FINAL SEDIMENT TRANSPORT REPORT FOR THE NEW RIVER AND SKUNK CREEK

Prepared for:

U.S. Army Corps of Engineers
Los Angeles District
P. O. Box 2711
Los Angeles, CA 90053

Submitted by:

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4030 Birch Street
Suite 103
Newport Beach, California 92660

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TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

VII. SEDIMENT ROUTING ANALYSIS

VIII. SUMMARY AND CONCLUSIONS

LIST OF FIGURES

LIST OF FIGURES (continued)

LIST OF TABLES

LIST OF TABLES (continued)

LIST OF TABLES (continued)

I. INTRODUCTION

1.1 Problem Statement

The Phoenix metropolitan area has been and continues to be a high growth rate area. As in other urbanizing areas, changes are occurring in the watersheds and along conveyances that influence water and sediment runoff. The New River, a major tributary to the Agua Fria River, and Skunk Creek, a major tributary of New River, collectively drain a watershed of about 340 square miles northwest of Phoenix. The potential development growth along the lower reaches of the New River and Skunk Creek are motivating the need for flood control measures. Recent changes that have occurred along Skunk Creek include a recreational baseball complex, a large horse breeding farm with a training track and a new sand and gravel operation. Anticipated changes along the New River include the Desert Harbor residential development, the Plaza Del Rio development, the "outer loop" freeway and a municipal airport. These developments in combination with existing bridge crossings, residential developments, sand and gravel operations, etc., encroach on the existing floodplain and may necessitate channelization, construction of levees and/or implementation of other flood control measures. To adequately evaluate channel response to proposed flood control measures a comprehensive hydraulic, erosion and sedimentation of the New River and Skunk Creek is required.

1.2 Scope of Work

To meet the need for comprehensive analysis of New River and Skunk Creek the Corps of Engineers (COE) contracted Simons, Li and Associates, Inc. (SLA). The study reach on the New River was defined from the confluence with the Agua Fria River upstream to approximately one mile above the confluence with Skunk Creek. The study reach on Skunk Creek was defined from the confluence with the New River to approximately one mile above the confluence with the Arizona Canal Diversion Channel (ACDC). The total study reach distance was about 12 river miles, nine on the New River and three on Skunk Creek.

The solution procedure involved three levels of analysis: 1) qualitative geomorphic analysis; 2) quantitative geomorphic analyses; and 3) sediment routing analysis. To adequately evaluate the proposed flood control alternative it was first necessary to establish as-is conditions. This was accomplished through application of the qualitative and quantitative geomorphic levels of analysis. Results of these analyses contributed to development of the preferred flood control alternative. This alternative, as established by the COE, was then evaluated using the sediment routing analysis. The specific scope of work for the qualitative and quantitative geomorphic analysis of the as-is conditions involved:

1. Site visits to familiarize key personnel with study reach.

2. Collection and assembly of pertinent data necessary to conduct all analyses. This data included aerial photographs, topographic maps and information, climatical hydrological, hydraulic, geologic and soils data, bridge plans and other structural information, reservoir information, etc.

3. Application of the HEC-2 computer program to establish hydraulic data for as-is conditions. Model application was made for peak discharge of 10-, 25-, 50- and 100-year floods, considering all bridge crossings in the study reach. Manning values appropriate for a flood plain study and values appropriate for a sediment transport study were utilized.

4. Grouping of HEC-2 cross sections with similar hydraulic, geometric and sediment characteristics and providing summaries of average hydraulic conditions for each reach.

5. Assessment of the adequacy of bridge crossings to pass 100-year flood with sufficient freeboard and summarizing breakout areas, levee overtoppings, pressure flow conditions and other problem areas.

6. Completion of a qualitative geomorphic analysis of river system for as-is conditions, including discussion of site visit observations, analysis of aerial photographs, sediment particle size analysis, etc.

7. Completion of quantitative geomorphic analysis of river system for as-is conditions, including evaluation of sediment transport capacity and sediment supply, equilibrium slope analysis, etc.

The specific scope of work for the sediment routing analysis of flood control alternative involved:

1. Application of QUASED, the SLA-developed water and sediment routing program, to the proposed floodway for the 100-year hydrograph.

2. Completion of plots of the channel bed response (at the end of the design flood) against the channel invert profile.

3. Tabulation of sediment rating curves throughout the study reach based on sediment transport rates computed in the QUASED program.

4. Establishment of sediment rating curves throughout the study reach based on sediment transport rates computed in the QUASED program.

5. Computation of average annual sediment yields for each reach with the sediment rating curves established in Task 4. An incremental probablistic methodology was utilized.

All analyses were based on existing conditions as of September, 1983. Results of the analysis of the as-is condition were presented in the "Qualitative and Quantitative Geomorphic Analysis of the New River and Skunk Creek for As-is Conditions" (SLA, August, 1984). Results of the analysis of the flood control alternative were presented in the "Draft Sediment Transport Report" (SLA, December, 1984). These two reports were then merged, resulting in this "Final Sediment Transport Report for the New River and Skunk Creek."

II. PHYSICAL CHARACTERISTICS OF STUDY AREA

2.1 Watershed

The New River is a major tributary of the Agua Fria River which flows into the Gila River in central Arizona. With its headwaters in the New River Mountains about 40 miles north of Phoenix, the New River flows generally south for about 40 miles to its confluence with Agua Fria River, about 15 miles west of Phoenix. Skunk Creek is the major tributary to the New River. Water from the ACDC enters the New River drainage via Skunk Creek. Total drainage area of the New River watershed is about 430 square miles, with Skunk Creek (including the ACDC) contributing about 200 square miles. Figure 2.1 is a location map for the study area.

Elevations in the basin range from a little over 5,000 feet in the New River Mountains to about 1,040 feet at the confluence of the New and Agua Fria Rivers. Stream gradients on the New River range from about 370 feet per mile in the mountains to ten feet per mile in the valley. Skunk Creek Stream gradients range from about 450 feet per mile in the New River Mountains to about 30 feet per mile in the valley.

Existing and proposed flood control works in the system include the New River Dam (New River), Adobe Dam (Skunk Creek), Cave Buttes Dam (Cave Creek), the Dreamy Draw Dam (Dreamy Draw Wash), the Arizona Canal and the ACDC. There are 164 square miles above the New River Dam site, 41 percent of which are mountainous. Forty-one percent of the 90 square miles above Adobe Dam on Skunk Creek are mountainous.

The ACDC will be just upstream from and nearly parallel to the Arizona Canal. It will extend about 17.3 miles from Cudia City Wash, at the upstream end, to Skunk Creek, about 2 miles upstream of the confluence of Skunk Creek and New River. Outflow from both Cave Buttes Dam and Dreamy Draw Dam will flow into the ACDC.

2.2 Geology And Physiography

About one-third of the New River drainage area is mountainous, and the remaining two-thirds is valley. The mountainous areas of the New River Mountains above about 3,000 feet are characterized by rugged terrain and steep gradients. The lower areas consist of fairly flat valley land with regular

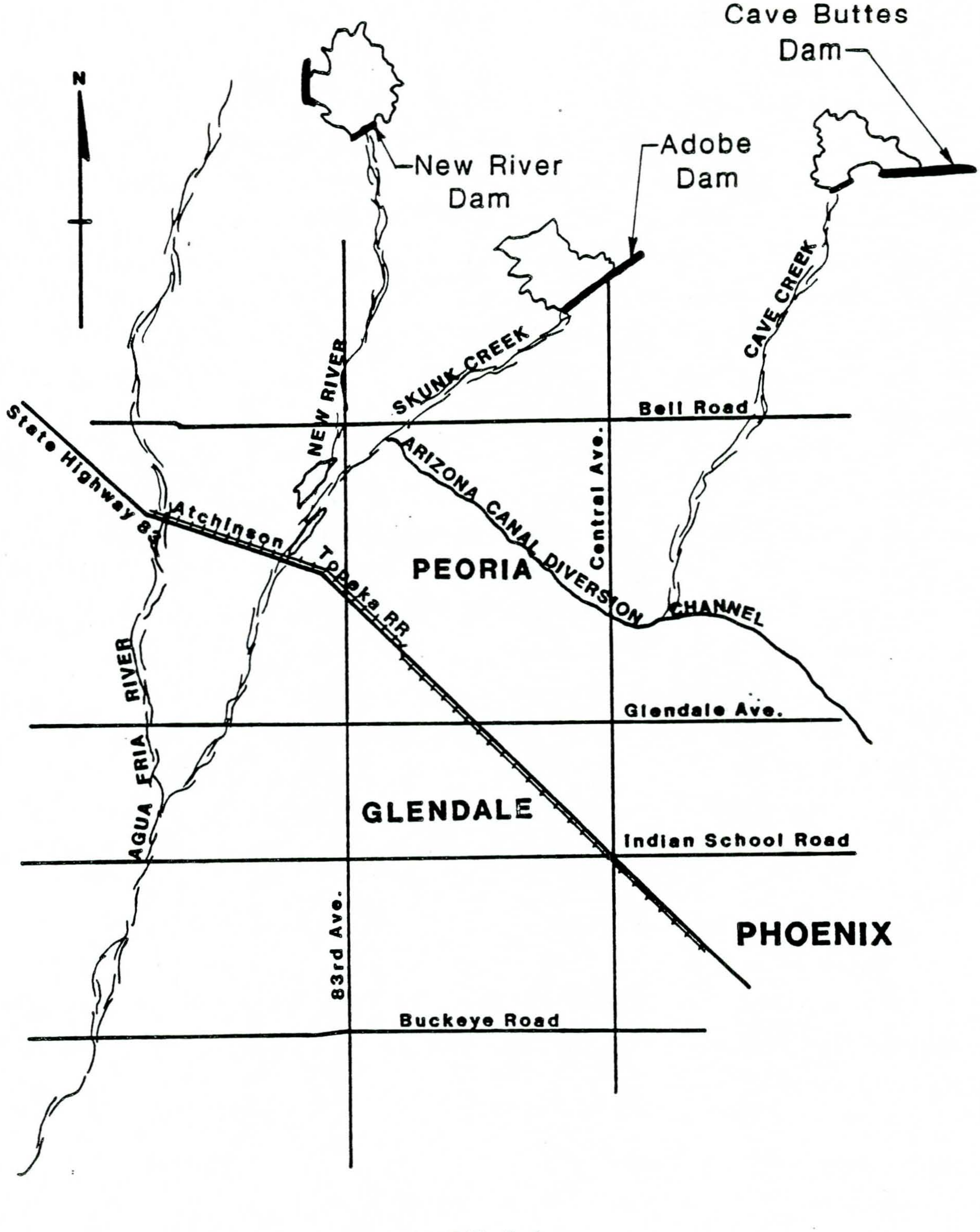
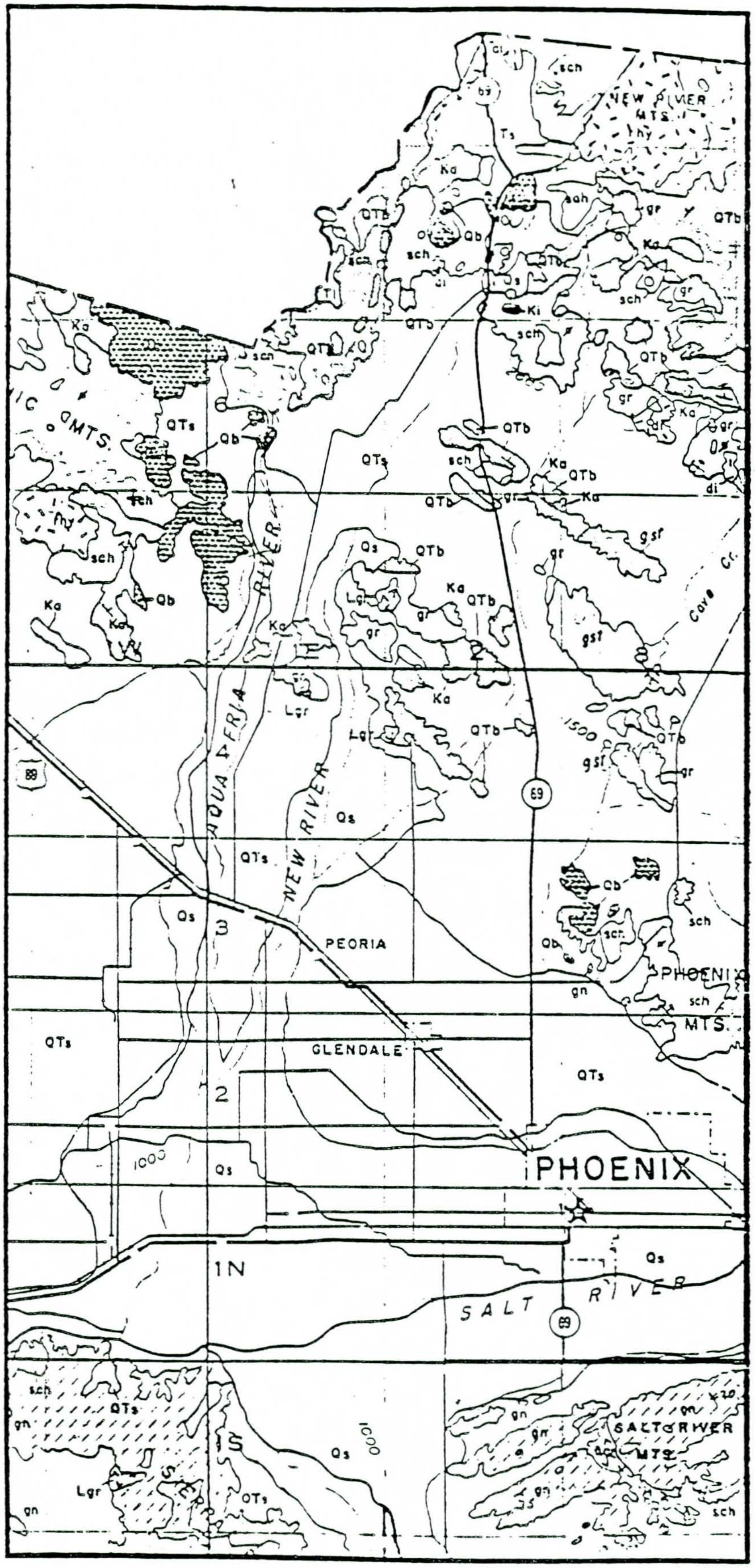


FIGURE 2.1
LOCATION MAP

alluvial slopes. The general geology and physiography of the New River and its watershed are illustrated in Figure 2.2.

In the mountainous regions the basement complex is composed predominantly of Precambian Schistose and massive metaigneous rocks with lesser amounts of gneiss and quartzite. These are covered and intruded by tertiary igneous rocks including granite, Andesite, Red Rock Rhyolite, and other related crystalline rocks. Alluvium fills the valleys and covers the slopes of the hills and mountains. The older alluvium which consists of moderately to well consolidated residual and talus debris is generally found along the side slopes of the valleys and underlies the recent alluvium. The flood plain deposits overlie or are cut into the alluvial valley deposits. These deposits consist of silts, sand, and gravel (unit Qs on Figure 2.2) 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 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 are moderately alkaline and slightly to strongly calcareous.



EXPLANATION

Sedimentary Rocks

Qs	Silt, sand and gravel
QTs	Sand, gravel and conglomerate
Ts	Sand, gravel and conglomerate
Tl	Lake deposits

Igneous Rocks

Qb	Basalt
QTb	Basalt
Ki	Dikes and plugs
Ka	Andesite
gr	Granite and related crystalline rocks
di	Diorite porphyry
sch	Schist
rhy	Red Rock ryholite
gs	Greenstone
gn	Granite gneiss

SCALE

0 5

miles

Contour interval: 500 feet m.s.l.

FIGURE 2.2
GEOLOGIC MAP (FROM WILSON et al., 1957)

III. HYDROLOGY

3.1 General Characteristics

Similiar to other watercourses in the Phoenix area, the New River and Skunk Creek are ephemeral streams. Runoff most often occurs during and following relatively heavy precipitation. Mean annual precipitation in the Phoenix area ranges from 7 inches in the desert areas to over 22 inches in the mountains. Precipitation generally occurs nearly equally in two distinct seasons, summer (June through September) and winter (December through March). Three basic storm types produce precipitation. General Winter storms are normally low intensity, long duration events covering a large aerial extent. Orographic effects are significant with mountains receiving as much as four to ten times the precipitation as desert areas. Precipitation in mountains above 6,000 feet generally occurs as snow. General summer storms typically consist of numerous locally heavy storm cells embedded in more widespread, generally light to moderate rain. The aerial extent and duration are usually less than general winter storms, but intensities may be higher. Similar to general winter storms, orographic effects can be significant. Local storms consist of heavy downpours of rain over relatively small areas for short time periods. Although they may occur at any time of year, they are most prevalent during the summer (July to September).

Runoff characteristics of general winter and general summer storms are similar, although because summer infiltration rates are typically higher, summer runoff volumes are often lower. Local storm runoff typically consists of a high peak and a low runoff volume which can result in serious flash floods.

3.2 Flood History

Runoff records are available at four locations on the New River and at one location on Skunk Creek. Table 3.1 gives the U.S. Geological Survey (USGS) gage number, drainage area, period of record and maximum discharge. Figure 3.1 illustrates the gaging locations.

Gaging station data and historical accounts indicate that damaging floods have occurred in the Gila River basin. Table 3.2 identifies those floods that have occurred from general storms and those from local storms. Information on many of these floods is detailed in COE General Design memorandums (COE, 1974 and COE, 1982); however, available information for floods prior to establish-

Table 3.1. Stream Gages in the Study Region.

Stream Gage Name	USGS No.	Drainage Area (sq. miles)	Period of Record	Maximum Discharge (cfs)	Date of Maximum Discharge
New River near Rock Spring, AZ	09513780	67.3	1962 to present	18,600	Sept. 5, 1970
New River at New River, AZ	09513800	85.7	1960 to present	19,500	Sept. 5, 1970
New River at Bell Road, near Peoria, AZ	09513835	187.0	1965 to present	14,600	Dec. 19, 1967
New River near Glendale, AZ	09513910	323.0	1961 to present	19,800	Dec. 19, 1967
Skunk Creek near Phoenix, AZ	09513860	64.6	1960 to present	11,500	Aug. 1, 1964

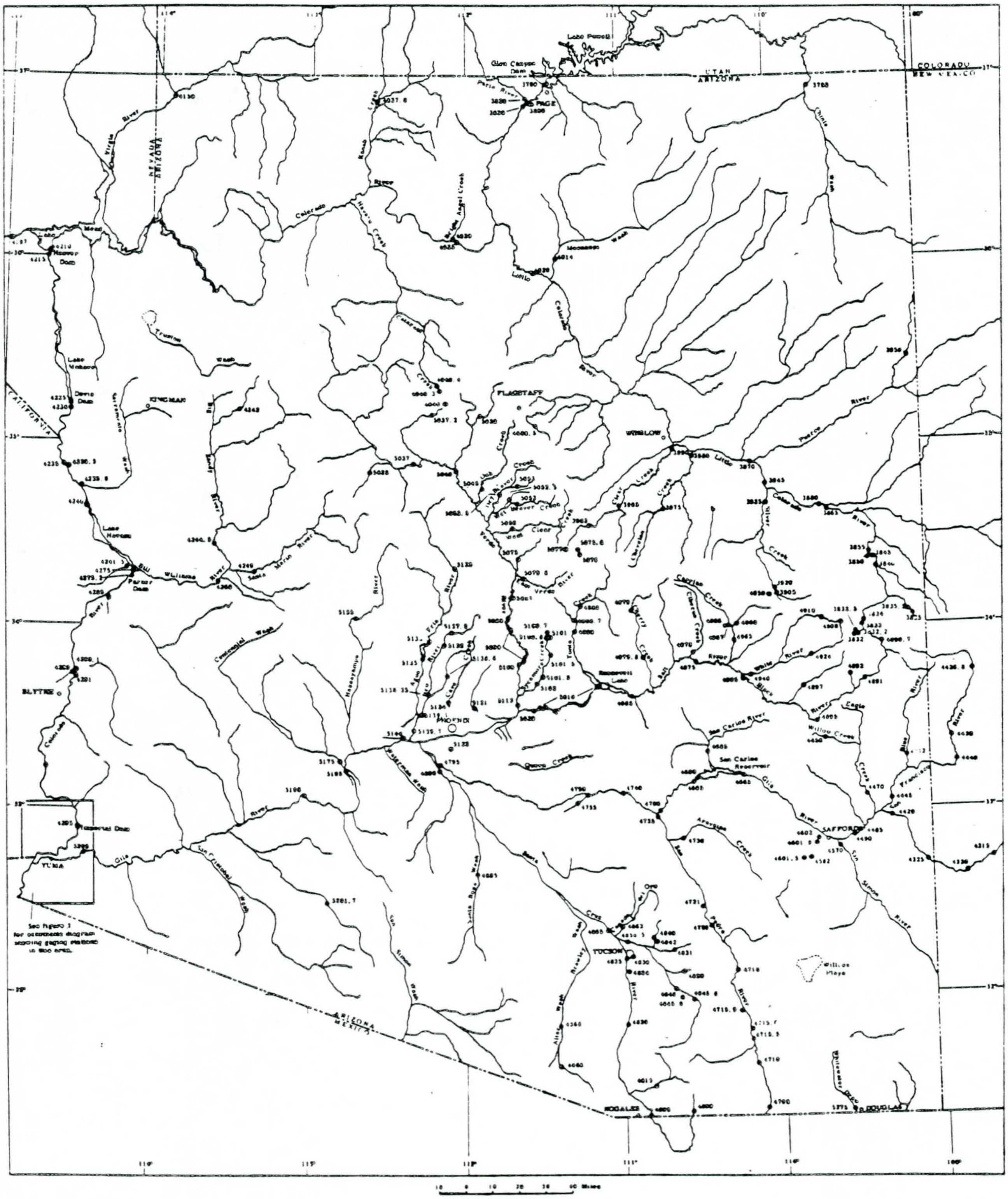


FIGURE 3.1
MAP OF ARIZONA SHOWING ACTIVE GAGING STATIONS

Table 3.2. Historical Floods in Study Region.

Floods in Gila River Basin Resulting From General Storms	Floods in Phoenix Area Resulting From Local Storms
February, 1884	1921
February, 1891	1935
January, 1916	1936
February-March, 1938	1939
	1943
	1951
	1955
	1956
	1957
	1963
	1964
	1967
	1969
	1970
	1972

ment of the stream gaging network (1960) was based primarily on historical accounts and only limited information was provided for the New River and Skunk Creek.

Similiar to other channels in the Gila River basin, flood peaks in the New River attenuate in the downstream direction. Factors causing this attenuation include 1) channel storage losses; 2) large infiltration losses; and 3) insignificant lateral inflow.

3.3 Flood Peak Information

Flood peak information for various return periods was provided by the COE. These flows were used for the hydraulic and sedimentation analysis and are presented in Table 3.3. The peak flows derived in the study were based on the following dams and drainage channels being operational.

1. New River dam on the New River.
2. Adobe dam on Skunk Creek.
3. Arizona Canal Diversion Channel.
4. Cave Buttes dam on Cave Creek.
5. Dreamy Draw dam on Dreamy Draw Wash.

Figure 3.2 shows the existing and proposed dams and flood control channels that were considered in the hydrologic analysis.

3.4 Flood Hydrographs

The shapes of the 100-year hydrographs for various locations along the study reach are presented in Figures 3.3a through 3.3f. These hydrographs were constructed based on the storm of August, 1951. On the New River and Skunk Creek above ACDC, the hydrographs have a duration of approximately ten hours with the severe portion of the flood lasting about two to three hours. On the New River downstream of Skunk Creek, the hydrographs have a duration of approximately 15 hours with the severe portion of the flood lasting about five hours. Hydrographs for the 50-, 25-, and 10-year floods were obtained by using a direct ratio of the peak flows $\left(\frac{Qpi}{Q\ 100}\right)$ to obtain the hydrograph ordinates for the more frequent floods.

It should be noted that the major portion of the inflow to New River from Skunk Creek is contributed by the ACDC.

Table 3.3. Design Flood Discharge-New River and Skunk Creek from Bell Road to Agua Fria River.

Location	PEAK DISCHARGE (cfs)			
	100-year Flood	50-year Flood	25-year Flood	10-year Flood
ACDC U/S of Confluence With Skunk Creek	29,000	20,000	13,500	7,700
Skunk Creek U/S of Confluence With ACDC	13,000	9,100	6,000	3,400
Skunk Creek D/S of Confluence With ACDC	35,000	25,000	17,000	9,200
New River U/S of Confluence With Skunk Creek	19,000	14,000	5,900	5,100
New River D/S of Confluence With Skunk Creek	41,000	29,000	19,000	10,500
New River Near Confluence With Agua Fria River	39,000	27,000	18,000	10,800

SOURCE: U.S. Army Corps of Engineer, Los Angeles. 4/12/84.

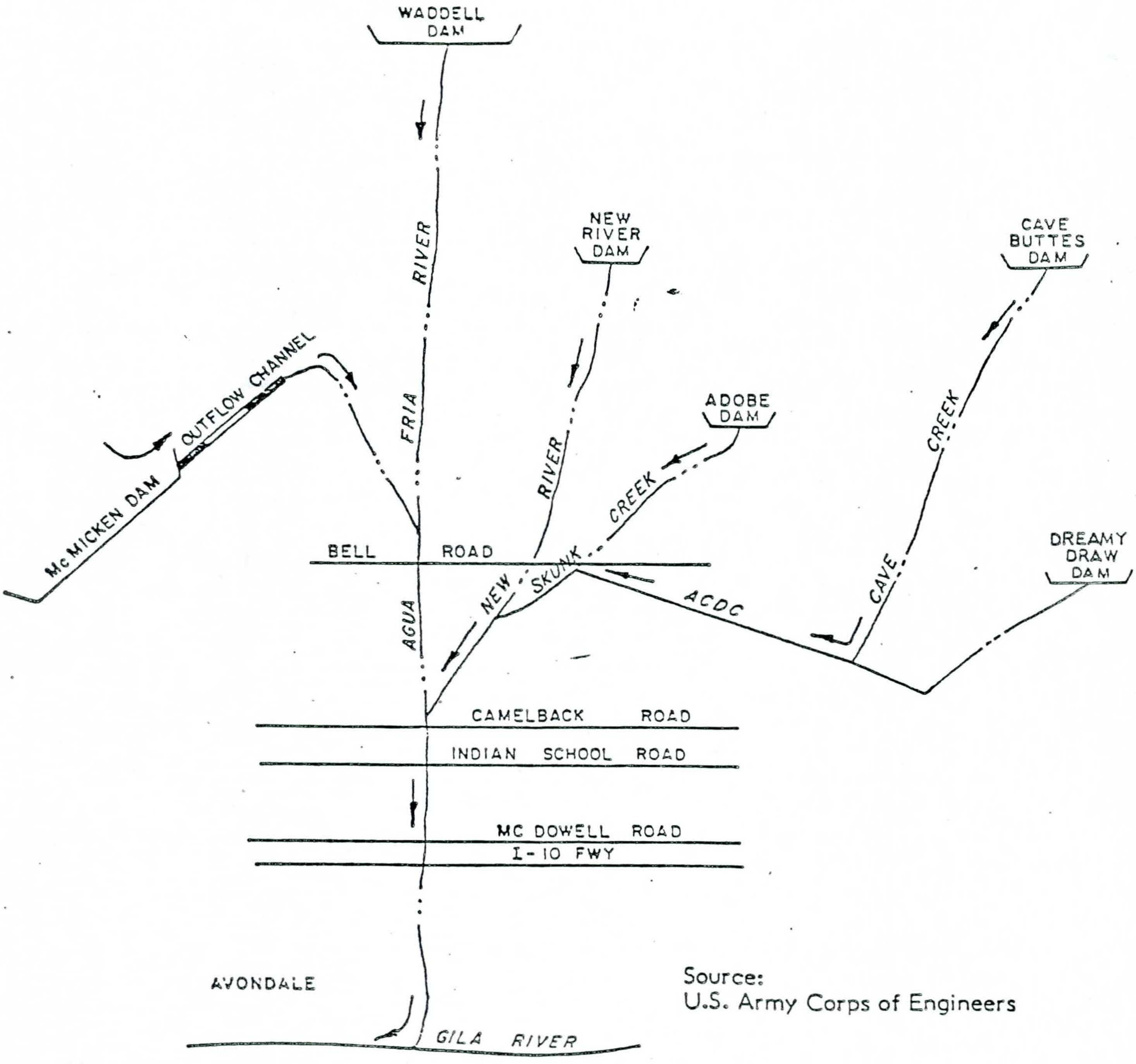


FIGURE 3.2
EXISTING AND PROPOSED DAMS
AND FLOOD CONTROL CHANNELS

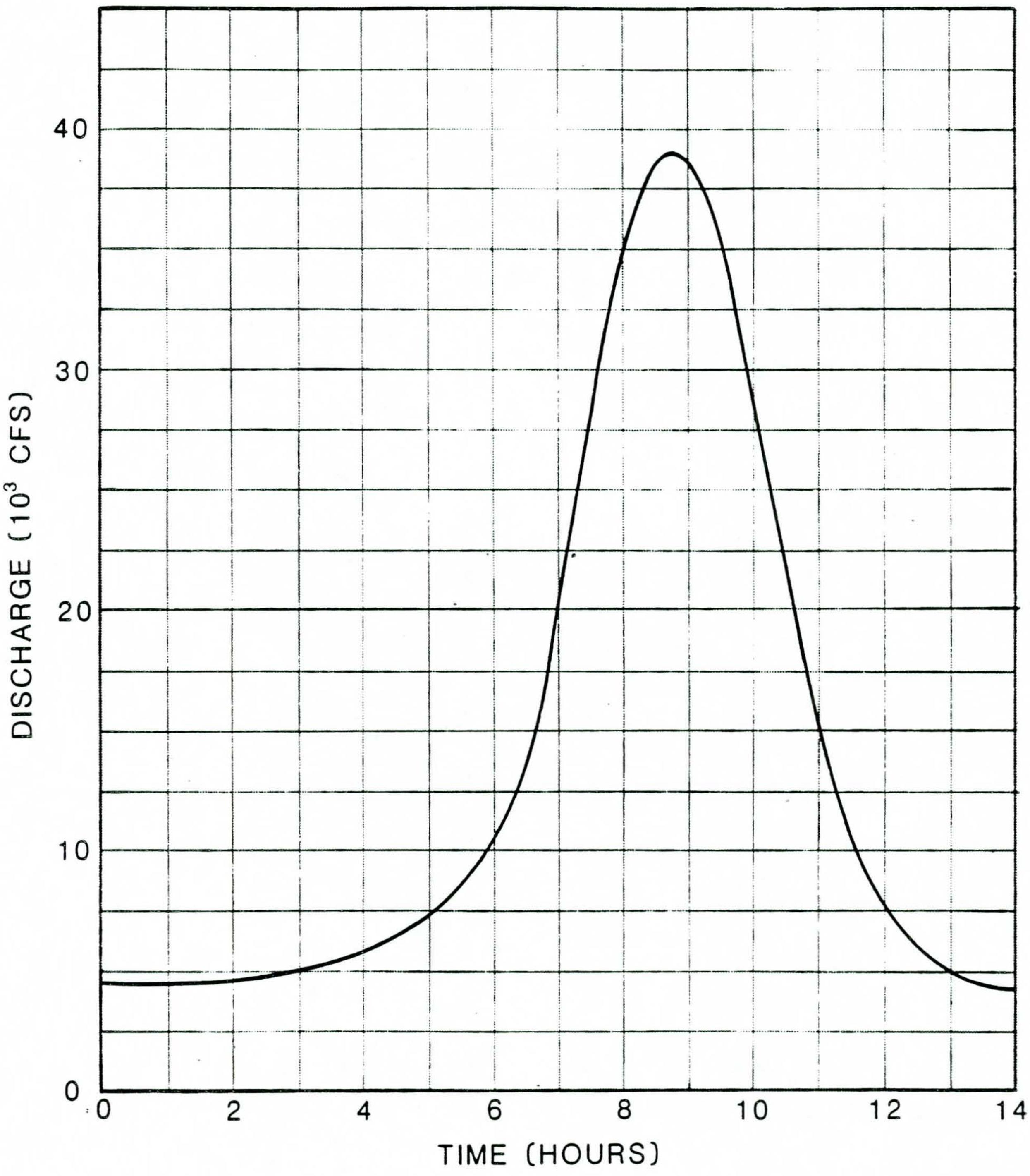


FIGURE 3.3a
100-YR FLOOD HYDROGRAPH AT NEW RIVER U/S OF AGUA FRIA RIVER

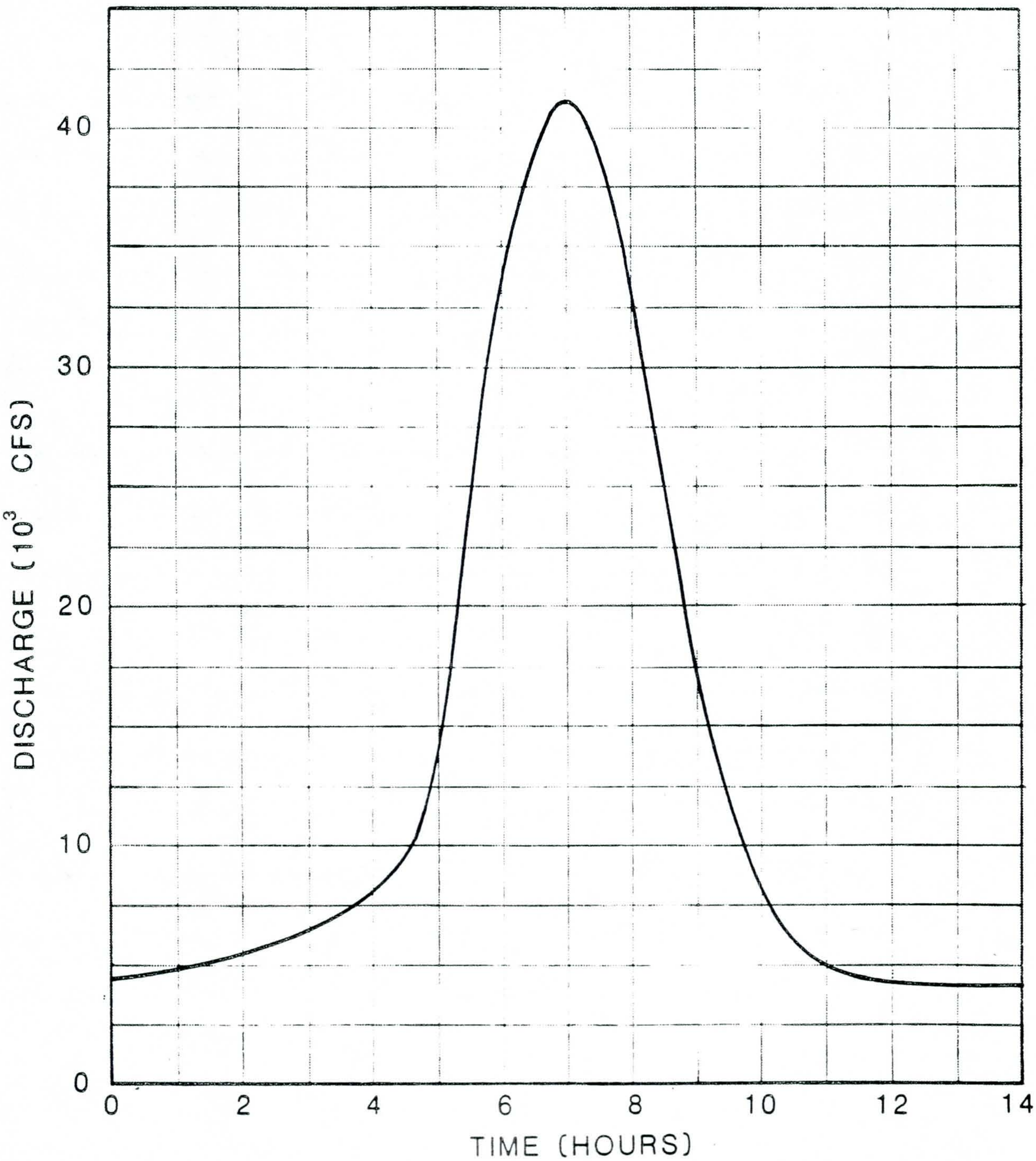


FIGURE 3.3b
100-YR FLOOD HYDROGRAPH AT NEW RIVER D/S OF SKUNK CREEK

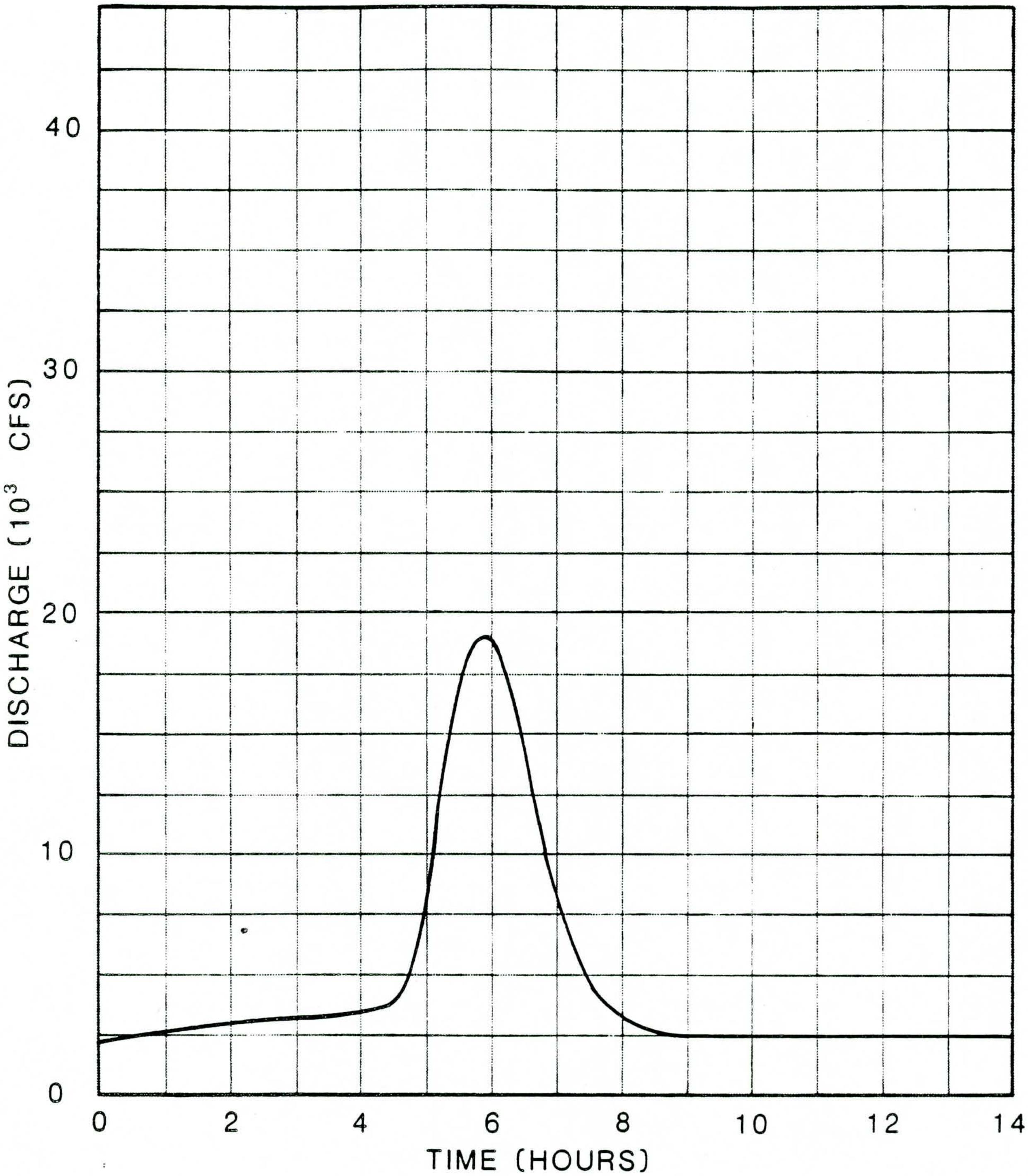


FIGURE 3.3c
100-YR FLOOD HYDROGRAPH AT NEW RIVER U/S OF SKUNK CREEK

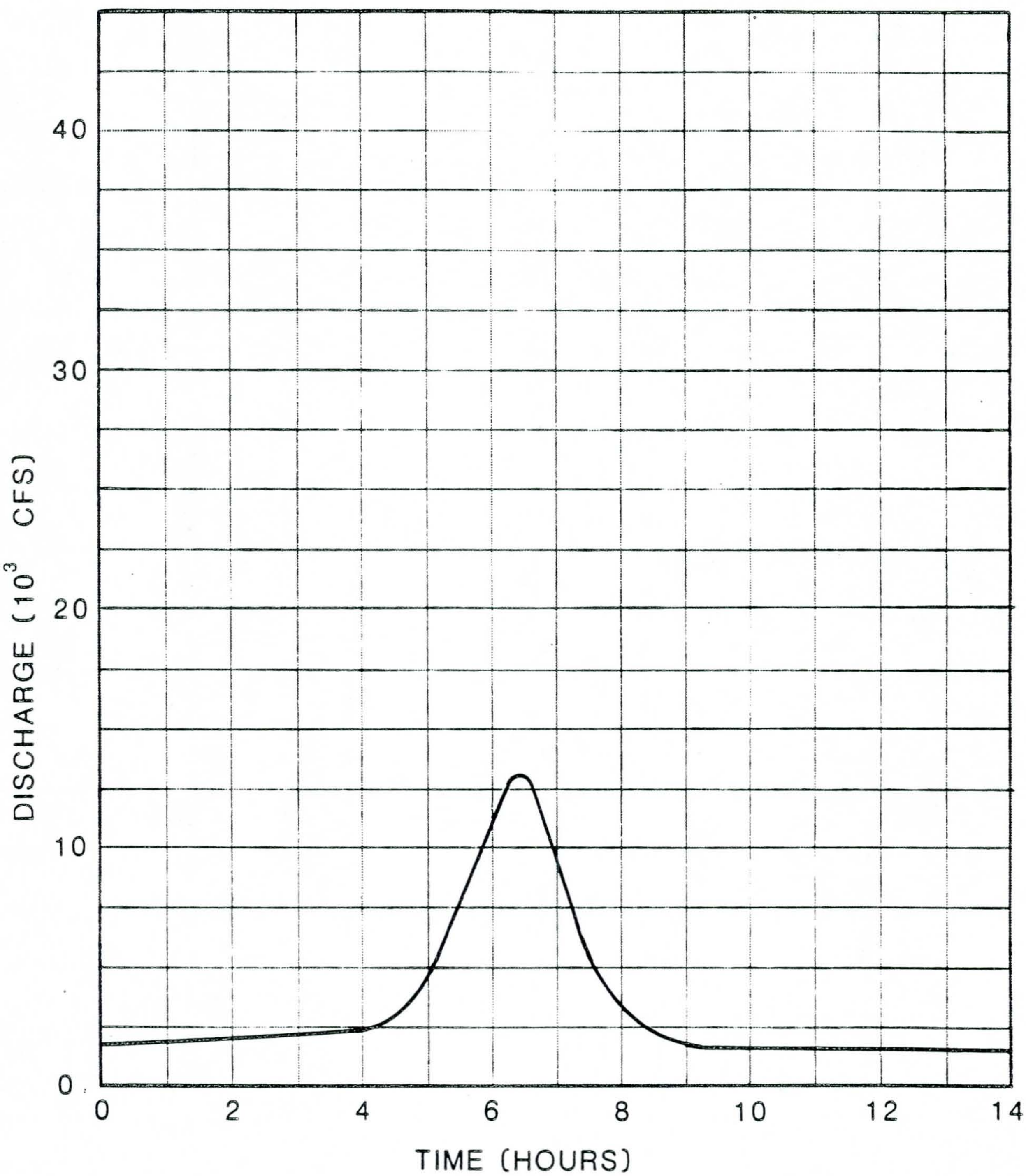


FIGURE 3.3d
100-YR FLOOD HYDROGRAPH AT SKUNK CREEK
U/S OF ARIZONA CANAL DIVERSION CHANNEL

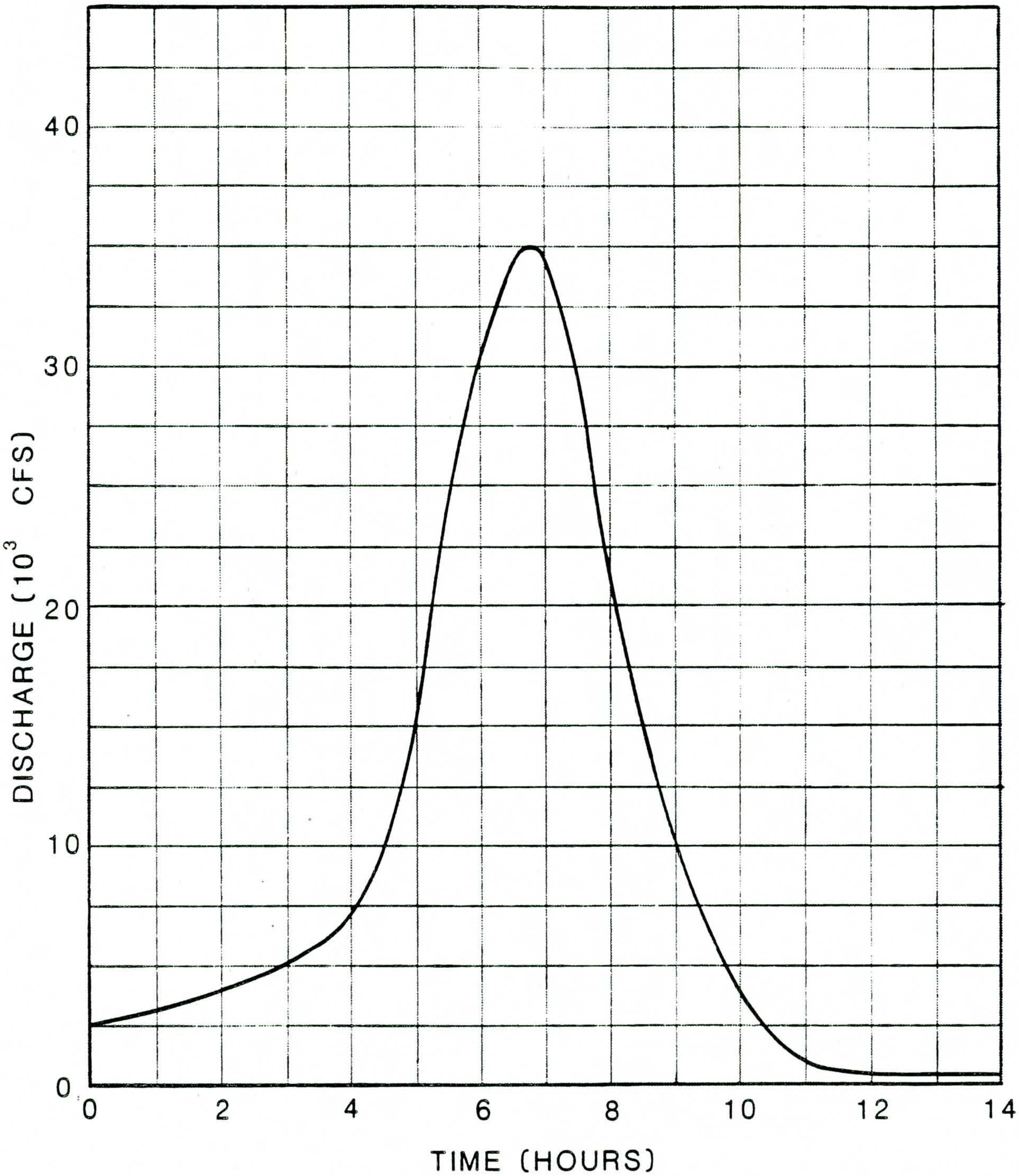


FIGURE 3.3e
100-YR FLOOD HYDROGRAPH AT SKUNK CREEK
D/S OF ARIZONA CANAL DIVERSION CHANNEL

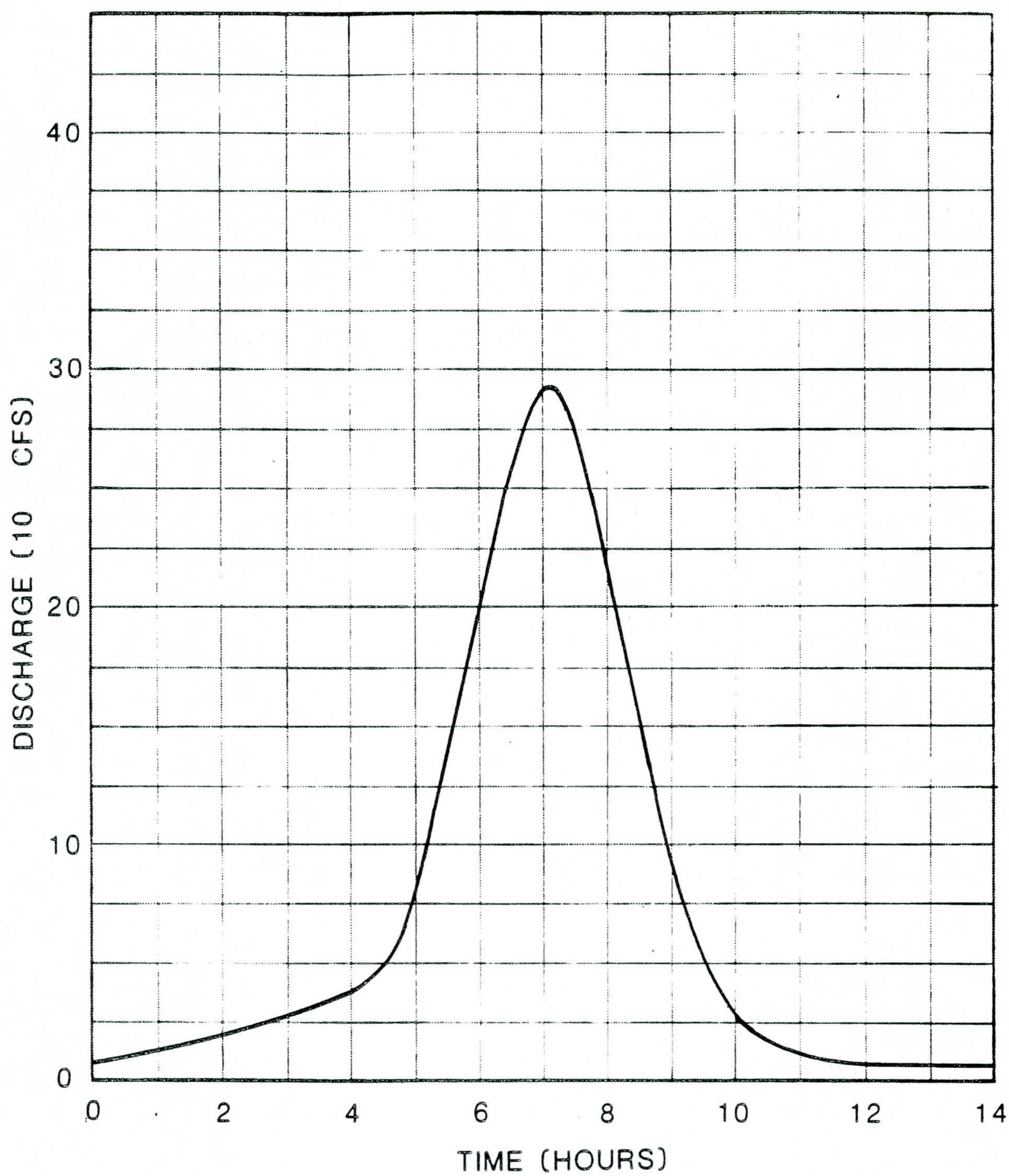


FIGURE 3.3f
100-YR FLOOD HYDROGRAPH AT ARIZONA CANAL
DIVERSION CHANNEL U/S OF SKUNK CREEK

IV. HYDRAULICS

4.1 General

Hydraulic characteristics of the New River and Skunk Creek between Bell Road and the confluence with the Agua Fria River were evaluated for the 10-, 25-, 50-, and 100-year flood peaks using the COE HEC-2 backwater profile program. Hydraulic variables such as flow velocity, flow depth, top width, and main channel and overbank discharge were used to describe the flow characteristics.

The cross-sectional data used for backwater profile computation was obtained from the 1980, 1981, and 1983 topographic maps. For the New River, from Bell Road to the confluence with the Skunk Creek, the 1980 topographic map was used; and from the confluence with the Skunk Creek to the confluence with the Agua Fria River, the 1981 topographic map was used as a base, with the September, 1983 topographic map updating the main channel portion of each cross section for selected reaches. For the Skunk Creek the 1981 topographic map was used. Figure 4.1 identifies the topographic mapping available at different locations in the study reach. A total of 156 cross sections were used to compute backwater profiles. Locations of cross sections are shown in the plates attached to this report.

4.2 Application of HEC-2

4.2.1 Manning Roughness Coefficient

The project scope of work and objectives required hydraulic information for both analysis of flooding problems and for analysis of sediment transport characteristics. To address flooding problems, a Manning roughness coefficient (Manning n) of 0.035 was selected for the main channel, since larger n values produce higher stages and therefore conservative analysis of flood conditions. In contrast, an n value of 0.025 was selected for sediment transport analyses since lower n values produce higher velocities and therefore more conservative sediment transport results. For both applications, floodplain roughness coefficients varied from 0.045 to 0.060, with most of the floodplain area having a value of 0.045.

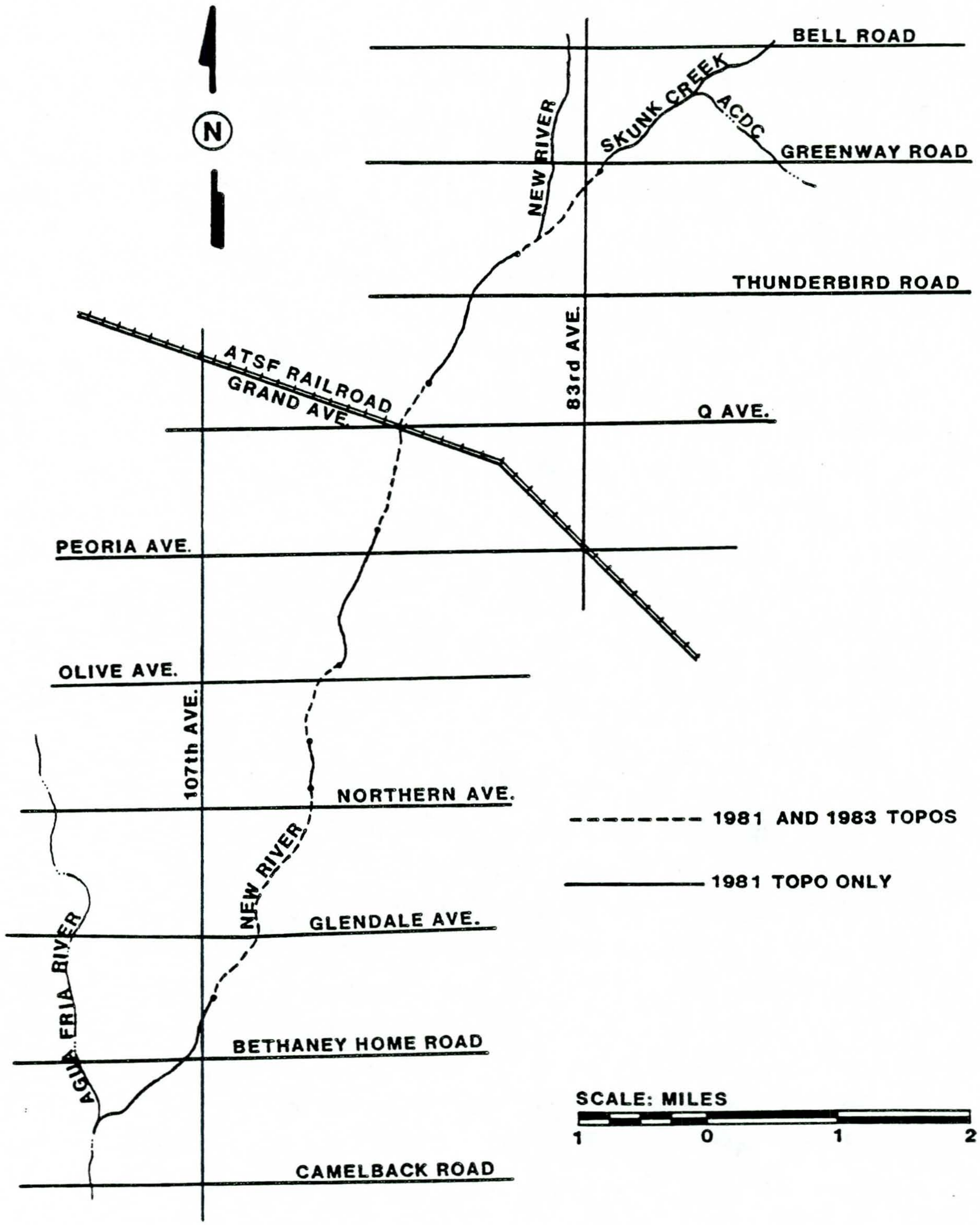


FIGURE 4.1
AVAILABLE TOPOGRAPHIC MAPS FOR
NEW RIVER AND SKUNK CREEK

4.2.2 Bridge Crossing Data

Bridge crossings in the study reach of the new River include Bell Road, Thunderbird Road, Atchison, Topeka, and Santa Fe Railroads, Grand Avenue, Peoria Avenue, Olive Avenue, and Glendale Avenue. Bridge crossings in the study reach of Skunk Creek include Bell Road and 83rd Avenue. Table 4.1 summarizes for each bridge the pier diameter or width, the length the pier extends across the bridges, the bottom elevation of the piers, the September 1983 thalweg elevation, the approximate skew angle (angle at which flow attacks the bridge piers), and the low-chord elevation of the bridge.

4.3 Discussion of Results

4.3.1 Analysis of Flooding Problems

Analysis of flooding problems is based on HEC-2 results using an n value of 0.035. The 100-year flood plain is plotted and shown in the 1" = 500' scale topographic maps that supplement this report. Also shown in the maps are the 10-year and 100-year water-surface profiles and the thalweg profile. Flow depths vary from 6.5 feet to 22.7 feet, main channel velocities range from 3.3 feet-per-second to 18.3 feet-per-second and top widths range from 250 feet to 5,000 feet for the 100-year flood peak.

Within the study reach, various problems such as pressure flow at bridges, flow breakouts, inundation of urbanized areas, overtopping of existing levees, etc., are clearly shown in the HEC-2 analysis. These problems are discussed in the following paragraphs.

Just downstream of the confluence of the Skunk Creek with the ACDC on the north overbank, flow breakout will occur for the 100- and 50-year flood events. Approximately 11,000 cfs and 4,000 cfs will leave the main channel at the 100- and 50-year flood peaks, respectively. Once water breaks out, it flows over agricultural fields on the north (right) floodplain and eventually joins the main channel flow near 83rd Avenue. Some flooding on the south (left) bank immediately downstream of the confluence is also expected.

The 83rd Avenue bridge on Skunk Creek has limited conveyance capacity. For the 100- and 50-year flood events, water will overtop the bridge during flood peaks. The backwater pool formed upstream of the bridge decreases flow velocity significantly and causes flow breakout on the left bank.

Table 4.1. Pertinent Data of Existing Bridges.

Location	Pier Width or Diameter	Pier Length	Bottom of Pier Footing	Thalweg Elevation	Skew of Piers to Flow Direction	Low Chord
New River						
Bell Road	1.2'	27.5'	1186.3'	1188.6'	10°	1205.5'
Thunderbird Road	3'	91.4'	1090.5'	1135.6'	-	1155.3'
ATSF Railroad	6'	15'	1113.0'	1120'	11°	1134.9'
Grand Avenue	5'	34'	1114.5'	1119.5'	15°	1134.8'
Peoria Avenue	2'	77'	1089.8'	1094.3'	2°	1112.8'
Olive Avenue	2'	*	1057.5'	1079.9'	10°	1088.9'
Glendale Avenue	1.3'	52.5'	1043.8'	1047.0'	7°	1062.4'
Skunk Creek						
Bell Road	1.5'	50'	1198.0'	1208'	3°	1218.0'
83rd Avenue	1.5'	70'	1156.0'	1162'	3°	1178.0'

*Circular Piers

The breakout flow is estimated to be approximat
flood peak and possibly returns to the main cha

Extensive flooding can be expected for bot
plains immediately upstream of Thunderbird Road
event. The flooding problem on the left overba
breakout flow from upstream of the 83rd Avenue
on the right overbank upstream of Thunderbird R
with water, by extending the flood boundary con

Downstream of Thunderbird Road the water b
the 100-year flood event. Pressure flow occurs
Grand Avenue bridge, but overtopping does not c
mainly over agricultural fields.

Immediately downstream of the Grand Avenue
left and right banks and flows through an urbar
50-year flood peaks. This water returns to the
1,500 feet downstream of the Grand Avenue brid
pit.

For the reach between Peoria Avenue and O
vely well-confined in the main channel for the
exception is the left overbank area immediatel
where the flood plain extends approximately 1,
by the bridge. Pressure and weir flow occurs
during 100-year flood peak.

Downstream of a short, narrow reach locat
Northern Avenue, flow breaks out heavily on th
the left overbank for the 100- and 50-year flc
the agricultural fields and a gravel mining ar
wide as 2,500 feet.

A significant amount of water overtops th
Home Road and the confluence with the Agua Fr
flood events. Approximately 7,000 cfs and 2,
bank during the 100- and 50-year peaks. This
Agua Fria River near Camelback Road. The flo
feet wide at the 100-year flood peak near the
the Agua Fria River.

Table 4.2 summarizes the available freeboard at all river crossings in the study reach for the 100-year flood. Pressure flow occurs at the Atchison, Topeka and Santa Fe Railroad bridge, Grand Avenue bridge and Olive Avenue bridge on the New River and 83rd Avenue bridge on the Skunk Creek. The Thunderbird Road Bridge, Peoria Avenue Bridge and Bell Road Bridge on Skunk Creek have less than three feet of freeboard for the 100-year flood peak.

4.3.2 Analysis for Sediment Transport Application

Hydraulic information for use in sediment transport analysis is presented in this section, based on HEC-2 results using a main channel Manning n of 0.025. For many sediment transport analyses it is beneficial to delineate the study reach into a limited number of subreaches. Delineation of the channel into subreaches is based on consideration of (1) physical characteristics of the channel such as top width and slope, (2) hydraulic parameters, particularly velocity, (3) sediment characteristics, (4) areas of interest, and (5) the desire to maintain reach lengths as uniform as possible throughout the system. Since the majority of flow in the New River below the confluence with Skunk Creek results from the ACDC, subreaches were defined consecutively beginning in Skunk Creek and continuing in the New River. The New River reach above the Skunk Creek confluence was considered separately. For the former case, 12 subreaches were established, as illustrated by Figure 4.2. Table 4.3 defines subreach boundaries. For the New River above Skunk Creek the short distance to the study boundary eliminates the necessity to define any subreaches.

Average flow velocities, effective widths, hydraulic depths and discharges for the 10-, 25-, 50-, and 100-year floods for each of the 13 subreaches (12 on Skunk Creek and New River below the confluence and one on New River above the confluence) are summarized in Tables 4.4 to 4.7.

Table 4.2. Summary of Freeboard Available at Bridge for 100-year flood.

Crossing	Low Chord Elevation (ft)	100-year Water Surface (ft)	Freeboard Height (ft)
New River			
Bell Road	1205.5	1200.3	5.2
Thunderbird Road	1155.3	1153.2	2.1
ATSF Railroad	1134.9	1137.6	*
Grand Avenue	1134.8	1136.0	*
Peoria Avenue	1112.8	1110.8	2.0
Olive Avenue	1088.9	1092.5	*
Glendale Avenue	1062.4	1058.4	4.0
Skunk Creek			
Bell Road	1218.0	1216.1	1.9
83rd Avenue	1178.0	1184.2	*

* pressure and weir flow.

Figure 4.2. Subreach delineation.

Location	Cross Section Number	Reach Number	Reach Length (ft)
Agua Fria River	501.45	12	5,695
	13.00		
	20.00		
	26.80		
	32.60		
	38.00		
	45.00		
	51.70		
New River at	54.00		
Bethany Home Road	59.00		
	63.00		
	70.50		
	77.00		
	84.00		
	88.70	11	6,091
	95.50		
	100.70		
New River at	107.00		
Glendale Avenue	117.20		
	118.00		
	120.00		
	125.00		
	131.40		
	135.00		
	140.00		
	143.00		
	146.00	10	6,221
	149.60		
	158.00		
	167.00		
	170.00		
New River at	174.10		
Northern Avenue	178.50		
	181.60		
	185.20		
	190.00		
	194.60		
	198.40		
	202.40		
	206.00	9	5,172
	210.50		
	216.00		
New River at	218.00		
99th Avenue	220.80		
	226.00		
New River at	229.00		
Olive Avenue	231.50		

Figure 4.2. Subreach delineation (continued).

Location	Cross Section Number	Reach Number	Reach Length (ft)
New River at Olive Avenue	232.60		
	235.00		
	238.80		
	240.80		
	245.80		
	251.00		
	256.00		
	258.80	8	5,820
	262.00		
	265.00		
	270.50		
	273.50		
	277.00		
	281.80		
New River at	288.00		
Peoria Avenue	290.00		
	291.00		
	293.00		
	297.00		
	302.00		
	305.00		
	307.00		
	311.50		
	314.50		
	318.00		
	323.00		
	330.00	7	7,033
	334.00		
	337.00		
	340.50		
	342.00		
New River at	343.00		
Grand Avenue	345.00		
	346.50		
New River at	346.90		
ATSF Railroad	347.00		
	349.00		
	352.00		
	358.00		

Figure 4.2. Subreach delineation (continued).

Location	Cross Section Number	Reach Number	Reach Length (ft)
	365.00		
	370.00		
	373.00		
	377.00		
	384.00		
	388.00		
	391.50		
	395.00	6	6,371
	402.00		
	406.00		
	409.00		
New River at	409.82		
Thunderbird Road	410.00		
	412.00		
	418.00		
	423.00		
	428.00		
	434.00		
	440.00	5	2,450
	443.20		
Confluence of Skunk Creek	449.40		
with New River	0.00		
	4.50		
	10.00		
	15.00	4	2,434
	19.00		
	21.00		
Skunk Creek at	23.00		
83rd Avenue	24.00		
	26.00		
	28.00		
	30.00	3	2,856
	35.00		
	41.00		
	45.00		
	48.00		

Figure 4.2. Subreach delineation (continued).

Location	Cross Section Number	Reach Number	Reach Length (ft)
	55.00		
	60.00		
	65.00		
	71.50	2	3,430
	75.00		
	78.00		
Confluence of Skunk Creek	84.00		
with ACDC	86.60		
	91.90		
	97.20		
	102.60		
	107.80		
	113.10		
Skunk Creek at	118.50	1	3,836
Bell Road	120.60		
	121.80		
	122.60		
	127.90		

Table 4.3. Subreach Definition.

Reach	Description
1.	Skunk Creek, from Bell Road to the confluence with ACDC.
2.	Skunk Creek, from the confluence with ACDC to 3,400 feet downstream of the confluence.
3.	Skunk Creek, from 3,400 feet downstream of the confluence with ACDC to 2,400 feet upstream of the confluence with the New River.
4.	Skunk Creek, from 2,400 feet upstream of the confluence with the New River to the confluence with the New River.
5.	New River, from the confluence with the Skunk Creek to 2,450 feet downstream of the confluence.
6.	New River, from 2,450 downstream of the confluence with the Skunk Creek to 1,450 feet upstream of the ATSF Railroad Bridge.
7.	New River, from 1,450 feet upstream of the ATSF Railroad Bridge to Peoria Avenue.
8.	New River, from Peoria Avenue to Olive Avenue.
9.	New River, from Olive Avenue to Northern Avenue.
10.	New River, from Northern Avenue to Glendale Avenue.
11.	New River, from Glendale Avenue to Bethany Home Road.
12.	New River, from Bethany Home Road to the confluence with the Agua Fria River.

Table 4.4. Average Flow Velocity, Hydraulic Depth, Effective Width and Discharge for the 10-year Flood Event.

	Left Flood Plain				Main Channel				Right Flood Plain			
Reach	Vel. (ft/sec)	Hyd. Depth (ft)	Eff. Width (ft)	Water Disc. (cfs)	Vel. (ft/sec)	Hyd. Depth (ft)	Eff. Width (ft)	Water Disc. (cfs)	Vel. (ft/sec)	Hyd. Depth (ft)	Eff. Width (ft)	Water Disc (cfs)
1	0.00	0.00	0	0	8.34	3.06	133	3400	0.00	0.00	0	0
2	0.00	0.00	0	0	10.40	5.87	147	8950	0.00	0.00	0	0
3	0.00	0.00	0	0	8.50	5.93	183	9200	0.00	0.00	0	0
4	0.00	0.00	0	0	7.03	5.62	232	9160	0.28	1.11	125	40
5	0.00	0.00	0	0	10.39	4.75	212	10500	0.00	0.00	0	0
6	0.00	0.00	0	0	7.98	4.86	271	10500	0.00	0.00	0	0
7	0.00	0.00	0	0	9.00	3.83	304	10500	0.00	0.00	0	0
8	0.00	0.00	0	0	8.07	4.36	292	10260	0.00	0.00	0	0
9	0.00	0.00	0	0	6.76	4.38	347	10250	0.00	0.00	0	0
10	0.00	0.00	0	0	8.22	4.11	311	10500	0.00	0.00	0	0
11	0.00	0.00	0	0	6.64	4.08	399	10800	0.00	0.00	0	0
12	0.65	0.30	269	50	6.15	3.67	476	10750	0.00	0.00	0	0
NRAS*	0.00	0.00	0	0	6.75	2.27	332	5100	0.00	0.00	0	0

*NRAS = New River Above Skunk Creek. See Table 4.3 for reach definition.

Table 4.5. Average Flow Velocity, Hydraulic Depth, Effective Width and Discharge for the 25-year Flood Event.

Reach	Left Flood Plain				Main Channel				Right Flood Plain			
	Vel (ft/sec)	Hyd. Depth (ft)	Eff. Width (ft)	Water Disc (cfs)	Vel (ft/sec)	Hyd. Depth (ft)	Eff. Width (ft)	Water Disc. (cfs)	Vel (ft/sec)	Hyd. Depth (ft)	Eff. Width (ft)	Water Disc (cfs)
1	0.00	0.00	0	0	9.03	4.42	150	6000	0.00	0.00	0	0
2	0.00	0.00	0	0	12.22	7.95	170	16520	0.00	0.00	0	0
3	0.00	0.00	0	0	9.67	8.07	218	17000	0.00	0.00	0	0
4	0.00	0.00	0	0	8.16	7.71	253	15950	1.16	2.56	353	1050
5	0.00	0.00	0	0	11.27	6.75	244	18550	1.45	1.23	211	380
6	0.00	0.00	0	0	9.47	6.58	305	19000	0.10	0.06	6	0
7	0.00	0.00	0	0	10.60	5.46	329	19000	0.05	0.04	7	0
8	0.00	0.00	0	0	9.44	6.22	315	18520	0.00	0.00	0	0
9	0.00	0.00	0	0	7.98	6.17	376	18500	0.00	0.00	0	0
10	0.00	0.00	0	0	9.66	5.36	352	18260	0.00	0.00	0	0
11	0.00	0.00	0	0	7.89	5.26	434	18000	0.00	0.00	0	0
12	1.15	0.55	282	180	7.63	4.55	510	17720	0.73	0.50	0	100
NRAS*	0.18	0.12	25	1	7.31	2.91	419	8895	0.07	0.25	154	4

*NRAS = New River above Skunk Creek. See Table 4.3 for reach definition.

Table 4.6. Average Flow Velocity, Hydraulic Depth, Effective Width and Discharge for the 50-year Flood Event.

	Left Flood Plain				Main Channel				Right Flood Plain			
Reach	Vel. (ft/sec)	Hyd. Depth (ft)	Eff. Width (ft)	Water Disc. (cfs)	Vel. (ft/sec)	Hyd. Depth (ft)	Eff. Width (ft)	Water Disc. (cfs)	Vel. (ft/sec)	Hyd. Depth (ft)	Eff. Width (ft)	Water Disc. (cfs)
1	0.00	0.00	0	0	9.28	5.72	171	9100	0.00	0.00	0	0
2	1.21	0.61	16	10	12.86	9.71	189	23580	1.48	1.02	472	710
3	1.03	1.03	11	10	9.82	9.58	250	23550	1.75	1.53	539	1440
4	0.56	0.50	151	40	9.31	8.67	263	21270	1.77	3.44	605	2690
5	1.31	1.30	391	670	10.77	9.41	249	25190	2.29	2.09	626	3000
6	0.00	0.00	0	0	10.51	8.02	344	28990	0.39	0.26	94	10
7	0.49	0.30	75	10	11.52	7.23	348	28990	0.00	0.00	0	0
8	0.48	0.48	27	10	10.61	8.07	331	28310	0.00	0.00	0	0
9	0.41	0.32	131	20	9.03	7.84	400	28280	0.00	0.00	0	0
10	0.69	0.41	664	190	10.62	7.09	364	27450	0.79	0.49	151	60
11	0.00	0.00	0	0	8.85	6.32	482	27000	0.00	0.00	0	0
12	1.52	0.56	701	600	8.87	4.67	617	25530	1.77	1.02	485	870
NRAS*	0.42	0.23	60	5	8.26	3.49	485	13975	0.22	0.46	187	20

*NRAS = New River above Skunk Creek. See Table 4.3 for reach definition.

Table 4.7. Average Flow Velocity, Hydraulic Depth, Effective Width and Discharge for the 100-year Flood Event.

	Left Flood Plain				Main Channel				Right Flood Plain			
Reach	Vel. (ft/sec)	Hyd. Depth (ft)	Eff. Width (ft)	Water Disc. (cfs)	Vel. (ft/sec)	Hyd. Depth (ft)	Eff. Width (ft)	Water Disc. (cfs)	Vel. (ft/sec)	Hyd. Depth (ft)	Eff. Width (ft)	Water Disc. (cfs)
1	0.57	0.34	268	50	9.56	7.33	185	12940	0.16	0.29	168	10
2	1.27	1.23	21	30	12.98	11.58	193	29060	2.60	2.35	803	4910
3	1.42	1.97	117	330	9.23	10.43	276	26550	2.75	2.95	1000	8120
4	1.87	2.66	238	1180	9.98	9.85	263	25880	2.25	4.92	718	7940
5	1.98	2.35	432	2010	11.55	11.12	254	32610	2.72	3.20	708	6160
6	0.51	0.40	323	70	11.14	9.68	378	40790	0.83	0.78	220	140
7	0.62	1.02	255	160	11.43	9.76	364	40640	0.73	1.10	251	200
8	0.82	0.86	43	30	11.69	10.02	341	40000	0.00	0.00	0	0
9	0.90	0.76	234	160	10.27	9.34	415	39830	0.46	0.41	44	10
10	1.55	1.24	1243	2390	10.98	8.51	391	36590	1.62	1.27	270	560
11	0.80	0.47	199	80	9.90	7.59	517	38910	0.36	0.36	72	10
12	2.12	1.21	781	2000	9.77	5.48	653	34970	2.33	1.51	576	2030
NRAS*	0.77	0.42	121	40	8.68	3.49	491	18895	0.37	0.68	253	65

*NRAS = New River above Skunk Creek. See Table 4.3 for reach definition.

V. QUALITATIVE GEOMORPHIC ANALYSIS

5.1 General

Qualitative geomorphic analysis employed in Level I relies strongly on expertise and practical experience. Geomorphology is the study of surficial features of the earth and the physical and chemical processes of changing land forms, while fluvial geomorphology is the geomorphology (and mechanics) of watershed and river systems. A qualitative Level I analysis provides insight into complicated fluvial response mechanisms and gives understanding and direction to the Level II and III quantitative analyses.

Qualitative geomorphic techniques are primarily based on a well-founded understanding of the physical processes governing watershed and river response. Therefore, an important first step is to assemble and review previous work and data applicable to the study area, and for key project participants to become familiar with the study area. A site visit by key personnel ensures identification of important characteristics of the study area. After completing the necessary site visits, there are a number of simplified concepts and procedures that contribute to a qualitative analysis. These include aerial photograph analysis, historical land-use patterns, and relatively simple relationships describing basic geomorphic concepts.

5.2 Site Visit Observations

5.2.1 New River From Bell Road to Skunk Creek Confluence

The New River at Bell Road (upper study reach limit) is a relatively wide, shallow channel as compared to downstream reaches. Bank height is low, typically about 5-10 feet. The bed material is a coarse, gravelly alluvium with a maximum size of about 6 inches. Deposition of 2.5 to 3.0 feet of sediment at the USGS stream gage suggests this reach is aggradational. Plate 5.1 illustrates conditions on New River at Bell Road.

Channelization by Plaza Del Rio and Desert Harbor developers begins in the reach below Thunderbird Avenue and extends upstream past the Skunk Creek confluence. This channelization was completed after September, 1983 and therefore, was not considered in the analyses of this report. However, for purposes of completeness and accurate discussion of site visit observations, it is mentioned in this section. Significant excavation has occurred throughout the channelized section and the resulting channel is now relatively wide and deep with shallow sides (3 or 4 on 1). Channel conveyance in this

PLATE 5.1

PLATE 5.2

reach appears relatively high; however, the lack of bank protection may result in channel stability problems.

5.2.2. Skunk Creek From Bell Road To New River Confluence

At Bell Road the Skunk Creek bed material is relatively fine as compared to armored reaches below the Arizona Canal confluence and near 83rd Street bridge; however, some caliche rock outcrops exist in this area, that may provide some geologic control.

At and below the confluence with the Arizona Canal, the bed material is coarser than material immediately above confluence and an armor layer has developed (Plate 5.2). Channelization below the Arizona Canal has resulted in a channel of relatively high conveyance (larger area, less resistance than upstream or downstream reaches). Plate 5.3 shows the channelization near the sports complex. Bed material conditions in this channelized section are finer than upstream conditions with maximum particle sizes about 3-4 inches.

At 83rd Avenue, the bed is heavily armored with particle sizes as large as 6-8 inches; however, scour is occurring at the piers (Plate 5.4). Pilings are exposed on the downstream side of the second span (from North abutment) and on the upstream side of the third span. Plate 5.4 also illustrates the armoring that has occurred around the bridge.

Above the confluence with New River the Skunk Creek channel is narrow, deep and heavily vegetated (Plate 5.5). Conveyance appears to be less in this reach compared to upstream reaches.

5.2.3 The New River From Skunk Creek Confluence to Olive Avenue

Channelization by the developers in this reach extends downstream about 1,200 feet below Thunderbird Avenue. About five-hundred feet below the confluence an 8-10 foot drop in bed elevation occurs where a grade control structure is planned. If the drop structure is not built before the next significant flood, it is possible that the present abrupt change in grade may initiate an upstream headcut. Similar to upstream reaches the alluvium consists of a coarse, gravelly mixture.

The Thunderbird Road bridge is shown in Plate 5.6. The capacity of the bridge and the conveyance of the channelized reach above and below the bridge suggest that this bridge should be relatively stable.

PLATE 5.3

PLATE 5.4

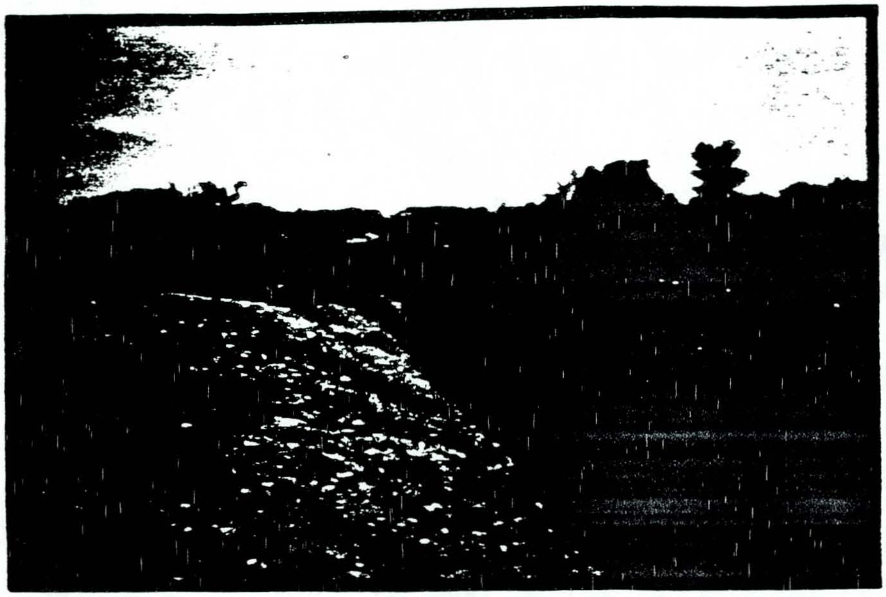


PLATE 5.5

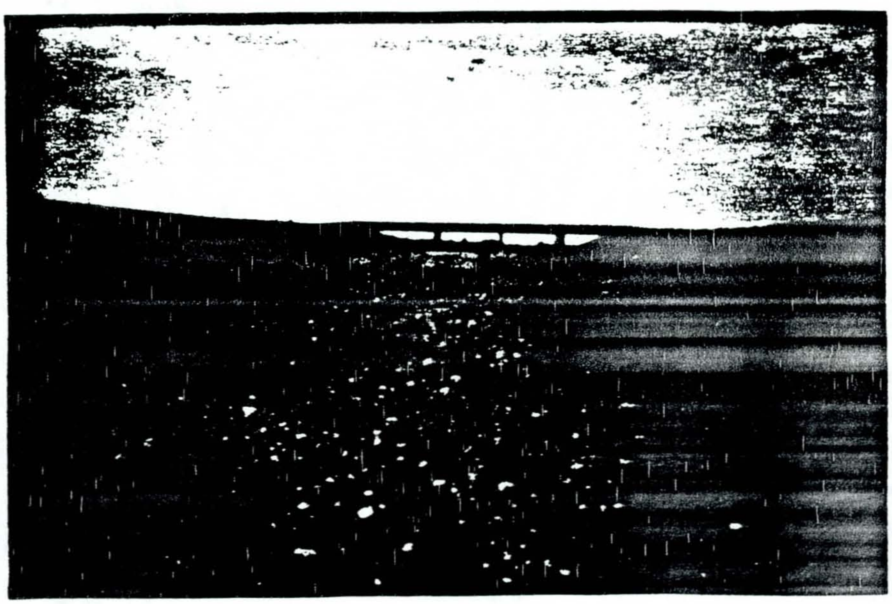


PLATE 5.6

At Grand Avenue (U.S. Highway 89), the Santa Fe and Atchison Topeka railroad also crosses New River. Grand Avenue consists of two bridges, one for each direction; therefore, with the railroad bridge there are a number of piers present in the channel (Plate 5.7). The trestle portion of the railroad bridge (right side looking downstream) is an ineffective flow area due to upstream channel banks blocking flow.

Between Grand Avenue and Peoria Avenue, the channel widens significantly, possibly the result of previous gravel mining activity. Plate 5.8 shows the reduction back to a more normal channel width at the downstream end of this reach. Near Peoria Avenue bridge are several caliche rock outcrops (Plate 5.9). About 50 feet upstream of the bridge a six-inch water line is exposed resulting from a 3 to 4 feet headcut.

5.2.4 New River From Peoria Avenue to Agua Fria Confluence

About 1,000 feet below Peoria Avenue bridge is an instream gravel operation. Plate 5.10 shows the pit, which is presently about 8-10 feet deep traversing the entire channel width. Below the gravel pit, about halfway between Peoria Avenue and Olive Avenue, the right bank has been built up with fill material (Plate 5.11). No bank protection was provided and there is some sloughing, particularly near the Peoria Motor Vehicle Registration and downstream apartments.

At Olive Avenue conveyance through the bridge appears limited by low chord clearance (Plate 5.12). Looking upstream from Olive Avenue, the channel is wide with poorly defined banks (Plate 5.13).

Downstream of Olive Avenue are two dip crossings. Heavy equipment has been working the channel at the 99th Avenue dip crossing, possibly for extraction of material (Plate 5.14). The Northern Avenue dip crossing appears to be acting as a grade control structure. Downstream of the crossing is a 3-4 foot drop in bed elevation (Plate 5.15) while upstream significant amounts of sand have been trapped (Plate 5.16). The sand extends upstream to where the transmission power lines cross the channel. It is possible that the dip crossing was constructed above the equilibrium invert elevation of the bed which has reduced the energy slope promoting deposition of the sand. Conversely, the dip crossing may have been constructed at the invert elevation and the material downstream scoured away or was mechanically removed; however,

PLATE 5.7

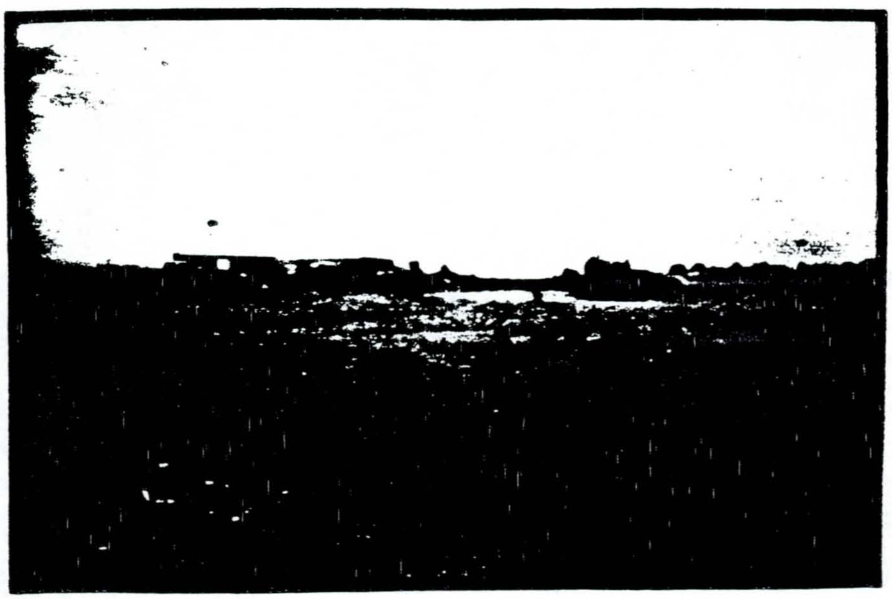


PLATE 5.8

PLATE 5.9

PLATE 5.10

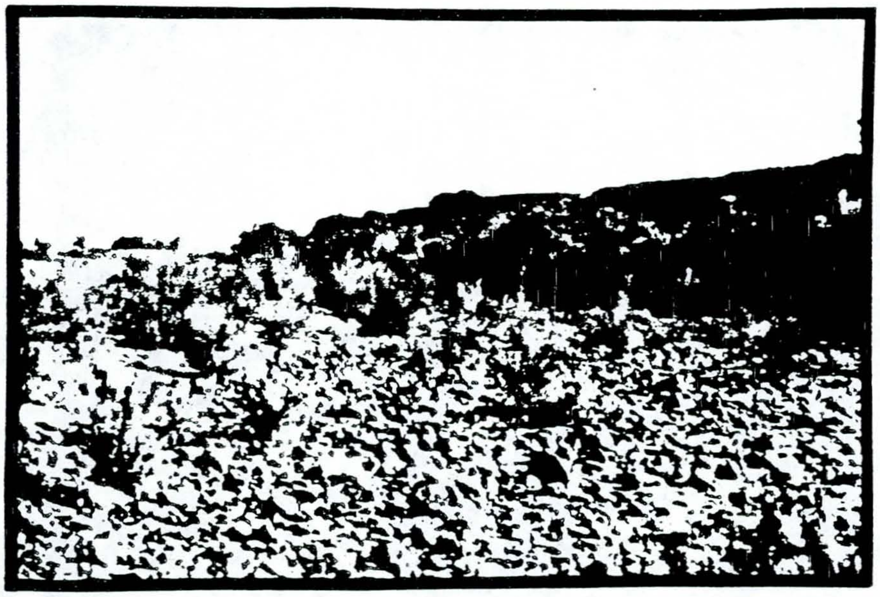


PLATE 5.11

PLATE 5.12

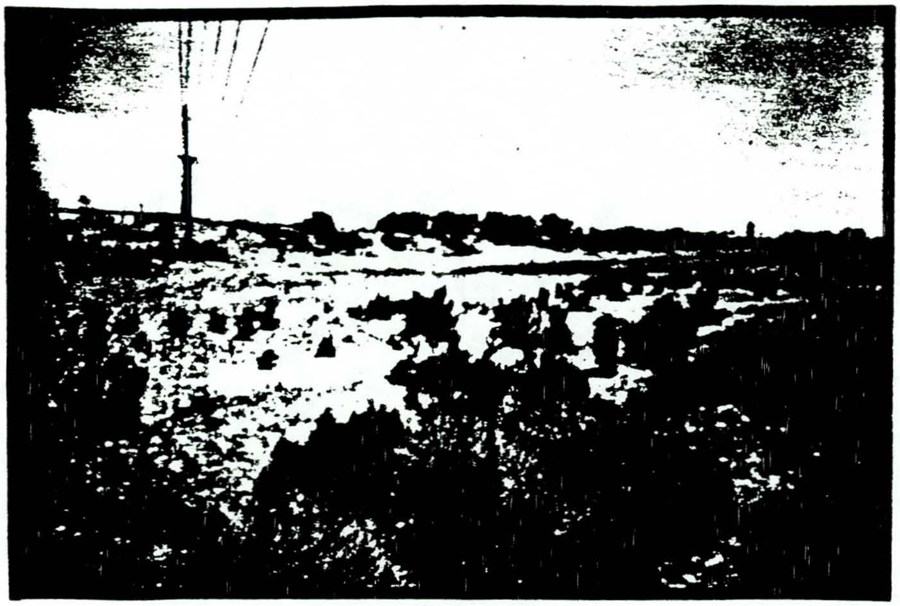


PLATE 5.13

PLATE 5.14

PLATE 5.15

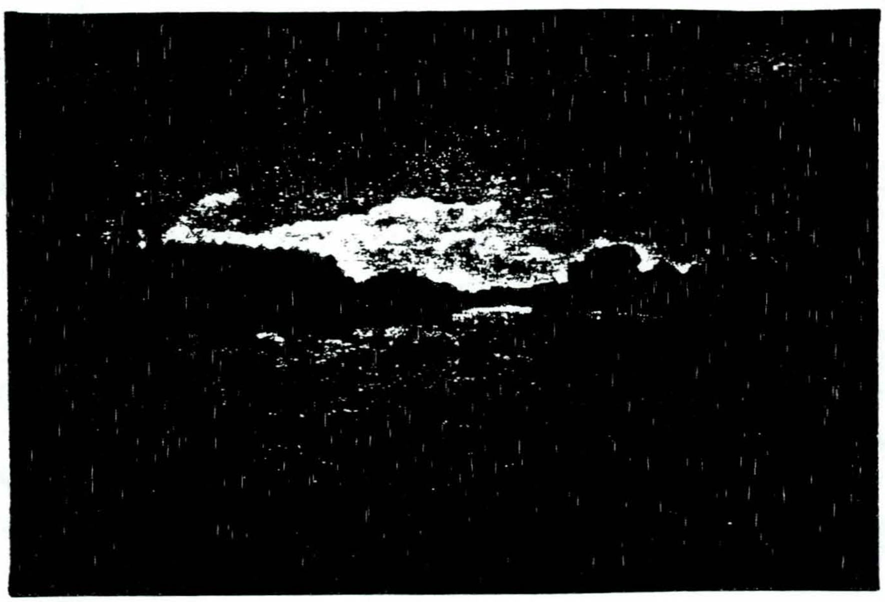


PLATE 5.16

this does not explain the rather unique occurrance of significant sand deposition upstream.

At Glendale Avenue, the bridge and channel appear to have relatively high conveyance (Plate 5.17). Downstream of Glendale Avenue about 500 feet a significant gravel operation is located on the right overbank. Instream gravel pits extend 20 to 30 feet below the channel bed.

From Bethany Home Road downstream to the confluence with the Agua Fria River, sand is more prevalent in the bed material. Patches of gravel and cobble outcrops exist, however, there is an abundance of sand (Plate 5.18). Upstream of Bethany Home Road, the bed material coarsens significantly.

Throughout the study reach a significant amount of vegetation exists between channel banks, especially below Grand Avenue. The vegetation is mostly brush with some cottonwoods. During a large flood it is expected that much of this vegetation may wash out.

5.3 Bed and Bank Material Analysis

5.3.1 Bed Material

Assessment of bed, as well as bank material, is important in evaluating aggradation/degradation trends, and general and local scour. Bed material is the sediment mixture of which the streambed is composed and can vary widely between river systems and even within a given river system, as evidenced by site visit observations on New River. Erodibility or stability of a channel depends on the size of particles in the bed. It is often not sufficient to know just the median bed-material size (D_{50}) in determining the potential for degradation. Knowledge of the bed-material size distribution is also important due to potential for armoring. As water flows over the bed, smaller particles that are more easily transported are carried away, while larger particles remain, armoring the bed. The armoring process is an important concept for understanding alluvial channel response.

To quantify bed material characteristics surface sediment samples were collected by SLA and particle size analysis completed. Due to the generally coarse nature of the alluvium it was necessary to utilize both grid and volumetric sampling and analysis techniques. A total of nine samples were collected, seven on the New River and two on Skunk Creek. Additionally, two samples on Skunk Creek, collected by Boyle Engineering for use in the ACDC study, are available.

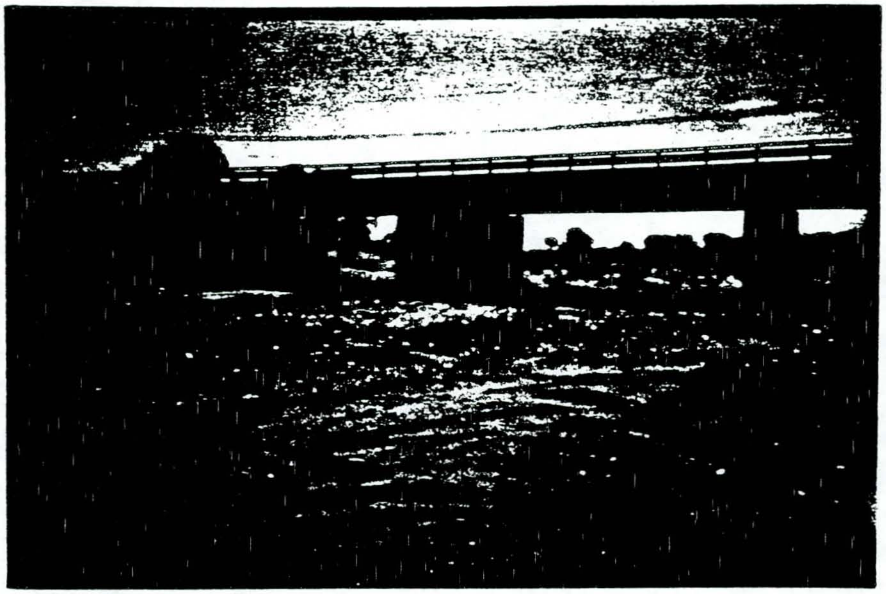


PLATE 5.17

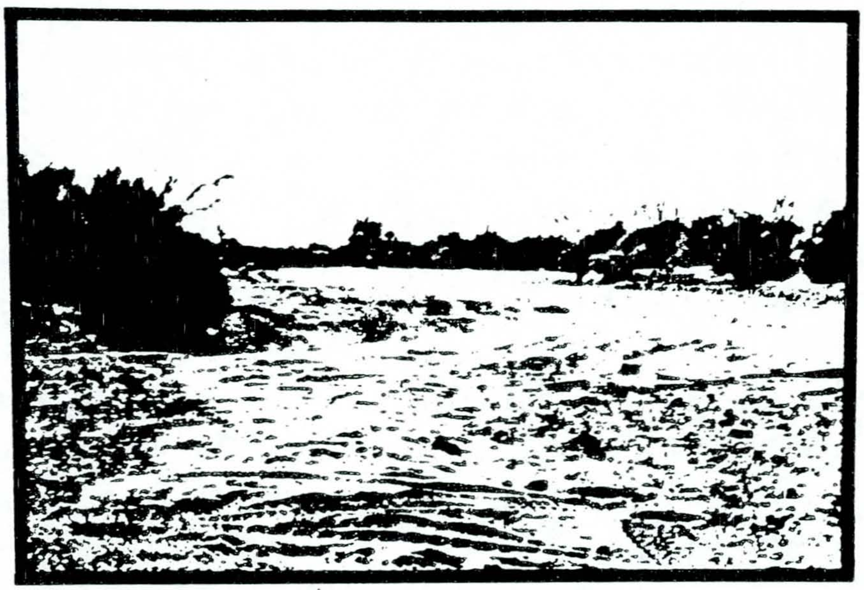


PLATE 5.18

Unlike the Agua Fria River where the bed material is mostly sands with only a trace of gravels and cobbles, the bed material in New River and Skunk Creek is coarser with significant gravels and cobbles present. In general, the bed material is relatively homogeneous throughout the study reach on New River. The only significant variations, as noted in the site visit observations, occur between the confluence with the Agua Fria River and Bethany Home Road, where the bed material characteristics are more similar to Agua Fria conditions, and for a short distance upstream of the Northern Avenue dip crossing where a large quantity of sand exists. In contrast, the Skunk Creek bed material characteristics are more variable, (i.e., changing more frequently), however, the changes are not as dramatic (i.e. not like the New River which changes from predominantly gravel to predominantly sand). The following discussion will illustrate these observations based on sediment sampling results.

Plate 5.19 and 5.20 illustrate bed material characteristics of the New River and Skunk Creek, respectively, at Bell Road (upper study limit). At this location alluvium is coarser in the New River than in Skunk Creek. Moving downstream on Skunk Creek, the material becomes significantly coarser at the Arizona Canal, as was illustrated in Plate 5.2. This material has probably been delivered to the Skunk Creek channel by the canal and as such represents a localized condition that is not indicative of the natural alluvium. This is substantiated by the bed material characteristics between the Arizona Canal and 83rd Avenue in the channelized section of Skunk Creek. Here, the alluvium is again finer, resembling that at Bell Road (Plate 5.21). At the 83rd Avenue bridge, the coarseness of the armor layer is evident in Plate 5.22. The upstream and downstream extent of this armor layer suggests it may have developed from contractual scour resulting from encroachment of the bridge. The lack of a local scour hole around pilings that are presently exposed also suggests contractual scour that has caused degradation throughout the bridge section. Therefore, further scour at the bridge site is unlikely until a flow occurs that is large enough to disrupt the armor layer.

Results of particle size analysis on Skunk Creek are given in Figure 5.1. Additionally, a Boyle Engineering sample is shown on Figure 5.1. The sample at Bell Road is considered representative of the natural alluvium in the Skunk Creek study reach due to the assumed local effects influencing the samples at 83rd Avenue and near the ACDC.

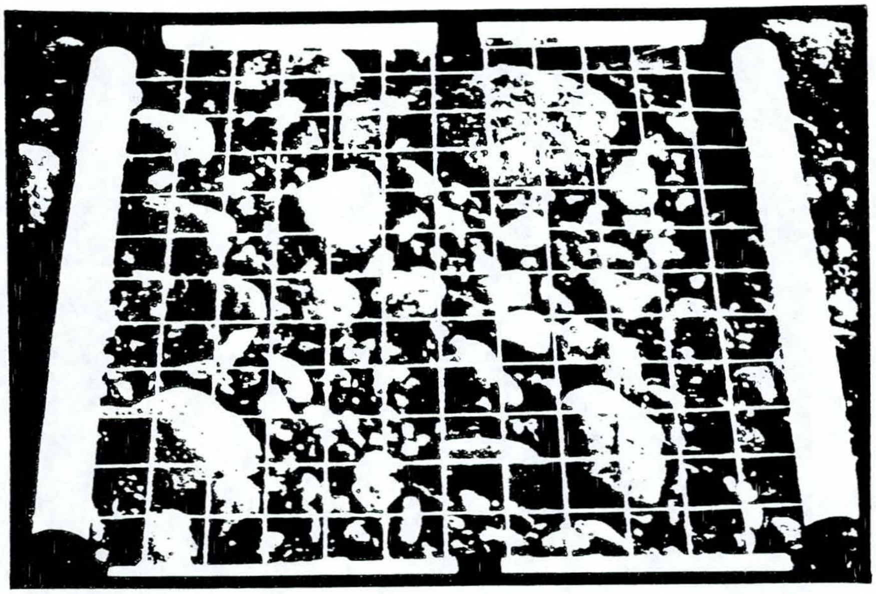


PLATE 5.19

PLATE 5.20

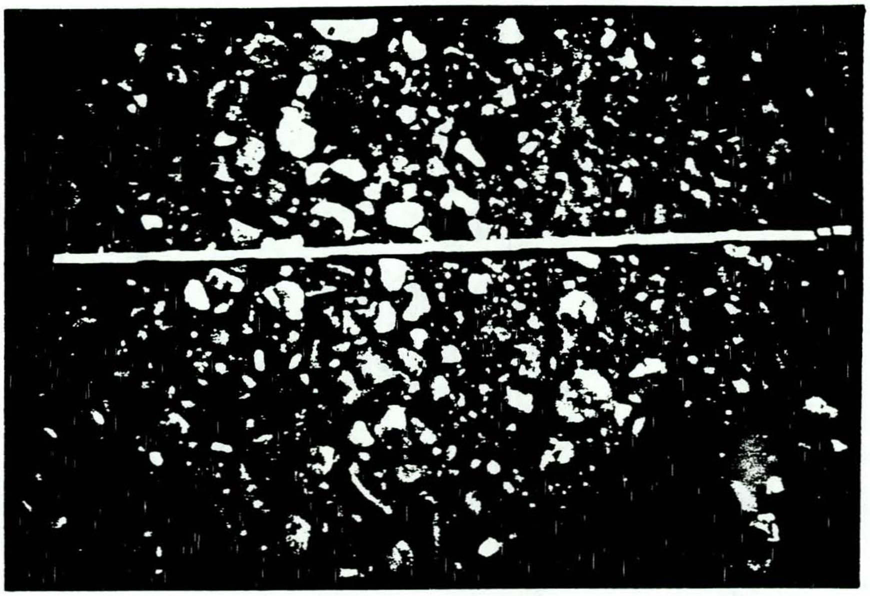


PLATE 5.21

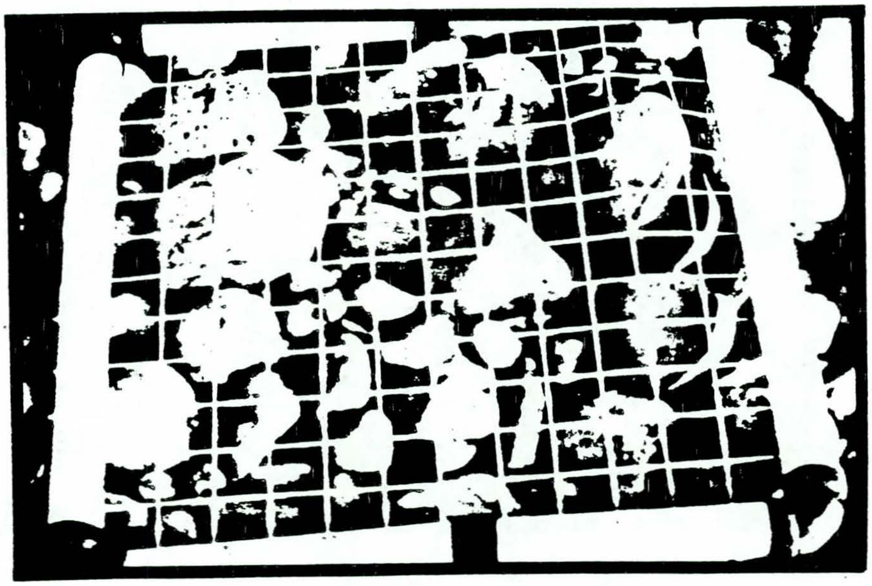


PLATE 5.22

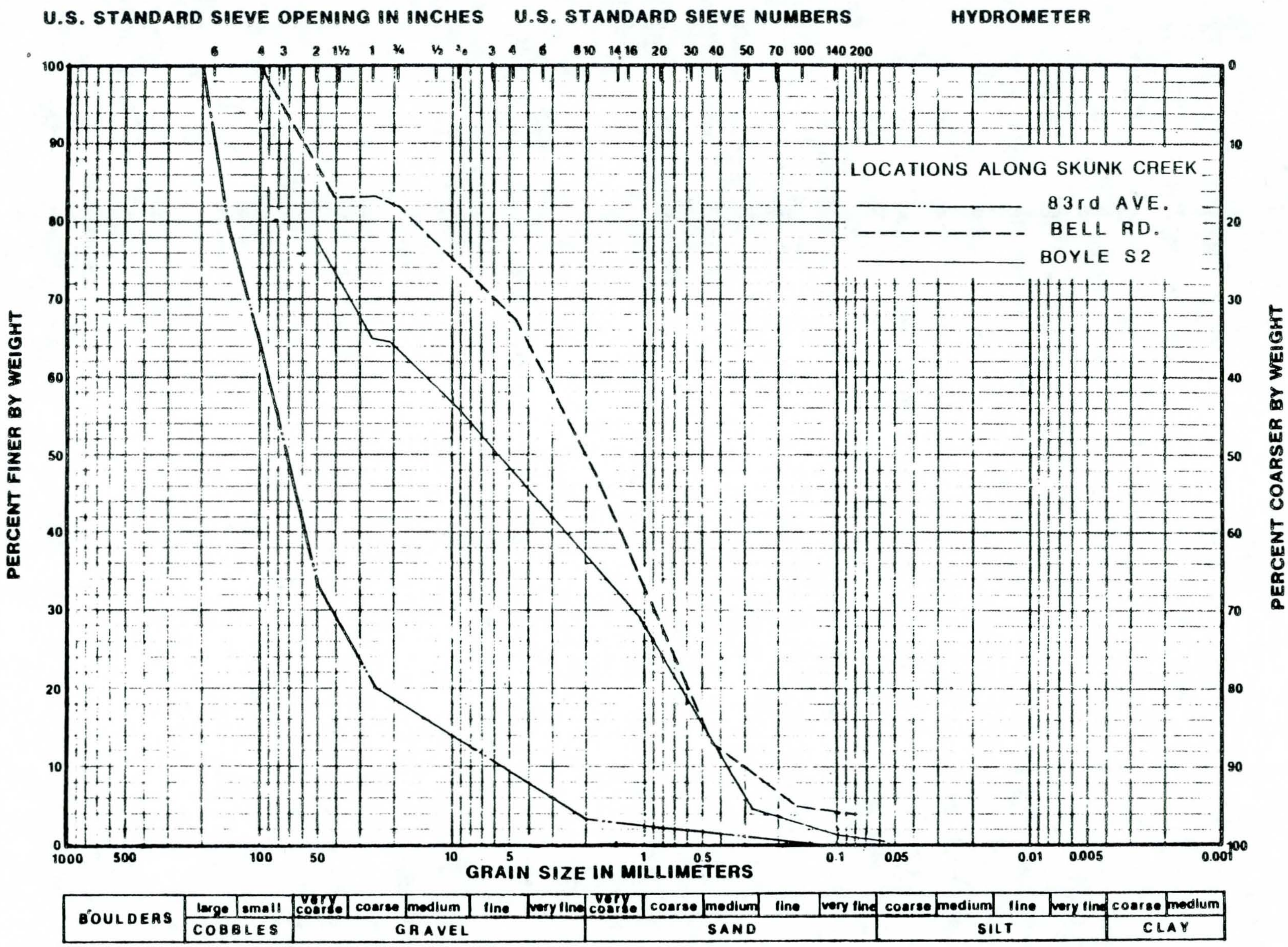


FIGURE 5.1 SKUNK CREEK PARTICLE SIZE GRADATION

On the New River, the bed material becomes finer in the channelized section below the Skunk Creek confluence (Plate 5.23); however, by Grand Avenue the alluvium is again coarse resembling that at Bell Road (Plate 5.24). Except for the sand deposition above Northern Avenue, as was illustrated by Plate 5.16, the bed material remains coarse to Bethany Home Road (Plate 5.25). Then, as observed during site visits and as was depicted by Plate 5.18, the bed material becomes predominantly sand with outcrops of gravels and cobbles.

Results of particle size gradation analysis of samples on New River are given in Figure 5.2. The D_{50} variation with distance is given in Figure 5.3. A representative gradation for the natural alluvium of the New River was defined from consideration of all samples, except the one above Northern Avenue, which was assumed to be a localized effect due to the dip crossing. Figure 5.4 presents the representative bed material gradations for both the New River and Skunk Creek.

In summary, the bed material in Skunk Creek is slightly finer than New River, except where significant armoring has occurred (at confluence of the Arizona Canal and at 83rd Avenue). In the New River, the bed material characteristics are relatively homogeneous above Bethany Home Road. In both the New River and Skunk Creek armoring has occurred and will continue to occur, significantly influencing degradation potential.

5.3.2. Bank Material

The characteristics of the material forming the banks of channels are highly variable, although bank material particle sizes are typically equal to or smaller than the bed particles. Bank material deposited in a channel can be broadly classified as cohesive, non-cohesive, and stratified. Cohesive material is more resistant to surface erosion and has low permeability that reduces the effects of seepage, piping, frost heaving, and subsurface flow on the stability of the banks. However, such banks when undercut and/or saturated are more likely to fail due to mass wasting processes such as sliding or block failure.

Non-cohesive bank material tends to be removed grain by grain from the bank line. The rate of particle removal, and hence the rate of bank erosion, is affected by factors such as the direction and magnitude of the velocity adjacent to the bank, turbulent fluctuations, magnitude and fluctuations in

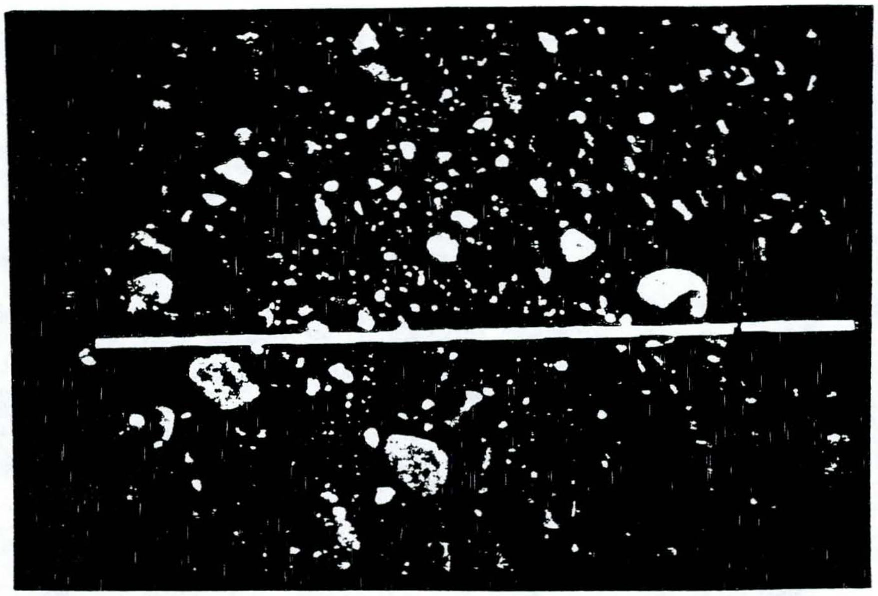


PLATE 5.23

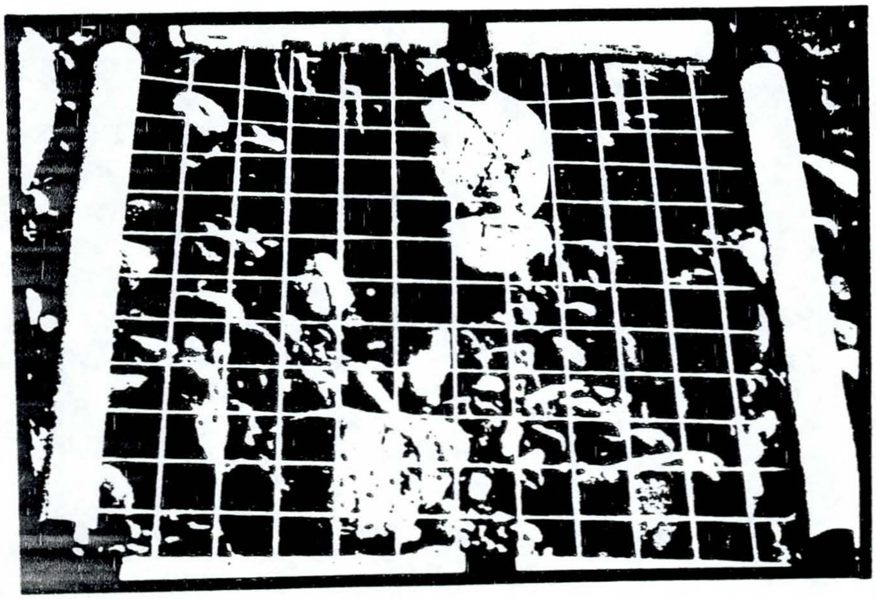


PLATE 5.24

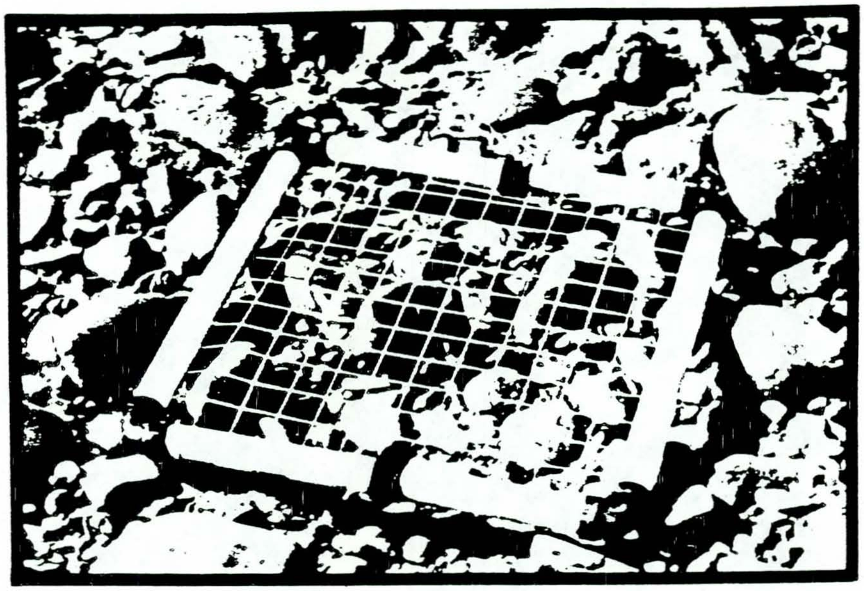


PLATE 5.25

PLATE 5.26

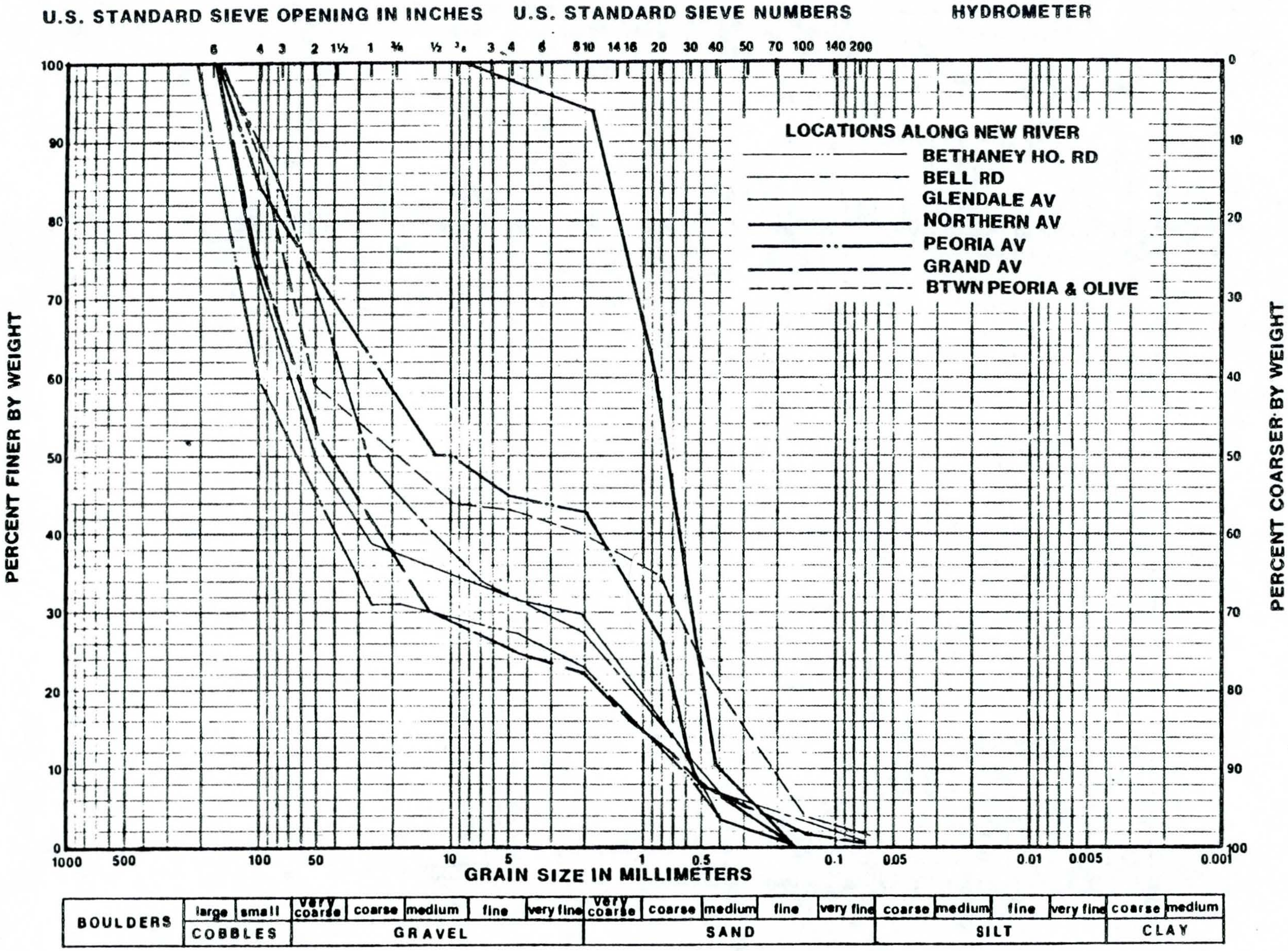


FIGURE 5.2 NEW RIVER PARTICLE SIZE GRADATION

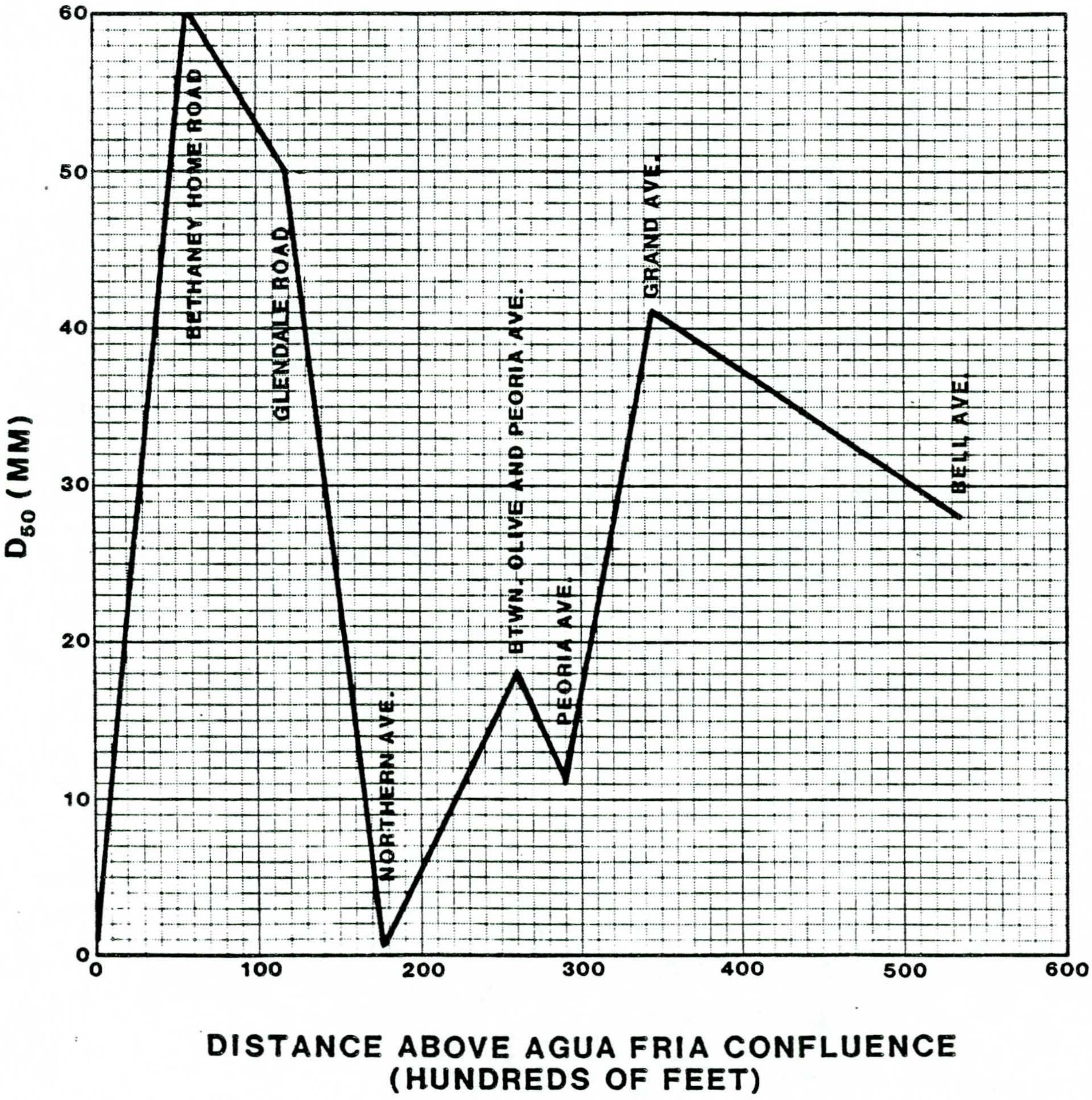


FIGURE 5.3

NEW RIVER PARTICLE SIZE VARIATION

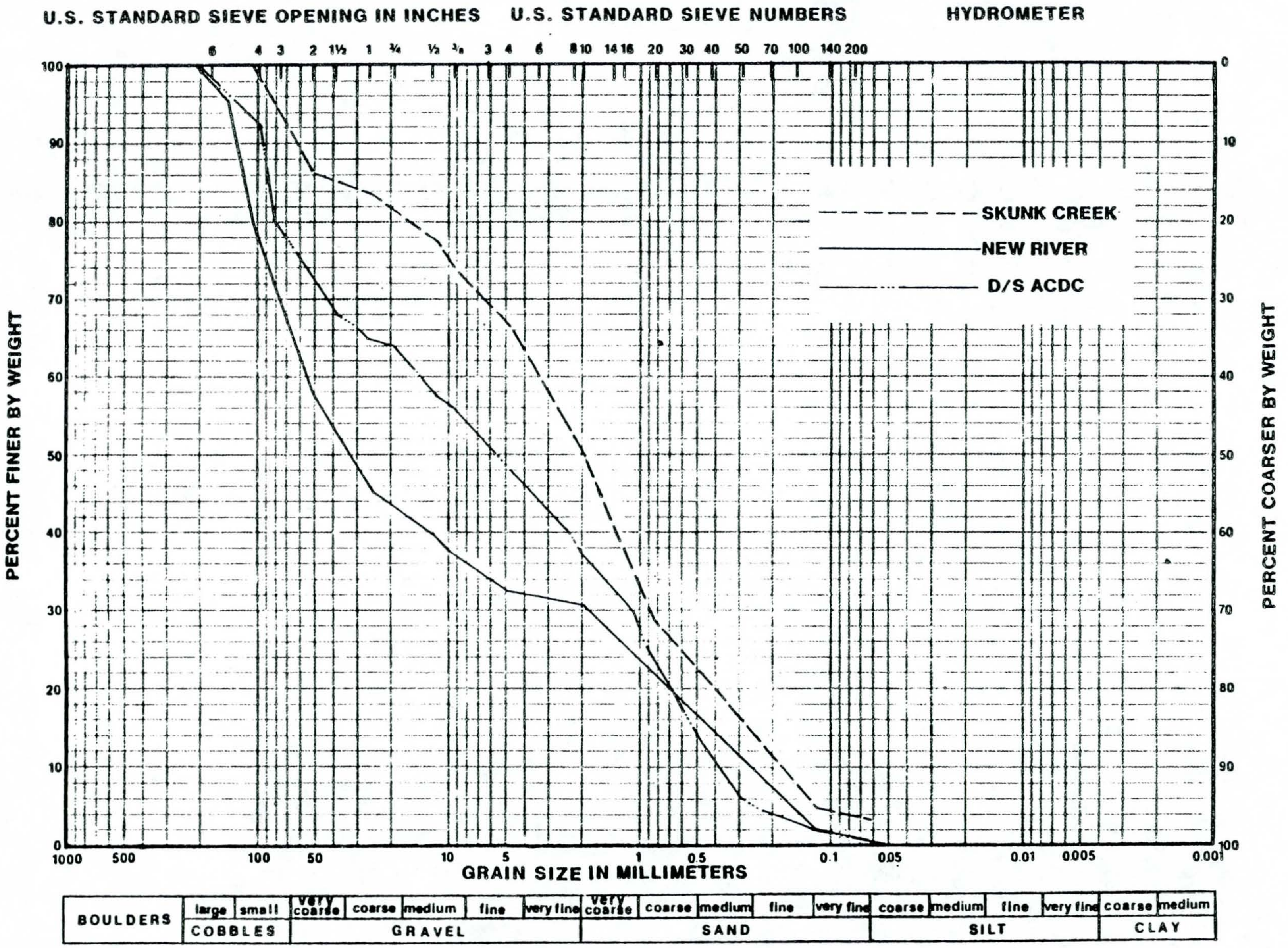


FIGURE 5.4 REPRESENTATIVE PARTICLE SIZE GRADATION

the shear stress exerted on the banks, seepage forces, and piping and wave forces, many of which may act concurrently.

Stratified banks are very common in alluvial channels and generally are the produce of past transport and deposition of sediment. More specifically, these types of banks consist of layers of materials of various sizes, permeability, and cohesion. Layers of noncohesive material are subject to surface erosion, but may be partly protected by adjacent layers of cohesive material. This type of bank is also vulnerable to erosion and sliding as a consequence of subsurface flows and piping.

Banks in the New River and Skunk Creek are typically stratified. In areas where the alluvium has not been recently disturbed or re-worked, the cohesive material present stabilizes the banks resulting in steep or vertical sideslopes. Plate 5.26 illustrates this condition for the low flow bank upstream of the Northern Avenue dip crossing (note the cobbles in this bank). As discussed in Section 2.2, the source of the coarse gravels and cobbles that armor the bed in New River and Skunk Creek are distributed throughout the alluvium in this region.

5.4 Aerial Photographs

5.4.1 General

Aerial photographs provide valuable information for qualitative analysis of hydraulic parameters and channel geometry changes. Furthermore, availability of aerial photographs over a span of years provides time-sequenced documentation of historical trends and changes. For the New River, six sets of aerial photographs covering a time period of 30 years were available. The dates, quality, and scale of these photos are summarized in Table 5.1.

For analysis of long term changes the 1953 and 1983 photographs were compared. The photograph of unknown date and the poor quality 1974 mosaic were not useful to the study.

5.4.2 Skunk Creek from Bell Road to the New River Confluence

Major changes in this reach since 1953 include addition of Bell Road and 83rd Street bridges, a horse track on left side downstream of Arizona Canal, a sports complex on right side, as well as levees on both sides upstream at 83rd Avenue crossing. The river is straighter (less sinuous) than its historical condition due to this channelization. At the confluence of Skunk Creek and New

Table 5.1. Aerial Photos Available for New River and Skunk Creek.

Date	Type	Scale	Quality
3/22/53	Physical Mosaic	1"=1,200'±	Ave. B/W
Unknown	Physical Mosaic	1"=500'±	Good B/W
1974	Physical Mosaic	1"=500'±	Poor Blueprint
9/9/81	Stereo	1"=500'±	Good Color
11/24/81	Stereo	1"=500'±	Good Color
9/27/83	Stereo	1"=500'±	Good B/W

River, major changes have occurred and are currently taking place due to sand and gravel mining. Significant urbanization has taken place in Glendale upstream of the Arizona Canal.

5.4.3 New River From Bell Road to Peoria Avenue

There has been a definite straightening of the river in this reach due to earthen levees on both sides. However, in one spot midway between the confluence and Bell Road, the river has broken out and gone back to a path that was a meander scar in 1953.

From Skunk Creek to Grand Avenue, the New River has maintained its relatively straight path. This has been aided by the channelization upstream of Thunderbird Road. Also, from Grand Avenue to Peoria Avenue, the river path as remained relatively constant, with a meander bend on the left side midway between Grand Avenue and Peoria Avenue. However, major urbanization has taken place adjacent to the river in Sun City and the city of Peoria.

5.4.4 Peoria Avenue to Confluence with Agua Fria

With the exception of gravel mining activity and encroachments due to bridge crossings, there has been little change in river plan form as far downstream as Glendale Avenue. A 1953 gravel pit upstream of Olive Avenue on the right side has been filled in. Since 1953 bridges have been added at Peoria, Olive, and Glendale Avenues.

Downstream of Glendale Avenue, the river in 1953 exhibited a braided condition. However, this reach currently shows a meandering pattern with vegetation up to the low flow channel. Since a braided channel can result from an overload of sediment, this change in plan form may have resulted from removal of sediment immediately downstream of Glendale Avenue in the extensive on going, in channel sand and gravel mining operation. Only a small gravel pit existed in this area in 1953.

The confluence of New River and Agua Fria River is basically the same now as it was in 1953, except that the low flow channel is better defined with vegetation right up to the banks.

5.5 Land Use Changes

5.5.1 General

Water and sediment yield from a watershed is a function of land use practices. Thus, knowledge of land use and historical changes in land use is essential to understanding water and sediment sources in the watershed. In a rapidly growing area such as the Phoenix region, the effects of urbanization can be significant on watershed and channel response. Effects of urbanization include greater runoff volumes with peak flows occurring sooner. Potential damages from flood events also increase as the property value subject to damage increases. The following sections discuss land use changes in the immediate vicinity of the channel and in upper watershed areas.

5.5.2 Land Use Changes in the Immediate Study Area

A report on a proposed flood control master plan by the Flood Control District of Maricopa County (1983) provides insight on recent changes in land use and channel conditions in the study reach. On Skunk Creek immediately downstream of the ACDC, a baseball complex and other recreation facilities have been build on the north bank. The natural channel of Skunk Creek from 83rd Avenue to 75th Avenue has been enlarged, including lowering the invert elevation by about four feet and construction of a levee adjacent to the sports complex. However, it is stated that the levee and channel modification are inadequate to contain the 100-year flood of 31,000 cfs downstream of the ACDC. Additionally, the bridge at 83rd Avenue and the channel downstream to the confluence cannot pass the 100-year flood.

On the New River from the Skunk Creek confluence to Grand Avenue, residential developments have impacted the channel. The Desert Harbor development, initiated in 1981, includes channelization of the New River from Greenway Road to about 1,200 feet upstream of Thunderbird Road. The channel and the new bridge at Thunderbird Road were designed to convey a 100-year flood of about 58,000 cfs. South of Thunderbird Road is the Plaza Del Rio development, who hope to excavate fill material from the channel as a source of borrow.

On the New River from Grand Avenue to Olive Avenue considerable development has occurred in recent years along both banks. In contrast, below Olive Avenue as far as Glendale Avenue the area is still primarily agricultural. Below Glendale Avenue is the site of the future Glendale Municipal Airport. As part of the airport construction, about one million yards of borrow will be taken from the New River and Agua Fria floodplains and the west bank of New River will be revetted (subsequent to September, 1983 and not considered in this analysis).

These developments have altered the water and sediment runoff characteristics of the surrounding watershed and have directly encroached on the channel. Compared to the natural land and agricultural land of pre-development times, more water and less sediment are now being delivered to the channel from local inflow.

5.5.3 Upstream Developments

Upper watershed areas contributing to the New River have been and continue to be impacted by reservoirs (See Figure 3.2). Table 5.2 summarizes both existing and planned reservoirs and their discharge capacity, storage capacity and flood control potential. Reservoir operation can affect sediment delivery in two ways. First, the reservoir can trap significant amounts of sediment, inducing degradation downstream. Second, reduced flood peaks from flood control benefits can significantly reduce transport capacity, limiting downstream degradation. Therefore, the combined action of these two impacts may not significantly alter channel conditions, other than reducing sediment delivery. Reduced sediment delivery will result in a lower safe yield of sand and gravel by mining.

In the New River watershed the flood control benefit provided by the upstream reservoirs is apparent from Table 5.2. Therefore, it is reasonable to assume that sediment delivery in lower reaches has been reduced. Simultaneously, urban growth and the need for sand and gravel have been increasing. If current extraction rates exceed the safe yield of the river, degradation induced by over-extraction may have occurred. Unfortunately, historical profile data are not available to establish if this situation has occurred. (See Section 5.6.2)

5.6 Other Geomorphic Analyses

5.6.1 Plan Form Classification

Classification of a waterway based on plan form characteristics yields insight to the nature and character of the watershed and drainage system. Waterways may be generally classified as straight, meandering, braided, or some combination of these. Reaches of a river that are relatively straight

Table 5.2. Reservoir Capacities.

		Discharge Capacity			Storage Capacity			Flood Control	
Dam	Status	Outlet Works (cfs)	Spillway (cfs)	Total (cfs)	Flood Control (AF)	Sediment Pool (AF)	Total (AF)	SPF Inflow (cfs)	SPF Outflow (cfs)
New River Dam	Planned	2,665	30,335	33,000	38,600	4,920	43,520	45,000	2,665
Adobe Dam	Finished	1,890	10,110	12,000	15,650	2,700	18,350	66,000	1,890
Dreamy Draw Dam	Finished	220	7,000	7,220	281	36	317	3,600	220
Cave Buttes Dam	Finished	486	100,114	100,600	40,900	5,700	46,600	54,000	486

over a long distance are generally unstable, as are divided flow reaches and braided channels.

Examination of aerial photographs suggests that of the three classification categories, the New River and Skunk Creek most closely resemble straight channels with some braided reaches. Application of the slope-discharge relation (Friedkin, 1945; Leopold and Wolman, 1957; and Lane, 1957) provides insight on the potential change in plan form that could occur with the significant increase of water to be provided by the ACDC. The relationship is given in Figure 5.5. For quantitative application, either the mean annual discharge or mean annual flood are required (Schumm, 1977). A purely qualitative application of this relationship assuming a constant bed slope suggests that with the increase in discharge resulting from the ACDC, Skunk Creek and New River may become more strongly braided channels.

5.6.2 Profile Analysis

Only limited profile analysis was possible due to the lack of historical data. Figure 5.6a and b show the 1981 thalweg profile with the update of this profile in 1983. Reaches not re-surveyed in 1983 were determined relatively unchanged by COE personnel. The degradation occurring below Glendale Avenue represents changes from gravel mining activity. At Northern Avenue, topographic mapping suggests the dip crossing was re-constructed, with the centerline of the road being lowered about 1.0 feet. Other changes depicted

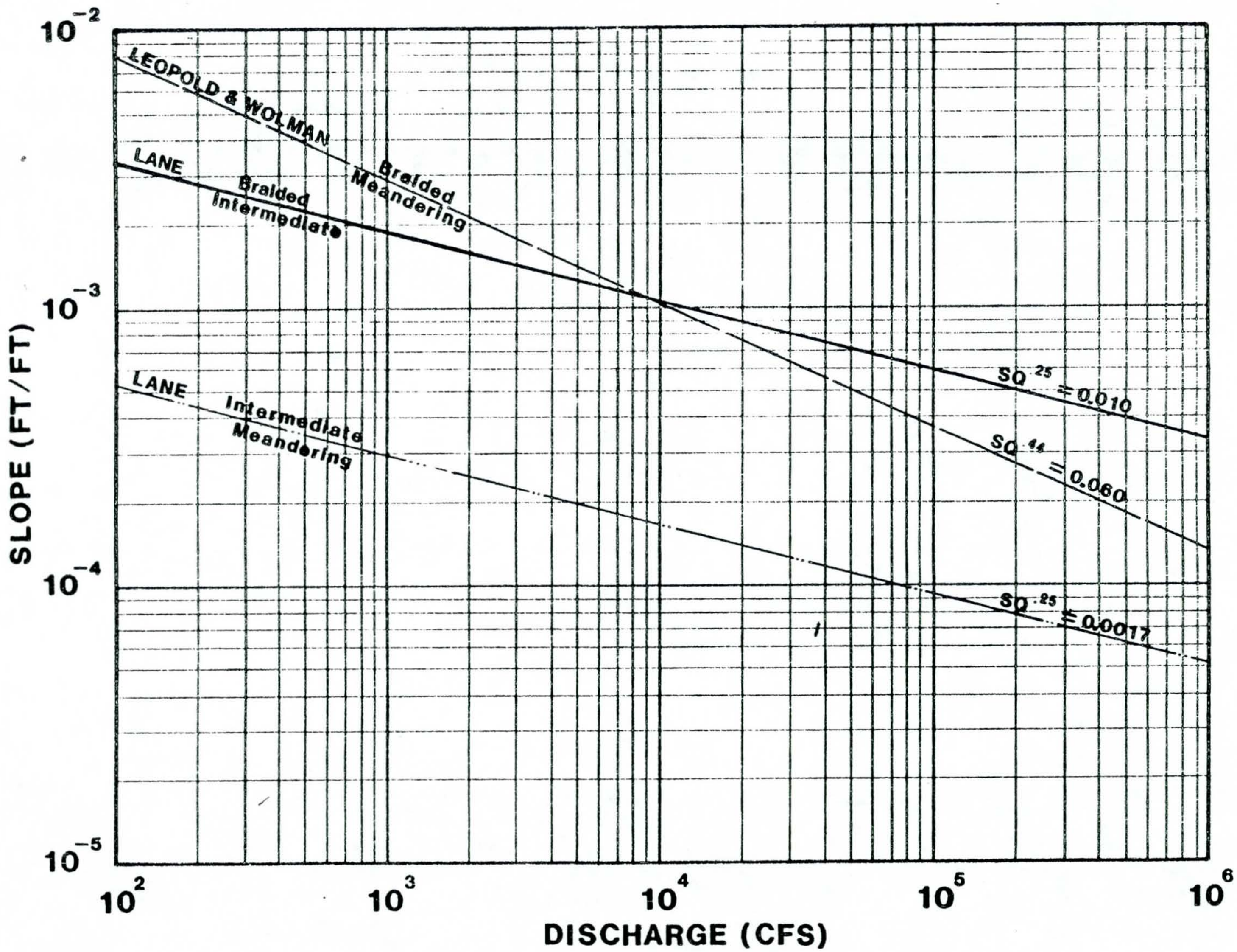


FIGURE 5.5 SLOPE DISCHARGE RELATIONS

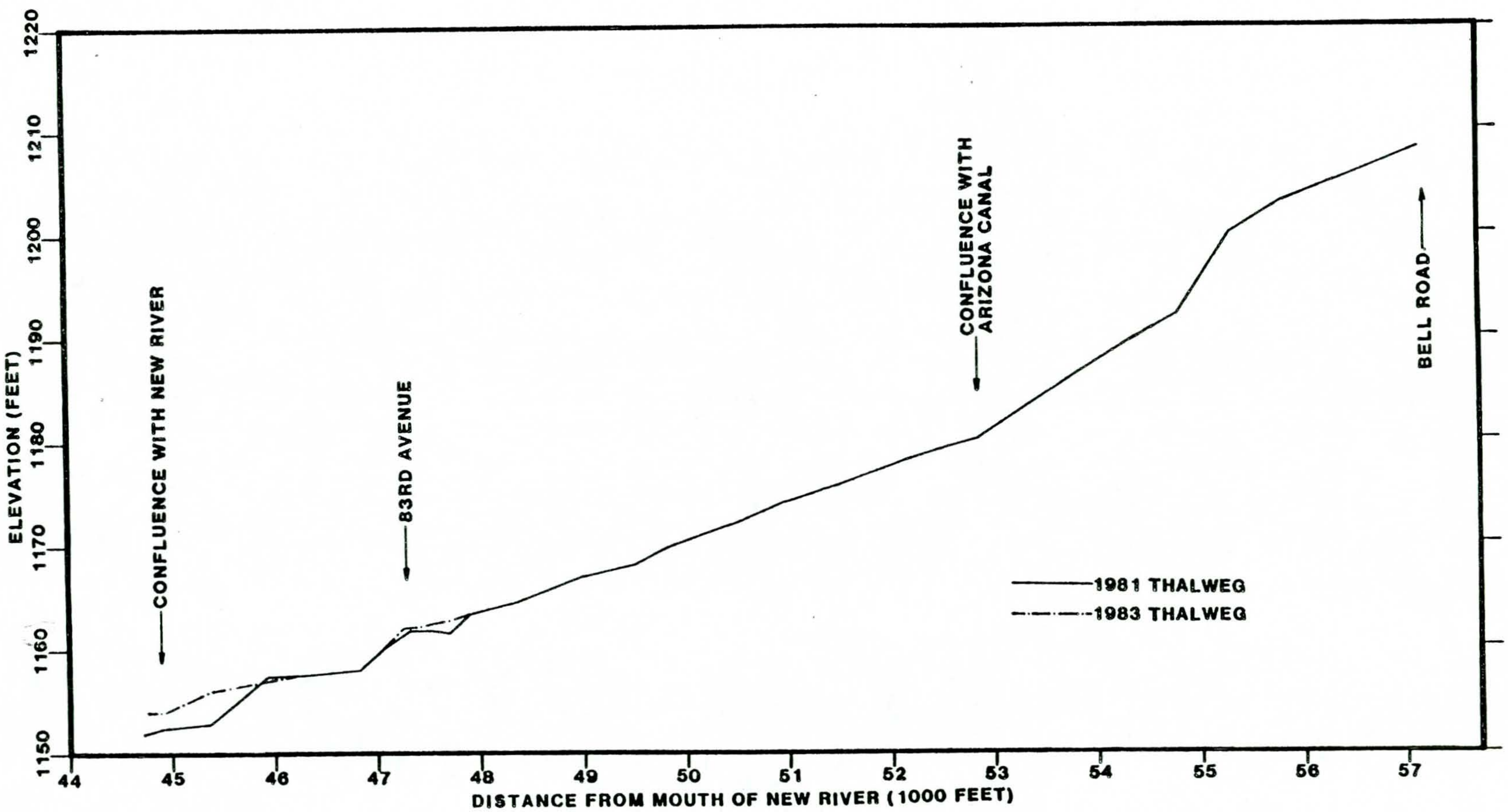


FIGURE 5.6 b

COMPARISON OF 1981 AND 1983 THALWEG ELEVATIONS ALONG SKUNK CREEK

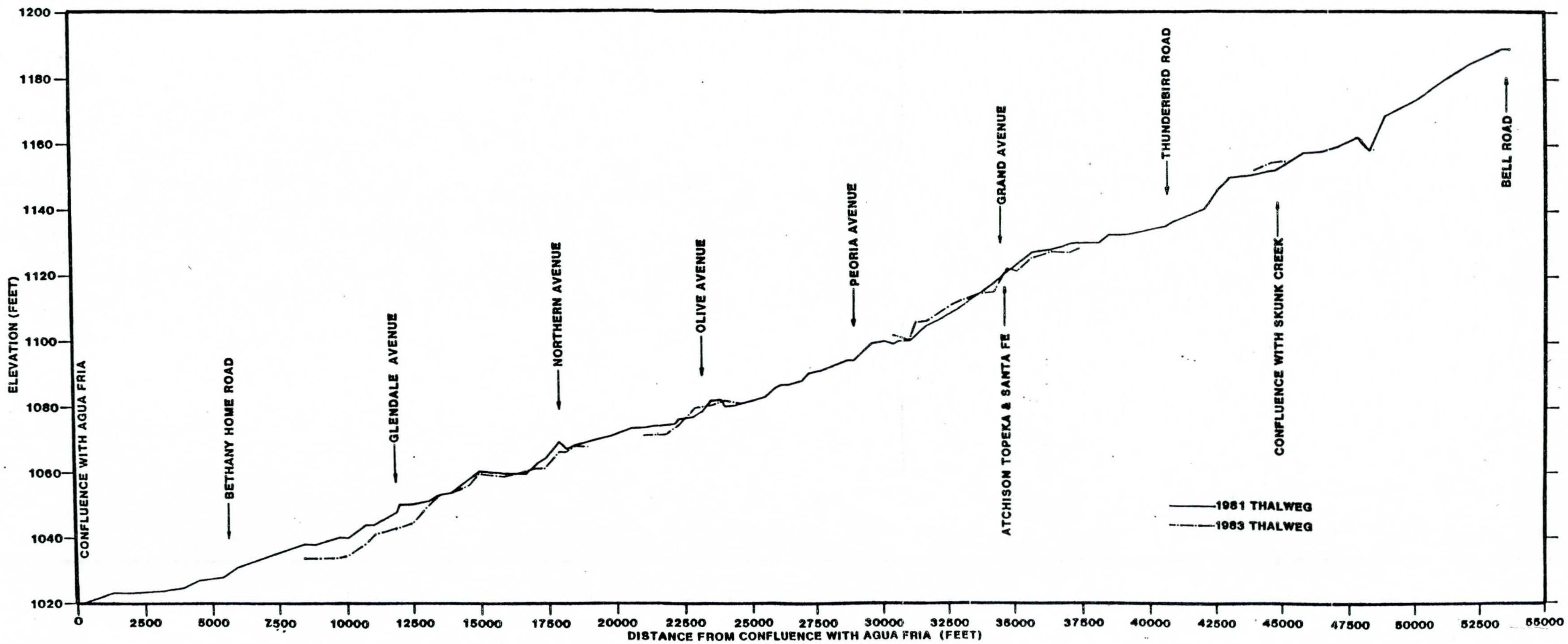


FIGURE 5.6a COMPARISON OF 1981 AND 1983 THALWEG ELEVATIONS ALONG NEW RIVER

VI. QUANTITATIVE GEOMORPHIC ANALYSIS

6.1 General

Quantitative geomorphic analysis is the second level of analysis of a watershed/river system. In Level I, geomorphic principles are applied to predict watershed and stream response and do not require detailed data, only a general understanding of the direction of change of the stream conditions. Geomorphic principles can also be applied to available data to more accurately evaluate watershed or channel responses. This analysis, when coupled with traditional type analyses involving basic engineering relationships, allows an initial quantitative evaluation of response. Analysis techniques used in Level II involve applications of the Shields relation and other geomorphic/engineering relations, application of sediment transport equations and the sediment continuity principle, evaluation of equilibrium slope, frequency analysis of water and sediment transport data, etc. After the channel response is quantified, initial recommendations can be made for flood control measures.

6.2 Sediment Transport Relationships

The Meyer-Peter, Muller (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 New River and Skunk Creek. No bed material or suspended sediment load measurements have been made on the New River and Skunk Creek 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 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 continuous transfer between the two zones. Although there is no distinct line between these two 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. Results of particle size analysis for Skunk Creek and the New River (see

Figures 5.1, 5.2 and 5.4) show very limited amounts of very fine sand (VFS, 0.0625 mm to 0.125 mm) and fine sand (FS, 0.125mm to 0.250 mm) in the surface sediment samples. Therefore, in this study both very fine sand and fine sand will be considered as wash load and will not be included in the development of sediment transport equations.

Sediment transport capacity can be conveniently described as a power function of flow velocity, flow depth and top width. To develop equations of this form, sediment transport capacities for a range of flow conditions and bed material characteristics likely to occur in the New River were determined. Regression on these data then provided the following equations:

$$\text{New River=} \quad Q_s = 1.55 \times 10^{-6} \cdot V^{4.624} \cdot Hy^{-0.044} \cdot TW \tag{6.1}$$

$$\text{Skunk Creek=} \quad Q_s = 2.45 \times 10^{-6} \cdot V^{4.413} \cdot Hy^{0.125} \cdot TW \tag{6.2}$$

where

Q_s is the sediment tranport capacity without VFS and Fs in cfs.

V is the average flow velocity in ft/sec.

Hy is the hydraulic depth in ft.

TW is the top width in ft.

The regressions were derived using the representative particle size gradation curves shown in Figure 5.4. The river bed material of New River has a D_{50} of 32 mm and a gradation coefficient of 34. The gradation coefficient is a measure of the uniformity of the bed material and is defined as:

$$G = \frac{1}{2} \left(\frac{D_{84}}{D_{50}} + \frac{D_{50}}{D_{16}} \right) \tag{6.3}$$

where

G is the gradation coefficient

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

The bed material in the Skunk Creek is considerably finer than that of the New River. The D_{50} sediment size in the Skunk Creek is 2mm and the gradation coefficient is approximately 11.

6.3 River Bed Response

6.3.1 General Approach

The general river bed response (i.e. aggradation/degradation) can be evaluated by comparing the sediment transport capacity from reach to reach. A reach is defined as a group of cross sections with similar hydraulic properties. In the qualitative geomorphic analysis, the New River and Skunk Creek were broken down into 12 reaches. The New River was divided into 8 reaches starting from the confluence with the Agua Fria River to the confluence with Skunk Creek. The reach on the New River between the confluence with the Skunk Creek to Bell Road was treated separately as a tributary. From the confluence with the New River to Bell Road, Skunk Creek was divided into 4 reaches.

The ACDC joins Skunk Creek as a tributary carrying large quantities of water and potentially significant sediment. Two sedimentation basins were proposed in the Arizona Canal Diversion Channel (ACDC) project to reduce the sedimentation problems in the ACDC. These sedimentation basins are located on tributaries with large sediment delivery capacity. The proposed sedimentation basin on Cudia City Wash will trap virtually 100 percent of bed material load ($\geq$ MS), and the proposed sedimentation basin on Cave Creek has a trap efficiency of 91 percent for medium sand (MS), 95 percent for coarse sand (CS) and 100 percent for particles coarser than coarse sand (CS). Thus, sediment inflow from these two tributaries will be reduced significantly.

Short term impacts of these two sedimentation basins on Skunk Creek will be minimal because sediment supply from the ACDC will come from the unlined channel of the ACDC immediately upstream of its confluence with Skunk Creek. This unlined channel, approximately 18,000 feet long and 220 feet wide (bottom width), has a large sediment transport capacity which will cause degradation in the unlined channel and provide sediment supply for Skunk Creek. Degradation in the unlined channel will cause changes in channel geometry, slope, etc., and an equilibrium condition with reduced sediment transport capacity will be reached after a long period of time.

Therefore, the ultimate long term impact of these two sedimentation basins will be significantly reduced sediment supply to Skunk Creek. However, detailed evaluation of sediment supply from the ACDC was beyond the scope of this project. For purposes of this project, short-term supply from the ACDC was based on the transport capacity of the unlined reach. Ultimate long-term supply will be that provided from the outflow of the Cave Creek sedimentation basin; however, the amount of time required to achieve this condition is unknown. For purposes of this project and considering the projected life of most engineering projects, the long-term ACDC supply was assumed as 75 percent of the short-term supply, and the ultimate supply as that from the outflow of the Cave Creek dam. Similarly, due to the proposed New River and Adobe dams, the long-term and ultimate upstream supply from New River and Skunk Creek were assumed as 75 percent of the short-term supply.

6.3.2 Evaluation of Transport Capacity

Using equations 6.1 and 6.2 in combination with the average hydraulics of the subreaches, as was summarized in Tables 4.4 through 4.7, the sediment transport capacity for the 10-, 25-, 50-, and 100- year flood peaks were computed on a reach-by reach basis for the main channel and floodplains. The sediment inflow from the ACDC was obtained from the report "Summary Sedimentation Study Report, Arizona Canal Diversion Channel, Phoenix and Vicinity, Maricopa County, Arizona (SLA & Boyle, June 22, 1983). Tables 6.1 through 6.4 summarize the sediment transport capacities by reach for the main channel and left and right floodplains for the 10-, 25-, 50-, and 100-year flood peaks, respectively.

The sediment transport capacity for the reach immediately downstream of the confluence of Skunk Creek and ACDC (Reach 2) and the reaches between the ATSF Railroad bridge and Olive Avenue (Reaches 6, 7 and 8) is significantly higher than the other subreaches of the river. This condition exists because the effective width of the main channel is narrower in these subreaches resulting in higher flow velocity and sediment transporting capacity.

6.3.3 Short-Term River Bed Response

Short-term river bed response is assessed by comparing the sediment transport capacity of a given reach with that of the reach immediately upstream. Table 6.5 shows the expected short-term river bed responses for the 10- and 100-year flood peaks for all 12 subreaches. As indicated, the bed response varies throughout the study area. At the 10-year flood peak, most of the reaches exhibit a tendency to aggrade, while at the 100-year peak the number of reaches in an aggrading mode and degrading mode are about the same.

Table 6.1. Sediment Transport Capacity for New River, 10-Year Flood[a].

	Left Overbank		Main Channel		Right Overbank	
Reach[b]	Water Discharge (cfs)	Sediment Discharge (cfs)	Water Discharge (cfs)	Sediment Discharge (cfs)	Water Discharge (cfs)	Sediment Discharge (cfs)
1	0	0.0	3,400	4.4	0	0.0
ACDC	0	0.0	7,720	3.5	0	0.0
2	0	0.0	8,950	13.8	0	0.0
3	0	0.0	9,200	7.1	0	0.0
4	0	0.0	9,160	3.9	40	0.0
NRAS[c]	0	0.0	5,100	3.4	0	0.0
5	0	0.0	10,500	15.4	0	0.0
6	0	0.0	10,500	5.8	0	0.0
7	0	0.0	10,500	11.5	0	0.0
8	0	0.0	10,260	6.6	0	0.0
9	0	0.0	10,250	3.5	0	0.0
10	0	0.0	10,500	7.7	0	0.0
11	0	0.0	10,800	3.7	0	0.0
12	50	0.0	10,750	3.1	0	0.0

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

(b) See Table 4.3 for reach definition.

(c) NRAS = New River above Skunk Creek.

Table 6.2. Sediment Transport Capacity for New River, 25-Year Flood[a]

	Left Overbank		Main Channel		Right Overbank	
Reach[b]	Water Discharge (cfs)	Sediment Discharge (cfs)	Water Discharge (cfs)	Sediment Discharge (cfs)	Water Discharge (cfs)	Sediment Discharge (cfs)
1	0	0.0	6,000	7.3	0	0.0
ACDC	0	0.0	13,550	11.3	0	0.0
2	0	0.0	16,520	33.8	0	0.0
3	0	0.0	17,000	15.5	0	0.0
4	0	0.0	15,950	8.4	1050	0.0
NRAS[c]	1	0.0	8,895	6.1	4	0.0
5	0	0.0	18,550	25.4	380	0.0
6	0	0.0	19,000	14.2	0	0.0
7	0	0.0	19,000	26.1	0	0.0
8	0	0.0	18,520	14.5	0	0.0
9	0	0.0	18,500	8.0	0	0.0
10	0	0.0	18,260	18.2	0	0.0
11	0	0.0	18,000	8.8	0	0.0
12	180	0.0	17,720	8.9	100	0.0

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

(b) See Table 4.3 for reach definition.

(c) NRAS = New River above Skunk Creek.

Table 6.3. Sediment Transport Capacity for New River, 50-Year Flood[a].

	Left Overbank		Main Channel		Right Overbank	
Reach[b]	Water Discharge (cfs)	Sediment Discharge (cfs)	Water Discharge (cfs)	Sediment Discharge (cfs)	Water Discharge (cfs)	Sediment Discharge (cfs)
1	0	0.0	9,100	9.7	0	0.0
ACDC	0	0.0	20,620	26.9	0	0.0
2	10	0.0	23,580	48.3	710	0.0
3	10	0.0	23,550	19.4	1,440	0.0
4	40	0.0	21,270	15.9	3,690	0.0
NRAS[c]	5	0.0	13,975	12.4	20	0.0
5	670	0.0	25,190	20.7	3,000	0.0
6	0	0.0	28,990	25.8	10	0.0
7	10	0.0	28,990	40.0	0	0.0
8	10	0.0	28,310	25.9	0	0.0
9	20	0.0	28,280	14.9	0	0.0
10	190	0.0	27,450	28.8	60	0.0
11	0	0.0	27,000	16.5	0	0.0
12	600	0.0	25,530	21.6	870	0.0

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

(b) See Table 4.3 for reach definition.

(c) NRAS = New River above Skunk Creek.

Table 6.4. Sediment Transport Capacity for New River, 100-Year Flood[a].

Reach[b]	Left Overbank		Main Channel		Right Overbank	
	Water Discharge (cfs)	Sediment Discharge (cfs)	Water Discharge (cfs)	Sediment Discharge (cfs)	Water Discharge (cfs)	Sediment Discharge (cfs)
1	50	0.0	12,940	12.3	10	0.0
ACDC	0	0.0	29,000	54.6	0	0.0
2	30	0.0	29,060	52.5	4,910	0.1
3	330	0.0	26,550	16.5	8,120	0.2
4	1,180	0.0	25,880	22.0	7,940	0.1
NRAS[c]	40	0.0	18,895	15.6	65	0.0
5	2,010	0.0	32,610	29.0	6,160	0.1
6	70	0.0	40,790	36.7	140	0.0
7	160	0.0	40,640	39.8	200	0.0
8	30	0.0	40,000	41.4	0	0.0
9	160	0.0	39,830	27.7	10	0.0
10	2,390	0.0	36,590	35.8	560	0.0
11	80	0.0	38,910	29.4	10	0.0
12	2,000	0.1	34,970	35.5	2,030	0.0

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

(b) See Table 4.3 for reach definition.

(c) NRAS = New River above Skunk Creek.

Table 6.5. Expected Short-Term River Bed Responses For The New River And Skunk Creek.

Reach	Sed. Transport (cfs)		Bed Response	
	10-Yr.	100-Yr.	10-Yr.	100-Yr.
1	4.4	12.3	-	-
ACDC	3.5	54.6	-	-
2	13.8	52.5	Degrade	Aggrade
3	7.1	16.5	Aggrade	Aggrade
4	3.9	22.0	Aggrade	Degrade
NRAS	3.4	15.6	-	-
5	15.4	29.0	Degrade	Aggrade
6	5.8	36.7	Aggrade	Degrade
7	11.5	39.8	Degrade	Degrade
8	6.6	41.4	Aggrade	Slight Degrade
9	3.5	27.7	Aggrade	Aggrade
10	7.7	35.8	Degrade	Degrade
11	3.7	29.4	Aggrade	Aggrade
12	3.1	35.5	Aggrade	Degrade

NRAS = New River above Skunk Creek

6.3.4 Long-Term River Bed Response

Over long time periods, a river system will adjust to meet the sediment supply provided by upstream reaches. Therefore, in the analysis of long-term bed response the sediment transport capacity of all downstream reaches are compared with the upstream sediment supply reach rather than the reach immediately upstream.

Table 6.6a summarizes the expected long-term river bed response for the 10- and 100-year flood peaks for the New River and Skunk Creek, assuming no reduction in New River, Skunk Creek and ACDC supply from as-is conditions (September, 1983 conditions). At the 10-year flood peak, most of the reaches are aggradational. At the 100-year flood peak, all the reaches show a tendency to aggrade. The overall aggradational trend is mainly due to sediment carried into the system by the ACDC.

As discussed in Section 6.3.1, over the long term sediment supplied from the ACDC, Skunk Creek above ACDC and New River above Skunk Creek will be reduced due to sediment trapping in the proposed ACDC sedimentation basins and from Adobe Dam and the proposed New River Dam. During the interim period occurring throughout much of the project life, some sediment supply will be available from the unlined ACDC channel near the confluence. Therefore, the long-term (over the project life) river bed response was again evaluated with 25 percent reduction in sediment supply from all three sources. Table 6.6b summarizes river bed responses under this assumption. The overall response is almost the same as before, except Reaches 2, 3 and 7.

Ultimately, after the unlined ACDC channel establishes a new equilibrium the sediment supply from the ACDC will be significantly reduced. At this time, sediment supply from the ACDC into the Skunk Creek will be controlled by the sediment outflow from the proposed Cave Creek sedimentation basin. Under this condition and with the assumed 25 percent reduction in the New River and Skunk Creek supplies, the overall river bed response in the New River and Skunk Creek will most likely be degradational throughout the entire system (Table 6.6c).

Table 6.6a. Expected Long-Term River Bed Responses for the New River and Skunk Creek, Assuming No Reduction in Sediment Supply from As-Is Conditions.

Reach	Sed. Transport (cfs)		Bed Response	
	10-Yr.	100-Yr.	10-Yr.	100-Yr.
1	4.4	12.3	-	-
ACDC	3.5	54.6	-	-
2	13.8	52.5	Degrade	Aggrade
3	7.1	16.5	Aggrade	Aggrade
4	3.9	22.0	Aggrade	Aggrade
NRAS	3.4	15.6	-	-
5	15.4	29.0	Degrade	Aggrade
6	5.8	36.7	Aggrade	Aggrade
7	11.5	39.8	Equilibrium	Aggrade
8	6.6	41.4	Aggrade	Aggrade
9	3.5	27.7	Aggrade	Aggrade
10	7.7	35.8	Aggrade	Aggrade
11	3.7	29.4	Aggrade	Aggrade
12	3.1	35.5	Aggrade	Aggrade

NRAS = New River above Skunk Creek

Table 6.6b. Expected Long-Term (Over Project Life) River Bed Response for the New River and Skunk Creek with a 25 Percent Reduction in Sediment Supplies.

Reach	Sed. Transport (cfs)		Bed Response	
	10-Yr.	100-Yr.	10-Yr.	100-Yr.
1	3.3	9.2	-	-
ACDC	2.6	41.0	-	-
2	13.8	52.5	Degrade	Slight Degrade
3	7.1	16.5	Degrade	Aggrade
4	3.9	22.0	Aggrade	Aggrade
NRAS	2.6	11.7	-	-
5	15.4	29.0	Degrade	Aggrade
6	5.8	36.7	Aggrade	Aggrade
7	11.5	39.8	Degrade	Aggrade
8	6.6	41.4	Aggrade	Aggrade
9	3.5	27.7	Aggrade	Aggrade
10	7.7	35.8	Aggrade	Aggrade
11	3.7	29.4	Aggrade	Aggrade
12	3.1	35.5	Aggrade	Aggrade

Table 6.6c. Expected Ultimate River Bed Responses for the New River and Skunk Creek*.

Reach	Sed. Transport (cfs)		Bed Response	
	10-Yr.	100-Yr.	10-Yr.	100-Yr.
1	3.3	9.2	-	-
ACDC	.0	3.4	-	-
2	13.8	52.5	Degrade	Degrade
3	7.1	16.5	Degrade	Degrade
4	3.9	22.0	Degrade	Degrade
NRAS	2.6	11.7	-	-
5	15.4	29.0	Degrade	Degrade
6	5.8	36.7	Equilibrium	Degrade
7	11.5	39.8	Degrade	Degrade
8	6.6	41.4	Degrade	Degrade
9	3.5	27.7	Aggrade	Degrade
10	7.7	35.8	Degrade	Degrade
11	3.7	29.4	Aggrade	Degrade
12	3.1	35.5	Aggrade	Degrade

* Assuming a 25% sediment reduction from Skunk Creek and New River, and supply from the ACDC equal to sediment outflow from Cave Creek sedimentation basin.

6.4 Annual Sediment Yield

6.4.1 Annual Sediment Yield from the New River

The annual sediment yield in the New River was established by assuming that the reach from Bell Road to the confluence with Skunk Creek was in equilibrium (i.e., transport capacity equal to sediment supply). The sediment yield from this reach for flood hydrographs of varying return periods were then computed and the average annual yield determined using a probability weighting procedure.

To simplify calculation of sediment yield for each hydrograph, a sediment rating curve between water discharge and sediment discharge was developed. This involved executing a multiple profile HEC-II run for a series of water discharges, and using the velocity, top width and hydraulic depth values for each water discharge to compute the sediment discharge (Equation 6.1). A regression was then computed using the water discharges and the sediment discharges. The resultant rating curve for the supply reach on the New River is:

$$Q_s = 1.070 \times 10^{-4} \cdot Q^{1.209} \qquad (6.4)$$

where

Q_s is the sediment ($\geq$ MS) discharge in cfs

Q is the water discharge in cfs

The sediment volumes for the 10-, 25-, 50-, and 100-year flood hydrographs in the supply reach were computed by discretizing the hydrographs and then applying Equation 6.4. The sediment volumes and water yields for the 2- and 5- year flood events in the supply reach were extrapolated (log-probability extrapolation) using the results of the 10-, 25-, 50- and 100-year flood events. Table 6.7 summarizes the water and sediment yield volumes for the 2-, 5-, 10-, 25-, 50- and 100-year flood events in the supply reach.

The sediment volumes were then weighted by the incremental probability of occurrence within a year, and by the average annual water volume divided by the incremental probabilistic water volume of the 2-, 5-, 10-, 25-, 50-, and 100-year flood hydrographs, to determine the average annual sediment yield. This is expressed mathematically as:

Table 6.7 Water And Sediment Yields For The 2-, 5-, 10-, 25-, 50-, and 100 Year Flood Events For The New River Supply Reach.

Flood Event	Water Yield (Acre - Feet)	Sediment Yield (Acre - Feet)
2-Year	280	0.11
5-Year	820	0.38
10-Year	1,430	0.74
25-Year	2,500	1.44
50-Year	3,930	2.49
100-Year	5,340	3.61

$$\forall = K \left[0.01 \; (\forall_{s_{100}}) + 0.01 \left(\frac{\forall_{s_{100}} + \forall_{s_{50}}}{2}\right) + 0.02 \left(\frac{\forall_{s_{50}} + \forall_{s_{25}}}{2}\right) + 0.06 \left(\frac{\forall_{s_{25}} + \forall_{s_{10}}}{2}\right) + 0.1 \left(\frac{\forall_{s_{10}} + \forall_{s_{5}}}{2}\right) + 0.3 \left(\frac{\forall_{s_{5}} + \forall_{s_{2}}}{2}\right) + 0.5 \left(\frac{\forall_{s_{2}}}{2}\right)\right] \quad (6.5)$$

where

$\forall_{s_a}$ is the average annual sediment volume,

$v_{s_{100}}, \forall_{s_{50}}, \forall_{s_{25}}, \forall_{s_{10}}, \forall_{s_5}, \forall_{s_2}$ are the sediment volumes of the 100-, 50-, 25-, 10-, 5-, and 2-year floods, repectively, and

K is the correction factor to account for the numerical errors.

Usually, the correction factor K is computed as $\forall_{meas}/\forall_a$, where $\forall_{meas}$ is the mean annual water volume determined from gaging station data of sufficient length and $\forall_a$ is average annual water volume, computed by substituting for sediment volume in Equation 6.5 the water volume of the same return period. The gaging station at Bell Road only has 14 years of record for annual water volume and three of these, 1978, 1979, and 1980, were extremely wet years. Additionally, the calculated water yield is based on hydrology that incorporates the increased water delivered by the ACDC, which is not reflected in the measured data. For these reasons it was decided not to use this information to compute the correction factor K. From past experience in working with similar river systems, a K factor of 1.5 was judged to be a reasonable value for this study. Applying the K factor of 1.5 with corresponding sediment volumes in Equation 6.5, the average annual sediment yield obtained is 0.53 acre-feet from the New-River supply reach. This translates into an average annual sediment concentration of 2,000 parts per million by weight.

6.4.2 Annual Sediment Yield from Skunk Creek

Applying the same methodology, the average annual sediment yield from Skunk Creek was computed. The supply reach in Skunk Creek was defined from Bell Road to the confluence with the ACDC. The sediment rating curve for Skunk Creek is:

$$Q_s = 3.206 \times 10^{-3} \cdot Q^{0.8888} \quad \text{for } Q \leq 6{,}000 \text{ cfs}$$

$$Q_s = 2.146 \times 10^{-2} \cdot Q^{0.6701} \quad \text{for } Q \geq 6{,}000 \text{ cfs} \qquad (6.6)$$

Table 6.8 summarizes the water sediment yields from Skunk Creek sediment supply reach for the 2-, 5-, 10-, 25-, 50-, and 100-year floods. As a result of the same concerns as in the New River, a K factor of 1.5 was used in the Skunk Creek calculations to compute the average annual sediment yield. The resulting average annual sediment yield from Skunk Creek is 1.04 acre-feet, with an average annual sediment concentration of 6,060 parts per million by weight.

6.4.3 Annual Sediment Yield from the ACDC

The average annual sediment yield from the ACDC was taken from the report entitled, "Final Summary Sedimentation Study Report, Arizona Canal Diversion Channel, Pheonix and Vicinity, Maricopa County, Arizona," (SLA & BOYLE, June 22, 1983). The unlined trapezoidal channel immediately upstream of the confluence with Skunk Creek was considered as the supply reach of the ACDC.

Table 6.9 summarizes the sediment yields from the ACDC sediment supply reach (sta. 14975) for the 2-, 5-, 10-, 25-, 50-, and 100-year floods. The average annual sediment yield is also included in this table.

6.5 Equilibrium Slope

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

Table 6.8. Water and Sediment Yields For the 2-, 5-, 10-, 25-, 50-, and 100-year Flood Events For The Skunk Creek Supply Reach.

Flood Event	Water Yield (Acre - Feet)	Sediment Yield (Acre - feet)
2-Year	180	0.34
5-Year	530	0.83
10-Year	930	1.34
25-Year	1640	2.22
50-Year	2490	3.14
100-Year	3560	4.17

Table 6.9. Water and Sediment Yields For the 2-, 5-, 10-, 25-, 50-, and 100-year Flood Events for the ACDC Supply Reach.

Flood Event	Water Yield (Acre - Feet)	Sediment Yield (Acre - feet)
2-Year	73	0.40
5-Year	428	0.22
10-Year	1,313	0.67
25-Year	4,219	2.15
50-Year	10,088	5.15
100-Year	20,406	10.44
ANNUAL	569	1.47

The equilibrium slope analysis is usually determined for the dominant discharge in the river, or the discharge that most influences the channel characteristics. Althrough it is difficult to precisely establish the dominant discharge, the value is typically between the 5- and 10-year events for intermittent and ephemeral streams. For the New River and Skunk Creek, the 10-year flood peak discharge was selected as the dominant discharge because most of the flow is contained within the channel.

Table 6.10 summarizes for each reach the existing slope, the sediment transport capacity for the 10-year flood peak, the average hydraulics, and the equilibrium slope to which the reach will adjust. Reach 2 exhibits a large degradation potential which is expected due to the narrow channel width and high flow velocity in this area. In contrast, Reaches 4, 6, 8, 9, 10, 11, and 12 exhibit a large aggradation potential due to the smaller sediment transport capacity of these reaches.

6.6 Armor Control Analysis

6.6.1 Potential for Armor Control

The equilibrium slopes shown in Table 6.10 were computed on the existing sediment supply from Reach 1 (Skunk Creek above the ACDC) and Reach NRAS (New River above Skunk Creek). This sediment supply, however, may be reduced due to river bed armoring resulting from trapping of sediment in Adobe Dam and the proposed New River Dam. Therefore, it was necessary to conduct an armor control analysis for both supply reaches.

When the shear stress over the bed attains or exceeds the critical value, particle motion begins. Sheilds criterion for incipient motion of sediment particles was used to compute the approximate critical velocities for transporting very fine gravels to small cobbles. Table 6.11 shows the results of the incipient motion analysis. As can be seen from the table, the incipient motion of small cobbles occurs at a velocity of 10 fps. Previous hydraulic analysis showed that main channel velocities ranged from 6 to 11 fps for the 10-year flood peak and from 7 to 13 fps for the 100-year flood peak in the New River and Skunk Creek sediment supply reaches. Therefore, considering the availabiltiy of these particles, the armoring potential of small cobbles in the sediment supply reaches is quite high.

Table 6.10. Summary of the Equilibrium Slope Analysis for the 10-Year Peak Discharge.

Reach	Description	Top Width (ft)	Water Discharge (cfs)	Flow Velocity (fps)	Sediment Discharge (cfs)	Conc. by Weight (ppm)	Slope Existing	Slope Equil.
1.	Skunk Creek, from Bell Road to the confluence with ACDC	135	3,400	8.3	4.4	3,430	.0064	.0064
ACDC	ACDC above Skunk Creek	-	-	-	3.5	1,200	.0004	-
2.	Skunk Creek, from the confluence with ACDC to 3,400 feet downstream of the confluence	145	8,950	10.4	13.8	4,090	.0035	.0023
3.	Skunk Creek, from 3,400 feet downstream of the confluence with ACDC to 2,400 feet upstream of the confluence with the New River	185	9,200	8.5	7.1	2,050	.0035	.0038
4.	Skunk Creek, from 2,400 feet upstream of the confluence with the New River to the confluence with the New River	230	9,200	7.0	3.9	1,120	.0035	.0061
NRAS	New River above Skunk Creek	335	5,100	6.0	3.4	990	.0043	-
5.	New River, from the confluence with the Skunk Creek to 2,450 feet downstream of the confluence	210	10,450	10.4	15.4	3,900	.0039	.0031
6.	New River, from 2,450 downstream of the confluence with Skunk Creek to 1,450 feet upstream of the ATSF Railroad Bridge	270	10,500	8.0	5.8	1,460	.0020	.0032
7.	New River, from 1,450 feet upstream of the ATSF Railroad Bridge to Peoria Avenue	305	10,500	9.0	11.5	2,900	.0049	.0048
8.	New River, from Peoria Avenue to Olive Avenue	290	10,250	8.1	6.6	1,710	.0026	.0038
9.	New River, from Olive Avenue to Northern Avenue	345	10,250	6.8	3.5	910	.0018	.0042
10.	New River, from Northern Avenue to Glendale Avenue	310	10,500	8.2	7.7	1,940	.0032	.0042
11.	New River, from Glendale Avenue to Bethany Home Road	400	10,800	6.6	3.7	910	.0030	.0067
12.	New River, from Bethany Home Road to the confluence with the Agua Fria River	475	10,800	6.2	3.1	760	.0018	.0045

Table 6.11. Critical Velocities for Incipient Motion of Sediment Particles.

Sediment Particle	Size (mm)	Critical Velocity (fps)
Very fine gravel	2-4	1.8
Fine gravel	4-8	2.5
Medium gravel	8-16	3.5
Coarse gravel	16-32	5.0
Very coarse gravel	32-64	7.0
Small cobble	64-130	10.0

In most cases, armoring of the entire supply reach is not likely to occur. Yet, the sediment supply can be reduced due to partial armoring or bed material coarsening. To account for this possible reduction of sediment supply in the future, the equilibrium slopes were recomputed assuming a 25 percent reduction in sediment supply. Table 6.12 shows the resultant equilibrium slopes with reduced sediment supply. Under this condition, degradation problems become worse while aggradation problems are alleviated to some extent.

Other armor control analyses include the computation of static equilibrium slopes and the determination of the armoring potential by particle size.

6.6.2 Static Equilibrium Slope

The static equilibrium slopes were computed utilizing Shields relationship for incipient motion, assuming there is adequate coarse material available in the subsurface to remain motionless during large flows. In this study, the subsurface material was assumed to be similar to that of surface material observations and particle gradation analysis of surface samples. From Figures 5.1 and 5.4, a representative armor particle size of 3 inches was selected for the Skunk Creek above the ACDC and 4 inches for all the remaining reaches.

Using Shields relation,

$$\tau_c = 0.047\ (\gamma s - \gamma)\ d_s \qquad (6.7)$$

τ_c is the critical shear stream stress initiating particle movement

γs is the unit weight of sediment

γ is the unit weight of water

d_s is the representative particle size

and applying the representative particle size of armor material, a critical shear can be computed. Equating the shear stress in the Darcy-Weisbach resistance equation to the critical shear stress,

$$\tau_c = \frac{1}{8}\ \rho\ f\ V^2 \qquad (6.8)$$

Table 6.12. Equilibrium Slopes Considering a 25 Percent Reduction from the Supply Reach for the 10-Year Peak Discharge.

Reach	Existing Slope	Sediment Transport Capacity (cfs)	Equilibrium Slope
1	.0064	3.3	.0064
ACDC	.0004	2.6	-
2	.0035	13.8	.0018
3	.0035	7.1	.0030
4	.0035	3.9	.0048
NRAS	.0043	2.6	-
5	.0039	15.4	.0026
6	.0020	5.8	.0026
7	.0049	11.5	.0039
8	.0026	6.6	.0031
9	.0018	3.5	.0034
10	.0032	7.7	.0034
11	.0030	3.7	.0054
12	.0018	3.1	.0037

where

τ is the bed shear stress

ρ is the density of water

f is the Darcy-Weisbach friction factor, and

V is the flow velocity,

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

$$S = \left(\frac{V^{5/3}\ n}{1.48\ q^{2/3}}\right)^2 \qquad (6.9)$$

where

S is the bed slope

V is the flow velocity

n is the Manning flow resistance

q is the unit width discharge

Table 6.13 summarizes the static equilibrium slopes computed for the 100-year flood peak discharge for each reach assuming the representative particle size armor material will form on the surface. Reaches 2, 5, and 8 have extremely flat static equilibrium slopes as a result of narrower channel conditions producing large unit width discharges. Degradation of this magnitude is unlikely to occur without bank failure, which would widen the channel, reduce the unit width discharge and steepen the equilibrium slope. Therefore, the calculated static equilibrium slopes are not expected to be achieved in these reaches.

6.6.3 Particle Armoring Method

The particle size armoring method provides an estimate of the depth of degradation that will occur before development of an armor layer. Assuming an armor layer 2 particle diameters thick forms on the channel bed, the degradation depth will be:

Table 6.13. Static Equilibrium Slope Analysis for the 100-Year Peak Discharge.

Reach	Existing Slope	Top Width (ft)	Water Discharge (cfs)	Unit Discharge (cfs/ft)	Static Equilibrium Slope
1	.0064	185	12,940	69.9	.0016
2	.0035	195	29,060	149.0	.0009
3	.0035	275	26,550	96.5	.0016
4	.0035	265	25,880	97.7	.0016
5	.0039	255	32,610	127.9	.0011
6	.0026	380	40,790	107.3	.0014
7	.0049	365	40,640	111.3	.0014
8	.0026	340	40,000	117.6	.0013
9	.0018	415	39,830	96.0	.0017
10	.0032	390	36,590	93.8	.0017
11	.0030	515	38,910	75.6	.0023
12	.0018	655	34,970	53.4	.0036

$$D_{sc} = \frac{2\ d_s}{P_c} \qquad (6.10)$$

where

D_{sc} is the depth of scour

d_s is the size of armoring material

P_c is the percent of material coarser than the armoring size

Using 3 inches as the armor particle size in Skunk Creek above ACDC, and the gradation curve for sediments in Skunk Creek at Bell Road (Figure 5.1), the depth to armor formation in Skunk Creek above ACDC was computed as 10.0 feet. For all remaining reaches an armor particle size of 4 inches was assumed. Using the gradation for Skunk Creek below the ACDC (Figure 5.1) the depth to armor layer formation below ACDC and above New River is 8.3 feet. For the New River, using the representative gradation curve (Figure 5.4) the depth is 8.0 feet.

6.7 Controlling Bed Response

Table 6.14 summarizes the expected aggradation/degradation response along the New River/Skunk Creek for the dynamic equilibrium, static equilibrium and particle armoring size methodologies. The dynamic equilibrium slope will control the grade at all reaches. The dynamic equilibrium slope presented in Table 6.14 is based on the present sediment supply condition. Should the reduction of sediment supply occur due to the armoring of river bed, the dynamic equilibrium slopes will become flatter than those predicted in Table 6.14. Under the reduced sediment supply condition the channel will become more entrenched and the banks will become unstable.

6.8 Local Scour Analysis

6.8.1 Local Scour at Bridge Crossings

Local scour at bridge piers is a result of vortex systems developed at the pier and can be evaluated using Shen's and Neil's equations. These equations were developed from extensive laboratory experiments on sand-bed channels and provide reasonable estimations for local scour depths. Shen's equation takes the following form:

Table 6.14. Comparison of Bed Response Using Different Methods.

Reach	Reach Length (ft)	Existing Slope	Dynamic Equilibrium Slope	Static Equilibrium Slope	Particle Armoring Method (ft)
1	3,863	.0064	.0064	.0016	10.0
2	3,430	.0035	.0023	.0009	8.3
3	2,856	.0035	.0038	.0016	8.3
4	2,434	.0035	.0061	.0016	8.3
5	2,450	.0039	.0031	.0011	8.0
6	6,371	.0020	.0032	.0014	8.0
7	7,033	.0049	.0048	.0014	8.0
8	5,820	.0026	.0038	.0013	8.0
9	5,172	.0018	.0042	.0017	8.0
10	6,221	.0032	.0042	.0017	8.0
11	6,091	.0030	.0067	.0023	8.0
12	5,695	.0018	.0045	.0036	8.0

$$d = K\ 0.00073\ R^{0.619}$$

where

d_s is the local scour depth

k is a multiplying factor to account for skewness of piers (see Table 6.15)

R is the pier Reynolds number

$$R = \frac{Va}{\nu} \tag{6.11}$$

where

V is the average flow velocity upstream of the bridge pier

a is the frontal width of the pier

ν is the kinematic viscosity of the water

Neil's equation takes the following form:

$$d_s = d_w\ (2)\ (\frac{a}{d_w})^{0.65}\ F_r^{0.43}\ k \tag{6.12}$$

where

k is a multiplying factor to account for skewness of piers (see Table 6.15)

d_w is the upstream depth of flow

a is the frontal width of the pier

F_r is the upstream Froud number

Results of local scour computations for the 100-year peak discharge are shown in Table 6.16. Velocity, depth and flow skewness used for local scour computations are included in the table. To account for debris accumulation near bridge piers, 2 feet was added to either side of the piers, or equivalently, a total of 4 feet to each bridge pier. As indicated in Table 6.16, a strict application of the local scour equations suggests scour depths as large as 36 feet are possible; however, these results must be properly interpeted based on engineering judgement and experience. The basic formulas were developed from laboratory data in sand bed channels, where scour depths would typically be larger than those expected in a coarser allumium, such as that in the New River drainage. Additionally, in a coarse alluvium the local scour depth

Table 6.15. 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 Stenurk, F., Sediment Transport Technology, Water Resource Publications, 1977.

Table 6.16. Approximate Local Scour Depths at Bridge Crossings for the 100-Year Flood.

Bridge Crossing	Pier Diameter or Width (ft)	Flow Velocity (fps)	Flow Depth (ft)	Flow Skewness (°)	Skew Factor	Local Scour Depth (ft)		Average Scour Depth (ft)
						Shen	Neil	
NEW RIVER								
Bell Road	1.2	11.2	11.4	10	2.9	32.6	31.5	32.0
Thunderbird Road	3	12.0	16.8	0	1.0	14.1	14.3	14.2
ATSF Railroad	6	9.8	17.7	11	1.2	18.6	20.0	19.3
Grand Avenue	5	8.4	16.6	15	1.9	25.1	27.4	26.3
Peoria Avenue	2	12.1	15.7	2	1.0	12.9	12.9	12.9
Olive Avenue	2	12.7	12.3	-	1.0	13.3	12.7	13.0
Glendale Avenue	1.3	13.0	11.2	7	3.0	37.6	35.2	36.4
SKUNK CREEK								
Bell Road	1.5	9.9	7.5	3	1.0	10.8	10.1	10.4
83rd Avenue	1.5	5.8	22.1	3	1.0	7.7	9.3	8.5

may be limited by armor control. Finally, the calculated scour is significantly effected by the skew factor correction; however, due to the large length-width ratio on several of the New River bridges, a significant extrapolation of information in Table 6.15 was required. Considering all these factors, it is reasonable to limit local scour at any bridge in the study reach to the depth of armor layer development.

Armor layer calculations for the main channel (see Section 6.6.3) were based on the development of an armor layer 2 particle diameters thick. Given the potential in a scour hole development for larger velocity and/or velocity vectors with significant incident angles, it is reasonable to utilize an armor layer thickness of 3 particle diameters. This assumption results in an armor depth of approximately 15 feet at the bridge crossings. Therefore, local scour at the Bell Road (New River), ATSF Railroad, Grand Avenue, and Glendale Avenue bridges will be limited by armor control to about 15 feet. Table 6.17 summarizes thalweg elevation, elevation at bottom of pier footing, scour depth and expected scour elevations is lower than the bottom of bridge pier footing in all bridges with exception of Thunderbird Road and Olive Avenue bridges. Once the scour hole reaches below the pier footing the stability of bridge becomes questionable, therefore, a riprap blanket protection around piers should be provided to ensure the bridge safety.

6.8.2 Local Scour Around Bridge Abutments

An empirical equation for evaluation of local scour around embankments was developed in the laboratory by Liu, et. al., (1961). The equilibrium local scour depth may be computed by:

$$S = d_1 \, (1.1) \, \left(\frac{a}{d_1}\right)^{0.4} \, F_{r_1}^{0.33} \qquad (6.13)$$

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

Table 6.17. Expected Local Scour Elevation at Bridge Crossings for the 100-Year Flood.

Bridge Crossing	Thalweg Elevation (ft)	Bottom of Pier Footing (ft)	Average Scour Depth (ft)	Expected Scour Elevation (ft)
NEW RIVER				
Bell Road	1,188.6	1,186.3	15.0	1,173.6
Thunderbird Road	1,135.6	1,090.5	14.2	1,121.4
ATSF Railroad	1,120.0	1,113.0	15.0	1,105.0
Grand Avenue	1,119.5	1,114.5	15.0	1,104.5
Peoria Avenue	1,094.3	1,089.8	12.9	1,081.4
Olive Avenue	1,079.9	1,057.5	13.0	1,066.9
Glendale Avenue	1,047.0	1,043.8	15.0	1,032.0
SKUNK CREEK				
Bell Road	1,208.0	1,198.0	10.4	1,197.6
83rd Avenue	1,162.0	1,156.0	8.5	1,153.5

Tables 6.18 and 6.19 summarizes results of the local scour analysis around the bridge abutments for the New River and Skunk Creek, respectively. Local scour was not evaluated for the left (east) abutment at the ATSF Railroad Bridge and Grand Avenue bridge because the local topographic conditions create an ineffective flow area in these locations. As discussed in the bridge pier scour section (Section 6.8.1), the scour depth will be limited by armor control to about 15 feet at Thunderbird Road and Grand Avenue. Toe down elevations are available for the Thunderbird Road Bridge and 83rd Avenue Bridge only, and protection is needed for both bridges. Additional information concerning toe down elevation for other bridges are necesary in order to perform an adequate evaluation.

Table 6.18. Local Scour at Bridge Abutments on New River for 100-Year Flood Peak.

	Abutment Protrudement to Flow (ft)	Water Surface Elevation (ft)	Riverbed Elevation Near Abutment (ft)	Water Depth Abutment (ft)	Velocity (cfs)	Froude Number	Local Scour (ft)
Bell Road							
left abut	90	1,200.0	1,192	8.3	11.2	0.69	10.6
right abut	100	1,200.0	1,190	10.3	11.2	0.61	12.7
Thunderbird							
left abut	500	1,152.4	1,136	16.4	12.0	0.52	19.2
right abut	230	1,152.4	1,136	16.4	12.0	0.52	19.2
ATSF							
left abut	-	1,137.7	-	-	9.8	-	-
right abut	100	1,137.7	1,121	16.7	9.8	0.42	16.3
Grand Ave.							
left abut	100	1,136.1	-	-	8.4	-	-
right abut	100	1,136.1	1,120	16.1	8.4	0.37	19.8
Peoria Ave.							
left abut	140	1,110.0	1,098	12.0	12.1	0.62	13.2
right abut	70	1,110.0	1,098	12.0	12.1	0.62	13.2
Olive Ave.							
left abut	30	1,092.2	1,080	12.2	12.7	0.64	15.3
right abut	20	1,092.2	1,080	12.2	12.7	0.64	15.3
Glendale Ave.							
left abut	150	1,058.2	1,050	8.2	13.0	0.80	9.9
right abut	350	1,058.2	1,050	8.2	13.0	0.80	9.9

Table 6.19. Local Scour at Bridge Abutments on Skunk Creek for 100-Year Flood Peak.

	Bell Road		83rd Avenue	
	left abut	right abut	left abut	right abut
Abutment protrudement to flow (ft)	30	30	500	600
Water surface elevation (ft)	1,215.5		1,184.1	
Riverbed elevation near abutment (ft)	1,209	1,208	1,064	1,064
Water Depth near abutment (ft)	6.5	7.5	20.1	20.1
Velocity (fps)	9.9		5.8	
Froude number	0.68	0.64	0.23	0.23
Local scour (ft)	9.10	10.3	16.0	16.0

VII. SEDIMENT ROUTING ANALYSIS

7.1 General

The qualitative and quantitative geomorphic analysis of the New River and Skunk Creek (Chapters V and VI) indicated that under the existing conditions and with the ACDC in operation, most of the reaches exhibit aggradational potential with exception of Reaches 2 and 5, which showed a degradation trend. Assuming a 25 percent reduction of the sediment supply, all reaches still exhibit the same response as those under the current condition. The exceptions are Reaches 3 and 7, which exhibit a degradation potential under this new condition. Local scour analysis for the 100-year flood peak discharge suggested that bridge pier protection (i.e., riprap blanket) would be beneficial at all bridges with exception of Olive Avenue bridge and Thunderbird Avenue bridge. In addition, abutment protection would be beneficial at all bridges.

In consideration of these results and other relevant information, the COE developed the preferred flood control alternative and prepared the HEC-2 input deck reflecting the proposed improvements. Briefly, the flood control alternative consisted of channel enlargement and installation of a stabilization structure in the vicinity of Grand Avenue. The proposed floodway channel is about 3,400 feet in length (approximately centered at Grand Avenue), with a bottom width of 310 feet and a side slope of 3:1 (3H to 1V). A stabilization structure is proposed immediately downstream of Grand Avenue to protect the bridge piers from local scour and possible exposure of pier footings during a major flood. The elevation of the top of this structure is 1119.0 feet M.S.L.

The SLA QUASED model was used to asess the aggradation/degradation response of the New River and Skunk Creek floodway to the 100-year flood. The following sections discuss the basic model, its application and the results of the analysis.

7.2 Model Concept

7.2.1 Basic Sediment Transport Theory

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 greater rates than larger particles under the same flow conditions). The second process is the supply of sediment entering the reach. Sediment supply is determined by the nature of the channel and watershed above the study reach.

When sediment supply is less than sediment transport capacity, 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.

7.2.2 Sediment Routing Procedure

The sediment routing procedure is considered quasidynamic because flow is assumed constant for a given time interval but varies from subreach to subreach. For example, a given flood event is broken into a number of time intervals, each with a different flow, but during each interval the flow is considered steady. Hydraulic conditions were calculated using the COE HEC-2 water-surface profile program. Sediment transport by size fraction is determined during each time interval for the overbanks and main channel portions of the cross section, then summed to give the total transport capacity within a subreach.

The aggradation or degradation volume within a subreach is computed as 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. Therefore, the movable nature of the alluvial boundary is accounted for throughout the flood.

7.2.3 Armoring

For this study, the sediment particle size range in the channel is large, necessitating consideration of the armoring process for realistic determination of the river response. The QUASED model determines transport capacity of the channel by size fractions. This not only provides for more accuracy in determining sediment discharge, but also allows 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.

7.3 Model Application and Results for 100-Year Flood

7.3.1 Model Application

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

It is reasonable to assume that the sediment transport characteristics of the Agua Fria River, New River, and Skunk Creek should be similar since they belong to the same watershed system (i.e., the New River discharges directly into the Agua Fria River and Skunk Creek is the major tributary of New River). Since the QUASED model simulated the channel trends accurately for the Agua Fria River, the reliability of the model to predict the New River/Skunk Creek floodway responses to the 100-year flood should be adequate.

The sediment inflow from the ACDC for analysis of the 100-year event was based on the short term assumption of supply equal to the transport capacity of the unlined ACDC reach near the confluence (See Section 6.3.1). For model simulation, the 100-year hydrographs for the New River/Skunk Creek were discretized into twelve time steps and the study reach was subdivided into 15 subreaches. Figures 7.1a to 7.1f show the discretized hydrographs at various points along the New River and Skunk Creek. The subreaches were defined as follows:

Reach 1 : Skunk Creek, from the confluence with ACDC to 2,200 feet downstream of the confluence.

Reach 2 : Skunk Creek, from 2,200 feet downstream of the confluence with ACDC to 1,540 feet upstream of 83rd Avenue.

Reach 3 : Skunk Creek, from 1,540 feet upstream of 83rd Avenue to 83rd Avenue.

Reach 4 : Skunk Creek, from 83rd Avenue to the confluence with the New River.

Reach 5 : New River, from the confluence with Skunk Creek to 2,450 feet downstream of the confluence.

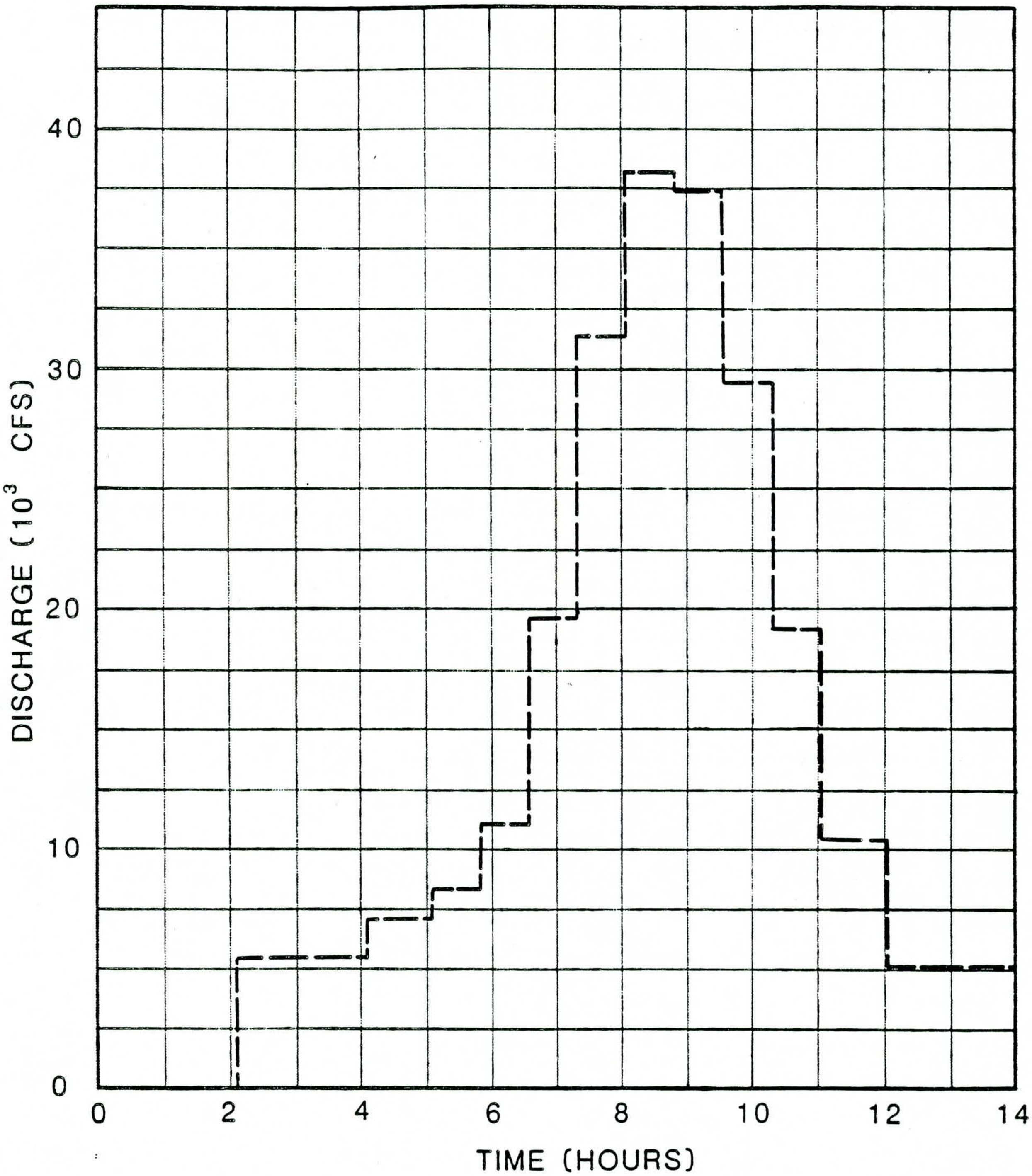


FIGURE 7.1a
DISCRETIZED 100-YR FLOOD HYDROGRAPH
AT NEW RIVER U/S OF AGUA FRIA RIVER

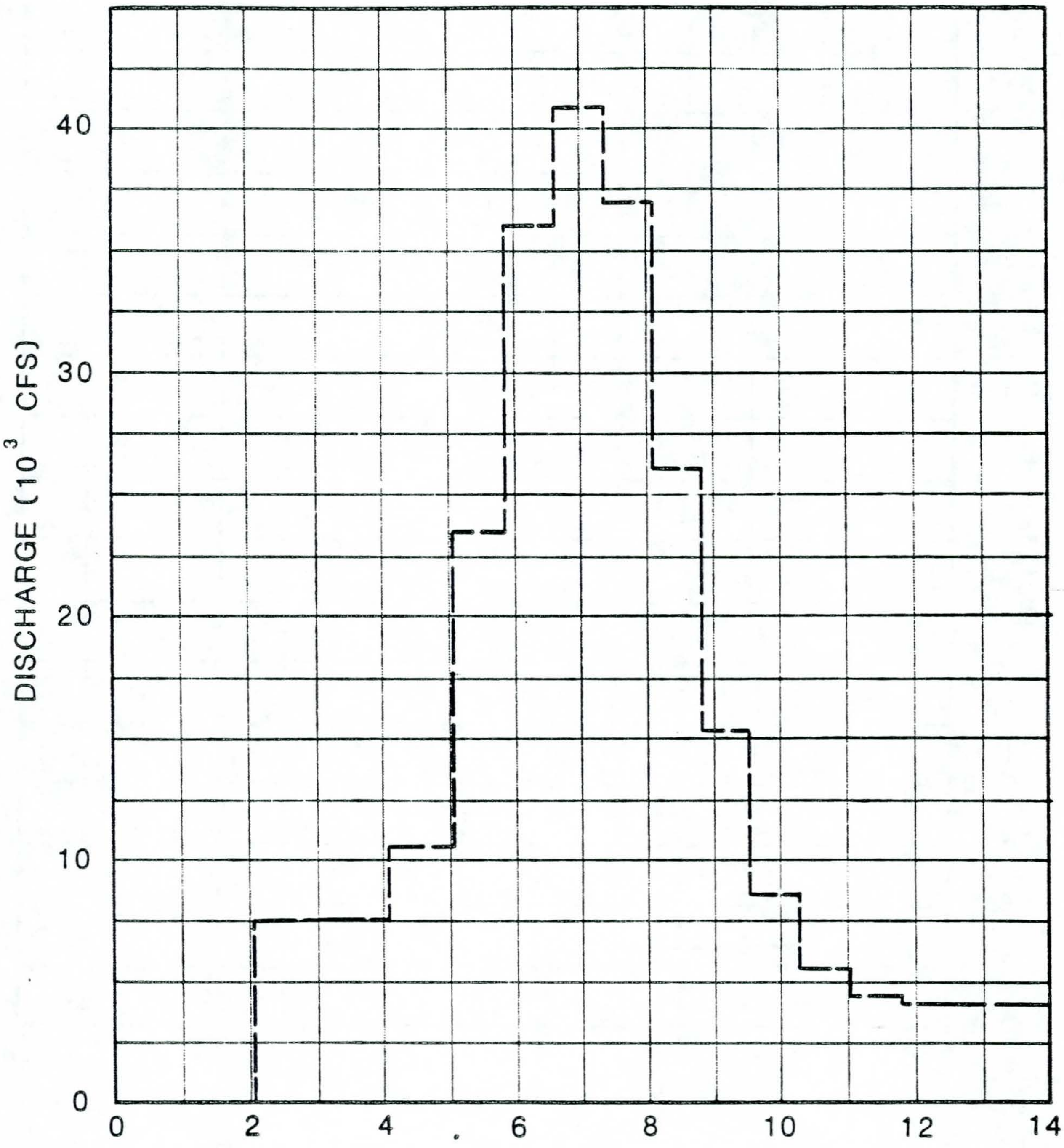


FIGURE 7.1b
DISCRETIZED 100-YR FLOOD HYDROGRAPH
AT NEW RIVER D/S OF SKUNK CREEK

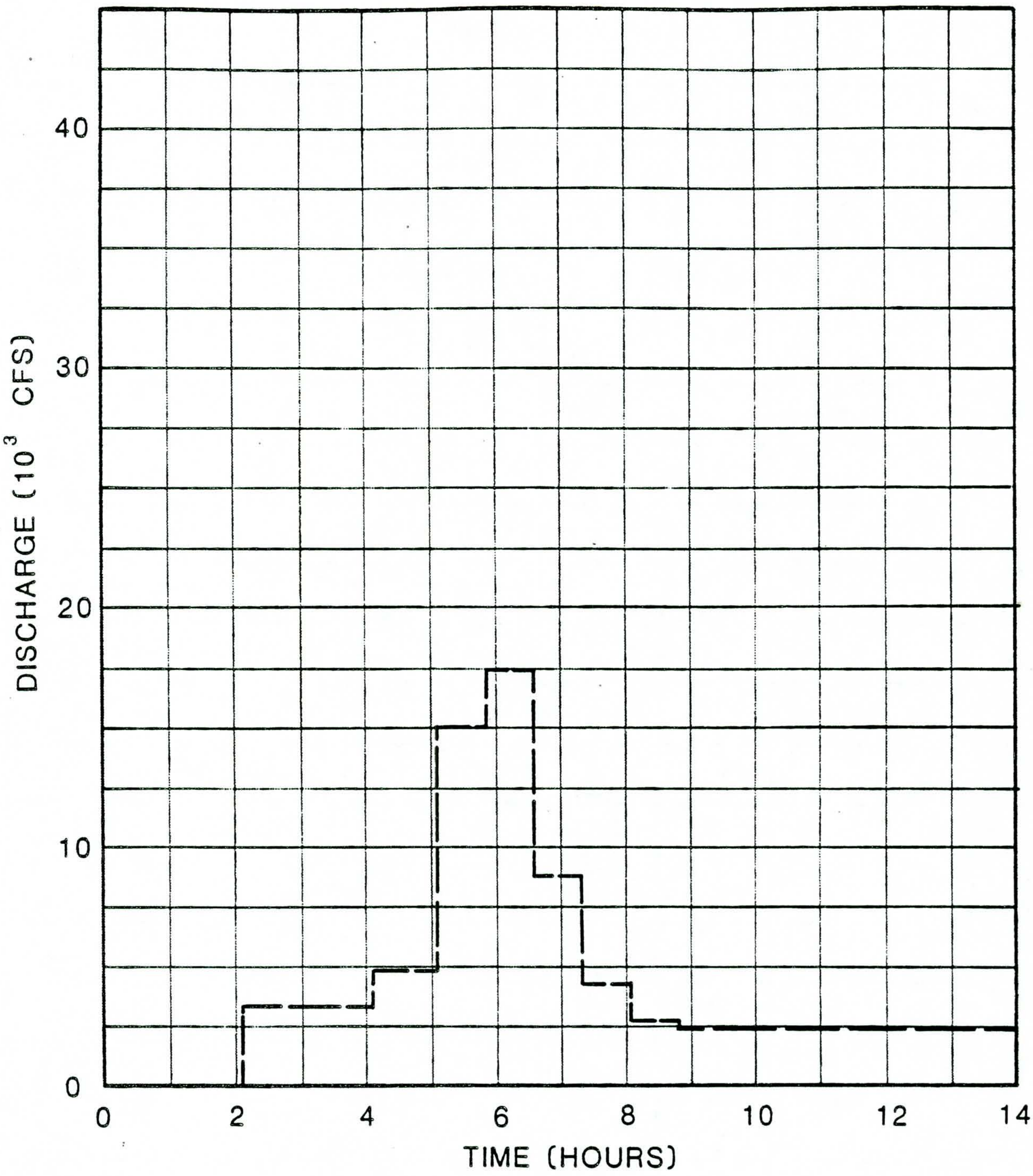


FIGURE 7.1c
DISCRETIZED 100-YR FLOOD HYDROGRAPH
AT NEW RIVER U/S OF SKUNK CREEK

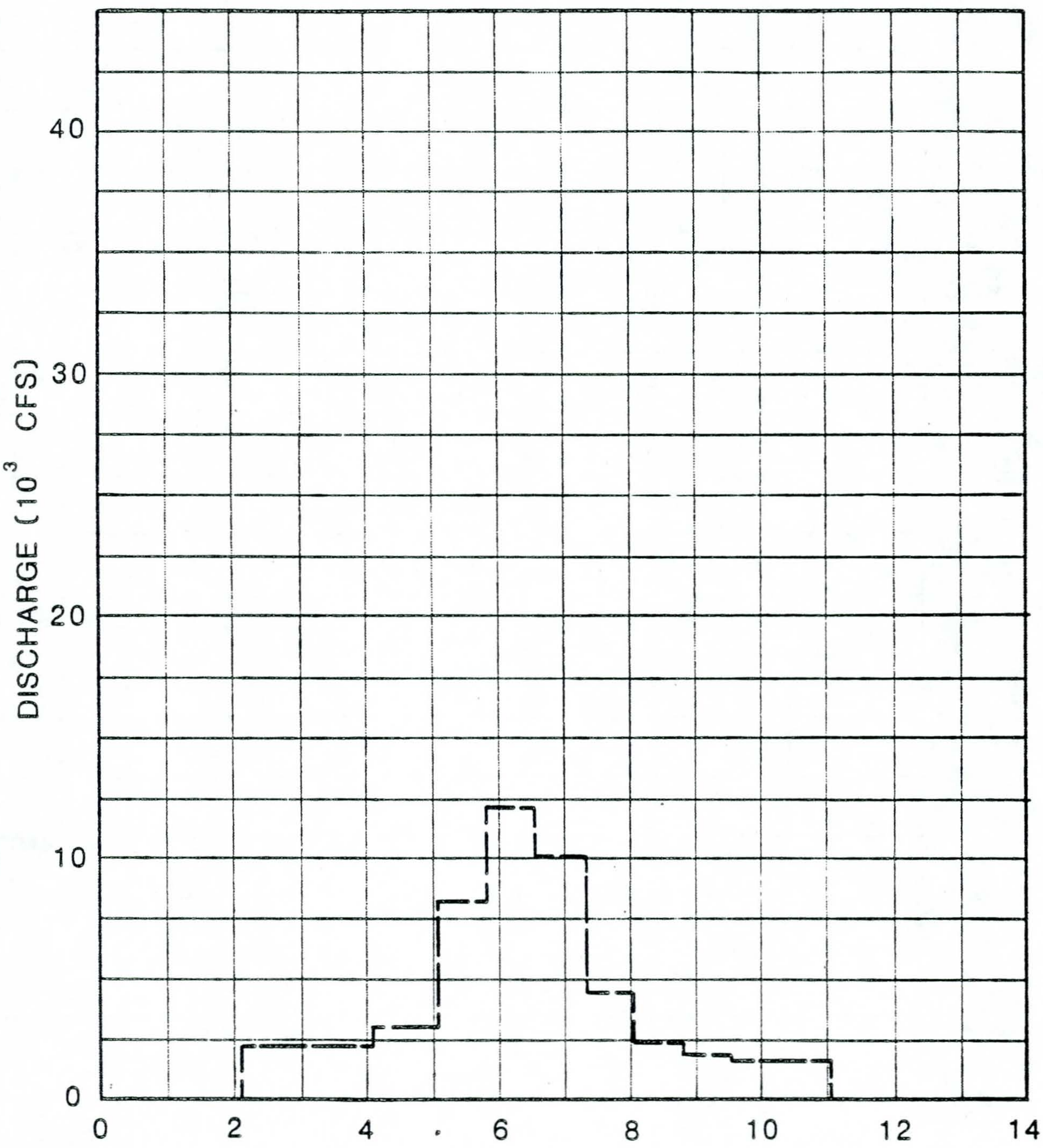


FIGURE 7.1d
DISCRETIZED 100-YR FLOOD HYDROGRAPH AT SKUNK CREEK
U/S OF ARIZONA CANAL DIVERSION CHANNEL

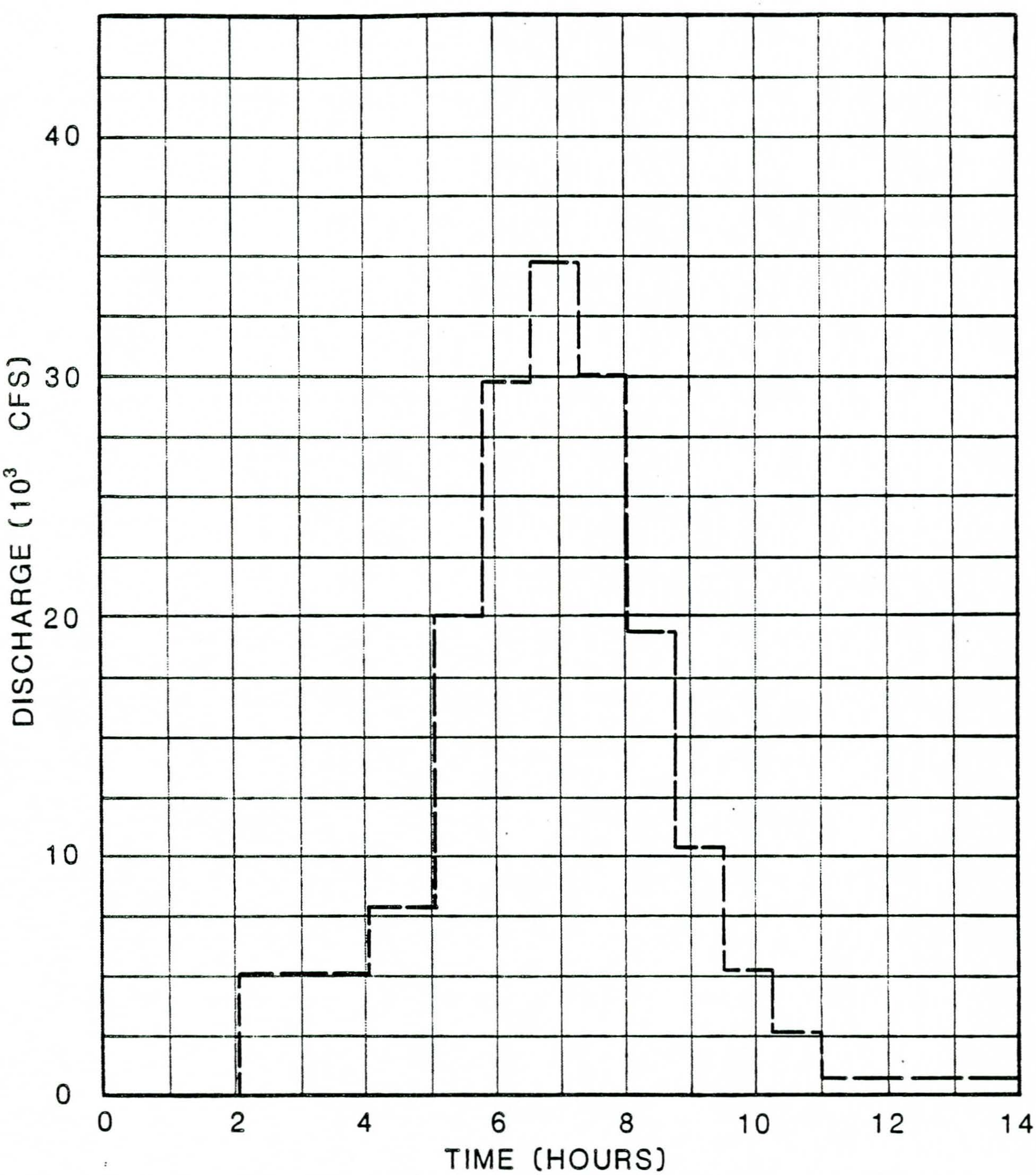


FIGURE 7.1e
DISCRETIZED 100-YR FLOOD HYDROGRAPH AT SKUNK CREEK
D/S OF ARIZONA CANAL DIVERSION CHANNEL

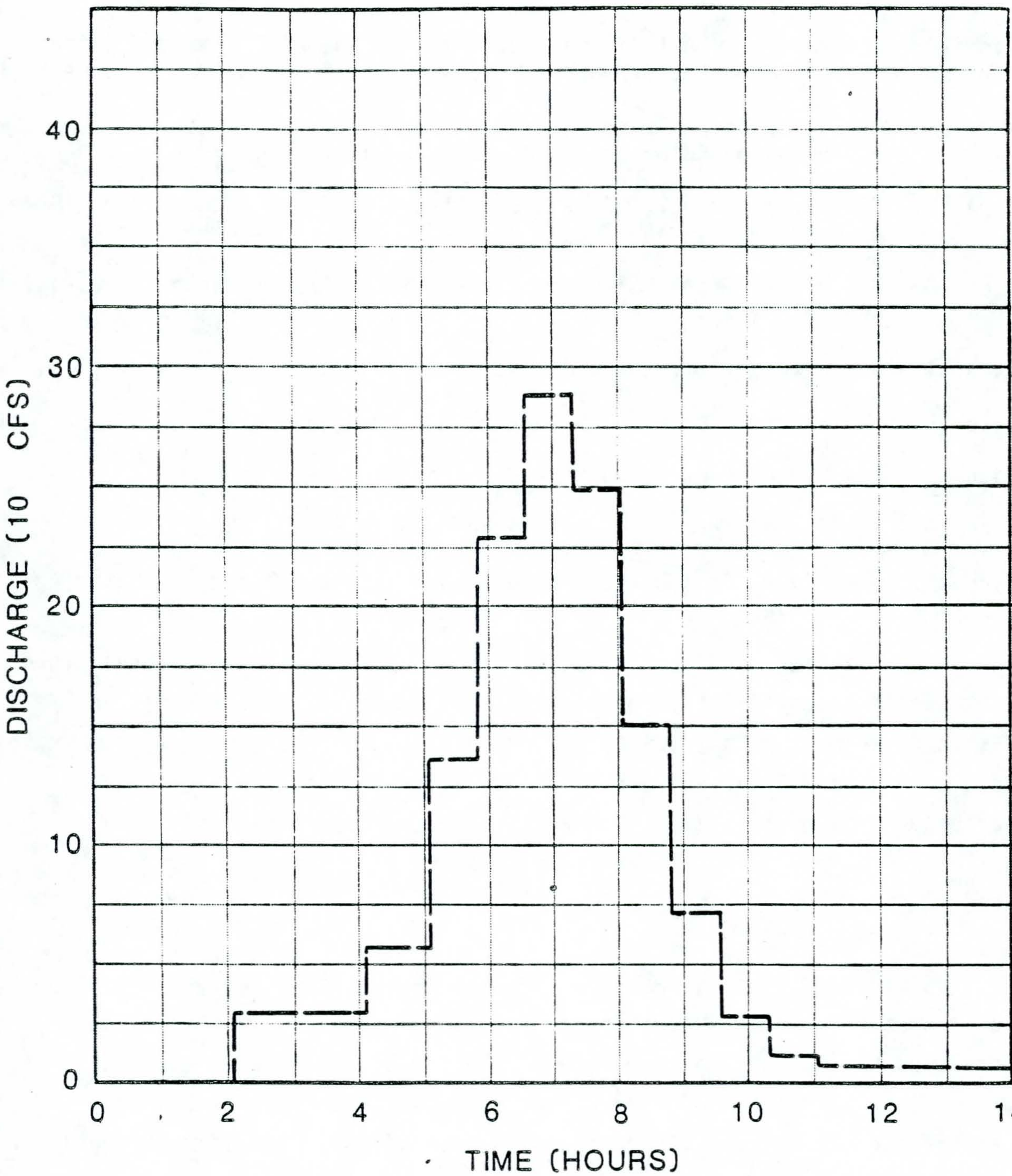


FIGURE 7.1f
DISCRETIZED 100-YR FLOOD HYDROGRAPH AT
ARIZONA CANAL DIVERSION CHANNEL U/S OF SKUNK CREEK

Reach 6 : New River, from 2,450 feet downstream of the confluence with Skunk Creek to 1,450 feet upstream of the ATSF Railroad Bridge.

Reach 7 : New River, from 1,450 feet upstream of the ATSF Railroad Bridge (upstream end of proposed COE channelization) to Grand Avenue.

Reach 8 : New River, from Grand Avenue to 1,700 feet downstream of Grand Avenue (downstream end of proposed COE channelization).

Reach 9 : New River, from 1,700 feet downstream of Grand Avenue to 3,670 feet downstream of Grand Avenue.

Reach 10: New River, from 3,670 feet downstream of Grand Avenue to Peoria Avenue.

Reach 11: New River, from Peoria Avenue to Olive Avenue.

Reach 12: New River, from Olive Avenue to Northern Avenue.

Reach 13: New River, from Northern Avenue to Glendale Avenue

Reach 14: New River, from Glendale Avenue to Bethany Home Road.

Reach 15: New River, from Bethany Home Road to the confluence with the Agua Fria River.

Reach division used in the sediment routing analysis is different than that used in the qualitative geomorphic and quantitative geomorphic analysis. This difference is mainly due to the proposed channelization work near Grand Avenue. Figure 7.2 shows reach delineation for sediment routing. Reach delineation used in qualitative and quantitative geomorphic analysis is also presented in Figure 7.2 for comparison purposes.

7.3.2 Model Results

The sediment routing results show that changes in river bed elevation, i.e., aggradation or degradation, are relatively small in most of the subreaches throughout the study area. The largest aggradation occurs in Reach 3, immediately upstream of 83rd Avenue, because of the small sediment transporting capacity of the channel in comparison to the sediment supply from the upstream reach (Reach 2). The small sediment transporting capacity of Reach 3 is the result of a backwater pool caused by the constricted flow area of 83rd Avenue bridge, while the large sediment supply is provided by the high flow velocity in the narrow channel of Reach 2. Aggradation in this reach ranges from 0.5 feet to 2.8 feet. Flood water breaks out on the right bank approximately 3,800 feet upstream of 83rd Avenue during the 100-year flood

Figure 7.2. Subreach Delineation for Sediment Routing.

Location	Cross-Section Number	Reach Number
Agua Fria River	501.45	
	13.00	
	20.00	
	26.80	
	32.60	15(12)
	38.00	
	45.00	
	51.70	
Bethany Home Road	54.00	
	59.00	
	63.00	
	70.50	
	77.00	
	84.00	
	88.70	
	95.50	14(11)
	100.70	
	107.00	
	110.70	
	117.20	
Glendale Avenue	118.00	
	120.00	
	125.00	
	131.40	
	135.00	
	140.00	
	143.00	
	146.00	
	149.60	
	158.00	
	167.00	13(10)
	170.00	
	174.10	
	178.00	
Northern Avenue	178.50	

(n) = Indicates reach number used in qualitative and quantitative geomorphic analysis for as-is conditions.

Figure 7.2 (continued). Subreach Delineation for Sediment Routing.

Location	Cross-Section Number	Reach Number
	181.60	
	185.20	
	190.00	
	193.60	
	198.40	
	202.40	
	206.00	
	210.50	
	216.00	12(9)
99th Avenue	218.00	
	220.80	
	223.00	
	229.00	
Olive Avenue	231.50	
	232.60	
	235.00	
	238.80	
	240.80	
	245.80	
	251.00	
	256.00	
	258.80	
	262.00	11(8)
	265.00	
	270.50	
	273.50	
	277.00	
	281.80	
	288.00	
Peoria Avenue	290.00	
	291.00	
	293.00	
	297.00	10(7)
	302.00	
	305.00	
	307.00	

(n) = Indicates reach number used in qualitative and quantitative geomorphic analysis for as-is conditions.

Figure 7.2 (continued). Subreach Delineation for Sediment Routing.

Location	Cross-Section Number	Reach Number
	311.50	
	314.00	
	318.00	9(7)
	323.00	
	328.00	
	330.00	
	334.00	
	343.00	8(7)
	344.90	
Grand Avenue	345.00	
	346.00	
ATSF Railroad	346.50	
	347.00	7(7)
	349.00	
	352.00	
	358.00	
	365.00	
	370.00	
	373.00	
	377.00	
	384.00	
	388.00	
	391.50	
	395.00	
	402.00	6(6)
	406.00	
	409.00	
Thunderbird Road	409.01	
	409.82	
	410.00	
	412.00	
	418.00	
	423.00	

(n) = Indicates reach number used in qualitative and quantitative geomorphic analysis for as-is conditions.

Figure 7.2 (continued). Subreach Delineation for Sediment Routing.

Location	Cross-Section Number	Reach Number
	425.00	
	428.00	
	434.00	
	440.00	5(5)
	443.20	
New River	449.40	
	0.00	
	4.50	
	10.00	
	15.00	4(4)
	19.00	
	21.00	
83rd Avenue	23.00	
	24.00	
	26.00	
	28.00	3(3)
	30.00	
	35.00	
	41.00	
	45.00	
	48.00	2(2)
	55.00	
	60.00	
	65.00	
	71.50	
	75.00	1(1)
	78.00	
ACDC	84.00	

(n) = Indicates reach number used in qualitative and quantitative geomorphic analysis for as-is conditions.

event. Sediment carried by this slow moving water will deposit on the overbank area along Skunk Creek, causing slight aggradation.

The largest degradation occurs in Reaches 4 and 5 due to reduced sediment supply from Reach 3 and a higher transport capacity of the entrenched channel in these reaches. Degradation in these reaches ranges from 0.4 feet to 1.5 feet. Because of the proposed stabilization structure located at the downstream end of Reach 7, it was assumed that neither aggradation nor degradation will occur in this reach, therefore the original cross-sectional geometry was not altered. An appreciable amount of aggradation occurs in Reach 9, immediately downstream of the proposed channel, due to the large supply of sediment coming from channelized reach and the small sediment transport capacity of the reach. The small sediment transport capacity in Reach 9 is due to the low flow velocity caused by the wider channel found in this reach. The approximate maximum channel width in this reach is as much as 1,300 feet.

Table 7.1 shows the river bed response compared to the original channel invert elevation at various time periods during the hydrograph (near the peak discharge, at maximum change and at the end of the 100-year flood event). The peak discharge does not occur at the same time step for all subreaches because the study reach is relatively long (about 10 miles) and due to flood attenuation processes. As a result, the occurrence of the peak discharge ranges from the fourth time step for the upstream subreach (Reach 1) to seventh time step for the downstream subreach (Reach 15). Based on changes of channel invert elevation and center of focus of this study, the fifth time step was selected as the representative time step of peak discharge for the entire study area. It is also important to realize that the maximum aggradation/degradation does not always occur at the peak discharge, as indicated by the results given in Table 7.1. Figure 7.3 compares the bed response to the original channel invert elevation at the end of the 100-year flood event.

QUASED has the limitation of not modeling local scour at river crossings. Thus, the recommendations regarding local scour protection of bridges do not change from the results of the quantitative geomorphic analysis (Section 6.8).

7.4 Annual Aggradation/Degradation Analysis

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

Table 7.1. Bed Response to the 100-Year Flood as Simulated by QUASED.

Cross-Section Number	Original Bed Elevation	Agg/Deg Near Peak (ft)	Max. Agg/Deg feet	Max. Agg/Deg Timestep	Agg/Deg At end (ft)
84.00	1180.9	0.02	-0.12	3	-0.01
78.00	1180.0	-0.03	-0.20	3	-0.06
75.00	1179.3	-0.08	-0.32	3	-0.16
71.50	1178.0	-0.03	-0.32	3	-0.11
65.00	1175.5	-0.07	-0.43	3	-0.20
60.00	1174.0	-1.08	-1.36	12	-1.36
55.00	1172.0	-1.00	-1.31	12	-1.31
48.00	1169.5	-0.71	-0.92	12	-0.92
45.00	1168.0	-0.63	-0.81	12	-0.81
41.00	1167.0	-0.76	-0.94	12	-0.94
35.00	1164.4	1.87	2.82	12	2.82
30.00	1163.2	1.06	1.61	12	1.61
28.00	1161.5	1.47	2.22	12	2.22
26.00	1161.8	1.38	2.00	12	2.00
24.00	1161.8	0.39	0.48	12	0.48
23.00	1161.4	0.39	0.48	12	0.48
21.00	1160.0	-0.61	-1.07	12	-1.07
19.00	1161.2	-0.53	-0.93	12	-0.93
15.00	1160.7	-0.55	-0.97	12	-0.97
10.00	1158.0	-0.85	-1.52	12	-1.52
4.50	1156.0	-0.40	-0.68	12	-0.68
0.00	1154.0	-0.21	-0.38	12	-0.38
449.40	1152.0	-0.29	-0.67	12	-0.67
443.20	1151.4	-0.39	-0.88	12	-0.88
440.00	1150.4	-0.49	-1.04	12	-1.04
434.00	1150.0	-0.64	-1.48	12	-1.48
428.00	1145.2	-0.42	-0.97	12	-0.97
425.00	1142.0	-0.35	-0.82	12	-0.82
423.00	1140.0	0.07	0.17	12	0.17
418.00	1138.0	0.08	0.19	12	0.19
412.00	1136.7	0.07	0.20	12	0.20
410.00	1135.6	0.06	0.18	12	0.18
409.82	1135.6	0.06	0.18	12	0.18
409.01	1135.2	0.08	0.19	12	0.19
409.00	1135.2	0.06	0.16	12	0.16
406.00	1134.5	0.04	0.11	12	0.11
402.00	1133.8	0.04	0.11	12	0.11
395.00	1132.8	0.05	0.12	12	0.12
391.50	1132.2	0.09	0.21	12	0.21
388.00	1132.4	0.07	0.19	12	0.19
384.00	1130.0	0.10	0.24	12	0.24
377.00	1128.4	0.07	0.17	12	0.17
373.00	1127.1	0.04	0.13	12	0.13
370.00	1127.3	0.05	0.14	12	0.14

Table 7.1 (continued). Bed Response to the 100-Year Flood as Sumulated by QUASED.

Cross-Section Number	Original Bed Elevation	Agg/Deg Near Peak (ft)	Max. Agg/Deg feet	Max. Agg/Deg Timestep	Agg/Deg At end (ft)
365.00	1127.4	0.06	0.16	12	0.16
358.00	1125.6	0.00	0.00	12	0.00
352.00	1121.3	0.00	0.00	12	0.00
349.00	1120.3	0.00	0.00	12	0.00
347.00	1119.6	0.00	0.00	12	0.00
346.50	1119.5	0.00	0.00	12	0.00
346.00	1119.3	0.00	0.00	12	0.00
345.00	1119.0	0.00	0.00	12	0.00
344.90	1116.3	-0.73	-0.89	7	-0.82
343.00	1115.7	-0.65	-0.80	7	-0.74
334.00	1112.6	-0.65	-0.80	7	-0.74
330.00	1111.3	-0.58	-0.72	7	-0.67
328.00	1110.0	0.96	1.22	7	1.15
323.00	1108.7	0.56	0.71	7	0.65
318.00	1106.0	0.62	0.79	7	0.71
314.00	1104.0	0.74	0.94	7	0.86
311.50	1102.0	1.13	1.43	7	1.31
307.00	1102.0	-1.17	-1.24	6	-1.08
305.00	1101.6	-1.26	-1.34	6	-1.17
302.00	1100.0	-1.28	-1.36	6	-1.18
297.00	1099.6	-0.93	-0.99	6	-0.88
293.00	1096.0	-1.08	-1.13	6	-1.00
291.00	1094.3	-0.89	-0.95	6	-0.85
290.00	1094.0	-0.93	-0.99	6	-0.88
288.00	1094.0	0.11	0.17	4	-0.14
281.80	1092.0	0.17	0.25	4	-0.18
277.00	1090.7	0.14	0.21	4	-0.18
273.50	1090.0	0.09	0.13	4	-0.11
270.50	1088.0	0.09	0.14	4	-0.13
265.00	1086.5	0.06	0.09	4	-0.08
262.00	1086.5	0.11	0.17	4	-0.15
258.80	1085.5	0.10	0.15	4	-0.14
256.00	1082.7	0.14	0.20	4	-0.14
251.00	1082.0	0.13	0.17	4	-0.06
245.80	1081.1	0.14	0.19	4	-0.09
240.80	1080.6	0.19	0.24	4	-0.04
238.80	1080.6	0.13	0.17	4	-0.07
235.00	1080.0	0.11	0.16	4	-0.13
232.60	1080.0	0.13	0.19	4	-0.16
231.50	1080.0	0.19	0.33	12	0.33
229.00	1079.8	0.10	0.17	12	0.17
223.00	1074.0	0.13	0.22	12	0.22
220.80	1073.0	0.10	0.19	12	0.19
218.00	1072.0	0.09	0.16	12	0.16

Table 7.1 (continued). Bed Response to the 100-Year Flood as Sumulated by QUASED.

Cross-Section Number	Original Bed Elevation	Agg/Deg Near Peak (ft)	Max. Agg/Deg feet	Max. Agg/Deg Timestep	Agg/Deg At end (ft)
216.00	1072.0	0.14	0.24	12	0.24
210.50	1071.0	0.16	0.28	12	0.28
206.00	1073.2	0.21	0.34	12	0.34
202.40	1072.2	0.18	0.31	12	0.31
198.40	1070.8	0.15	0.25	12	0.25
193.60	1070.0	0.16	0.26	12	0.26
190.00	1067.6	0.11	0.18	12	0.18
185.20	1067.7	0.17	0.30	12	0.30
181.60	1066.2	0.18	0.32	12	0.32
178.50	1067.7	-0.01	-0.10	12	-0.10
178.00	1060.0	-0.03	-0.20	12	-0.20
174.10	1059.6	-0.06	-0.40	12	-0.40
170.00	1059.3	-0.04	-0.26	12	-0.26
167.00	1059.0	-0.03	-0.24	12	-0.24
158.00	1059.6	-0.02	-0.13	12	-0.13
149.60	1060.0	-0.02	-0.13	12	-0.13
146.00	1058.0	-0.02	-0.15	12	-0.15
143.00	1058.0	-0.01	-0.10	12	-0.10
140.00	1054.0	-0.01	-0.09	12	-0.09
135.00	1051.9	-0.01	-0.11	12	-0.11
131.40	1050.1	-0.01	-0.11	12	-0.11
125.00	1046.0	-0.02	-0.13	12	-0.13
120.00	1046.0	-0.02	-0.15	12	-0.15
118.00	1047.5	-0.01	-0.09	12	-0.08
117.20	1046.2	0.11	0.19	7	0.10
110.70	1041.3	0.11	0.19	7	0.13
107.00	1038.0	0.08	0.14	7	0.09
100.70	1034.7	0.15	0.26	7	0.16
95.50	1034.0	0.10	0.17	7	0.12
88.70	1034.0	0.09	0.15	7	0.10
84.00	1035.7	0.08	0.14	7	0.09
77.00	1036.0	0.15	0.24	7	0.16
70.50	1034.0	0.09	0.14	7	0.10
63.00	1032.0	0.10	0.16	7	0.12
59.00	1030.7	0.07	0.12	7	0.08
54.00	1028.0	0.00	-0.03	12	-0.03
51.70	1027.4	0.00	-0.04	12	-0.04
45.00	1026.8	0.00	-0.04	12	-0.04
38.00	1024.9	0.00	-0.03	12	-0.03
32.60	1024.0	0.00	-0.05	12	-0.05
26.80	1023.7	0.00	-0.03	12	-0.03
20.00	1023.4	0.00	-0.05	12	-0.05
13.00	1023.5	0.00	-0.04	12	-0.04
501.45	1019.2	0.00	0.00	12	0.00

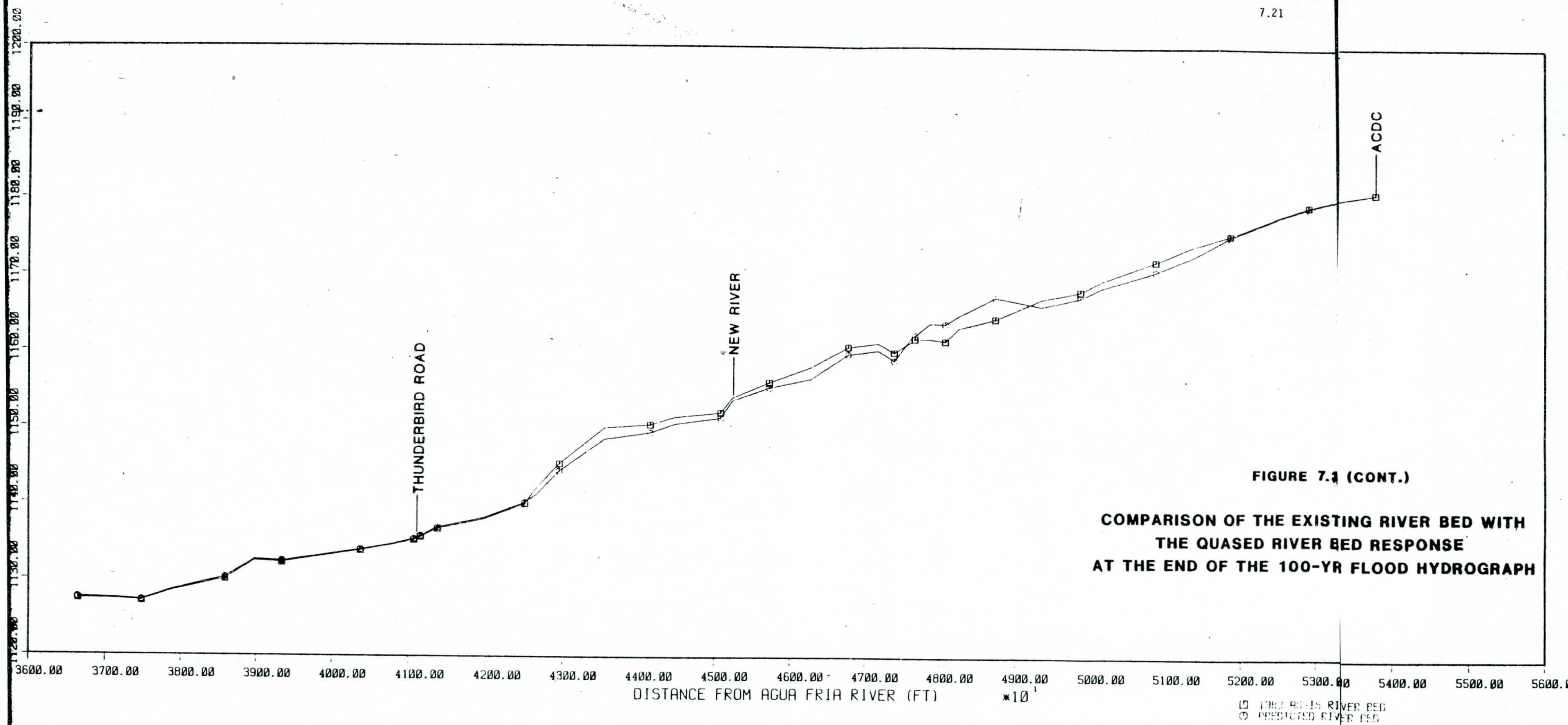


FIGURE 7.1 (CONT.)

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

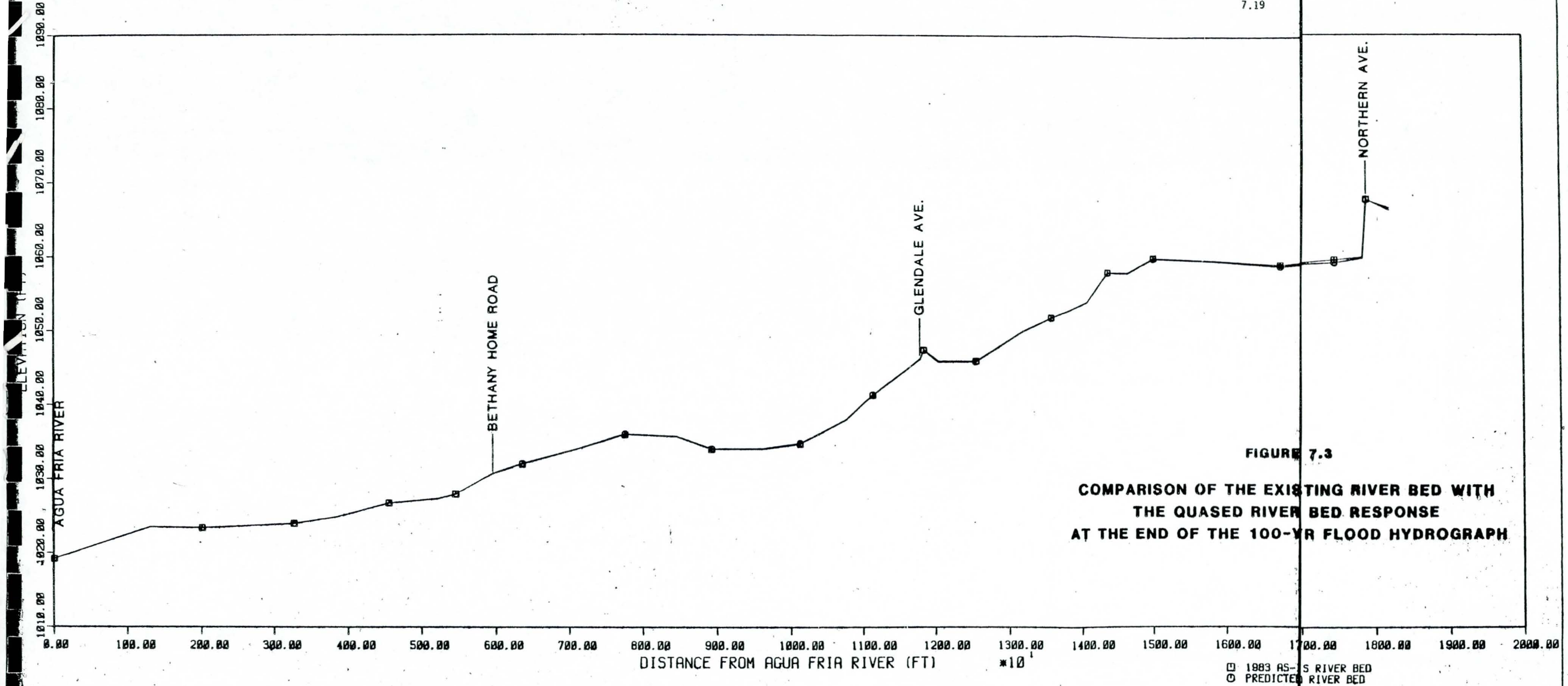


FIGURE 7.3

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

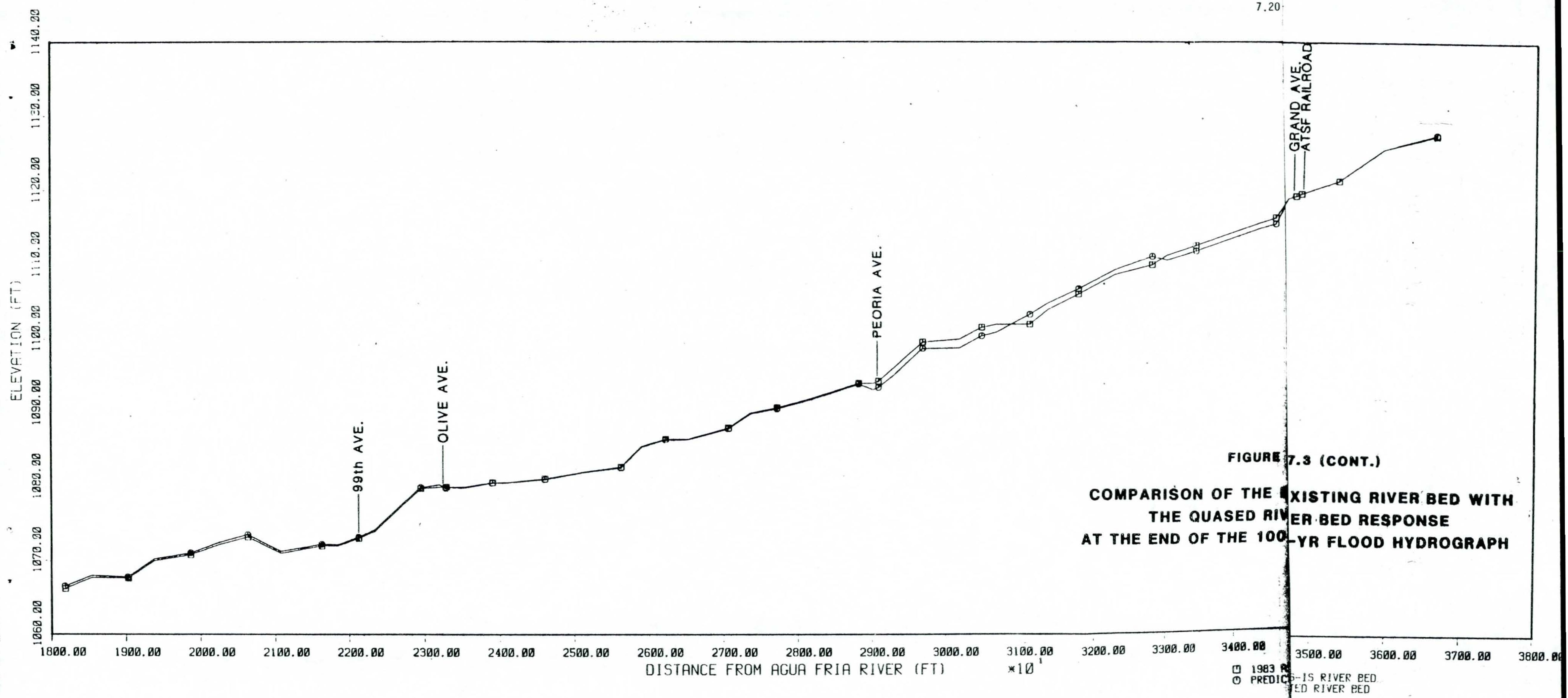


FIGURE 7.3 (CONT.)

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

1. Develop a sediment rating curve from the computed sediment discharges and water discharges from the 100-year QUASED run for each of the 15 reaches.

2. Use the sediment rating curves to determine the sediment yields for the 2-, 5-, 25-, 50-, and 100-year flood events.

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

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

The developed sediment rating curves were of the following form:

$$Q_s = aQ^b \tag{7.1}$$

where Q_s is the sediment transport capacity (without very fine sand and fine sand) in cfs, Q is the water discharge in cfs, and a and b are the best fit coefficient and exponent. Table 7.2 lists the coefficients and exponents, a and b, for each of the 15 reaches.

Sediment yields for the 10-, 25-, 50-, and 100-year floods were determined by applying Equation 1 to the discretized flood hydrographs. The sediment yields for the 2- and 5-year flood events were extrapolated (log-log extrapolation) using the results of the 10-, 25-, 50 and 100-year flood events. The average annual sediment yield was then computed using the weighted incremental probability of occurence of the floods defined by Equation 6.5.

Table 7.3 summarizes the average annual sediment yields for each reach and compares the yields with the supply reach to determine the net aggradation/degradation response. Results of this analysis agree with the quantitative geomorphic analysis. The net average annual degradation/aggradation is relatively small in all reaches and the largest net change is in Reach 3 with an estimated average annual aggradation of 0.3 feet.

Ultimately, sediment supply from the ACDC will reduce when the unlined channel immediately upstream of the confluence of the ACDC with Skunk Creek approaches an equilibrium condition (See Section 6.3.1). Table 7.4 shows the average annual aggradation/degradation response under the condition of reduced sediment supply from ACDC. In this case, Reaches 1, 2, 4, and 5 become degradational and the aggradational amounts in remaining reaches are smaller.

Table 7.2. Coefficients and Exponents of Sediment Rating Curves for Each Reach of the New River and Skunk Creek.

Reach	a	b
1	1.366×10^{-5}	1.448
2	2.456×10^{-6}	1.662
3	1.310×10^{-7}	1.838
4	4.095×10^{-5}	1.318
5	2.948×10^{-5}	1.387
6	5.018×10^{-7}	1.767
7	5.018×10^{-7}	1.767
8	1.442×10^{-8}	2.140
9	2.816×10^{-4}	1.075
10	1.633×10^{-5}	1.407
11	1.398×10^{-6}	1.675
12	4.263×10^{-7}	1.746
13	1.524×10^{-6}	1.639
14	1.388×10^{-7}	1.864
15	2.314×10^{-7}	1.821

Table 7.3. Average Annual Aggradation/Degradation Response for Study Reach.

Reach	Sediment Yield (ac-ft)	Degradation/ Aggradation (ac-ft)	Length (ft)	Average Depth* of Deg/Agg (ft)
Supply: Skunk Creek	1.04	-	-	-
ACDC	1.47	-	-	-
1	1.42	1.09	2345	0.1
2	1.75	0.76	2545	0.1
3	0.47	2.04	1540	0.3
4	1.36	1.15	2325	0.1
Supply: New River	0.53	-	-	-
5	2.47	0.57	2590	<0.1
6	1.37	1.67	6270	<0.1
7	1.37	1.67	1720	0.1
8	1.37	1.67	1690	0.1
9	1.55	1.49	1975	0.1
10	1.63	1.41	2000	0.1
11	1.63	1.41	5680	<0.1
12	1.02	2.02	5210	<0.1
13	1.30	1.74	6235	<0.1
14	0.95	2.09	6095	<0.1
15	1.07	1.97	5695	<0.1

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

Table 7.4. Average Annual Aggradation/Degradation Response for Study Reach with 25 Percent Reduction in Sediment Supply.

Reach	Sediment Yield (ac-ft)	Degradation/ Aggradation (ac-ft)	Length (ft)	Average Depth* of Deg/Agg (ft)
Supply: Skunk Creek	0.78	-	-	-
ACDC	1.10	-	-	-
1	1.42	0.46	2345	< 0.1
2	1.75	0.13	2545	< 0.1
3	0.47	1.41	1540	0.2
4	1.36	0.52	2325	< 0.1
Supply: New River	0.40	-	-	-
5	2.47	-0.19	2590	>-0.1
6	1.37	0.91	6270	< 0.1
7	1.37	0.91	1720	< 0.1
8	1.37	0.91	1690	< 0.1
9	1.55	0.73	1975	< 0.1
10	1.63	0.65	2000	< 0.1
11	1.63	0.65	5680	< 0.1
12	1.02	1.26	5210	< 0.1
13	1.30	0.98	6235	< 0.1
14	0.95	1.33	6095	< 0.1
15	1.07	1.21	5695	< 0.1

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

VIII. SUMMARY AND CONCLUSIONS

A sediment transport study has been completed for the New River and Skunk Creek. Qualitative and quantitative geomorphic analyses were conducted on the as-is conditions (prior to September, 1983) to establish the characteristics of the fluvial system and to assist in developing flood control alternatives. Based on results of these analyses and other relevant information, the COE developed the preferred flood control alternative.

QUASED, the SLA water and sediment routing model, was then executed for the 100-year flood using the 100-year floodway as developed by the COE. The purpose of this analysis was to simulate the channel bed responses under project conditions of the New River from its confluence with the Agua Fria River to its confluence with Skunk Creek, and Skunk Creek from its confluence with the New River to Bell Road. The bed response to the 100-year flood showed relatively small aggradation/degradation amounts. The largest degradation occured in the reach immediately upstream of the confluence of Skunk Creek with the New River, where scour depths ranged from 0.4 feet to 1.5 feet. The largest aggradation occured in the reach immediately upstream of 83rd Avenue where sediment deposition ranged from 0.5 feet to 2.8 feet.

Sediment rating equations were then established from the results of the QUASED simulation of the 100-year flood. Utilizing these sediment rating equations and an incremental probabilistic weighting scheme, average annual sediment yields were computed for each of the reaches. Comparison of the average annual sediment yields to initial sediment supply conditions indicate only slight aggradational tendencies in all reaches as a result of the COE floodway. Therefore, minimal sediment removal is expected throughout the system. Over long term conditions, the proposed Cudia City Wash and Cave Creek sedimentation basins will significantly reduce sediment supply to Skunk Creek/New River. This reduction in supply is noticed in the reaches downstream of the confluence of Skunk Creek with ACDC, reversing the initial aggradation/degradation trends.