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PIPELINE DESIGN FLOOD REPORT

PREPARED FOR
NORTHWEST ALASKAN PIPELINE COMPANY

PREPARED BY
NORTHERN TECHNICAL SERVICES

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PIPELINE DESIGN FLOOD REPORT

SUMMARY

The purpose of this report is to present a methodology for the estimation of a Pipeline Design Flood (PDF) for streams crossed by the Northwest Alaskan Pipeline. As examples, the PDF has been computed for nine streams in the Tanana River basin which will be crossed by the Northwest Alaskan Pipeline. The drainage areas and PDF magnitudes of these nine streams are as follows:

Stream No.	Stream	Drainage Area (mi^2)	PDF Magnitude (cfs)
5-172	Gerstle River	209.5	24,200
5-175	Johnson River	371.5	37,400
5-185	Bear Creek	86.1	11,800
5-187	Robertson River	572.3	49,000
6-199	Cathedral Rapids #1	9.7	3,100
6-201	Yerrick Creek	33.9	7,000
6-205	Tok River	907.	64,600
6-207	Upper Tanana River	6800.	136,000
6-219	Gardiner Creek	323.5	21,800

Procedures described in this report were used for all streams except the Upper Tanana River. A separate approach was used for the Upper Tanana River. For some basins, the PDF is primarily a snowmelt flood. Those stations usually are the larger basins, and are to be treated in a manner similar to the Upper Tanana Basin. Basins with smaller drainage areas usually have major floods which result from intense rainstorms, such as that for Fairbanks in August 1967. Such basins are to have their PDF computed by unit hydrograph analysis and the use of the HEC-1 model.

1.0 INTRODUCTION

The purpose of this report is to present the procedures and results of studies conducted to establish discharges upon which to base pipeline river crossing design for nine (9) selected streams crossed by the Northwest Alaskan Gas Pipeline.

The procedure starts with hydrograph reconstitutions. The results of this work have been reported in "Completion Report, Hydrograph Reconstitution," Rev. 2, 1981 (Reference 1). Basin characteristics were developed which, when combined with reconstitution relationships, gave the necessary hydrographic parameters. The hydrographic parameters used in conjunction with probable maximum precipitation estimates and time-area curves provide an estimate of the pipeline design flood (PDF) magnitude.

The design of a structure or facility crossing a stream is greatly influenced by the amount of water expected to flow in that stream.

This amount of water or discharge has a direct influence on water level and scour at and near the proposed structure or facility.

Depending upon the type of structure, a failure will have varying degrees of environmental, economic and safety impact. For this reason, not all structures are designed assuming the same design flood; for example, the design flood for a pipeline may not be the same as for a bridge. Therefore the design floods for the proposed gas pipeline, access road bridges, highway bridges, and floodplain material site analysis were sized differently. In this report, the development of the pipeline design flood (PDF) is presented.

The pipeline design flood is defined as "an estimate representing flood discharges that may be expected from the most severe combination of meteorological and hydrologic events that are considered reasonably characteristic of the geographical region involved, excluding extremely rare combinations" (U.S. Army Corps of Engineers, 1962 - Reference 2). It is derived by applying the most severe precipitation or snowmelt conditions to a mathematical model of the runoff characteristics of the particular watershed involved. This flood is not developed from a statistical analysis of the frequency of historic floods and no probability can be realistically attached to such an event. A discussion of the data required to derive the PDF follows.

2.0 DATA REQUIREMENT AND PREPARATION

2.1 Pipeline Design Storm

A general estimate of storm precipitation for drainage areas up to 400 square miles is provided by the U.S. Weather Bureau (Miller, 1963 and Miller, 1965 - References 3 and 4). Additionally, precipitation and snowmelt sequences are available for floods that have been developed by the U.S. Army Corps of Engineers (References 5 and 6), The U.S. Department of Interior (Childers et al, 1972 - Reference 7), the Bureau of Reclamation (Miller, 1959 - Reference 8), and the U.S. Weather Bureau/Service (References 9 and 10) for various projects. By using those data sources, the probable maximum precipitation (PMP) in basins along the pipeline route were estimated.

The step-by-step procedure for estimating probable maximum precipitation follows:

A. Drainage Divide Outline

Drainage divides were drawn on U.S. Geological Survey (USGS) topographic contour maps. The scale used was generally 1:63,360 for small drainage areas, and 1:250,000 for large drainage basins. Topographic maps at a scale of

1:50,000 from the Canada Map Office were used for streams with portions of their basins in Canada. Drainage divides for nine streams in the Tanana River basin, results of which are presented earlier in the summary, are shown in Figures 1 through 9.

B. Determination of the Drainage Area of the Basin

Standard planimeter procedures were used to measure total basin areas (U.S. Department of Interior, 1961 - Reference 11).

C. Determination of the Location of a Point Representative of the Basin

The point selected as being representative should be at about the mean basin elevation near the basin centroid. In this study, the centroid was used. The procedures used for locating the centroid of the drainage area are given in Appendix 1.

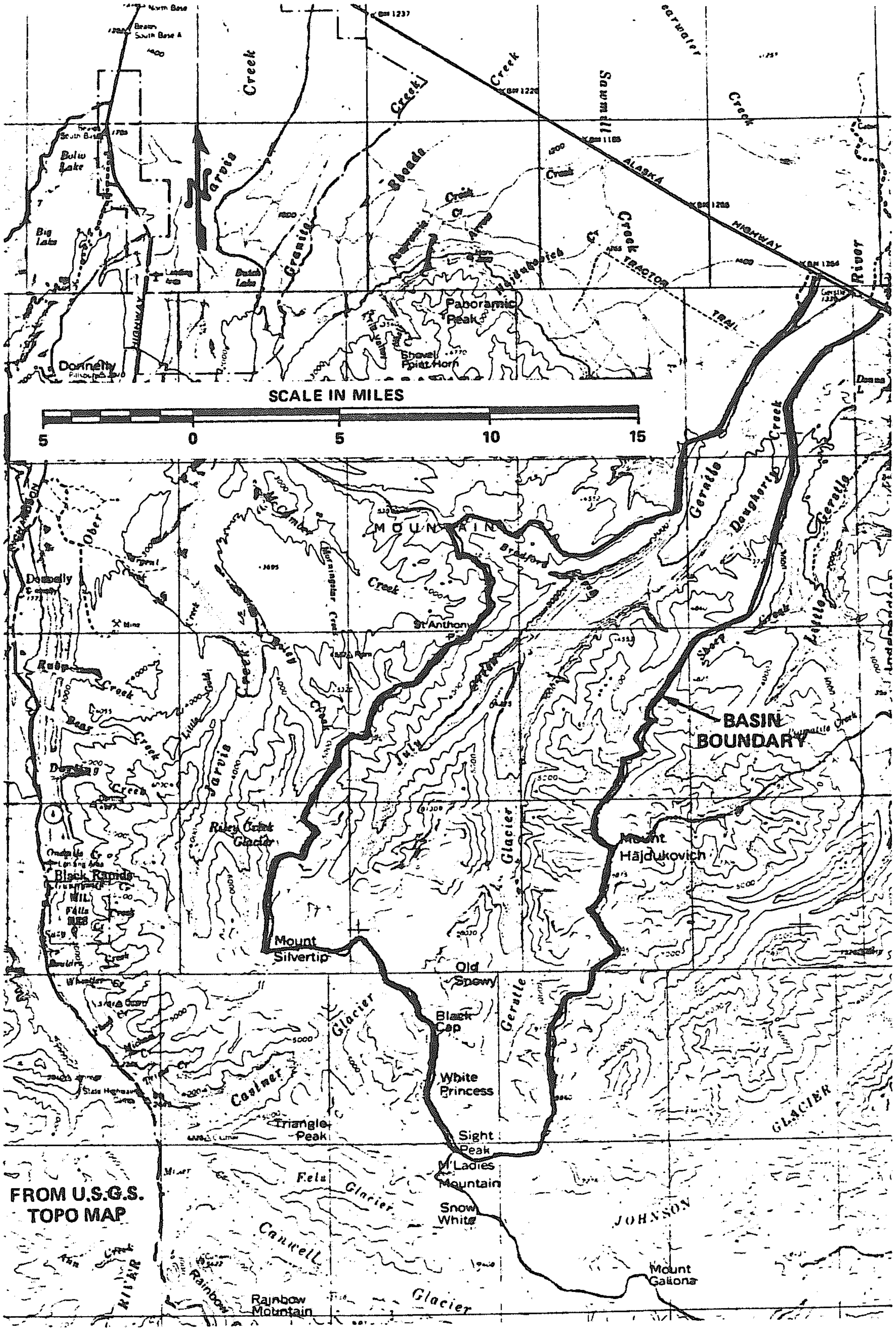

FIGURE 1 GERSTLE RIVER DRAINAGE BASIN

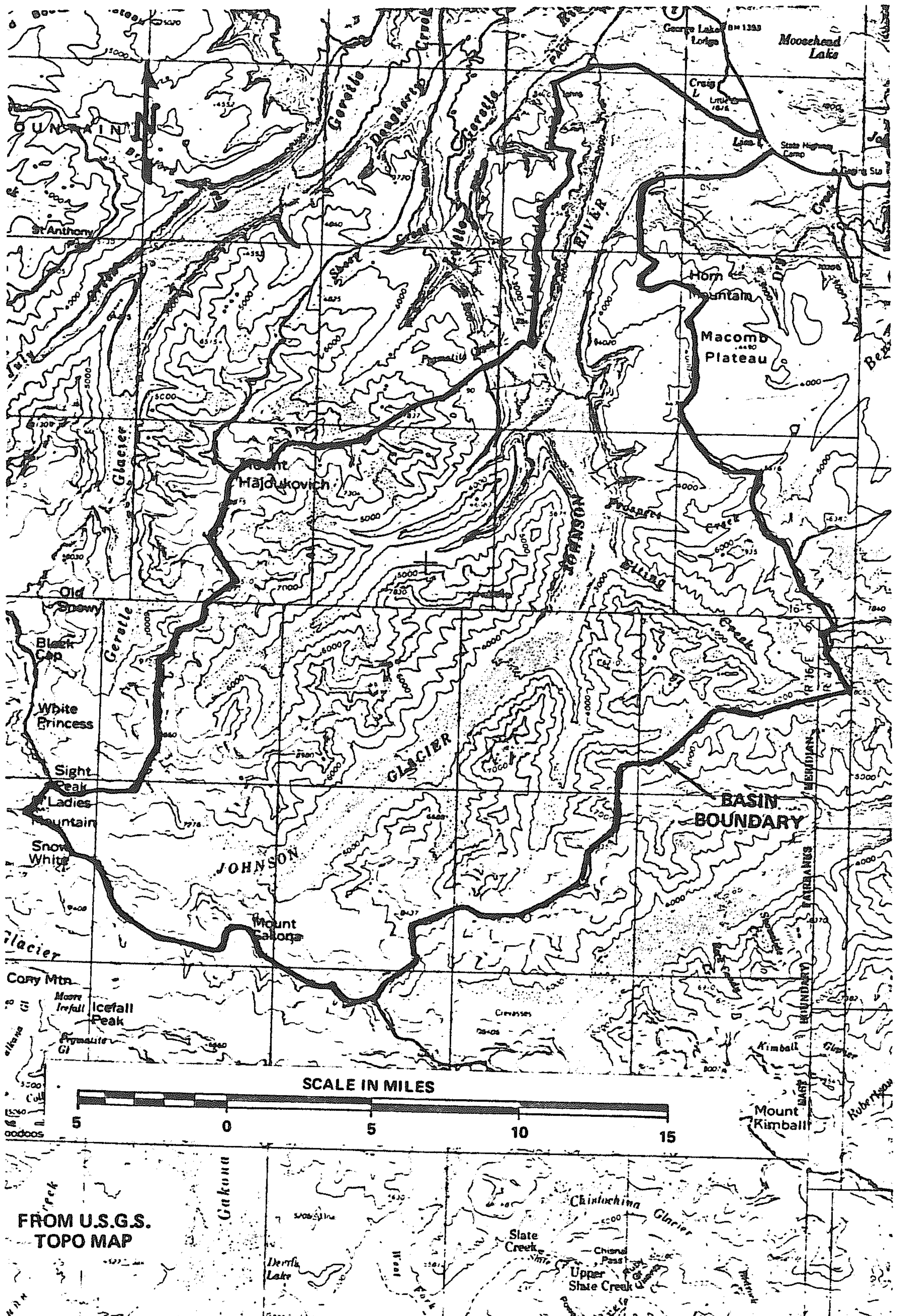

FIGURE 2 JOHNSON RIVER DRAINAGE BASIN

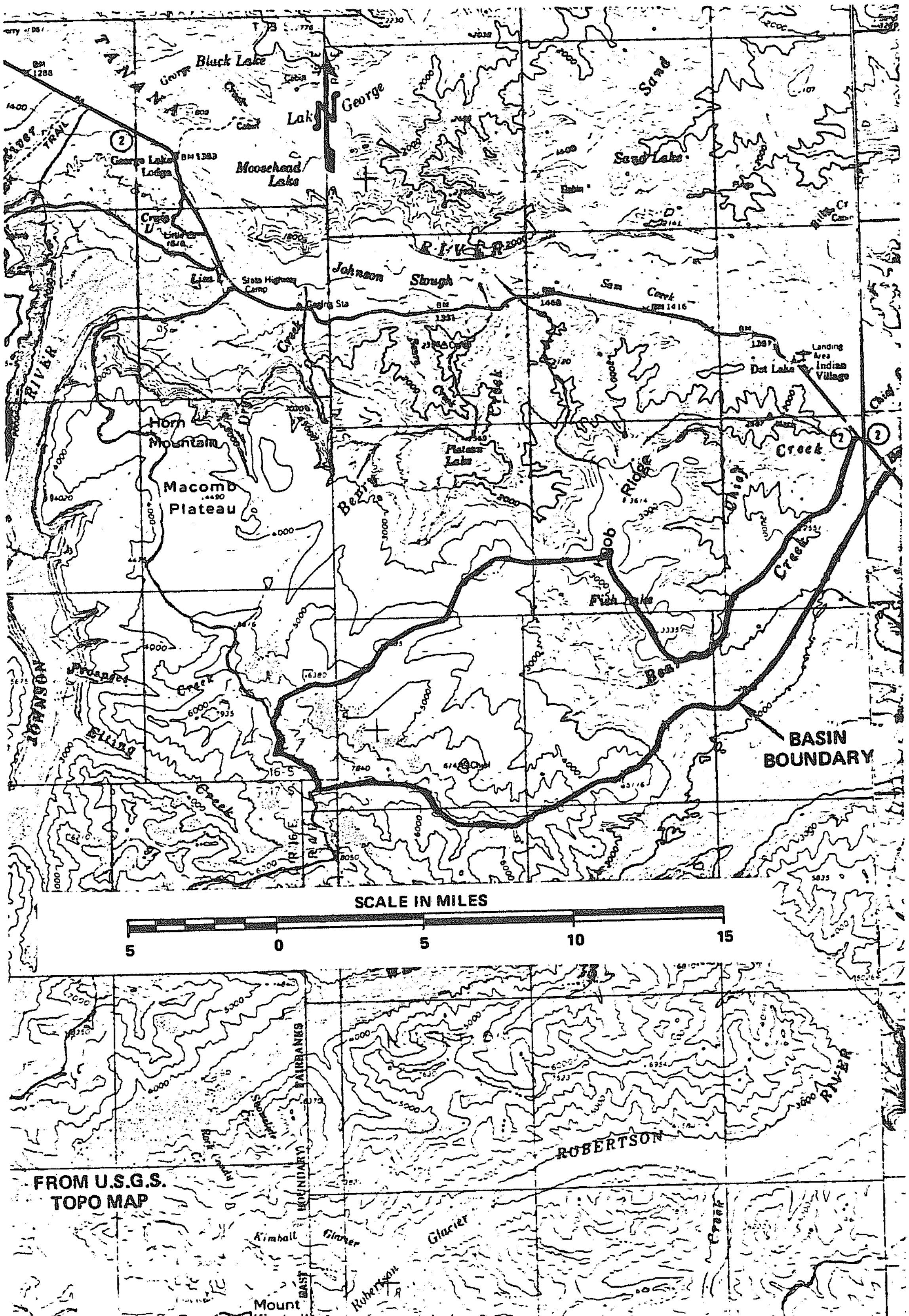

FIGURE 3 BEAR CREEK DRAINAGE BASIN

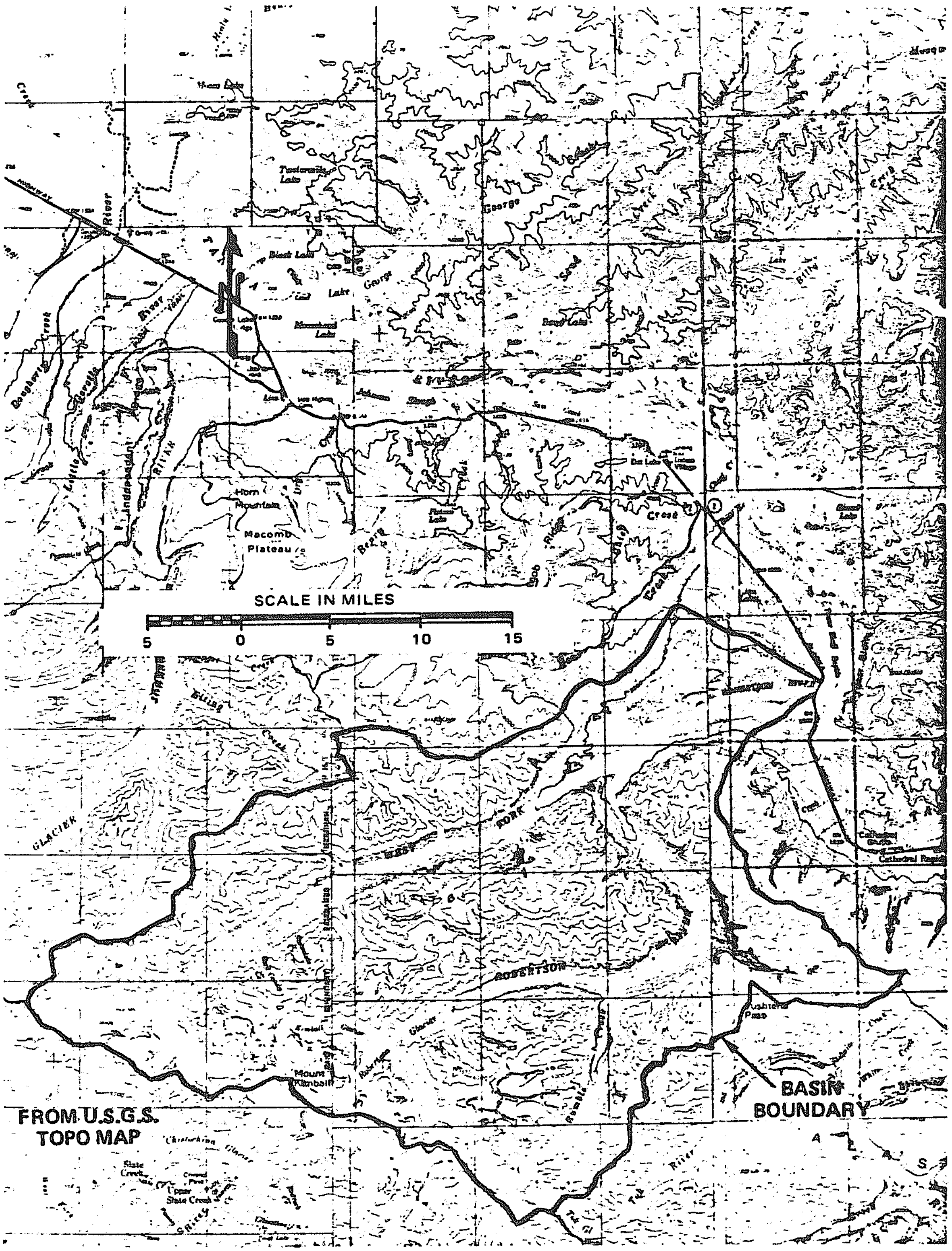

FIGURE 4 ROBERTSON RIVER DRAINAGE BASIN

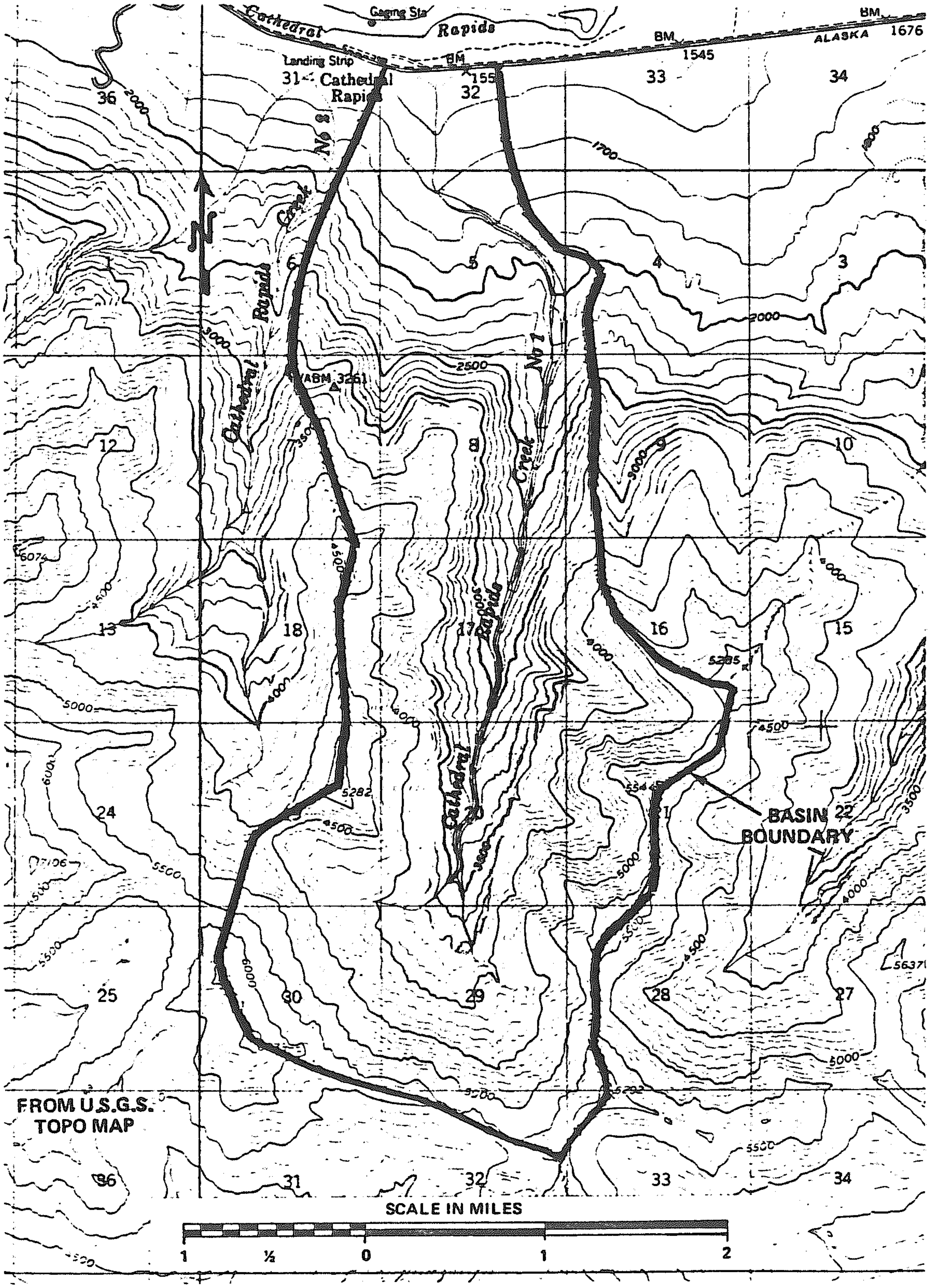

FIGURE 5 CATHEDRAL RAPIDS No. 1 DRAINAGE BASIN

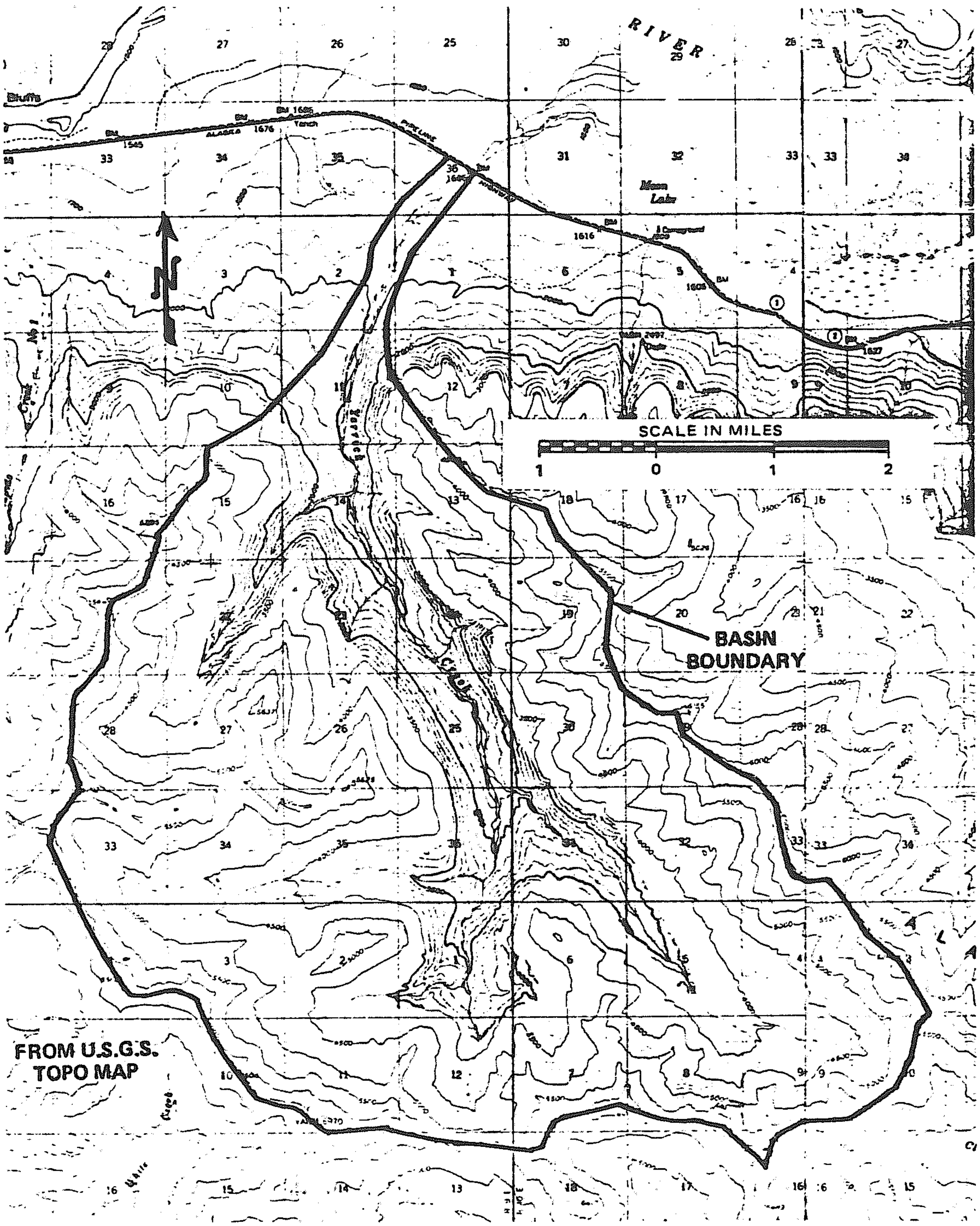

FIGURE 6 YERRICK CREEK DRAINAGE BASIN

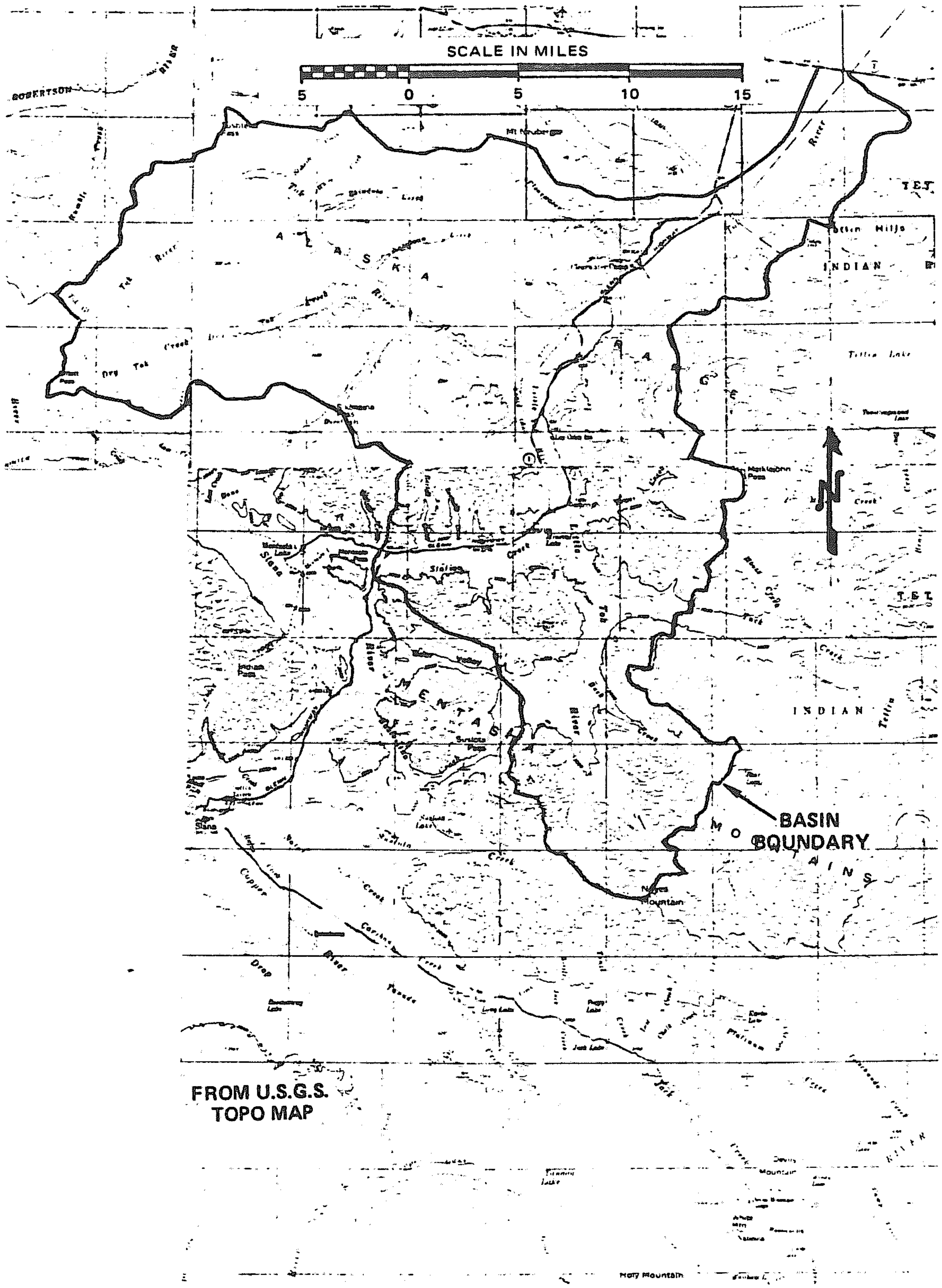

FIGURE 7 TOK RIVER DRAINAGE BASIN

FIGURE 8 UPPER TANNANA DRAINAGE BASIN

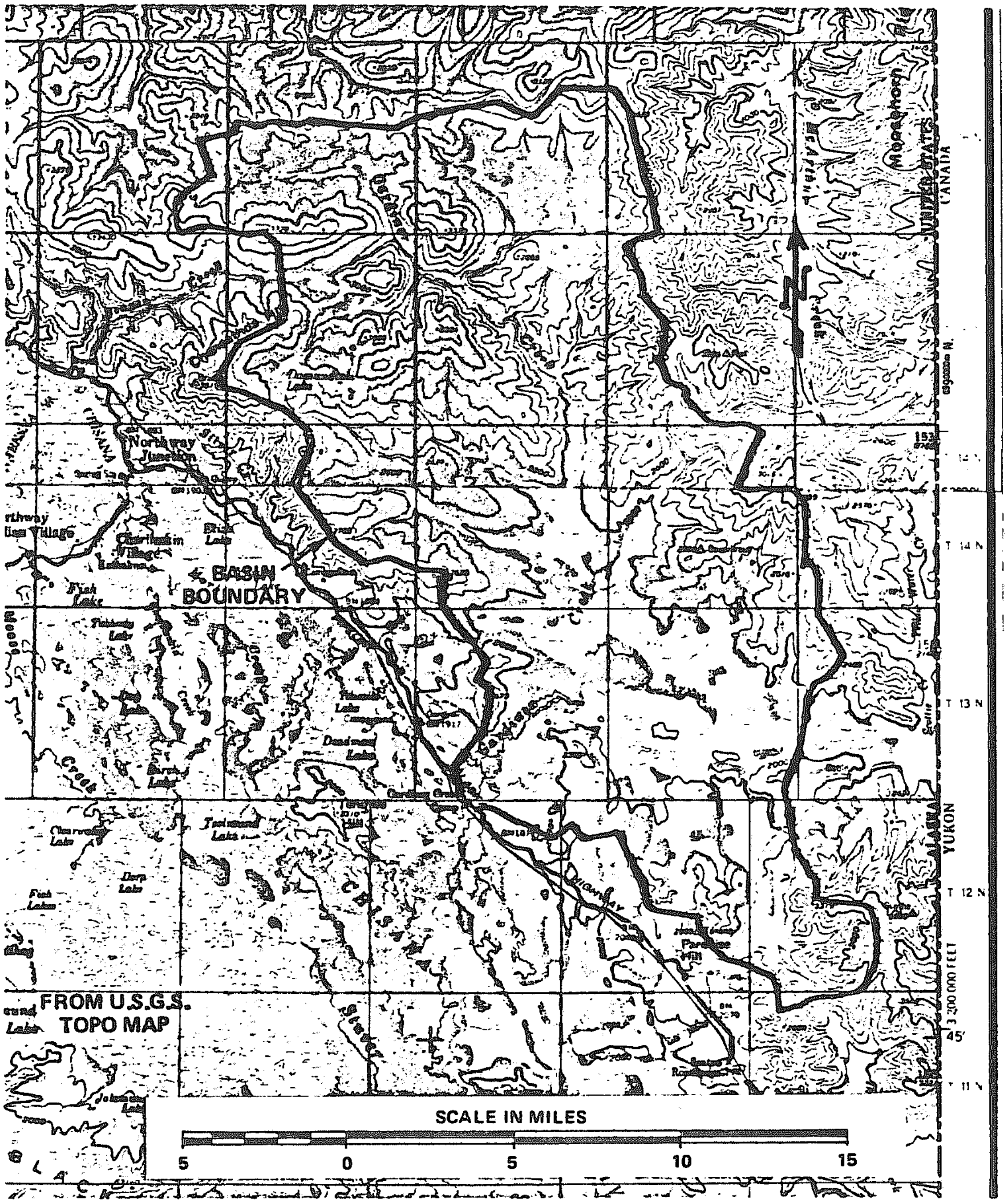

FIGURE 9 GARDINER CREEK DRAINAGE BASIN

D. Determination of the Probable Maximum 24-Hour Point Precipitations

The generalized estimate of Probable Maximum Precipitation (PMP) is Figure 2-12 contained in Technical Paper 47 (Miller, 1963 - Reference 3). By using the basin centroid and Figure 2-12 from TP-47, the 24-hour point PMP was derived.

E. Reduction of "Point" Probable Maximum Precipitation (PMP) to "Areal" PMP

Point PMP was reduced to areal PMP by multiplication by the appropriate ratios given below (see Appendix 2 for discussion).

Drainage Area (mi^2)	Ratio to Point Precipitation
1	1.00
10	0.99
20	0.98
50	0.96
100	0.93
200	0.90
500	0.86
1,000	0.83
2,000	0.80

For larger drainage areas, such as the Upper Tanana River, major floods include snowmelt, and it is inappropriate to apply the unit hydrograph theory. Therefore, the discussion of reduction of point PMP on larger basis will not be addressed.

F. Determination of the Ratios of Probable Maximum Precipitation (PMP) for 6-, 12-, 24-, 48-, 72-, and 96-Hour Duration to that for 24-Hour Duration

The probable maximum precipitation for 6-hour, 12-hour, and 24-hour storms can be derived directly from TP-47. For longer durations of 48-, 72-, and 96-hours, the derivation (see Appendix 3) was based on TP-47 and the distribution used by the Corps of Engineers (U.S. Army Corps of Engineers, 1971a - Reference 5) on the Tanana basin. The following ratios of PMP for durations of 6-, 12-, 24-, 48-, 72-, and 96-hour to 24-hour PMP were adopted:

Duration (Hours)	Ratio to 24-Hour
6	0.70
12	0.80
24	1.00
48	1.38
72	1.66
96	1.86

G. Adjustment to Derived PMP

Adjustments to published precipitation estimates such as in TP-47 were made based on the experience gained in recent years. The 1967 flood in the Fairbanks area was used as a base to adjust the PMP obtained from TP-47 in some drainage basins.

H. Determination of Pipeline Design Storm

The pipeline design storm is analogous to the standard project storm utilized by the Corps of Engineers. The Corps of Engineers (1962 - Reference 2) states that standard project storm precipitation is roughly between 40 percent and 60 percent of the probable maximum precipitation. A ratio of 50 percent was used south of the Brooks Range. Due to the lack of data north of the Brooks Range, 100 percent of probable maximum precipitation was adopted to compensate for the uncertainty. A transposition coefficient can be either an input quantity or can be computed by using the equation described in HEC-1 users manual (U.S. Army Corps of Engineers, 1973a - Reference 12). However, the values determined by the equation tend to lower the PDF values. The value of 1.0 was therefore adopted for all cases.

2.2 Time Area Curve

The shape of the hydrograph for flood discharge from a drainage basin depends on the travel time through the basin and on the shape and storage characteristics of the basin. Excess rainfall is inflow to the basin, and the hydrograph represents outflow.

By dividing the basin into zones and treating each zone separately, the outflow from one zone becomes inflow to the adjacent lower zone. These zones may be defined by lines of equal travel time (isochrones) from the outlet. The area of each isochronal area is then measured, and a time-area curve is plotted. This curve may be viewed as inflow to a hypothetical reservoir with storage characteristics equivalent to those of the basin and located at the basin outlet. The hydrograph which would result from an instantaneous rainfall of one inch uniformly distributed over the basin is called an instantaneous unit hydrograph (IUH). It can be converted into a unit hydrograph for any duration,t, by integrating ordinates over t units of time and plotting the integral at the end of the period.

The HEC-1 program provides an option which generates and applies a synthetic time-area curve. It was felt that these optional synthetic time-area curves should be tested before using that option in the PDF computation. The details of the comparison test are contained in Appendix 4. For cases where the synthetic time-area curve is found to be inappropriate, a distributed parameter rainfall-runoff model based on the physical measures of the basin morphology may be used to develop the IUH (Dawdy et al, 1978 - Reference 13).

2.3 Basic Hydrograph Parameters

The basic hydrograph parameters, time of concentration (TC), storage coefficient (R), starting flow (STRTQ), recession flow (QRCSN), and recession constant (RTIOR) required in any flow computation are estimated from hydrograph reconstitutions. Multiple regression analyses were performed to optimize these parameters for a satisfactory reproduction of past flood events on gaged streams. The records from drainages with known rainfall and outflow hydrographs were used to obtain the basic hydrograph parameters (see "Completion Report, Results of Hydrograph Reconstitution," Rev. #2, March 1981 - Reference 1). The optimized parameters obtained from these flood reconstitutions were then transferred to similar drainages. The transfer was achieved by the following derived regression equations:

$TC = R = 0.7\ A^{0.5}$ North of Brooks Range

$TC = R = 1.4\ A^{0.5}$ South of Brooks Range

$QRCSN = 4.17\ A$

$STRTQ = 0.47\ A^{1.33}$

$RTIOR_6 = 2.54\ A^{-0.1}$

where

A = Drainage area in square miles.

The recession constant ($RTIOR_{NHR}$) for time interval (NHR) other than 6 hours was derived from the relationship

$$RTIOR_{NHR} = e^{10(NHR)/k},$$

where

$k = 60/\ln(RTIOR_6)$.

The computation time interval (NHR) was determined from the following relationship:

If TC exceeds 45, use NHR = 6

If TC is between 35 and 45, use NHR = 3

If TC is between 15 and 35, use NHR = 2

If TC is less than 15, use NHR = 1

In order to test the results derived above, pipeline design floods were computed for streams that were used for the reconstitution. In all 17 cases, the pipeline design floods exceed the maximum flood of record. Comparisons are shown in Table 1. The PDF derived by Alyeska is included for comparison where available.

Table 2 shows the derived values of these parameters for eight of the streams currently under study. The Upper Tanana River drainage area was treated as a special case, and the discharge was picked from Figure 10, as described later in this report.

2.4 Loss Rates

Before computing the pipeline design flood, it is necessary to determine loss rates used to obtain the precipitation excess. The loss rate is not needed if the net rain is available for direct input. For gauged basins, HEC-1 allows the user to input rainfall and runoff data from which loss rate parameters are optimized to give a best fit to the information provided. For ungauged basins, the estimation of loss rate depends upon judgement.

Loss rates derived from the flood hydrograph reconstitution study were not used in computing the pipeline design flood. An initial loss of 0.11 inches and a uniform loss rate 0.03 in/hr, derived from the data used by the Corps of Engineers in the Chena study, were used. Those values are generally conservative values appropriate for wet conditions which should exist during a major storm such as the PDF.

TABLE 1
RECORD FLOODS AND DERIVED PDF ON STREAMS
USED FOR HYDROGRAPH RECONSTITUTIONS

Stream	Date of Flood	Maximum Known Flood cfs	Computed Flood cfs	Alyeska PDF cfs
Berry Creek Nr. Dot Lake	July 19, 1964	2,800	8,900	-
Boulder Creek Nr. Central	August 13, 1967	1,150	5,200	5,700
Caribou Creek Nr. Chatanika	May 12, 1975	117	2,800	-
Chena River at Fairbanks	August 15, 1967	74,400	98,800	98,800
Chena River Nr. North Pole	May 14, 1975	12,300[a]	79,100	-
Chena River Nr. Two Rivers	May 12, 1975	16,800[b]	63,200	-
Dry Creek Nr. Dot Lake	July 10, 1964	2,200	8,800	-
Jim River Nr. Bettles	June 1, 1977	12,800	40,300	30,100
Little Chena River Nr. Fairbanks	August 13, 1967	17,000	38,300	39,300
Melozitna River Nr. Ruby	September 3, 1962	28,200	127,500	68,300 *
Middle Fork Koyukuk River Nr. Wiseman	Before 1970	33,000	73,100	67,000
Nenana River Nr. Healy	July 25, 1967	46,800	96,200	79,000

TABLE 1 (Continued)

Stream	Date of Flood	Maximum Known Flood cfs	Computed Flood cfs	Alyeska PDF cfs
Poker Creek Nr. Chatanika	May 12, 1975	240	7,000	-
Salcha River Nr. Salchaket	August 14, 1967	97,000	123,400	102,000
Teklanika River Nr. Lignite	June 25, 1967	33,100	41,000	37,000
Wiseman Creek Nr. Wiseman	June 6, 1976	686	10,200	10,800
Wood River Nr. Fairbanks	August 13, 1976	5,510[c]	65,400	-

* Deviated from general relationship between Alyeska PDF and drainage area.

a The flood of August 13, 1967 was the largest flood since 1905; however, the gage was established in May 1972, and therefore no discharges are available for the 1967 flood.

b Gage height for 16,000 cfs is 21.05 feet. Gage height for the flood of August 13, 1967 reached a stage of 26.6 feet, from floodmarks, but discharge was not determined.

c Gage height for 5,510 cfs is 8.98 feet. Gage height for the flood of August 1967 reached a stage of 11.8 feet, from floodmarks, but discharge was not determined.

3.0 COMPUTER PROGRAM APPLICATION

The pipeline design flood can be derived by a number of methods. A computer model, Flood Hydrograph Package (HEC-1), developed by the Hydrologic Engineering Center and adopted by the Corps of Engineers, was used (U.S. Army Corps of Engineers, 1973a, 1973b - References 12 and 14).

3.1 Required Input Data For PDF Computations

In order to use the HEC-1 computer program, the following input data are required.

Code	Description
A	Job Title
B	Job Specifications
K	Computations Specification for Modeling
M	Subarea Runoff Computation Specification
P	Standard Project or Probable Maximum Storm Precipitation Data
T	Precipitation Loss Rate Data
V	Clark Unit Graph Coefficients
X	Hydrograph Starting and Recession Characteristics

3.2 Results

The pipeline design floods (PDF) were computed for eight (8) selected streams: Gerstle River, Johnson River, Bear Creek, Robertson River, Cathedral Rapids #1, Yerrick Creek, Tok River, and Gardiner Creek. These are shown in Table 2. As an example, the printout from the computer run for Gerstle River is included in Appendix 5.

Due to the fact that major floods in the Tanana result from snowmelt, the pipeline design flood for the Upper Tanana River crossing was determined from the Corps of Engineers' studies.

Standard Project Flood (SPF) determinations were available (Corps of Engineers, 1971a, 1971b - References 5 and 6) for the following rivers:

Station	Drainage Area	PDF
Chena River @ Fairbanks	1,980 square miles	91,000 cfs
Tanana River @ Fairbanks	19,000 square miles	255,000 cfs
Yukon River @ Rampart	199,000 square miles	1,600,000 cfs

These three points are shown as triangles in Figure 10, and the curve shows their relation of PDF to drainage area. It represents SPF for snowmelt conditions. The drainage area above the

Tanana River near Tok Junction (USGS Gauging Station No. 15472000) is 6,800 square miles, which gives a unit discharge of 20 cfs/mi^2 from the curve, for a PDF of 136,000 cfs. As a check against that value, a summary of maximum discharges for gauging stations in Alaska with drainage areas over 1,000 square miles (Lamke, 1979 - Reference 15) and within the East Arctic, Koyukuk, Yukon and Tanana regions, is given in Table 3. Those data are also shown on Figure 10. Several historical floods, when plotted on Figure 1, approach or exceed the curve. These are as follows:

No.	Stream	Date of Peak	Peak Flow cfs	D.A. (sq. mi.)
1.	Koyukuk R @ Hughes	6/6/64	266,000	18,700
2.	Porcupine R nr Fort Yukon	5/24/73	299,000	29,500
3.	Salcha R nr Salchaket	8/14/67	97,000	2,170
4.	Birch Creek nr Central	8/14/67	84,000	2,150
5.	Chena R @ Fairbanks	8/15/67	74,400	1,980
6.	Chena R ab L Chena	8/13,14/67	105,000	1,370

As a further check of the PDF discharge for Tanana River at Tok Junction, the Lamke (1979) value for the 50-year flood for the Tanana River at Tanacross would be 40,400 cfs. The PDF for Tanana River at Tanacross is 3.4 times that estimate of the 50-year flood.

Shown as crosses in Figure 10 are several PDF's determined for this report (refer to Table 1) for basins on the order of 1,000 to 2,000 square miles. They are:

No.	Station	PDF (cfs/mi^2)
A	Chena R at Fairbanks	49.9
B	Chena R nr North Pole	55.3
C	Melozitna R nr Ruby	47.3
D	Nenana R nr Healy	50.4
E	Salcha R nr Salchaket	56.9
F	M.F. Koyukuk nr Wiseman	51.3

Those data, determined for rainfall conditions, plot near the curve and indicate that this analysis yields similar results for both the snowmelt and rainfall floods for that size of drainage area.

In conclusion, for basins with drainage areas smaller than 2,000 square miles, the methods presented in this report, which use the HEC-1 computer program, will be used to compute the PDF. For areas 2,000 square miles and larger, Figure 10 will be used to compute the PDF.

TABLE 2
INPUT PARAMETERS AND PDF
FOR EIGHT SELECTED DELTA SOUTH STREAMS

Stream	A mi^2	L mi	NHR hrs	PMS in	TC hrs	R hrs	STRTQ cfs	QRCSN cfs	$RTIOR_6$	PDF cfs
Gerstle River	209.5	40.0	2	6.1	20.3	20.3	575	874	1.14	24,200
Johnson River	371.5	35.0	2	6.5	27.0	27.0	1,231	1,549	1.12	37,400
Bear Creek	86.1	22.0	1	5.3	12.9	12.9	176	359	1.08	11,800
Robertson River	572.3	38.0	2	6.4	33.5	33.5	2,186	2,387	1.12	49,000
Cathedral Rapids #1	9.7	5.1	1	5.9	4.4	4.4	10	40	1.12	3,100
Yerrick Creek	33.9	8.8	1	5.8	8.2	8.2	51	141	1.10	7,000
Tok River	907.	78.0	3	6.2	42.2	42.2	4,034	3,782	1.13	64,600
Gardiner Creek	323.5	36.0	2	4.4	25.2	25.2	1,024	1,349	1.13	21,800

TABLE 3
MAXIMUM KNOWN FLOODS IN EAST ARCTIC, KOYUKUK, YUKON AND TANANA AREAS

Station No.	Stream	Drainage Area (mi2)	Maximum Flood of Record (cfs)	Ratio of Observed Discharge to Drainage Area (cfs/mi2)
15348000	Fortymile R at Steel C	5,880	84,000	14.3
15356000	Yukon R at Eagle	113,500	545,000	4.8
--	Coleen R nr Rampart House	1,700	20,000	11.8
--	Sheenjek R nr Artik Village	2,230	18,000	8.1
15389000	Porcupine R nr Fort Yukon	29,500	299,000	10.1
15389500	Chandalar R nr Venetie	9,330	62,800	6.7
15446000	Birch C nr Central	2,150	84,000	39.1
15468000	Yukon R at Rampart	199,400	950,000	4.8
15470000	Chisana R at Northway Jct	3,280	12,000	3.7
15472000	Tanana R nr Tok Jct	6,800	35,700	5.3
15476000	Tanana R nr Tanacross	8,550	39,100	4.6
15478000	Tanana R at Big Delta	13,500	62,800	10.4
15484000	Salcha R nr Salchaket	2,170	97,000	44.7
--	Chena R ab L Chena nr Eielson AFB	1,370	105,000	76.6
15493500	Chena R nr North Pole	1,430	12,300	8.6
15514000	Chena R at Fairbanks	1,980	74,400	37.6
15515500	Tanana R at Nenana	25,600	186,000	7.3
15518000	Nenana R nr Healy	1,910	46,800	24.5
15518300	Nenana R nr Rex	2,450	63,000	25.7
15564600	Melozitna R nr Ruby	2,693	28,200	10.5
15564800	Yukon R at Ruby	259,000	970,000	3.7
15564875	M F Koyukuk R nr Wiseman	1,426	17,100	12.0
15564875	M F Koyukuk R nr Wiseman	1,426	33,000	23.1
15564900	Koyukuk R at Hughes	18,700	266,000	14.2
15565200	Yukon R at Kaltag	296,000	1,030,000	3.5
15896000	Kuparuk R nr Deadhorse	3,130	82,000	26.2
15896000	Kuparuk R nr Deadhorse	3,130	100,000	31.9
15910000	Sagavanirktok R nr Sagwon	2,208	34,900	15.8
15910000	Sagavanirktok R nr Sagwon	2,208	62,000	28.1
--	Shaviorik R nr Deadhorse	1,580	22,000	13.9
--	Canning R nr Artic Village	1,326	22,000	16.6
--	Canning R nr Deadhorse	1,871	53,000	28.3

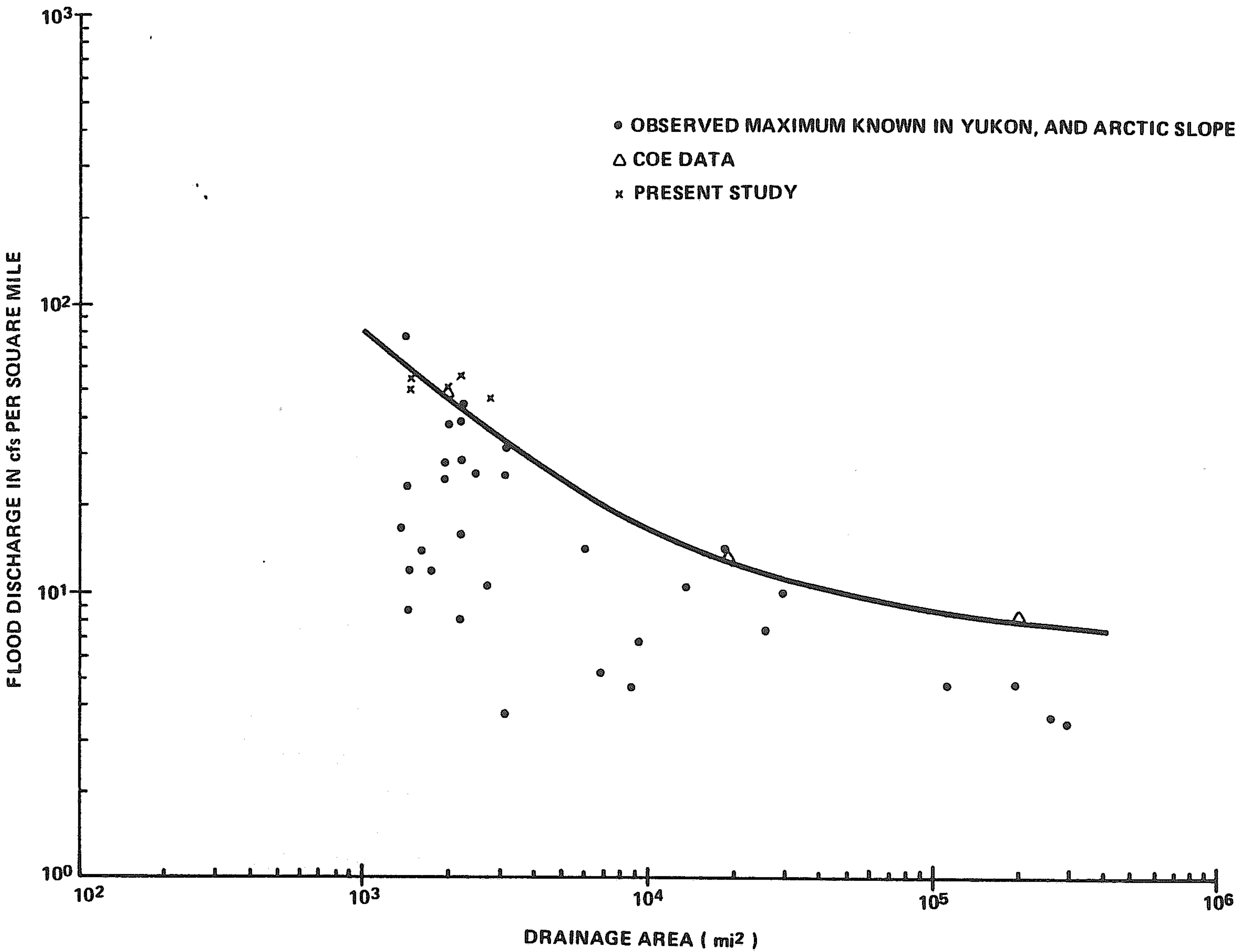

FIGURE 10

ALASKAN NORTHWEST NATURAL GAS TRANSPORTATION COMPANY

RELATION BETWEEN UNIT DISCHARGE AND DRAINAGE AREA

ALASKA SEGMENT OF THE ALASKA NATURAL GAS TRANSPORTATION SYSTEM

DRAWING NUMBER

REV.

FLUOR PROJECT MANAGEMENT CONTRACTOR

NORTEC NORTHERN TECHNICAL SERVICES

DES./DWN.	/	DATE	REV.	BY	APP.	DATE
CHK./APP.	/					
PMC/NWA	/					

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APPENDIX 1

LOCATION OF THE CENTROID OF DRAINAGE BASINS

The centroid for each drainage basin was located in the following manner:

a. Tracing paper was used to overlay the map and the drainage divides were traced.

b. Drainage areas were then cut out with scissors.

c. Two pinpoints were selected on the cut-out.

d. A pin attached to a light string stretched by a heavy object, was used to fasten the cut-out of the drainage area on a vertical board.

e. A line was drawn along the string after the cut-out was in a balanced position.

f. Steps d and e were repeated on another pinpoint.

g. The intersection of these two string lines was then located. This was the centroid of the drainage area.

h. The centroid was then transferred back to the map and the longitude and latitude were determined.

APPENDIX 2

DEPTH-AREA RELATIONS

Precipitation averaged over an area is less than that which occurs at the storm center, which receives the maximum amount. Depth-area relations are developed to define how average rainfall for a storm or storms varies as the area under study increases. The depth-area relations are primarily a function of the type of storm dominant in the area. Thunderstorms predominate in arid regions and produce extremely intense rainfall over a small area of a few tens of square miles. Large frontal systems produce relatively uniform precipitation over hundreds of square miles. They are the predominant storms of the Pacific Northwest region of the Continental United States.

A number of relations resulting from large storms have been investigated by the National Weather Service and other government agencies. Curves applicable to broad geographic regions have been derived. One relationship applicable to drainage areas in Alaska of up to 400 square miles is provided by Technical Paper 47 (Miller, 1963). The paper indicates the relation was not derived from the sparse Alaskan data but rather was adapted from curves developed for the Pacific Coastal areas. To test the conservatism of the estimate and to provide a basis for extrapolation to larger areas, the recommended curve as well as curves recommended by other sources for other regions were plotted. All original data were adjusted to a uniform base for 10 square miles. Data from the following sources are presented in Plate 2-1:

1. Miller, J.F., 1963, "Probable Maximum Precipitation and Rainfall-Frequency Data for Alaska". National Weather Service, Technical Paper 47, Figure 2-15, 2-16 for durations of 6-, 12-, and 24-hours.

2. U.S. Bureau of Reclamation, 1977, "Design of Small Dams". Chapter III, Flood Studies, Figure 19 presents three general storm conversion ratios for 3 areas in the western United States. These ratios pertain to a 6-hour duration and areas to 1000 square miles.

3. Donald M. Gray, 1973, "Handbook on the Principles of Hydrology". It presents data from Canadian prairie rainstorms. These data are for durations from 6 to 84-hours and areas up to about 50,000 square miles.

The plot shows that use of the 24-hour curve from Miller, 1961, extended logarithmically provides a good estimate of depth-area relations. A slight reduction in short duration precipitation rates was justified by considering the variability of the depth-area relation with duration. However, in the interest of conservatism, this was not done.

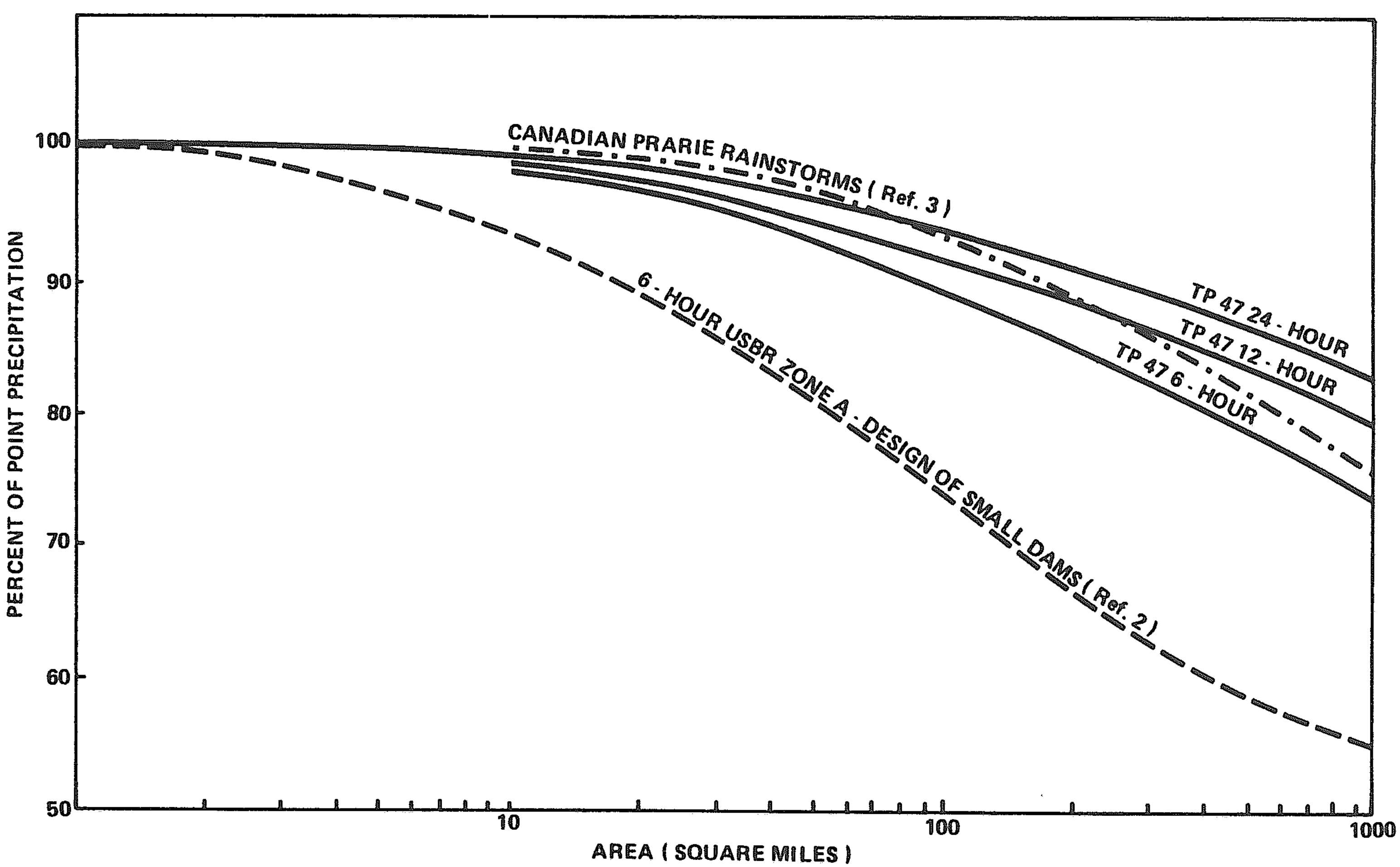

PLATE 2 - 1

FLUOR
PROJECT MANAGEMENT CONTRACTOR

NORTEC
NORTHERN TECHNICAL SERVICES

		DATE	REV.	BY	APP.	DATE
DES./DWN.	/					
CHK./APP.	/					
PMC /NWA	/					

ALASKAN NORTHWEST NATURAL GAS TRANSPORATION COMPANY

COMPARISON OF PUBLISHED DEPTH - AREA RELATIONS

ALASKA SEGMENT OF THE ALASKA NATURAL GAS TRANSPORTATION SYSTEM

DRAWING NUMBER	REV.

APPENDIX 3

LONG DURATION PROBABLE MAXIMUM PRECIPITATION

Note: The derivation of Plate 3-1 in this Appendix was based on the data from National Weather Service, Technical Paper 47 and Corps of Engineers for the Chena Lakes Flood Control Project.

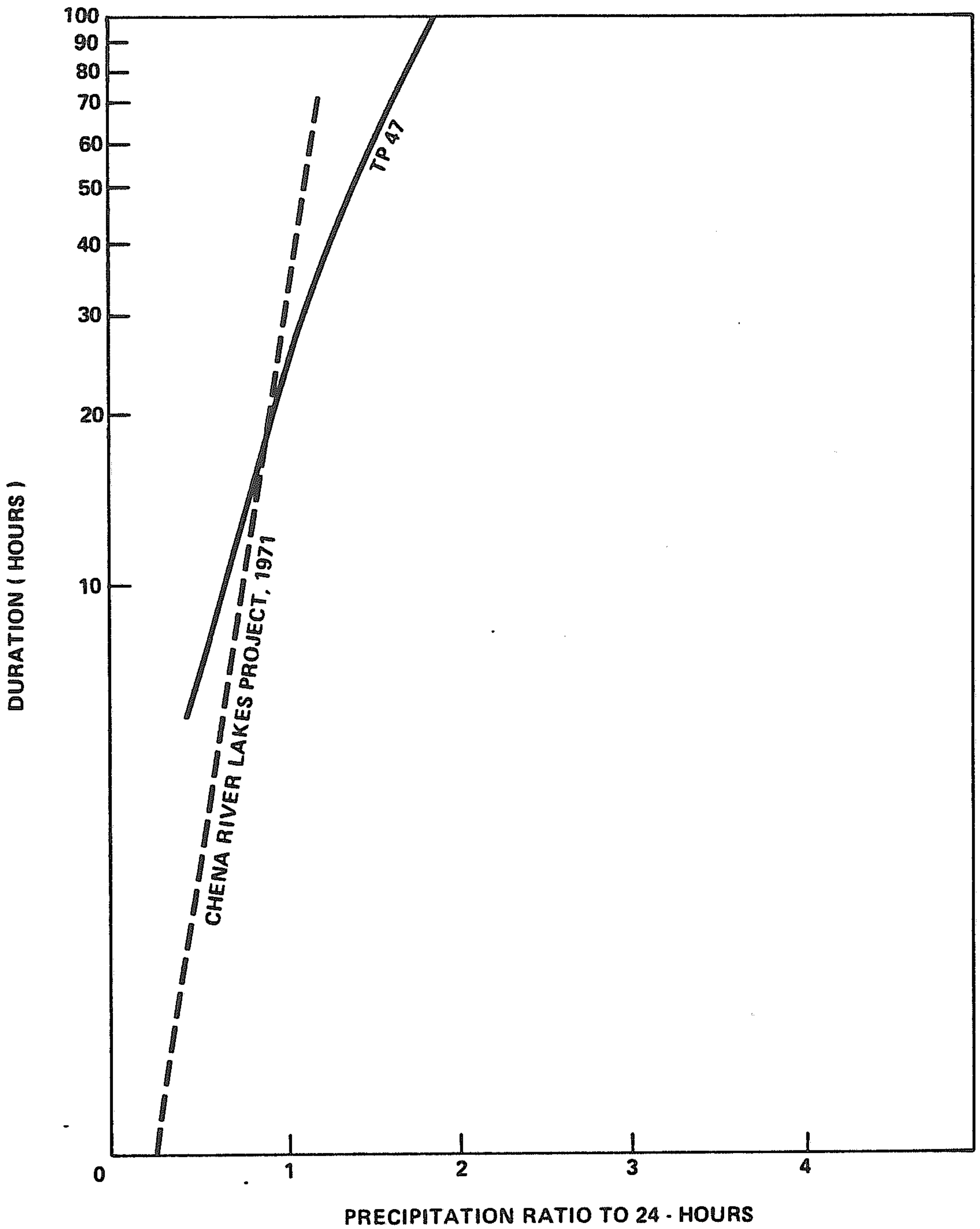

PLATE 3 - 1

FLUOR
PROJECT MANAGEMENT CONTRACTOR

NORTEC
NORTHERN TECHNICAL SERVICES

ALASKAN NORTHWEST NATURAL GAS TRANSPORATION COMPANY

DEPTH - DURATION RELATIONS

ALASKA SEGMENT OF THE ALASKA NATURAL GAS TRANSPORTATION SYSTEM

		DATE	REV.	BY	APP.	DATE
DES./DWN.	/					
CHK./APP.	/					
PMC /NWA	/					

DRAWING NUMBER	REV.

APPENDIX 4

REAL

AND

SYNTHETIC

TIME-AREA CURVES

TABLE OF CONTENTS

CONCLUSIONS

The results of three tests strongly indicated that the synthetic curve generated in the HEC-1 program could fulfill the needs for pipeline design flood computations. The result also showed that considerable manpower and effort could be saved by using the option. However, for some basins that deviate substantially from the generalized shape upon which the synthetic curves were based, a time-area curve must be determined.

4-1.0 INTRODUCTION

The time-area curve for a basin relates the travel time from within the basin to the outflow point to the cumulative area of the basin with that travel time or greater. The HEC-1 program provides an option which generates and applies a synthetic time-area curve. A comparison was made of the results from the use of the optional synthetic time-area curves with time-area curves derived from standard basin analyses. That study was used to determine whether any accuracy was lost by the use of the HEC-1 program.

4-2.0 REAL TIME-AREA CURVE

4-2.1 Description

The shape of the hydrograph for a drainage basin depends on the travel time through the basin and on the shape and storage characteristics of the basin. Excess rainfall is the inflow to a basin and the hydrograph is the outflow.

The Clark unit hydrograph procedure utilizes a time-area curve for the basin under study. This time-area curve may be obtained by the use of measured or calculated flow travel times from various locations in the basin. An expedient approximation to the time-area curve is the distance-area curve. When the distance-area curve is used instead of the time-area curve, the assumption is made that flow time from a given location is proportional only to the travel distance to the basin outlet.

The basin is divided into zones. These zones are bounded by lines of equal travel time (isochrones) from the outlet. The area between isochrones is then measured, and a time-area curve is plotted. This curve is a translation hydrograph which is considered as inflow to a hypothetical reservoir located at the basin outlet. That reservoir has storage characteristics equivalent to those of the basin.

The hydrograph which flows out of the reservoir as the result of an instantaneous rainfall is called an instantaneous unit hydrograph (IUH). The IUH can be converted into a time unit hydrograph (TUH) for any duration,t, by averaging ordinates t units of time apart and plotting the average at the end of the period.

4-2.2 Construction of Isochrones

Steps used in the construction of isochrones are as follows:

1. Determine time of concentration by using either: (1) Manning's formula, (2) the observed hydrograph or, (3) hydrograph reconstitution.

2. Subdivide the time of concentration into equal parts.

3. Plot a profile of the longest water course.

4. Estimate the elevation of the intersections of the isochrones with the main channel.

5. Transfer the intersections to the topographic map along the main channel.

6. Locate similar intersections on the major tributaries.

7. Draw contour lines on the positions of isochrones.

For simplicity, the isochrones are usually drawn equal distances apart from the outflow location to the uppermost head of the basin. The number of isochrones used is ordinarily chosen so that a convenient scale may be used and reasonably good definition of the time-area relation obtained. If the basin is reasonably uniform throughout its length, this simplified method produces acceptable accuracy. If large variations exist between very flat and very steep portions of the basin, velocities must be estimated and used to construct the isochrones.

4-2.3 Construction of Time-Area Curve

Steps used in construction of a time-area curve are as follows:

1. Determine the contributed area between isochrones.

2. Plot the accumulated area vs. travel time in percent.

3. Tabulate increments between points one computation interval apart.

4-3.0 SYNTHETIC TIME-AREA CURVE

4-3.1 Description

In many cases, it has been found that it is not necessary to use the actual time-area curve as described above in the analysis. Instead, a distance-area curve of general shape can be used to represent the time-area curve of the basin. This generalized distance-area curve is referred to as a synthetic time-area curve.

In the HEC-1 program, a synthetic curve is computed from an assumed parabolic shape, with accumulated area as the 1.5 power of time, up to half of the time of concentration, and as a symmetrical curve for the last half of the time of concentration. From a study of a large number of basins it has been found, by others, that this synthetic curve represents a shape common to most watersheds. As a note of caution, for basins that deviate substantially from the generalized shape, a time-area curve based on physical data from the basin should be used.

4-4.0 SAMPLE TESTS

4-4.1 Description

In order to test the reliability of the HEC-1 program option, the isochrones for Gerstle River, Little Chena River and Salcha River basins were developed by conventional methods. These have been plotted on Plates 4-1, 4-2 and 4-3, respectively. These three basins were selected to represent the range of basin shapes along the proposed pipeline route. Time-of-concentration (TC), which is used to determine the number of isochrones, was obtained from hydrograph reconstitutions. The isochrones were constructed by linear interpolation between points. In developing the time-area curve, all points along each isochrone were placed at equal distances from the basin outlet.

4-4.2 Results

Six computer runs were performed. Three of the runs used real time-area curves and three used the synthetic curves. Computer printouts are shown on Plate 4-4 and comparisons are shown in Table 4-1. For all three basins, the flood peaks computed from both approaches were very close. By comparison of the two hydrographs, it can be seen that the shapes of computed versus synthetic hydrographs are very consistent.

TABLE 4-1
COMPARISON BETWEEN REAL AND SYNTHETIC TIME-AREA CURVES

Stream	Peaks (cfs)	
	Real Time-Area Curve	Synthetic Time-Area Curve
Gerstle River	24,700	24,000
Little Chena River	24,200	23,800
Salcha River	130,000	130,300

ISOCHRONES ON GERSTLE RIVER BASIN

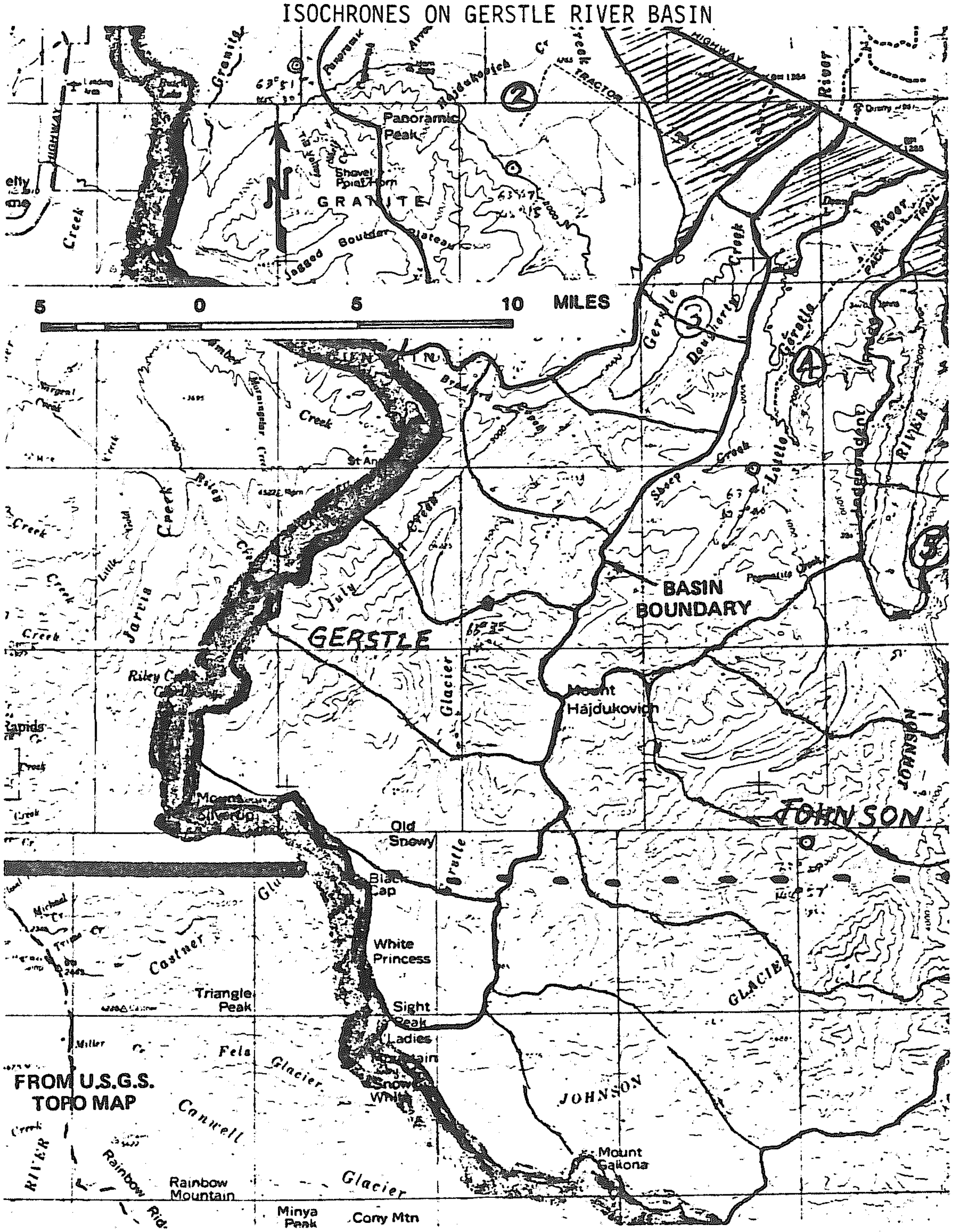

PLATE 4-1

ISOCHRONES ON LITTLE CHENA RIVER BASIN

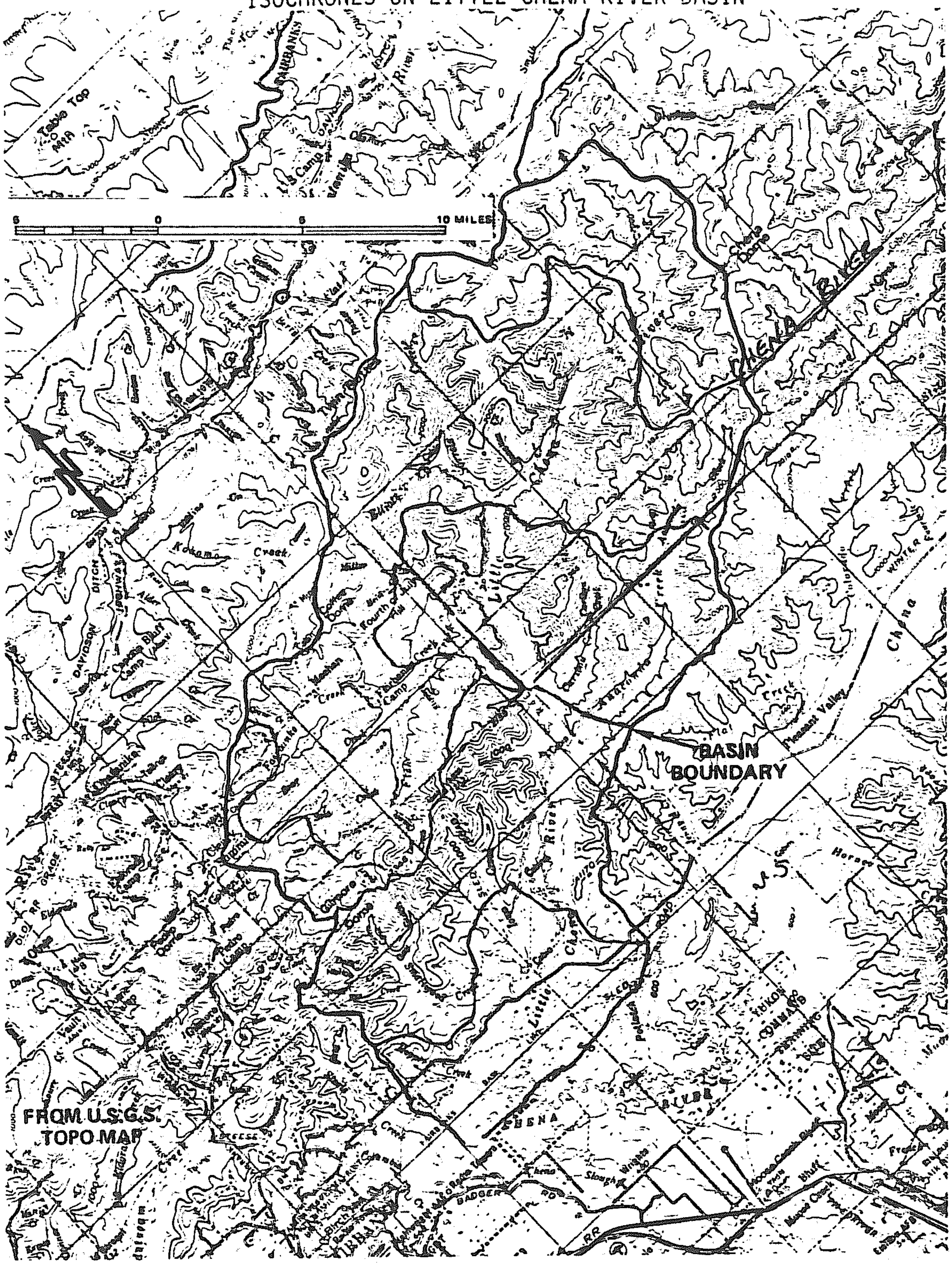

PLATE 4-2

ISOCHRONES ON SALCHA RIVER BASIN

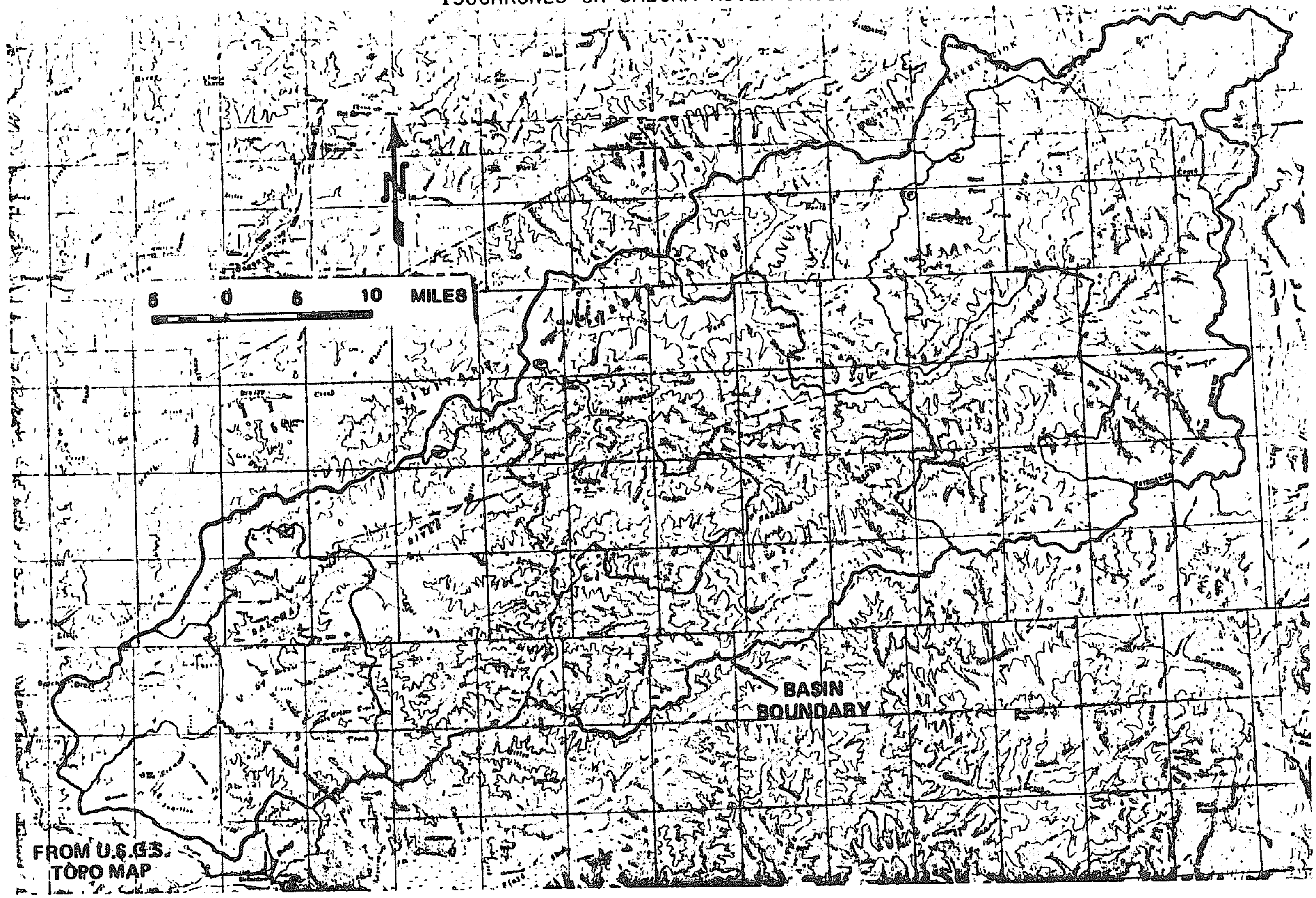

PLATE 4-3

APPENDIX 5

EXAMPLE PDF COMPUTATION
(GERSTLE RIVER)

STREAM ANALYSIS
SUMMARY DATA SHEET
PIPELINE DESIGN FLOOD

STREAM NAME Gerstle River FILE NO. NT-5-172

PIPELINE MILEPOST 576.55 ALIGNMENT SHEET 102 REV. 3

1. a) Drainage Area (TAREA) at River Mile 22.7

Area of Lakes - sq. mi.

Area of Forests - sq. mi.

Area of Swamps - sq. mi.

Area of Glaciers 61.9 sq. mi.

Area of Land 147.6 sq. mi.

Total Drainage Area 209.5 sq. mi.

b) Main Channel Length 33.6 miles

c) Channel Slope 166.0 ft/mile

2. Centroid of Basin

Latitude 63 ° 35 '

Longitude 145 ° 16 '

3. Probable Maximum Storm (PMS)

6-hr 4.3 in. 12-hr 4.9 in. 24-hr 6.1 in.

48-hr 8.4 in. 72-hr 10.1 in. 96-hr 11.4 in.

4. a) Initial Loss (STRTL) 0.11 inches

b) Uniform Rainfall Loss Rate (CNSTL) 0.03 in/hr.

5. Time of Concentration (TC) 20.3 hrs.

6. Storage Coefficient (R) 20.3 hrs.

7. a) No. of Time-Area Coordinates (NTA) ______ dimensionless

b) Program Option used? (Yes x No ___)

8. Starting Flow (STRTQ) 575 cfs (Winter ___ Summer x)

9. Flow at which Recession occurs (QRCSN) 874 cfs

10. Ratio of Recession Flow (RTIOR) 1.14 dimensionless

11. Pipeline Design Flood (PDF) Magnitude 24,200 cfs

```
****************************
HEC-1 VERSION DATED JAN 1973
UPDATED AUG 74
CHANGE NO. 01
****************************
```

GERSTLE RIVER
PIPELINE DESIGN FLOOD
MARCH 1981

JOB SPECIFICATION

NQ	NHR	NMIN	IDAY	IHR	IMIN	METRC	IPLT	IPRT	NSTAN
150	2	0	0	0	0	0	2	1	0

JOPER	NWT
3	0

*********** *********** *********** *********** ***********

SUB-AREA RUNOFF COMPUTATION

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
0	0	0	0	0	0	0

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
1	0	209.50	0.00	209.50	1.00	0.000	0	0	0

PRECIP DATA

SPFE	PMS	R6	R12	R24	R48	R72	R96
0.00	6.10	70.00	80.00	100.00	138.00	166.00	186.00

LOSS DATA

STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0.00	0.00	1.00	0.00	0.00	1.00	.11	.03	0.00	0.00

UNIT HYDROGRAPH DATA
TC= 20.30 R= 20.30 NTA= -0

RECESSION DATA
STRTQ= 575.00 QRCSN= 874.00 RTIOR= 1.14

UNIT HYDROGRAPH 60 END-OF-PERIOD ORDINATES, LAG= 18.50 HOURS, CP= .57 VOL= 1.00

139.	518.	1052.	1670.	2344.	3013.	3575.	3975.	4206.	4241.
4014.	3645.	3303.	2992.	2711.	2457.	2226.	2017.	1828.	1656.
1501.	1360.	1232.	1116.	1012.	917.	831.	753.	682.	618.
560.	507.	469.	416.	377.	342.	310.	281.	254.	230.
209.	189.	171.	155.	141.	129.	116.	105.	95.	86.
78.	71.	64.	58.	53.	48.	43.	39.	35.	32.

END-OF-PERIOD FLOW

TIME	RAIN	EXCS	COMP Q
1	.03	0.00	568.
2	.03	0.00	560.
3	.03	0.00	553.
4	.04	0.00	546.
5	.04	0.00	539.
6	.04	0.00	532.
7	.22	.16	547.
8	.45	.39	656.

10	.05	0.00	1250.
11	.05	0.00	1659.
12	.05	0.00	2099.
13	.06	.00	2527.
14	.06	.00	2887.
15	.06	.00	3145.
16	.08	.02	3288.
17	.08	.02	3298.
18	.08	.02	3169.
19	.42	.36	3016.
20	.86	.80	3064.
21	.34	.28	3422.
22	.09	.03	4023.
23	.09	.03	4780.
24	.09	.03	5628.
25	.16	.10	6485.
26	.16	.10	7247.
27	.16	.10	7848.
28	.20	.14	8270.
29	.20	.14	8477.
30	.20	.14	8454.
31	1.11	1.05	8455.
32	2.26	2.20	9011.
33	.90	.84	10381.
34	.24	.18	12381.
35	.24	.18	14763.
36	.24	.18	17350.
37	.05	0.00	19875.
38	.05	0.00	21982.
39	.05	0.00	23445.
40	.06	0.00	24167.
41	.06	0.00	24012.
42	.06	0.00	22931.
43	.31	.25	21326.
44	.63	.57	19757.
45	.25	.19	18437.
46	.07	.01	17355.
47	.07	.01	16488.
48	.07	.01	15810.
49	0.00	0.00	15246.
50	0.00	0.00	14696.
51	0.00	0.00	14095.
52	0.00	0.00	13412.
53	0.00	0.00	12601.
54	0.00	0.00	11642.
55	0.00	0.00	10624.
56	0.00	0.00	9660.
57	0.00	0.00	8781.
58	0.00	0.00	7980.
59	0.00	0.00	7252.
60	0.00	0.00	6593.
61	0.00	0.00	5995.
62	0.00	0.00	5453.
63	0.00	0.00	4962.
64	0.00	0.00	4516.
65	0.00	0.00	4112.
66	0.00	0.00	3746.
67	0.00	0.00	3409.
68	0.00	0.00	3097.
69	0.00	0.00	2822.
70	0.00	0.00	2576.
71	0.00	0.00	2352.
72	0.00	0.00	2150.
73	0.00	0.00	1966.

74	0.00	0.00	1799.
75	0.00	0.00	1649.
76	0.00	0.00	1510.
77	0.00	0.00	1385.
78	0.00	0.00	1271.
79	0.00	0.00	1158.
80	0.00	0.00	1043.
81	0.00	0.00	953.
82	0.00	0.00	879.
83	0.00	0.00	863.
84	0.00	0.00	852.
85	0.00	0.00	841.
86	0.00	0.00	830.
87	0.00	0.00	819.
88	0.00	0.00	809.
89	0.00	0.00	799.
90	0.00	0.00	788.
91	0.00	0.00	777.
92	0.00	0.00	767.
93	0.00	0.00	757.
94	0.00	0.00	748.
95	0.00	0.00	738.
96	0.00	0.00	728.
97	0.00	0.00	719.
98	0.00	0.00	709.
99	0.00	0.00	700.
100	0.00	0.00	691.
101	0.00	0.00	682.
102	0.00	0.00	673.
103	0.00	0.00	664.
104	0.00	0.00	656.
105	0.00	0.00	647.
106	0.00	0.00	639.
107	0.00	0.00	630.
108	0.00	0.00	622.
109	0.00	0.00	614.
110	0.00	0.00	606.
111	0.00	0.00	598.
112	0.00	0.00	590.
113	0.00	0.00	583.
114	0.00	0.00	575.
115	0.00	0.00	568.
116	0.00	0.00	560.
117	0.00	0.00	553.
118	0.00	0.00	546.
119	0.00	0.00	539.
120	0.00	0.00	532.
121	0.00	0.00	525.
122	0.00	0.00	518.
123	0.00	0.00	511.
124	0.00	0.00	505.
125	0.00	0.00	498.
126	0.00	0.00	491.
127	0.00	0.00	485.
128	0.00	0.00	479.
129	0.00	0.00	473.
130	0.00	0.00	466.
131	0.00	0.00	460.
132	0.00	0.00	454.
133	0.00	0.00	448.
134	0.00	0.00	443.
135	0.00	0.00	437.
136	0.00	0.00	431.
137	0.00	0.00	426.

137	0.00	0.00	426.
138	0.00	0.00	420.
139	0.00	0.00	415.
140	0.00	0.00	409.
141	0.00	0.00	404.
142	0.00	0.00	399.
143	0.00	0.00	393.
144	0.00	0.00	388.
145	0.00	0.00	383.
146	0.00	0.00	378.
147	0.00	0.00	373.
148	0.00	0.00	368.
149	0.00	0.00	364.
150	0.00	0.00	359.
SUM	11.32	8.65	647862.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	24167.	23875.	20594.	13814.	647862.
INCHES		1.06	3.66	7.36	9.59
AC-FT		11845.	40868.	82239.	107140.

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	0	24167.	23875.	20594.	13814.	209.50