including a 10-inch pipeline crossing near Buckeye Road, approximately 150 feet upstream of the Southern Pacific Railroad, and a 20-inch pipeline on the east flood plain of the Agua Fria between Camelback and ISRB running north and south. The Southern Pacific Pipeline Incorporated has two pipelines near the Agua Fria including a 12-inch pipeline just south of Buckeye Road and a 6-inch pipeline parallel and adjacent to Thomas Road. The City of Avondale has a 16-inch water line parallel and adjacent to Thomas Road. Locations of these pipelines are shown in the topographic maps attached to this report.

The Roosevelt Irrigation District has a flume crossing in the river approximately 2,200 feet downstream of ISRB. The flume has previously been discussed in Section 3.3.

5.7.2 Local Scour Around Transmission Towers

Local scour computations were made for the Tucson Electric Power Company and Salt River Project transmission towers located within the 100-year flood plain using Shen's and Neil's equations. Tucson Electric Power Company has 40 transmission towers within the 100-year flood plain. The towers vary from 80 to 105 feet above the existing ground and have 5-foot diameter pier footings. Several of the towers have been reinforced with sheet pile. The obstruction width for the reinforced tower legs is 10 feet. Table 5.13 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 and Neil equations, the adopted local scour, the approximate ground elevation near the tower and the expected elevation after local scour.

Table 5.13. Local Scour Around Tucson Electric Power Company Transmission Towers for the 100-year Flood.

Tower No.	Obstruction Width (ft)	Velocity (ft/sec)	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
73	5	2.1	0.6	4.0	4.5	4.3	1,034.3	1,030.0
74	5	7.8	10.7	9.1	11.8	10.4	1,023.1	1,012.7
75	5	7.9	8.6	9.2	11.5	10.3	1,022.6	1,012.3
76	5	2.1	3.1	4.1	5.7	4.9	1,026.5	1,021.6
77	5	2.8	4.9	4.8	6.8	5.8	1,022.0	1,016.2
78	5	2.3	2.0	4.3	5.6	4.9	1,021.0	1,016.1
79	5	0.9	0.7	2.4	3.3	2.8	1,020.0	1,017.2
81	5	1.0	5.5	2.5	4.4	3.4	1,011.0	1,007.6
82	5	0.4	6.7	1.5	3.1	2.3	1,009.0	1,006.7
83	5	1.5	6.4	3.3	5.4	4.4	1,006.5	1,002.1
84	5	1.7	3.2	3.6	5.2	4.4	1,004.0	999.6
85 (rein)	10	1.2	2.3	4.4	6.7	5.5	1,002.0	996.5
86	5	2.3	3.0	4.3	5.9	5.1	999.5	994.4
87	5	2.8	3.8	4.8	6.6	5.7	997.0	991.3
THOMAS 88 (rein)	10	2.9	3.0	7.5	10.1	8.8	996.0	987.2
89 (rein)	10	10.1	13.0	16.4	21.2	18.8	983.4	964.6
McDowell 90 - Levee	5	3.2	3.6	5.2	6.9	6.1	990.5	984.4
91 (rein)	5	10.4	11.4	16.7	21.0	18.9	980.6	961.7
92	5	11.3	11.0	11.5	13.9	12.7	978.7	966.0
93	5	8.0	9.2	9.3	11.7	10.5	977.9	967.4
94 In channel	5	6.9	8.9	8.4	10.9	9.7	977.0	967.3
95	5	6.4	11.9	8.0	11.0	9.5	972.7	963.2
I-10 96 (rein)	10	8.5	12.9	14.8	19.7	17.2	970.0	952.8
97	5	9.3	12.7	10.2	13.0	11.6	966.5	954.9
98	5	8.5	13.6	9.6	12.6	11.1	962.8	951.7
99 (rein)	10	9.2	13.2	15.6	20.5	18.0	960.5	942.5
100 (rein)	10	7.0	9.7	13.1	17.4	15.3	962.0	946.7

Table 5.13. Local Scour Around Tucson Electric Power Company Transmission Towers for the 100-year Flood. (continued)

Tower No.	Obstruction Width (ft)	Velocity (ft/sec)	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
101 (rein)	10	6.6	11.5	12.6	17.4	15.0	959.0	944.0
102 (rein)	10	12.2	10.6	18.5	22.4	20.4	956.1	935.7
Buckeye Rd. 103 (rein)	10	10.6	9.7	16.9	20.8	18.8	953.5	934.7
104	5	9.3	8.9	10.1	12.4	11.3	951.0	939.7
105	5	6.8	9.2	8.4	10.9	9.6	948.5	938.9
106	5	7.8	6.9	9.1	11.1	10.1	948.0	937.9
107	5	6.1	8.8	7.8	10.3	9.1	942.0	932.9
108	5	7.0	10.2	8.6	11.2	9.9	937.4	927.5
109	5	6.3	9.9	8.0	10.7	9.3	934.0	924.7
110	5	7.2	8.9	8.7	11.1	9.9	931.3	921.4
111	5	5.4	10.5	7.3	10.1	8.7	926.3	917.6
112	5	6.2	14.9	7.9	11.1	9.5	920.0	910.5
113	5	8.1	12.4	9.4	12.2	10.8	920.1	909.3

rein = reinforced foundations

The Salt River Project has 26 transmission towers located within the 100-year flood plain. The footings of these towers are 3 feet in diameter. Table 5.14 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 and Neil 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.

5.7.3 Pipeline Crossings

Several pipelines cross or parallel the Agua Fria 100-year flood plain. From the predicted short- and long-term bed responses of the river, most of the crossings would appear to be safe.

El Paso Gas Company has two pipelines in the study reach. A 20-inch line running north and south on the east overbank between Camelback and Indian School Road would appear to be safe, while the 10-inch line crossing the Agua Fria 150 feet upstream of the Southern Pacific Railroad may have to be lowered. The 20-inch pipeline is on the overbank where velocities are low and degradation response is minimal. The 20-inch line would be in danger if the river migrated several hundred feet laterally to the east, however, past aerial photographs have not indicated this trend in the area. The 10-inch pipeline is located in a degradation reach. The amount of cover above the pipeline is not exactly known, so it is difficult to ascertain whether the pipeline needs to be lowered. The depth of burial should be field verified before judgment is made. However, the pipeline is located in the downstream portion of the reach where the degradation may not warrant lowering of the pipe.

The Southern Pacific Pipeline Incorporated has two pipeline crossings in the study reach, including a 6-inch high pressure line crossing at Thomas Road and a 12-inch pipeline just south of Buckeye Road. The 6-inch line is located in a reach that is in near equilibrium and therefore should be safe. The 12-inch pipeline is located in an aggradation reach and should be safe for existing conditions.

Table 5.14. Local Scour Around Salt River Project Transmission Towers for the 100-Year Flood.

Tower No.	Obstruction Width (ft)	Velocity (ft/sec)	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
42	3	2.1	0.6	2.9	3.2	3.1	1,034.3	1,031.2
43	3	7.8	10.7	6.6	8.4	7.5	1,023.1	1,015.6
44	3	7.9	8.6	6.7	8.2	7.5	1,022.6	1,015.1
45	3	2.1	3.1	3.0	4.1	3.5	1,026.5	1,023.0
46	3	2.8	4.9	3.5	4.9	4.2	1,022.0	1,017.8
47	3	3.3	2.8	3.9	4.9	4.4	1,022.0	1,017.6
48	3	0.9	0.7	1.8	2.3	2.1	1,020.0	1,017.9
49	3	1.4	2.0	2.3	3.3	2.8	1,016.5	1,013.7
51	3	1.1	3.6	1.9	3.1	2.5	1,013.0	1,010.5
52	3	0.7	6.0	1.5	2.7	2.1	1,010.0	1,007.9
53	3	0.3	6.2	1.0	2.0	1.5	1,009.0	1,007.5
54	3	1.5	6.4	2.4	3.9	3.2	1,006.5	1,003.3
55	3	1.7	3.2	2.6	3.7	3.2	1,004.0	1,000.8
56	3	1.2	2.3	2.1	3.1	2.6	1,002.0	999.4
57	3	2.4	3.1	3.2	4.3	3.8	1,000.0	996.2
58	3	2.5	3.5	3.3	4.4	3.9	998.0	994.1
THOMAS 59	3	2.8	1.9	3.6	4.3	3.9	998.0	994.1
60	3	8.5	13.9	7.0	9.1	8.0	984.2	976.2
61	3	11.8	12.0	8.6	10.2	9.4	982.5	973.1
McDowell 62	3	10.1	12.3	7.8	9.6	8.7	980.8	972.1
63	3	10.7	11.2	8.1	9.8	8.9	979.9	971.0
64	3	2.7	5.2	3.4	4.8	4.1	984.0	979.9
65	3	3.7	3.5	4.2	5.2	4.7	983.0	978.3
66	3	2.7	3.0	3.5	4.5	4.0	979.0	975.0
67	3	2.8	2.2	3.5	4.4	3.9	977.0	973.1
68	3	2.1	2.3	2.9	3.9	3.4	975.0	971.6

VI. SEDIMENT ROUTING ANALYSIS

6.1 General

The third level of analysis involves applying the detailed sediment routing model developed by Simons, Li & Associates, Inc. to the study reach of the Agua Fria River to assess the aggradation/degradation response of the system. The sediment routing model is called QUASED. In using the QUASED model, the main river is subdivided into a series of computational reaches. Each of the subreaches is selected as a portion of the main river where hydraulic and geomorphic characteristics are similar. For this study, each subreach had sediment discharge input from the upstream portion of the main river. Hydraulic conditions were calculated using the Army Corps of Engineers HEC-2 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. This 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 quasidynamic where the flow is assumed constant for a given time increment but varies from subreach to subreach. The flood event 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 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-2 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 for more 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 Verification

Prior to simulating the channel response to the 100-year flood, QUASED was executed for the December 1978, January 1979 and February 1980 floods. The 1973 cross sections were used to simulate the pre-flood conditions. Only small flows occurred in the Agua Fria between 1973 and 1978, therefore the 1973 cross sections were considered representative of the bed prior to the three floods. The simulated response to the three floods was then compared to the 1981 topographic map to verify the predictive capability of QUASED.

Table 6.1 compares the simulated and measured thalweg elevations at several cross sections between ISRB and McDowell Road. QUASED satisfactorily predicted the aggradation/degradation trends for the 1978, 1979 and 1980 floods, and will give reasonable sedimentation predictions for the 100-year flood.

Table 6.1. Model Verification Results - Simulation of Channel Change from 1973 to 1981.

Section Number 1973 data	1973 Thalweg Elevation (ft MSL)	1981 Thalweg Elevation (ft MSL)	Difference Between 1981 & 1973 Thalweg Elevations (ft)	Simulated 1981 Thalweg Elevation (ft MSL)	Difference Between Simulated 1981 & 1973 Thalwegs (ft)
6.33*	978.0	976.8	-1.2	976.6	-1.4
6.71	981.0	978.0	-3.0	978.5	-2.5
6.96	983.0	980.8	-2.2	980.6	-2.4
7.12	986.0	982.2	-3.8	983.9	-2.1
7.36	990.5	985.6	-4.9	988.4	-2.1
7.65*	992.0	988.4	-3.6	989.0	-3.0
7.88	995.0	988.6	-6.4	990.1	-4.9
8.18*	995.5	994.0	-1.5	992.4	-3.1
8.19	995.0	994.2	-0.8	992.5	-2.5
8.41	994.5	997.0	+2.5	995.2	+0.7
8.50	996.0	998.0	+2.0	996.2	+0.2
8.60*	1000.0	999.8	-0.2	1000.2	+0.2

* Section 6.33 is McDowell Road
Section 7.65 is Thomas Road
Section 8.18 is RID flume
Section 8.60 is ISRB

6.4 Model Application for 100-Year Flood

After verification, QUASED simulated the aggradation/degradation response of the Agua Fria River for the 100-year flood. The hydrograph shown in Figure 2.3 was digitized into 17 time steps and the study reach was divided into 10 reaches corresponding to the subreaches defined in Figure 3.1.

The sediment routing results indicate that degradation occurred in the constricted reaches from ISRB to Buckeye Road (reaches 3 to 7), while deposition occurred immediately downstream of the Buckeye Road bridge (reach 8). Figure 6.1 shows the thalweg profile change resulting from the 100-year flood routing. Maximum degradation occurred near the RID flume.

The channel degradation/aggradation response to the 100-year flood is consistent with the predicted response from the engineering geomorphic analysis. Table 6.2 compares the sediment transport rates at the 100-year flood peak for the engineering analysis and model simulation. The sediment transport rates compare reasonably well.

QUASED has the limitation of not modeling the local scour at bridge crossings. Thus to determine total scour depths at bridge crossings the local scour depth needs to be added to the model prediction. The total scour depths at the five bridge crossings are summarized in Table 6.3.

CHANGE IN THALWEG ELEVATION FOR 100YR FLOOD EVENT - AGUA FRIA RIVER, AS-IS

Table 6.2. Comparison of Sediment Transport at the 100-Year Flood Peak.

Reach	Sediment Routing Q_s (cfs)	Sediment Routing ΔQ_s (cfs)	Engineering Analysis Q_s (cfs)	Engineering Analysis ΔQ_s (cfs)
1	369		379	
2	263	+106	254	+125
3	319	-56	446	-192
4	495	-176	541	-95
5	491	+4	581	-40
6	428	+63	538	+43
7	447	-19	676	-138
8	391	+56	465	+211
9	368	+23	446	+19
10	385	-17	492	-46

Table 6.3. Summary of Total Scour Depths Expected at Bridge Crossings in the Agua Fria River for the 100-Year Flood Peak.

Bridge Crossing	QUASED General Degradation Prediction (ft)	Local Scour Neil's Method (ft)	Expected Total Scour (ft)
Indian School Road	1.3	32.1	33.4
Roosevelt Irrigation District Flume	3.8	16.5	20.3
I-10	---	13.9	13.9
Southern Pacific Railroad	0.8	29.2	30.0
Buckeye Road	0.8	21.9	22.7

The City of Avondale has a 16-inch water line crossing the Agua Fria at Thomas Road. The line is adjacent and parallel to the 6-inch high pressure gas line of the Southern Pacific Railroad and should be safe for existing conditions.

VII. CONCLUSIONS AND RECOMMENDATIONS

The following are conclusions regarding the hydraulic and geomorphic analysis of the Agua Fria River.

1. The Agua Fria River is an ephemeral braided stream with a wide 100-year flood plain. The width of the 100-year flood plain varies from 3,000 to 8,000 feet in the study reach from the confluence with the Gila River to the confluence with the New River.

2. An armor layer of gravel and cobble-sized materials is evident on the river bed surface of the Agua Fria River from Waddell Dam to Bethany Home Road. The armor layer has formed largely due to upstream sediment supply being trapped in Waddell Dam, causing downstream degradation and subsequent armoring of the bed.

3. Future developments upstream of the study reach might include a new Waddell Dam, New River Dam and Arizona Canal Diversion Channel. Of these developments, the new Waddell Dam will have the greatest impact on reducing downstream flood peaks in the study reach and changing the morphology of the channel. The 100-year flood peak on the Agua Fria River at Camelback Road will reduce approximately in half if a new Waddell Dam is built.

4. The thalweg has lowered from 0.5 to 3 feet throughout the study reach from 1973 to 1981. Not only has the thalweg dropped, but the overall cross section has also lowered. Part of the reason for the bed drop stems from the extensive instream gravel mining operations that have taken place in the study reach. It should be noted that contour intervals on the 1972 map are 4 feet and on the 1981 map the contour intervals are 2 feet. The accuracy of the 1972 map is ± 2 feet and the 1981 map is ± .5 feet. Thus, the magnitude of the difference in thalwegs is masked by the ± 2.5 feet combined map tolerance.

5. The qualitative geomorphic, engineering geomorphic and computer modeling analysis showed the following bed response for reaches between the confluence with the New River and the confluence with the Gila River.

    a. Approximately equilibrium conditions between the confluence with the New River and ISRB.

    b. Slight degradation potential between ISRB and Buckeye Road.

    c. Aggradation potential between Buckeye Road and Broadway Road.

    d. Approximately equilibrium conditions between Broadway Road and the confluence with the Gila River.

    The largest degradation potential exists between ISRB and the RID flume, and the reach between Thomas Road and McDowell Road.

6. Due to the significant armoring of the river bed between Waddell Dam and Bethany Home Road, the upstream sediment supply into the study reach is limited to the bed and bank material available from Bethany Home Road to Camelback Road, and the sediment supply from the New River. This supply may be reduced if the bed material coarsens through sediment sorting by future floods. Under the reduced supply condition downstream degradation becomes severe.

7. For present channel conditions ISRB and the RID flume do not possess adequate capacity to pass the 100-year peak discharge.

8. The local scour depths for the 100-year flood peak around the bridge piers at ISRB exceed the present burial depth of pier foundations 1-12. Local scour depths at the RID flume, and the Southern Pacific Railroad crossings are approaching the present pier foundation burial depths. Some protection around these piers will be required to withstand the 100-year flood peak.

The following recommendations regarding flood control alternatives are suggested:

1. With future developments likely along the Agua Fria River, narrowing the 100-year flood plain can be achieved with channelization.

2. Considering the flow velocities in the main channel average between 7 and 10 feet per second for the 100-year flood peak, and the parent material consists of coarse sands and fine gravels, some sort of bank protection will be required on the levees.

3. Between ISRB and Buckeye Road the channel bed response was slight degradation. With channelization in this reach, the degradation response will be accelerated and some grade control structures will be required to prevent excessive degradation.

4. Some protection of bridge piers at ISRB, RID flume and the SPRR are necessary for these structures to withstand the 100-year flood peak.

5. If channelization proceeds between Camelback Road and Buckeye Road, instream gravel mining should be limited to removal of bars and islands. No pits in the main channel should be allowed to endanger levees, utility crossings or bridge crossings.

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.

4. Einstein, H. A., 1950. "The Bed Load Function for Sediment Transportation in Open Channel Flows," U.S. Department of Agriculture, Soil Conservation Service, T. B. No. 1026.

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.

<u>Topographic Maps</u>

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.

1957 (photo revised 1974) USGS quadrangle map of Tolleson, 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.

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.

3/83 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 Willdon 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 & Associates, Inc.

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

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

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

Hydrographs

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 print-out dated March 7, 1981.

APPENDIX A

HYDRAULICS OF AGUA FRIA AND NEW RIVER

Figures A.1 and A.2 are schematic diagrams of cross section locations used in the HEC-II data input for the Agua Fria and New Rivers. Summarized in the schematic diagram A.1 is the distance between cross sections and the accumulated distance from the confluence with the Gila River. Summarized in schematic Figure A.2 is the distance between cross sections and the accumulated distance from the confluence with the Agua Fria River.

In Chapter III the water-surface profiles for the 10- and 100-year peak discharges in the Agua Fria were illustrated in Figure 3.2. Table A.1 lists the water-surface elevations for the 25- and 50-year return events for the Agua Fria at all cross section locations within the study reach.

Table A.2 summarizes the average flow velocities for the main channel and the entire cross section for the 10- and 100-year flood peaks in the Agua Fria study reach. The average cross section velocity is considerably smaller than the main channel velocity due to the lower overbank velocities.

Table A.3 summarizes the bankfull top width for all of the cross sections in Agua Fria between Glendale Avenue and the confluence with the Gila River. Bankfull top widths range from 455 feet to 3,300 feet. These widths are considerably less than the 100-year flood plain shown in Figure 3.3.

X-SECTION NUMBER	DISTANCE (FT)	ACCUMULATED DIST. (FT)	REMARK
			GILA RIVER
	715		
7.15		715	
	655		
13.70		1370	
	630		
20.00		2000	
	690		
26.90		2690	SOUTHERN AVENUE
	830		
35.20		3520	
	930		
44.60		4450	
	900		
53.60		5350	
	830		
61.90		6180	
	840		
70.40		7020	
	470		
75.00		7490	
	760		BROADWAY ROAD
82.60		8250	
	1120		
93.80		9370	
	1010		
103.90		10380	
	1345		
117.35		11725	
	410		
121.45		12135	
	920		
130.65		13055	
	475		
135.40		13530	LOWER BUCKEYE ROAD
	1595		
151.35		15125	
	2000		
171.35		17125	
	1020		
181.55		18145	
	865		
190.20		19010	
	1000		
200.20		20010	
	75		
201.00		20085	BUCKEYE ROAD
	100		
202.00		20185	
	30		
202.30		20215	S.P.R.R.

Figure A.1. Schematic diagram of 1981 cross sections - Agua Fria River.

X-SECTION NUMBER	DISTANCE (FT)	ACCUMULATED DIST. (FT)	REMARK
202.30		20215	S.P.R.R.
	1035		
212.85		21250	
	1510		
227.95		22760	
	1215		
240.20		23975	
	1410		
254.30		25385	VAN BUREN STREET
	1250		
266.80		26635	
	835		
275.15		27470	
	775		
281.50		28245	
	1		
281.60		28246	
	180		
283.40		28426	I-10 BRIDGE
	1		
283.50		28427	
	1050		
294.00		29477	
	1200		
303.00		30677	
	800		
308.30		31477	
	700		
316.20		32177	
	1300		
323.20		33477	McDOWELL ROAD
	1100		
334.20		34577	
	1000		
344.20		35577	
	440		
348.60		36017	
	970		
358.30		36987	
	1220		
370.50		38207	THOMAS ROAD
	1080		
381.40		39287	
	410		
385.50		39697	
	660		
392.10		40357	
	590		
398.00		40947	
	570		
403.75		41517	

Figure A.1 (continued)

X-SECTION NUMBER	DISTANCE (FT)	ACCUMULATED DIST. (FT)	REMARK
403.75		41517	
	25		
403.90		41542	RID FLUME
	550		
409.45		42092	
	830		
417.75		42922	
	920		
426.95		43842	
	40		
427.35		43882	ISRB
	615		
433.50		44497	
	595		
439.45		45092	
	530		
444.75		45622	
	785		
452.60		46407	
	690		
459.50		47097	
	710		
466.60		47807	
	670		
473.30		48477	
	360		
476.90		48837	
	640		
483.30		49477	CAMELBACK ROAD
	760		
490.90		50237	
	580		
496.70		50817	
	475		
501.45		51292	NEW RIVER
	885		
510.30		52177	
	990		
520.20		53167	
	1100		
525.20		54267	
	1350		
544.70		55617	
	1390		
558.60		57007	
	1010		
568.70		58017	
	1145		
580.15		59162	
	910		
589.25		60072	

Figure A.1 (continued)

X-SECTION NUMBER	DISTANCE (FT)	ACCUMULATED DIST. (FT)	REMARK
	200		AGUA FRIA RIVER
0.20		200	
	300		
0.50		500	
	500		
1.00		1000	
	500		
1.50		1500	
	500		
2.00		2000	
	500		
2.50		2500	
	500		
3.00		3000	
	500		
3.50		3500	
	500		
4.00		4000	
	500		
4.50		4500	
	500		
5.00		5000	
	500		
5.50		5500	

Figure A.2. Schematic diagram of 1981 cross section - New River.

Table A.1. Water Surface Elevation for 25- and 50-Year Flood Peak Discharge - Agua Fria River.

X-SECTION NUMBER	WATER SURFACE ELEVATION (FT) 25-YEAR	WATER SURFACE ELEVATION (FT) 50-YEAR	REMARK
			GILA RIVER
7.15	916.00	917.50	
13.70	917.49	918.72	
20.00	919.96	920.85	
26.90	921.35	922.33	SOUTHERN AVENUE
35.20	922.78	923.90	
44.60	924.19	925.20	
53.60	926.40	927.26	
61.90	928.58	929.51	
70.40	931.05	932.18	
75.00	932.26	933.18	BROADWAY ROAD
82.60	933.37	934.21	
93.80	935.16	936.00	
103.90	937.15	938.12	
117.35	941.29	942.22	
121.45	942.26	943.13	
130.65	944.57	945.38	
135.40	945.85	946.72	LOWER BUCKEYE ROAD
151.35	949.10	949.92	
171.35	953.36	954.12	
181.55	955.69	956.54	
190.20	956.99	957.88	
200.20	959.05	960.13	
201.00	959.16	960.34	BUCKEYE ROAD
202.00	959.38	960.61	
202.30	959.59	960.95	S.P.R.R.
212.85	961.94	963.37	
227.95	967.69	969.10	
240.20	969.48	970.64	
254.30	971.90	972.87	VAN BUREN STREET
266.80	974.53	975.51	
275.15	975.87	976.77	
281.50	977.17	978.25	
281.60	977.38	978.43	
283.40	977.78	978.81	I-10 BRIDGE
283.50	977.73	978.68	
294.00	980.95	981.96	
303.00	982.81	983.76	
308.30	983.71	984.58	

Table A.1 (continued)

X-SECTION NUMBER	WATER SURFACE ELEVATION (FT) 25-YEAR	WATER SURFACE ELEVATION (FT) 50-YEAR	REMARK
316.20	984.98	985.73	
323.20	987.11	987.80	McDOWELL ROAD
334.20	990.43	991.25	
344.20	992.47	993.31	
348.60	992.93	993.71	
358.30	995.77	997.08	
370.50	998.77	999.79	THOMAS ROAD
381.40	1000.48	1001.41	
385.50	1000.84	1001.77	
392.10	1001.46	1002.60	
398.00	1003.66	1004.17	
403.75	1004.20	1005.00	
403.90	1004.26	1006.34	RID FLUME
409.45	1006.29	1007.59	
417.75	1010.86	1013.73	
426.95	1011.94	1014.35	
427.35	1011.96	1014.37	ISRB
433.50	1012.45	1014.95	
439.45	1013.02	1015.23	
444.75	1013.78	1015.54	
452.60	1015.85	1016.72	
459.50	1017.41	1018.22	
466.60	1019.23	1019.94	
473.30	1021.30	1021.96	
476.90	1022.14	1022.76	
483.30	1023.49	1024.13	CAMELBACK ROAD
490.90	1025.12	1025.74	
496.70	1025.99	1026.58	
501.45	1026.96	1027.64	NEW RIVER
510.30	1029.80	1030.21	
520.20	1032.32	1033.05	
525.20	1034.53	1035.21	
544.70	1037.04	1037.71	
558.60	1040.76	1041.32	
568.70	1044.07	1044.67	
580.15	1046.08	1046.85	
589.25	1047.59	1048.57	

Table A.2. Flow Velocity for 10- and 100-Year Flood Peak Discharge - Agua Fria River.

X-SECTION NUMBER	FLOW VELOCITY (FPS) 10-YEAR CHANNEL	10-YEAR AVERAGE	100-YEAR CHANNEL	100-YEAR AVERAGE	REMARK
					GILA RIVER
7.15	3.47	3.47	7.08	6.53	
13.70	7.48	7.48	8.93	8.23	
20.00	4.78	4.78	7.51	6.33	
26.90	4.57	4.57	7.93	7.25	SOUTHERN AVENUE
35.20	4.39	4.39	5.85	5.05	
44.60	5.42	5.40	7.95	6.32	
53.60	5.75	5.75	9.74	8.25	
61.90	6.23	6.23	7.75	5.81	
70.40	5.54	5.54	8.12	6.87	
75.00	4.33	4.33	5.51	4.46	BROADWAY ROAD
82.60	4.04	3.87	6.16	5.37	
93.80	3.32	3.32	5.44	5.44	
103.90	5.86	5.86	7.66	7.66	
117.35	4.81	4.69	6.25	6.07	
121.45	4.91	4.82	6.34	6.10	
130.65	4.27	4.27	7.27	7.27	
135.40	5.04	4.85	7.04	6.43	LOWER BUCKEYE ROAD
151.35	3.91	3.29	6.11	4.78	
171.35	5.78	5.36	7.79	7.04	
181.55	3.83	3.46	6.05	4.80	
190.20	5.95	4.83	8.93	6.40	
200.20	5.11	5.11	9.59	9.59	
201.00	5.14	5.14	9.20	9.20	BUCKEYE ROAD
202.00	5.05	5.05	8.85	8.85	
202.30	5.01	5.01	8.33	8.33	S.P.R.R.
212.85	10.66	10.66	15.02	15.02	
227.95	5.61	5.61	6.61	4.07	
240.20	5.46	5.46	7.04	4.38	
254.30	6.68	6.34	9.24	6.56	VAN BUREN STREET
266.80	6.93	6.64	8.50	4.76	
275.15	4.94	4.86	8.13	5.71	
281.50	7.73	7.73	9.33	7.68	
281.60	7.77	7.77	8.44	6.72	
283.40	7.50	7.50	9.23	7.19	I-10 BRIDGE
283.50	8.33	8.33	10.34	8.84	
294.00	5.83	4.93	8.52	5.48	
303.00	4.37	3.75	6.37	4.05	
308.30	6.39	4.76	7.02	4.44	

Table A.2 (continued)

X-SECTION NUMBER	FLOW VELOCITY (FPS) 10-YEAR CHANNEL	10-YEAR AVERAGE	100-YEAR CHANNEL	100-YEAR AVERAGE	REMARK
316.20	5.06	3.78	6.73	4.31	
323.20	7.83	5.31	11.82	6.09	McDOWELL ROAD
334.20	8.94	5.68	10.36	5.01	
344.20	6.53	4.51	9.89	5.29	
348.60	6.47	6.13	11.76	7.24	
358.30	7.52	7.26	8.46	6.42	
370.50	6.81	6.23	9.05	5.46	THOMAS ROAD
381.40	4.66	4.46	6.59	3.99	
385.50	4.93	4.93	6.92	4.65	
392.10	8.16	8.16	10.42	5.25	
398.00	6.66	4.86	10.99	6.25	
403.75	8.21	8.21	12.75	6.42	
403.90	8.35	8.35	11.59	5.37	RID FLUME
409.45	13.27	13.27	12.54	5.47	
417.75	5.88	5.88	7.78	3.62	
426.95	4.27	4.27	6.50	5.55	
427.35	4.26	4.26	6.21	4.99	ISRR
433.50	5.47	5.47	4.01	2.53	
439.45	5.08	4.90	4.12	2.57	
444.75	5.87	5.39	5.37	3.37	
452.60	4.37	4.37	6.32	4.93	
459.50	4.55	4.55	6.43	5.22	
466.60	5.34	5.34	7.49	6.30	
473.30	4.59	4.52	5.84	4.19	
476.90	4.00	3.52	5.58	4.41	
483.30	6.61	6.32	6.58	4.47	CAMELBACK ROAD
490.90	4.20	4.20	4.59	3.27	
496.70	4.45	4.45	6.62	4.62	
501.45	5.23	5.23	8.33	5.12	NEW RIVER
510.30	6.14	4.56	7.89	4.24	
520.20	5.64	2.77	7.77	3.53	
525.20	4.59	2.52	6.48	3.47	
544.70	4.26	3.18	5.70	4.06	
558.60	5.85	4.32	8.56	5.33	
568.70	5.97	3.65	6.74	3.91	
580.15	5.35	5.15	8.27	5.88	
589.25	6.21	6.21	10.46	10.46	

Table A.3. Bankfull Width of 1981 Cross Sections.

X-SECTION NUMBER	BANKFULL WIDTH (FT)	REMARK
		GILA RIVER
7.15	1640	
13.70	1835	
20.00	1900	
26.90	1780	SOUTHERN AVENUE
35.20	1940	
44.60	1630	
53.60	1160	
61.90	1470	
70.40	2110	
75.00	2290	
		BROADWAY ROAD
82.60	2525	
93.80	3300	
103.90	3435	
117.35	2850	
121.45	3010	
130.65	2765	
135.40	2525	LOWER BUCKEYE ROAD
151.35	1890	
171.35	2430	
181.55	1960	
190.20	1030	
200.20	1188	
201.00	1188	BUCKEYE ROAD
202.00	1214	
202.30	1214	S.P.R.R.

Table A.3 (continued)

X-SECTION NUMBER	BANKFULL WIDTH (FT)	REMARK
212.85	920	
227.95	1050	
240.20	1030	
254.30	1100	VAN BUREN STREET
266.80	780	
275.15	1093	
281.50	1495	
281.60	1495	
283.40	1495	I-10 BRIDGE
283.50	1495	
294.00	1020	
303.00	1295	
308.30	1260	
316.20	1170	
323.20	580	McDOWELL ROAD
334.20	455	
344.20	535	
348.60	630	
358.30	1265	
370.50	890	THOMAS ROAD
381.40	1000	
385.50	1133	
392.10	808	
398.00	705	
403.75	550	

Table A.3 (continued)

X-SECTION NUMBER	BANKFULL WIDTH (FT)	REMARK
403.90	535	RID FLUME
409.45	455	
417.75	600	
426.95	1055	
427.35	1055	ISRR
433.50	1950	
439.45	2080	
444.75	2000	
452.60	2560	
459.50	2580	
466.60	2830	
473.30	2790	
476.90	3165	
483.30	1750	CAMELBACK ROAD
490.90	2550	
496.70	2250	
501.45	1570	NEW RIVER
510.30	1445	
520.20	880	
525.20	1350	
544.70	2710	
558.60	1870	
568.70	1490	
580.15	1270	
589.25	1310	

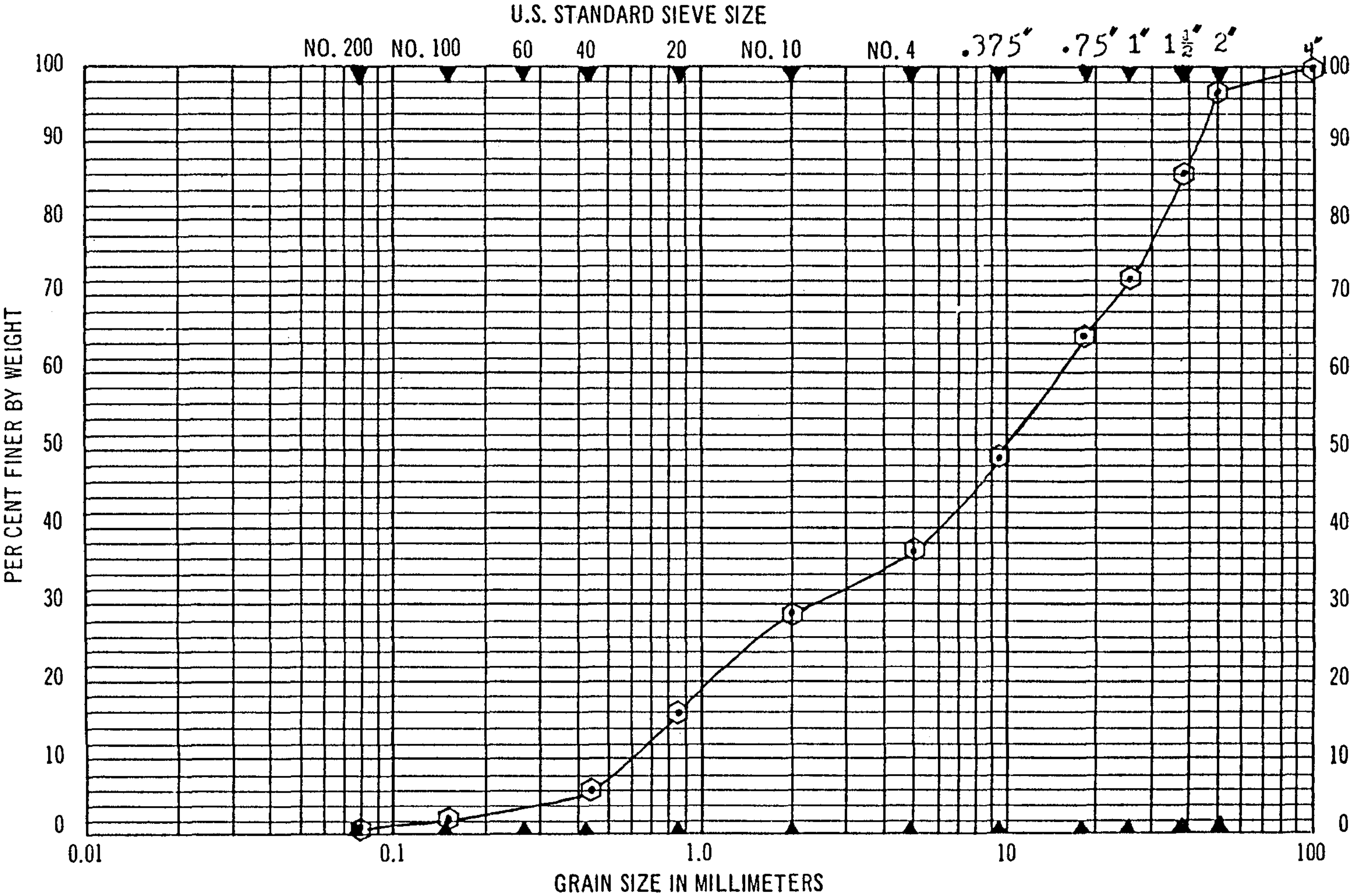
APPENDIX B

SEDIMENT SAMPLES

Several soil samples were taken at selected locations along the Agua Fria River and the New River for this study. The majority of the samples were taken by SLA and the sieve analyses performed by Desert Earth Engineering of Tucson. The grain size distributions are presented in this appendix.

JOB NO. 83-48-3 BY Kossack DATE 4-19-1983

KEY	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
⊙		-3'		Sandy Gravel with trace clay
				Location: 1 mile S. Glendale



SILT OR CLAY	SAND			GRAVEL		*
	FINE	MEDIUM	COARSE	FINE	COARSE	

*COBBLES

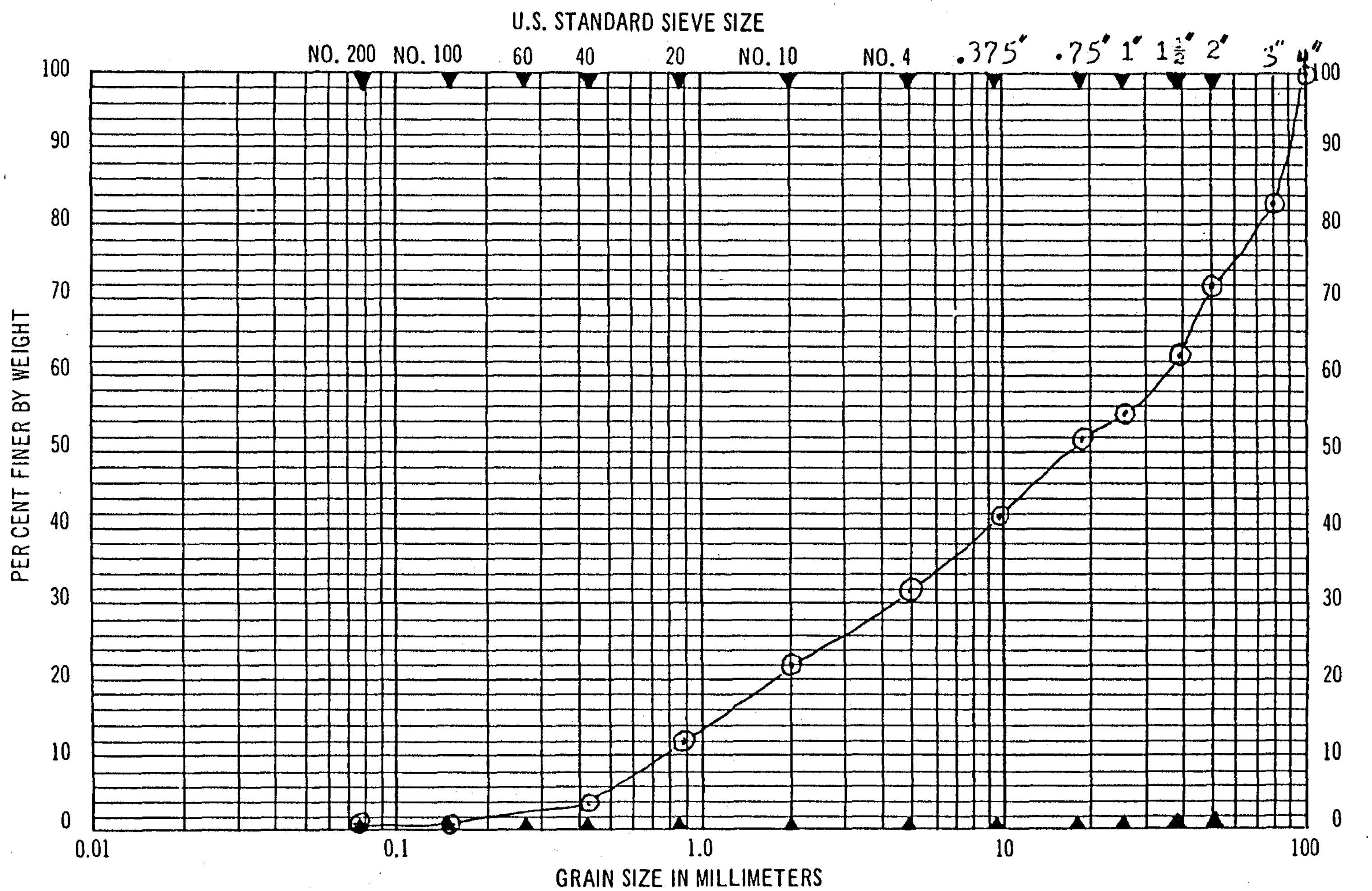
Figure B.1a. Grain size distribution of sediment sample located in Agua Fria River one mile south of Glendale Avenue.

GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

desert earth engineering

JOB NO. 83-48-8 BY Kossack DATE 4-19-1983

KEY	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
⊙		5'		Sandy Gravel with some cobbles and trace clay
				Location: 1 mile S. Glendale



SILT OR CLAY	SAND			GRAVEL		*
	FINE	MEDIUM	COARSE	FINE	COARSE	

*COBBLES

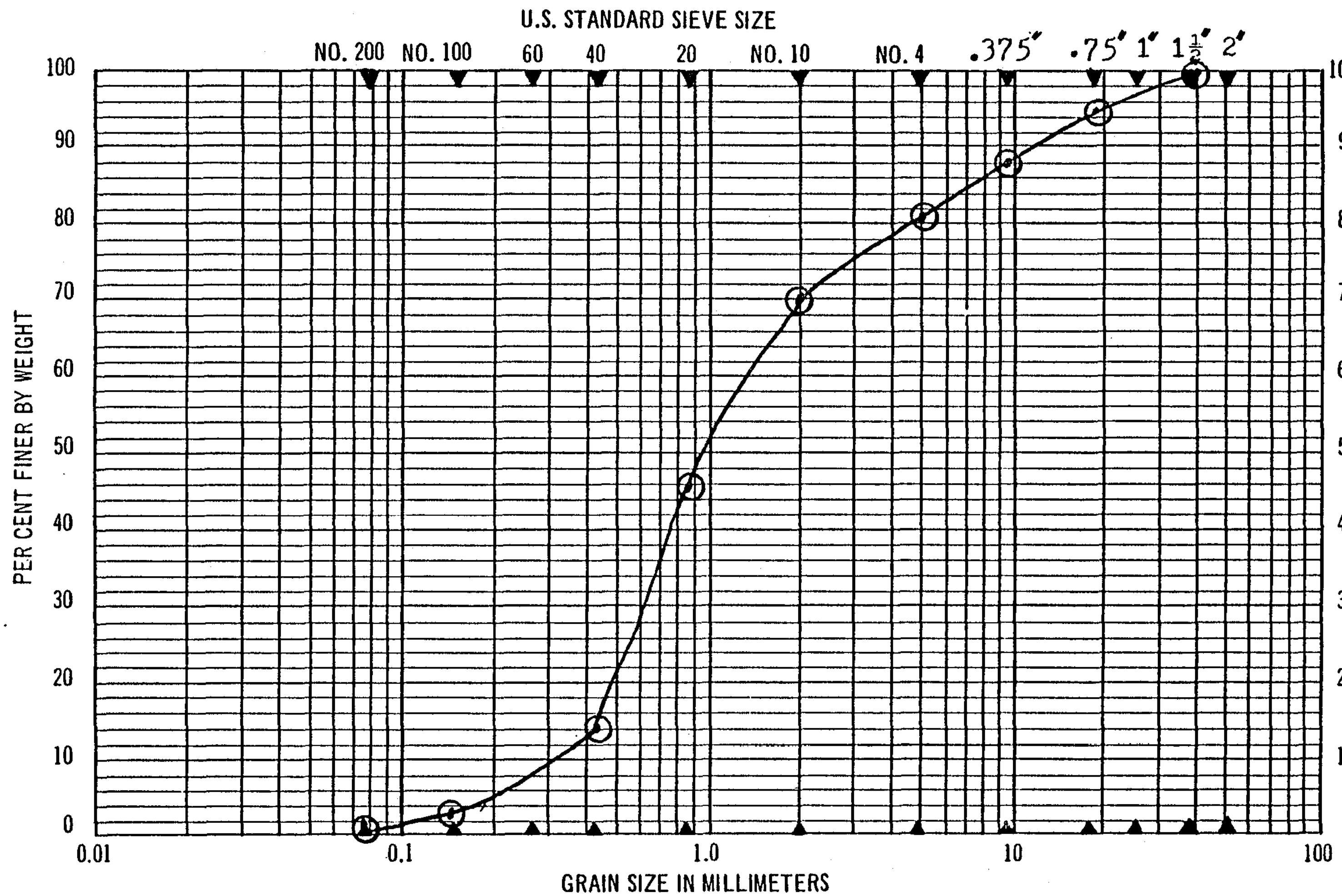
Figure B.1b. Grain size distribution of sediment sample located in Agua Fria River one mile south of Glendale Avenue.

GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

desert earth engineering

JOB NO. 83-30 BY KOSSACK DATE 2-28-83

KEY	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
⊙	83-30-5	12" to 15"	N.A.	Sand with some gravel and trace silt
				Location: 300' Downstream from Confluence
				of New River



SILT OR CLAY	SAND			GRAVEL		*
	FINE	MEDIUM	COARSE	FINE	COARSE	

*COBBLES

Figure B.2. Grain size distribution of sediment sample - Agua Fria River 300 feet downstream of the confluence with New River.

desert earth engineering

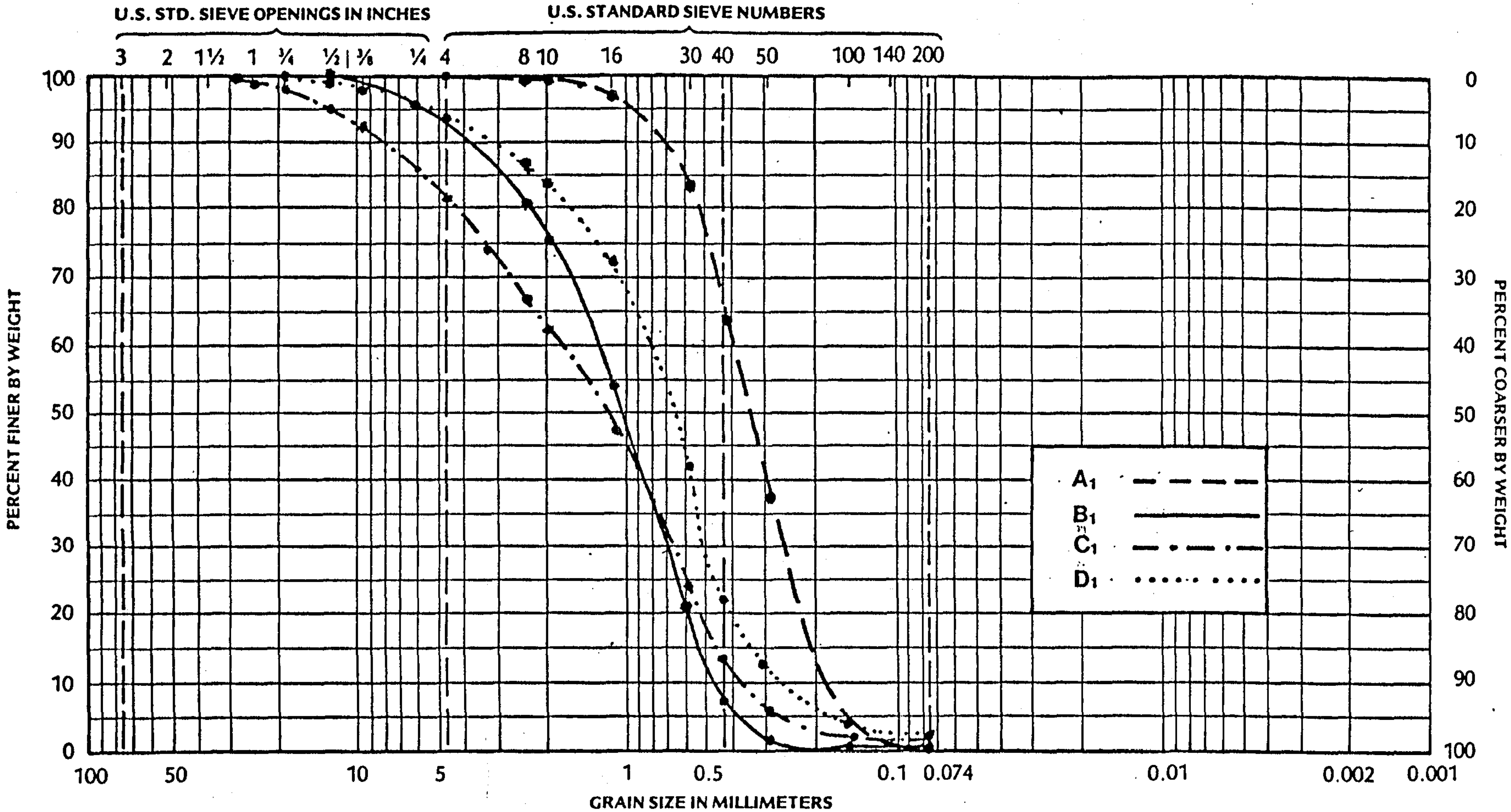
GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

Type of Material Alluvial Deposits Job No. 2121J023

Source of Material Camelback Road Bridge Crossing of Aqua Fria River Lab/Inv. No. 21200569-1

Test Procedure ASTM C136 Tested/Calc. By T. Brennan Date 7-27-81

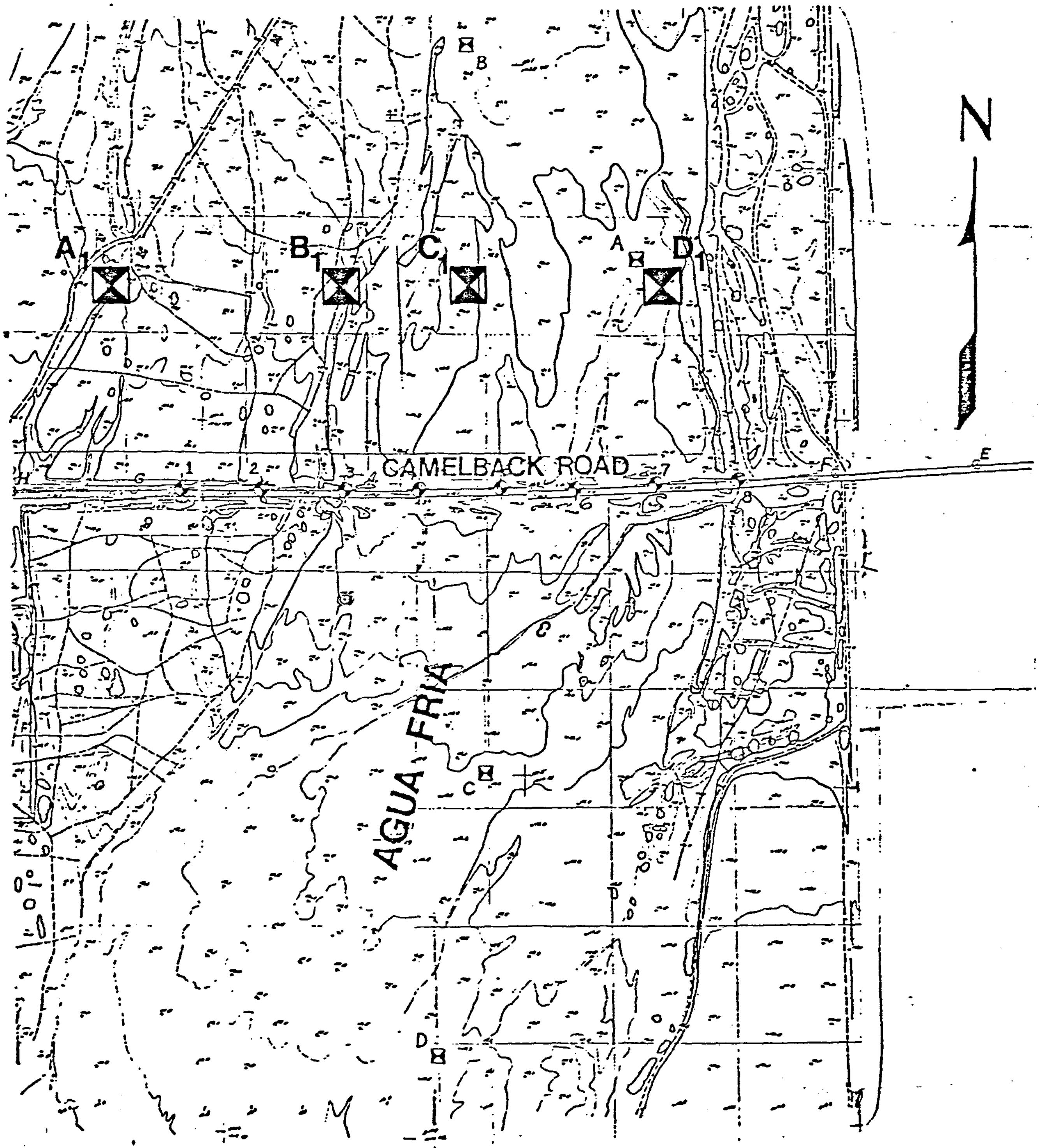
Reviewed By R. Anderson Date 7-27-81



Unified	Gravel	Coarse Sand	Medium Sand	Fine Sand	Silt or Clay	
AASHTO	Gravel		Coarse Sand	Fine Sand	Silt	Clay

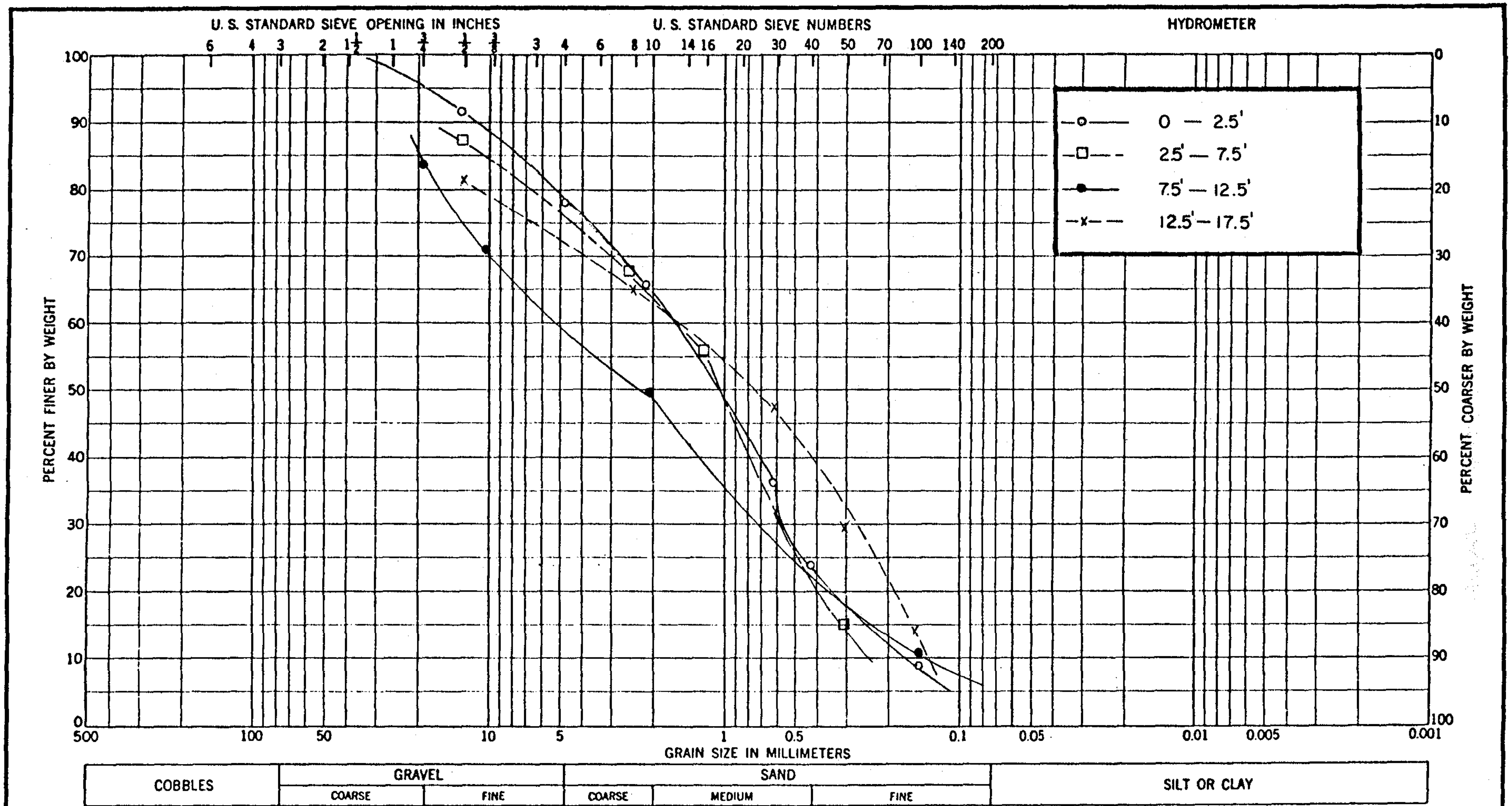
PARTICLE SIZE DISTRIBUTION CHART

Figure B.3. Grain size distribution of sediment sample - Agua Fria River near Camelback Road Bridge Crossing (sampling locations are shown in Figure B.4).



Legend: Test Boring Location · Test Pit Location · Subgrade Sample · Surface Sediment Sample

Figure B.4a. Sampling location map of Agua Fria River near Camelback Road Bridge crossing.



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI

Project AGUA FRIA RIVER

Area INDIAN SCHOOL ROAD

Boring No.

Date

GRADATION CURVES

ENG FORM 1 MAY 63 2087

Figure B.4b. Sample Test by Maricopa County Highway Department near Indian School Road Bridge.

Agua Fria - Thomas Road (upper 6" ~ i.e. surface)

JOB NO. 83-48-7 BY ECP DATE 4/11/83

KEY	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
				Brown GRAVEL with some sand and trace cobbles

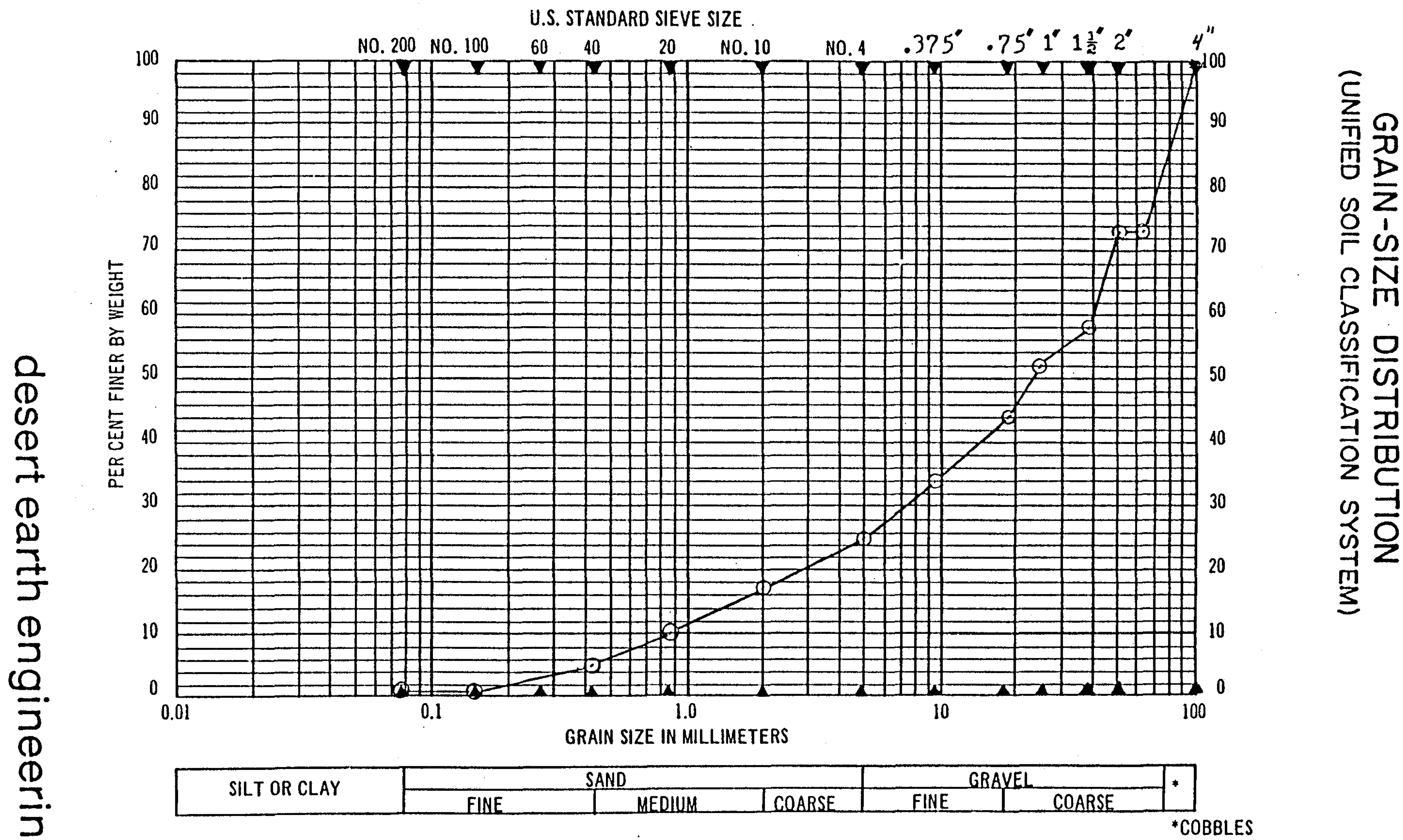
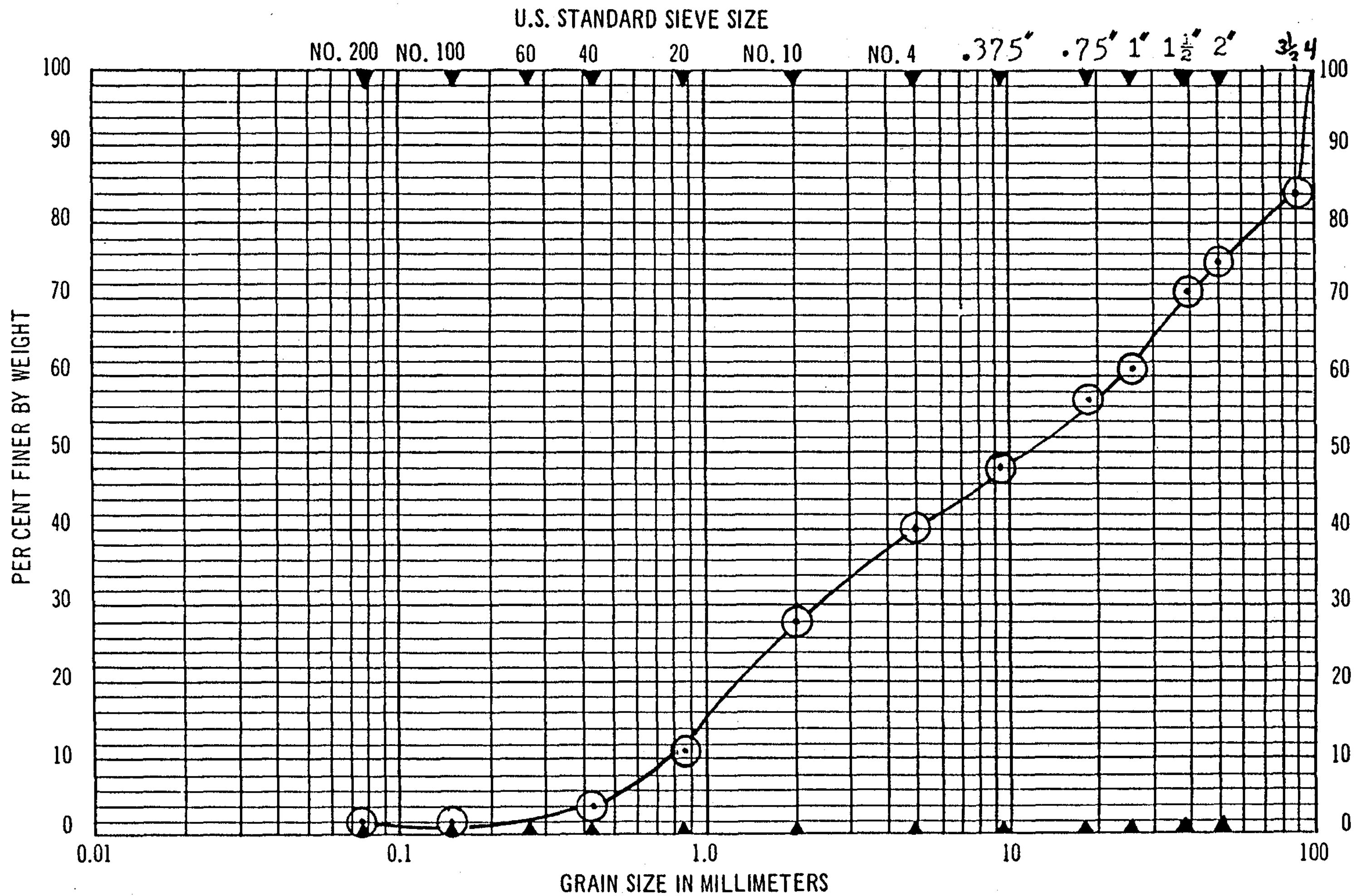


Figure B.5a. Grain size distribution of sediment sample - Agua Fria River near Thomas Road.

JOB NO. 83-48 BY CP DATE 4/12/83

KEY	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
⊙		6'		Gravelly Sand with trace cobbles



SILT OR CLAY	SAND			GRAVEL		*
	FINE	MEDIUM	COARSE	FINE	COARSE	

*COBBLES

Figure B.5b. Grain size distribution of sediment sample - Agua Fria River near Thomas Road.

GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

desert earth engineering

Agua Fria River - Thomas Road (Depth: 8')

JOB NO. 83-48-12 BY ECP DATE 4/11/83

KEY	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
				Brown SANDY CLAY

GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

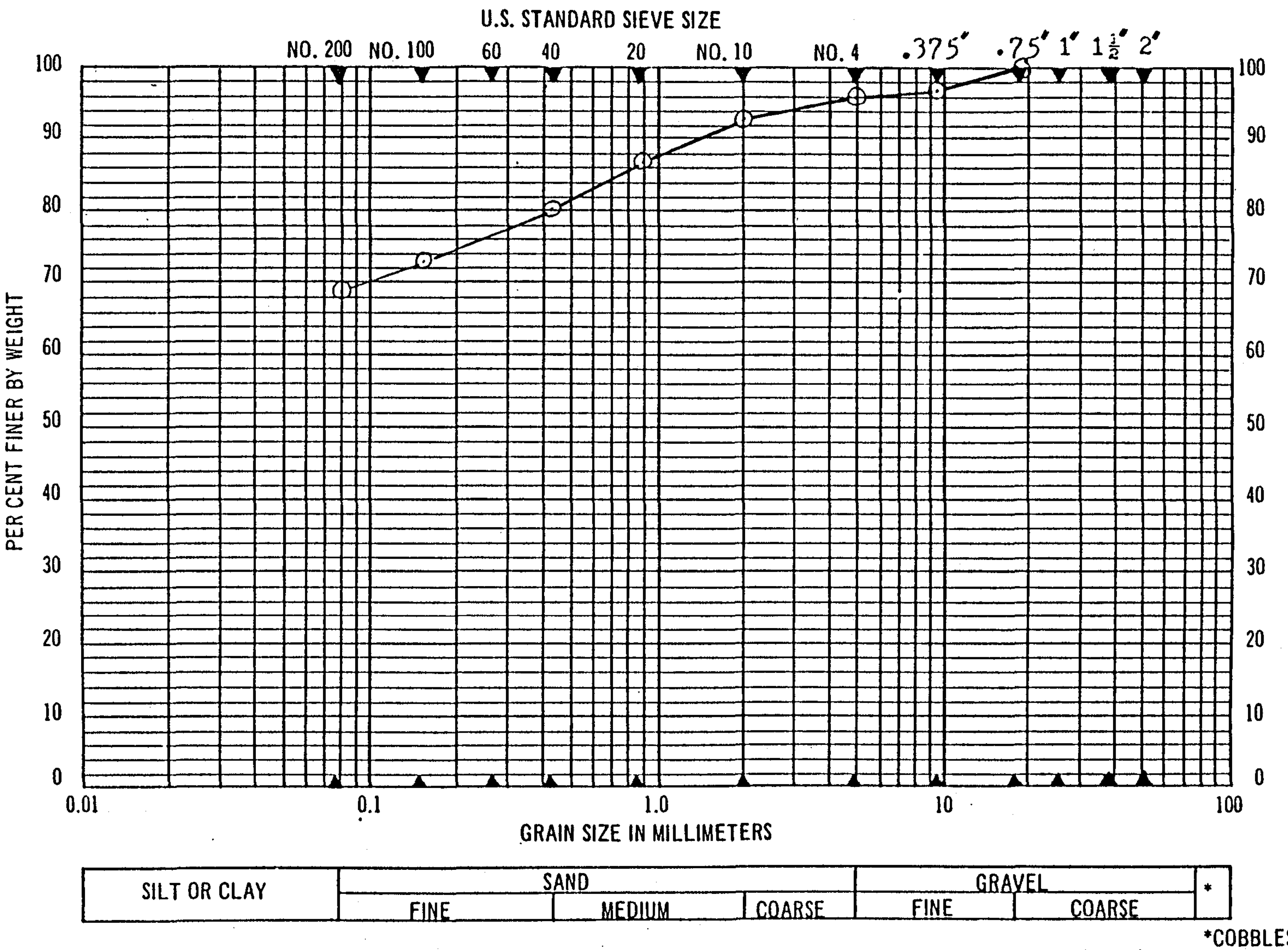


Figure B.5c. Grain size distribution of sediment sample - Agua Fria River near Thomas Road.

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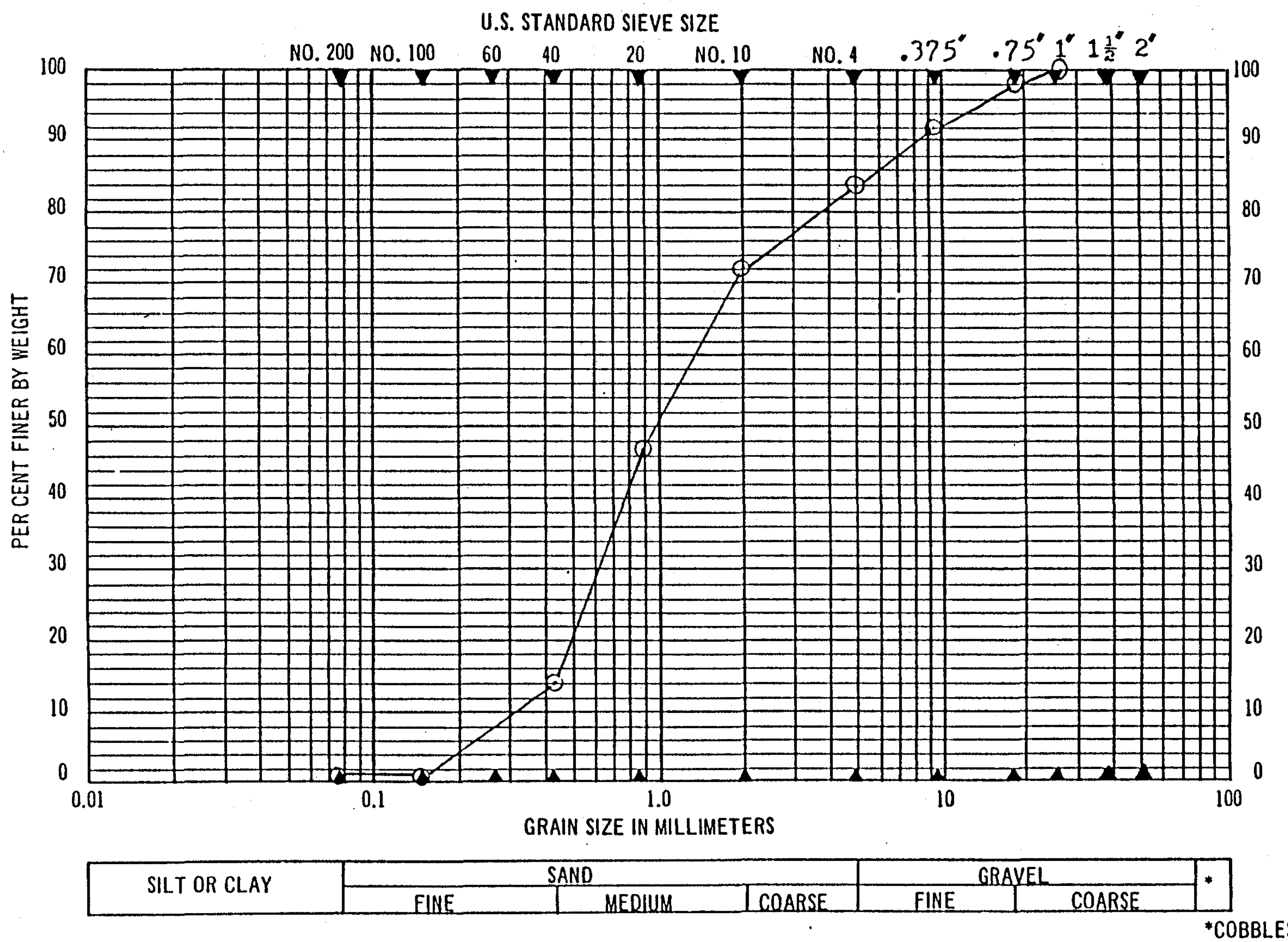
Agua Fria River - 1000' u.s. McDowell (Depth 2' to 7')

JOB NO. 83-48-13 BY ECP DATE 4/9/83

KEY	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
				Brown SAND with some fine gravel



GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

Figure B.6a. Grain size distribution of sediment sample - Agua Fria River 1000 feet upstream of McDowell Road.

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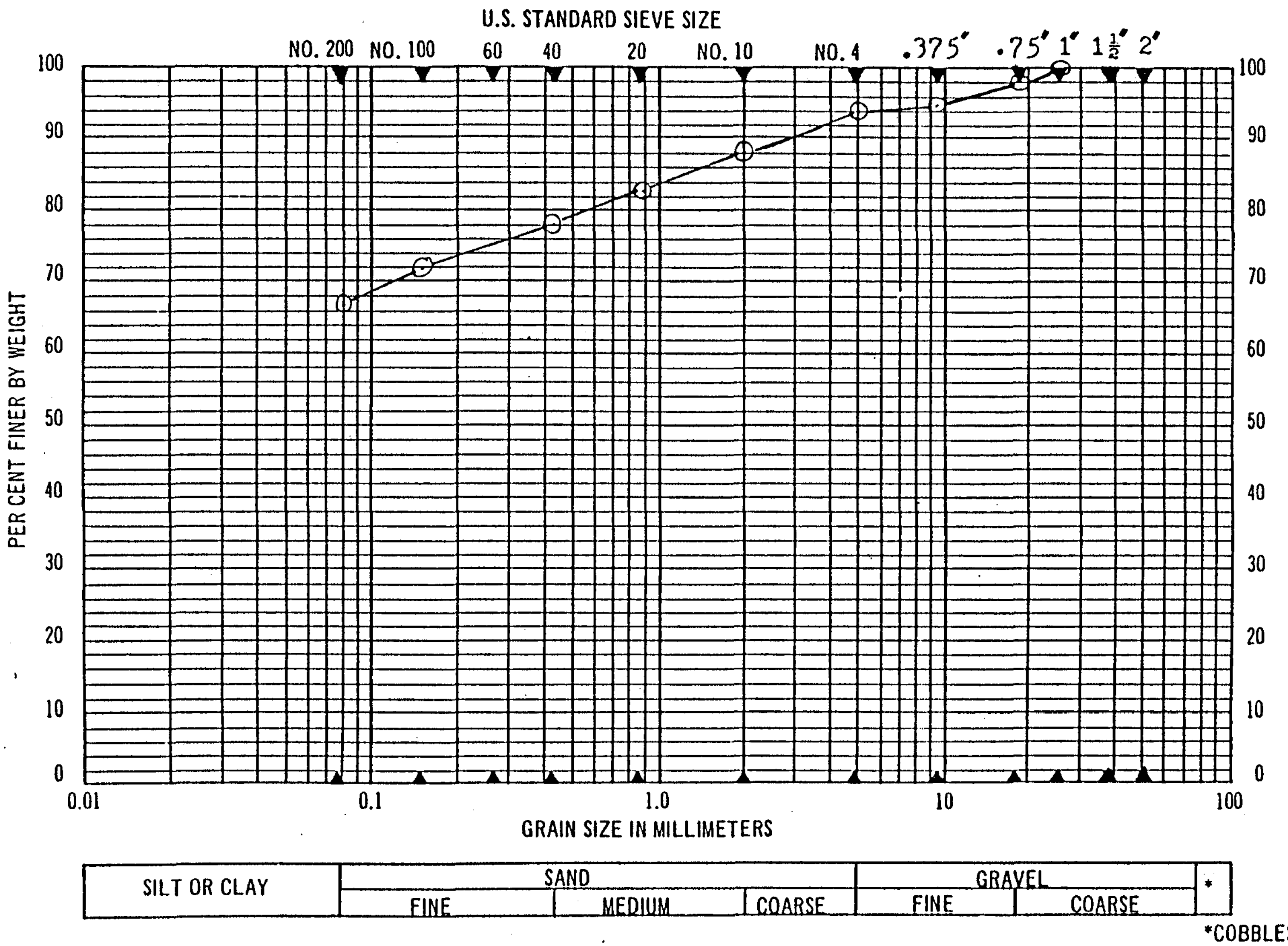
Agua Fria River - 1000' u.s. McDowell (Depth: 8')

JOB NO. 83-48-14 BY ECP DATE 4/9/83

KEY	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
				Brown SANDY CLAY with trace fine gravel



GRAIN-SIZE DISTRIBUTION (UNIFIED SOIL CLASSIFICATION SYSTEM)

desert earth engineering

B.12

Figure B.6b. Grain size distribution of sediment sample - Agua Fria River 1000 feet upstream of McDowell Road.

JOB NO. 83-30 BY KOSSACK DATE 3-2-83

KEY	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
□	83-30-3	12 to 15"	N.A.	Medium to coarse Sand with some gravel
				Location: Buckeye 3 or 1 000' N.

GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

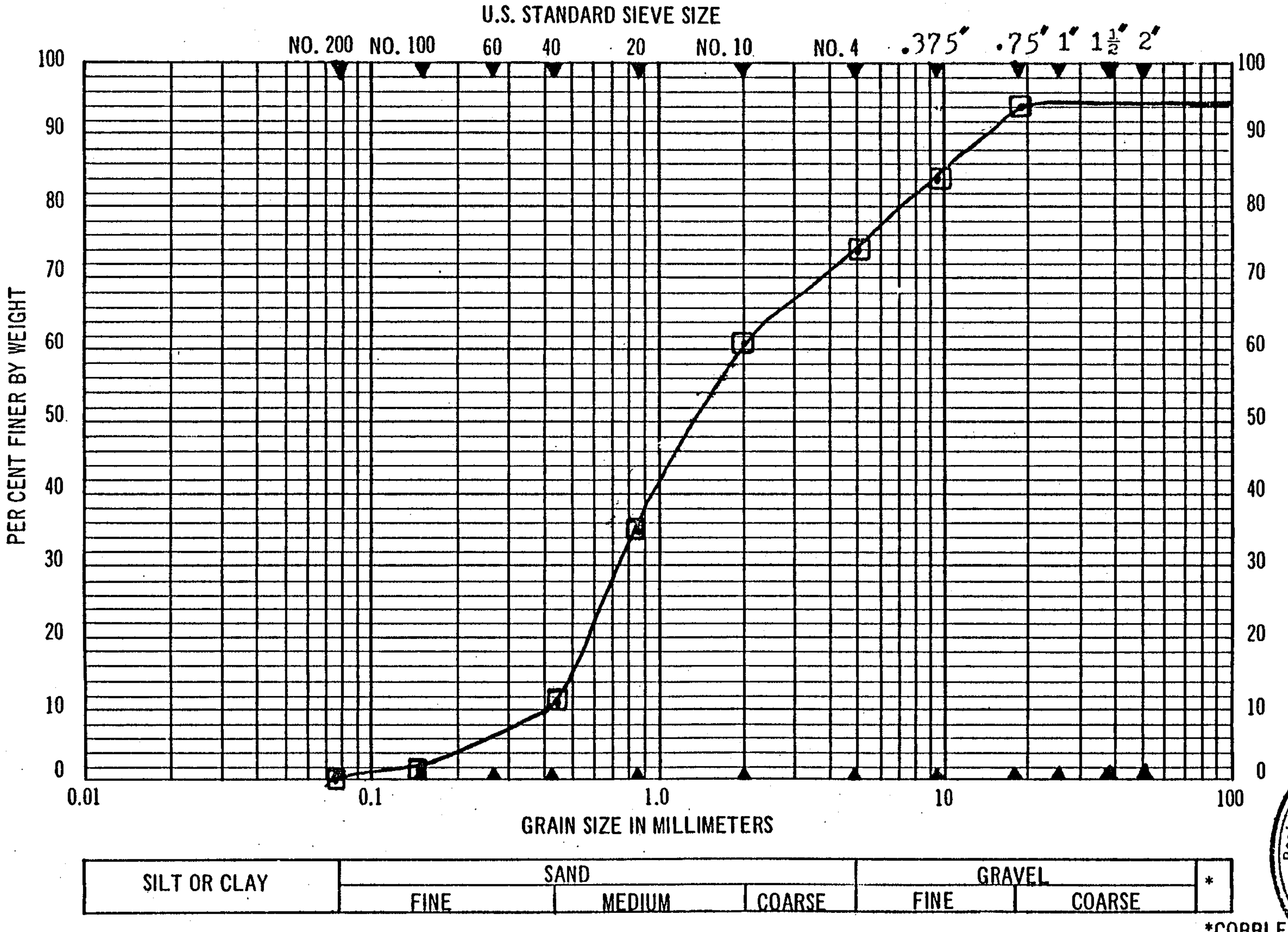


Figure B.7a. Grain size distribution of sediment sample - Agua Fria River 1000 feet upstream of Buckeye Road.

Registered Professional Engineer (Civil) CERTIFICATE NO. 10296 ROBERT L. SOGGE Date Signed 3/3/83 ARIZONA, U.S.A.

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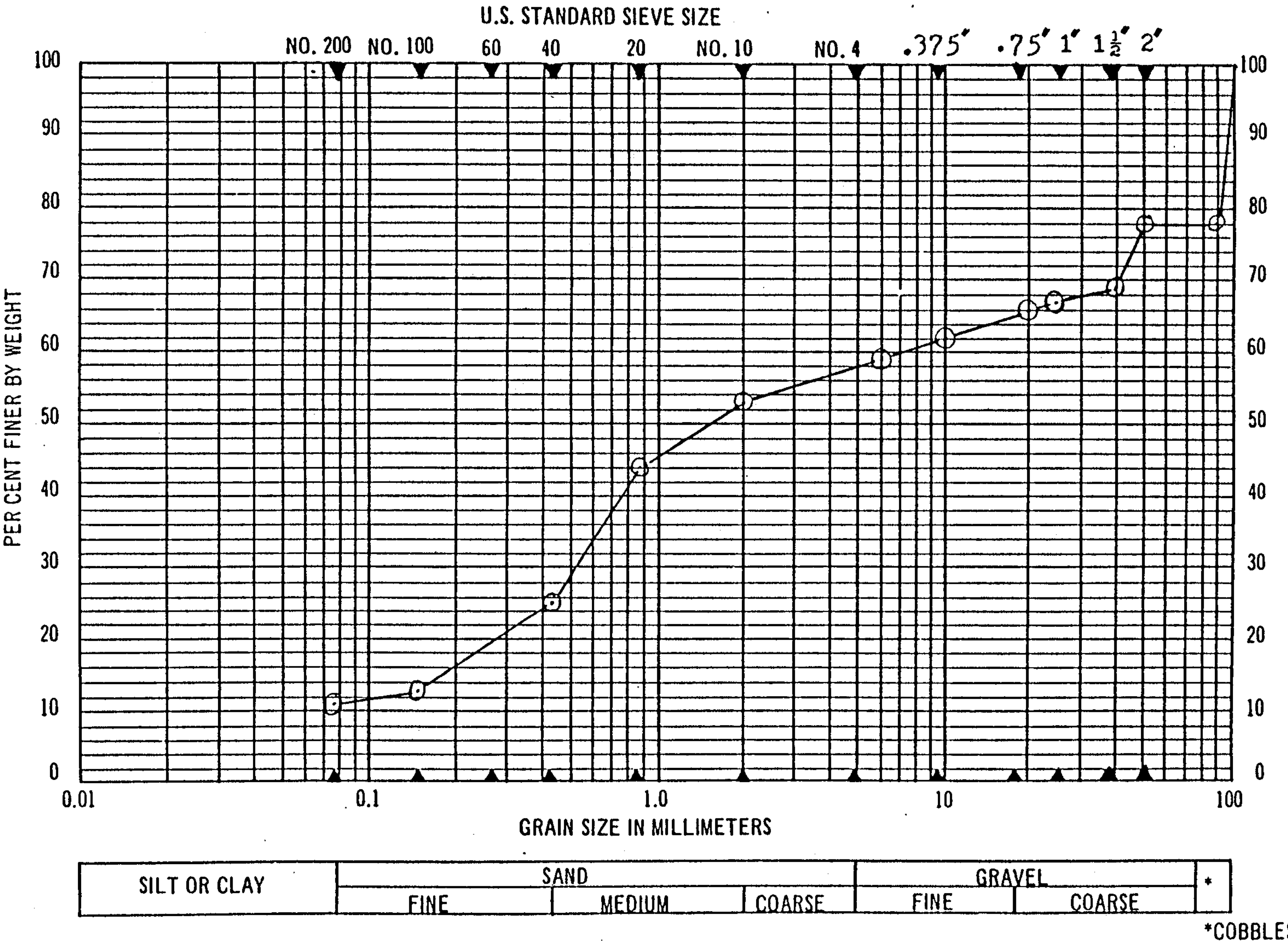
Agua Fria River - 500' u.s. Buckeye Road (Depth: 2')

JOB NO. 83-48-2 BY ECP DATE 4/11/83

KEY	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
				Brown COBBLE and SAND mixture with some gravel and trace silt



GRAIN-SIZE DISTRIBUTION (UNIFIED SOIL CLASSIFICATION SYSTEM)

B.14

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Figure B.7b. Grain size distribution of sediment sample - Agua Fria River 500 feet upstream of Buckeye Road.

Agua Fria River - 500' u.s. Buckeye (Depth 3' to 10')

JOB NO. 83-48-15 BY ECP DATE 4/8/83

KEY.	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
				Brown fine-medium SAND with trace fine
				gravel

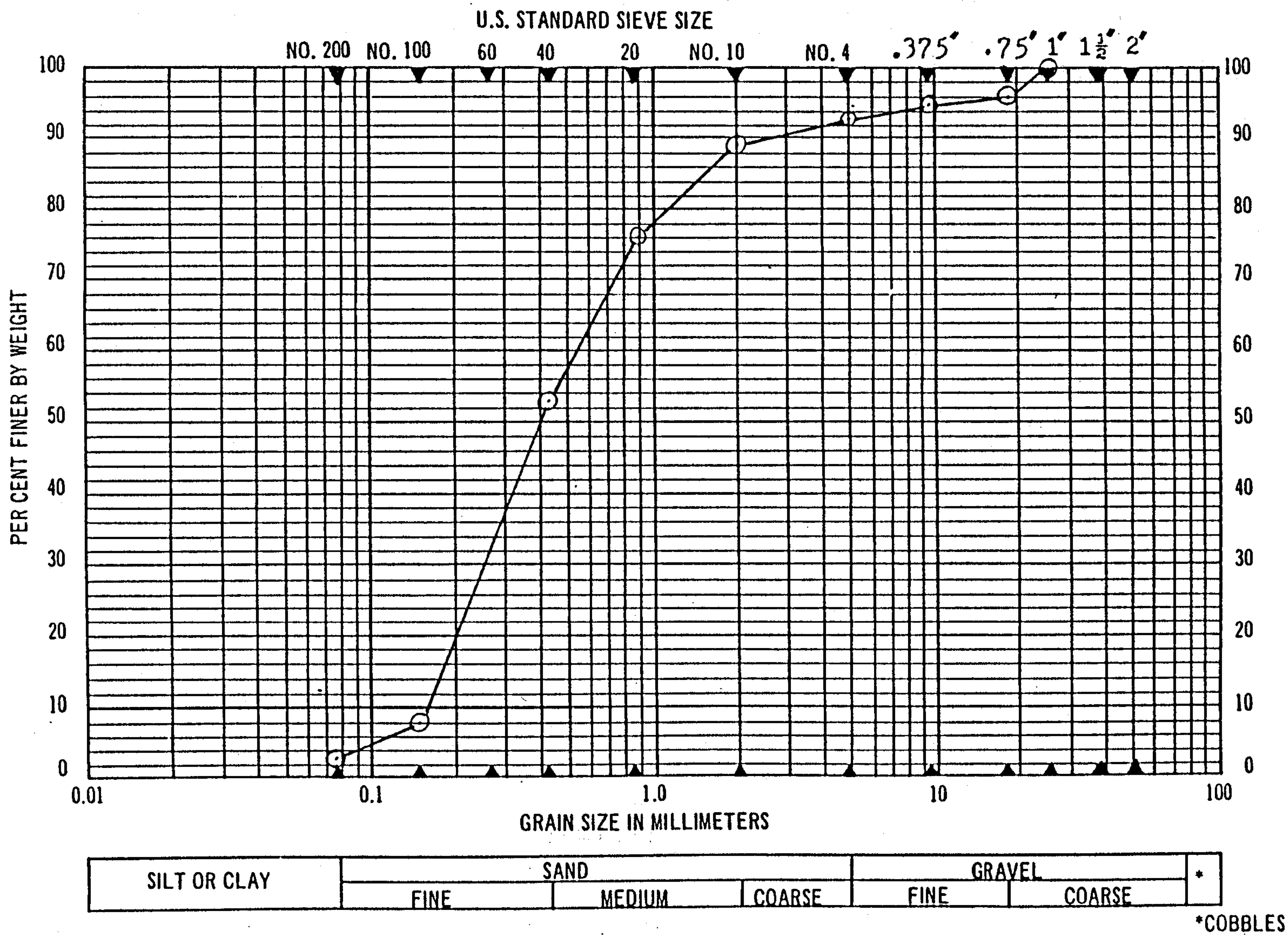

Figure B.7c. Grain size distribution of sediment sample - Agua Fria River 500 feet upstream of Buckeye Road.

GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

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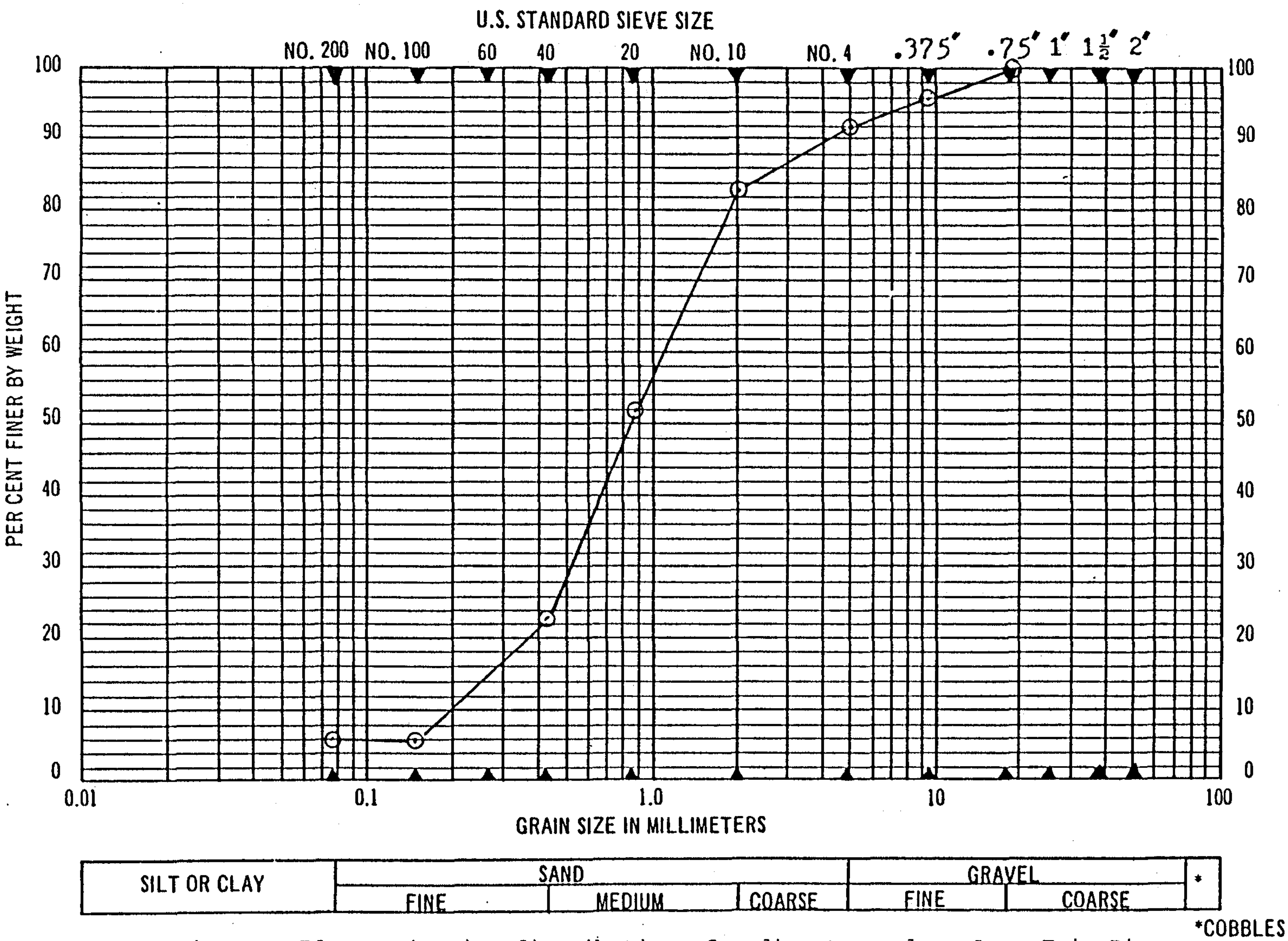
Agua Fria River - 500' north Buckeye (Depth(11'))

JOB NO. 83-48-11 BY ECP DATE 4/9/83

KEY	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
				Red-brown SAND with trace fine gravel



GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

B.16

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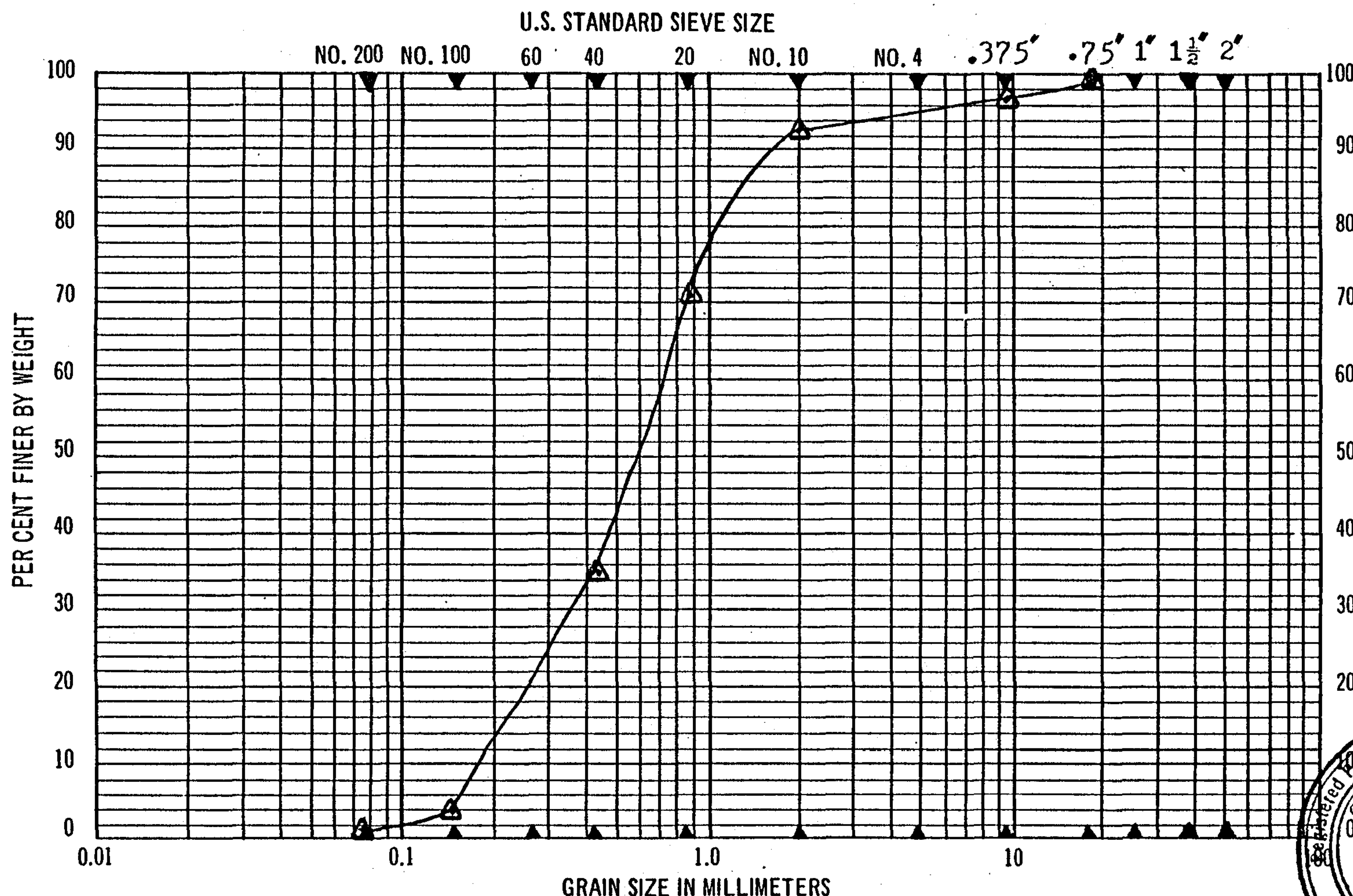
Figure B.7d. Grain size distribution of sediment sample - Agua Fria River 500 feet upstream of Buckeye Road.

JOB NO. 83-30 BY Kossack DATE 2-25-83

KEY	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
△	83-30-1	4 to 6"	NA	Fine to medium Sand with trace gravel and silt
				Location: 500' Upstream from Gila
				Confluence

GRAIN-SIZE DISTRIBUTION (UNIFIED SOIL CLASSIFICATION SYSTEM)



SILT OR CLAY	SAND			GRAVEL		*
	FINE	MEDIUM	COARSE	FINE	COARSE	

*COBBLES

Registered Professional Engineer (Civil)
CERTIFICATE NO. 10296
ROBERT L. SOGGE
Date Signed 3/3/83
ARIZONA, U.S.A.

Figure B.8a. Grain size distribution of sediment sample - Agua Fria River 500 feet upstream of the confluence with Gila River.

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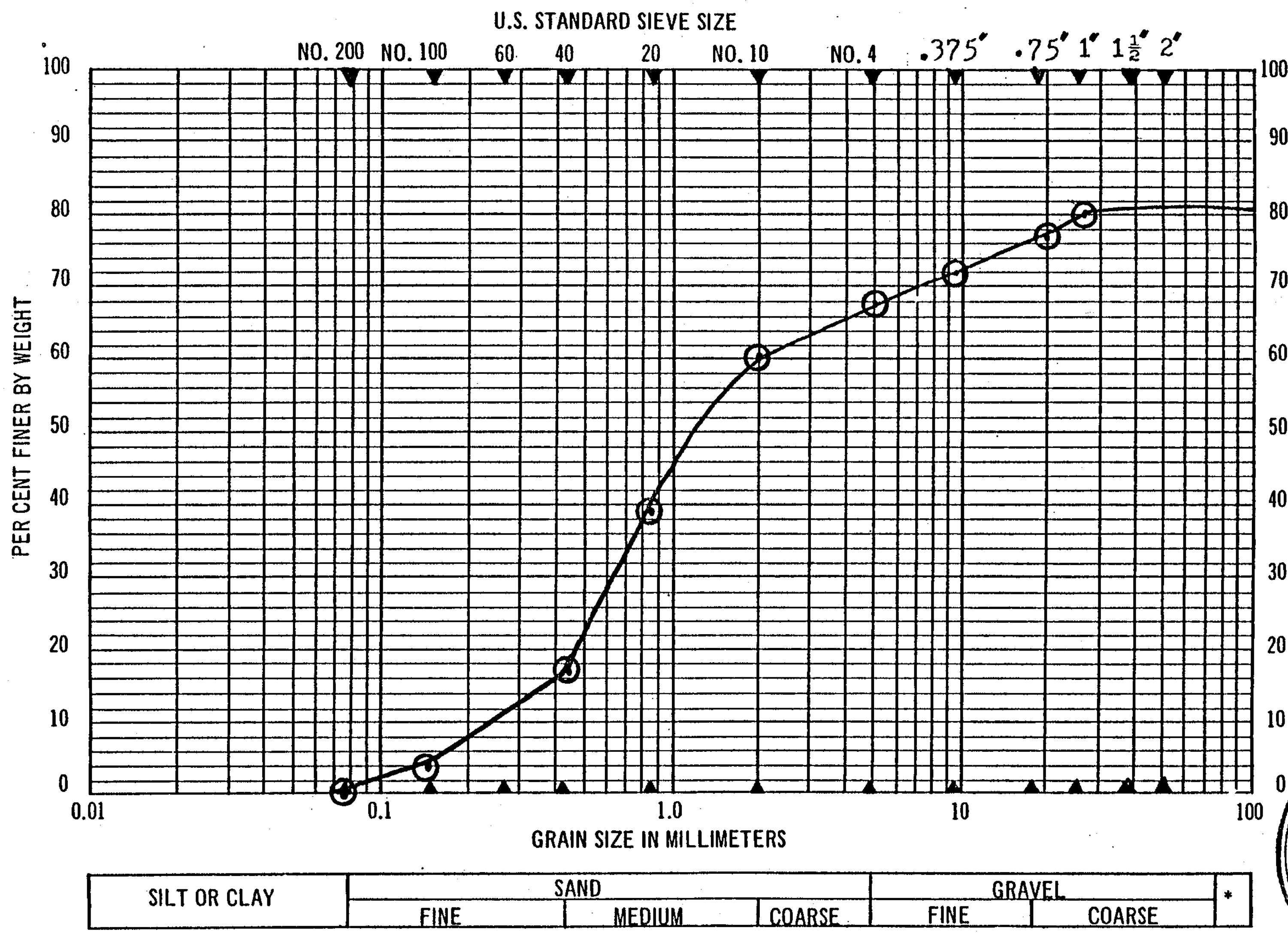
JOB NO. 83-30 BY KOSSACK DATE 3-2-83

KEY	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
O	83-30-2	12 to 15"	N.A.	Gravelly Sand
				Location: 500' Upstream from Gila Confluence



*COBBLES

GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

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Registered Professional Engineer (Civil)
CERTIFICATE NO. 10296
ROBERT L. SOGGE
Date Signed 3/3/83
ARIZONA, U.S.A.

Figure B.8b. Grain size distribution of sediment sample - Agua Fria River 500 feet upstream of the confluence with Gila River.

New River - 500' u.s. from confluence with Agua Fria River (Depth: 3')

JOB NO. 83-48-10 BY ECP DATE 4/9/83

KEY	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
				Dark brown GRAVELLY SAND with trace cobbles

GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

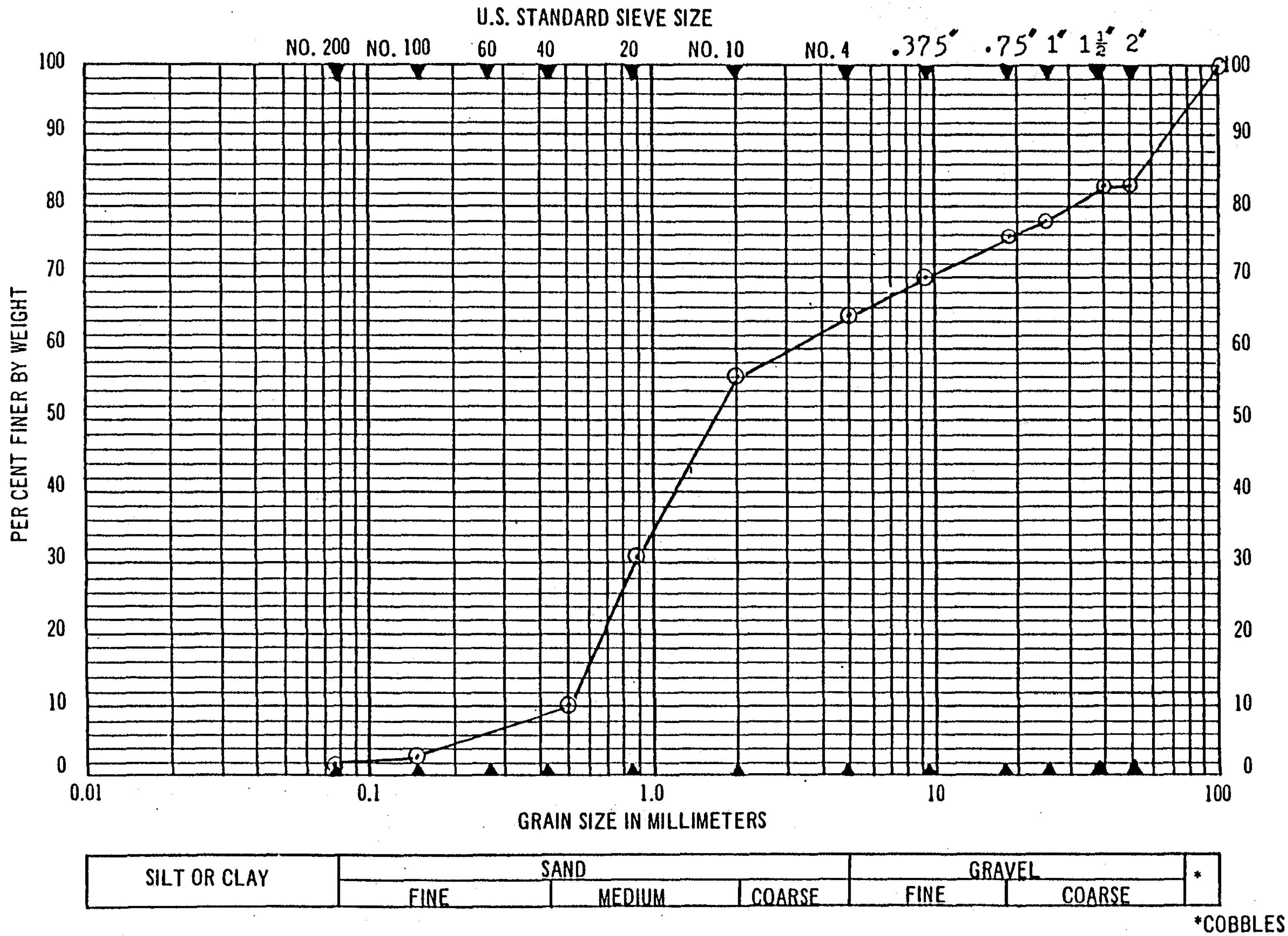

Figure B.9. Grain size distribution of sediment sample - New River 500 feet upstream of the confluence with Agua Fria River.

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New River - 1 mile + South Glendale (Depth: 18 inch)

JOB NO. 83-48-4 BY ECP DATE 4/11/83

KEY	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
				Brown SANDY COBBLY GRAVEL

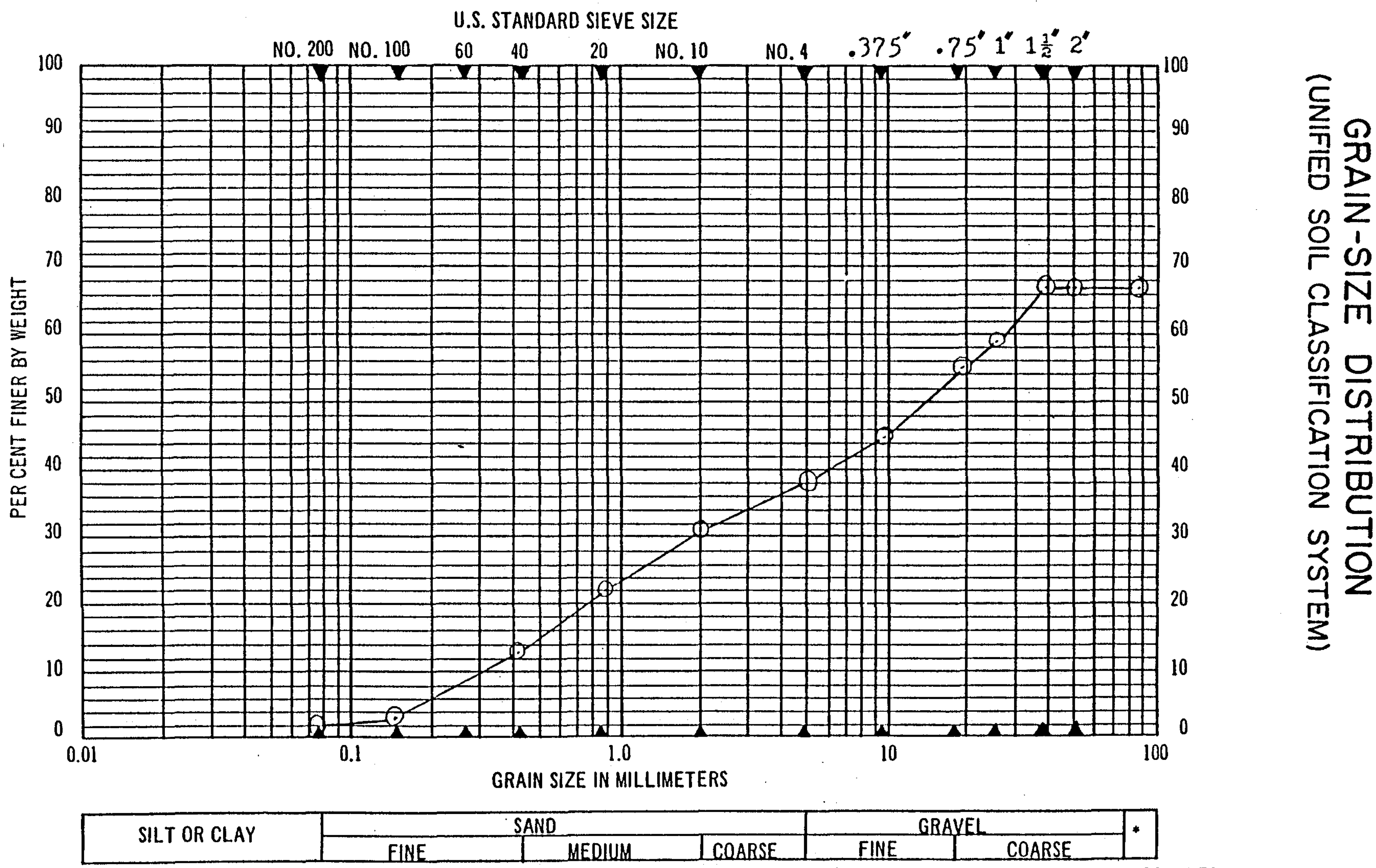

Figure B.10a. Grain size distribution of sediment sample - New River one mile downstream of Glendale Avenue.

New River- 1mi. South of Glendale (Depth: 7')

JOB NO. 83-48 BY KOSSACK DATE 4/19/1983

KEY	BORING	DEPTH	ELEV.	SOIL CLASSIFICATION
⊙		-7'		Cobbly Gravel with trace sand and trace clay
				Location: 1 mile and South Glendale

GRAIN-SIZE DISTRIBUTION
(UNIFIED SOIL CLASSIFICATION SYSTEM)

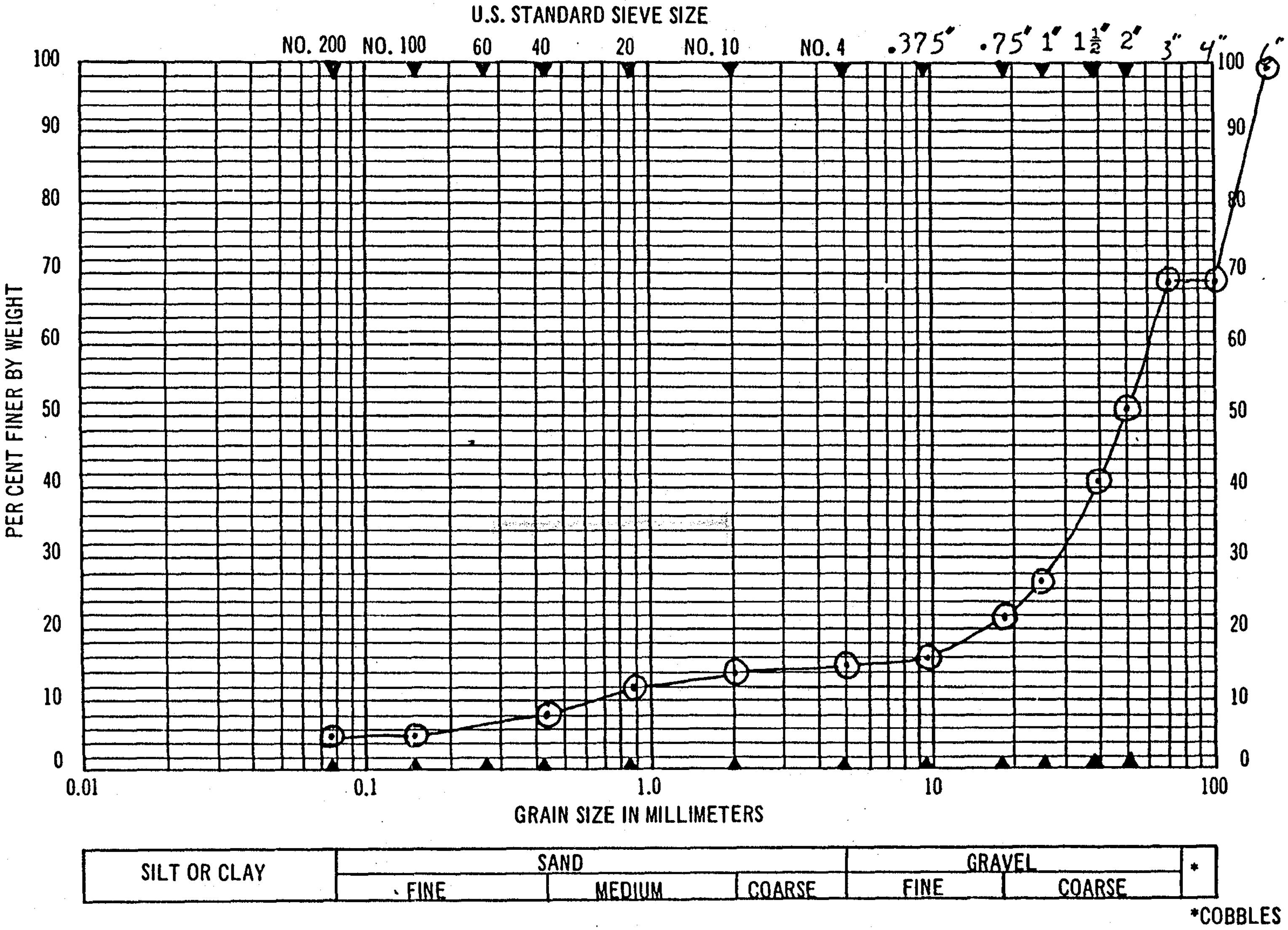

Figure B.10b. Grain size distribution of sediment sample - New River one mile downstream of Glendale Avenue.

desert earth engineering

SIMONS, LI & ASSOCIATES, INC.

3555 STANFORD ROAD
POST OFFICE BOX 1816
FORT COLLINS, COLORADO 80522

TELEPHONE (303) 223-4100
TLX: 469370 SLA FTCN CI
CABLE CODE: SIMONSLI

September 19, 1983

Mr. Richard G. Perreault
Flood Control District of Maricopa County
3335 West Durango Street
Phoenix, AZ 85009

Re: Response to Review of "Hydraulic and Geomorphic Analysis of the Agua Fria River." (Our Project Number AZ-MC-05)

Dear Dick:

Simons, Li & Associates, Inc. (SLA) has reviewed the comments of the Flood Control District of Maricopa County and the Los Angeles District of the Corps of Engineers and have incorporated the comments in the final report or have addressed them in this letter. The comments are attached to this letter.

The twenty comments of the Flood Control District have been addressed in the report, with the exception of comments 4 and 7. SLA does not have a map to replace Figures 1.2 and 3.3. We apologize for the clarity of the map, however, this was the best available map that showed the entire study reach that was available to SLA. In regards to comment 20, we did find a slight error in the HEC-II input deck at the RID flume and I-10 bridges and we made the appropriate changes for water surface elevations considering 10 and 20 percent debris blockages at the bridge crossings. The proper water surface elevations can be found in Table 5.9.

In regards to the Los Angeles District, Corps of Engineers' comments on the report, we offer the following number-by-number replies:

1. Page 11 (Executive Summary) - Your point regarding the accuracy of the contour levels on the topographic maps is well taken. However, once a thalweg is established, the difference from the starting thalweg and the post 100-year flood hydrograph thalweg gives a good quantitative indication of bed response. Thus, with the thalweg established, the erosion and sediment model can give an estimate to relative aggradation/degradation changes within the river bed system for the selected thalweg and will give meaningful results.

2. Page 2.8, paragraph 2.3.1 - This item has been corrected on page 2.8 of the final report.

3. Page 4.2, paragraph 4.2 - This item has been corrected on page 4.2 of the final report.

4. Page 4.14, paragraph 4.6 - A precautionary note has been added regarding the accuracy of contour intervals of topographic maps.

5. Page 4.24, paragraph 4.7 - A precautionary note has been added to conclusion #2 regarding the accuracy of contour intervals in the topographic maps used.

6. Page 5.5, paragraph 5.4 - The equilibrium slope concept is based on the <u>dominant</u> discharge in a river that has the most influence in shaping the cross section. In a sand bed channel, the <u>dominant</u> discharge is one that has the probability of occurring often enough to cause equilibrium conditions to exist. A 100-year return flood will not occur often enough for actual equilibrium conditions to be achieved, while the 2-year return level along the Agua Fria is not bankfull. Thus, a 10-year flood was chosen for equilibrium slope analysis because it is within the range of discharge in which the Agua Fria is at bankfull, and the 10-year return flood has a high enough probability of occurrence for equilibrium conditions to be achieved.

   In a cobble bed stream the <u>dominant</u> discharge may be a flood with a significantly higher return interval than the 10-year. The armor layer may stay intact until the velocities associated with a flood with a large return interval begins transporting the cobble material and altering the channel shape. Thus, the dominant discharge concept is very dependent on the type of bed and bank material, magnitude of bankfull discharge and whether the channel is ephemeral or perennial.

7. Page 5.12, paragraph 5.6.3 - An explanation was inserted in the report detailing how the bridge skewness and pier length were taken into account in the Shen and Neil local scour methodologies.

8. Page 6.2, paragraph 6.3 - If thalweg comparisons are not valid for comparing aggradation/degradation response of the bed in time, it is highly suspect whether or not cross-sectional area comparison would be very meaningful. If the entire cross section has a $\pm$ 2 feet vertical accuracy and the effective channel width is 1,000 feet, the cross-sectional area can be $\pm$ 2,000 square feet. This could lead to erroneous conclusions regarding bed response and would not necessarily be any more accurate than comparing bed response with thalweg.

9. Page 6.4, paragraph 6.4 - The 100-year flood was modeled to get an indication of the bed response to an extreme event. We acknowledge that it would be more accurate to model a scenario of floods, during a 100-year cycle, to evaluate the probable changes in the Agua Fria bed for long-term response. However, for existing conditions we felt the equilibrium slope analysis would be an adequate indicator of the long-term response. Since existing conditions will be modified with proposed channelization, more modeling emphasis was placed in analyzing channelized conditions for future channel bed response. A report documenting expected maintenance in the Agua Fria for future channelization measures will discuss long-term bed response as a result of modeling floods in the 100-year cycle.

   Again, the emphasis in this report was to establish long-term bed response trends and identify problem areas. This can be adequately accomplished through the use of the equilibrium slope concept.

10. Page 7.1, paragraph 7-4 - A precautionary note was added to this conclusion on page 7.1 of the text.

If you have any questions regarding our responses or the final report, please do not hesitate to call.

Sincerely yours,

Ruh-Ming Li
Executive Vice President and
General Manager

MJB,RML:sm
Enclosures
RD118:R347

C.

DEPARTMENT OF THE ARMY
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
P. O. BOX 2711
LOS ANGELES, CALIFORNIA 90053

August 8, 1983

REPLY TO
ATTENTION OF:

SPLED-DM

Mr. D. E. Sagramoso
Chief Engineer and General Manager
Flood Control District of Maricopa County
3335 West Durango Street
Phoenix, Arizona 85009

Dear Mr. ~~Sagramoso~~: Dan,

This is in response to your request for review and comment on the "Hydraulic and Geomorphic Analysis of the Agua Fria River" prepared by Simons, Li and Associates and dated May 16, 1983. The report has been reviewed by our Hydraulics Section, and the inclosed list of comments are provided for your use.

Sincerely,

Norman Arno
Norman Arno
Chief, Engineering Division

Enclosure

FLOOD CONTROL DISTRICT
RECEIVED

AUG 15 '83

\	CH ENG	2a	HYDRO
2	ASST		LMgt
	ADMIN		SUSP
	C & O	5	FILE
			DESTROY
	FINANCE	2	RGP
REMARKS			

LH1.4

Incl #2

Los Angeles District, Corps of Engineers
Comments on "Hydraulics and Geomorphic
Analysis of the Agua Fria River" a Report
Prepared by Simons, Li and Associates and
Dated May 16, 1983

pg XI - Math Modeling (Executive Summary). Summary reference is made to 1.6 and 3.0 feet average channel degradation. Please refer to detail comments under pg 4.14 para 4.6 below.

pg 2.8 para 2.3, 1. For existing conditions (without ACDC), Dreamy Draw Reservoir and Cave Buttes Reservoir cannot contribute to the Agua Fria design flood calculations. However, present and future discharges on the Agua Fria River are considered to be the same with or without the COE project for flood frequencies shown in Table 2.3.

pg 4.2, para 4.2. A new heading such as "other Phoenix Area Drainage Systems" should have been used to categorize the drainage systems outside of the Agua Fria River system. To our knowledge no structure called the "Union Hills Diversion Channel" is being constructed as stated. Earlier planning alternatives for the COE project included such a channel but it was not included in the authorized plan. The apparent confusion may result from current construction of Skunk Creek Channels and Levees which are part of the Adobe Dam feature.

pg 4.14, para 4.6. The 1973 thalweg elevations are taken from 1972 topographic maps with 4 foot contour interval. The normal map elevation accuracy between contours would be ± 2 feet. The 1981 thalweg elevations were digitized from 2 foot contour interval maps and thus have an elevation accuracy of about ± 0.5 feet. We do not quarrel with the general degradation of the river thalweg from 1972 to 1981, but the magnitude is masked by the ± 2.5 feet combined map vertical tolerance. A precaution note should be added to this effect.

pg 4.24 para 4.7. Comment above for pg 4.14 is equally applicable to conclusion #2.

pg 5.5 para 5.4. The definition of "equilibrium slope" should be expanded to include all Agua Fria discharges contained within the channel bank extensions at each flood stage, rather than just the 10-year flood. The main channel, even when it overtops its banks, transports the bulk of the sediment load and thus controls the channel equilibrium slope at all flood stages. It would have been helpful to have been able to compare the existing slope with the channel equilibrium slope at a range of flood stages.

RGP 5.12 5.6.3
pg ~~5.14~~ para ~~5.63.~~ It is not clear how the bridge skewness and pier length are taken into account by the Shen's and Neil's equations.

Incl 1

pg 6.2 para 6.3. The 1973 thalweg elevations (1972 topo maps) have a map accuracy of ± 2 feet as mentioned in pg 4.14 comments above. It is questionable whether such a "coarse base line standard" can be used to calibrate and/or verify the model to the level of accuracy of the results shown in table 6.1. There are several methods available to determine thalweg elevations from contour maps, and table 6.1 compares two different methods. Thalwegs are often taken as minimum spot elevations, which generally are small depressions, and thus, not necessarily representative of the localized channel bottom elevation.

The net aggradation or degradation across the full width of the channel determines the time function sediment yield. A comparison of the cross-section area changes would be a more accurate indicator of sediment transport characteristics, and may or may not confirm thalweg changes. As a very minimum, all thalweg elevation differences less than the elevation tolerance limits, should be grouped in a class as having "no determinable aggradation or degradation." Thus, half of the table 6.1 "measured" thalweg changes would be classified as qualifying rather than quantifying measurements.

pg 6.4 para 6.4. The 100-year flood model represents a single event. A scenario of floods, during a 100-year cycle, is required to evaluate the probable changes in the Agua Fria channel bottom elevation.

RGP

pg 7.1 para VIII-4. Comments above for pg 6.2 apply equally to the conclusions about the accuracy of thalweg lowering.

FLOOD CONTROL DISTRICT
of Maricopa County Comments
on the "Hydraulic and Geomorphic
Analysis of the Agua Fria River".

1.) Page IX; Third paragraph; Change to read: "...include a new proposed Waddell Dam, ... and a partially constructed I-10 collector channel."

2.) Page X; Third paragraph; typo: should read "... RID Flume,...

3.) Page 1,2; Paragraph 1.1; typo: should read "... Yavapai County...

4.) Page 1.4; Figure 1.2; replace with better map, if possible.

5.) Page 2.8; Paragraph 2.3.1; Change to read: "...hydrology studies in conjunction with the New River and Phoenix City Streams Project and has documented ..." Note: The last sentence should read, "..., as derived by the Corps of Engineers, is ..."

6.) Page 2.9; Table 23; the 10-year flood D/S New River Confluence is 32,000 ≠ 18,000.

7.) Page 3.9; Figure 3.3; See comment #4.

8.) Page 4.2; First paragraph; Change: "Glendale-Maryvale Canal" to Agua Fria-Dysart Drain". Delete the last sentence; ie: the Union Hills Diversion Channel was a Corps of Engineers' alternative that was dropped.

9.) Page 4.2; Paragraph 4.3; last sentence of the third sub-pragraph should read "... Glendale, Goodyear, and Avondale."

10.) Page 4.3; Table 4.1; DELETE the references to the gravel pits at Thomas Road and Broadway Road.

11.) Page 4.18; Paragraph 4.6.1.4; Paragraph needs to be re-written. Adobe Dam is on Skunk Creek.

12.) Page 4.18; Paragraph 4.6.1.5; Change the second sentence to read: "... 164 square miles of the existing 340 square mile ..."

13.) Page 4.19; Figure 4.11; Indian Bend Wash should be extended from the Arizona Canal to the Salt River. (See attached Figure)

14.) Page 4.20; Paragraph 4.6.2; Delete Cashion from second subparagraph.

15.) Page 4.23; Figure 4.13; Replace with better photograph, if possible.

~~16.) Page 4.25; Figure 4.14; Indicate north arrow on~~ the photograph.

17.) Page 4.26 and 4.27; Figures 4.15 & 4.16; see comment #15.

18.) Page 4.29; Paragraph 4.7(6); see comments #1 & 11.

19.) Page 5.8; typo; end of first line.

20.) Page 5.13; Table 5.9; check the w/s elevations for the RID Flume and I-10. A statement as to why the elevations do not change with the increased blockage may be helpful. The available freeboard for Buckeye Road should be 3.3 feet ≠ 3.4 feet.

Figure 4.11. Location of the Arizona Canal and the study reach.