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Cumulative Erosion Impacts Analysis for the Missouri River Master Water Control Manual Review and Update Study

By Thomas J. Pokrefke, David A. Abraham,
Peggy H. Hoffman, WES

William A. Thomas, Mobile Boundary Hydraulics

Stephen E. Darby, Colin R. Thorne, University of Nottingham

WES

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Prepared for U.S. Army Engineer Division, Northwestern

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by Thomas J. Pokrefke, David A. Abraham, Peggy H. Hoffman

U.S. Army Corps of Engineers
Waterways Experiment Station
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

William A. Thomas

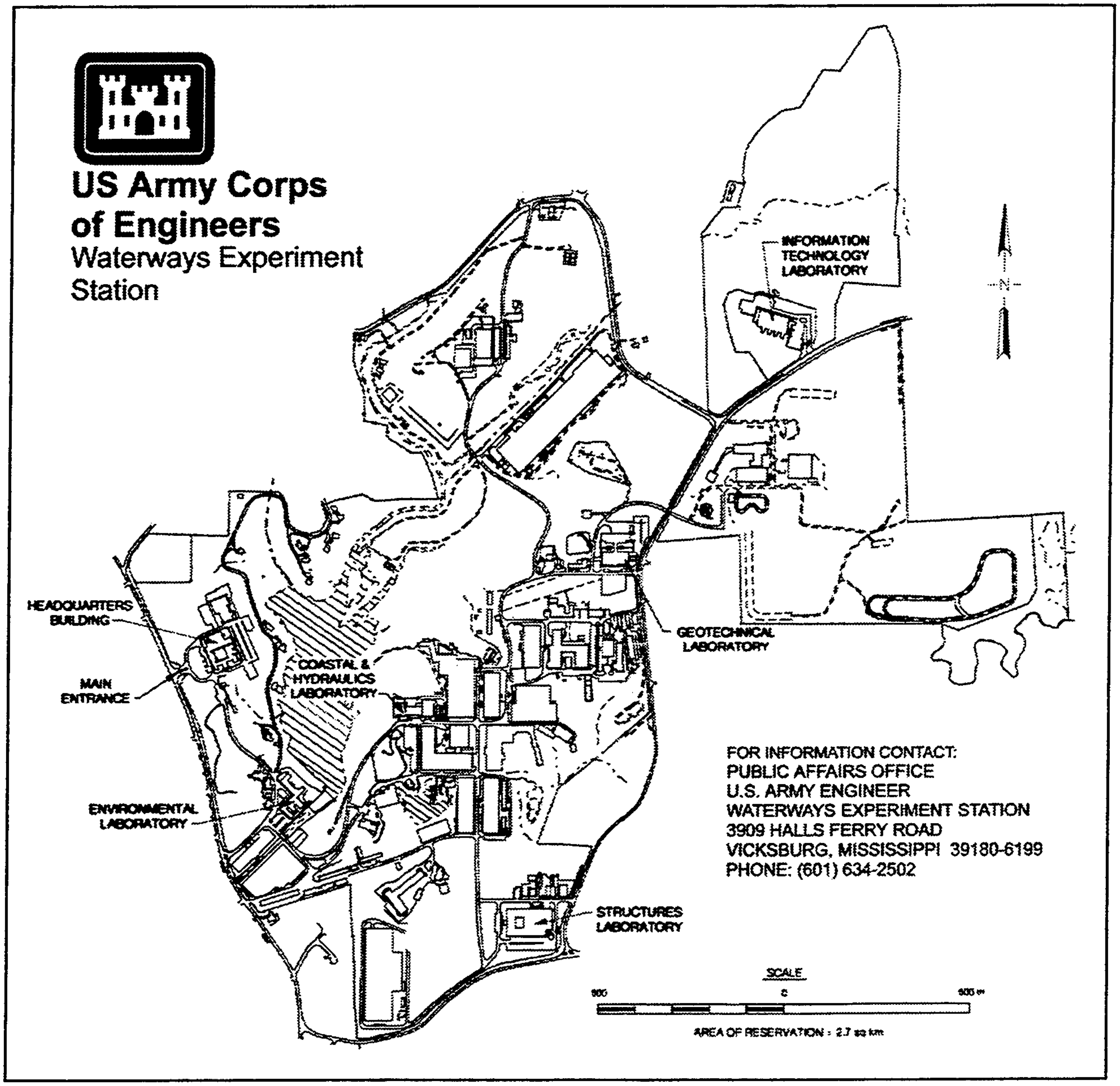
Mobile Boundary Hydraulics
606 Highway 80 West
Clinton, MS 39056-4183

Stephen E. Darby, Colin R. Thorne

University of Nottingham
University Park, Nottingham NG7 2RD
United Kingdom

Final report

Prepared for U.S. Army Engineer Division, Northwestern
Missouri River Region
Omaha, NE 68144-3869



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Preface

The study reported herein was conducted for the U.S. Army Engineer Division, Missouri River Region (MRR), in the Coastal and Hydraulics Laboratory (CHL), U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, MS, during the period July 1995 to September 1997. MRR is the former U.S. Army Engineer Division, Missouri River. The investigation was conducted under the general supervision of Messrs. F. A. Herrmann, Jr., former Director, Hydraulics Laboratory (HL), R. A. Sager, Acting Director, HL; and Dr. J. R. Houston, Director, CHL; and under the direct supervision of Mr. W. H. McAnally, Chief, Waterways and Estuaries Division, HL. The engineer in immediate charge of the study was Mr. T. J. Pokrefke, Jr., Waterways and Estuaries Division. Mr. Pokrefke was assisted by Mr. D. A. Abraham and Ms. P. Hoffman, River Engineering Branch, Waterways and Estuaries Division; Mr. W. A. Thomas, Mobile Boundary Hydraulics, Clinton, MS; and Drs. C. R. Thorne and S. E. Darby, University of Nottingham, United Kingdom. Mr. Thomas, who has years of experience developing and applying numerical models, analyzed and reduced the historical channel cross sections. To address issues of bank stability, a contract was established with Drs. Darby and Thorne, who are recognized throughout the world for their work on stream bank analysis and the development of a bank erosion algorithm that is often referred to as the Darby-Thorne bank stability model. This report was prepared by Messrs. Pokrefke, Abraham, and Thomas; Ms. Hoffman; and Drs. Thorne and Darby.

This report is being published by the WES Coastal and Hydraulics Laboratory (CHL). The CHL was formed in October 1996 with the merger of the WES Coastal Engineering Research Center and Hydraulics Laboratory. Dr. James R. Houston is the Director of the CHL, and Messrs. Richard A. Sager and Charles C. Calhoun, Jr., are Assistant Directors.

During the course of the study, MRR was kept informed of the progress through monthly progress reports. Messrs. A. R. Swoboda and R. F. McAllister, MRR, visited WES to discuss study results and coordinate the study program.

This report was reviewed by Messrs. Swoboda, McAllister, and W. R. Stern, MRR, and Mr. J. I. Remus of the U.S. Army Engineer District, Omaha. An in-house WES technical review of this report was performed by Dr. R. R. Copeland, River Engineering Branch.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin, and Commander was COL Robin R. Cababa, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acres	4,046.873	square meters
acre-feet	1,233.489	cubic meters
cubic feet	0.02831685	cubic meters
degrees (angle)	0.01745329	radians
feet	0.3048	meters
miles (U.S. statute)	1.609347	kilometers
pounds (force) per square foot	47.88026	pascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
square feet	0.09290304	square meters
square miles	2.589998	square kilometers
tons (2,000 lb, mass)	907.1847	kilograms

1 Introduction

Background

The U.S. Army Engineer Division, Northwestern, Missouri River Region, (MRR), is currently conducting the Missouri River Master Water Control Manual Review and Update Study (Master Manual Study) in which it is trying to determine if the Current Water Control Plan (CWCP) or an alternative plan best meets the current needs of the Missouri River. MRR identified a potential increase in streambank erosion rates if the Corps were to change from the CWCP to the alternative identified as the Preferred Alternative (PA) in the Draft Environmental Impact Statement (DEIS) for the Master Manual Study. These potential impacts, however, were not quantified in the DEIS. Following the public's review of the DEIS, the U.S. Fish and Wildlife Service (Service) expressed a concern on another, somewhat related matter--the issuance of permits for additional streambank erosion control measures without some sort of assessment of the effects of some ultimate level of erosion control along the banks of the Missouri River. Service concerns for based on the unidentified cumulative impacts of additional control measures on environmental resources for fish and wildlife species with special concerns for the threatened and endangered species: the interior least tern, the piping plover, and the pallid sturgeon.

Scope of Study

MRR proposed a study to address the cumulative impacts on erosion of changing the operation of the main stem dams and adding additional streambank erosion control measures. The U.S. Army Engineer Waterways Experiment Station (WES) was tasked with defining the existing trends for streambank and channel bed erosion in relation to historic discharges. After these trends were quantified, the impacts of these two erosion types under the mode of operation of the PA were quantified. Four reaches totaling 362 miles[1] downstream from Fort Peck (189 miles), Garrison (79 miles), Fort Randall (36 miles), and Gavins Point (58 miles) Dams were included in this evaluation. As part of a habitat evaluation

[1] A table of factors for converting non-SI units of measure to SI units is found on page xiv.

effort, environmental features such as islands, sandbars, and backwater and chute habitats were analyzed. Other factors such as channel degradation, channel geometry, and turbidity were examined to the extent possible. Finally, the study also addressed the impacts of future streambank protection on the natural channel processes.

Study Tasks

The following study tasks were accomplished:

a. Existing data and prepared time lines for closure and filling of reservoirs, construction of bank protection, and operation schedules (discharges) were reviewed.

b. Existing data on rates of bank erosion, degradation, aggradation, and channel geometry changes were reviewed.

c. Aerial photographs to document the movement and/or size of vegetated islands, sandbars, backwater/chute habitat, and bank lines were analyzed.

d. The relationship between sandbar areal exposure and discharge were identified.

e. The Darby-Thorne bank erosion algorithm (Darby and Thorne 1995) was used to assess the relative potential for bank erosion with the present and PA operating schedules .

f. Where possible, turbidity trends were identified.

g. Where possible, a sediment budget for a reach to establish the relative importance of bank erosion and other factors was developed.

h. Bank and bed erosion, island and sandbar formation or movement, turbidity, channel geometry (area, width, and depth), and backwater or chute habitat were correlated.

i. Future trends with and without the PA were predicted.

j. Impacts were projected for various levels of increased stabilization of the bank lines within the four study reaches.

Data Available for Cumulative Impacts Study

Numerous technical studies by MRR for the Master Manual supported the DEIS. Much of the data and results presented in these MRR studies were used for the cumulative erosion impacts study conducted by WES and reported herein. This approach was followed for continuity between the earlier technical studies and the cumulative erosion impacts study. These earlier technical studies provided an excellent foundation for the cumulative erosion impacts study.

The following are data examples from MRR, the U.S. Army Engineer District, Omaha, and other sources as the cumulative impacts study progressed:

a. Aerial photography for Fort Peck Reach included 1974, 1975, 1976, 1983, 1990, and 1991; for Garrison Reach 1975, 1976, 1980, 1981, and 1990; for Fort Randall Reach 1975, 1976, 1982, 1991, and 1994; and Gavins Point Reach 1972, 1975, 1977, 1979, 1981, 1985, 1990, and 1994. These photographs were provided in hard copy to WES.

b. Channel cross-section surveys for Fort Peck Reach included 1948, 1949, 1950, 1951, 1952, 1953, 1954, 1955, 1956, 1958, 1960, 1966, 1973, and 1978; for Garrison Reach 1946, 1954, 1955, 1956, 1957, 1958, 1959, 1960, 1962, 1970, 1976, and 1985; for Fort Randall Reach 1952, 1953, 1954, 1955, 1956, 1957, 1958, 1959, 1960, 1962, 1967, 1970, 1975, 1985, and 1995; and Gavins Point 1955, 1960, 1965, 1970, 1974, 1980, 1986, and 1994. These data were provided to WES in digital form using the HECDSS (Data Storage System) format.

c. Water-surface profile and discharge data for various stations in the four study reaches were provided to WES in digital form.

d. "Missouri River Master Water Control Manual Review and Update, Draft Environmental Impact Statement, Executive Summary," July 1994, U.S. Army Engineer Division, Missouri River.

e. "Missouri River Master Water Control Manual Review and Update, Draft Environmental Impact Statement, Technical Report, Volume 5: Aggradation, Degradation, and Water Quality Conditions," July 1994, U.S. Army Engineer Division, Missouri River.

f. "Aggradation, Degradation, and Water Quality Conditions, Missouri River Mainstem Reservoir System," January 1994, U.S. Army Engineer District, Omaha.

g. "Fort Peck Dam - Downstream Degradation/Aggradation and Sediment Trends Study," April 1988, Draft Report prepared by Darrel Dangberg and Associates and River Pros, for the U.S. Army Engineer District, Omaha.

h. "1994-1995 Annual Operation Plan and Summary of Actual 1993-1994 Operations," December 1994, U.S. Army Engineer Division, Missouri River.

i. "Draft Biological Opinion on the Missouri River Master Water Control Manual Review and Study and Operations of the Missouri River Main Stem System," August 1994, U.S. Fish and Wildlife Service.

j. Water Quality and Sediment Data from U.S. Geological Survey offices in Montana, North Dakota, South Dakota, and Nebraska. Data were provided in hard copy or digital form.

k. Water Quality, Physical, and Sediment Data from the U.S. Environmental Protection Agency STORET System. Data were provided in hard copy or digital form.

l. Other miscellaneous data including, but not limited to, aerial mosaics, telephone conversations, written correspondence, and other reports prepared for the DEIS.

2 Time Lines and Data Reduction

Time Lines

Historic erosion trends were evaluated using time lines prepared for each study reach. Each time line began with the initiation of dam construction, ended with the year of the last aerial photograph used in the analysis, and included information relative to closure and pool filling, construction and location of bank protection, and yearly discharge information. Time frames covered on each reach included the periods used in the analysis relative to channel cross sections and aerial photographs. Discharge information includes (a) maximum discharge for the year, (b) days the discharge was less than the minimum discharge of either the PA or CWCP, (c) days the discharge was greater than the average daily discharge for the reach, (d) if applicable, the days the discharge was greater than the maximum CWCP discharge, and (e) if applicable, days the discharge was greater than the maximum PA discharge.[1]

The Omaha District supplied the digital data, in DSS format, used to analyze observed stages, water-surface elevations, and daily flow rates for the four study reaches. HECDSS was used to plot these data, and every DSS file covering reaches along the Missouri River was plotted, evaluated, and analyzed. Data from each gauge location covered varied periods. For instance, the Missouri River below Fort Peck Dam has observed daily flow for the years 1934-1995, while the Missouri River, Bismarck, ND, gauge location has observed daily flow for the years 1928-1995. Observed daily flow data were tabulated in a spreadsheet and sorted from largest to smallest discharge for the selected time periods in each reach. These rankings determined the number of days the flows exceeded the predetermined maximum or minimum CWCP and PA discharge values within the

[1] All references to discharges for the CWCP and the PA in this report are to the average monthly values for a 96-year period of modeling using inflows to the Missouri River main stem system of dams from 1898 to 1993. The maximum or minimum flow, in terms of average monthly value for a given month in a single year, can be much higher or lower than the identified maximum or minimum monthly average value.

reaches; however, while these data are in tables for analysis, they are not included in this report.

The Fort Peck Reach time line from 1960 River Miles (RM) 1,771 to 1,582 is shown in Table 1. The discharges *Q* were obtained at the discharge range listed in the Omaha District data as "Below Ft. Peck Dam" for 1955-1990. The following tabulation is a summary and categorization of the discharges of Table 1:

Year(s)	Flow Classification	Explanation
1956, 1957, 1958, 1962, 1963, 1964, 1987	Low	200 days or more with Q<7,000 cfs
None	Medium	200 days or more with Q<9,000 cfs Less than 100 days with Q<11,000 cfs
1965, 1966, 1967, 1968, 1969, 1970, 1971, 1972, 1974, 1975, 1976, 1978, 1979, 1980, 1981, 1982, 1984, 1985, 1989	High	200 days or more with Q>9,000 cfs 100 days or more with Q>11,000 cfs
1965, 1966, 1969, 1970, 1971, 1972, 1975, 1976, 1978, 1979, 1981, 1982	Very high	200 days or more with Q>9,000 cfs 100 days or more with Q>11,000 cfs 50 days or more with Q>14,000 cfs
1957 and 1958	Lowest maximum	7,500 cfs
1975	Highest maximum	35,400 cfs

Criteria used in delineation of the discharges were based on 200 days representing more than half a year; 100 days, approximately 3 months; and 50 days, more than 1 month but less than 2 months. The PA and CWCP blocked discharges are based on a 1-month time frame; therefore, 50 days appeared a reasonable delineation point. Based on the limits for flow classifications, the years of 1955, 1959, 1960, 1961, 1973, 1977, 1983, 1986, 1988, and 1990 did not fall into a particular flow class. A review of Table 1 indicates that 1955, 1961, 1973, 1977, 1983, and 1988 are near medium flow conditions. The years of 1959, 1960, 1986, and 1990 are close to low-flow conditions.

The Garrison Reach time line (RM 1,390 to 1,311) is shown in Table 2. The discharges were obtained at the discharge range listed "Bismarck, North Dakota," RM 1,314.2 downstream of the study reach for 1956-1990. The following tabulation is a summary and categorization of the discharges of Table 2. In this case, 50 days was used instead of 200 days to show discharges less than the PA minimum of 12,500 cfs since there were only 4 years (1956, 1960, 1962, and 1963) with more than 100 days where the discharge was less than 12,500 cfs. This is probably a function of using the tabulated discharges at Bismarck rather than closer to Garrison Dam. It should be noted that the years of 1958, 1959, 1964, 1973, 1977, 1981, 1983, 1985, 1987, and 1989 did not fall within the flow classification limits. A review of Table 2 indicated that 1958, 1964, 1973, 1977, 1983, 1985, 1987, and 1989 had flows close to medium-flow conditions while 1959 was near the low-flow classification.

Year(s)	Flow Classification	Explanation
1956, 1957,1960, 1961, 1962, 1963, 1966, 1988, 1990	Low	50 days or more with Q<12,500 cfs
1974, 1980, 1984, 1986	Medium	200 days or more with Q>22,500 cfs Less than 100 days with Q>29,000 cfs
1965, 1967, 1968, 1969, 1970, 1971, 1972, 1975, 1976, 1978, 1979, 1982	High	200 days or more with Q>22,500 cfs 100 days or more with Q>29,000 cfs
1965, 1967, 1969, 1970, 1972, 1975, 1976, 1978, 1979, 1982	Very high	200 days or more with Q>22,500 cfs 100 days or more with Q>29,000 cfs 50 days or more with Q>31,500 cfs
1959	Lowest maximum	21,700 cfs
1975	Highest maximum	68,800 cfs

Table 3 shows the Fort Randall Reach time line (RM 880 to 844). The discharges were obtained at the discharge range listed "At Fort Randall Dam" for 1954-1994. The following tabulation is a summary and categorization of the discharges of Table 3:

Year(s)	Flow Classification	Explanation
1954, 1956, 1957, 1958, 1959, 1960, 1961, 1962, 1963, 1964, 1965, 1989, 1990, 1991, 1992, 1993	Low	100 days or more with Q<13,300 cfs
1967, 1968, 1974, 1977, 1979, 1980, 1981	Medium	200 days or more with Q>25,500 cfs Less than 100 days with Q>35,400 cfs
1969, 1970, 1971, 1972, 1975, 1976, 1978	High	200 days or more with Q>25,500 cfs 100 days or more with Q>35,400 cfs
1969, 1970, 1971, 1972, 1975, 1976, 1978	Very high	200 days or more with Q>25,500 cfs 100 days or more with Q>35,400 cfs 50 days or more with Q>37,300 cfs
1993	Lowest maximum	26,000 cfs
1975	Highest maximum	60,600 cfs

The years 1955 (no data available), 1966, 1973, 1982-1988, and 1994 are not included because they did not fit within the classification limits. However, a review of Table 3 indicates that 1966, 1973, 1982-1988, and 1994 are close to the medium flow classification.

Table 4 shows the Gavins Point Reach time line (RM 811 to 753). The discharges were obtained at the discharge range listed "At Yankton, South Dakota" and covers 1960-1994. The following tabulation is a summary and categorization of the discharges of Table 4.

Year(s)	Flow Classification	Explanation
1960, 1961, 1962, 1963, 1964, 1965, 1981, 1989, 1990, 1991, 1992, 1993	Low	100 days or more with $Q<16{,}000$ cfs
1967, 1968, 1974, 1977, 1980, 1982, 1985, 1987, 1988	Medium	200 days or more with $Q>27{,}500$ cfs Less than 100 days with $Q>37{,}000$ cfs
1969, 1970, 1971, 1972, 1975, 1976, 1978, 1979, 1986	High	200 days or more with $Q>27{,}500$ cfs 100 days or more with $Q>37{,}000$ cfs
1969, 1970, 1971, 1972, 1975, 1976, 1978, 1979, 1986	Very high	200 days or more with $Q>27{,}500$ cfs 100 days or more with $Q>37{,}000$ cfs 50 days or more with $Q>40{,}000$ cfs
1993	Lowest maximum	24,100 cfs
1975	Highest maximum	63,400 cfs

The years 1966, 1973, 1983, 1984, and 1994 are not included because they did not fit in the flow classification limits. However, a review of Table 4 indicates they are close to the medium-flow classification.

River Mileage

Data provided for this study were based on mileages determined in 1941 and 1960. The cross-sectional data were based on 1941 river miles; however, almost all other data and referenced reports used 1960 river miles. Table 5 presents specific locations in the various reaches and the conversion from 1941 to 1960 river miles by Omaha District. Table 6 was prepared by WES and presents the conversion from 1941 to 1960 river miles for the historical channel-sectional data, also provided by Omaha District. For the study presented herein, only 1960 river miles will be used.

Aerial Photographs

Numerous aerial photographs were used to evaluate islands, sandbars, chute fills, and other channel processes because an adequate analysis required that the photographs for a given reach be taken with similar flow conditions as well as covering the greatest time period. Therefore, the aerial photographs in the following tabulation were used for the specific reaches indicated.

Reach	Date of Photograph	Discharge, cfs
Fort Peck	16 August 1974 25 to 26 October 1990	12,200 7,900
Garrison	10 October 1976 25 October 1990	13,400 10,300
Fort Randall	17 October 1976 4 May 1994	38,000 29,500
Gavins Point	6 June 1981 5 May 1994	32,000 30,600

Aerial photographs were analyzed as follows: (a) the photographs were assembled, (b) the vegetated islands and sandbars, chute fills, channel border fills, and tributary fills were identified, (c) the water to "land" limits were digitized in AutoCAD, and (d) the areas of the islands and sandbars, chute fills, channel border fills, and tributary fills were computed in acres in AutoCAD (note that the nonvegetated beaches and other areas attached to islands were included as part of the island area). For this study, islands are defined as vegetated channel attributes more or less in the river channel surrounded by water at all flows. Sandbars are unvegetated channel attributes similar to but usually smaller than islands. Chute fills are depositional areas between an island and the bank line that can tend to become vegetated over time. Channel border fills are depositional areas adjacent to the bank line that can tend to become vegetated over time. Tributary fills are the depositional, deltalike attributes resulting from tributary sediments depositing near the confluence of the tributary and the Missouri River. In some instances, over the time frames covered by the aerial photographs, chute fills and channel border areas became attached to the bank lines and were indistinguishable from their original classification. Therefore, those ending aerial photographs were not digitized, and it was assumed those habitats had been converted, more or less, to terrestrial habitats. The tributary fills were not used in this study, but were digitized to ensure that they were accounted for in the total coverage or not attributed to another attribute such as channel border fills. Examples of the digitized results for one segment of the Fort Randall Reach for 1976 and 1994 are illustrated in Figures 1 and 2, respectively.

Channel Cross Sections

The historical channel cross sections were analyzed and reduced by Thomas[1] using the options in some numerical model codes. The majority of the discussion in "Channel Cross Sections" was taken from Thomas.

The bed and bank erosion was analyzed using measured cross sections called Sedimentation Ranges. Calculations were made using the $VOLUME option in

[1] William A. Thomas. (1996). "The analysis of sedimentation ranges and turbidity along the Missouri River," prepared by Mobile Boundary Hydraulics, Clinton, MS, for U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

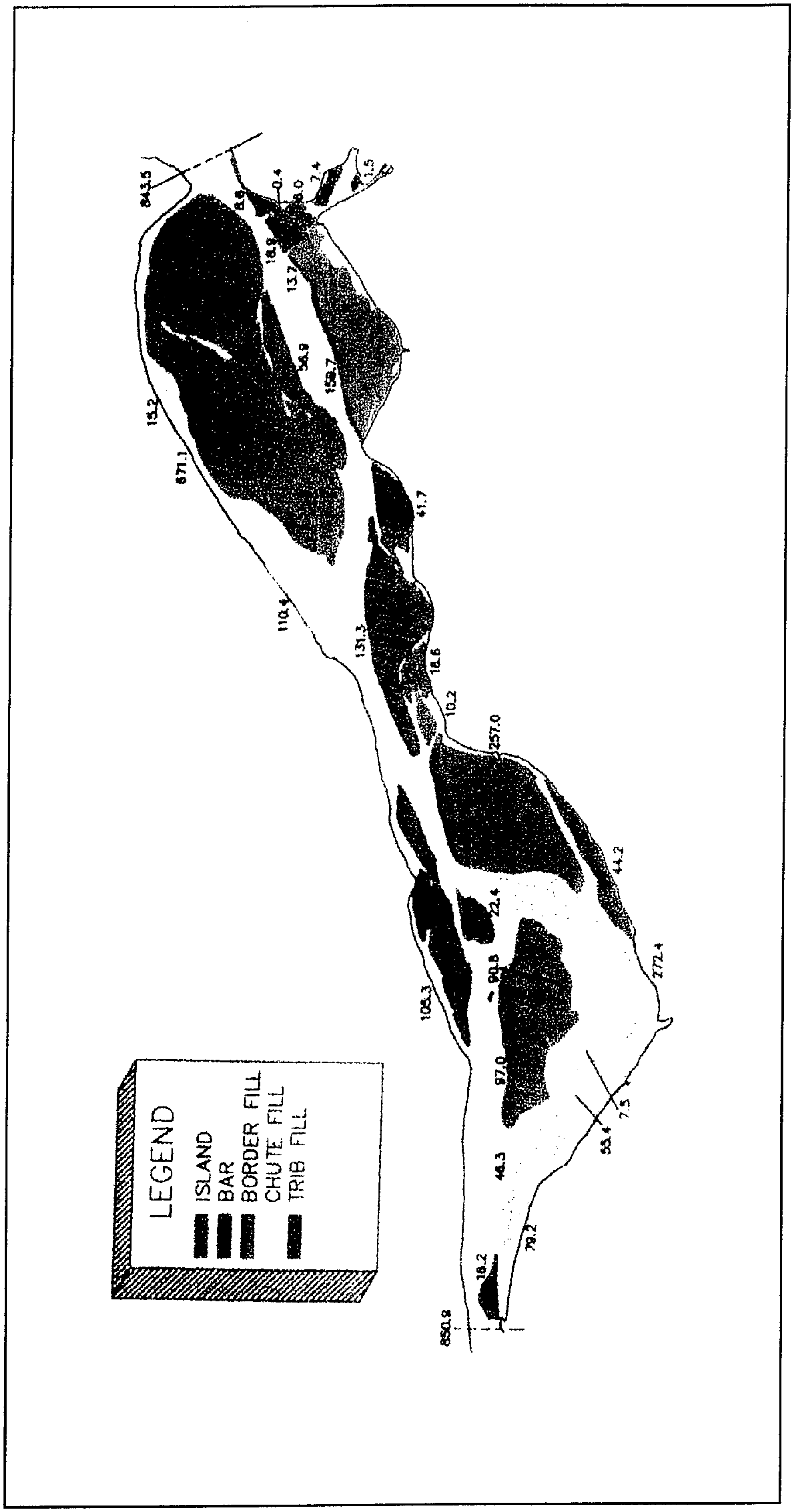

Figure 1. Example of initial digitized aerial photographs for Fort Randall Reach, 1976

Figure 2. Example of final digitized aerial photographs for Fort Randall Reach, 1994

the computer program, "Sedimentation in Stream Networks (HEC-6T)," which was originally developed to calculate the reduction in reservoir storage capacity curves resulting from sediment deposits. In this analysis, the sloping plane feature was used rather than the horizontal plane calculation that is appropriate for reservoir storage calculations. Geometric calculations in the HEC-6 program determine cross-sectional end areas and top widths from HEC-2 style input data sets using trapezoidal integration rather than making sedimentation calculations. Data were converted to HEC-2 style input allowing a more efficient use of the Omaha District sedimentation range data stored in DSS files.

Basic data

The sedimentation ranges were furnished to Mr. Thomas by WES in the following DSS files:

File Name	Bytes	Date	Time	Description
DGARDE.DSS	592,380	11-06-95	2:26p	Garrison Reach
DGAVDE.DSS	517,116	11-18-94	2:37p	Gavins Point Reach
DPECDE1.DSS	319,484	10-21-95	3:36p	Fort Peck Reach, File 1 of 2
DPECDE2.DSS	487,420	04-28-92	9:39a	Fort Peck Reach, File 2 of 2
DRANDE.DSS	488,444	05-10-94	1:44p	Fort Randall Reach

Plates 11, 38, 80, and 101 of U.S. Army Engineer Division, Northwestern, Missouri River Region (USAEDNMRR) (1994a) show the locations of these monumented ranges. The cross-sectional area and top width were saved for each range and then calculated by subsection. That is, each cross section was partitioned into three parts, called subsections: left bank subsection, main channel subsection, and right bank subsection.

Selection of survey dates

The periods used for this analysis were selected to coincide with available aerial photographs discussed in the section, "Aerial Photographs." The resulting changes in cross-sectional area due to erosion and deposition were used to compare changes in island density, sandbar density, and off-channel deposits that were obtained from measuring those features on successive aerial photographs. Three years of sedimentation range data were selected for each reach as shown in the following tabulation. For the Garrison and Fort Randall Reaches, a significant number of cross sections were missing from the years when these projects began operation. All available cross sections were used, and missing ones were replaced with other cross sections obtained from succeeding years.

Reach	1960 River Mile		Year		
	From	To	1	2	3
Fort Peck	1,771.0	1,582.0	1955	1966	1978
Garrison	1,390.0	1,311.0	1956-58	1976	1985
Fort Randall	880.0	844.0	1954-60	1975	1985
Gavins Point	811.0	753.0	1960	1974	1986

Approach

The data from each of the 3 years and each cross section were overplotted for each reach; the left and right channel stations were selected. However, for this step, the channel bottom stations were selected rather than the top bank stations; therefore, end areas that are printed for the left bank are actually the end area of bank erosion for the left side of the cross section. Likewise, end areas that are printed for the right bank are the end area of bank erosion for the right side of the cross section. End areas printed for the channel subsection are actually erosion of the channel bed. An example of a plotted cross section is shown in Plate 1. Left and right channel bed stations are shown by circles.

The change in cross-sectional end areas was calculated by placing a sloping plane over each survey and calculating the end area of each cross section beneath that plane. This sloping plane is called the computational plane, and its height was selected to ensure that all cross-section stations that changed during the period of the surveys were lower than the plane elevation. Starting elevation[1] and slope of the computational plane are shown in the following tabulation:

Reach	Sedimentation Range at Downstream End	Elevation of Computational Plane at Downstream End	Slope of Computational Plane
Fort Peck	1,599.0	1,920	0.0001821
Garrison	1,336.2	1,675	0.0001244
Fort Randall	844.2	1,234	0.0001618
Gavins Point	753.1	1,142	0.0002139

When end area changes are calculated by subtracting successive surveys, it is imperative that all data sets for a given range start and stop at the same cross-section station; however, surveys are not usually that precise. Therefore, each range was tested, and the stations were extended to the maximum and minimum values for that range in all cases when the survey fell short. This effort used all sedimentation ranges, a total of 157, in the total distance of approximately 370 river miles. Cross-sectional areas were calculated at three points in time; the results were subtracted to determine how much change in end areas had occurred

[1] All elevations (el) cited herein are in feet referred to mean sea level (msl).

during each time period. Changes in end areas were calculated in three parts: left bank, channel bed, and right bank. WES interpreted the results and used them in erosion or deposition volume computations presented later in this report.

Results

The change in the cross-sectional area was calculated for each period of time. Tables 7-14 show the results and the change in channel cross-sectional area at each sedimentation range in the four reaches between the survey dates. Column 1 shows sedimentation ranges using 1941 river miles. This is the reference used in the basic data files (DSS). Column 2 shows the 1960 river miles for those ranges. Columns 9, 10, 11, and 12 show the calculated erosion (negative values) or deposition (positive values).

Column 9 is labeled LOB, which stands for left bank, but is actually the left bank of the cross section because the left channel station was selected at the toe of the bank rather than at the top of the bank. Column 10 shows bed erosion or deposition. Column 11 applies to the right bank of the cross section in a similar fashion to column 9. Column 12 is the total change in cross-sectional end area calculated by summing columns 9, 10, and 11. Column 13 is the total change in bank cross-sectional area obtained by adding columns 9 and 11.

The cross-sectional area for the bed and banks at each sedimentation range was used to compute the volume contributed by each to the documented aggradation in the reservoir immediately downstream. However, before the volume of bed and bank material scoured was computed, the link between the area scoured at individual cross sections and planform had to be established. The approach taken was to compute the river surface area between adjacent sedimentation ranges and compare that to measurements and results of cumulative bank erosion from previous reports. The Fort Peck Reach was analyzed somewhat differently from the Garrison and Fort Randall Reaches due to previous studies.

The DSS cross-section files plotted by WES and used by Thomas[1] had varied time frames. Some surveys were made within a year and some had longer intervals between them. Creation of these plots produced an enormous amount of data that were used later during the analysis. In addition to separate cross sections plotted, various dated cross sections at the same range were overplotted to determine any changes that occurred.

Bed and Bank Erosion Volume Computations

Thomas' channel bed and bank erosion or deposition analysis[1] provided data in a format for additional analysis. These data were used to generate various graphs

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[1] Thomas, op. cit.

of total degradation and aggradation of the channel bed and banks at each sedimentation range versus island and sandbar density. The significance of the signs (negative and positive) shown in Tables 7-14 was maintained when adding the area of the left and right banks. Any graphs developed in this effort are presented later in this report.

Volume changes of the channel banks and bed were determined by calculations based on the cross-section end area at each sedimentation range discussed in the preceding section. Volumes were computed in acre-feet by the end-area method using the following general equations (between two consecutive ranges at RM 1 and RM 2, for this example):

a. Bank Volume Equation:

$$(RM1 - RM2) * 5280 * \frac{(RM1LOB + RM1ROB + RM2LOB + RM2ROB)/2}{43560} \qquad (1)$$

b. Channel Volume Equation:

$$(RM1 - RM2) * 5280 * \frac{(RM1CH + RM2CH)/2}{43560} \qquad (2)$$

LOB represents the left bank, ROB the right bank, and CH the channel bed area. Through all these computations, the numerical signs of the input and the results were maintained because a negative sign indicates erosion or degradation and a positive sign indicates aggradation. Calculations were performed on all appropriate sedimentation ranges for the four study reaches. It should be noted that in some years no sedimentation range data were available. For instance, for the 1955 and 1978 ranges, there were no cross-sectional data at the Fort Peck Reach RM 1,707.7 and 1,707.5 (1978 set). Therefore, those ranges were not included in the volume computations for that time frame.

Bank and channel bed calculated volume computations, in units of acre-feet, were input into Tables 15-18, which display the values between each range along the reach. Computations were made for two time frames in the four study reaches, and those results are presented in Chapter 5 relative to the discussions of volumes scoured from the bed and banks over time.

3 Operation Plans and Hydrographs

Introduction

The MRR model for the CWCP for the Missouri River computes monthly riverflow and reservoir release data between 1898 and 1993. Operating criteria for the CWCP were first published in the Master Manual for the Missouri River in 1960, and they have been revised to meet changing needs for the Missouri River. The PA included in the DEIS prepared for the current review of the Master Manual was based on developing another method for operation of the dams following numerous studies on various impacts of altering the CWCP. In fact, a very large number of alternatives resulted from the 1989 MRR study. Final PA selection was based on the analysis of numerous criteria considering navigation, flood control, recreation, social, economic, and environmental issues.

This chapter analyzes similarities and dissimilarities between the CWCP and PA in the DEIS (USAEDNMRR 1994a). Also, because the CWCP is what the name implies–a plan–this study considered differences between the CWCP and the flows that actually passed through the four reaches. While operating plans are useful for general guidance, it was necessary to evaluate the actual hydrographs. Even with the wide potential latitude available in the operation of the Missouri River Main Stem System, the natural occurrences of floods and droughts have required short- or long-term modifications to the CWCP for the various project purposes associated with the Missouri River Main Stem System.

Current Water Control Plan

The modeling output on the CWCP consists, in part, of monthly average discharges for the present plan of operation of the Missouri River Main Stem System. The monthly average discharges over the 96-year modeling period of 1898 to 1993 for the Fort Peck, Garrison, Fort Randall, and Gavins Point Reaches are presented in the following tabulation:

Monthly Average Discharges, CWCP, 1,000 cfs, 1898-1993				
Month	**Fort Peck Reach**	**Garrison Reach**	**Fort Randall Reach**	**Gavins Point Reach**
January	10.09	21.73	14.45	16.02
February	10.13	21.87	13.59	15.75
March	8.11	20.92	16.50	19.82
April	7.28	21.93	25.69	28.52
May	8.96	22.24	27.45	30.12
June	8.14	21.73	28.15	30.71
July	7.66	23.23	31.69	32.91
August	7.71	21.62	35.23	36.22
September	10.62	31.49	35.43	36.95
October	11.10	26.11	33.54	35.46
November	9.08	18.36	29.67	31.58
December	8.99	19.48	16.35	17.85

This tabulation shows, relative to the entire year, that (a) Fort Peck Reach discharges increase in the winter and fall; (b) Garrison Reach discharges increase in the fall; (c) Fort Randall Reach discharges increase also in the summer and fall; and (d) Gavins Point Reach discharges increase in the summer and fall in support of downstream navigation. Plates 2-5 show the plots of the CWCP and PA discharges as annual hydrographs for the four reaches.

Preferred Alternative

The PA was the operational plan proposed for the Missouri River Main Stem System in the DEIS. This plan has also been modeled on a monthly basis over the same 96-year period; therefore, monthly average discharges have been computed for this plan. It should be noted that the average daily discharge, based on the entire year for the CWCP and the PA, is approximately the same for the four individual reaches. Therefore, both hydrographs provide the same total volume of water each year, but do so in different distributions during the year. Average monthly discharges for Fort Peck, Garrison, Fort Randall, and Gavins Point Reaches are presented in the following tabulation.

This tabulation shows, relative to the entire year, that (a) Fort Peck Reach discharges increase in the winter and early summer; (b) Garrison Reach discharges increase in the spring and summer; (c) Fort Randall Reach discharges increase in the spring and early fall; and (d) Gavins Point Reach discharges increase in the spring and early fall. Plates 2-5 show the PA average monthly discharge plots as annual hydrographs for the four reaches superimposed on the CWCP discharge.

Monthly Average Discharges, PA, 1,000 CFS, 1898-1993				
Month	Fort Peck Reach	Garrison Reach	Fort Randall Reach	Gavins Point Reach
January	9.43	21.03	14.32	15.88
February	9.42	20.08	13.30	15.46
March	8.51	23.26	20.78	24.10
April	7.70	26.74	36.10	38.95
May	9.95	28.31	37.34	40.03
June	14.09	29.05	30.20	32.76
July	8.27	23.73	27.62	28.84
August	7.36	20.27	29.93	30.93
September	8.80	25.68	33.62	35.15
October	8.65	20.81	27.67	29.59
November	6.91	12.72	18.96	20.87
December	8.61	18.01	15.69	17.17

Actual Average Hydrographs

The cumulative erosion impacts addressed in this report were assumed to be significantly influenced by the discharges in the four study reaches. Other factors such as geotechnical conditions, freeze-thaw processes, bank stabilization, etc., have an influence, but the energy provided by the flowing water is a much greater influence. To analyze those impacts, it is imperative to know what flow conditions have occurred on the river. The responses of islands, sandbars, chute fills, channel border fills, banks, and the channel bed that are presented should not be attributed to the CWCP, but rather to the actual flow conditions in the reaches.

Inspection of daily discharge hydrographs over many years to evaluate the actual flow conditions is extremely difficult and virtually impossible. In lieu of this very difficult process, historic daily average discharges for varying periods for the four reaches were computed and used in this analysis. Those daily averages were then plotted as yearly hydrographs and compared to the plots for the CWCP and PA. Two other data sets were used in this overall study–aerial photographs and channel cross sections. However, because the data for certain periods were limited, the time frames for the aerial photographs and channel cross sections overlapped only to a limited degree. Therefore, average annual hydrographs for two different periods were computed and plotted for each of the four reaches. The first hydrograph covered the time period used in the analysis of the channel cross-section data, and the second hydrograph covered the period used in the analysis of the aerial photographs.

Daily average discharge hydrographs were developed by averaging the discharges on 1 January of each year over the period and repeating the process for 2 January and so on to 31 December. For continuity, the data for 29 February (leap years) were dropped from the computations; therefore, all years used in the averages had 365 days. For the Fort Peck Reach, the years between 1955 and 1978 (channel cross-section period) and between 1974 and 1990 (aerial photograph period) were averaged. Daily average hydrograph plots for 1955-1978 and 1974-1990 were superimposed on the CWCP and PA hydrographs (Plates 6 and 7, respectively). For the Garrison Reach, the years between 1956 and 1985 (channel cross-section period) and between 1976 and 1990 (aerial photograph period) were averaged. Daily average hydrograph plots for 1956-1985 and 1976-1990 were superimposed on the CWCP and PA hydrographs (Plates 8 and 9, respectively). For the Fort Randall Reach, the years between 1954 and 1985 (channel cross-section period) and between 1976 and 1994 (aerial photograph period) were averaged. Daily average hydrograph plots for 1954-1985 and 1976-1994 were superimposed on the CWCP and PA hydrographs (Plates 10 and 11, respectively). For the Gavins Point Reach, the years between 1960 and 1986 (channel cross-section period) and between 1981 and 1993 (aerial photograph period) were averaged. Daily average hydrograph plots for 1960-1986 and 1981-1993 were superimposed on the CWCP and PA hydrographs (Plates 12 and 13, respectively).

One final set of daily average discharge hydrographs was also calculated to assist in channel cross-section data analysis of bed and bank scour. As discussed in the section "Selection of Survey Dates," three sets of channel cross sections over a relatively wide time span were analyzed for each study reach. Documentation of the average daily discharges in the two periods between those three channel cross-section sets would help to quantify responses. Although the hydrographs developed during this effort are not presented in this report, the time frames chosen to compute the additional average daily discharges were as follows: (a) Fort Peck Reach, 1955-1966 and 1967-1978; (b) Garrison Reach, 1956-1976 and 1977-1985; (c) Fort Randall Reach, 1954-1974 and 1975-1985; and (d) Gavins Point Reach, 1960-1974 and 1975-1986. These additional computations were used to help quantify and classify the flow conditions in those time frames.

Annual Water Volumes

Comparing various hydrographs is informative; however, it does not necessarily lend itself to appreciating differences between hydrographs. For example, comparing the CWCP and PA monthly discharge hydrographs for the Fort Peck Reach (Plate 2) does not provide a true understanding of these differences. Differences and similarities will be put together to provide a basis for the various analyses to follow.

River engineers typically use annual average water volumes as a method to quantify total riverflow for an entire year; therefore, different years can be compared. High water years are years the river passed the largest or greater than average water volume. Low water years are years the river passed less water volume than average. To quantify these volumes, the average annual water volumes for the CWCP, PA, overall channel cross-section time frame, aerial photograph time frame, and the first and second time frames between the channel cross-section analysis for each study reach were further analyzed and the data reduced to acre-feet of water.

The CWCP and PA annual average water volumes were computed using the monthly averages and number of days in the respective month for that flow. Annual water volume for the average daily hydrographs was obtained by summing the individual days within the average daily hydrograph. The results of those computations are given in the following tabulation.

The total water volume passing downstream for the CWCP and PA is essentially equal for each of the four reaches, as shown by the data in the tabulation. Fort Peck Reach time frames for 1955-1978, 1974-1990, and 1967-1978 would be classified as high water periods and 1955-1966 as a low water period relative to the CWCP and PA. All four periods for the Garrison Reach would be classified as high water conditions relative to the CWCP and PA. However, as stated previously, the discharges used for the Garrison Reach were obtained from the Bismarck, ND, gauge and may be somewhat higher due to tributary inflow between the dam and the gauge. Fort Randall Reach, between the time frames 1954 and 1985, 1976 and 1994, and 1954 and 1974 would be classified as low water periods while 1975-1985 would be classified as a high water period relative to the CWCP and PA. Gavins Point Reach time frame 1960-1986 would be classified as a relatively average period. However, considering the separate periods of 1960-1974, a low water period, and 1975-1986, a high water period, which constitute the entire period of 1960-1986, it can be seen that separating the time frames has some value. The time frame 1981-1993 in the Gavins Point Reach would be classified as a low flow period.

Average Channel Velocities

Since there is virtually no difference between the CWCP and PA total water volumes passing downstream, a final effort to quantify any differences between the two hydrographs was conducted by analyzing average velocities. This approach is reasonable since water velocity, not discharge, provides an indication of the energy available to the stream.

At any given point in a stream, the average velocity V is equal to the discharge Q divided by the cross-sectional area A of the stream; therefore, there is a direct relationship between velocity and discharge in a stream. River engineers typically associate the discharge of the stream with particular stream attributes in determining the hydraulic geometry of the stream. Hydraulic geometry is the required width and depth that a particular stream needs to carry the flows imposed on it.

Hydrograph	Water Volume millions of acre-ft
Fort Peck Reach	
CWCP (1898-1993)	6.50
PA (1898-1993)	6.49
1955-1978	7.06
1974-1990	7.50
1955-1966	5.88
1967-1978	8.24
Garrison Reach	
CWCP (1898-1993)	16.33
PA (1898-1993)	16.28
1956-1985	17.29
1976-1990	17.21
1956-1976	17.07
1977-1985	17.82
Fort Randall Reach	
CWCP (1898-1993)	18.61
PA (1898-1993)	18.47
1954-1985	17.34
1976-1994	16.68
1954-1974	16.10
1975-1985	19.59
Gavins Point Reach	
CWCP (1898-1993)	20.06
PA (1898-1993)	19.93
1960-1986	20.26
1981-1993	17.95
1960-1974	18.77
1975-1986	22.12

Over the years, geomorphologists and river engineers have determined that the channel width, depth, and sediment-carrying capabilities of a stream are related to stream discharge. Leopold, Wolman, and Miller (1964) presented the following equations:

$$w = aQ^b \tag{3}$$

$$d = cQ^f \tag{4}$$

$$G = pQ^j \tag{5}$$

where

w = channel width

Q = discharge

d = channel depth

G = sediment load

a, b, c, f, p, j = numerical coefficients

Because the velocity V and the discharge Q are directly related, a relationship also exists between the velocity and the width, depth, or sediment load. However, the exact relationship is beyond the scope of this study and is not needed for the analysis. The important issue is that the parameters used to describe the channel (width and depth) are power functions of the discharge or velocity as is the sediment load carried by the stream. Therefore, an increase in the discharge (or velocity) may precipitate an increase in any or all the parameters: width, depth, or sediment load. Conversely, a decrease in the discharge (or velocity) may cause a decrease in the required magnitudes of the parameters.

Average velocities expected with the CWCP or PA values were computed for the 12 monthly flows in each plan for the four study reaches. Discharges Q were obtained directly from the CWCP or PA. A specific stage-discharge rating curve was then used to determine the water-surface elevation for each discharge. The following tabulation lists the stage-discharge rating curve information used in this study:

Reach	Location Description	Curve Dates	USGS Gauge Number	Gauge Zero ft above msl
Fort Peck	Missouri River below Fort Peck Dam (2.61 m downstream)	1987 to present	06132000	2,018.00
Garrison	Missouri River at Bismarck, ND	1991 to present	06342500	1,618.28
Fort Randall	Missouri River at Greenwood, SD	1987 to present	RM 907.6	Rating curve read in actual elevations
Gavins Point	Missouri River at Yankton, SD	1990 to present	06467500	1,139.68

The average velocity for each flow was based on dividing the discharge by the computed channel cross-sectional area at that location. Area computations for the four reaches were obtained using the following: at Fort Peck Reach, the channel cross section obtained by the U.S. Geological Survey (USGS) in May 1996 during discharge measurements; at Garrison Reach, the October 1996 USGS discharge channel cross section; at Fort Randall Reach, the Omaha District sedimentation range for September 1985 cross section (RM 865.1) at Greenwood, SD; and at Gavins Point Reach, the August 1986 channel cross section in the Omaha District sedimentation range data at Yankton, SD (RM 805.8).

Percent exceedence curves were developed for velocities for the CWCP and PA. Plates 14-17 show those results for the Fort Peck, Garrison, Fort Randall, and Gavins Point Reaches, respectively. Plate 14 shows that the Fort Peck Reach CWCP and PA velocities are essentially identical to about the 33 percent exceedence point, then the PA velocities are less than the CWCP velocities except for the maximums, which differ by about 0.75 fps. The Garrison Reach PA average velocities are lower than the CWCP average velocities to about the 50 percent exceedence point (Plate 15). From there, the PA average velocities are higher than the CWCP average velocities, but the maximum deviation between the two plans is about 0.15 fps. The Fort Randall Reach average CWCP and PA velocities are nearly identical with the largest variance about 0.12 fps (Plate 16). The Gavins Point Reach has similar deviations between the CWCP and PA average velocities. The maximum difference is about 0.25 fps; however, the majority of the two curves are virtually superimposed on each other (Plate 17).

Wolf Point and Culbertson, MT, are USGS discharging stations in the Fort Peck Reach. The comparison of velocity differences between the CWCP and PA at those locations provides an opportunity to evaluate erosion potential due to changes in velocities starting downstream of the dam (at USGS Gauge No. 06132000 discussed previously) and proceeding downstream past Wolf Point to Culbertson. The pertinent data for the rating curves used at these two locations are listed in the following tabulation:

Location Description	Curve Dates	USGS Gauge No.	Gauge Zero ft above msl
Missouri River near Wolf Point, MT	1984 to present	06177000	1958.57
Missouri River near Culbertson, MT	1990 to present	06185500	1883.4

The channel cross sections used for the average velocity computation were obtained from the USGS discharge measurement notes for October 1996 at both gauges.

Plate 18 is a plot of the average velocities versus exceedence for the CWCP and PA for the Wolf Point and Culbertson locations. The similar plot from the Fort Peck Reach below Fort Peck Dam (Plate 14) is included in Plate 18 for comparison. This plate indicates that from upstream to downstream in the Fort Peck Reach, the average velocities and the differences at the gauge sites between CWCP and PA average velocities decrease. The greatest difference in average velocities is at the maximum for all locations, but that difference decreases from 0.76 fps at the Fort Peck gauge to 0.40 fps and 0.27 fps at Wolf Point and Culbertson, respectively. The decrease in average velocities between the CWCP and PA indicates that erosion should decrease in the downstream direction and the potential for erosion in the PA should be about what it is with the CWCP.

Summary

There are obvious and consistent differences between the CWCP and PA for each of the four study reaches. The PA tends to move the peak plan discharges into the spring while the CWCP tends to have the hydrograph peaks occur later in the year. A comparison of the two plans shows that the 96-year average monthly peak discharges for the Fort Peck and Gavins Point Reaches will increase approximately 3,000 cfs while the peak discharge for the Fort Randall Reach will increase approximately 2,000 cfs. The Garrison Reach 96-year average monthly PA peak discharge will be about 2,500 cfs lower than the CWCP peak. In each reach, both plans will convey about the same annual water volume. Based on the average velocity computations presented, for the range of flows in the CWCP and PA for the entire year, there are insignificant differences in the magnitude of velocities between the two plans or the differences occur for a short period of time. Relative to the amount of energy provided by the water in the stream to the bed and banks, which is a function of the average velocities, the PA should not have any greater impacts than the CWCP. Because the channel width, depth, and sediment load are functions of the discharge or velocity, the PA should have little impact on those parameters versus what has developed using the CWCP. Based on the average annual water volumes, there is significantly greater variation among actual annual hydrographic events than the variations between the CWCP and the PA.

4 Impacts on Islands, Sandbars, and Channel Border Fills

Introduction

The impact of erosion changes on islands and sandbars is a major point of interest in this study, specifically whether any relationships exist between channel bed or bank scour and increases or decreases in island and sandbar density (acres per mile). The islands and sandbars for all four reaches were studied using aerial photograph mosaics, sedimentation range cross sections, digitized aerial photographs, and observed riverflows.

As degradation downstream of the dams occurs over time, one possible geomorphic response could be to produce islands and sandbars with very little sediment transport. As the channel degrades and becomes incised, former bed forms could become exposed as sandbars and islands. Another process taking place in these reaches is eroded material from the upstream and middle reaches becoming available to deposit in the downstream portion of the reach where the water-surface slopes are flatter. This deposition has the potential to increase island and sandbar densities in the downstream portion.

Islands and Sandbars

Fort Peck Reach

This analysis started with the data from the digitized aerial 1974 and 1990 photographs. Table 19 shows island and sandbar densities in acres per mile. There are no island or sandbar densities for RM 1,746-1,682; 1,661-1,653; and 1,638-1,631 because aerial photographs were not available. The data show that the average island density throughout the remainder of the reach decreased by 2.2 acres per mile, while the sandbar density increased by 2 acres per mile. The maximum change in island density was a loss of approximately 75 acres per mile between RM 1,603.0 and 1,599.0. In that particular segment, a cutoff occurred between the times the aerial photographs were taken that removed the islands from the main river channel. Were it not for the cutoff, the average change in island density during the study period would have been an average gain of 1.4 acres per mile. Another notable change for islands was a gain of 10.3 acres per mile of islands between RM 1,631.0 and 1,625.6. Based on the time frame analyzed, this slight change seems to indicate a relatively stable system in terms of its islands.

The maximum change in sandbar density was a gain of 13.9 acres per mile, which occurred in the downstream reach between RM 1,599.0 and 1,596.0. This is the downstream limit of the Fort Peck Reach, which could be turning into a portion of the downstream aggradation reach. Between RM 1,631.0 and 1,625.6, which is immediately downstream from a tributary, the sandbar area increased by 5.8 acres per mile. Therefore, it is highly probable that the tributary is the source of sediments causing the aggradation in this reach. With the building of sandbars near the tributary and in the lower section of the reach, the average change in sandbar density for the reach was 2.0 acres per mile. This increase is very similar to the increase in the average island densities of about 1.4 acres per mile (if the cutoff event is ignored). Thus, the Fort Peck Reach islands and sandbars appeared to be in a relatively stable condition during the period 1974-1990.

As discussed in Chapter 2, the sedimentation ranges were analyzed to obtain the area in square feet of channel bed and bank erosion at each range. From 1966 to 1978, the most recent period used for the Fort Peck Reach, 29 cross sections in the bed degraded while 14 cross sections aggraded. A plot of channel bed erosion and aggradation from 1966 to 1978 and island densities for 1974 and 1990 is shown in Plate 19. From about RM 1,682 to the downstream limits of the reach, the trend to degrade decreased. Based on these results, there is no apparent relationship to the channel bed scouring and changes in island densities. For example, in the upstream segment of the reach, there was little or no channel bed scour and the island density increased. A plot of the channel bed erosion and aggradation from 1966 to 1978 and sandbar densities for 1974 and 1990 is shown in Plate 20. Inspection of this plate indicates that sandbar densities increased in areas where the channel bed aggraded or eroded very little. This result is logical, because sandbars are the channel bed forms, sand dunes, and waves. As the channel bed aggrades, those features will develop closer to the water surface and may become exposed as sandbars.

The banks during the same time frame showed 28 cross sections scouring and 15 cross sections rebuilding. The bank scour and aggradation from 1966 to 1978 versus island densities for 1974 and 1990 are shown in Plate 21. Based on the results, there appears to be a general, although weak, relationship between bank scour and change in island densities since several ranges of the middle segment of the reach did experience erosion and the island densities in the downstream segment increased. However, in the upstream segment of the reach, island density increased and little to no bank scouring occurred. A similar trend is demonstrated in Plate 22, which is a plot of the bank scour and aggradation versus sandbar densities. Once again, sandbar densities increased in the downstream segment with several ranges in the middle segment exhibiting bank scour.

In an effort to evaluate the total overall impacts of scour and aggradation of the channel bed and banks, the channel bed and bank scour and aggradation were combined for each sediment range, then compared to the island and sandbar densities. Plate 23 is the combined aggradation and degradation versus island densities, and Plate 24 is the combined aggradation and degradation versus sandbar densities. These plates show that the upstream segment of the reach had less total scour than the middle segment, and the downstream segment tended toward deposition or less erosion. The island and sandbar densities (Plates 23 and 24) increased slightly in the upstream and downstream segments. Therefore, there appears to be very little relationship between the channel bed scour and the change in island or sandbar densities, but there is a relationship between sandbar density and channel bed aggradation with sandbar density increasing where the bed aggrades.

Plate 25 shows the relative amounts of channel bed and bank aggradation and degradation for the same cross sections throughout the reach as they relate to the total aggradation and degradation for each sediment range. No regular pattern of bed scour versus bank erosion is immediately obvious from the plot with the exception that channel bed scour occurred more frequently in the upstream segment of the reach while bank scour occurred more frequently in the middle segment. The downstream segment was a mix of scour and deposition from the channel bed or banks.

To address the impacts of discharges on the islands and sandbars in the Fort Peck Reach, reference is made to the time line presented in Chapter 2 and the hydrographs presented in Chapter 3. For the time frame 1974-1990, the time line indicated very high flow conditions in the Fort Peck Reach for 1975, 1976, 1978, 1979, 1981, and 1982 with high flow conditions for 1974, 1980, 1984, 1985, and 1989. In that time frame, only 1987 was classified as a low flow year, but 1959, 1960, 1986, and 1990 had flow conditions close to the low classification. The averaged daily hydrograph for the 1974-1990 time frame (Chapter 3) characterized that period as having greater than average flow conditions compared with the CWCP (Plate 7). As previously discussed , the analysis of Fort Peck Reach island and sandbar densities indicated that they appeared relatively stable during 1974-1990 (Table 19). Therefore, it can be concluded, that although that time period was one of relatively high flow conditions, those flows had no major negative impacts on the islands or sandbars.

Garrison Reach

The analysis for the Garrison Reach was identical to the Fort Peck Reach. The tabulation of island and sandbar densities based on the 1976 and 1990 aerial photographs is presented in Table 20. The data show that the average island density throughout the reach increased by approximately 6.3 acres per mile, while sandbar density decreased by 0.8 acre per mile. The maximum change in island density was a loss of 36 acres per mile in the reach between RM 1,371.7 and 1,364.0. This major change was the result of a large island being converted to channel border fill. Just downstream between RM 1,364.0 and 1,355.3, some sandbars were converted to islands at a rate of 34.4 acres per mile. These were the two largest changes in island acreage in the reach during the time period and, being nearly equal in magnitude with similar reach lengths, their net effect in terms of island building and/or loss was basically no change.

The maximum change in sandbar density was a loss of 44.1 acres per mile between RM 1,371.7 and 1,364.0, the same location as the greatest decrease in island density. In this segment of the river and upstream to the dam, the sandbar density decreased due to a combination of removal or conversion to islands. Between RM 1,346.8 and 1,325.2 more sandbars were being actively formed than lost. The net average change for the reach was a decrease of 0.8 acres per mile. Thus the islands and sandbars appeared to be relatively stable during the period 1976 to 1990, with the density of sandbars showing a slight tendency to decrease in the reach's upper segments.

Garrison Reach had the same bed and bank analysis as the Fort Peck Reach. In the time period 1976-1985, 27 cross sections in the bed degraded while 11 cross sections showed aggradation. A plot of channel bed erosion and aggradation from 1976 to 1985 and island densities for 1976 and 1990 is shown in Plate 26. The upstream segment includes a mix of aggradation and degradation. Bed scour was the greatest in the upstream portion of the middle segment. The middle and lower segments had some notable bed aggradation. There is no apparent relationship between the scouring of channel bed and changes in island densities. A plot of the channel bed erosion and aggradation from 1976 to 1985 and sandbar densities for 1976 and 1990 is shown in Plate 27. Inspection of this plate indicates no relationship between channel bed aggradation or degradation and sandbar densities.

The banks during the period between 1976 and 1985 showed 27 cross sections scouring and 11 cross sections recovering. The bank scour and aggradation from 1976 to 1985 versus island densities for 1976 and 1990 are shown in Plate 28. The results do show a definite trend toward widening in the upper segments of the reach. Based on these results, there appears to be a general relationship between bank scour and change in island densities since several ranges of the upstream segments of the reach experienced erosion and the middle and downstream island densities increased. The opposite trend is demonstrated in Plate 29, which is a bank scour and aggradation versus sandbar densities plot. With bank scour in the upstream segment, the sandbar density in the middle segment decreased. The major increases in sandbar densities were in the downstream segment with only intermittent ranges in the middle segment exhibiting bank scour.

Plate 30 shows the relative amounts of bed and bank aggradation and degradation as a percent of the whole for the same cross sections throughout the reach. This plate shows an obvious trend toward degradation and widening throughout the reach and, at several ranges, a predominance of channel bed degradation;

however, several other ranges are virtually all bank erosion. Based on Plate 30, it appears that many ranges exhibiting scour are either channel bed or bank scour but not necessarily a combination of both at many ranges.

In the Garrison Reach time line (Chapter 2) from 1976 to 1990, flow conditions were classified as very high in 1976, 1978, 1979, and 1982 with low flow conditions for 1988 and 1990. The 1976-1990 averaged daily hydrograph (Chapter 3) characterized that period as having greater than average flow conditions compared to the CWCP (see Plate 9). As discussed previously, the analysis of the Garrison Reach island and sandbar densities indicated that they appeared relatively stable during 1976-1990 (Table 20). Therefore, it can be concluded that, although the time period started out with relatively high flow conditions and ended with some low flow conditions, those flows had no major negative impacts on the islands or sandbars. Also, during this period of flows, one island was converted to a channel border fill, which is not an elimination of an island feature, but rather modification of the channel attributes.

Fort Randall Reach

Table 21 tabulates island and sandbar density based on the 1976 and 1994 aerial photographs in acres per mile. The data show that average island density throughout the reach decreased 18.4 acres per mile and the sandbar density decreased 40.7 acres per mile. The maximum change in island density was a loss of 109 acres per mile between RM 864.5 and 861.9. This change was the result of a large island being converted to a channel border fill. The last two segments of the reach between RM 854.8 and 843.5 also showed some loss of islands. Inspection of the aerial photographs indicated that this was a reduction in the size of the original islands and not a loss due to the islands becoming border fill. Up to this point of the analysis, this was the first area to show such a change. In general, the upstream islands appeared relatively stable during the period 1976-1994; however, the downstream islands indicated a loss of density.

The maximum change in sandbar density was a loss of 125.1 acres per mile between RM 864.5 and 861.9, the same location as the greatest change for islands. Although this river segment had the greatest decrease in sandbar density, there was an average sandbar density loss of 40.7 acres per mile in the entire Fort Randall Reach. Therefore, from 1976 to 1994, there was a definite trend over the entire Fort Randall Reach for the density of the sandbars to decrease significantly.

In the channel bed and bank scour and aggradation analysis, the time period 1975-1985 was used. In this time frame, 18 cross sections of the bed degraded while 8 cross sections aggraded. A channel bed erosion and aggradation plot from 1975 to 1985 and island densities for 1976 and 1994 are shown in Plate 31. The trend in the upstream and middle segments is toward some degradation and some notable bed aggradation in the lower segment of the reach. Because the island densities remained about the same in segments that had degradation and decreased in the other segment that had aggradation, there is no apparent relationship to the scouring of channel bed and changes in island densities. A channel bed erosion and aggradation plot from 1975 to 1985 and sandbar densities for 1976 and 1994 are shown in Plate 32. Inspection of this plate indicates that sandbar densities decreased in areas where the channel bed aggraded or degraded. Therefore, like the island density in this reach, there appears to be no relationship between channel bed aggradation and degradation.

The banks from 1975 to 1985 indicated scouring at 19 cross sections and aggradation at 7 cross sections. It should be noted that the area of banks scoured was significantly less than the area of channel bed scoured for the reach. The bank scour and aggradation from 1975 to 1985 versus island densities for 1976 and 1994 are shown in Plate 33. The results show that bank scour occurred throughout the reach with some notable aggradation near RM 870.0 and 849.0. The bank recovery between RM 874.0 and 869.7 appears to be due to a chute that filled, while the bank recovery near RM 849.0 is probably due to the inflow from the Ponca Creek tributary. Based on these results, there appears to be no definitive reason for the changes in island densities in the downstream segment. The trends presented in Plate 34, which is a plot of the bank scour and aggradation versus sandbar densities, indicate that, with bank scour or aggradation present, the sandbar densities decreased for the entire reach. The loss of sandbar and island densities in the downstream portion of the Fort Randall Reach may be attributed to the backwater effects of the Niobrara River delta. Stage-discharge rating curves in this area have shifted upward as much as 4 ft, and this stage increase may have submerged islands and sandbars that would have otherwise been exposed.

In an effort to evaluate the total overall impacts of scour and aggradation of the channel bed and banks, the channel bed and bank scour and aggradation were combined for each sediment range and compared to the island and sandbar densities. Plate 35 is the combined aggradation and degradation versus island densities, and Plate 36 is the combined aggradation and degradation versus sandbar densities. Inspection of these plates indicates that the upstream and middle segments of the reach tended to be erosional, and the downstream segment tended to be depositional. However, the depositional downstream segment lost both island and sandbar densities. Therefore, there appears to be very little relationship between the scour of the channel bed or banks and the change in island or sandbar densities.

Plate 37 shows the relative amounts of channel bed and bank aggradation and degradation for the same cross sections throughout the reach as they relate to the total for each sediment range. It is obvious from this plot that the channel bed contributed the largest portion of the scour material in the 1975 to 1985 time frame. This plot shows a strong trend toward degradation in the majority of the reach with less degradation and increased aggradation in the downstream portion.

In the Fort Randall Reach time line (Chapter 2), for the time frame 1976-1994, the flow conditions were classified as very high in 1976 and 1978 with low flow conditions for 1989, 1990, 1991, 1992, and 1993. The averaged daily hydrograph for 1976-1994 (Chapter 3) characterized that period as having less than average flow conditions compared to the CWCP (see Plate 11). As discussed previously, the analysis of the island and sandbar densities for the Fort Randall Reach (Table 21) indicated that island densities tended to decrease in most of the reach and sandbar densities for the entire reach decreased during the period 1976 to 1994. Therefore, it can be concluded that with average flow conditions less than experienced with the CWCP, there was a strong tendency for the islands and sandbars to decrease in size. It is not clear if the conversion of the one island at RM 864.5 to a channel border fill was a result of low flow conditions or the natural movement of the channel over this period.

Gavins Point Reach

Of the four study reaches, the Gavins Point Reach is unique because no downstream reservoir exists and the navigation channel impacts the downstream limits. Navigation channel development has included cutoffs, channel contraction using dikes and river training structures, and channel bank stabilization measures such as revetments. The combination of these measures in the navigation channel impacts the downstream portion of the Gavins Point Reach by introducing additional channel degradation and the potential for headcutting beyond that created by the construction of Gavins Point Dam. Therefore, the navigation channel and its effects on the Gavins Point Reach should be considered in the evaluation of change in channel attributes.

The island and sandbar densities computed for the Gavins Point Reach for the time frame for 1981 and 1994 are tabulated in acres per mile in Table 22. The island density throughout the reach decreased 2.1 acres per mile, and the sandbar density decreased by 7.6 acres per mile. Other than at the downstream end of the reach, the island density decreased from upstream to downstream with the largest decreases near the downstream end.

The maximum change in sandbar density was an increase of 33.9 acres per mile between RM 763.0 and 753.7. This segment is immediately downstream of a segment having a loss of 27.1 acres per mile between RM 769.0 and 763.0. The majority of the reach segments indicated a decrease in sandbar density. Some sandbar loss was due to being converted to islands or channel border fill, but the majority was due to sandbar erosion. There was a general tendency for sandbar erosion during the time frame.

In the Gavins Point Reach, the time period covered for the channel bed and banks analysis was from 1974 to 1986. In this time frame, 30 cross sections in the bed degraded while 10 cross sections aggraded. A channel bed erosion and aggradation plot from 1974 to 1986 and island densities for 1981 and 1994 are shown in Plate 38. The plot indicates a tendency for the channel bed degradation to increase from upstream to downstream with notable bed aggradation in one location at the center of the reach and in three locations at the downstream reach segment. Since the island densities remained about the same in the upstream and middle segments and decreased in the downstream segment, there is no apparent relationship to channel bed scouring and changes in island densities. A channel bed erosion and aggradation plot from 1974 to 1986 and sandbar densities for 1981 and 1994 are shown in Plate 39. Inspection of this plate indicates that sandbar densities increased and decreased in areas where the channel bed degraded. Therefore, like the island density in this reach, there appears to be no relationship between sandbar density and channel bed aggradation or degradation.

The banks during the time frame 1974-1986 showed 34 cross sections scouring and 6 cross sections recovering. It should be noted that the area of banks scoured was slightly greater than the area of channel bed scoured for the reach. The bank scour and aggradation from 1974 to 1986 versus island densities for 1981 and 1994 are shown in Plate 40. The results show that bank scour occurred throughout the reach and increased from upstream to downstream. As was the case with the channel bed results for this reach, although bank scour increased from upstream to downstream, the island densities remained about the same in the upstream segment, increased in the middle segment, and decreased in the downstream segment. Therefore, there is no apparent relationship between the scouring of banks and changes in island densities. The trends presented in Plate 41, which is a plot of the bank scour and aggradation versus sandbar densities, indicate that even though the bank scour increased from upstream to downstream, the majority of the sandbars tended to decrease in size throughout in the reach.

Plates 42 and 43 show a combination of the channel bed and bank scour and aggradation for each sediment range compared to the island and sandbar densities, respectively. Inspection of these plates indicates that increased total degradation from upstream to downstream showed no relationship to the changes in the islands or sandbars.

Plate 44 shows the channel bed and bank aggradation and degradation amounts for the same cross sections throughout the reach as a percentage of the total for each sediment range. It is obvious from this plot that the channel bed contributed the largest portion of the scour material in the 1974-1986 time frame. This plot shows a strong trend toward degradation in the majority of the reach with somewhat less degradation and slightly more aggradation in the downstream portion.

In the Gavins Point Reach time line (Chapter 2) for the time frame 1981-1994, the flow conditions were classified as very high in 1986 with low flow conditions for 1981, 1989, 1990, 1991, 1992, and 1993. The averaged daily hydrograph for the 1981-1993 time frame (Chapter 3) characterized that period as less than average flow conditions compared to the CWCP (see Plate 13). As discussed previously, the density analysis of islands and sandbars for the Gavins Point Reach (Table 22) indicated that overall they tended to decrease during 1976-1994. Therefore, it can be concluded that with average flow conditions less than the CWCP, there was a tendency for the islands to decrease or stay about the same size and for the sandbars to decrease slightly in size. This conclusion may be somewhat in error, however, due to navigation channel impacts on the downstream portions of the reach.

Summary of All Four Reaches

In the preceding sections, the four study reaches were considered separately. Conclusions that might be drawn from the four reaches when taken as a whole are discussed in this section. It should also be remembered that many of the changes were not a loss of material, but a conversion from one attribute to another, i.e., islands to channel border fills.

The first issue is relative to the islands and the overall response of the islands over time. Based on Tables 19-22, the absolute value percentage of the increase or decrease in the final island density relative to the initial values was computed. The percent change for the four reaches starting upstream with the Fort Peck Reach and ending downstream with Gavins Point Reach, were -13, +25, -23, and -5 percent, respectively. This indicates that over periods of approximately 15 years for each reach, the islands in all the reaches are relatively stable.

The second issue is relative to the sandbars. Here the percent change for the four reaches in the same order were +65, -2, -88, and -30 percent, respectively. The 65 percent change for the Fort Peck Reach is somewhat misleading, since the magnitude of this change is small, and the largest change was 13.9 acres per mile increase in the most downstream section of the reach. This particular reach segment is near the aggradation reach and may be in the process of becoming part of the aggradation reach. Exclusion of this segment between RM 1,599.0 and 1,596.0 would produce a change of 47 percent, which is more in line with the other reaches. The 88 percent change in the Fort Randall Reach is significant since the magnitude of the decrease in average sandbar density of about 41 acres per mile is much larger than in any other reach.

The overall channel bed and bank changes can be evaluated by reviewing Plates 25, 30, 37, and 44. Above the horizontal zero line, aggradation and narrowing are represented; and below the line, degradation and widening are represented. Without exception, every reach for the time windows represented shows from two and one-half to eleven times more degradation and widening than aggradation and narrowing. This is not surprising because the river is trying to re-establish some type of dynamic equilibrium after the closure of the dams.

Such effects of dam closure are well documented by the USGS in Williams and Wolman (1984). This document clearly shows that rivers will normally experience significant degradation of banks and bed for some distance downstream of the dam both in space and time. The distance downstream of the dams in these study reaches is already well documented by the numerous surveys conducted by Omaha District personnel. Figure 3, reprinted from the USGS, clearly shows that degradation does not occur uniformly over time. Therefore, a critical issue is determination of the positions of the study reaches on their individual degradation curve.

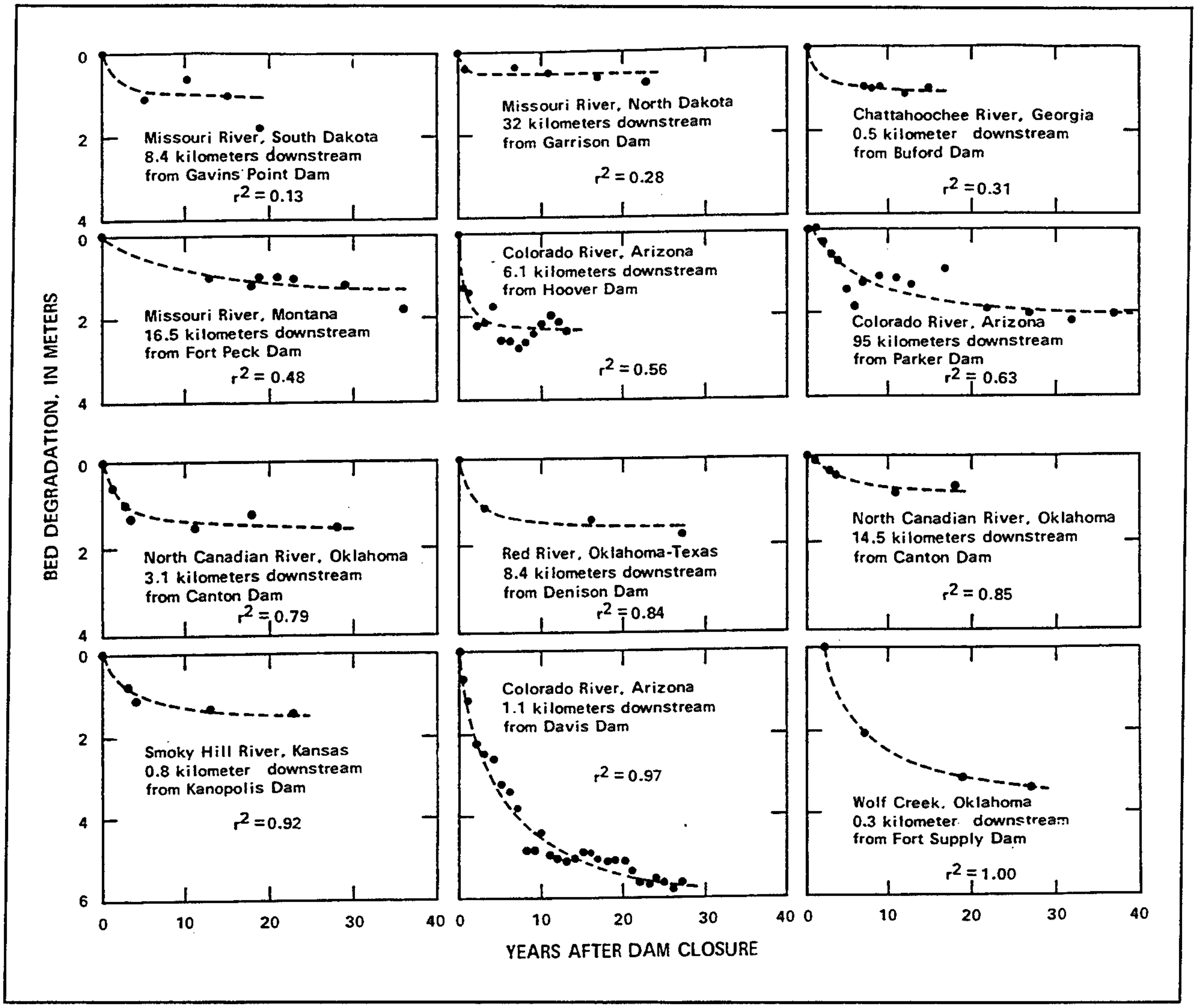

Figure 3. USGS regression curves at selected sites (from Williams and Wolman 1984)

It is evident that the closer to the time after dam closure the data are taken, the higher the rate of degradation. For the most part, the aerial photographs and cross sections used in the study reaches are at least 20 years after dam closure. So it is reasonable to suggest that major changes to the river due to adjustment to the dams occurred prior to the time the data for this analysis were obtained. This does not mean that the river has attained its equilibrium, but simply that the rates of change are probably considerably lower.

Impact of Preferred Alternative on Islands and Sandbars

An estimate of whether differing flow rates or channel morphology had the biggest influence on the changes identified in the time frames discussed in this chapter can be made with the data at hand. In all four reaches for example, the time frame with the highest and longest duration of high flows in this study occurred between 1969 and 1978. These are very nearly the same dates framed by the portion of this study dealing with channel bed and bank changes and the channel cross-section data used for the Fort Peck Reach. The amount of degradation versus aggradation in the reach was not greater than observed in the other three reaches. In fact, the observation time frames for the other three reaches were slightly later than those for the Fort Peck Reach; thus, the changes in channel features that occurred in their respective time periods were not the result of all the highest flow years. Yet these reaches showed equal or greater percentages of island and sandbar density changes compared with the Fort Peck Reach. Also, the Fort Randall Reach had the largest reduction in island and sandbar densities with a flow condition significantly lower than the CWCP. These data seem to point to the quest of the river for dynamic equilibrium, which has been a dominant factor in changing the morphological features of the river, along with periods of low or high flows. Therefore, it would appear that overall, morphological changes greater than those that have occurred in the past probably will not be forced on the system by the PA since it is shorter in duration and lower in magnitude than the high flows of the 1969-1978 period. Some significant morphological changes occurring in isolated river sections due to various controls are not precluded.

Chute and Channel Border Habitats

In the analysis of impacts on islands and sandbars, two additional channel attributes were also addressed: chute fills and channel border fills. For this study, chute fills were defined as the fills occurring in the secondary channels around islands, and channel border fills were defined as the fills adjacent to the channel banks not associated with an island. Channel border fills tend to be present in overwidened river reaches and probably start out as sandbars. Typically they evolved in such a manner that they either became attached to the riverbank or were separated from the bank by a relatively narrow channel that probably would carry flow only during higher flow events. Also, unlike sandbars, the channel border fills tended not to be transitory. As was the case in the islands and sandbars analysis, the areas for the chute and channel border fills reported in this report were obtained by digitizing the aerial photographs.

The areas presented in the following tabulation are the total for the study reaches. In the cases where chute or channel border fills became attached to the bank, it was virtually impossible to delineate that particular area and no measurement was made; however, the photographs and digitized data were inspected to determine locations where cases occurred. The results of those inspections are included in the analysis.

The time line for the periods listed in the tabulation shows that in the Fort Peck Reach, 1975, 1976, 1978, 1979, 1981, and 1982 were very high flow years; 1980, 1984, 1985, and 1989 were high flow years; and 1987 was a low flow year. One would expect, in considering sedimentation processes, that several years with

Reach	Photo Date	Discharge, cfs	Chute Fill, acres	Channel Border Fills, acres
Fort Peck	August 1974 October 1990	12,200 7,900	39.3 92.4	756.7 455.8
Garrison	October 1976 October 1990	13,400 10,300	393.8 471.0	1,144.9 1,915.1
Fort Randall	October 1976 May 1994	38,000 29,500	757.9 409.3	692.9 523.2
Gavins Point	June 1981 May 1994	32,000 30,600	86.8 13.0	2,383.7 2,258.0

above-normal flows would tend to increase shoaling in areas that tended to be depositional such as island chutes. Relative to the chute fills, as addressed previously, the island density of this reach increased from 1976 to 1990 when the cutoff segment is not included. Therefore, additional secondary channels associated with those new islands would have provided lower velocity areas and conditions favorable for deposition. Upon inspection of the photographs and digitized plots, it became apparent that, in several reach segments, the channel border fills became attached to the bank line and could no longer be visually delineated from the river bank.

The time line in the Garrison Reach shows 1978, 1979, and 1982 as very high flow years; 1980, 1984, and 1986 as medium flow years; and 1988 and 1990 as low flow years. This reach had the same trend as the Fort Peck Reach relative to chute fills and an opposite trend for channel border fills. Island density increased between 1976 and 1990, providing additional islands and secondary channels suited for chute filling. Inspection of the aerial photographs and digitized plots indicated that, in some reach segments, channel border fills were converted to river bank lines. However, in several segments, sandbars became channel border fills, thus increasing the total channel border area over time.

The time line for the Fort Randall Reach shows very high flow in 1978; medium flows for 1977, 1979, 1980, and 1981; and low flows for 1989, 1990, 1991, 1992, and 1994. Therefore, in this reach, there were long periods of average and low flows that one would expect to scour material from the various channel attributes. Island and sandbar densities decreased from 1976 to 1994 just as did the chute fills and channel border fills areas. This indicates that adjustment of the channel is still an ongoing process, which is discussed in the "Banks and Channel Bed Erosion" section in Chapter 5. It should be noted that the Fort Randall Reach differs from the other three reaches because significant portions of this reach are aligned along the bluff line and few tributaries are present. Both these conditions contribute to scouring of these habitats due to a limitation of available sediments.

The Gavins Point Reach time line shows very high flows in 1986; medium flows in 1982, 1985, 1987, and 1988; medium to low flows in 1981; and low flows for 1989, 1990, 1991, 1992, and 1993. Even though the flow characteristics are similar to those in the Fort Randall Reach, the Gavins Point Reach showed little change in island density between 1981 and 1994 or in the area of channel border fills. There was a significant decrease in chute fills; however, initially there were very few examples of such attributes in the reach. The "Banks and Channel Bed Erosion" section presented later in this report discusses the process of channel adjustment taking place in this reach. That adjustment is influencing the reaction of the chute and channel border fills and makes correlation to discharges virtually impossible at this time.

Relationship of Sandbar and Island Exposure and Discharge

The amount of sandbar exposed for various flow conditions was a particular concern. The analysis presented in the previous section addressed the density of sandbars based on acres of sandbar per mile for various study reach segments. Originally MRR requested that this study determine if some relationship between sandbar exposure and discharge exists. Initially, analysis to determine if such a relationship exists seemed reasonable. However, as this analysis proceeded, it became very apparent that no relationship between sandbar exposure and discharge could be developed.

It must be realized that sandbars are channel attributes deposited by the river during various flow events. The composition of the material in the sandbars is the sediment load moved along the bed. The origin of those materials is relatively unimportant. Typically the sandbars are bed forms (dunes) that are growing and moving with the riverflow. Thus, the sandbars are channel bed features that develop, sometimes enlarge, and sometimes degrade (depending on the particular flow conditions, location of areas conducive to deposition, and other factors, such as wind and waves). As the river discharge increases and the bed sediment load starts movement, that movement will be maintained until a segment of the river is reached that is conducive to deposition. That may be an overwidened river segment or one where part of the channel turns and the main thread of flow moves in one direction leaving one side of the channel with lower velocities. Based on this study's aerial photograph analysis, it was apparent that for the four reaches, some segments tended to have more sandbars than other segments.

As the discharge and stage increase, provided a sufficient sediment supply is available, the sandbar can build to a higher elevation. This process will more than likely continue as long as increases in the discharge and stage continue. Once the flow crest is reached and stages and discharges start to decrease, the potential for sandbar exposure is reached. To that point, the sandbar has been completely submerged. If the stage decreases relatively rapidly, the sandbar may become exposed and continue in that condition. There may or may not be scour along the sides and upstream face of the sandbar and, following that rapid fall of the stage, the sandbar may continue to be exposed or may be scoured away. If all or a portion of the sandbar remains, debris may collect in the general area or vegetation may establish itself on the sandbar. In either or both cases, the sandbar may become more resistant to future erosion when similar flow conditions return.

However, if the stage decreases slowly, the sandbar top and side surfaces may scour with the receding water-surface elevation, and the top elevation will merely "track" with the water surface. In this case, the sandbar may remain submerged either partially or totally at lower stages. The same situation may exist in an area that is only somewhat conducive to establishment of sandbars where the flow direction or local magnitude may vary during the various stages.

The purpose of this discussion is to show the dynamic nature of sandbars. The height or area of an exposed sandbar has little to do with the discharge or stage at the time of observation. In fact, flows that occurred in previous months may have created the sandbar. Once it is created, then the size, shape, and amount of exposure are subject to modifications by riverflows, wind, waves, ice, and other factors. In an effort to address this dynamic nature, part of the study was directed toward an analysis of the islands in the four reaches. While sandbars and islands may not be directly related, an attempt to quantify the response of islands to various flow conditions may be helpful to various resource agencies.

The results of the island exposure analysis are presented in Tables 23-26 for the Fort Peck, Garrison, Fort Randall, and Gavins Point Reaches, respectively. A total of 72 islands were analyzed to determine what happened to them over time. It should be noted that any island that existed in the oldest set of aerial photographs for a particular reach continued to exist in the newest set of photographs. This observation is important, because it shows that for all the changes that have occurred relative to channel bed degradation, bank scour, wide variation in flows, and activities by man, those islands have been able to survive. The changes in island densities were addressed earlier in this report. This section will address the changes over time in specific islands within the four reaches.

Fort Peck Reach

Thirty-nine islands were documented in the Fort Peck Reach (Table 23). Of those 39 islands, 18 existed in 1947 and 7 did not, based on an aerial mosaic. Due to the mosaic photograph quality and possibly the size or lack of existence of the other 14 islands, they could not be interpreted as existing in 1947. Of the seven islands known not to exist in 1947, six of them were documented on the 1990 aerial photographs. The seventh island (RM 1,674.0) was developed by 1956, enlarged to 15.3 acres in 1976, and was gone by 1990. One island (RM 1,597.0) was not obvious until the 1990 photograph. Returning to the time line discussion, 1974 was a high water year while 1975 and 1976 were very high water years. Between the 1974 photograph (12,200-cfs discharge) and the 1976 photograph (22,100-cfs discharge), 25 of the 39 islands indicated smaller areas, 11 islands indicated larger areas, and 2 islands indicated little or no change. The island at RM 1,597.0 was not included in this time frame. The time line from 1976 to 1990 indicated that most of these years were high or very high flow years with only 1987 characterized as a low flow year. Between 1976 and the 1990 photograph taken at a 7,900-cfs discharge, 19 islands increased in size, 12 islands got smaller, 3 changed very little or not at all, 4 became attached to the adjacent river bank, and one was scoured away. Because the reaction of the Fort Peck Reach islands to the flow conditions varied dramatically in magnitude and short- or long-term trends, the results indicate that enlargement or reduction of islands cannot be directly attributed solely to discharge. It can be stated, however, that any islands that existed in 1947 continued to exist in 1990.

Garrison Reach

In the Garrison Reach, 18 islands were documented (Table 24). Of those 18 islands, 7 were islands, 10 were sandbars, and one did not exist in 1956, based on aerial mosaics. The one island that did not exist in 1956 was present on the 1990 aerial photograph. It should be noted that the island at RM 1,344.0 was not present on the 1976 photograph. Based on the time line, 1976, 1978, and 1979 were very high water years. Between the 1976 photograph (13,400-cfs discharge) and the 1981 photograph (24,100-cfs discharge), nine islands indicated smaller areas and nine islands indicated larger areas including the island developed at RM 1,344.0. The time line between 1981 and 1990 indicated that the flow conditions during that period were relatively normal. Between 1981 and the 1990 photograph (10,300-cfs discharge), 15 islands increased in size, 2 islands got smaller, and one became attached to the adjacent riverbank. Overall from 1976 to 1990, 13 islands increased in size, one changed very little, and 4 got smaller. In general, the reaction of the Garrison Reach islands to the flow conditions has been that higher flow conditions tended to decrease island size and more normal flow conditions tended to increase island size. Over the period from 1976 to 1990 many more islands increased in size than decreased, and from 1956 to 1990, any islands that existed in 1956 as an island or a sandbar continued to exist in 1990.

Fort Randall Reach

In the Fort Randall Reach eight islands were documented (Table 25). No mosaic was available for this reach. All eight islands existed on the 1975 photograph. Based on the time line, 1975 and 1976 were very high water years. Between the 1975 photograph (60,000-cfs discharge) and the 1976 photograph (38,000-cfs discharge), seven islands enlarged and one island changed very little. The time line between 1976 and 1994 indicated that the flow conditions during that period were relatively normal except for 1989, 1990, 1991, 1992, and 1994, which were low flow years. Between 1976 and the 1994 photograph (29,500-cfs discharge), one island increased in size, four islands got smaller, two islands got slightly smaller, and one island changed very little. In general, the reaction of the Fort Randall Reach islands to the flow conditions has been that higher flow conditions tended to increase island size and more normal to low flow conditions tended to decrease island size. From 1975 to 1994, six islands increased in size while only one decreased. As in the previous two reaches, in the Fort Randall Reach any islands that existed in 1975 continued to exist in 1994.

Gavins Point Reach

Seven islands were documented in the Gavins Point Reach (Table 26). Of those seven islands, four were islands, two were sandbars, and one was not present in 1956 based on an aerial mosaic. The two sandbars and the island not present in 1956 were present as islands in 1972. For the Gavins Point Reach, a 1977 set of aerial photographs was available for this analysis. However, ice was present on the river that made delineation of the island limits somewhat difficult. It should be noted that the island at RM 804.0 was submerged in the 1972 photograph and no area was determined there. On the Gavins Point Reach time line, 1972-1977, the years 1972, 1975, and 1976 were very high water years. Between the 1972 photograph (46,500-cfs discharge) and the 1977 photograph (15,000-cfs discharge), all but one island increased in size with the island at RM 760.0 showing little change. The time line from 1977 to 1981 indicated very high flow years in 1978-1979 and a low flow year in 1981. Between 1977 and the 1981 photograph (32,000-cfs discharge), all but one island decreased in size with the island at RM 760.0 showing little change. The time line from 1981 to 1994 indicated a very high flow year in 1986 and low flow years in 1989, 1990, 1991, 1992, and 1993. Between 1981 and the 1994 photograph (30,600-cfs discharge), five islands decreased in size, one island increased, and the island at RM 755.0 showed little change. In general, the reaction of the Gavins Point Reach islands to the flow conditions has been that higher flow conditions tended to increase island size and low flow conditions tended to decrease island size. From 1972 to 1994 three islands increased in size, three decreased, and one changed very little. As in the previous reaches, any islands that existed in 1956 as an island or a sandbar continued to exist in 1994.

5 Impacts on Banks and the Channel Bed

Channel Attributes

To determine the impacts of the PA on various channel attributes, it is necessary to first determine the historical impacts on those attributes. In this section, the amounts of bank and channel bed erosion that can be documented were analyzed and associated with the flow conditions over the periods used for this analysis.

Bank and Bed Scour

As documented in USAEDNMRR (1994a), and supported in this study, the construction of the Missouri River dams has caused a tendency for the channel invert (bed) to degrade, a trend that was anticipated in the overall project. Installing the dams trapped the Missouri River sediments previously supplied to the study reaches in the reservoirs. The capability of a stream to carry sediment Q_s is directly related to the stream discharge Q and the stream slope S, and inversely related to the sediment size D_{50}. In river engineering, the relationship of these various parameters is referred to as Lane's relationship. That relationship is presented in the following form:

$$Q_s \propto \frac{QS}{D_{50}} \qquad (6)$$

Based on Lane's relationship and the need to maintain the proportion, if the flow discharge Q or the slope S is reduced and the other parameters are unchanged, then the sediment discharge Q_s will be reduced. If the sediment size D_{50} is increased, the sediment discharge will also be reduced. The Missouri River dams provide flood control benefits, which means that the peak discharge from the dam is generally less than what the natural river carried. Although the peak discharge is reduced, the flow still has the capacity to carry a certain volume of sediment, but a volume lower than previously carried by the river. Therefore, immediately downstream of the dams the sediment discharge Q_s and peak river discharge Q are reduced. The streamflow still has the capability to carry some sediment; and the channel bed scours, which reduces the channel slope from upstream to downstream. Also, the reduced discharge does not have the capacity to carry all the sediment previously transported. The larger sizes of bed material are no longer transported or are transported at a lower rate, resulting in an increase in the sediment size D_{50} and a trend to armor the channel bed. Armoring is the process by which the finer materials in the bed are scoured, leaving behind the coarser fractions that the river is incapable of moving with those particular flow conditions.

Low flows with the dams in place have a slightly different result. The minimum discharge in postdam conditions was increased, which would increase the sediment discharge Q_s over predam minimum flow conditions. Therefore, over the entire range of discharges, postdam conditions will tend to reduce sediment discharge data due to reduced channel slopes, armored channel bed, and retained riverbed material in the upstream reservoir. Increasing the minimum flow conditions would offset some of the overall sediment discharge, except that sediment discharge uses a power function of the water discharge or velocity. Therefore, the higher water discharges have a greater impact on the sediment discharge. In summary, the reaches downstream of the Missouri River dams have adjusted significantly to the dams and, as discussed previously, the present degree of that adjustment is decreasing (Chapter 4).

Tributary discharges and sediment supplies add to the complexity of the process, as does riverbed armoring. This armor layer can be moved during periods when higher discharges capable of moving large bed material are released from the dams. So, over time, the channel bed profile will adjust to the new conditions imposed on it, and the channel invert will lower downstream of the dams. At the same time, bank and bed materials will accumulate downstream in the headwater of the next reservoir. This combined process results in the flattening of the channel slope. Because the channel degradation immediately downstream of the dams or the aggradation in the downstream reservoir headwaters (that aggradation will move farther downstream into the reservoir) cannot continue indefinitely, an adjusted slope will be reached that is in concert with the discharges and other conditions imposed on the stream including base level changes. From that point in time, there will be local or slight overall adjustments in the slope during possible long periods of lower or higher than normal discharges, but overall the channel slope should remain relatively "stable."

During the period when the channel slope is adjusting, bank erosion will increase due to lowering of the channel bed and water-surface elevations downstream of the dams. This lowering often creates a situation where, even with their inherent geotechnical properties, the banks are at an elevation higher than they are capable of maintaining. Therefore, the banks also react to the immediate changes in discharge but, generally speaking, at a slower rate than the bed. Once the channel invert approaches its condition of ultimate adjustment, the material scoured from the banks will become interactive with sediments supplied by tributaries.

Those materials will, at times, be stored temporarily within the channel as sandbars and, either temporarily or permanently depending on local conditions, as chute or border fills or fills along islands. They will also be deposited on point bars as border fills, depending on local conditions, that will eventually become new channel bank lines. It should be noted that this entire process is dynamic in the context of time and space. In some years, depending on the river hydrology, some segments of the study reaches may have bank scour and the subsequent segments downstream may be building sandbars and channel border fills. Following this train of events, those sandbars may later move to accumulate around an island as a chute fill, and the channel border fill may establish natural vegetation conducive to additional aggradation, become attached to the existing channel bank line, and become the new channel bank.

Banks and Channel Bed Erosion

Fort Peck Reach

On the Fort Peck Reach, the water-surface area was computed using the channel width measured on the channel cross sections multiplied times the distance between cross sections. This was done for the 1955, 1966, and 1978 cross sections. Previous work performed by Darrel Dangberg and Associates and River Pros for the Omaha District[1] addressed five Fort Peck Reach segments as follows: RM 1,739.3-1,724.7; RM 1,721.1-1,696.3; RM 1,688.7-1,678.4; RM 1,651.1-1,630.0; and RM 1,605.3-1,596.9. As stated previously, the total reach for this cumulative erosion impacts study covers RM 1,771-1,582.

The computations for the WES study from the 1955, 1966, and 1978 cross sections for the five segments resulted in an increase in "wetted" surface area between 1955 and 1966 of 758 acres and an increase between 1966 and 1978 of 393 acres for a total between 1955 and 1978 of 1,151 acres. From the Darrel Dangberg and Associates and River Pros report[1], the erosion between 1955 and 1966 was 747 acres and between 1966 and 1978 was 734 acres for a total between 1955 and 1978 of 1,481 acres. The report does not explain how these streambank erosion amounts were determined, but it is assumed they were obtained through comparison of aerial photographs. Therefore, the streambank erosion amounts would include the total riverine planform and subsequent total bank line erosion. The comparison indicated that the computations made in this existing study, using only channel cross sections, accounted for about 78 percent (1,151/1,481) of the total channel included in the overall channel planform, or the computations based on the cross sections needed to be increased by a factor of 1.29 (1,481/1,151) to account for the total area.

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[1] Darrel Dangberg and Associates and River Pros. (1988 (April)). "Missouri River Fort Peck Dam - downstream degradation/aggradation and sediment trends study," prepared for U.S. Army Engineer District, Omaha, Omaha, NE.

It should be noted that this method for applying a planform correction to the computed volumes was used on all of the study reaches for both the channel banks and bed. No exact data were available for the planform changes and river meandering; however, aerial photographs and mosaics indicated that the four study reaches had typical meandering planforms with lateral and down valley channel movement. Therefore, it was decided that the planform correction factor determined based on the banks should also be applied to the channel bed volumes.

Using these cross-sectional areas for the bed and banks and the distance between adjacent cross sections, the volume of material scoured from the bed and banks was computed, then multiplied times the 1.29 factor to account for planform changes not present in the cross sections. The volumes shown in the following tabulation are for the five segments analyzed in the Darrel Dangberg and Associates and River Pros report.[1] The total volume scoured from the banks of the Fort Peck Reach is addressed later in this section.

	Volume, acre-ft	
Time Span	**Bank Scour**	**Bed Scour**
1955-1966	9,924	6,818 (fill)
1966-1978	6,671	2,718
1955-1978	16,595	4,100 (fill)

These computations indicate that, since about 1955, the volume of material scoured from the banks has been significantly greater than that scoured from the bed and that the erosion of the banks has decreased over time. In the time frame of this study, the channel bed filled (aggraded) during the first 11 years and then scoured the next 12 years.

Total volume of material scoured from the banks since the project went into operation was determined by calculating the average bank height. The volume of bank scour from 1955 to 1978 (16,595 acre-ft) was divided by the total erosion area over the same time from the Darrel Dangberg and Associates and River Pros report (1,481 acres),[1] which gives an average bank height of approximately 11.2 ft. Based on the Darrel Dangberg and Associates and River Pros report, the total accumulated bank erosion from 1933 to 1983 for the five segments was 3,184 acres. Multiplying that accumulated area times the average bank height gives a volume of 35,661 acre-ft scoured from the banks. Note that this is only for the five segments covering 79.3 miles of the 189 miles of the Fort Peck Reach.

The volume of material removed from the banks over time needs to be estimated to some reasonable degree. In the Darrel Dangberg and Associates and River Pros report,[1] another set of computations was made. They computed the

1 Op. cit.

total bank erosion area over the entire Fort Peck Reach from 1975 to 1983; however, no computations were made to include erosion for the entire reach from 1933 to 1975. The relationship between the five segments and the entire reach for the 1975-1983 time frame was used to determine a ratio for use with the five-segment reach for the 1933-1983 time frame. In the report, the five segments from 1975 to 1983 had an accumulated erosion area of 322.3 acres while the entire reach from 1975 to 1983 had an accumulated erosion area of 680.9 acres. Using the ratio of these two values and equating it to the ratio of the five-segment accumulated area from 1933 to 1983 (3,184 acres) over the unknown entire reach accumulated area from 1933 to 1983 produced an area of 6,727 acres. Multiplying that value times 11.2 ft gave an accumulated volume for the entire reach from 1933 to 1983 of 75,342 acre-ft. The method used in determining this accumulated volume is not ideal. It assumes that the erosion rate of the five segments is related to the erosion rate of the entire Fort Peck Reach between 1975 and 1983 and that the erosion rate of the five segments between 1975 and 1983 is related to the erosion rate of the five segments between 1933 and 1983. While this method may include some errors, it appeared to be the only method available to use as a basis for the WES study of the Fort Peck Reach.

It should be noted that during the WES study for the entire reach, a total volume of material scoured from the banks was computed using the end-area method described earlier in the report. Using the planform correction factor of 1.29, those computations resulted in a volume of bank scour between 1955 and 1978 of 28,464 acre-ft. Comparison of this computed scour volume with the 75,342 acre-ft volume for the entire reach from 1933 to 1983 based on the Darrel Dangberg and Associates and River Pros report[1] is very reasonable. That comparison also supports the conclusion that bank scour in the Fort Peck Reach has decreased over time.

The total volumes computed for the bank and bed scour in the entire Fort Peck Reach using the planform factor of 1.29 are listed in the following tabulation:

	Volume, acre-ft	
Time Span	**Bank Scour**	**Bed Scour**
1955-1966	18,025	7,990 (fill)
1966-1978	10,439	5,597
1955-1978	28,464	2,393 (fill)

These computations support the conclusions for the five segments presented previously for the entire reach with bank erosion decreasing over time and the channel bed filling, then scouring, indicating some degree of dynamic equilibrium.

[1] Op. cit.

The 1994-1995 Annual Operating Plan for the Missouri River Main Stem Reservoirs (USAEDNMRR 1994b) states that the total storage loss in Garrison Reservoir, which is downstream from Fort Peck Reach, has been 907,000 acre-ft, as of the 1988 survey. The computations based on the Darrel Dangberg and Associates and River Pros report[1] (75,342 acre-ft) indicate that approximately 8 percent of the material in Lake Sakakawea behind Garrison Dam came from Fort Peck Reach banks. Therefore, the remainder of the storage loss in the Garrison Reservoir came from material delivered to the reservoir from the channel bed, tributaries, and other sources.

The Fort Peck Reach time line presented previously showed that 1956, 1957, 1958, 1962, 1963, and 1964 were low flow years with over one-half of the year (200 days) having a discharge less than 7,000 cfs (the minimum average monthly 1898-1993 PA discharge). The time line also indicated that 1966, 1969, 1970, 1971, 1972, 1975, 1976, and 1978 were very high flow years with discharges greater than 14,000 cfs (the maximum average monthly 1898-1993 PA discharge) for 50 days or more. This indicates that, for the Fort Peck Reach, significant bank erosion and channel bed aggradation took place during a low flow period and that bank erosion was reduced and channel bed scour took place during high flow conditions.

Time has had an influence on the magnitudes of bank and bed scour or fill, which is evident from the preceding discussion. Besides time, there is a spacial impact on bed and bank scour starting at the various Missouri River Main Stem dams and moving downstream. Table 15 was the basis to consider the accumulated erosion or deposition of the bed and banks over each study reach. Plate 45 shows the cumulative changes from upstream to downstream in bank and bed volumes for 1955-1978 for the Fort Peck Reach. This is a plot of each sedimentation range volume change presented in Table 15 and was accumulated (numerically added considering the sign of the change) from the most upstream range to farthest downstream range with the planform factor of 1.29 applied to the data. Plate 45 indicates that bank erosion is greatest from about RM 1,730 to 1,670 and RM 1,620 to the downstream range near RM 1,600 and very slight from RM 1,770 to 1,740 and RM 1,670 to about 1,620. Bed erosion was the greatest between RM 1,760 and 1,740 with deposition starting near RM 1,710 to about 1,660. These data indicate that the channel bed is approaching some degree of stability or equilibrium as well as the upstream 30 miles of the banks. The cumulative bed erosion between 1966 and 1978 is also presented in Plate 45. Comparison of the bed erosion over the 1955-1978 time frame to the 1966-1978 time frame indicates that the portion of the reach from RM 1,770 to about RM 1,690 has remained relatively stable since 1955. From RM 1,690 to the downstream limits near RM 1,600 the channel bed appears to be oscillating toward a reasonable level of equilibrium. A review of Table 15 shows that between sedimentation ranges at RM 1,728.1 and 1,669.5, the bank erosion was slightly less from 1966 to 1978 than it had been between 1955 and 1966. The same trend was true from

[1] Op. cit.

sedimentation ranges at RM 1,620.9 to 1,599.0. Therefore, even though these segments of the Fort Peck Reach have the greatest erosion rates, the bank erosion has decreased over the most recent documented period. Linking that decrease with the trend toward channel bed stability implies that the overall bank erosion rates in the Fort Peck Reach are also working toward stability.

Garrison Reach

On the Garrison Reach, the water-surface area was computed using the channel width measured on the channel cross sections, multiplied times the distance between cross sections. This was done for the 1956, 1976, and 1985 cross sections. Those computations resulted in an increase between 1956 and 1976 of 1,308 acres and an increase between 1976 and 1985 of 199 acres for a total between 1956 and 1985 of 1,507 acres. On Plate 58 of USAEDNMRR (1994a), the accumulated erosion between 1956 and 1976 was 1,481 acres and between 1976 and 1985 was 383 acres for a total between 1956 and 1985 of 1,864 acres. These streambank erosion rates were determined by comparing aerial photographs and, therefore, would include the total riverine planform and subsequent total bank line erosion. The comparison indicated that the computations made in this study using only channel cross sections accounted for about 81 percent (1,507/1,864) of the total channel included in the overall channel planform, or the computations based on the cross sections needed to be increased by a factor of 1.24 (1,864/1,507) to account for the total area.

Using the cross-sectional areas for the bed and banks described in the preceding paragraph and the distance between adjacent cross sections, the volume of material scoured from the bed and banks was computed, then multiplied times this factor of 1.24 to account for planform changes not present in the cross sections. The volumes computed for the Garrison Reach are listed in the following tabulation:

	Volume, acre-ft	
Time Span	**Bank Scour**	**Bed Scour**
1956-1976	28,438	36,507
1976-1985	6,951	5,472
1956-1985	35,389	41,979

These computations indicate that, since the project started power generation in about 1956, the volume of material scoured from the bed has been slightly greater than that from the banks and that the rate of banks and bed erosion has decreased significantly over time.

Calculation of the average bank height can be used to determine the total volume of material scoured from the banks since the project went into operation.

The bank scour volume from 1956 to 1985 (35,389 acre-ft) was divided by the total erosion area over the same time from Plate 58 of USAEDNMRR (1994a) (1,864 acres), which gives an average bank height of approximately 19.0 ft. Based on Plate 58, the total bank erosion from 1955 to 1990 for the Garrison Reach was 1,966 acres. Multiplying that accumulated area times the average bank height gives a volume of 37,354 acre-ft scoured from the banks. That value agrees exceptionally well with the value computed using the end-area method for 1956-1985.

The 1994-1995 Annual Operating Plan for the Missouri River Main Stem Reservoirs (USAEDNMRR 1994b) states that the total storage loss in the Oahe Reservoir, which is downstream from Garrison Reach, has been 614,000 acre-ft, as of the 1989 survey. The volumes computed previously using the 1955-1990 period for the banks and the 1956-1985 period for the bed indicate that approximately 6 percent of the material in the Oahe Reservoir came from the banks and another 7 percent came from the channel bed. Therefore, the remainder of the storage loss in the Oahe Reservoir came from material delivered to the reservoir from tributaries and other sources.

The Garrison Reach time line presented previously shows that from 1956 to 1976, there were about 7 years of low flow conditions and 7 years of very high flow conditions. The time line also indicated that from 1976 to 1985 there were no low flow conditions and 4 years of very high flow conditions. This indicates that for the Garrison Reach, significant bank erosion and channel bed scour took place during the period with a wide variation in flow conditions, and significantly less bank and channel bed erosion took place during the period when conditions tended toward higher flows. Significant bank stabilization work took place in the Garrison Reach from 1976 to 1981, which could have also contributed to the reduction in bank scour between 1976 and 1985. Bank protection is addressed in Chapter 8 of this report.

Cumulative bed and bank erosion with the planform factor of 1.24 applied for the Garrison Reach is shown in Plate 46. That plate indicates that, between 1956 and 1985, the bank erosion along the reach has been relatively constant. Bed erosion was consistent from the upstream reach limits to about RM 1,352 where bed erosion reduced significantly. A review of Table 16 indicates that bank erosion between sedimentation ranges 1,375.7 and 1,356.2 reduced significantly as in the 1976-1985 period compared to 1956-1976 and virtually all bed erosion in this reach segment (Plate 46) occurred from 1956 to 1976. Normally one would expect the greatest rate of bank erosion to occur in the area where the greatest bed erosion is present. However, there is no indication of such a trend in the Garrison Reach. This implies that some parameter other than bed erosion influences bank erosion to a greater degree. As stated previously, that parameter is likely the particular flow conditions and influences from bank stabilization.

Fort Randall Reach

On the Fort Randall Reach, the water-surface area was computed using the same method described previously for the 1954, 1975, and 1985 cross sections. Those computations resulted in an increase between 1954 and 1975 of 743 acres and an increase between 1975 and 1985 of 439 acres for a total between 1954 and 1985 of 1,182 acres. In Plate 99 of the DEIS (USAEDNMRR 1994a) the total erosion between 1955 and 1975 was 1,087 acres and between 1975 and 1985 was 194 acres for a total between 1955 and 1985 of 1,281 acres. It should be noted that 1984 was the last year on this plate, but for comparison to the cross sections, a straight-line extrapolation was made to extend the data to 1985. Again, these streambank erosion rates were determined by comparing aerial photographs. The comparison indicated that the computations made in this study using only channel cross sections accounted for about 93 percent (1,182/1,281) of the total channel included in the overall channel planform, or the computations based on the cross sections needed to be increased by a factor of 1.08 (1,281/1,182) to account for the total area.

Using the cross-sectional areas for the bed and banks described previously and the distance between adjacent cross sections, the volume of material scoured from the bed and banks was computed, then multiplied times this factor of 1.08 to account for planform changes not present in the cross sections. The volumes computed for the Fort Randall Reach are listed in the following tabulation:

	Volume, acre-ft	
Time Span	Bank Scour	Bed Scour
1954-1975	13,475	8,121
1975-1985	3,097	7,757
1954-1985	16,572	15,878

These computations indicate that since the project started power generation in about 1954, the volume of material scoured from the bed and banks has been about equal and the rate of the bank erosion has decreased significantly over time. The rate of bed scour increased in the shorter, second period with the total volume about equal to the longer, first period.

The average bank height was calculated to determine the total volume of material scoured from the banks of the Fort Randall Reach since the project went into operation. This was done by dividing the volume of bank scour from 1954 to 1985 (16,572 acre-ft) by the cumulative erosion area over the same time from Plate 99 of the DEIS (USAEDNMRR 1994a) (1,281 acres), which gives an average bank height of approximately 12.9 ft. Based on Plate 99 of the DEIS (USAEDNMRR 1994a) the total accumulated bank erosion from 1955 to 1985 for the Fort Randall Reach was 1,272 acres. Multiplying that accumulated area times the average bank height gives a volume of 16,409 acre-ft scoured from the banks. That volume agrees exceptionally well with the volume computed using the end-area method for the period 1954-1985.

The 1994-1995 Annual Operating Plan for the Missouri River Main Stem Reservoirs states that the total storage loss in Gavins Point Reservoir, which is downstream from Fort Randall Reach, has been 83,000 acre-ft, as of the 1985 survey (USAEDNMRR 1994b). The volume computations using the end-area values indicate that approximately 19 percent of that material came from the banks and another 18 percent from the channel bed. Therefore, the remainder of the storage loss, 63 percent, in the Gavins Point Reservoir, came from material delivered to the reservoir from tributaries and other sources.

The Fort Randall Reach time line presented previously shows that from 1954 to 1975 there were about 11 low flow years at the beginning of that period and 5 very high flow years at the end of the period. The time line also indicates that from 1975 to 1985 there were no low flow conditions and 3 years of very high flow conditions. This indicates that for the Fort Randall Reach, significant bank erosion and channel bed scour took place during the period with a wide variation in flow conditions. Significantly less bank scour and slightly less channel bed erosion took place during the period when conditions tended toward medium and higher flows. However, the rate of bed scour was greater for the second period than the first period with a wide variation in flow conditions. It should be noted that all bank protection in the Fort Randall Reach was constructed between 1978 and 1982. That protection accounts for 9 percent of the reach bank lines. Installing bank protection may have influenced the bank scour reduction between 1975 and 1985, but it is unclear as to the overall significance on the major reduction in the volume of material scoured. See Chapter 8 for additional discussions relative to bank protection.

Cumulative bed and bank erosion in the Fort Randall Reach from 1954 to 1985 is presented in Plate 47. The values plotted are from Table 17 for the Fort Randall Reach with the planform factor of 1.08 applied to those volumes. Bank erosion is relatively constant from the upstream limits to about RM 865 and then reduces from that point to the downstream limits of the reach. Bank erosion is essentially zero between RM 875 and 870 and between RM 860 and 856, but it should be noted that the river is against the bluff line in those locations, which eliminates the potential for bank erosion (Table 3). Bed reaction varies significantly in the Fort Randall Reach. Bed erosion is greatest from the upstream limits to about RM 862, relatively constant from about RM 862 to 851, and then becomes aggradational approaching Ponca Creek and the Niobrara River (at RM 843.55) at this reach's downstream limits. Therefore, even though significant channel bed erosion occurred in the upstream segment of the reach and deposition in the downstream portion of the reach, the bank erosion rate remained relatively constant over the entire reach.

Gavins Point Reach

On the Gavins Point Reach, the water-surface area was computed for the 1960, 1974, and 1986 cross sections. Those computations resulted in an increase between 1960 and 1974 of 2,070 acres and an increase between 1974 and 1986 of 754 acres for a total between 1960 and 1986 of 2,824 acres. In Plate 117 of the DEIS (USAEDNMRR 1994a), the total erosion was 2,688 acres between 1960 and 1974 and 1,228 acres between 1974 and 1986, for a total of 3,916 acres between 1960 and 1986. It should be noted that 1985 was the last year included in this plate; but for comparison to the cross sections, a straight-line extrapolation was made to extend the data to 1986. Again, these streambank erosion rates were determined through comparing aerial photographs. The comparison indicated that the computations made in this study using only channel cross sections accounted for about 72 percent (2,824/3,916) of the total channel included in the overall channel planform, or the computations based on the cross sections needed to be increased by a factor of 1.39 (3,916/2,824) to account for the total area.

Using these cross-sectional areas for the bed and banks and the distance between adjacent cross sections, the volume of material scoured from the bed and banks was computed, then multiplied times this factor of 1.39 to account for planform changes not in the cross sections. The volumes computed for the Gavins Point Reach are listed in the following tabulation:

	Volume, acre-ft	
Time Span	**Bank Scour**	**Bed Scour**
1960-1974	54,867	30,654
1974-1986	37,932	21,995
1960-1986	92,799	52,649

These computations indicate that since the project started power generation in about 1956, the volume of material scoured from the banks has been greater than that scoured from the channel bed and that bank erosion has decreased over time.

The average bank height was calculated to determine a total volume of material scoured from the banks of the Gavins Point Reach since the project went into operation. The volume of bank scour from 1960 to 1986 (92,799 acre-ft) was divided by the cumulative erosion area over the same time from Plate 117 of the DEIS (USAEDNMRR 1994a) (3,916 acres), which gives an average bank height of approximately 23.7 ft. Based on Plate 117, the total bank erosion from 1956 to 1986 for the Gavins Point Reach was 4,319 acres. Multiplying that area times the average bank height gives a volume of 102,360 acre-ft scoured from the banks. That volume agrees very well with the volume computed using the end-area method for 1960-1986.

The Gavins Point Reach time line presented previously shows that from 1960 to 1974, there were about six low flow years at the beginning of that period and four very high flow years at the end of the period. The time line also indicates that from 1974 to 1986, there was 1 year with low flow conditions and 4 years of very high flow conditions. This indicates that for the Gavins Point Reach, significant bank erosion and channel bed scour took place during the period with a wide variation in flow conditions, and slightly less bank scour and channel bed erosion took place during the period when conditions tended toward medium and higher flows. It should also be noted that this reach does not have any hydropower peaking flows, but of the four study reaches, it has the highest rates of bank and bed erosion. Bank protection covering approximately 22 percent of the reach bank lines was installed between 1978 and 1982. Installation of the bank protection may have also contributed to the reduction in bank scour between 1974 and 1986. See Chapter 8 for additional discussions on bank protection.

Plate 48 presents the cumulative bed and bank erosion for the Gavins Point Reach for the time period between 1960 and 1986. The cumulative volumes plotted are from the Gavins Point Reach data presented in Table 18 with the planform factor of 1.39 applied. Both bed and bank erosion are relatively consistent over the entire reach with only minor or local variations. It should be noted that, like the Fort Peck Reach, bank erosion was significantly greater than bed erosion. That tendency is opposite to the Garrison and Fort Randall Reaches where bed erosion was greater than bank erosion. Improvements to the navigation channel downstream of the Gavins Point Reach including realignments, contraction, and stabilization of the banks have impacted the reach and probably account for a great degree of the consistency of bed and bank erosion rates. This is also supported by the bank and bed scour volumes presented in the preceding paragraph. From 1974 to 1986 bank and bed erosion decreased slightly compared with the period from 1960 to 1974; however, it appears that those changes in erosion volumes were not impacted by the changes in flow conditions in the Gavins Point Reach as much as in the other three study reaches.

Summary of Bank and Bed Scour

Bank scour

Based on the volumes of material scoured from the banks computed using the cross sections in this study and the cumulative erosion plots from the DEIS (USAEDNMRR 1994a) and the Darrel Dangberg and Associates and River Pros report,[1] there is a definite trend for the rate of bank erosion in all four reaches to decrease. The volumes computed in the second period were less than those in the first period for all four reaches; and the plots in the referenced reports for the Fort Peck, Garrison, and Fort Randall Reaches show a flattening of the total curves. Based on the volumes computed for the Gavins Point Reach, it appears bank

[1] Op. cit.

stability is lagging somewhat behind the other three reaches. In this case, time is probably not a factor because Garrison, Fort Randall, and Gavins Point Dams power generation started about the same time, 1954-1956. Although not addressed in this study, the conditions downstream of the Gavins Point Reach are probably having an effect on the stabilization of the banks. Without a reservoir downstream, the reach must adjust to the downstream navigation project and the adjustment of that project to the various engineering works such as dikes, revetments, and channel shortening. Bank erosion is a function of many physical and hydrologic factors such as bank material composition, groundwater conditions, streamflow, bed material discharge, rate-of-stage changes, and other factors. As in any alluvial river, bank erosion and bank building are natural phenomena and a continuing process driven by these factors. Therefore, the four reaches discussed are probably not headed to a situation in which all the banks are stable without erosion. Rather the volume of bank material being eroded will equal the volume of riverbed material added to the banks in another location, and the total erosion of the banks will be fairly constant. So, in the future some bank lines will continue to move landward, some will move riverward, and those locations will probably change from time to time. The relative elevation of the bank lines has been influenced by the dams. High flood flows have been significantly reduced, which reduced or eliminated overbank flooding. Therefore, the new bank lines are at a lower elevation due to the reduced flood flows and general channel degradation. Then the new bank lines will appear as a bench or berm adjacent to the preproject bank lines.

Bed scour

Based on the volumes of material scoured from the bed (computed using cross sections), the Fort Peck Reach actually filled (aggraded) during the first period and scoured during the second period. This indicates the potential toward bed stabilization following installation of Fort Peck Dam and a long-term degradation of the reach. Plates 30-33 in the DEIS (USAEDNMRR 1994a) present the average bed profile in this reach for 1956, 1966, and 1978 and indicate minor bed scour changes except for a few specific locations. The channel bed on the Garrison Reach was scoured significantly during the first period with a wide variation of flows and bed scour decreased significantly during the second, higher flow period. Plate 54 in the DEIS (USAEDNMRR 1994a) presents the average bed profile in this reach for 1958, 1964, 1975, and 1985. This plate shows that major degradation occurred from 1958 to 1975, minor degradation (approximately 3 ft) occurred in the upstream portion of the reach, and slight aggradation occurred in the downstream portion of the reach from 1975 to 1985. Based on these volumes computed, the Fort Randall and Gavins Point Reaches appear to be still in the adjustment phase. Plate 95 in the DEIS (USAEDNMRR 1994a) presents the average bed profile in the Fort Randall Reach for 1954, 1967, 1975, and 1985. This plate shows significant degradation over the reach from RM 879 to 860 and aggradation from RM 850 to 844. Of the four reaches studied, the Gavins Point Reach continues to be the most active and is probably the furthest from reaching adjustment. For the two periods analyzed, the bed scour volumes had about equal magnitude, even though the flow conditions were significantly different in those periods. Plate 113 in the DEIS (USAEDNMRR 1994a) presents the average bed profile in the Gavins Point Reach for 1959, 1965, 1974, and 1986. This plate shows that from 1959 to 1986 major degradation, as great as 10 ft, occurred. However, from RM 810 to 776 the majority of the degradation occurred between 1959 and 1974 with significantly less bed scour after 1974. Between RM 765-754, significant scour occurred between 1974 and 1986. As mentioned, this reach is responding to the development and stabilization of the downstream navigation channel and lacks a base level as constant as those of the other three study reaches.

6 Impacts on Turbidity Trends

Introduction

Turbidity analysis is the tool used to determine if there is a link between reservoir operation and Missouri River turbidity and, if there is, how the river will react to the PA. Historical turbidity data were analyzed to determine any reasonable relationships. The historical turbidity data were analyzed and reduced by Mr. William A. (Tony) Thomas of Mobile Boundary Hydraulics.[1] The majority of the description provided in this chapter is based on Mr. Thomas' analysis.

Data and Study Locations

WES conducted a survey of Missouri River basin stream gauges to determine if there were adequate field data for this analysis. Table 27 shows the 67 data sets (all that were identified in the WES data search) that were assembled and plotted for this section. All the graphs were made and the statistics were calculated using the spreadsheet computer program, EXCEL, Version 5, by Microsoft Corporation.

Two Missouri River locations were selected for this part of the study: the Missouri River at Bismarck, ND, and the Missouri River at Sioux City, IA. Data review at the Bismarck gauge showed an abundance of sediment and turbidity data as well as a major tributary to the Missouri River, the Heart River, entering near Bismarck. The Heart River has a stream gauge with both sediment and turbidity data records and was used as a "control group" for comparison with the Missouri River data. Sioux City, IA, was selected because it is downstream of the entire Missouri River dams system.

Turbidity has historically been measured on the Missouri River using Jackson Turbidity Units (JTU) and Nephelometric or Formazin polymer Turbidity Units (NTU or FTU). According to American Public Health Association (APHA)

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[1] Thomas, op. cit.

(1995), there is a basic limitation in the Jackson candle turbidimeter that gives JTU measurements:

> "The standard method for the determination of turbidity has been based on the Jackson candle turbidimeter. However, the lowest turbidity value that can be measured directly on this instrument is 25 units. With turbidities of treated water generally falling within the range of 0 to 5 units, indirect secondary methods have been required to estimate turbidities on such samples."

In 1978 on the Missouri River, turbidity instrumentation changed to a nephelometric basis, which measures only that light intensity scattered at 90 deg. Readings from these instruments are expressed in HACH FTU units. There were a few measurements in JTU units during 1978, but FTU units have been used exclusively since 1979. Again referencing APHA (1995), relative to JTU and FTU measurements:

> "Since there is no direct relationship between the intensity of light scattered at 90 degrees and the Jackson candle turbidity, there is no valid basis for the practice of calibrating a nephelometer in terms of candle units... Formazin polymer, which has gained acceptance as the turbidity standard reference suspension in the brewing industry, is used as the reference turbidity standard suspension for water. It is easy to prepare and is more reproducible in its light-scattering properties than the clay or turbid natural water standards previously used. The turbidity of a given concentration of formazin suspension is defined as 40 nephelometric units. This same suspension of formazin has an approximate turbidity of 40 Jackson units when measured on the candle turbidimeter; therefore, nephelometric turbidity units based on the formazin preparation will approximate units derived from the candle turbidimeter but will not be identical to them."

Therefore, this basic difference between the Jackson candle turbidimeter (JTU) and the HACH nephelometric methods (FTU) makes it necessary to present data as two separate sets rather than one combined data set.

Turbidity Analyses

Heart River at Mandan

As mentioned, the data from the Heart River at Mandan, ND, were used as the "control group" for this analysis. The Heart River is located across the Missouri River from Bismarck, ND, and enters the Missouri River at Bismarck. The drainage area of the Heart River at the Mandan gauge, number 06486000, is 3,310 square miles.

The recorded period for the Heart River at the Mandan gauge, which recorded all values in FTU units, is presented in Plate 49. The turbidity values are generally low and, while there is a trend line in Plate 49, the correlation between time and turbidity is not significant. The relationship between turbidity measurements and water discharge for the Heart River indicates a fairly reasonable relationship exists with an R^2 parameter of 0.82, which is significant (Plate 50). R^2 as it is used in this report, is defined as an indicator of the goodness of fit of the developed trend line to represent the data points used in the development of the trend line or equation. An R^2 value of 0.00 means there is no relationship between the data and the equation, while an R^2 value of 1.00 means there is a perfect relationship between the data points and the equation and that all points are on the trend line. Plate 51 shows no correlation between turbidity and season of the year; however, there is a tendency toward higher values, in the scatter, during the spring and early summer months (March to June). Therefore, the relationship between turbidity and water discharge is reasonable because the Heart River flows are likely to be higher in the spring and early summer.

Missouri River at Bismarck

The turbidity data of the Missouri River at Bismarck, ND, were retrieved from STORET using gauge numbers 370033 and 06342500 and were collected with two different types of instruments. All the turbidity measurements versus time are presented in Plate 52. Between 1957 and 1971, the data came from gauge number 370033. These values do not correlate with time using single parameter regression techniques with Plate 52 showing wide scatter over this time frame. By visual inspection, the number and magnitude of high values become smaller as time passes. During the period from 1957 through 1961, very low JTU turbidity values were recorded; however, between 1964 and 1969 there are no values less than 25 JTU. For the period between 1974 and 1982, the data are from gauge number 06342500. Plate 52 shows that during this period turbidity values do not exceed 20.

Because of uncertainties about the earlier data, this study will focus on data during the period 1974 through 1982. As stated in the preceding paragraph, these data came from STORET using gauge number 06342500 and are shown versus time in Plate 53. It should be noted that between 1957 and 1977, turbidity was measured in JTU units and beginning with 1977 the turbidity was measured in FTU units. It is obvious from Plate 53 that no relationship of turbidity versus time exists.

Plate 54 shows the relationship between water discharge and turbidity for the period between 1974 and 1982. Trend lines are shown in this plate to highlight the two types of instruments used in collecting the data. There is no significant correlation between water discharge and turbidity.

Seasonal variations in turbidity are presented in Plate 55. While there is no measurable trend, turbidity values tended to be lower during January and February than during the rest of the year. Also, the highest JTU values were in May through July.

Missouri River at Sioux City

Turbidity measurements for the Missouri River at Sioux City, IA, collected between 1974 and 1987 at gauge number 06486000 are shown in Plate 56. Note that, as at Bismarck, there was an instrumentation change in 1977. There is a trend line on this plate, but single parameter linear regression shows that time does not explain a significant portion of the scatter in these turbidity data.

A turbidity plot versus water discharge at Sioux City is shown in Plate 57. As in Bismarck, there is no significant correlation between these variables. The seasonal trend in turbidity at Sioux City, shown in Plate 58, indicates a tendency toward higher values in the scatter during March through July.

Knife River

Analysis performed to this point indicated that on the Missouri River at Bismarck and Sioux City, there was no significant relationship between turbidity and water discharge. However, on a smaller watershed, the Heart River at Mandan, data indicated a significant relationship between turbidity and water discharge. Therefore, it was decided to address the potential for relationships between the water discharge of a Missouri River tributary and turbidity measurements obtained on a Missouri River gauge downstream of that tributary.

Based on the data available, the Knife River, which enters the Missouri River near RM 1,375.7, was used as the Missouri River tributary for this analysis. The water discharge measurements obtained by the USGS were compared to corresponding turbidity measurements available in STORET for the Bismarck gauge previously used. Two days were added to the Knife River measurements to account for travel time between the water discharge readings on the Knife River and turbidity readings on the Missouri River at Bismarck. A plot of the data is presented in Plate 59. The trend lines show that, using either JTU or FTU units, there is no significant correlation of Knife River discharge and Bismarck turbidity measurements. The R^2 values vary from about 0.08 to 0.12 for the FTU and JTU readings, respectively, indicating insignificant correlation.

Relationship of Turbidity and Suspended Sediment Concentration

One study task was to determine any trends relative to turbidity. It is obvious from the turbidity data presented that on the Missouri River, there are no significant relationships between turbidity and time or water discharge, but there is somewhat of a relationship between turbidity and the seasons of the year. Therefore, additional relationships were investigated to determine if some measured parameter is related to turbidity.

Because suspended sediment concentrations have been historically obtained on the Missouri River, a decision was made to expand the study in that direction. The following explanation of turbidity was obtained (APHA 1995):

> "Turbidity in water is caused by the presence of suspended matter, such as clay, silt, finely divided organic and inorganic matter, plankton, and other microscopic organisms. Turbidity is an expression of the optical property that causes light to be scattered and absorbed rather than transmitted in straight lines through the sample. Attempts to correlate turbidity with the weight concentration of suspended matter are impractical because the size, shape, and refractive index of the particulate materials are important optically but bear little direct relationship to the concentration and specific gravity of the suspended matter."

In general, measurements of water quality parameters are abundant in the Missouri River data sets, but the synoptic data required to perform a scientific study are lacking. For example, the entire STORET record was retrieved for the Missouri River gauge at Sioux City, IA (gauge number 06486000). The file contains 1.5 megabytes of data and spans the period from 1971 through 1986. Only one complete measurement of the data types is needed to explore a possible relationship between water discharge, turbidity, and the concentration of fine sediments. This statement by APHA (1995) concerning the lack of correlation between sediment concentration by weight and light refraction seems reasonable. For the general case, sediment measuring instruments must be based on mass and not on particle surface area. However, there have been specific sites where turbidity was used as a surrogate parameter for sediment concentration.

Suspended Sediment Concentrations

Heart River at Mandan

As in the previous section, the Heart River at Mandan continued to act as the control group for this analysis. In the river engineering and sedimentation field, the term fines, or the phrase concentration of fines, refers to the concentration of silt and clay particles in the water. The sedimentation diameter of silt and clay particles is less than 0.062 mm, as determined by laboratory tests.

The concentration of fine sediments and turbidity measurements are compared in the plot in Plate 60. The statistical correlation using the single-parameter linear regression produced a low R^2 value, and Plate 60 shows an exceptionally large number of low-value measurements.

Plate 61 shows an excellent correlation of water discharge versus suspended sediment concentrations on the Heart River. The trend line on this plot has an R^2 value of 0.92.

Plate 62 shows the fine sediment measurements taken in the Heart River between 1978 and 1993. This plate shows that the concentration of fines is not correlated with time statistically. However, visually the trend seems to decline with time on the Heart River. That assessment is based on the absence of higher values, 100 mg/ℓ and above, in the 1990's compared with the 1980's.

A plot of suspended sediment versus the concentration of fines is presented in Plate 63. The R^2 value of 0.95 indicates a significant correlation between these variables. However, a plot of the concentration of fines versus the months of the year indicated no seasonal correlation on the Heart River (Plate 64). In fact, there is not even a visual trend in the data to imply a correlation.

Missouri River at Bismarck

When turbidity and the concentration of fines at Bismarck were plotted against each other, the single-parameter linear regression technique explained only 40 percent of the scatter in these data (Plate 65). Water discharge and suspended sediment concentrations at Bismarck are compared in Plate 66. The R^2 value is low; however, the plot indicates that the discharge could change by 50 percent and the concentration would remain within the scatter of the measured data.

A plot of suspended sediment concentration and the concentration of fines in those samples indicates a low correlation (Plate 67). Single-parameter linear regression correlation is regarded as low because the graph includes the concentrations of fines in both variables (the concentrations of fines is one component of the suspended sediment concentrations). The reason for this low correlation is not obvious. A plot of the concentration of fines versus the months of the year (Plate 68) shows a significant amount of the scatter in the concentration of fine sediments. This trend is different from similar plots using turbidity, which show a trend toward higher spring values.

Suspended sediment measurements were available to investigate the long-term sediment yield for the period from 1972 to 1980. Annual sediment yield depends on the runoff hydrology and mean daily discharges for the period being analyzed. Total annual water yields for the time frame from 1972 to 1980 are presented in Plate 69. Two years with about equal volumes of water runoff are 1974 and 1980. The total water volume in 1974 was about 19.3 million acre-ft and in 1980 about 17.7 million acre-ft of water, a difference of about 8 percent. Note that both these years were somewhat greater than the annual water volume for the CWCP or PA. The flow duration curves for 1974 and 1980 are shown in Plate 70.

Using the suspended sediment measurements and flow duration curves, a sediment discharge duration curve was developed (Plate 71). It should be noted that the "step" or "jump" between about 4,000 and 12,000 tons/day on the 1974 sediment discharge curve in Plate 71 was present in the data supplied to WES. The reason behind the inconsistency was not apparent; however, a review of the discharges and suspended sediment measurements indicated that the flat portion of the curve just before the "step" occurred in April through May of 1974. Over this period the measurements remained relatively constant while the discharge increased slightly. Therefore, since the measurements remained relatively constant over a specific period of time, it was surmised that possibly there was a problem with the measuring device. However, since there was no proof of such a malfunction and it occurred over a short period of time, the curve for 1974 was used as presented in Plate 71. Sediment yield decreased from 4.8 million tons per year in 1974 to 3.2 million tons per year in 1980. Therefore, the decrease in sediment yield was about 33 percent for the 8 percent decrease in total water volume.

Using these procedures for 1974 and 1980, similar computations were performed for the other years in the period from 1972 to 1980. Total water and sediment yields for each year are plotted in Plate 69. Plate 69 shows that in the period from 1972 through 1980 the greatest water yield occurred in 1975 and the greatest sediment yield occurred in 1972.

As discussed previously, the total annual average water volume for the Garrison Reach for the CWCP and PA is approximately 16.3 million acre-ft. Plate 69 indicates that 1973 had a total annual water volume approximately equal to the CWCP and PA with a corresponding sediment yield of about 3.9 million tons. For comparison, 1979 had a total annual water volume slightly greater than 1973 with a sediment yield of about 3.6 million tons. The differences computed here are a function of the flow duration curves and the amount of time during the year that flows with high sediment-carrying capabilities were taking place versus other flow conditions. Therefore, it is documented that year to year the discharge hydrographs vary and the corresponding sediment yields will also vary.

Using the data computed from Plate 69, a plot of water yield versus sediment yield was developed (Plate 72). This plot indicates that for the period from 1975 through 1980, there is a good, linear relationship between water and sediment yields. The period from 1972 through 1974 appears to have a much greater variability than the later 6 years. The reduction in sediment yields implies that erosion is decreasing and some sort of equilibrium is being approached. It should be noted that in Chapter 2, 1975, 1976, 1978, and 1979 were classified as very high flow years.

Missouri River at Sioux City

Suspended sediment measurements are available in the STORET data file for the period 1971-1984 (Plate 73). The statistical correlation using single-parameter linear regression does not explain a significant portion of the scatter in the data. However, visually the trend is toward lower concentrations of fines with respect to time. That observation is based somewhat on the lowest values in

Plate 73. Plate 73 shows that the lowest concentration for 1972 was about 90 mg/ℓ, and by 1982 the low values had declined to less than 25 mg/ℓ. Such a decline could be associated with the Missouri River reservoirs; however, a similar trend was also present in the same type comparison on the Heart River. This suggests that factors other than the Missouri River reservoirs are operating in this region to reduce sediment yield.

A plot of concentration of fine sediment versus turbidity is presented in Plate 74. Whereas there is no correlation between those variables using JTU measurements, there is a correlation when the FTU measurements are plotted. Relative to the FTU units the R^2 is 0.70, which indicates a reasonable relationship.

Comparison of water discharge versus the suspended sediment concentration at Sioux City produced the plot in Plate 75. The R^2 shows no significant correlation, which is typical for these variables. Concentration of fines versus the concentration of suspended sediment (Plate 76) plot indicates no statistical significance between those variables. Comparison of the concentration of fine sediments to the months of the year to determine a seasonal relationship proved insignificant also (Plate 77).

Computations of sediment yields of the Missouri River at Sioux City were not made due to insufficient data.

Turbidity Related to Dissolved Solids

During this analysis, numerous variables were compared with one another. One particular variable, dissolved solids, provided interesting correlations, which are presented here. A plot of turbidity versus dissolved solids is presented in Plate 78. The Bismarck data used in this plot span a relatively short range, and the linear regression relationship does not adequately explain the scatter in the data.

The interest in dissolved solids is twofold. First, the presence of dissolved solids, depending on types or quantities, may affect the turbidity directly. Second, their presence, again depending on types or quantities, may affect the interaction between water chemistry and the clay minerals in the water column. Therefore, another plot for Bismarck comparing dissolved solids and concentration of fine sediments was prepared (Plate 79). There is an excellent correlation between these two variables, and the trend line on the plot has an R^2 value of 0.92, which is significant. More work is needed to explain the physics of the processes, but there appears to be a link between dissolved solids and turbidity at this gauge.

Summary

Analysis of data for the Heart River at Mandan, ND, indicates a good correlation between turbidity and water discharge and suspended sediment concentrations and water discharge. Missouri River data at Bismarck, ND, and Sioux City, IA, and at the Heart River indicated a seasonality to turbidity with a tendency to be the highest in the spring and early summer. At the two Missouri River gauges, there was no correlation between turbidity and water discharge nor suspended sediment concentrations and water discharge. There was no correlation between Knife River discharges and turbidity measurements at Bismarck. Of the other variables compared, there may have been some slight or low correlation, but none significant enough to use as a predictive tool.

Since there is a correlation between turbidity and water discharge on the Heart River and none on the Missouri River at Bismarck or Sioux City (both have large watersheds upstream), perhaps such correlations are possible only on smaller watersheds. Also, numerous tributaries influence turbidity or the lack of it with their inflows to the river. Therefore, it is highly likely that the resulting Missouri River turbidity is as much a "mirror" of the accumulated turbidity quantities it collects from its tributaries as it is a direct result of the turbidity it creates for itself. One additional effect of the Missouri River Main Stem dams is that the cleaner, less turbid releases may actually dilute the turbidity supplied by the tributaries.

The annual sediment yield at a given location varies significantly from year to year and is dependent upon the duration of flow conditions capable of carrying riverine sediments.

Impact of the Preferred Alternative on Turbidity

This analysis indicates no relationship between turbidity and the discharges in the Missouri River; therefore, it is highly unlikely that changing operation from the CWCP to the PA will affect turbidity. Differences caused by a change from the CWCP to the PA are insignificant when compared to the influences of tributaries or increased Missouri River flows in high flow years.

7 Impacts on Bank Stability

Introduction

A cumulative erosion impacts issue of concern on the Missouri River was the present degree of bank stability and the determination of bank stability in the future if the CWCP were continued. Also, MRR was interested in a determination of future bank stability if the PA were to be adopted. The majority of this chapter was taken from Darby and Thorne.[1]

The research and analysis performed in this effort were for the Fort Peck and Garrison Reaches. Trends identified for these two reaches could possibly be applicable to the Fort Randall and Gavins Point Reaches. Darby and Thorne were to determine whether or not a change from the existing to an alternative operating plan might cause discernible impacts on bank erosion processes, bank stability with respect to mass failure, and rates of bank line migration.

The objectives of this part of the erosion study were to (a) establish the channel form and bank stratigraphy and present status of riverbank stability along the study reaches to locate critical sites experiencing accelerated bank erosion and mass failure; (b) identify the erosion processes and failure mechanisms responsible for retreat; (c) estimate the short-term (1 to 5 years) impacts of changes in the regulated flow regime from the CWCP to the PA on key bank hydrological parameters, and hence stability with respect to mass failure; (d) estimate the long-term (50 years) impacts of the CWCP and PA flow regimes on bank erosion at 18 selected study sites; and (e) estimate the impact of long-term (50 years) bank erosion (estimated in (c)) on bank geometry and bank stability with respect to mass failure at the 18 selected study sites.

[1] S. E. Darby and C. R. Thorne. (1996). "Bank stability analysis for the Upper Missouri River," prepared by University of Nottingham, Nottingham, England, for U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Study Approach

The bank stability study was divided into three elements. First, a field reconnaissance of the two study reaches was undertaken to record existing channel conditions, locate unstable sites, identify dominant erosion processes and failure mechanisms, and collect the data required to perform numerical bank stability analyses. Second, historical records of channel response to flow regulation in the study reaches were collected and interpreted to establish past response of the river to regulation and determine rates of bank erosion and bed scour. These historical data were then used as a basis for predicting future rates. Finally, a numerical model was applied to estimate present and future conditions of bank stability with respect to mass failure under the CWCP and PA.

Stream Reconnaissance

A stream reconnaissance was made by boat in accordance with guidelines documented by Thorne (1993). Approximately 160 miles of bank (62 percent of the two study reaches) were classified according to stratigraphy, profile, failure mechanism, and overall stability. Bank failure mechanisms were classified as being of the planar, rotational, cantilever, piping/sapping, or pop-out type (Plate 80). Failure mechanisms were recorded on 1:24,000-scale aerial photographs as the boat progressed downstream. These photographs were later used to estimate the locations and percentage lengths of stable and unstable banks, and the proportion of unstable bank line in each failure category. Notes were made regarding the geomorphic context of bank retreat at each location and, particularly, how failure categories related to position at channel bends. Photographs were taken at 126 sites along the two study reaches.

Data Used in Bank Stability Analysis

Channel morphology data for the periods 1956 to 1978 (Fort Peck Reach) and 1958 to 1985 (Garrison Reach) have been collected by Omaha District and compiled in the DEIS (USAEDNMRR 1994a). This report includes bed elevation, bed width, and bed material size data through time by river mile for both study reaches. These surveys were supplemented by additional mean bed elevation and bed width data obtained from a report by Williams and Wolman (1984), extending the period of record from 1953 to 1985 in the Garrison Reach. Flow and sediment transport data were supplied by Omaha District and/or obtained from published USGS stream gauge records.

Channel and watershed characteristics are summarized in Table 28. In both reaches, the bed material is predominantly sand, with coarser gravel located only in the upstream portions, close to the dams. Bank materials are composed of fine sand or silt (also observed by Williams and Wolman 1984), and have little intrinsic cohesive strength (Table 29). Table 29 also shows geotechnical data from other sandy/silty streambanks in other regions of the United States. In this study, data were collected and analyzed for reaches with unprotected riverbanks. Riverbank protection is rather limited in the Fort Peck Reach but covers significant portions of banks along the Garrison Reach.

Annual monthly average (1898-1993) hydrographs for the CWCP and the PA for the Fort Peck and Garrison Reaches are shown in Plates 2 and 3, respectively. The PA would have the effect of increasing discharge releases during spring and summer, but decreasing discharge at other times. Maximum average monthly release rates will be increased from approximately 11,000 cfs to 14,000 cfs in the Fort Peck Reach, but will be decreased from approximately 31,500 cfs to 29,000 cfs in the Garrison Reach. In both reaches, minimum average monthly releases will be reduced with the PA, and the overall impact of proposed changes in dam operation will be to increase flow duration for relatively large magnitude flows and reduce flow duration for lower magnitude flows (Plate 81).

Bank Erosion Mechanics and Flow Regulation

Three aspects of bank erosion mechanics may be influenced by changes in flow regulation:

a. Changes in the flow regime could alter the operational shear strength of the bank materials. Bank stability is increased by negative pore-water pressures in the bank during low flow in the channel and by confining hydrostatic pressure of water in the channel during high flows. Conversely, stability is decreased by excess positive pore-water pressures in the bank during rapid drawdown in the channel following a high flow event. Such hydrological impacts on stability with respect to mass failure could occur almost immediately after implementation of the PA.

b. Changes in the rates of bank erosion can be expected if the magnitude and/or frequency of flows generating fluvial erosion are altered. Such changes could begin immediately following implementation of the PA, and could continue until a new condition of dynamic equilibrium was reached.

c. Changes in rates of bed scour and sedimentation resulting from changes in the regulated regime might alter the profiles of banks along the study reaches, leading to changes in stability with respect to mass failure. Such changes would also begin immediately following implementation of the PA, but their effects on bank stability with respect to mass failure might become discernible only some years later.

Estimation of Impact of Key Hydrological Parameters on Bank Erosion

Worst-case bank hydrology parameters, corresponding to the conditions most likely to trigger bank failure, occur during the largest drawdown event of the annual hydrograph, because rapid drawdown results in relatively high phreatic-surface elevations with concurrent relatively low water-surface elevations. This condition generates maximum excess positive pore-water pressures and minimum hydrostatic confining pressures simultaneously. Inspection of the hydrograph (Plate 2) shows that for the CWCP, maximum drawdown occurs between October and November in the Fort Peck Reach (approximate decrease in discharge from 11,000 cfs to 9,000 cfs). In the Garrison Reach, maximum drawdown (Plate 3) also occurs between October and November (approximate decrease in discharge from 26,100 cfs to 18,400 cfs). It should be noted that there is a similar change in discharges in the February to March time frame. The differences in discharges in this spring period may be more critical than the fall because (a) groundwater levels may be higher in the very early spring; (b) the difference between the groundwater level and river stage may be greater because of the lower spring discharge; and (c) the change in stage may be greater as the difference in stage is generally greater for the same discharge increment at lower discharges. Therefore, there is the potential for large variations in the relative position of the stage and ground-water elevations due to these factors. These factors will introduce stochastic variability into the estimates listed in the following tabulation, which should be regarded as approximate, long-term average values.

Study Reach	CWCP EI		PA EI	
	Groundwater	Surface Water	Groundwater	Surface Water
Fort Peck Dam	0.75*H*	0.75*H* - 0.98	0.75*H*	0.75*H* - 1.97
Garrison Dam	0.75*H*	0.75*H* - 2.10	0.75*H*	0.75*H* - 2.49

Measured water-surface profiles along each study reach from MRR were used to convert these discharge values to ground- and water-surface elevations. The pre-drawdown water surface was assumed to represent the groundwater elevation, with the postdrawdown surface representing the channel water surface. It was found that the predrawdown water-surface elevation was equivalent to approximately 75 percent of the bank height H in feet at most study sites. To simplify the calculations, the groundwater elevation was, therefore, equated to this value at all sites. Bank hydrology parameters for the proposed flow regime were estimated using the same procedure, but substituting discharge and water-surface elevations appropriate for the PA.

Bank hydrology parameters for each flow regime were taken to be constant for bank stability analyses projected into the future, even though the estimates were based on measured water-surface profile data that will actually change as channel morphology adjusts. However, no data are available to estimate future changes in bank hydrology parameters caused by possible future changes in water-surface profiles, and the technology does not presently exist to predict such data reliably.

Fort Peck Reach

The primary morphological response of the channel to river regulation in the Fort Peck Reach has been bed degradation. This channel response downstream of a dam has been widely observed on many rivers, and is consistent with conclusions reached by Williams and Wolman (1984) and Borah and Bordoloi (1989), who attribute bed degradation to reduction in sediment supply following dam closure. Bed degradation during the 1955 to 1978 period varied from about 2 ft between Fort Peck Dam and the Milk River confluence, to about 1-2 ft downstream of the Milk River (Plate 82). With the exception of localized cases of narrowing or widening, little variation in active channel width relative to overall channel widening through time had been observed up to the date of the latest available survey in 1978 (Plate 83 and Plate 34 of the DEIS (USAEDNMRR 1994a)). The most recent survey (1978) indicates bed aggradation only in the furthest downstream portions of the study reach.

Garrison Reach

The primary morphological response of the channel in the Garrison Reach during the period 1953 to 1985 was also bed degradation. This finding is also consistent with data reported by Williams and Wolman (1984) and Borah and Bordoloi (1989). Plate 82 shows that degradation has been greatest close to Garrison Dam (approximately 8 ft). Degradation decreases with distance downstream (approximately 3 ft at RM 1,340). Downstream of RM 1,365, there appears to have been a recovery of bed elevation by 1-2 ft between the 1975 and 1985 surveys. Plate 83 and Plate 55 of the DEIS (USAEDNMRR 1994a) indicate channel bed width reduction in the upstream reaches during the period 1975 to 1985, associated with bed incision observed in this period. Further downstream, the relationship between channel bed width and time is unclear.

Projected Bank Erosion and Bed Scour

Channel survey data for the periods 1955 to 1978 (Fort Peck Reach) and 1953 to 1985 (Garrison Reach) (Plates 82 and 83) were used to construct regression relationships between mean bed elevation versus time and bed width versus time at the 18 bank stability study sites (Table 30). Changes in mean bed elevation versus time were assumed to be representative of changes in near-bank bed elevation through time, while changes in half bed width through time were assumed to be representative of changes in flow erosion of the bank toe through time. It is recognized that this may not be realistic for sites with highly nonuniform cross sections, or at sites subject to local scour or flow impingement. However, this procedure appears reasonable, because aerial photographs and notes made during the field reconnaissance indicate that 14 of the 18 study sites are not subject to significant streamline curvature, flow impingement, or other discernible local controls on bank line migration or near-bank bed scour.

Exponential and logarithmic regression curves were fit to the data so obtained. The regression relationship that most closely fit the survey data (highest R^2 value) was selected for use in extrapolating future channel response to the regulated flow regime. Estimates of cumulative amounts of near-bank bed degradation (ΔZ) and bank toe erosion ($\Delta W/2$) compared to estimated channel bed conditions at the present time (1995) projected 1 (1996), 5 (2000), 10 (2005), 20 (2015), and 50 (2045) years into the future (Table 31) were obtained by extrapolation of the empirical regression curves listed in Table 30. It is stressed that the 1995 reference values of mean bed elevation and bed width are themselves extrapolated estimates, because the dates of the last surveys used to construct the regression curves are 1978 and 1985 for the Fort Peck and Garrison Reaches, respectively.

At some sites, bed elevation and/or bed width were observed to be steady. In such cases, future bed elevations and bed widths were predicted to be constant and equal to the historical values. In all cases, estimates of lateral fluvial erosion increments were obtained by distributing predictions of overall change in channel bed width equally between both banks. Extrapolation of fitted curves to predict future channel response has no physical basis, but empirical studies have indicated that, assuming boundary conditions do not change during the period of channel adjustment, fitted regression curves often describe the time evolution of morphological parameters quite well (e.g., Williams and Wolman 1984; Lohnes 1991; Simon and Hupp 1992). Despite this, it should be recognized that the extrapolation approach to estimating future near-bank channel bed conditions is an approximate technique subject to limitations, and statistical error and uncertainty (Table 31). Hence, a range of ΔZ and $\Delta W/2$ values, based on the extrapolated values plus or minus the error estimates obtained at 95 percent confidence limits, was used to support bank stability computations.

Predictions of future fluvial bank erosion and near-bank bed scour for the CWCP (Table 31) were obtained by extrapolating the regression curves listed in Table 30. Statistical uncertainty in these extrapolations is represented by the 95 percent confidence interval. Amounts of bed scour after 50 years of channel adjustment ranged between 0.2 ft of bed deposition and 2.5 ft of scour for the 13 Fort Peck study sites. Bed scour in the Garrison Reach (study sites 14-18) after 50 years ranged between 0.7 and 2.6 ft of scour. The rate of bed scour over the 50-year period averaged 0.004 ft/year and 0.03 ft/year in the Fort Peck and Garrison Reaches, respectively. Bank erosion in the 13 Fort Peck sites ranged between zero and 20.6 ft over the 50 years, while bank erosion in the 5 Garrison sites ranged between zero and 29.7 ft. The rate of bank scour over the 50-year period averaged 0.09 ft/year and 0.20 ft/year for the Fort Peck and Garrison Reaches, respectively. Mean rates of bed scour and bank erosion are low, indicating that the channel is at or approaching a condition of dynamic equilibrium. At some specific study sites (sites 8, 10, 11, 14, and 16), fluvial bank erosion rates are higher due to local conditions. There are also some study sites (sites 14 to 17) downstream of Garrison Dam that are predicted to experience higher rates of bed scour. This may indicate continued adjustment of the bed downstream of the Garrison Dam.

As stated at the beginning of this chapter, the sites selected during the 1995 field reconnaissance were based on what the researchers considered as being critical and experiencing accelerated bank erosion and mass failure. The sites were selected to cover a range of observed bank profiles, stratigraphies, and geomorphic locations, representative of the proportion of the bank line in each category of bank stability. As shown in Tables 30 and 31, of the 18 sites selected for the time frames available, 9 sites in the Fort Peck Reach and 3 sites in the Garrison Reach had constant channel widths. Therefore, at those 12 sites, either no bank erosion occurred or bank erosion and bank building occurred at the same rate producing a net zero gain in channel width. It should also be noted, as discussed earlier, that those new banks would be at a lower elevation than the older river banks and increased material would be delivered to the channel due to bank erosion.

Using the volumes presented in Tables 15 and 16, the planform correction factors presented in Chapter 5 (1.29 and 1.24 for the Fort Peck and Garrison Reaches, respectively), the computed average bank heights (11.2 ft and 19.0 ft for the Fort Peck and Garrison Reaches, respectively), and the reach lengths, the following average bank erosion rates were computed:

Period	Bank Erosion Rate, ft/year
Fort Peck Reach	
1955-1966	6.4
1966-1978	3.4
Garrison Reach	
1956-1976	7.8
1976-1985	4.2

While these computed bank erosion rates are significantly greater than those previously computed, they support the trends of the rates being greater in the Garrison Reach and a decrease in the rates over time. The discrepancy between these bank erosion rates and the rates based on the regression curves is probably due to the limited number of sites used in the regression analysis and the fact that so many sites indicated no <u>change</u> in channel width over the data period. It should also be noted that the field reconnaissance took place in 1995 at which time the researchers interpreted the banks as being active while those particular banks may not have been as active over the previous data periods.

Estimation of Long-term Changes in Bank Geometry

Bank stability simulations were based on estimates of the values of future bank profile parameters for the CWCP and PA, in conjunction with bank hydrological parameters corresponding to the CWCP and PA.

Future (1995-2045) bed elevation and bank toe erosion trends for the PA were estimated by comparing hydraulic and sedimentary regimes corresponding to existing and proposed flow regimes. Base data used to define the flow and sedimentary regimes of the study reaches are the flow duration curves for the existing and proposed Fort Peck Dam and Garrison Dam operating plans (Plate 81), and suspended sediment transport data from USGS gaging stations in the study reaches. In large rivers a substantial fraction of the total load is wash load. However, it is the erosion, transport, and deposition of bed material that is fundamental to the hydraulic shaping of the channel (Leopold 1992; Thorne, Russel, and Alam 1993). Suspended bed material transport rates were estimated by excluding the fraction of measured load finer than 0.062 mm. The silt may be viewed as wash load passing through the system without playing a significant role in forming the channel. Data collected between 1958 and 1980 from gauges located at Culbertson (RM 1,620) and Bismarck (RM 1,320) in the Fort Peck and Garrison Reaches, respectively, were used to develop bed material load rating curves:

$$Q_s = -11440 + 5982 \log\left(\frac{Q}{35.3}\right) \qquad (R^2 = 0.84) \tag{7}$$

$$Q_s = -70366 + 28786 \log\left(\frac{Q}{35.3}\right) \qquad (R^2 = 0.7 \tag{8}$$

where

Q_s = suspended bed material transport rate, tons/day

Q = discharge, cfs

Annual bed material load data corresponding to average monthly discharges for the CWCP and PA were obtained by multiplying flow duration (converted to days) by bed material transport rate. Based on these equations, the suspended bed material transport rate will be zero for Fort Peck Reach discharges of 2,885 cfs or less and Garrison Reach discharges of 9,823 cfs or less.

Computations were made using the flow duration curves for the CWCP and PA (Plate 81) with Equations 7 and 8. It should be noted that these historical flow duration curves include discharges greater and less than those presented in the average monthly discharges for the CWCP and PA in Chapter 3 and in Plates 2 and 3. The total annual volume of bed material load was determined for the Culbertson gauge in the Fort Peck Reach and the Bismarck gauge in the Garrison

Reach. The results of those computations are presented in the following tabulation:

Reach	Annual Bed Material Load, tons		Difference in Load	
	With CWCP	With PA	Tons	Percent
Fort Peck Reach at Culbertson	987,930	944,440	43,490	4.4
Garrison Reach at Bismarck	3,479,340	3,280,170	199,170	5.7

As the tabulation indicates, redistribution of the flow duration curve at Culbertson slightly decreased the total bed material by about 4 percent, while at Bismarck the flow redistribution decreased the total bed material load about 6 percent. Table 32 presents the detailed information used to develop this tabulation.

In Chapter 6 similar computations were made at Bismarck for the period from 1972 through 1980 (Plate 69). Plate 69 indicates that over the period the annual total bed material load varied from about 2 million tons to about 12.5 million tons. It should be noted that the values computed in the tabulation were based on data collected between 1958 and 1980. A review of the flow classifications presented in Chapter 2 and Table 2 indicates that from 1958 through 1964 and 1966 were low or medium flow years. The years from 1965 through 1972 were high or very high flow; and from 1973 through 1980, flows varied between medium and very high flow conditions. Therefore, even though the sediment yield volumes were computed based on different time frames, using somewhat different methods, and by different researchers, the results compare favorably.

Although the developed bed material load rating curves may not completely describe present conditions, those curves are useful in evaluating impacts of the PA versus the CWCP. Considering the average monthly releases for the Fort Peck Reach (Plate 2) and for the Garrison Reach (Plate 3), monthly bed material loads were computed using Equations 7 and 8, respectively. The results of those computations are presented in Table 33. The results presented in the table indicate that for the Fort Peck and Garrison Reaches, the total annual bed material load will be slightly reduced with the PA, less than 2 percent, compared with the CWCP. This indicates that the flows outside of the CWCP and PA range, either of greater or less magnitude or of longer or shorter duration, have significant impact on the total bed material load. Since those flows and durations are not within the control of the water control plan, the impacts of such circumstances should not be considered in evaluating the CWCP and PA.

One interesting point to be noted, however, is that comparing the total annual bed material load for the Garrison Reach CWCP of approximately 3.8 million tons with the medium flow years presented in Plate 69 shows that the 3.8 million tons per year is in the range of actual measured values. On the plate 1973, 1974, 1977, and 1980 would be classified as medium flow years. The corresponding annual sediment yields presented in Plate 69 for those years were 3.9, 4.9, 1.9, and 3.2 million tons. Those values compare relatively well with the CWCP total annual bed material load computed previously. The variation in the values is an indication of the annual variability within the Missouri River system particularly from one reach to another.

A number of studies have indicated that the discharge transporting the most bed material (termed the effective discharge) is the channel-forming or dominant discharge (Wolman and Miller 1960; Hey 1975; Andrews 1980; Biedenharn and Thorne 1994; Thorne, Russel, and Alam 1993). Many authors have also developed empirically based hydraulic geometry equations (Leopold and Maddock 1953; Simons and Albertson 1960; Ackers and Charlton 1970) relating stable channel dimensions to dominant discharge Q using power equations of the form:

$$W = aQ^{b} \tag{9}$$

$$D = cQ^{f} \tag{10}$$

$$V = kQ^{m} \tag{11}$$

where

W = stable channel width

D = stable channel depth

V = mean velocity

a, c, k, b, f, m = empirical coefficients and exponents whose values are determined by regression

The effective discharge for the Fort Peck Reach was determined to be 7,000 cfs for the CWCP and PA and for the Garrison Reach, 18,500 cfs for the CWCP and PA. Because the effective discharge values estimated for the CWCP and PA are unchanged, the impacts of a change in flow regime on channel morphology are likely to be negligible.

Darby-Thorne Bank Stability Analysis

The riverbank stability analysis developed and tested by Darby and Thorne (in preparation) is suitable for use in this study. Simulations are based on bank profiles deformed by combinations of near-bank bed scour and direct fluvial entrainment (Plate 84). Upper bank failures can also be simulated, and the effects of pore-water and hydrostatic confining pressures are included in the analysis. The analysis has also been shown to have better predictive ability than the alternative models of Lohnes and Handy (1968), Huang (1983), or Osman and Thorne (1988)

(Darby and Thorne, in preparation) (Table 34). The analysis is valid for cohesive, steep (bank angles greater than 50 deg), eroding, nonlayered riverbanks that fail along planar failure surfaces. Based on the results of the field reconnaissance, this failure is the type most commonly observed along the study reaches (Table 35 and Plate 85). Input data parameters corresponding to bank conditions at the time of observation (September 1995) were obtained at 18 sites (13 at Fort Peck Reach and 5 at Garrison Reach) during field reconnaissance.

To apply the analysis, bank height, tension crack depth, relic tension crack depth, and angle of the uneroded bank slope are required to describe the geometry of the bank profile (Plate 84). Cohesion, friction angle, and unit weight values are used to characterize geotechnical soil properties. Groundwater and surface water elevations are used to simulate the effects of bank hydrology on stability.

Sites were selected to cover a range of observed bank profiles, stratigraphies, and geomorphic locations, representative of the proportion of the bank line in each category of bank stability (Table 35). Cantilever, rotational, pop-out, and piping/sapping type failures were not used in the computations for the 18 study sites, because the numerical model is valid for planar failures only. This is justified because, on unstable banks, planar failures are the most common of the observed failure types (Table 35). Bank heights and tension crack depths were measured by standard surveying techniques and/or direct measurement with a survey rod. Average bank angles were obtained using a clinometer resting on a survey rod laid along the bank profile.

A hand-held shear vane tester was used to obtain in situ measurements of bank material shear strength on exposed bank faces. Ten measurements of bank material shear strength were obtained at six separate sites. Mean values of shear strength at the six sites were all close to 108.6 lbf/ft^2 (Table 29). Bank shear strength along the entire study reach was, therefore, characterized using this value.

Shear strength values can be resolved into cohesion and friction angle components using the Coulomb equation:

$$s = c + \sigma \tan \phi \qquad (12)$$

where

s = soil shear strength, lbf/ft^2

c = soil cohesion, lbf/ft^2

σ = normal stress, lbf/ft^2

ϕ = friction angle, deg

The value of normal stress is unknown when the hand-held shear vane testing device is used, but can be computed based on back calculations of observed failure block geometry (Plate 84), for which $\sigma = W_t \cos \beta = \gamma V \cos \beta$; where V is failure block volume per unit reach length, ft^3/ft; W_t is the weight of the failure block, lb/ft; and γ is soil unit weight, lb/ft^3. In addition to observations of failure block volume and failure plane angle, assumptions regarding the nature of the soils under worst-case conditions were, therefore, required to estimate cohesion and friction angle components. Worst-case conditions refer to the values of cohesion, friction angle, and unit weight when the soil is saturated and most likely to fail (Thorne, Murphey, and Little 1981).

Friction angle was assumed equal to 20 deg for worst-case conditions, because observations made during field reconnaissance indicate saturated bank materials come to rest at angles close to this value. This estimate is, therefore, considered reliable and accurate. Unit weight values were measured using laboratory analysis of samples taken from the field and are reliable and accurate. Worst-case cohesion values were obtained by estimating values of the normal stress in Equation 12 at the six bank material sampling sites by reconstructing failure block geometry based on measured bank profiles at those sites. Using the values $s = 108.6$ lbf/ft^2, $\gamma = 134.4$ lb/ft^3, $V = 2.8$ ft^3/ft, $\beta = 50$ deg, and $\phi = 20$ deg (estimated using the assumption described in the previous paragraph), a value of $c = 83.5$ lbf/ft^2 was obtained. Because the estimated worst-case cohesion value was based on a back calculation using measured bank profile parameters (failure plane angle and failure volume) together with an estimated value of friction angle, worst-case soil properties used in this study should be representative of soils at the study sites. Also, soil property values derived and used in this study are comparable to values obtained by measurement on similar alluvial riverbanks (Table 29). Close correspondence between simulated and observed bank stability conditions at the study sites for September 1995 conditions also supports the validity of this procedure. On this basis, it may be concluded that soil properties estimated using these procedures are reliable and sufficiently accurate to predict the impacts of river regulation on bank stability with respect to mass failure.

Summary of Field Reconnaissance Rates

Contemporary conditions of bank line stability are summarized in Plate 85 and Table 35. The conditions indicate that 57 percent of the banks reconnoitered in the Fort Peck Reach display evidence of instability with respect to mass instability, compared to 41 percent in the Garrison Reach. Planar failures are the most common mode of collapse, accounting for 45 percent and 59 percent of unstable banks in the Fort Peck and Garrison Reaches, respectively. Pop-out (33 percent in the Fort Peck Reach and 14 percent in the Garrison Reach) and cantilever-type (19 percent in the Fort Peck Reach and 27 percent in the Garrison Reach) failures are also observed along shorter, but still significant, lengths of unstable bank line in both reaches. It should be noted that very few rotational failures were observed at any site during the field reconnaissance, and the 3 percent of such documented failures were spread over several locations. Therefore, no rotational failures are included in Plate 85.

Plate 85 shows a general tendency for the severity of bank instability observed during September 1995 to decrease with distance downstream of Fort Peck Dam. In contrast, bank instability increases with distance downstream of Garrison Dam (Plate 85). Planar failures are the most common mechanism of bank collapse in this Fort Peck Reach (with the exception of subreaches between RM 1,640-1,670 and 1,690-1,710). Planar failures are dominant in three of five sampled subreaches of the Garrison Reach, although classifications are based on a relatively small sample size in the other two subreaches.

Bank Stability Analysis Results

The Darby-Thorne bank stability analysis (Darby and Thorne, in preparation) was applied at each of the 18 trial bank sites to produce quantitative estimates of bank stability for (a) existing conditions; (b) conditions reflecting the short-term impact of the PA on bank hydrological parameters; and (c) conditions corresponding to long-term (1995 to 2045) channel changes under either flow regime.

Banks were classified into one of four categories:

a. Stable banks have simulated factors of safety, defined by the ratio of resisting to driving forces acting on the incipient failure block, greater than 1.3. Bank line retreat of geotechnically stable banks occurs only through fluvial erosion, the movement of the bank material by the water forces, and not geotechnical bank failure and subsequent removal of the failed bank material by the riverflow.

b. Marginal banks have a simulated factor of safety between 1.1 and 1.3. Bank line retreat of marginal banks occurs through fluvial erosion, but they are vulnerable to geotechnical destabilization through relatively small increases in toe scour.

c. Upper-bank banks have simulated factors of safety less than 1.1 with failure planes confined to the upper half of the bank. Bank line retreat occurs through combinations of fluvial erosion and mass instability. Rates of bank retreat in this category are frequently more severe than those in categories *a* and *b*, but are usually less severe than those of category *d*.

d. Unstable banks have simulated factors of safety less than 1.1 with failure planes intersecting the lower half of the bank profile. Bank line retreat occurs through combinations of fluvial erosion and deep-seated mass instability. Rates of bank retreat in this category are commonly severe.

The factor of safety differentiating unstable and marginal banks is set here at 1.1, rather than the theoretical value of 1.0. This adjustment was made specifically to account for the tendency of the Darby-Thorne model to overpredict factor of safety (Darby and Thorne, in preparation; Table 34). It should be noted that bank instability and bank erosion are not necessarily proportional. Bank instability is a geotechnical phenomenon, while bank erosion requires a sediment transport capability with bank material being a major factor in determining erodibility. For example, a vertical or nearly vertical bank may be technically unstable, but may not be eroding appreciably. Conversely, some banks that are low and somewhat sloped may experience annual erosion. While the Darby-Thorne stability analysis is a qualitative methodology that is helpful in indicating likely types of future failures and predicting the amount of unstable bank lines, it cannot be used to determine erosion rates and quantities.

Existing conditions

Results of the Darby-Thorne analysis for existing (1995) conditions are presented in Table 36. Bank profile, geotechnical, ground- and surface-water elevation input data, and corresponding simulated bank stability output data for each of the sections analyzed are listed. The analysis of contemporary bank stability is based on observed bank profile, geotechnical, and bank hydrology parameters measured during the September 1995 stream reconnaissance.

Six sites (three in the Fort Peck Reach and three in the Garrison Reach) are predicted to be stable. Three sites, all located in the Fort Peck Reach, are predicted to be marginal at present. Nine sites are predicted to be unstable, of which six are subject to upper bank failures. In the Fort Peck Reach, the seven unstable sites are divided between three deep-seated and four upper bank failures. The two unstable sites in the Garrison Reach are predicted to be subject only to upper bank failures at present.

Of the sites in the Fort Peck and Garrison Reaches, 54 and 40 percent, respectively, are predicted to be subject to mass instability. These values are similar to the observed overall lengths of unstable bank line (57 and 41 percent in Table 35). Discrepancies between predicted and observed failure categories occur at 5 (38 percent) of 13 sites (Table 36). Two of these involve inconsistencies between sites predicted to be marginal, but observed to be stable. At two of the remaining sites, the error is due to incorrect simulation of failure plane *location* on banks that are otherwise correctly predicted to be unstable. These discrepancies are within acceptable bounds for a reconnaissance study of this type.

Short- and long-term bank stability conditions

Bank stability analyses were conducted using input parameters for conditions projected 1 (1996), 5 (2000), 10 (2005), 20 (2015), and 50 (2045) years into the future. These simulations represent the effects of bank hydrological conditions in isolation because cumulative changes in bank profile parameters are too small at this time to affect the simulations. Because estimates of changes in perimeter erosion rates under the PA are negligible, the short- and long-term responses are identical for the CWCP and the PA for bank stability.

After 1 year into the simulation (1996), there are no significant differences between factors of safety for the flow regimes (Plate 86). After 5 years (2000), differences in factor of safety become discernible (Plate 86), but are still small. These differences are insignificant because the predicted change is insufficient to result in a shift in bank stability classification.

Bank profile data for future conditions were obtained by modifying bank profile parameters measured during the September 1995 field reconnaissance (denoted by the subscript *o* in the following equations) by the appropriate amounts of cumulative fluvial erosion and/or bed scour (Table 31):

$$H = H_o + \Delta Z \tag{13}$$

$$H' = H_o - (\Delta W/2) \tan \alpha \tag{14}$$

Values of ΔZ and $\Delta W/2$ used in Equations 13 and 14 were obtained from Table 30. Simulations were conducted using a range of H and H' values, based on ranges of ΔZ and $\Delta W/2$ corresponding to 95 percent confidence intervals of the extrapolated regression curves. Simulations also accounted for the effects of bank hydrological conditions.

Bank stability results at each successive date in the simulation are shown in Plate 86. The error bars in this plate reflect the uncertainty introduced into the factor of safety computations that results from using a range of values of ΔZ and $\Delta W/2$ in the bank simulation. After 50 years (2045), between 10 and 12 of the 18 study sites (56 and 67 percent) are predicted to be subject to bank instability (Table 37). These data compare with the observation that about 54 percent of existing (1995) bank lines are subject to mass instability (Table 35). This indicates that the extent of bank line subject to mass bank failure will increase slightly over a 50-year period, under either the CWCP or PA.

Conclusions

Stream reconnaissance suggests that at the present time (September 1995), 57 and 41 percent of the banks in the Fort Peck and Garrison Reaches, respectively, exhibit evidence of bank instability and mass wasting. Field measurements of geotechnical characteristics indicate that bank material properties along the study reaches are relatively uniform. Bank materials are weakly cohesive (mean shear strength = 108.6 lbf/ft^2) sandy-silts. Planar failure due to toe scour and oversteepening by fluvial bank erosion is the most common mechanism of collapse in both study reaches.

The short-term (<5 years) impact on bank stability with respect to mass failure was analyzed by simulating changes in key bank hydrological parameters. Excess bank pore-water pressures and hydrostatic confining pressures generated under the PA flow regime are found to be indiscernible from those under the CWCP flow regime. Short-term impacts on bank stability with respect to mass failure are predicted to be negligible.

In predicting long-term (up to 50 years) bed scour and fluvial bank erosion rates, it is essential to consider the historical context of channel adjustment trends along the study reaches. This is because existing trends of channel adjustment will drive ongoing channel adjustment under the existing flow regime. Conversely, any major alterations from the existing flow regime will produce divergence from these existing channel adjustment trends.

Amounts of bed scour after 50 years of channel adjustment ranged between 0.2 ft of bed deposition and 2.5 ft of scour for the 13 Fort Peck study sites. Bed scour in the Garrison Reach (study sites 14-18) after 50 years ranged between 0.7 and 2.6 ft. The rate of bed scour over the 50-year period averaged 0.004 ft/year and 0.03 ft/year in the Fort Peck and Garrison Reaches, respectively. Bank erosion in the 13 Fort Peck sites ranged between zero and 20.6 ft over the 50 years, while bank erosion in the 5 Garrison sites ranged between zero and 29.7 ft. The rate of bank scour over the 50-year period averaged 0.09 ft/year and 0.20 ft/year for the Fort Peck and Garrison Reaches, respectively. Mean rates of bed scour and bank erosion are low, indicating that the channel is at, or approaching, a condition of dynamic equilibrium. At some specific study sites (sites 8, 10, 11, 14, and 16), fluvial bank erosion rates are higher due to local conditions. There are also some study sites (sites 14 to 17) downstream of Garrison Dam that are predicted to experience higher rates of bed scour. This may indicate continued adjustment of the bed downstream of the Garrison Dam.

Analysis of the sediment regime of the study reaches under the CWCP and PA flow regimes using measured data suggests that the annual suspended bed-material load will decrease under the PA by about 4 and 6 percent for the Fort Peck and Garrison Reaches, respectively. The dominant discharge was found to be about 7,000 cfs and 18,500 cfs in the Fort Peck and Garrison Reaches, respectively. These dominant discharge values are identical under the CWCP and PA.

Estimates of the possible divergent effects of the PA from the extrapolated trends of bed scour and fluvial bank erosion are based on alterations to the annual sediment load of the river. If perimeter erosion due to changes in sediment load is distributed uniformly along the study reaches, then estimated resulting increases in adjustment rates are negligible over a 50-year period.

Long-term changes in bank stability with respect to mass failure with the CWCP and PA are predicted using the Darby-Thorne bank stability model. Simulations are based on estimating the future values of bank hydrological parameters and bank geometry parameters under the two flow regimes. Bank geometry parameters 1, 5, 10, 20, and 50 years from September 1995 were obtained using the measured bank profiles deformed by cumulative amounts of bed scour and fluvial bank erosion appropriate for the flow regime. By the year 2045, the total length of unstable bank line in the study reaches is predicted to be approximately 56 to 67 percent.

The upper Missouri River has been regulated for the past 60 years. The channel is continuing to respond to the imposed flow and sediment regimes through erosion and deposition. Historical data indicate that rates of morphological adjustments through bed scour and fluvial bank erosion are decreasing with time. Bank instability with respect to mass failure will increase somewhat during the next 50 years due to cumulative effects of bed scour and toe erosion. Implementation of the PA will have no discernible effect on any of these ongoing channel adjustments, compared with those predicted to continue with the CWCP.

On the basis of reconnaissance-level morphological field and modeling analyses performed in this study, it has been concluded that about half of the bank lines along the study reaches of the upper Missouri River currently (1995) exhibit evidence of mass instability. Historical trends of channel adjustment indicate that the channel is approaching a condition of dynamic equilibrium, and on this basis it is unlikely that rates and extent of bank line retreat under the existing flow regime will increase significantly in the short term. The modeling studies indicate a small increase in the extent of bank line instability with respect to mass failure, but this is within the range of uncertainty for a study of this type. On balance, the results of morphological projections and bank stability modeling for the PA flow regime suggest that the impacts on bed scour, fluvial bank erosion, and bank stability with respect to mass failure will be indiscernible from those of the CWCP flow regime.

8 Impacts of Possible Future Bank Stabilization Measures

Introduction

Another task in this study addressed the impacts of additional bank stabilization on various attributes such as bank erosion and islands, sandbars, chute fills, and channel border fills. This task assumed that an additional zero, 10, 20, 30, and 40 percent of the bank lines would be stabilized.

To determine the historical role of bank protection in reducing bank scour, the total bank protection installed and the dates of installation were computed based on Tables 1 through 4. The following tabulation summarizes the various types of bank protections installed in the four Missouri River reaches:

Bank Protection miles	Year Installed	Total River Miles Both Banks	Bank Line Protected percent
Fort Peck Reach			
1.6	1985	345.46	0.4
Garrison Reach			
7.9	1966 through 1975	107.98	7.3
14.9	1976 through 1981		13.8
20.4	1981 to Present		18.9
		Total	40.0
Fort Randall Reach			
6.5	1978 through 1982	71.96	9.0
Gavins Point Reach			
5.7	1955 w/project	115.84	4.9
25.9	1978 through 1982		22.4
		Total	27.3

While not exact, it is relatively accurate to say that the Fort Peck Reach has zero percent bank protection, the Fort Randall Reach has about 10 percent of the banks protected, the Garrison Reach has about 40 percent of the banks protected, and the Gavins Point Reach has about 30 percent of the banks protected. Therefore, these four reaches will be analyzed individually to address the potential impacts of future additional stabilization.

Overall Documented Response in the Study Reaches

Fort Peck Reach

Impacts of historic flows on islands, sandbars, chute fills, and border and channel border fills were analyzed over the time frame from 1974 to 1990, as presented in Chapter 4. This period would have included construction of the Fort Peck Reach revetment, but as stated previously, with the limited amount of revetment constructed, zero percent of the bank stabilization was assumed to be in place.

Analysis of the scour and aggradation of channel bed and banks, relative to the specific flow conditions as they applied to the periods analyzed, was presented in Chapter 5. On the Fort Peck Reach, the overall time frame was 1955-1978 with an intermediate survey in 1966. Again, the one revetment in the reach was constructed in 1985.

Plate 87 summarizes all of the changes in hydraulic, channel geometry, and channel attributes, with most of these changes presented in percentages rather than absolute values. It demonstrates that less than CWCP or PA averaged annual flows (for the 96-year period evaluated for the DEIS (USAEDNMRR 1994a)) produced channel bed filling and bank erosion in the reach. When average annual flows were greater than those for the CWCP or PA, channel bed scour (versus filling) occurred and less bank erosion took place. Continuation of average annual flows greater than those of the CWCP or PA resulted in increased island and sandbar densities. Total area of chute filling increased while channel border fills decreased; however, part of the channel border fills decrease resulted from some fills being attached to bank lines and becoming a new riverbank. These channel bed and bank responses are only for the five segments included in the Darrel Dangberg and Associates and River Pros report[1]. When the entire reach was considered for this present study, there was significantly less bank erosion in the 1966-1978 time frame than in the 1955-1966 time frame. At the same time, the channel bed filling and scouring trends were maintained through these two periods.

[1] Op. cit.

For the entire survey analysis period, the yearly flow classification determined in Chapter 2 is also presented in Plate 87. The flow classifications indicate that during 1955-1966 all but 2 years were classified as low or medium flow years with all those flows at the beginning of the period. From 1966 to 1978 there were no low flow years and only two medium flow years with the remaining 10 years either high or very high flow years.

Garrison Reach

Chapter 4 presents historic trends of changes to islands, sandbars, chute fills, and channel border fills over the time frame from 1976 to 1990. This period included the second increment of Garrison Reach bank protection between 1976 and 1980 but not the bank protection constructed between 1966 and 1975. It should be noted that between 1981 and 1990 most of the bank protection was constructed by landowners or other local interests. Chapter 5 presents analysis results for the scour and aggradation of channel bed and banks for the overall time frame of 1956-1985, with an intermediate survey in 1976.

Plate 88 summarizes the geomorphic changes with most of the changes presented in percentages rather than absolute values. It demonstrates that, over the entire period, the average flows were greater than the longer term CWCP or PA flows. With all the periods of averaged flows being greater than the longer term CWCP or PA flows, island densities increased, sandbar densities decreased very slightly (the decrease being attributed to sandbars that were converted to channel border fills), and total areas of chute filling and channel border fills increased from 1976 to 1990. In general, the higher flow conditions tended to reduce channel bed and bank scour and increase the density or sizes of the channel attributes addressed in the analysis.

For the first survey between 1956 and 1976, the flow classifications indicate that the first part of that period had basically only low flow conditions and ended with mainly high and very high flow conditions. The second survey period, 1976 through 1985, had no low flow conditions, but either medium or very high flow conditions.

Fort Randall Reach

The impacts of historic flows on islands, sandbars, chute fills, and channel border fills were analyzed over the time frame from 1976 to 1994 (Chapter 4). Fort Randall Reach bank protection was completed between 1978 and 1982. In Chapter 5, the scour and aggradation of channel bed and banks were analyzed relative to the specific flow conditions as they applied to the periods analyzed. On the Fort Randall Reach, the overall time frame for that analysis was 1954-1985, with an intermediate survey in 1975.

Plate 89 summarizes these geomorphic changes and shows that in the period when the averaged flows were greater than the longer term CWCP or PA flows (1975-1985), the bank erosion decreased significantly while the bed scour decreased slightly. In a period with averaged flows less than the longer term CWCP or PA flows, island and sandbar densities and areas of chute and channel border fills decreased. The most significant reductions occurred in the sandbar density and chute fills. In general, the higher flow conditions tended to reduce bank scour erosion significantly and reduce channel bed scour slightly while the lower flow conditions tended to decrease the density or sizes of the channel attributes.

The flow classifications between 1954 and 1966 showed that the period began with all low medium flow conditions. Near the end of the period there were five very high flow conditions mixed with a few medium flow conditions. The second period, 1975 through 1985, started with very high flows and ended with seven consecutive medium flow conditions.

Gavins Point Reach

The impacts on islands, sandbars, chute fills, and channel border fills were analyzed over the time frame from 1981 to 1994 (Chapter 4). The second increment of the Gavins Point Reach bank protection was completed between 1978 and 1982, with the first increment completed with the dam project around 1955. Therefore, the response of those attributes would have included any impacts from the total protection plan with approximately 80 percent of the total bank protection completed between 1978 and 1982. The scour and aggradation of channel bed and banks were analyzed (Chapter 5) relative to the specific flow conditions as they applied to the periods analyzed. On the Gavins Point Reach, the overall time frame for that analysis was 1960-1986 with an intermediate survey in 1974.

Plate 90, which summarizes this information, shows that when the averaged historic flows were greater than the longer term CWCP or PA flows (1974-1986), the bank erosion decreased significantly while the bed scour decreased slightly. The volume of bank erosion decreased significantly from 1974 to 1986 compared with the period between 1960 and 1974. Because the later time frame continued to have a significant erosion volume of over 32,000 acre-ft, the process of bank erosion continued at a higher rate than any of the other three study reaches. In a period with averaged flows less than (about 90 percent total volume) the longer term CWCP or PA flows, island and sandbar densities decreased and chute and channel border fills also decreased. As was the case in the Fort Randall Reach, the most significant reductions occurred in the sandbar density and chute fills. In general, it can be concluded that the higher flow conditions tended to reduce bank scour erosion significantly and reduce channel bed scour slightly while the lower flow conditions tended to decrease the density or sizes of the channel attributes.

The first survey period, 1960 through 1974, started with six consecutive low flow conditions followed by 3 years of medium flow. As was the case in the

Fort Randall Reach, the first survey period ended with a mixture of very high and medium flows. The survey period between 1974 and 1986 started with very high and medium flows and ended with mainly medium flows, with one low flow in about the middle of the period.

Summary for the four study reaches

General conclusions reached from reviewing the data for the Fort Peck, Garrison, Fort Randall, and Gavins Point Reaches are as follows:

a. For all reaches with flow conditions greater than the 96-year average, there were less bank erosion, greater island and sandbar densities, greater chute filling, and an increase in channel border sizes.

b. For flow conditions greater than the 96-year average that extended for a long period of time (Fort Peck and Garrison Reaches), the island densities were greater.

c. Flow conditions greater than the 96-year average caused less channel bed scour, excluding the Fort Peck Reach (because the dam has been in place for a significantly longer time).

d. Flow conditions less than the 96-year average produced lower island and sandbar densities and less chute filling.

It should be noted that bed and bank erosion in the study reaches is impacted by numerous complex and interrelated variables that cannot be overlooked. One explanation for the finding that higher average monthly flows correlated with observed reduced bed and bank erosion is that as average monthly flows are increased, the flexibility and magnitude of hydropower plant peaking capability are decreased. When average monthly releases approach hydropower plant capacities, the plants are flat loaded, i.e., not used as peaking plants, which significantly reduces or eliminates the downstream daily fluctuations in river stages. Conversely, the drawdown concept that occurs when high flows are suddenly reduced to lower flow conditions, such as when hydropower plant production is reduced, results in bank instability because of the higher differential head associated with the groundwater and river stage. That drawdown issue was addressed in Chapter 7. The main point to be realized here is that this study is addressing the long-term, cumulative impacts and that the daily or even hourly variations are included in the 96-year averages within the CWCP or PA. Therefore, such variations are included in the analysis by inference, if not directly addressed.

Approach

To evaluate the impacts of potential additional bank protection in the four study reaches, the present bank erosion rates had to be determined. However, as stated several times previously in this report, the amount of past bank erosion is a result of more than just the operating plan. It is obvious that low and/or high flows have an impact that cannot be isolated from the operating plan. In fact the operating plan is intertwined within the annual hydrographs that have occurred on the four Missouri River study reaches. However, if the assumption is made, generally speaking, that bank protection will have the greatest impact on the volume of material that can be scoured from the riverbanks and other factors such as impacts on channel bed would be relatively minor, then some average or projected values can be developed from the data. The additional assumption made is that if the dams were operated under a different operating plan than the CWCP, flow conditions greater or less than the long-term 96-year average will have the same impact in the future as they have had in the past.

It must be noted that in Chapter 3 the comparison of the average velocities for the CWCP and PA showed that the changes were fairly insignificant (Plates 14-18). Also in Chapter 7 the annual total bed material load computed for the Fork Peck and Garrison Reaches for the CWCP and PA indicated little or no change in volumes of material in movement (Table 33). These conclusions imply that, since velocities will not change significantly and the ability of the stream to carry bed material load will remain constant, bank erosion should remain relatively constant also. Remember that the bank erosion that has or has not occurred has been analyzed over relatively long periods of time and is a result of a wide range of flow conditions or classifications.

The volumes of bank material scoured between the various survey periods (Tables 15-18) were used to compute an average annual bank volume in acre-ft/year and compared to the percentage of bank stabilization in place between the surveys. The results of that analysis are presented in Table 38. The following analysis is based on the computations in Table 38.

Background on the Projection of Potential Effectiveness of Additional Stabilization Measures

Limited data exist from which to make projections on the effectiveness of additional stabilization measures in the four study reaches. Table 38 presents the data on the effectiveness of existing measures, and one can readily see that there are no data for the Fort Peck Reach and only two data points for each of the other three reaches. In lieu of making no effort to address this study objective, two methodologies were implemented, and the results of this effort will be presented in this part of the report.

Data on the effectiveness of the existing measures are presented in Table 38 and shown in Plates 91-94. Examination of the four plates provides some insight on the effectiveness of the absence of measures in the Fort Peck Reach and the existing measures in the Garrison, Fort Randall, and Gavins Point Reaches. Essentially no stabilization measures have been installed in the Fort Peck Reach; yet there has been a 47 percent reduction in the erosion rate. This may be an indication of the effectiveness of attaining equilibrium in this reach. The trends shown for the Garrison and Fort Randall Reaches indicate that there were significant reductions in the erosion rate for relatively low percentages of stabilization. In the Garrison Reach, an increase of 14 percent stabilization resulted in a 46 percent reduction of the erosion rate. In the Fort Randall Reach, installation of measures in 9 percent of the reach resulted in a 52 percent reduction in the erosion rate. Not all of the reduction may be attributed to the measures themselves as the data in the Fort Peck Reach demonstrate. Effectiveness of measures in the Gavins Point Reach portrays a completely different picture. Stabilization of an additional 22 percent of this reach resulted in only a 19 percent reduction in the erosion rate. This relative ineffectiveness is likely an indication of the impacts of not being near equilibrium in this reach. The Gavins Point Reach has had the least time to react to the dam closure (Table 4), and the channel is also reacting to the navigation channel realignment and improvements downstream (Chapter 5). Analysis of the data and of several other factors resulted in one methodology being implemented in the Fort Peck, Garrison, and Fort Randall Reaches and a second methodology for the Gavins Point Reach. Each of these four reaches will be discussed individually to provide the background on the unique factors in each reach and to present the results of the projections.

Projection of Results

Fort Peck Reach

In the Fort Peck Reach, no stabilization was present during the first survey period, 1955-1966, and none was added during the second survey period, 1966-1978. However, the average annual volume of bank material scoured decreased from 1,639 acre-ft/year to 870 acre-ft/year. As previously stated, this could be an indicator that a reach approaching equilibrium may also experience a reduced rate of bank erosion. This phenomenon would likely continue to play a role in the future with or without the construction of stabilization measures in this river reach. Areas experiencing the worst erosion are the most likely candidates for stabilization, and the stabilization of those reaches will have a relatively high impact on the overall erosion rate in the total reach. This stabilization combined with the reduction that may occur as equilibrium continues to set in leads to the conclusion that the rate of reduction will be most significant for the initial stabilization efforts and that the rate will diminish with time. Thus some sort of exponential relationship can be assumed for the construction of additional stabilization in the Fort Peck Reach. Two other factors were considered as the projection was developed for this reach. First, elimination (100 percent reduction) of bank erosion will not occur until the reach is 100 percent stabilized. This assumption does not affect the projection significantly as the rate of return for this end of the projection per each additional 10 percent reduction is relatively small. Second, the value of 90 percent reduction in the erosion rate with 60 percent stabilization was selected as one point on the curve. This may be a little low as the analysis that Darby and Thorne[1] conducted for this reach for this study determined that 57 percent of the banks in the Fort Peck Reach are currently exhibiting evidence of bank instability and mass wasting and none of this reach is presently stabilized. Again, selection of another point for the development of the projection would not have a significant impact on the projection for the reason just described: the curve does not change much once the higher levels of stabilization for an exponential relationship are reached.

Plate 91 shows the projection based on these assumptions. The curve was visually fit but is fairly accurate based on what the assumptions would require. Based on the curve in Plate 91, the following reductions in the erosion rates would occur with 10, 20, 30, 40, and 50 percent stabilization of the Fort Peck Reach:

Projection of the Effects of Bank Stabilization Measures in the Fort Peck Reach

Percent Stabilization	Resulting Erosion Rate acre-ft/year	Percent Erosion Reduction
0	870	0
10	490	44
20	310	64
30	200	77
40	140	84
50	110	87

The rate of return for each additional 10 percent diminishes to less than 10 percent after the first 30 percent stabilization, and the reduction is only 7 and 3 percent for 40 and 50 percent stabilization, respectively.

Garrison Reach

Significant stabilization measures have already been constructed in the Garrison Reach. In the period following the stabilization of 7 percent of the reach, the erosion rate was 1,422 acre-ft/year. This rate was reduced to 772 acre-ft/year after the stabilization of another 14 percent (for a total of 21 percent) of the 79-mile reach. The reduction in the erosion rate is assumed to be directly related

[1] S. E. Darby and C. R. Thorne. (1996). "Bank stability analysis for the Upper Missouri River," prepared by University of Nottingham, Nottingham, England, for U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

to the stabilization measures and, potentially, the movement toward equilibrium of the channel in this reach.

An exponential relationship was also assumed to apply to this reach. The need to stabilize 100 percent of the reach is based on the fact that high flows can create erosional problems to all of the riverbanks; therefore, 100 percent stabilization to attain 100 erosion reduction is assumed. The attainment of 90 percent reduction in the erosion rate for 60 percent stabilization is also assumed. This latter assumption seems appropriate as Darby and Thorne[1] estimated for this study that 41 percent of the banks in the Garrison Reach currently exhibit evidence of bank instability and mass wasting with 21 percent of the banks already stabilized, and the unstable percent is projected to increase in the future with no additional stabilization (Chapter 7). Again, using another rate will not have a significant impact on the projection.

Plate 92 presents the projection made for this reach. Based on the projection back to the zero-stabilization point, the initial assumed rate with no stabilization is 2,000 acre-ft/year. Based on this curve and the initial rate of erosion of 2,000 acre-ft/year, the following reductions in the erosion rates would occur with an additional 10, 20, 30, 40, and 50 percent stabilization:

Projection of the Effects of Bank Stabilization Measures in the Garrison Reach

Percent Stabilization	Resulting Erosion Rate acre-ft/year	Percent Erosion Reduction
0	2,000	0
7	1,422	29
21	772	61
31	550	72
41	380	81
51	280	86
61	190	90
71	140	93

This projection is similar to the one for the Fort Peck Reach in that the percent reduction in the erosion rate for the next 10 percent additional stabilization drops below 10 percent at about the 30 percent stabilization point. The rates of return for an additional 20, 30, 40, and 50 percent of additional stabilization result in additional erosion rate reductions of only 9, 5, 4, and 3 percent, respectively.

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[1] S. E. Darby and C. R. Thorne. (1996). "Bank stability analysis for the Upper Missouri River," prepared by University of Nottingham, Nottingham, England, for U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Fort Randall Reach

The Fort Randall Reach also has two points on which to base some sort of projection. The erosion rate was reduced from 642 acre-ft/year to 310 acre-ft/year with the stabilization of 9 percent of the reach. Again, this reduction could be due to the gradual approach of the channel to equilibrium in addition to the stabilization measures. Because the first erosion rate was determined when there was no stabilization, the initial erosion rate is 642 acre-ft/year. The 60 percent stabilization attaining a 90 percent erosion rate reduction and the need for 100 percent stabilization to eliminate the erosion were assumed as this exponential relationship was established. As with the two previous projections, the resulting projection was visually fit.

Plate 93 presents the resulting projection for the Fort Randall Reach. Based on this curve, the following reductions in the erosion rates would occur with an additional 10, 20, 30, 40, and 50 percent stabilization:

Projection of the Effects of Bank Stabilization Measures in the Fort Randall Reach

Percent Stabilization	Resulting Erosion Rate acre-ft/year	Percent Erosion Reduction
0	642	0
9	310	52
19	185	71
29	120	81
39	100	84
49	85	87
59	70	89

As with the previous two projections, the rate of erosion reduction for an additional 10 percent stabilization drops below 10 percent once 30 percent of the reach is stabilized. The rates of return for an additional 30, 40, and 50 percent stabilization in the Fort Randall reach result in additional erosion rate reductions of only 4, 2, and 2 percent, respectively.

Gavins Point Reach

Approximately one-fourth of the Gavins Point Reach has been stabilized; however, this stabilization has not been as effective as in the other three reaches, based on the effectiveness of the most recent stabilization efforts. With 5 percent of the reach stabilized in the period from 1960 to 1974, the annual loss of bank materials averaged 3,919 acre-ft/year. An additional 22 percent stabilization in the 1974-1986 period reduced the erosion rate to 3,161 acre-ft/year, a reduction of only 19 percent. This response to stabilization does not follow the exponential-type response the Garrison and Fort Randall Reaches exhibited. In fact, extension of the observed rate of reduction would indicate that this reach would need to be stabilized more than 100 percent to attain a 100 percent reduction in the erosion rate. The erosion rates presented in Table 38 are the average over the 12-year period during which the stabilization measures were installed; therefore, the actual erosion rate at the end of the period would likely have been less than that at the beginning of the period. The effectiveness of the stabilization measures, the majority of which were installed from 1978 to 1981 (the middle of the period of analysis), was likely much more than the 19 percent would indicate. If 100 percent of the Gavins Point Reach were stabilized with the appropriate stabilization measures, bank erosion would likely be eliminated; therefore, the selected method of projection for this highly unstable reach was a straight-line projection from the most recent data point to the point where 100 percent stabilization attains 100 percent erosion reduction. The resulting projection is depicted in Plate 94, and the 10, 20, 30, 40, and 50 percent additional stabilization values are as follows:

Projection of the Effects of Bank Stabilization in the Gavins Point Reach

Percent Stabilization	Resulting Erosion Rate acre-ft/year	Percent Erosion Reduction[1]
5	3,919	N/A
27	3,161	N/A
37	2,730	13.7
47	2,295	13.7
57	1,860	13.7
67	1,430	13.7
77	995	13.7

[1] The percent reduction was computed assuming 3,161 acre-ft/year as the initial point. The remaining 73 percent divides into 7.3 sets of 10, which equates to a 13.7 percent improvement required to completely eliminate bank erosion for every 10 percent of the remainder of the reach being stabilized.

Based on the historic effectiveness of stabilization measures in the Gavins Point Reach, the projection is very optimistic. This reach is considerably different from the other three reaches, and other factors may need to change. As this reach gets somewhat closer to equilibrium, bank stabilization may be much more effective, as it has been in the other reaches.

Summary

The following summarizes the discussions and conclusions reached in this chapter:

a. Based on the analysis of the average annual bank erosion rate in the Fort Peck Reach, providing no additional bank stabilization will result in a decrease in the yearly eroded volume of bank material due to the effects of this channel in this reach moving closer to equilibrium.

b. The average annual bank material eroding is a function of the stability of the channel bed in that reach. The more stable the bed, the more dependable the projections on the impacts of additional bank protection.

c. For the more stable reaches, an exponential projection of the effectiveness of bank stabilization measures seems to be appropriate. The two primary factors supporting this conclusion are as follows: (1) stability is an indicator of the approach of channel equilibrium, which assists with bank stability and reduced bank erosion (supported by Fort Peck Reach data), and (2) the most highly eroding areas are usually next to be stabilized.

d. For the stable reaches with exponential projections, once 30 percent of the reach is stabilized, the effectiveness of additional stabilization is significantly reduced. In the Fort Peck, Garrison, and Fort Randall Reaches, stabilization of an additional 10 percent of the reach beyond the 30 percent amount is projected to result in less than a 10 percent reduction in the erosion rate over the entire reach.

e. The channel in the Gavins Point Reach is still very unstable due to the effects of both the upstream dam and the downstream channel modifications. A straight-line projection was selected as the most appropriate for this reach.

f. Additional bank stabilization will reduce bank material eroded from the protected area. Based on the system variables, such as annual flows, this stabilization will not have any long-term impact on the other channel processes.

9 Results and Conclusions

Study Tasks

This study was undertaken to address various tasks and issues that MRR needed to identify the impacts of changing the operation of the Fort Peck, Garrison, Fort Randall, and Gavins Point Dams from the CWCP to the PA. The following tasks were accomplished within this study:

a. Existing data and prepared time lines for closure and filling of reservoirs, construction of bank protection, and operation schedules (discharges) were reviewed.

b. Existing data on rates of bank erosion, degradation, aggradation, and channel geometry changes were reviewed.

c. Aerial photographs to document the movement and/or size of vegetated islands, sandbars, backwater/chute habitat, and bank lines were analyzed.

d. The relationship between sandbar areal exposure and discharge was identified.

e. The Darby-Thorne bank erosion algorithm was used to assess the relative potential for bank erosion with the CWCP and PA operating plans.

f. Where possible, turbidity trends were identified.

g. Where possible, a sediment budget for a reach to establish the relative importance of bank erosion and other factors was developed.

h. Bank and bed erosion, island and sandbar formation or movement, turbidity, channel geometry (area, width, and depth), and backwater or chute habitat were correlated.

i. Future trends with and without the PA were predicted for various channel parameters and attributes.

j. Impacts were projected for various levels of increased bank stabilization within at least one of the four reaches.

A primary objective was to determine the impacts of changing the operation of the dams from the CWCP to the PA. Achieving this objective for specific items required that several tasks be completed; then the results of those analyses had to be integrated to provide a final conclusion. One of the most complex factors in the analyses was that virtually all data developed included many factors beyond the CWCP or the PA. There were years where the annual flow was greater or less than the 96-year average represented by the CWCP and PA; there were limitations in the timing of the data, such as survey or aerial photographs, that restricted analysis periods; and other data such as turbidity and suspended sediment data were somewhat limited. Relative to bank protection measures, there was concern as to whether the protection had been in place long enough in some reaches to evaluate the impacts. Nevertheless, a sincere effort was made to separate the independent variables as much as possible to accomplish these tasks and complete the study objective.

Throughout this report, results and conclusions have been presented. This chapter reiterates those results and conclusions previously provided. They are presented in the order that the topics were presented in the main body of the report and with headings as a reference for the reader.

Flow Classification Time Lines

The following tabulation summarizes the time lines for the four study reaches resulting in flow classifications:

	Flow Condition	
Reach	**Low**	**Very High**
Fort Peck	1956, 1957, 1958, 1959, 1960, 1962, 1963, 1964, 1986, 1987, and 1990	1965, 1966, 1969, 1970, 1971, 1972, 1975, 1976, 1978, 1979, 1981, and 1982
Garrison	1956, 1957, 1959, 1960, 1961, 1962, 1963, 1966, 1988, and 1990	1965, 1967, 1969, 1970, 1972, 1975, 1976, 1978, 1979, and 1982
Fort Randall	1954, 1956, 1957, 1958, 1959, 1960, 1961, 1962, 1963, 1964, 1965, 1989, 1990, 1991, 1992, and 1993	1969, 1970, 1971, 1972, 1975, 1976, and 1978
Gavins Point	1960, 1961, 1962, 1963, 1964, 1965, 1981, 1989, 1990, 1991, 1992, and 1993	1969, 1970, 1971, 1972, 1975, 1976, 1978, 1979, and 1986

Current water control plan

The average monthly releases (Plates 2-5) in the CWCP in the Fort Peck Reach have discharges increased in the winter and fall; in the Garrison Reach discharges are increased in the fall; in the Fort Randall Reach discharges are increased in the summer and fall; and in the Gavins Point Reach discharges are increased in the summer and fall in support of the downstream navigation project.

Preferred alternative

The flows in the PA in the Fort Peck Reach have increased discharges in the winter and early summer; on the Garrison Reach discharges are increased in the spring and summer; on the Fort Randall Reach discharges are increased in the spring and early fall; and in the Gavins Point Reach discharges are increased in the spring and early fall.

Annual Water Volumes

The following conclusions were reached for the water volumes analyzed over various periods during this study:

a. *The four reaches*. The total volume of water passed downstream for the CWCP and PA are essentially equal.

b. *Fort Peck Reach*. Relative to the 96-year average, the time frames for 1955-1978, 1974-1990, and 1967-1978 would be classified as high water periods and 1955-1966 as a low water period.

c. *Garrison Reach*. Relative to the 96-year average, all four periods, 1956-1985, 1976-1990, 1956-1976, and 1977-1985, would be classified as high water conditions.

d. *Fort Randall Reach*. Relative to the 96-year average, the time frames between 1954 and 1985, 1976 and 1994, and 1954 and 1974 would be classified as low water periods and 1975-1985 would be classified as a high water period.

e. *Gavins Point Reach*. Relative to the 96-year average, the time frame from 1960 to 1986 would be classified as a relatively average period. However, the period of 1960-1974 would be classified as a low water period, and 1975-1986 as a high water period. The time frame from 1981 to 1994 would be classified as a low flow period.

Average Channel Velocities

The average velocities in each reach channel were computed based on channel cross sections obtained from the USGS during their discharge measurements or from MRR sedimentation range surveys. Water-surface elevations were obtained from USGS or Omaha District data for computations of the average velocities for the CWCP and PA. It should be noted that the CWCP and PA are based on monthly averages over the 96-year period from 1898 to 1995.

a. *Fort Peck Reach*. The CWCP and PA velocities were essentially identical except for the maximums, which differed about 0.75 fps. The computed maximums had a magnitude of approximately 5.0 fps; therefore, the difference was relatively insignificant.

b. *Garrison Reach*. The PA average velocities were lower than the CWCP average velocities to about the 50 percent exceedance point (Plate 15). From there the PA average velocities were higher than the CWCP average velocities, but the maximum deviation between the two plans was about 0.1 fps.

c. *Fort Randall Reach*. The average CWCP and PA velocities were nearly identical with the largest variance about 0.12 fps.

d. *Gavins Point Reach*. The maximum difference between the CWCP and PA was about 0.25 fps; however, the majority of the velocities from the two plans were virtually identical.

e. *Velocity magnitudes*. Considering all four reaches and based on the average velocity computations for the range of flows in the CWCP and PA for the entire year, the differences in the magnitude of velocities between the two plans are insignificant.

f. *Variations*. There is significantly greater variation between actual annual hydrographic events than the variations between the CWCP and PA.

Islands and Sandbars Relative to Bed and Bank Scour

As presented previously, the following conclusions were developed concerning channel bed and bank scour and impacts on island and sandbars:

a. *Fort Peck Reach*. There appeared to be very little relationship between the scour of the channel bed or banks and the change in island or sandbar densities, but there is a relationship between sandbar density and channel bed aggradation. A cutoff occurred between RM 1,603.0 and 1,599.0

that removed islands from the main river channel. The islands and sandbars appeared to be in a relatively stable condition during the period 1974-1990.

b. *Garrison Reach*. The islands and sandbars appeared to be relatively stable during the period 1976-1990, with the density of sandbars showing a slight tendency to decrease in the upper segments of the reach. The island densities increased in segments that had degradation and in other segments that had aggradation; there is no apparent relationship between the scouring of channel bed and changes in island densities. There is a general relationship between bank scour and change in island densities since several ranges of the upstream segment of the reach experienced bank erosion and the island densities in the middle and downstream segments increased.

c. *Fort Randall Reach*. The islands in the upstream portion of the reach appeared to be in a relatively stable condition during the period 1976-1994; however, the downstream portion of the reach indicated loss of island densities. There was no apparent relationship between the scouring of channel bed and changes in island densities. From 1976 to 1994 there was a definite trend over the entire reach for the density of the sandbars to decrease significantly. The analysis indicated that, with bank scour or aggradation present, the sandbar densities decreased for the entire reach.

d. *Gavins Point Reach*. Other than at the downstream end of the reach, the island density decreased from upstream to downstream with the largest decreases near the downstream end. Some sandbar loss was due to conversion to islands or channel border fill, but the majority was due to erosion of the sandbars. There was a general tendency for the sandbars to erode during the time frame. No apparent relationship between the scouring of channel bed and changes in island densities or sandbar densities is evident. There was no apparent relationship between the scouring of banks and changes in island densities and, though the bank scour increased from upstream to downstream, the majority of the sandbars tended to degrade in the reach.

e. *All four reaches*. Over periods of approximately 15 years for each reach, the islands in all the reaches were relatively stable.

f. *Morphological changes*. Morphological changes greater than those that have occurred in the past probably will not be forced on the system in the future. As indicated previously, the yearly variations in total water volumes are significantly greater than variations between the CWCP and PA. Since the CWCP is integrated within the yearly hydrographs, overall channel morphology should react in a similar manner if the PA replaces the CWCP.

Chute and Channel Border Fills

Impacts on chute and channel border fills developed during this study follow:

a. *Fort Peck Reach.* From 1974 to 1990, total chute fill area increased by 135 percent and channel border fills decreased by 40 percent. In several areas channel border fills became attached to the bank line and could no longer be visually delineated from the river bank.

b. *Garrison Reach.* From 1976 to 1990, total chute fill area increased by 20 percent and channel border fill area increased by 67 percent. Some channel border fills were converted to river bank lines and, in other segments, sandbars became channel border fills.

c. *Fort Randall Reach.* From 1976 to 1994, total chute fill area decreased by 46 percent and channel border fill area decreased by 24 percent. The reduction of all of the channel attributes in this reach indicates that adjustment of the channel is still an ongoing process.

d. *Gavins Point Reach.* From 1981 to 1994, total chute fill area decreased by 85 percent and total channel border fill area decreased by 5 percent.

Relationship of Islands and Discharge

The river discharge and its relationship to islands developed during this study are presented as follows:

a. *Fort Peck Reach.* Because the reaction of the reach islands to the flow conditions varied dramatically in magnitude and short- or long-term trends, the results indicate that enlargement or reduction of islands cannot be directly attributed solely to discharge. However, any islands that existed in 1947 continued to exist in 1990.

b. *Garrison Reach.* The reaction of the islands to the flow conditions has been that higher flow conditions tended to decrease island size and more normal flow conditions tended to increase island size. Over the period from 1976 to 1990, many more islands increased in size than decreased, and any islands that existed in 1956 as an island or a sandbar continued to exist in 1990.

c. *Fort Randall Reach.* The reaction of the reach islands to the flow conditions has been that higher flow conditions tended to increase island size and more normal to low flow conditions tended to decrease island size. Any islands that existed in 1975 continued to exist in 1994.

d. *Gavins Point Reach.* The reaction of the islands to the flow conditions has been that higher flow conditions tended to increase island size and low flow conditions tended to decrease island size. As in the previous reaches, any islands that existed in 1956 as an island or a sandbar continued to exist in 1994.

Banks and Channel Bed Erosion

Using the channel cross sections, volumes of scour were computed and linked to the storage loss in the reservoirs downstream. The following conclusions were developed during this study:

a. *Fort Peck Reach.* For the five segments analyzed in the April 1988 Darrel Dangberg and Associates and River Pros report,[1] from 1955 to 1978 the volume of material scoured from the banks was significantly greater than from the bed, and the erosion of the banks has decreased over time. In the time frame of this 1988 study, the channel bed filled (aggraded) during the first 11 years and then scoured the next 12 years. From 1933 to 1983, for the five segments, a volume of 35,661 acre-ft was scoured from the banks, and for the entire reach from 1933 to 1983 a volume of 75,342 acre-ft was computed. The end-area method from the current study produced a volume computation for bank scour between 1955 and 1978 of 28,464 acre-ft. Based on a bank erosion loss of 75,342 acre-ft, approximately 10 percent of the storage capacity lost in Lake Sakakawea behind Garrison Dam came from the banks in the Fort Peck Reach.

b. *Garrison Reach.* Since the project started power generation, the volume of material scoured from the bed has been slightly greater than from the banks, and erosion of the banks and bed has decreased significantly over time. The total accumulated bank erosion volume from 1956 to 1985 was 35,389 acre-ft from the banks. The total storage loss in Oahe Reservoir, which is downstream of the Garrison Reach, was 614,000 acre-ft as of the 1989 survey. The volume computations using the 1955-1985 period for the banks and 1956-1985 period for the bed indicate that approximately 6 percent of that material in Lake Oahe came from the banks and another 7 percent from the channel bed.

c. *Fort Randall Reach.* Since 1954, the volume of material scoured from the bed and banks has been about equal, and erosion of the banks and bed has decreased significantly over time. The total accumulated bank erosion volume from 1954 to 1985 was 16,572 acre-ft. The total lost storage in Clark Lake behind Gavins Point Dam, which is downstream of the Fort Randall Reach, was 83,000 acre-ft as of the 1985 survey. The

[1] Op. cit.

volume computations indicate that approximately 19 percent of that material came from the banks.

d. Gavins Point Reach. The Gavins Point Reach has no reservoir downstream of the reach; however, bank and bed scour volumes were computed to provide some indication of overall trends on the Missouri River. Since the project started power generation in about 1956, the volume of material scoured from the banks has been greater than from the channel bed and the erosion of the banks has decreased over time. The total accumulated bank erosion volume from 1960 to 1986 was 92,799 acre-ft.

e. Bank scour. There is a definite trend for the rate of bank erosion in all four reaches to decrease. The volumes computed in the second period were less than in the first period for all four reaches, and cumulative erosion curves for the Fort Peck, Garrison, and Fort Randall Reaches show a flattening. Based on the volumes computed for the Gavins Point Reach, it appears that the stability of the banks there is lagging somewhat behind the other three reaches. The four reaches discussed are probably not headed to a situation where all the banks are stable and no erosion is occurring. Rather the volume of bank material being eroded will equal the volume of riverbed material added to the banks in another location, and the cumulative erosion of the banks will be fairly constant.

f. Channel bed scour. Based on the volumes of material scoured from the bed, the Fort Peck Reach actually filled (aggraded) during the first period and scoured during the second period. This indicates the potential trend toward bed stabilization. The Garrison Reach appears to be headed toward some degree of equilibrium since the volume of bed scour reduced significantly during the second period. At the same time the Fort Randall and Gavins Point Reaches appear to be still in the adjustment phase. Of the four reaches studied, the Gavins Point Reach continues to be the most active and is probably the furthest from reaching adjustment. This reach is responding to the development and stabilization of the downstream navigation channel and lacks a base level as constant as the other three study reaches.

Turbidity Relationships

Turbidity of the water and its potential relationship to various parameters are of interest to some of the resource agencies dealing with the Missouri River. The following conclusions were developed during this study:

a. On the Heart River at Mandan, ND, there were good correlations between turbidity and water discharge and suspended sediment concentrations and water discharge. There was also a seasonality to turbidity on the Heart River with turbidity highest in the spring and early summer.

b. On the Missouri River at Bismarck, ND, and Sioux City, IA, turbidity was seasonal, tending to be the highest in the spring and early summer.

c. At Bismarck and Sioux City, there was no correlation between turbidity and water discharge or between suspended sediment concentrations and water discharge.

d. There was no correlation between discharge from the Knife River, an upstream tributary, and the turbidity at Bismarck.

e. At Bismarck and Sioux City, any other variables compared resulted in some slight or low correlation, but none significant enough to use as a predictive tool.

f. Since there was a correlation between turbidity and water discharge on the Heart River and none on the Missouri River at Bismarck or Sioux City, it implies that perhaps such correlations are possible only on smaller watersheds.

g. Since the analysis indicated no relationship between turbidity and the discharges in the Missouri River, it is highly unlikely that changing operation from the CWCP to the PA will affect turbidity.

h. Since there is a trend toward seasonality of turbidity on the Missouri River, the higher spring flows, particularly in the Garrison, Fort Randall, and Gavins Point Reaches, will produce higher discharges without an increase in the turbidity.

i. Annual sediment yields computed at Bismarck during the period from 1972 to 1980 indicate that an 8 percent decrease in water volume resulted in a 33 percent reduction in sediment. Over the period from 1972 to 1980 annual sediment yields varied between 1.9 and 12.6 million tons.

j. At Bismarck there were good correlations between turbidity and dissolved solids and dissolved solids and concentration of fine sediments. The significance of these relationships do not appear to be relative to this study.

Bank Stability

Short- and long-term bank stability is of major interest in evaluation of potential impacts of the PA versus the CWCP. The following conclusions relative to bank stability were developed during this study:

a. Field measurements of geotechnical characteristics indicate that the bank material properties along the two study reaches, Fort Peck and Garrison, are relatively uniform. Bank materials are weakly cohesive sandy silts.

Planar failure due to toe scour and oversteepening by fluvial bank erosion is the most common mechanism of collapse in both study reaches.

b. Excess bank pore-water pressures and hydrostatic confining pressures generated under the PA were found to be indiscernible from those under the CWCP. Hence, short-term (less than 5 years) impacts on bank stability with respect to mass failure are predicted to be negligible.

c. Amounts of bed scour after 50 years of channel adjustment ranged between 0.2 ft of bed deposition and 2.5 ft of scour for the 13 Fort Peck study sites. Bed scour in the Garrison Reach (study sites 14-18) after 50 years ranged between 0.7 and 2.6 ft of scour. The rate of bed scour over the 50-year period averaged 0.004 ft/year and 0.03 ft/year in the Fort Peck and Garrison Reaches, respectively. Bank erosion in the 13 Fort Peck sites ranged between zero and 20.6 ft over the 50 years, while in the 5 Garrison sites bank erosion ranged between zero and 29.7 ft. The rate of bank scour over the 50-year period averaged 0.09 ft/year and 0.20 ft/year for the Fort Peck and Garrison Reaches, respectively. Mean rates of bed scour and bank erosion are low, indicating that the channel is at, or approaching, a condition of dynamic equilibrium. At some specific study sites (sites 8, 10, 11, 14, and 16), fluvial bank erosion rates are higher due to local conditions. There are also some study sites (sites 14 to 17) downstream of Garrison Dam that are predicted to experience higher rates of bed scour. This may indicate continued adjustment of the bed downstream of the Garrison Dam.

d. The dominant discharge is found to be about 7,000 cfs and 18,500 cfs in the Fort Peck and Garrison Reaches, respectively. These dominant discharge values are identical under the CWCP and PA.

e. Analysis of the sediment regime of the study reaches with the CWCP and PA suggests that the annual suspended bed material load will remain about the same for the Fort Peck and Garrison Reaches.

f. Stream reconnaissance suggests that at the present time 57 percent and 41 percent of the banks in the Fort Peck and Garrison Reaches, respectively, exhibit evidence of bank instability and mass-wasting.

g. Historical data indicate that rates of morphological adjustments through bed scour and fluvial bank erosion are decreasing with time. Bank instability with respect to mass failure will increase somewhat during the next 50 years due to cumulative effects of bed scour and toe erosion.

h. Long-term changes in bank stability with respect to mass failure with the CWCP and PA using the Darby-Thorne bank stability model based on bank geometry parameters for 1, 5, 10, 20, and 50 years into the future were obtained. The model results indicated that, by the year 2045, the total length of unstable bank line in the two study reaches is predicted to be approximately 56-67 percent.

i. On the evidence of the reconnaissance study and the results of this analysis, implementation of the PA will have no discernible effect on any of these ongoing channel adjustments, compared with those predicted to continue with the CWCP.

Additional Bank Stabilization

The following conclusions relative to potential impacts of adding bank stabilization measures to the reaches were developed during this study:

a. Based on the analysis of the average annual bank erosion rate in the Fort Peck Reach, providing no additional bank stabilization will result in a decrease in the yearly eroded volume of bank material due to the effects of this channel in this reach moving closer to equilibrium.

b. The average annual bank material eroding is a function of the stability of the channel bed in that reach. The more stable the bed, the more dependable the projections on the impacts of additional bank protection.

c. For the more stable reaches, an exponential projection of the effectiveness of bank stabilization measures seems to be appropriate. The two primary factors supporting this conclusion are as follows: (1) stability is an indicator of the approach of channel equilibrium, which assists with bank stability and reduced bank erosion (supported by Fort Peck Reach data), and (2) the most highly eroding areas are usually next to be stabilized.

d. For the stable reaches with exponential projections, once 30 percent of the reach is stabilized, the effectiveness of additional stabilization is significantly reduced. In the Fort Peck, Garrison, and Fort Randall Reaches, stabilization of an additional 10 percent of the reach beyond the 30 percent amount is projected to result in less than a 10 percent reduction in the erosion rate over the entire reach.

e. The channel in the Gavins Point Reach is still very unstable due to the effects of both the upstream dam and the downstream channel modifications. A straight-line projection was selected as the most appropriate projection to make for this reach.

f. Additional bank stabilization will reduce bank material eroded from the protected area. Based on the system variables, such as annual flows, this stabilization will not have any long-term impact on the other channel processes.

General Conclusions for the Four Study Reaches

After a review of many of the specific conclusions developed, the following general conclusions were reached for the four study reaches:

a. For all reaches with flow conditions greater than the 96-year average, there was less bank erosion.

b. For flow conditions greater than the 96-year average, island and sandbar densities were greater.

c. For flow conditions greater than the 96-year average, chute filling and increase in channel border sizes were greater.

d. For flow conditions greater than the 96-year average that extended for a long period of time (Fort Peck and Garrison Reaches), the island densities were greater.

e. For flow conditions less than the 96-year average, the island and sandbar densities were less.

f. For flow conditions less than the 96-year average, the chute filling was less.

g. Excluding the Fort Peck Reach (because the dam has been in place for a significantly longer time), flow conditions greater than the 96-year average caused less channel bed scour.

General Conclusions of CWCP Versus the PA

Since one of the primary purposes of this study was to delineate the difference in impacts on operation of the dams with the PA instead of the CWCP, this summary is provided as a collective review of the analysis and conclusions developed:

a. The average channel velocities for the CWCP and PA are essentially identical; therefore, no significant additional bank and channel bed erosion will probably occur if the PA is adopted.

b. Annual variations in the hydrographs are significant and vary considerably from the 96-year average of the CWCP and PA.

c. Based on the annual sediment yields, the PA will move approximately the same volume of material as the CWCP. This is based on computations in the Fort Peck and Garrison Reaches, but there is no reason to assume that the results would be different in the Fort Randall or Gavins Point Reaches.

d. Changing operation from the CWCP to the PA should have no impact on the amount of turbidity in the water.

e. Based on the data available, changing operation from the CWCP to the PA should not have a significant impact on island existence or sizes.

f. Likewise, sandbars and filling of chute channels should not be significantly impacted by going to the PA.

g. In the analysis there were numerous instances where channel borders became attached to the existing banks and in fact probably became the new lower bank line. Since this is a function of the channel processes and tied to deposition of eroded bed or bank material, it seems unlikely that changing from the CWCP to the PA will significantly influence this trend.

Recommendations

In various portions of this report several recommendations were made relative to the need for additional studies or analysis in the future. Those recommendations are repeated here for completeness:

a. Based on the bank stability analysis, it is recommended that the extent and severity of bank instability with respect to mass failure continue to be monitored to identify problems should they develop.

b. It is recommended that an additional set of cross-sectional data be obtained for each reach, and an analysis similar to the one conducted for this study be performed to better quantify impacts of various degrees of bank protection.

c. To help quantify rates of bank and bed erosion as they relate to discharge, it is suggested that shorter periods, such as 2 to 4 years, with similar flow conditions (high or low flows) be used to address volumes of material scoured. Using such a method will allow the evaluation to focus on much shorter term events and provide for more accurate conclusions.

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Table 1
Time Line for Missouri River Downstream of Fort Peck Dam

Year	**Background**
1933	Construction Started
1937	Closure
1940	Project placed in operation
1943	First powerhouse unit operational
1955	**Cross sections analyzed** Discharges below Fort Peck Dam Q_{max}=31,100 cfs, Q<7K for 123 days Q>9K for 188 days, Q> 11k for 180 days Q>14K for 137 days
1956	Discharges below Fort Peck Dam Q_{max}=10,400 cfs, Q<7K for 264 days Q>9K for 56 days
1957	Discharges below Fort Peck Dam Q_{max}=7,500 cfs, Q<7K for 214 days
1958	Discharges below Fort Peck Dam Q_{max}=7,500 cfs, Q<7K for 243 days
1959	Discharges below Fort Peck Dam Q_{max}=8,100 cfs, Q<7K for 23 days
1960	Discharges below Fort Peck Dam Q_{max}=9,700 cfs, Q<7K for 106 days Q>9K for 5 days
1961	Final powerhouse unit operational Discharges below Fort Peck Dam Q_{max}=15,700 cfs, Q<7K for 16 days Q>9K for 112 days, Q>11K for 47 days
1962	Discharges below Fort Peck Dam Q_{max}=12,500 cfs, Q<7K for 207 days Q>9K for 111 days, Q>11K for 75 days
1963	Discharges below Fort Peck Dam Q_{max}=13,000 cfs, Q<7K for 269 days Q>9K for 35 days, Q>11K for 10 days
1964	Discharges below Fort Peck Dam Q_{max}=13,400 cfs, Q<7K for 235 days Q>9K for 89 days, Q>11K for 39 days
1965	Discharges below Fort Peck Dam Q_{max}=15,300 cfs, Q<7K for 6 days Q>9K for 338 days, Q>11K for 301 days Q>14K for 121 days

Note: Significance of Fort Peck discharges:
The PA minimum average monthly discharge (1898-1993) is 7,000 cfs.
The CWCP minimum average monthly discharge (1898-1993) is 7,300 cfs.
The CWCP and the PA average annual discharge (1898-1993) is 9,000 cfs.
The CWCP maximum average monthly discharge (1898-1993) is 11,000 cfs.
The PA maximum average monthly discharge (1898-1993) is 14,000 cfs.

Table 1 (Continued)	
Year	**Background**
1966	**Cross sections analyzed** Discharges below Fort Peck Dam Q_{max}=15,700 cfs, Q<7K for 109 days Q>9K for 226 days, Q>11K for 158 days Q>14K for 72 days
1967	Discharges below Fort Peck Dam Q_{max}=14,900 cfs, Q<7K for 24 days Q>9K for 322 days, Q>11K for 247 days Q>14K for 14 days
1968	Discharges below Fort Peck Dam Q_{max}=14,600 cfs, Q<7K for 82 days Q>9K for 263 days, Q>11K for 184 days Q>14K for 79 days
1969	Discharges below Fort Peck Dam Q_{max}=14,700 cfs, Q<7K for 23 days Q>9K for 317 days, Q>11K for 207 days Q>14K for 79 days
1970	Discharges below Fort Peck Dam Q_{max}=15,300 cfs, Q<7K for 32 days Q>9K for 423 days, Q>11K for 265 days Q>14K for 209 days
1971	Discharges below Fort Peck Dam Q_{max}=15,300 cfs, Q<7K for 0 days Q>9K for 282 days, Q>11K for 188 days Q>14K for 108 days
1972	Discharges below Fort Peck Dam Q_{max}=14,900 cfs, Q<7K for 0 days Q>9K for 256 days, Q>11K for 148 days Q>14K for 56 days
1973	Discharges below Fort Peck Dam Q_{max}=15,000 cfs, Q<7K for 160 days Q>9K for 120 days, Q>11K for 71 days Q>14K for 32 days
August 1974	**Aerial photographs analyzed** Discharges below Fort Peck Dam Q_{max}=13,300 cfs, Q<7K for 48 days Q>9K for 257 days, Q>11K for 107 days
1975	Discharges below Fort Peck Dam Q_{max}=35,400 cfs, Q<7K for 19 days Q>9K for 319 days, Q>11K for 269 days Q>14K for 190 days
1976	Discharges below Fort Peck Dam Q_{max}=25,500 cfs, Q<7K for 0 days Q>9K for 365 days, Q>11K for 310 days Q>14K for 258 days
1977	Discharges below Fort Peck Dam Q_{max}=15,400 cfs, Q<7K for 157 days Q>9K for 143 days, Q>11K for 66 days Q>14K for 39 days
	(Sheet 2 of 3)

Table 1 (Concluded)

Year	Background
1978	**Cross sections analyzed** Discharges below Fort Peck Dam Q_{max}=15,300 cfs, Q<7K for 20 days Q>9K for 295 days, Q>11K for 236 days Q>14K for 119 days
1979	Discharges below Fort Peck Dam Q_{max}=28,900 cfs, Q<11K for 238 days Q9K for 270 days, Q>11K for 238 days Q>14K for 121 days
1980	Discharges below Fort Peck Dam Q_{max}=14,600 cfs, Q7K for 32 days Q>9K for 245 days, Q>11K for 179 days Q> 14K for 6 days
1981	Discharges below Fort Peck Dam Q_{max}=15,000 cfs, Q<7K for 0 days Q>9K for 335 days, Q>11K for 242 days Q>14K for 59 days
1982	Discharges below Fort Peck Dam Q_{max}=15,600 cfs, Q<7K for 39 days Q>9K for 260 days, Q>11K for 174 days Q>14K for 5days
1983	Discharges below Fort Peck Dam Q_{max}=14,400 cfs, Q<7K for 107 days Q>9K for 177 days, Q>11K for 80 days Q>14K for 5 days
1984	Discharges below Fort Peck Dam Q_{max}=13,800 cfs, Q<7K for 27 days Q>9K for 273 days, Q>11K for 137 days
1985	Discharges below Fort Peck Dam Q_{max}=14,600 cfs, Q<7K for 76 days Q>9K for 232 days, Q>11K for 142 days Q>14K for 23 days
1986	Discharges below For t Peck Dam Q_{max}=14,500 cfs, Q<7K for 137 days Q>9K for 63 days, Q>11K for 60 days Q>14K for 23 days
1987	Discharges below Fort Peck Dam Q_{max}=11,400 cfs, Q<7K for 215 days Q>9K for 79 days, Q>11K for 18 days
1988	Discharges below Fort Peck Dam Q_{max}=12,200 cfs, Q<7K for 173 days Q>9K for 79 days, Q>11K for 38 days
1989	Discharges below Fort Peck Dam Q_{max}=13,400 cfs, Q<7K for 37 days Q>9K for 220 days, Q>11K for 103 days
October 1990	**Aerial photographs analyzed** Discharge below Fort Peck Dam Q_{max}=13,100 cfs, Q<7Kfor 61 days Q>9K for 61 days, Q>11K for 50 days

(Sheet 3 of 3)

Table 2
Time Line for Missouri River Downstream of Garrison Dam

Year	Background
Natural (preproject)	Bluffs/rock on RT, RM 1,389-1,386.8 Bluffs/rock on LT, RM 1,355.5-1,352.0 Bluffs/rock on RT, RM 1,348.0-1,346.0 Bluffs/rock on RT, RM 1,339.0-1,337.0
1946	Construction started
1953	Closure
1955	Project placed in operation
1956	**Cross sections analyzed** First powerhouse unit operational Discharges at Bismarck, ND Q_{max}=37,400 cfs, Q<12.5K for 108 days Q>22.5K for 152 days, Q>29K for 75 days Q>31.5K for 25 days
1957	Discharges at Bismarck, ND Q_{max}=29,100 cfs, Q<12.5K for 73 days Q>22.5K for 33 days
1958	Discharges at Bismarck, ND Q_{max}=32,400 cfs, Q<12.5K for 31 days Q>22.5K for 97 days, Q>29K for 62 days
1959	Discharges at Bismarck, ND Q_{max}=21,700 cfs, Q<12.5K for 15 days
1960	Final powerhouse unit operational Discharges at Bismarck, ND Q_{max}=33,000 cfs, Q<12.5K for 173 days Q>22.5K for 13 days, Q>29K for 3 days Q>31.5K for 1 day
1961	Discharges at Bismarck, ND Q_{max}=29,400 cfs, Q<12.5K for 67 days Q>22.5K for 49 days
1962	Discharges at Bismarck, ND Q_{max}=31,800 cfs, Q<12.5K for 145 days Q>22.5K for 112 days, Q>29K for 8 days Q>31.5K for 4 days
1963	Discharges at Bismarck, ND Q_{max}=2,100 cfs, Q<12.5K for 127 days Q>22.5K for 79 days
1964	Discharges at Bismarck, ND Q_{max}=33,100 cfs, Q<12.5K for 17 days Q>22.5K for 128 days, Q>29K for 17 days Q>31.5K for 3 days

Note: Significance of Garrison Discharges:
The PA minimum average monthly discharge (1898-1993) is about 12,500 cfs.
The CWCP minimum average monthly discharge (1898-1993) is about 18,400 cfs.
The CWCP and the PA average annual discharge (1898-1993) is 22,500 cfs.
The CWCP maximum average monthly discharge (1898-1993) is 31,500 cfs.
The PA maximum average monthly discharge (1898-1993) is 29,000 cfs.

Table 2 (Continued)	
Year	**Background**
1965	Discharges at Bismarck, ND Q_{max}=37,200 cfs, Q<12.5K for 41 days Q>22.5K for 266 days, Q>29K for 194 days Q>31.5K for 104 days
1966	Dikes on LT, RM 1,362.0-1,361.8 Revetment on RT, RM 1,361.3 -1,360.9 Dikes on RT, RM 1,360.4-1,360.2 Revetment on RT, RM 1,360.2-1,359.9 Discharges at Bismarck, ND Q_{max}=34,300 cfs, Q<12.5K for 75 days Q>22.5K for 105 days, Q>29K for 20 days Q>31.5K for 10 days
1967	Dikes on RT, RM 1,371.4-1,371.0 Discharges at Bismarck, ND Q_{max}=41,300 cfs, Q<12.5K for 31 days Q>22,5K for 271 days, Q>29K for 206 days Q>31.5K for 169 days
1968	Discharges at Bismarck, ND Q_{max}=40,100 cfs, Q<12.5K for 12 days Q>22.5K for 199 days, Q>29K for 105 days Q>31.5K for 77 days
1969	Revetment on RT, RM 1,371.9-1,371.5 Discharges at Bismarck, ND Q_{max}=49,000 cfs, Q<12.5K for 0 days Q>22.5K for 305 days, Q>29K for 206 days Q>31.5K for 93 days
1970	Revetment on RT, RM 1,364.7-1,363.4 Discharges at Bismarck, ND Q_{max}=41,100 cfs, Q<12.5K for 0 days Q>22.5K for 322 days, Q>29K for 179 days Q>31.5K for 93 days
1971	Discharges at Bismarck, ND Q_{max}=42,400 cfs, Q<12.5K for 0 days Q>22.5K for 349 days, Q>29K for 220 days Q>31.5K for 148 days
1972	Revetment on LT, RM 1,351.4-1,349.3 Discharges at Bismarck, ND Q_{max}=44,100 cfs, Q<12.5K for 0 days Q>22.5K for 304 days, Q>29K for 192 days Q>31K for 154 days
1973	Revetment with Dikes on RT, RM 1,370.7-1,370.3 Discharges at Bismarck, ND Q_{max}=31,200 cfs, Q<12.5K for 0 days Q>22.5K for 150 days, Q>29K for 23 days Q>31.5K for 22 days
1974	Discharges at Bismarck, ND Q_{max}=33,000 cfs, Q<12.5K for 1 day Q>22.5K for 323 days, Q>29K for 96 days Q>31.5K for 22 days
	(Sheet 2 of 4)

Table 2 (Continued)	
Year	**Background**
1975	Revetment on LT, RM 1,352.4-1,349.3 Discharges at Bismarck, ND Q_{max}=68,800 cfs, Q<12.5K for 0 days Q>22.5K for 325 days, Q>29K for 255 days Q>31.5K for 209 days
1976	**Cross sections analyzed**
October 1976	**Aerial photographs analyzed** Dikes on RT, RM 1,351.5, 1,350.0, 1,349.7 Discharges at Bismarck, ND Q_{max}=42,500 cfs, Q<12.5K for 0 days Q>22.5K for 348 days, Q>29K for 272 days Q>31.5K for 236 days
1977	Discharges at Bismarck, ND Q_{max}=34,000 cfs, Q<12.5K for 2 days Q>22.5K K for 100 days, Q>29K for 24 days Q>31.5K for 13 days
1978	Revetment on LT, RM 1,369.3-1,368.3 Revetment on RT, RM 1,368.6-1,365.5 Dikes on LT, RM 1,365.1 and 1,363.5 Dikes on RT, RM 1,362.8 Revetment on LT, RM 1,361.9-1,361.2 Revetment on LT, RM 1,364.7-1,362.8 Revetment on LT, RM 1,357.9-1,357.5 Hard points on LT, RM 1,356.9-1,356.8 Revetment on LT, RM 1,356.6-1,356.3 Discharges at Bismarck, ND Q_{max}=43,500 cfs, Q<12.5K for 1 day Q>22.5K for 280 days, Q>29K for 219 days Q>31.5K for 174 days
1979	Revetment on RT, RM 1,348.8-1,348.2 Hard points on LT, RM 1,348.8-1,348.1 Longitudinal dike on RT, RM 1,345.6-1,345.3 Discharges at Bismarck, ND Q_{max}=53,300 cfs, Q<12.5K for 0 days Q>22,5K for 236 days, Q>29K for 166 days Q>31.5K for 127 days
1980	Revetment on LT, RM 1,374.1-1,373.4 Revetment on LT, RM 1,350.6-1,359.3 Revetment on LT, RM 1,359.2-1,358.6 Discharges at Bismarck, ND Q_{max}=31,300 cfs, Q<12.5K for 0 days Q>22.5K for 264 days, Q>29K for 61 days

(Sheet 3 of 4)

Table 2 (Concluded)	
Year	**Background**
1981	Revetment on RT, RM 1,385.7-1,385.6 Revetment on RT, RM 1,384.0-1,384.4 Revetment on RT, RM 1,380.3-1,380.0 Hard points on RT, RM 1,379.8-1,379.7 Revetment on LT, RM 1,379.1-1,378.9 Revetment on LT, RM 1,374.4-1,374.2 Revetment on LT, RM 1,351.2-1,350.9 Revetment on LT, RM 1,345.2-1,344.4 Revetment on RT, RM 1,343.8-1,343.1 Revetment on RT, RM 1,342.3-1,340.7 Revetment on RT, RM 1,339.8-1,339.4 Revetment on LT, RM 1,338.8-1,338.2 Discharges at Bismarck, ND Q_{max}=32,500 cfs, Q<12.5K for 0 days Q>22.5K for 175 days, Q>29K for 57 days Q>31.5K for 13 days
1982	Discharges at Bismarck, ND Q_{max}=37,700 cfs, Q<12.5K for 0 days Q>22.5K for 274 days, Q>29K for 108 days Q>31.5K for 79 days
1983	Discharges at Bismarck, ND Q_{max}=40,200 cfs, Q<12.5K for 0 days Q>22.5K for 174 days, Q>29K for 54 days Q>31.5K for 38 days
1984	Discharges at Bismarck, ND Q_{max}=31,000 cfs, Q<12.5K for 0 days Q>22.5K for 233 days, Q>29K for 61 days
1985	**Cross sections analyzed** Discharges at Bismarck, ND Q_{max}=33,800 cfs, Q<12.5K for 0 days Q>22.5K for 129 days, Q>29K for 38 days Q>31.5K for 14 days
1986	Discharges at Bismarck, ND Q_{max}=34,700 cfs, Q<12.5K for 23 days Q>22.5K for 129 days, Q>29K for 38 days Q>31.5K for 20 days
1987	Discharges at Bismarck, ND Q_{max}=30,700 cfs, Q<12.5K for 29 days Q>22.5K for 88 days, Q>29K for 23 days
1988	Discharges at Bismarck, ND Q_{max}=33,000 cfs, Q<12.5K for 66 days Q>22.5K for 85 days, Q>29K for 29 days Q>31.5K for 11 days
1989	Discharges at Bismarck, ND Q_{max}=26,200 cfs, Q<12.5K for 40 days Q>22.5K for 152 days
October 1990	**Aerial photographs analyzed** Discharges at Bismarck, ND Q_{max}=27,700 cfs, Q<12.5K for 77 days Q>22.5K for 44 days

(Sheet 4 of 4)

Table 3
Time Line for Missouri River Downstream of Fort Randall Dam

Year	Background
Natural (preproject)	Bluffs/rock on LT, RM 878.6-876.8 Bluffs/rock on RT, RM 874.5-869.5 Bluffs/rock on RT, RM 862.5-855.0 Bluffs/rock on LT, RM 851.5-850.0
1946	Construction started
1952	Closure
1953	Project placed in operation
1954	**Cross sections analyzed** First powerhouse unit operational Discharges at Fort Randall Dam Q_{max}=32,300 cfs, Q<13.3K for 132 days Q>25.5K for 95 days
with project	Revetment on RT, RM 879.0-874.5 Local protection on RT RM 853.0-851.0
1955	Discharges at Fort Randall Dam No discharge data available for 1955
1956	Final powerhouse unit operational Discharges at Fort Randall Dam Q_{max}=50,400 cfs, Q<13.3K for 134 days Q>25.5K for 162 days, Q>35.4K for 9 days Q>37.3K for 9 days
1957	Discharges at Fort Randall Dam Q_{max}=37,800 cfs, Q<13.3K for 171 days Q>25.5K for 104 days, Q>35.4K for 3 days Q>37.3K for 2 days
1958	Discharges at Fort Randall Dam Q_{max}=36,500 cfs, Q<13.3K for 155 days Q>25.5K for 118 days, Q>35.4K for 2 days
1959	Discharges at Fort Randall Dam Q_{max}=35,100 cfs, Q<13.3K for 149 days Q>25.5K for 113 days
1960	Discharges at Fort Randall Dam Q_{max}=40,300 cfs, Q<13.3K for 178 days Q>25.5K for 85 days, Q>35.4K for 1 day Q>37.3K for 1 day
1961	Discharges at Fort Randall Dam Q_{max}=35,200 cfs, Q<13.3K for 177 days Q>25.5K for 76 days

Note: Significance of Fort Randall discharges:
The PA minimum average monthly discharge (1898-1993) is 13,300 cfs.
The CWCP minimum average monthly discharge (1898-1993) is about 13,600 cfs.
The CWCP and PA average annual discharge (1898-1993) is 25,500 cfs.
The CWCP maximum average monthly discharge (1898-1993) is 35,400 cfs.
The PA maximum average monthly discharge (1898-1993) is 37,300 cfs.

Table 3 (Continued)	
Year	**Background**
1962	Discharges at Fort Randall Dam Q_{max}=30,800 cfs, Q<13.3K for 193 days Q>25.5K for 147 days, Q>35.4K for 4 days
1963	Discharges at Fort Randall Dam Q_{max}=37,000 cfs, Q<13.3K for 193 days Q>25.5K for 147 days, Q>35.4K for 4 days
1964	Discharges at Fort Randall Dam Q_{max}=34,800 cfs, Q<13.3K for 120 days Q>2.5K for 126 days
1965	Discharges at Fort Randall Dam Q_{max}=34,600 cfs, Q<13.3K for 45 days Q>25.5K for 132 days
1966	Discharges at Fort Randall Dam Q_{max}=35,800 cfs, Q<13.3K for 45 days Q>25.5K for 189 days, Q>35.4K for 4 days
1967	Discharges at Fort Randall Dam Q_{max}=40,700 cfs, Q<13.3K for 84 days Q>25.5K for 204 days, Q>35.4K for 31 days Q>37.3K for 11 days
1968	Discharges at Fort Randall Dam Q_{max}=39,700 cfs, Q<13.3K for 28 days Q>25.5K for 244 days, Q>35.4K for 28 days Q>37.4K for 6 days
1969	Discharges at Fort Randall Dam Q_{max}=52,000 cfs, Q<13.3K for 31 days Q>25.5K for 225 days, Q>35.4K for 134 days Q>37.3K for 125 days
1970	Discharges at Fort Randall Dam Q_{max}=43,800 cfs, Q<13.3K for 8 days Q>25.5K for 229 days, Q>35.4K for 155 days Q>37.3K for 149 days
1971	Discharges at Fort Randall Dam Q_{max}=50,500 cfs, Q<13.3K for 14 days Q>25.5K for 267 days, Q>35.4K for 213 days Q>37.3K for 209 days
1972	Discharges at Fort Randall Dam Q_{max}=48,200 cfs, Q<13.3K for 6 days Q>25.5K for 258 days, Q>35.4K for 212 days Q>37.3K for 194 days
1973	Discharges at Fort Randall Dam Q_{max}=36,300 cfs, Q<13.3K for 33 days Q>25.5K for 149 days, Q>35.4K for 2 days
1974	**Cross sections analyzed** Discharges at Fort Randall Dam Q_{max}=42,100 cfs, Q<13.3K for 26 days Q>25.5K for 226 days, Q>35.4K for 40 days Q>37.3K for 10 days
	(Sheet 2 of 4)

Table 3 (Continued)	
Year	**Background**
1975	Discharges at Fort Randall Dam Q_{max}=60,600 cfs, Q<13.3K for 29 days Q>25.5K for 232 days, Q>35.4K for 167 days Q>37.3K for 164 days
October 1976	**Aerial photographs analyzed** Discharges at Fort Randall Dam Q_{max}=41,400 cfs, Q<13.3K for 2 days Q>25.5K for 270 days, Q>35.4K for 114 days Q>37.3K for 65 days
1977	Discharges at Fort Randall Dam Q_{max}=41,700 cfs, Q<13.3K for 57 days Q>25.5K for 200 days, Q>35.4K for 114 days Q>37.3K for 1 day
1978	Toe revetment on LT, RM 8694-868.6 Section 32 Toe Revet with hardpoints on RT, RM 868.5-866.0 Discharges at Fort Randall Dam Q_{max}=53,200 cfs, Q<13.3K for 53 days Q>25.5K for 198 days, Q>35.4K for 174 days Q>37.3K for 169 days
1978-1982	Section 32 window revet plus 20 hardpoints on LT, RM 869.6-868.4
1979	Discharges at Fort Randall Dam Q_{max}=43,500 cfs, Q<13.3K for 25 days Q>25.5K for 48 days
1980	Discharges at Fort Randall Dam Q_{max}=41,700 cfs, Q<13.3K for 52 days Q>25.5K for 228 days, Q>35.4K for 43 days Q>37.3K for 11 days
1981	Discharges at Fort Randall Dam Q_{max}=36,000 cfs, Q<13.3K for 96 days Q>25.5K for 222 days, Q>35.4K for 1 day
1982	Discharges at Fort Randall Dam Q_{max}=39,500 cfs, Q<13.3K for 50 days Q>25.5K for 192 days, Q>35.4K for 20 days Q>37.3K for 12 days
1983	Discharges at Fort Randall Dam Q_{max}=37,200 cfs, Q<13.3K for 47 days Q>25.5K for 153 days, Q>35.4K for 10 days
1984	Discharges at Fort Randall Dam Q_{max}=44,600 cfs, Q<13.3K for 92 days Q>25.5K for 152 days, Q>35.4K for 145 days Q>37.3K for 141 days
1985	**Cross sections analyzed** Discharges at Fort Randall Dam Q_{max}=35,900 cfs, Q<13.3K for 24 days Q>25.5K for 136 days, Q>35.4K for 1 day
1986	Discharges at Fort Randall Dam Q_{max}=45,100 cfs, Q<13.3K for 83 days Q>25.5K for 216 days, Q>35.4K for 89 days Q>37.3K for 61 days

(Sheet 3 of 4)

Table 3 (Concluded)	
Year	**Background**
1987	Discharges at Fort Randall Dam Q_{maz}=32,300 cfs, Q<13.3.K for 14 days Q>25.5.K for 190 days
1988	Discharges at Fort Randall Dam Q_{max}=37,800 cfs, Q<13.3K for 73 days Q>25.5K for 189 days, Q>35.4K for 9 days Q>37.3K for 1 day
1989	Discharges at Fort Randall Dam Q_{max}=32,700 cfs, Q<13.3K for 116 days Q>25.5K for 187 days
1990	Discharges at Fort Randall Dam Q_{max}=31,100 cfs, Q<13.3K for 134 days Q>25.5K for 68 days
1991	Discharges at Fort Randall Dam Q_{max}=33,600 cfs, Q<13.3K for 141 days Q>25.5K for 95 days
1992	Discharges at Fort Randall Dam Q_{max}=29,500 cfs, Q<13.3K for 135 days Q>25.5K for 32 days
1993	Discharges at Fort Randall Dam Q_{max}=26,000 cfs, Q<13.3K for 204 days Q>25.5K for 10 days
May 1994	**Aerial photographs** Discharges at Fort Randall Dam Q_{max}=35,000 cfs, Q<13.3K for 22 days Q>25.5K for 151 days
	(Sheet 4 of 4)

Table 4
Time Line for Missouri River Downstream of Gavins Point Dam

Year	Background
Natural (preproject)	Bluffs/rock on RT, RM 787.6-786.8 Bluffs/rock on RT, RM 766.5-766.1 Bluffs/rock on RT, RM 764.3-762.5
1952	Construction started
1955	Project placed in operation
1956	First powerhouse unit operational
1957	Final powerhouse unit operational
with project	Revetment on RT, RM 811.0-807.9
with project	Revetment on LT, RM 811.0-810.6
1960	**Cross sections analyzed** Discharges at Yankton, SD Q_{max}=33,900 cfs, Q<16K for 149 days Q>27.5K for 50 days
1961	Discharges at Yankton, SD Q_{max}=29,400 cfs, Q<16K for 175 days Q>27.5K for 65 days
1962	Discharges at Yankton, SD Q_{max}=35,300 cfs, Q<16K for 176 days Q>27.5K for 15 days
1963	Discharges at Yankton, SD Q_{max}=32,200 cfs, Q<16K for 117 days Q>27.5K for 156 days
1964	Discharges at Yankton, SD Q_{max}=33,800 cfs, Q<16K for 112 days Q>27.5K for 133 days
1965	Discharges at Yankton, SD Q_{max}=34,000 cfs, Q<16K for 100 days Q>27.5K for 118 days
1966	Discharges at Yankton, SD Q_{max}=34,700 cfs, Q<16K for 55 days Q>27.5K for 184 days
1967	Discharges at Yankton, SD Q_{max}=36,700 cfs, Q<16K for 76 days Q>27.5K for 224 days
1968	Discharges at Yankton, SD Q_{max}=38,400 cfs, Q<16K for 35 days Q>27.5K for 258 days, Q>37K for 10 days

Note: Significance of Fort Randall discharges:
The PA minimum average monthly discharge (1898-1993) is 15,500 cfs.
The CWCP minimum average monthly discharge (1898-1993) is about 16,000 cfs.
The CWCP and PA average annual discharge (1898-1993) is 27,500 cfs.
The CWCP maximum average monthly discharge (1898-1993) is 37,000 cfs.
The PA maximum average monthly discharge (1898-1993) is 40,000 cfs.

Table 4 (Continued)	
Year	**Background**
1969	Discharges at Yankton, SD Q_{max}=55,700 cfs, Q<16K for 7 days Q>27.5K for 232 days, Q>37K for 130 days Q>40K for 116 days
1970	Discharges at Yankton, SD Q_{max}=46,100 cfs, Q<16K for 16 days Q>27.5K for 247 days, Q>37K for 147 days Q>40K for 133 days
1971	Discharges at Yankton, SD Q_{max}=54,500 cfs, Q<16K for 15 days Q>27.5K for 268 days, Q>37K for 212 days Q>40K for 207 days
1972	Discharges at Yankton, SD Q_{max}=51,200 cfs, Q<16K for 0 days Q>27.5K for 268 days, Q>37K for 227 days Q>40K for 211 days
1973	Discharges at Yankton, SD Q_{max}=33,800 cfs, Q<16K for 0 days Q>27.5K for 188 days
1974	**Cross sections analyzed** Discharges at Yankton, SD Q_{max}=37,400 cfs, Q<16K for 0 days Q>27.5K for 246 days
1975	Discharges at Yankton, SD Q_{max}=63,400 cfs, Q<16K for 0 days Q>27.5K for 255 days, Q>37K for 181 days Q>40K for 167 days
Between 1975 and 1978	Local revetment on LT, RM 807.4-806.8 Sect. 14 on LT, RM 806.8-805.8 Bank protection at Yankton Bridge Local revetment on LT, RM 772.2
1976	Discharges at Yankton, SD Q_{max}=41,700 cfs, Q<16K for 0 days Q>27.5K for 274 days, Q>37K for 274 days Q>40K for 2 days
1977	Discharges at Yankton, SD Q_{max}=36,700 cfs, Q<16K for 75 days Q>27.5K for 248 days
1978	Revet and hardpoint on RT, RM 799.7-797.7 Revetment on LT, RM 797.6-797.2 Discharges at Yankton, SD Q_{max}=53,500 cfs, Q<16K for 15 days Q>27.5K for 206 days, Q>37K for 188 days Q>40K for 163 days
	(Sheet 2 of 4)

Table 4 (Continued)	
Year	**Background**
1979	Revet and hardpoint on LT, RM 769.9-794.9 Revet and hardpoint on RT, RM 786.4-785.0 Revet and hardpoint on LT, RM 786.5-784.8 Discharges at Yankton, SD Q_{max}=43,900 cfs, Q<16K for 23 days Q>27.5K for 246 days, Q>37K for 161 days Q>40K for 45 days
1980	Revet and hardpoint on RT, RM 785.0-782.3 Revet and hardpoint on LT, RM 772.0-769.1 Discharges at Yankton, SD Q_{max}=38,500 cfs, Q<16K for 44 days Q>27.5K for 247 days, Q>37K for 56 days
1981	Revet and hardpoint on LT, RM 784.5-782.0 Revet and dike on RT, RM 776.1-774.8 Revet and hardpoint on RT, RM 762.5-759.0 Revet and hardpoint on LT, RM 757.4-753.5
June 1981	**Aerial photographs analyzed** Discharges at Yankton, SD Q_{max}=36,300 cfs, Q<16K for 128 days Q>27.5K for 227 days
1982	Revetment on RT, RM768.8-767.2 Discharges at Yankton, SD Q_{max}=44,600 cfs, Q<16K for 75 days Q>27.5K for 229 days, Q>37K for 51 days Q>40K for 31 days
1983	Discharges at Yankton, SD Q_{max}=39,100 cfs, Q<16K for 30 days Q>27.5K for 159 days, Q>37K for 116 days
1984	Discharges at Yankton, SD Q_{max}=47,700 cfs, Q<16K for 35 days Q>27.5K for 150 days, Q>37K for 144 days Q>40K for 142 days
1985	Rehab of revet and hardpoint on LT, RM 796.9-794.9 Rehab of revet and hardpoint on LT, RM 772.0-769.1 Rehab of revet and hardpoint on RT, RM 762.5-759.0 Rehab of revet and hardpoint on LT, RM 757.4-753.5 Rehab of revet and hardpoint on RT, RM 768.8-767.2 Discharges at Yankton, SD Q_{max}=41,200 cfs, Q<16K for 8 days Q>27.5K for 220 days, Q>37K for 7 days Q>40K for 1 day
1986	**Cross sections analyzed** Discharges at Yankton, SD Q_{max}=50,300 cfs, Q<16K for 3 days Q>27.5K for 236 days, Q>37K for 140 days Q>40K for 83 days
	(Sheet 3 of 4)

Table 4 (Concluded)

Year	Background
1987	Discharges at Yankton, SD Q_{max}=34,400 cfs, Q<16K for 0 days Q>27.5 K for 226 days
1988	Discharges at Yankton, SD Q_{max}=38,900 cfs, Q<16K for 46 days Q>27.5K for 230 days, Q>37K for 15 days
1989	Discharges at Yankton, SD Q_{max}=32,700 cfs, Q<16K for 115 days Q>27.5K for 182 days
1990	Discharges at Yankton, SD Q_{max}=33,400 cfs, Q<16K for 140 days Q>27.5K for 98 days
1991	Discharges at Yankton, SD Q_{max}=32,100 cfs, Q<16K for 145 days Q>27.5K for 122 days
1992	Discharges at Yankton, SD Q_{max}=29,100 cfs, Q<16K for 148 days Q>27.5K for 16 days
1993	Discharges at Yankton, SD Q_{max}=24,100 cfs, Q<16K for 217 days
May 1994	**Aerial photographs analyzed** Discharges at Yankton, SD Q_{max}=32,400 cfs, Q<16K for 0 days Q>27.5K for 126 days

(Sheet 4 of 4)

Table 5
Mileage Conversion, Missouri River

Location	1941 Mileage	1960 Mileage
Fort Peck Reach		
Fort Peck Dam	1868.7	1771.55
Nashua, MT	1860.8	1766.00
Milk River	1857.4	1761.50
Little Porcupine Creek	1840.6	1743.01
Wolf Creek	1808.0	1708.25
Wolf Point, MT, Highway Bridge	1802.1	1701.42
Brockton, MT	1746.5	1649.62
Big Muddy Creek	1725.7	1630.36
Culbertson, MT, Highway Bridge	1715.1	1620.76
Snowden, MT	1686.0	1591.27
Yellowston River	1680.7	1582.00
Garrison Reach		
Garrison Dam	1455.0	1389.86
Knife River	1440.8	1375.72
Stanton, ND	1440.6	1375.72
Fort Clark, ND	1431.7	1366.65
Mandan Lake Creek	1428.6	1364.75
Washburn, ND	1418.5	1354.70
Painted Woods Creek and Lake	1412.6	1348.88
Price, ND	1402.3	1338.58
Fort Randall Reach		
Fort Randall Dam	922.0	879.98
Greenwood, SD	907.6	865.00
Ponca Creek	891.6	849.00
Niobrara River	885.3	843.55
Gavins Point Reach		
Gavins Point Dam	846.5	811.05
Yankton, SD, Highway Bridge	840.4	805.76
James River	834.6	797.50
Vermillion River	806.1	772.00
Elk Point, SD	790.1	757.80

Table 6
1941 to 1960 Sediment Range Mileage Conversion

Channel Ranges		Channel Ranges	
1941 miles	1960 miles	1941 miles	1960 miles
Fort Peck Reach		Gavins Point Reach	
1865.7	1770.0	845.1	810.7
1864.8	1769.0	844.5	809.9
1863.5	1767.7	843.8	908.2
1862.8	1766.8	843.1	808.6
1861.1	1765.1	842.5	808.0
1860.1	1763.9	841.6	807.0
1857.5	1761.7	841.0	806.2
1855.8	1759.2	840.6	806.0
1853.7	1757.3	840.4	805.8
1851.2	1754.3	839.9	805.4
1847.7	1751.0	839.1	804.5
1845.1	1747.8	837.6	803.6
1842.8	1745.8	836.1	801.9
1840.7	1744.0	835.3	802.0
1838.5	1741.2	834.5	800.0
1834.4	1736.1	832.8	798.8
1832.2	1733.8	831.7	797.9
1829.4	1731.7	830.6	797.0
1826.0	1728.1	829.4	795.6
1823.6	1724.5	828.5	794.5
1823.1	1723.9	827.3	793.8
1818.5	1720.0	826.3	793.3
1813.9	1715.5	825.2	792.5
1810.6	1712.1	824.1	790.3
1807.6	1707.7	823.0	789.2
1807.5	1707.6	822.0	787.7
1807.4	1707.5	821.1	786.9
1801.2	1700.5	820.0	786.0
1794.4	1695.0	817.7	784.5

Table 6 (Continued)			
Channel Ranges		Channel Ranges	
1941 miles	1960 miles	1941 miles	1960 miles
Fort Peck Reach (Cont'd)		Gavins Point Reach (Cont'd)	
1785.8	1687.5	816.5	782.2
1779.0	1682.5	814.7	780.2
1772.1	1674.8	812.7	778.6
1766.3	1669.5	810.2	777.0
1759.2	1661.9	808.5	775.9
1750.2	1653.3	806.3	773.9
1744.1	1647.2	804.2	771.4
1739.0	1643.4	801.4	769.0
1734.4	1638.8	797.5	765.7
1726.6	1631.3	793.9	762.6
1719.8	1624.9	791.2	761.7
1717.2	1623.3	788.8	758.1
1714.4	1620.9	786.4	756.0
1711.0	1616.8	783.6	753.1
1707.8	1612.0		
1703.6	1607.7		
1699.9	1603.4		
1693.4	1599.0		
Garrison Reach		Fort Randall Reach	
1453.4	1388.3	920.9	879.3
1452.3	1387.1	920.5	787.6
1451.1	1386.0	920.0	877.5
1450.1	1385.0	919.3	876.8
1448.4	1383.4	918.6	876.4
1447.3	1382.3	917.8	875.8
1446.4	1381.4	917.1	875.2
1445.5	1380.6	915.9	874.8
1444.7	1379.7	915.0	871.8
1443.8	1378.9	914.1	872.0
1443.3	1378.5	912.8	870.4
			(Sheet 2 of 3)

Table 6 (Concluded)

Channel Ranges		Channel Ranges	
1941 miles	**1960 miles**	**1941 miles**	**1960 miles**
Garrison Reach (Cont'd)		**Fort Randall Reach (Cont'd)**	
1442.4	1377.4	911.9	869.8
1441.7	1376.5	910.6	868.0
1440.8	1375.7	909.5	867.0
1440.0	1374.9	907.6	865.1
1439.5	1374.4	906.0	863.5
1438.8	1373.8	904.3	862.6
1437.4	1372.3	902.8	861.5
1436.2	1371.5	900.9	859.5
1435.0	1370.5	899.0	856.3
1433.4	1369.1	897.5	854.7
1431.8	1367.6	895.6	853.1
1430.7	1366.5	893.1	850.8
1429.0	1365.0	891.7	849.0
1427.8	1364.1	890.2	848.1
1426.5	1362.7	888.9	847.5
1424.5	1361.3	886.8	845.1
1422.5	1358.5	885.6	844.2
1420.3	1356.2		
1417.7	1353.8		
1415.6	1351.7		
1413.3	1349.2		
1410.2	1346.3		
1408.5	1344.8		
1407.0	1343.3		
1405.0	1341.4		
1403.2	1339.8		
1401.6	1338.2		
1400.4	1337.2		
1399.5	1336.2		

(Sheet 3 of 3)

Table 7
Fort Peck Reach End Area and Cross Section Change, 1955-1978

River Mile		1955 End Area, sq ft			1978 End Area, sq ft			Erosion or Deposition, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1941	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
1865.0	1770.0	17529	80967	13360	17663	80579	13514	-134	388	-154	100	-288
1864.8	1769.0	10776	87396	6969	10942	87587	6925	-166	-191	44	-313	-122
1863.5	1767.7	25784	48202	6926	26514	48051	7294	-730	151	-368	-947	-1098
1862.8	1766.3	4751	68618	5569	4751	69061	5762	0	-443	-193	-636	-193
1861.1	1765.1	51482	83175	16666	51415	83938	17738	67	-763	-1072	-1768	-1005
1860.1	1763.9	2178	44125	3891	2071	45021	3877	107	-896	14	-775	121
1857.5	1761.4	4736	62349	5047	5303	64082	4691	-567	-1733	356	-1944	-211
1855.8	1759.2	5037	35508	37739	4622	36171	38427	415	-663	-688	-936	-273
1853.7	1757.3	16685	50283	564	16776	51941	280	-91	-1658	284	-1465	193
1851.2	1754.3	4192	67012	42689	4909	67749	42535	-717	-737	154	-1300	-563
1847.7	1751.0	48403	82592	2620	48388	48398	2756	15	-1806	-136	-1927	-121
1845.1	1747.8	14834	26723	1549	15384	26603	721	-550	120	828	398	278
1842.8	1745.8	115671	55848	14000	116409	58865	14318	-738	-3017	-318	-4073	-1056
1840.7	1744.0	10510	32081	24065	10491	32737	24701	19	-656	-636	-1273	-617
1838.5	1741.2	32761	28500	7522	33196	27490	8321	-435	1010	-799	-224	-1234
1834.4	1736.1	32151	57546	7727	32142	58926	8274	9	-1380	-547	-1918	-538
1832.2	1733.8	10129	41236	22625	10622	39335	23924	-493	1901	-1299	109	-1792
1829.4	1731.7	8099	52795	5418	8334	52415	4685	-235	380	733	878	498

Note: LOB = left bank of cross section.
ROB = right bank of cross section.
[1] Total of columns 9, 10, and 11
[2] Total of columns 9 and 11

Table 7 (Continued)

River Mile		1955 End Area, sq ft			1978 End Area, sq ft			Erosion or Deposition, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1941	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
1826.0	1728.1	9193	35311	36906	10532	35528	38622	-1339	-217	-1716	-3272	-3055
1823.6	1724.5	13893	58360	12120	17629	57786	12112	-3736	574	8	-3154	-3728
1823.1	1723.9	12723	54021	572	13066	54125	572	-343	-104	0	-447	-343
1818.5	1720.0	22294	70448	38044	22255	71568	42067	39	-1120	-4023	-5104	-3984
1813.9	1715.5	13462	50373	26860	13177	50065	29479	285	308	-2619	-2026	-2334
1810.6	1712.1	11680	28465	2296	12170	27607	2339	-490	858	-43	325	-533
1807.6	1707.7	4792	28689	37208	no data	no data	no data	no data	no data	no data	no data	no data
1807.5	1707.6	19579	17506	76353	21403	13784	74316	-1824	3722	2037	3935	213
1807.4	1707.5	8761	31249	6300	no data	no data	no data	no data	no data	no data	no data	no data
1801.2	1700.5	22095	51605	19616	22532	50619	22327	-437	986	-2711	-2162	-3148
1794.4	1695.0	5159	64938	4015	5365	65663	3699	-206	-725	316	-615	110
1785.8	1687.5	51834	65881	7372	53103	66160	7600	-1269	-279	-228	-1776	-1497
1779.0	1682.5	9027	63029	4340	10043	62549	4405	-1016	480	-65	-601	-1081
1772.1	1674.8	38623	56590	27209	40980	55174	28288	-2357	1416	-1079	-2020	-3436
1766.3	1669.5	9989	44481	8204	10886	44889	8539	-897	-408	-335	-1640	-1232
1759.2	1661.9	26470	32532	9619	26895	32045	9643	-425	487	-24	38	-449
1759.0	1661.7	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data
1750.2	1653.3	8845	27585	5015	8875	27985	4787	-30	-400	228	-202	198
1744.0	1647.2	25535	50913	24729	26234	52803	24597	-699	-1890	132	-2457	-567
1739.0	1643.4	29990	76574	14141	30330	77188	14215	-340	-614	-74	-1028	-414
1734.4	1638.8	13933	52846	7599	13612	53145	7571	321	-299	28	50	349

(Sheet 2 of 3)

Table 7 (Concluded)

River Mile		1955 End Area, sq ft			1978 End Area, sq ft			Erosion or Deposition, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1941	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
1726.6	1631.3	1046	54603	16585	no data	no data	no data	no data	no data	no data	no data	no data
1719.8	1624.9	12249	73564	18996	12417	74768	19187	-168	-1204	-191	-1563	-359
1717.2	1623.3	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data
1714.4	1620.9	5965	47170	25458	6304	46415	25555	-339	755	-97	319	-436
1711.0	1616.8	23840	59428	6536	26886	58625	6395	-3046	803	141	-2102	-2905
1707.8	1612.0	14440	47963	20642	14271	47093	22793	169	870	-2151	-1112	-1982
1703.6	1607.7	14321	69480	2337	15698	66340	2276	-1377	3140	61	1824	-1316
1699.9	1603.4	21219	56371	22198	25094	54632	21476	-3875	1739	722	-1414	-3153
1693.4	1599.0	6137	43032	14610	6595	42800	15248	-458	232	-638	-864	-1096

(Sheet 3 of 3)

Table 8
Fort Peck Reach End Area and Cross Section Change, 1966-1978

River Mile		1966 End Area, sq ft			1978 End Area, sq ft			Erosion or Deposition, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1940	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
1865.0	1770.0	17767	80596	13505	17663	80579	13514	104	17	-9	112	95
1864.8	1769.0	10802	87549	6894	10942	87587	6925	-140	-38	-31	-209	-171
1863.5	1767.7	26307	48285	7102	26514	48051	7294	-207	234	-192	-165	-399
1862.8	1766.3	4751	68774	5670	4751	69061	5762	0	-287	-92	-379	-92
1861.1	1765.1	51440	83220	17428	51415	83938	17738	25	-718	-310	-1003	-285
1860.1	1763.9	2085	44637	3893	2071	45021	3877	14	-384	16	-354	30
1857.5	1761.4	5071	62432	4782	5303	64082	4691	-232	-1650	91	-1791	-141
1855.8	1759.2	4589	35821	37753	4622	36171	38427	-33	-350	-674	-1057	-707
1853.7	1757.3	16728	51381	571	16776	51941	280	-48	-560	291	-317	243
1851.2	1754.3	4792	68796	42835	4909	67749	42535	-117	1047	300	1230	183
1847.7	1751.0	48316	83261	2727	48388	84398	2756	-72	-1137	-29	-1238	-101
1845.1	1747.8	15302	26707	1570	15384	26603	721	-82	104	849	871	767
1842.8	1745.8	115593	58084	14239	116409	58865	14318	-816	-781	-79	-1676	-895
1840.7	1744.0	10518	32122	24400	10491	32737	24701	27	-615	-301	-889	-274
1838.5	1741.2	33750	27715	7865	33196	27490	8321	554	225	-456	323	98
1834.4	1736.1	32541	56348	7847	32142	58926	8274	399	-2578	-427	-2606	-28
1832.2	1733.8	10365	39433	23246	10622	39335	23924	-257	98	-678	-837	-935
1829.4	1731.7	8049	51370	5097	8334	52415	4685	-285	-1045	412	-918	127

Note: LOB = left bank of cross section.
ROB = right bank of cross section.
[1] Total of columns 9, 10, and 11
[2] Total of columns 9 and 11

Table 8 (Continued)

River Mile		1966 End Area, sq ft			1978 End Area, sq ft			Erosion or Deposition, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1940	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
1826.0	1728.1	9682	35775	38052	10532	35528	38622	-850	247	-570	-1173	-1420
1823.6	1724.5	16132	57576	12144	17629	57786	12112	-1497	-210	32	-1675	-1465
1823.1	1723.9	12901	53890	572	13066	54125	572	-165	-235	0	-400	-165
1818.5	1720.0	22296	71661	39426	22255	71568	42067	41	93	-2641	-2507	-2600
1813.9	1715.5	12960	49974	28416	13177	50065	29479	-217	-91	-1063	-1371	-1280
1810.6	1712.1	11910	28147	2330	12170	27607	2339	-260	540	-9	271	-269
1807.6	1707.7	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data
1807.5	1707.6	20997	14890	75286	21403	13784	74316	-406	1106	970	1670	564
1807.4	1707.5	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data
1801.2	1700.5	22176	50228	20387	22532	50619	22327	-356	-391	-1940	-2687	-2296
1794.4	1695.0	5310	65111	3988	5365	65663	3699	-55	-552	289	-318	234
1785.8	1687.5	53102	63673	7353	53103	66160	7600	-1	-2287	-247	-2535	-248
1779.0	1682.5	9924	63052	4379	10043	62549	4405	-119	503	-26	358	-145
1772.1	1674.8	39290	54400	28393	40980	55174	28288	-1690	-774	105	-2359	-1585
1766.3	1669.5	10423	45242	8120	10886	44889	8539	-463	353	-419	-529	-882
1759.2	1661.9	27816	31022	9685	26895	32045	9643	921	-1023	42	-60	963
1759.0	1661.7	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data
1750.2	1653.3	8922	27627	4969	8875	27985	4787	47	-358	182	-129	229
1744.0	1647.2	25204	52798	24614	26234	52803	24597	-1030	-5	17	-1018	-1013
1739.0	1643.4	29977	77063	14660	30330	77188	14215	-353	-125	445	-33	92
1734.4	1638.8	14103	53132	7511	13612	53145	7571	491	-13	-60	418	431

(Sheet 2 of 3)

Table 8 (Concluded)

River Mile		1966 End Area, sq ft			1978 End Area, sq ft			Erosion or Deposition, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1940	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
1726.6	1631.3	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data
1719.8	1624.9	11256	74105	20565	12417	74768	19187	-1161	-663	1378	-446	217
1717.2	1623.3	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data
1714.4	1620.9	5926	46003	25845	6304	46415	25555	-378	-412	290	-500	-88
1711.0	1616.8	24383	59703	6845	26886	58625	6395	-2503	1078	450	-975	-2053
1707.8	1612.0	14177	46694	21290	14271	47093	22793	-94	-399	-1503	-1996	-1597
1703.6	1607.7	14831	69808	2286	15698	66340	2276	-867	3468	10	2611	-857
1699.9	1603.4	24657	54458	22054	25094	54632	21476	-437	-174	578	-33	141
1693.4	1599.0	6647	42709	15089	6595	42800	15248	52	-91	-159	-198	-107

(Sheet 3 of 3)

Table 9
Garrison Reach End Area and Cross Section Change, 1956-1985

River Mile		1956-58 End Area, sq ft			1985 End Area, sq ft			Cross Section Change, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1941	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
1453.4	1388.3	7899	49929	15211	9413	54632	21775	-1514	-4703	-6564	-12781	-8078
1452.3	1387.1	13318	67553	4871	13208	76259	4969	110	-8706	-98	-8694	12
1451.1	1386.0	6091	35404	25013	6695	38718	30437	-604	-3314	-5424	-9342	-6028
1450.1	1385.0	8349	45142	4494	8607	53255	4532	-258	-8113	-38	-8409	-296
1448.4	1383.4	9468	55190	7378	10733	64769	9672	-1265	-9579	-2294	-13138	-3559
1447.3	1382.3	12153	54863	5624	12733	65412	6272	-580	-10549	-648	-11777	-1228
1446.4	1381.4	5341	46543	3936	5281	51885	4139	60	-5342	-203	-5485	-143
1445.5	1380.6	5899	61980	31334	9000	65467	45109	-3101	-3487	-13775	-20363	-16876
1444.7	1379.7	66304	34353	16591	82248	36622	18247	-15944	-2269	-1656	-19869	-17600
1443.8	1378.9	17509	70317	1570	23666	75197	1815	-6157	-4880	-245	-11282	-6402
1443.3	1378.5	13260	33126	27934	14475	31611	26134	-1215	1515	1800	2100	585
1442.4	1377.4	14640	40558	7803	18002	42763	9382	-3362	-2205	-1579	-7146	-4941
1441.7	1376.5	7662	22996	78250	8822	25093	89146	-1160	-2097	-10896	-14153	-12056
1440.8	1375.7	81615	71799	8199	87536	78153	9375	-5921	-6354	-1176	-13451	-7097
1440.0	1374.9	4908	140344	9341	5478	151392	11189	-570	-11048	-1848	-13466	-2418
1439.5	1374.4	5424	74368	14817	7046	82096	20567	-1622	-7728	-5750	-15100	-7372
1438.8	1373.8	17812	66139	56761	22126	75222	59920	-4314	-9083	-3159	-16556	-7473
1437.4	1372.3	13099	39270	3845	13765	44242	3497	-666	-4972	348	-5290	-318

Note: LOB = left bank of cross section.
ROB = right bank of cross section.

[1] Total of columns 9, 10, and 11

[2] Total of columns 9 and 11

(Continued)

Table 9 (Concluded)

River Mile		1956-58 End Area, sq ft			1985 End Area, sq ft			Cross Section Change, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1941	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
1436.2	1371.5	8385	76771	23994	8251	91690	24476	134	-14919	-482	-15267	-348
1435.0	1370.5	58048	83592	13249	58466	95928	13024	-418	-12336	225	-12529	-193
1433.4	1369.1	35762	96448	15922	38434	108511	15798	-2672	-12063	124	-14611	-2548
1431.8	1367.6	5384	141447	5140	6245	151558	5095	-861	-10111	45	-10927	-816
1430.7	1366.5	8474	56440	24328	11592	62312	27013	-3118	-5872	-2685	-11675	-5803
1429.0	1365.0	11572	94569	9341	13279	108409	8626	-1707	-13840	715	-14832	-992
1427.8	1364.1	18403	39184	11693	22855	43599	16162	-4452	-4415	-4469	-13336	-8921
1426.5	1362.7	13626	66716	25099	13937	73115	29256	-311	-6399	-4157	-10867	-4468
1424.5	1361.3	9000	86746	53205	8015	89326	57041	985	-2580	-3836	-5431	-2851
1422.5	1358.5	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data
1420.3	1356.2	11500	78834	9652	11793	90033	9960	-293	-11199	-308	-11800	-601
1417.7	1353.8	4879	76719	20038	5222	81572	23758	-343	-4853	-3720	-8916	-4063
1415.6	1351.7	15963	60111	28119	17759	65822	33573	-1796	-5711	-5454	-12961	-7250
1413.3	1349.2	35737	21184	14710	44469	17973	17159	-8732	3211	-2449	-7970	-11181
1410.2	1346.3	12047	43483	4713	13005	47465	5041	-958	-3982	-328	-5268	-12860
1408.5	1344.8	7263	38148	37494	9934	39532	36610	-2671	-1384	884	-3171	-1787
1407.0	1343.3	44599	31372	21016	47121	27972	27313	-2522	3400	-6297	-5419	-8819
1405.0	1341.4	2837	55871	12314	3046	62101	15766	-209	-6230	-3452	-9891	-3661
1403.2	1339.8	9540	36392	39701	10340	34384	46568	-800	2008	-6867	-5659	-7667
1401.6	1338.2	7046	66773	4031	10646	72021	3989	-3600	-5248	42	-8806	-3558
1400.4	1337.2	6487	9463	54491	6903	7658	61872	-416	1805	-7381	-5992	-7797
1399.5	1336.2	6783	66814	3976	7677	69466	4444	-894	-2652	-468	-4014	-1362

Table 10
Garrison Reach End Area and Cross Section Change, 1976-1985

River Mile		1976 End Area, sq ft			1985 End Area, sq ft			Cross Section Change, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1941	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
1453.4	1388.3	9197	54525	17698	9413	54632	21775	-216	-107	-4077	-4400	-4293
1452.3	1387.1	13126	74480	5064	13208	76259	4969	-82	-1779	95	-1766	13
1451.1	1386.0	6671	38592	30103	6695	38718	30437	-24	-126	-334	-484	-358
1450.1	1385.0	8610	52325	4449	8607	53255	4532	3	-930	-83	-1010	-80
1448.4	1383.4	10261	63382	9454	10733	64769	9672	-472	-1387	-218	-2077	-690
1447.3	1382.3	12628	63319	6410	12733	65412	6272	-105	-2093	138	-2060	33
1446.4	1381.4	5223	51840	3870	5281	51885	4139	-58	-45	-269	-372	-327
1445.5	1380.6	8391	66023	40342	9000	65467	45109	-609	556	-4767	-4820	-5376
1444.7	1379.7	76065	37673	17778	82248	36622	18247	-6183	1051	-469	-5601	-6652
1443.8	1378.9	20823	74614	1809	23666	75197	1815	-2843	-583	-6	-3432	-2849
1443.3	1378.5	14065	32738	26600	14475	31611	26134	-410	1127	466	1183	56
1442.4	1377.4	16949	42997	8944	18002	42763	9382	-1053	234	-438	-1257	-1491
1441.7	1376.5	8226	24681	86320	8822	25093	89146	-596	-412	-2826	-3834	-3422
1440.8	1375.7	87523	78153	9375	87536	78153	9375	-13	0	0	-13	-13
1440.0	1374.9	5072	144985	10762	5478	151392	11189	-406	-6407	-427	-7240	-833
1439.5	1374.4	6360	82062	18465	7046	82096	20567	-686	-34	-2102	-2822	-2788
1438.8	1373.8	21001	71096	60143	22126	75222	59920	-1125	-4126	223	-5028	-902
1437.4	1372.3	13469	44100	3497	13765	44242	3497	-296	-142	0	-438	-296

Note: LOB = left bank of cross section.
ROB = right bank of cross section.

[1] Total of columns 9, 10, and 11

[2] Total of columns 9 and 11

(Continued)

Table 10 (Concluded)

River Mile		1976 End Area, sq ft			1985 End Area, sq ft			Cross Section Change, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1941	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
1436.2	1371.5	8328	87761	24499	8251	91690	24476	77	-3929	23	-3829	100
1435.0	1370.5	58466	95927	13024	58466	95928	13024	0	-1	0	-1	0
1433.4	1369.1	38088	102104	15810	38434	108511	15798	-346	-6407	12	-6741	-334
1431.8	1367.6	5600	155995	5108	6245	151558	5095	-645	4437	13	3805	-632
1430.7	1366.5	10649	64048	26044	11592	62312	27013	-943	1736	-969	-176	-1912
1429.0	1365.0	13628	105346	8705	13279	108409	8626	349	-3063	79	-2635	428
1427.8	1364.1	22505	43356	17137	22855	43599	16162	-350	-243	975	382	625
1426.5	1362.7	14446	72212	26753	13937	73115	29256	509	-903	-2503	-2897	-1994
1424.5	1361.3	10071	94255	54512	8015	89326	57041	2056	4929	-2529	4456	-473
1422.5	1358.5	no data	no data	no data	no data	no data	no data	no data	no data	no data	no data	0
1420.3	1356.2	11538	89308	11127	11793	90033	9960	-255	-725	1167	187	912
1417.7	1353.8	4957	80988	21381	5222	81572	23758	-265	-584	-2377	-3226	-2642
1415.6	1351.7	19031	62216	30700	17759	65822	33573	1272	-3606	-2873	-5207	-1601
1413.3	1349.2	46441	13909	16134	44469	17973	17159	1972	-4064	-1025	-3117	947
1410.2	1346.3	12739	46043	5010	13005	47465	5041	-266	-1422	-31	-1719	-297
1408.5	1344.8	10262	38555	36672	9934	39532	36610	328	-977	62	-587	390
1407.0	1343.3	44377	29116	28749	47121	27972	27313	-2744	1144	1436	-164	-1308
1405.0	1341.4	2934	60625	16803	3046	62101	15766	-112	-1476	1037	-551	925
1403.2	1339.8	10375	35426	44276	10340	34384	46568	35	1042	-2292	-1215	-2257
1401.6	1338.2	10654	68534	4063	10646	72021	3989	8	-3487	74	-3405	82
1400.4	1337.2	6910	10213	54489	6903	7658	61872	7	2555	-7383	-4821	-7376
1399.5	1336.2	7102	71102	4201	7677	69466	4444	-575	1636	-243	818	-818

Table 11
Fort Randall Reach End Areas and Cross Section Change, 1954-1985

River Mile		1954 End Area, sq ft			1985 End Area, sq ft			Erosion or Deposition, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1941	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
920.9	879.3	24075	49526	14865	32783	51881	14151	-8708	-2355	714	-10349	-7994
920.5	878.6	4956	54320	13556	7486	55100	14799	-2530	-780	-1243	-4553	-3773
920.0	877.5	45458	55237	8401	46590	64503	8597	-1132	-9266	-196	-10594	-1328
919.3	876.7	69237	39913	5049	68474	49162	5783	763	-9249	-734	-9220	29
918.6	876.4	51817	64351	6176	51552	74397	9252	265	-10046	-3076	-12857	-2811
917.8	875.8	42862	92457	3796	47850	119737	5338	-4988	-27280	-1542	-33810	-6530
917.1	875.2	25930	91664	16274	35575	104619	19936	-9645	-12955	-3662	-26262	-13307
915.9	874.8	5301	78647	35443	5440	93597	40246	-139	-14950	-4803	-19892	-4942
915.0	872.0	2843	37880	74303	2813	42296	76136	30	-4416	-1833	-6219	-1803
914.1	871.8	3619	49662	74697	3834	58196	76610	-215	-8534	-1913	-10662	-2128
912.8	870.4	4382	35106	92885	4682	43028	90814	-300	-7922	2071	-6151	1771
911.9	869.8	2876	67199	74039	3822	76229	74288	-946	-9030	-249	-10225	-1195
910.6	868.0	50446	37348	85948	68040	35023	88058	-17594	2325	-2110	-17379	-19704
909.5	867.0	76045	76933	11578	80681	84409	13681	-4636	-7476	-2103	-14215	-6739
907.6	865.1	87189	61927	1963	91222	69553	1879	-4033	-7626	84	-11575	-3949
906.0	863.5	48634	67936	13927	48825	74018	19718	-191	-6082	-5791	-12064	-5982
904.3	862.6	6531	86687	42178	5461	93611	44208	1070	-6924	-2030	-7884	-960
902.8	861.5	948	48503	1744	1236	49277	2099	-288	-774	-355	-1417	-643

Note: LOB = left bank of cross section.
ROB = right bank of cross section.
[1] Total of columns 9, 10, and 11
[2] Total of columns 9 and 11

(Continued)

Table 11 (Concluded)

River Mile		1956-58 End Area, sq ft			1954 End Area, sq ft			Erosion or Deposition, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1941	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
900.9	859.5	2011	58876	84730	2282	60187	85917	-271	-1311	-1187	-2769	-1458
899.0	856.3	2194	77997	2971	2138	82103	3052	56	-4106	-81	-4131	-25
897.5	854.7	2459	65630	109218	2479	70923	111550	-20	-5293	-2332	-7645	-2352
895.6	853.1	18176	70478	93940	21990	71399	96347	-3814	-921	-2407	-7142	-6221
893.1	850.8	5414	32387	924	9524	33773	1031	-4110	-1386	-107	-5603	-4217
891.7	849.0	61658	41899	21717	61683	37441	24255	-25	4458	-2538	1895	-2563
890.2	848.1	91804	33957	10905	88140	28528	10629	3664	5429	276	9369	3940
888.9	847.5	138264	18280	9727	140546	10642	11218	-2282	7638	-1491	3865	-3773
886.8	845.1	10458	31226	53112	16334	23300	50563	-5876	7926	2549	4599	-3327
885.6	844.2	2042	59821	75835	1798	46434	79625	244	13387	-3790	9841	-3546

Table 12
Fort Randall Reach End Area and Cross Section Change, 1975-1985

River Mile		1975 End Area, sq ft			1985 End Area, sq ft			Cross Section Change, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1941	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
920.9	879.3	32301	50969	13912	32783	51881	14151	-482	-912	-239	-1633	-721
920.5	878.6	5739	53433	14364	7486	55100	14799	-1747	-1667	-435	-3849	-2182
920.0	877.5	46105	62844	8537	46590	64503	8597	-485	-1659	-60	-2204	-545
919.3	876.7	68214	47768	5682	68474	49162	5783	-260	-1394	-101	-1755	-361
918.6	876.4	51577	71871	8117	51552	74397	9252	25	-2526	-1135	-3636	-1110
917.8	875.8	47217	108937	5332	47850	119737	5338	-633	-10800	-6	-11439	-639
917.1	875.2	no data	no data	no data	35575	104619	19936	no data	no data	no data	no data	no data
915.9	874.8	5073	91680	39000	5440	93597	40246	-367	-1917	-1246	-3530	-1613
915.0	872.0	2862	43634	75450	2813	42296	76136	49	1338	-686	701	-637
914.1	871.8	3786	56129	78443	3834	58196	76610	-48	-2067	1833	-282	1785
912.8	870.4	4489	41661	91684	4682	43028	90814	-193	-1367	870	-690	677
911.9	869.8	3690	73715	73207	3822	76229	74288	-132	-2514	-1081	-3727	-1213
910.6	868.0	71136	33053	84933	68040	35023	88058	3096	-1970	-3125	-1999	-29
909.5	867.0	78857	86547	13745	80681	84409	13681	-1824	2138	64	378	-1760
907.6	865.1	87906	66155	1913	91222	69553	1879	-3316	-3398	34	-6680	-3282
906.0	863.5	48848	66436	17149	48825	74018	19718	23	-7582	-2569	-10128	-2546
904.3	862.6	5625	88992	43571	5461	93611	44208	164	-4619	-637	-5092	-473
902.8	861.5	no data	no data	no data	1236	49277	2099	no data	no data	no data	no data	no data

Note: LOB = left bank of cross section.
ROB = right bank of cross section.
[1] Total of columns 9, 10, and 11
[2] Total of columns 9 and 11

(Continued)

Table 12 (Concluded)

River Mile		1975 End Area, sq ft			1985 End Area, sq ft			Cross Section Change, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1941	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
900.9	859.5	2164	61741	85569	2282	60187	85917	-118	1554	-348	1088	-466
899.0	856.3	2163	79779	3136	2138	82103	3052	25	-2324	84	-2215	109
897.5	854.7	2714	64415	110507	2479	70923	111550	235	-6508	-1043	-7316	-808
895.6	853.1	22512	71503	96093	21990	71399	96347	522	104	-254	372	268
893.1	850.8	9815	27615	977	9524	33773	1031	291	-6158	-54	-5921	237
891.7	849.0	61400	34472	25886	61683	37441	24255	-283	-2969	1631	-1621	1348
890.2	848.1	91290	30281	10687	88140	28528	10629	3150	1753	58	4961	3208
888.9	847.5	140859	12953	10516	140546	10642	11218	313	2311	-702	1922	-389
886.8	845.1	11905	26497	51808	16334	23300	50563	-4429	3197	1245	13	-3184
885.6	844.2	1919	55652	77771	1798	46434	79625	121	9218	-1854	7485	-1733

Table 13
Gavins Point Reach End Area and Cross Section Change, 1960-1986

River Mile		1960 End Area, sq ft			1985 End Area, sq ft			Cross Section Change, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1941	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
845.1	810.7	294636	119330	440	294636	122902	463	0	-3572	-23	-3595	-23
844.5	809.9	102013	59310	5963	102711	65688	6176	-698	-6378	-213	-7289	-911
843.8	809.2	190334	80538	9522	190420	86832	9714	-86	-6294	-192	-6572	-278
843.1	808.6	18764	41195	40555	19537	47756	40883	-773	-6561	-328	-7662	-1101
842.5	808.0	128728	82682	53830	128728	95105	55332	0	-12423	-1502	-13925	-1502
841.6	807.0	101276	135415	57445	114988	135757	57445	-13712	-342	0	-14054	-13712
841.0	806.2	13059	126116	34755	13741	137683	34793	-682	-11567	-38	-12287	-720
840.6	806.0	4054	79136	24793	4113	85842	24743	-59	-6706	50	-6715	-9
840.4	805.8	4188	58892	45964	3469	68945	46754	719	-10053	-790	-10124	-71
839.9	805.4	34142	67914	39014	34437	76091	39491	-295	-8177	-477	-8949	-772
839.1	804.5	157673	123044	69145	161217	128666	74809	-3544	-5622	-5664	-14830	-9208
837.6	803.9	187462	86779	10028	192506	94521	10048	-5044	-7742	-20	-12806	-5064
836.1	802.0	172732	41750	43124	174615	39194	67863	-1883	2556	-24739	-24066	-26622
835.3	801.1	81626	80644	90981	86037	81198	105666	-4411	-554	-14685	-19650	-19096
834.5	800.0	155518	97134	47557	158686	104742	46997	-3168	-7608	560	-10216	-2608
832.8	798.8	204599	57238	73130	207822	63615	76648	-3223	-6377	-3518	-13118	-6741
831.7	797.9	45696	96691	131344	45891	105971	132303	-195	-9280	-959	-10434	-1154
830.6	797.0	29205	73099	136916	34967	70800	156759	-5762	2299	-19843	-23306	-25605

Note: LOB = left bank of cross section.
ROB = right bank of cross section.
[1] Total of columns 9, 10, and 11
[2] Total of columns 9 and 11

(Continued)

Table 13 (Concluded)

River Mile		1960 End Area, sq ft			1986 End Area, sq ft			Cross Section Change, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1941	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
829.4	795.6	28769	40863	135094	30716	39982	154198	-1947	881	-19104	-20170	-21051
828.5	794.5	52118	130874	43932	57064	139866	44027	-4946	-8992	-95	-14033	-5041
826.3	793.3	61330	70024	48131	60246	75361	47995	1084	-5337	136	-4117	1220
825.0	792.5	36018	77042	89551	39900	78477	91014	-3882	-1435	-1463	-6780	-5345
824.1	790.3	91076	125917	70121	92756	129171	77493	-1680	-3254	-7372	-12306	-9052
823.0	789.2	30806	247697	52892	33794	267621	52959	-2988	-19924	-67	-22979	-3055
822.0	787.7	17411	90785	19130	20675	95466	19261	-3264	-4681	-131	-8076	-3395
821.1	786.9	28563	51417	8121	29715	55461	8444	-1152	-4044	-323	-5519	-1475
820.0	786.0	116908	30730	13444	124593	32064	17576	-7685	-1334	-4132	-13151	-11817
817.7	784.5	47159	54038	179794	49460	54327	186779	-2301	-289	-6985	-9575	-9286
816.5	782.2	31436	141532	22858	33045	148475	22900	-1609	-6943	-42	-8594	-1651
814.7	780.2	40795	130573	10188	42666	133589	119524	-1871	-3016	-17636	-22523	-19507
812.7	778.6	108667	159048	44780	115243	166121	51486	-6576	-7073	-6706	-20355	-13282
810.2	777.0	58875	76630	41249	81875	77428	41571	-23000	-798	-322	-24120	-23322
808.5	775.9	86874	78591	10014	87014	80063	13928	-140	-1472	-3914	-5526	-4054
806.3	773.9	58428	120614	122818	58405	120222	139775	23	392	-16957	-16542	-16934
804.2	771.4	13850	154151	40468	23004	164209	40607	-9154	-10058	-139	-19351	-9293
801.4	769.0	30656	108218	27577	31955	115315	33600	-1299	-7097	-6023	-14419	-7322
793.9	762.6	45382	75377	3693	45400	84619	4834	-18	-9242	-1141	-10401	-1159
791.2	761.7	no data	no data	no data	193356	34027	62560	no data	no data	no data	no data	no data
788.8	758.1	155250	34176	86559	156733	32754	110978	-1483	1422	-24419	-24480	-25902
786.4	756.0	46134	93025	30095	60558	96958	34622	-14424	-3933	-4527	-22884	-18951
783.6	753.1	4785	106793	4116	4785	131637	4065	0	-24844	51	-24793	51

Table 14
Gavins Point Reach End Area and Cross Section Change, 1974-1986

River Mile		1974 End Area, sq ft			1986 End Area, sq ft			Cross Section Change, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1941	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
845.1	810.7	294636	123339	440	294636	122902	463	0	437	-23	414	-23
844.5	809.9	102637	63497	6192	102711	65688	6176	-74	-2191	16	-2249	-58
843.8	809.2	190420	86451	9598	190420	86832	9714	0	-381	-116	-497	-116
843.1	808.6	19001	47386	40735	19537	47756	40883	-536	-370	-148	-1054	-684
842.5	808.0	128728	92700	55460	128728	95105	55332	0	-2405	128	-2277	128
841.6	807.0	111808	136604	57445	114988	135757	57445	-3180	847	0	-2333	-3180
841.0	806.2	13439	134235	34793	13741	137683	34793	-302	-3448	0	-3750	-302
840.6	806.0	4087	83165	24772	4113	85842	24743	-26	-2677	29	-2674	3
840.4	805.8	no data	no data	no data	3469	68945	46754	no data	no data	no data	no data	no data
839.9	805.4	34304	72997	39188	34437	76091	39491	-133	-3094	-303	-3530	-436
839.1	804.5	161288	124004	71194	161217	128666	74809	71	-4662	-3615	-8206	-3544
837.6	803.9	191068	90931	10098	192506	94521	10048	-1438	-3590	50	-4978	-1388
836.1	802.0	175716	38517	58733	174615	39194	67863	1101	-623	-9130	-8652	-8029
835.3	801.1	82930	81384	104488	86037	81198	105666	-3107	186	-1178	-4099	-4285
834.5	800.0	160255	99441	47341	158686	104742	46997	1569	-5301	344	-3388	1913
832.8	798.8	206075	60106	74158	207822	63615	76648	-1747	-3509	-2490	-7746	-4237
831.7	797.9	45868	100862	132258	45891	105971	132303	-23	-5109	-45	-5177	-68
830.6	797.0	36542	68827	147649	34967	70800	156759	1575	-1973	-9110	-9508	-7535

Note: LOB = left bank of cross section.
ROB = right bank of cross section.
[1] Total of columns 9, 10, and 11
[2] Total of columns 9 and 11

(Continued)

Table 14 (Concluded)

River Mile		1974 End Area, sq ft			1986 End Area, sq ft			Cross Section Change, sq ft			Total Change in Cross-Sectional End Area,[1] sq ft	Total Change in Bank Cross-Sectional Area,[2] sq ft
1941	1960	LOB	Channel	ROB	LOB	Channel	ROB	LOB	Channel	ROB		
829.4	795.6	30923	44122	141670	30716	39982	154198	207	4140	-12528	-8181	-12321
828.5	794.5	53041	136702	44104	57064	139866	44027	-4023	-3164	77	-7110	-3946
826.3	793.3	61186	71621	49139	60246	75361	47995	940	-3740	1144	-1656	2084
825.2	792.5	40549	74794	89797	39900	78477	91014	649	-3683	-1217	-4251	-568
824.1	790.3	92068	127542	71188	92756	129171	77493	-688	-1629	-6305	-8622	-6993
823.0	789.2	32910	261349	52959	33794	267621	52959	-884	-6272	0	-7156	-884
822.0	787.7	18678	90472	18689	20675	95466	19261	-1997	-4994	-572	-7563	-2569
821.1	786.9	28687	52430	8434	29715	55461	8444	-1028	-3031	-10	-4069	-1038
820.0	786.0	120388	31873	16129	124593	32064	17576	-4205	-191	-1447	-5843	-5652
817.7	784.5	47301	54812	181260	49460	54327	186779	2159	485	-5519	-7193	-7678
816.5	782.2	32568	146179	22798	33045	148475	22900	-477	-2296	-102	-2875	-579
814.7	780.2	41720	128714	106837	42666	133589	119524	-946	-4875	-12687	-18508	-13633
812.7	778.6	111160	167623	50708	115243	166121	51486	-4083	1502	-778	-3359	-4861
810.2	777.0	77509	73190	45041	81875	77428	41571	-4366	-4238	3470	-5134	-896
808.5	775.9	87014	80622	13398	87014	80063	13928	0	559	-530	29	-530
806.3	773.9	58382	122482	126527	58405	120222	139775	-23	2260	-13248	-11011	-13271
804.2	771.4	25296	152183	39921	23004	164209	40607	2292	-12026	-686	-10420	1606
801.4	769.0	31340	113735	33065	31955	115315	33600	-615	-1580	-535	-2730	-1150
793.9	762.6	45433	80057	4518	45400	84619	4834	33	-4562	-316	-4845	-283
791.2	761.7	187976	34930	52983	193356	34027	62560	-5380	903	9577	-14054	-14957
788.8	758.1	160884	35281	105622	156733	32754	110978	4151	2527	-5356	1322	-1205
786.4	756.0	48993	94954	34473	60558	96958	34622	-11565	-2004	-149	-13718	-11714
783.6	753.1	4785	124124	4145	4785	131637	4065	0	-7513	80	-7433	80

Table 15
Fort Peck Reach Volume Computation

River Mile		Volume Computation, acre-ft			
		1955 - 1978		1966 - 1978	
1941	1960	Bank	Channel	Bank	Channel
1865.0	1770.0				
1864.8	1769.0	-25	12	-5	-1
1863.5	1767.7	-96	-3	-45	15
1862.8	1766.3	-110	-25	-42	-4
1861.1	1765.1	-87	-88	-27	-73
1860.1	1763.9	-64	-121	-19	-80
1857.5	1761.4	-14	-398	-17	-308
1855.8	1759.2	-65	-319	-113	-267
1853.7	1757.3	-9	-267	-53	-105
1851.2	1754.3	-67	-435	77	89
1847.7	1751.0	-137	-509	16	-18
1845.1	1747.8	30	-327	129	-200
1842.8	1745.8	-94	-351	-16	-82
1840.7	1744.0	-183	-401	-128	-152
1838.5	1741.2	-315	60	-30	-66
1834.4	1736.1	-542	-113	21	-720
1832.2	1733.8	-336	75	-139	-358
1829.4	1731.7	-164	289	-102	-120
1826.0	1728.1	-553	35	-280	-173
1823.6	1724.5	-1480	78	-629	8
1823.1	1723.9	-148	17	-59	-16
1818.5	1720.0	-1023	-289	-654	-34
1813.9	1715.5	-1723	-221	-1058	1
1810.6	1712.1	-591	240	-319	93
1807.6	1707.7	No data	No data	No data	No data
1807.5	1707.6	-87	1249	80	449
1807.4	1707.5	No data	No data	No data	No data
1801.2	1700.5	-1263	2026	-745	308
1794.4	1695.0	-1013	87	-687	-314

(Continued)

Table 15 (Concluded)

River Mile		Volume Computation, acre-ft			
		1955 - 1978		1966 - 1978	
1941	1960	Bank	Channel	Bank	Channel
1785.8	1687.5	-630	-456	-6	-1290
1779.0	1682.5	-781	61	-119	-541
1772.1	1674.8	-2108	885	-807	-126
1766.3	1669.5	-1499	324	-792	-135
1759.2	1661.9	-770	36	37	-307
1759.0	2661.7	No data	No data	No data	No data
1750.2	1653.3	-131	46	624	-723
1744.0	1647.2	-136	-847	-290	-134
1739.0	1643.4	-226	-577	-212	-30
1734.4	1638.8	-18	-255	146	-38
1726.6	1631.3	No data	No data	No data	No data
1719.8	1624.9	-8	-1266	546	-569
1714.4	1620.9	-194	-110	32	-263
1711.0	1616.8	-824	384	-528	164
1707.8	1612.0	-1422	487	-1062	198
1703.6	1607.7	-859	1045	-640	800
1699.9	1603.4	-1165	1271	-187	858
1693.4	1599.0	-1133	526	9	-71
	Total	**-22065**	**1855**	**-8092**	**-4339**

Table 16
Garrison Reach Volume Computation

River Mile		Volume Computation, acre-ft			
		1956 - 1985		1976 - 1985	
1941	1960	Bank	Channel	Bank	Channel
1453.4	1388.3				
1452.3	1387.1	-587	-975	-311	-137
1451.1	1386.0	-401	-801	-23	-127
1450.1	1385.0	-383	-693	-27	-64
1448.4	1383.4	-374	-1716	-75	-225
1447.3	1382.3	-319	-1342	-44	-232
1446.4	1381.4	-75	-867	-16	-117
1445.5	1380.6	-825	-428	-277	25
1444.7	1379.7	-1881	-314	-656	88
1443.8	1378.9	-1164	-347	-455	22
1443.3	1378.5	-134	-77	-66	13
1442.4	1377.4	-296	-47	-97	92
1441.7	1376.5	-927	-235	-268	-10
1440.8	1375.7	-929	-410	-167	-20
1440.0	1374.9	-461	-844	-41	-311
1439.5	1374.4	-297	-569	-110	-195
1438.8	1373.8	-540	-611	-134	-151
1437.4	1372.3	-708	-1278	-109	-388
1436.2	1371.5	-32	-964	-10	-197
1435.0	1370.5	-33	-1652	6	-238
1433.4	1369.1	-233	-2070	-28	-544
1431.8	1367.6	-306	-2016	-88	-179
1430.7	1366.5	-441	-1066	-170	412
1429.0	1365.0	-618	-1792	-135	-121
1427.8	1364.1	-541	-996	57	-180
1426.5	1362.7	-1136	-918	-116	-97
1424.5	1361.3	-621	-762	-209	342
1422.5	1358.5	No data	No data	No data	No data
1420.3	1356.2	-1067	-4259	136	1299

(Continued)

Table 16 (Concluded)

River Mile		Volume Computation, acre-ft			
		1956 - 1985		1976 - 1985	
1941	1960	Bank	Channel	Bank	Channel
1417.7	1353.8	-678	-2335	-252	-190
1415.6	1351.7	-1440	-1345	-540	-533
1413.3	1349.2	-2793	-379	-99	-1162
1410.2	1346.3	-2191	-136	114	-964
1408.5	1344.8	-279	-488	8	-218
1407.0	1343.3	-964	183	-83	15
1405.0	1341.4	-1437	-326	-44	-38
1403.2	1339.8	-1098	-409	-129	-42
1401.6	1338.2	-1088	-314	-211	-237
1400.4	1337.2	-688	-209	-442	-56
1399.5	1336.2	-555	-51	-497	254
	Total	**-28540**	**-33854**	**-5605**	**-4413**

Table 17
Fort Randall Reach Volume Computation

River Mile		Volume Computation, acre-ft			
		1954 - 1985		1975 - 1985	
1941	1960	Bank	Channel	Bank	Channel
920.9	879.3				
920.5	878.6	-499	-133	-123	-109
920.0	877.5	-340	-670	-182	-222
919.3	876.7	-63	-898	-44	-148
918.6	876.4	-51	-351	-27	-71
917.8	875.8	-340	-1357	-64	-485
917.1	875.2	-721	-1463	No data	No data
915.9	974.8	-442	-676	-136	-771
915.0	872.0	-1145	-3286	-382	-98
914.1	871.8	-48	-157	14	-9
912.8	870.4	-30	-1396	209	-291
911.9	869.8	21	-616	-19	-141
910.6	868.0	-2280	-731	-135	-489
909.5	867.0	-1603	-312	-108	10
907.6	865.1	-1231	-1739	-581	-145
906.0	863.5	-963	-1329	-565	-1065
904.3	862.6	-379	-709	-165	-666
902.8	861.5	-107	-513	No data	No data
900.9	859.5	-255	-253	-176	-576
899.0	856.3	-288	-1051	-69	-149
897.5	854.7	-230	-911	-68	-856
895.6	853.1	-831	-603	-52	-621
893.1	850.8	-1455	-322	70	-844
891.7	849.0	-740	335	173	-996
890.2	848.1	75	539	249	-66
888.9	847.5	6	475	103	148
886.8	845.1	-1033	2264	-520	801
885.6	844.2	-375	1163	-268	677
	Total	**-15345**	**-14702**	**-2868**	**-7182**

Table 18
Gavins Point Reach Volume Computation

River Mile		Volume Computation, acre-ft			
		1960 - 1986		1974 - 1986	
1941	1960	Bank	Channel	Bank	Channel
845.1	810.7				
844.5	809.0	-45	-482	-4	-85
843.8	809.2	-50	-538	-7	-109
843.1	808.6	-50	-467	-29	-27
842.5	808.0	-95	-690	-20	-101
841.6	807.0	-922	-774	-185	-94
841.0	806.2	-700	-577	-169	-126
840.6	806.0	-9	-221	-4	-74
840.4	805.8	-1	-203	No data	No data
839.9	805.4	-20	-442	-16	-210
839.1	804.5	-544	-753	-217	-423
837.6	803.9	-519	-486	-179	-300
836.1	802.0	-3649	-597	-1084	-485
835.3	801.1	-2494	109	-672	-24
834.5	800.0	-1447	-544	-158	-341
832.8	798.8	-680	-1017	-169	-641
831.7	797.9	-431	-854	-235	-470
830.6	797.0	-1460	-381	-415	-386
829.4	795.6	-3959	270	-1685	184
828.5	794.5	-1739	-541	-1084	65
826.3	793.3	-278	-1042	-135	-502
825.2	792.5	-200	-328	74	-360
824.1	790.3	-1920	-625	-1008	-708
823.0	789.2	-807	-1545	-525	-527
822.0	787.7	-586	-2237	-314	-1024
821.1	786.9	-236	-423	-175	-389
820.0	786.0	-725	-293	-365	-176
817.7	784.5	-1918	-148	-1212	27
816.5	782.2	-1525	-1008	-1151	-252

(Continued)

Table 18 (Concluded)

River Mile		Volume Computation, acre-ft			
		1960 - 1986		1974 - 1986	
1941	**1960**	**Bank**	**Channel**	**Bank**	**Channel**
814.7	780.2	-2565	-1207	-1723	-869
812.7	778.6	-3180	-978	-1793	-327
810.2	777.0	-3549	-763	-558	-265
808.5	775.9	-1825	-151	-95	-245
806.3	773.9	-2544	-131	-1673	342
804.2	771.4	-3974	-1465	-1767	-1480
801.4	769.0	-3974	-1465	-1767	-1480
793.9	762.6	-3290	-6338	-556	-2382
791.2	761.7	No data	No data	-831	-200
788.8	758.1	-7380	-2133	-3526	748
786.4	756.0	-5709	-320	-1644	67
783.6	753.1	-3322	-5058	-2045	-1673
	Total	**-66762**	**-37877**	**-27289**	**-15824**

Table 19
Islands and Sandbar Density
Fort Peck Reach

River Miles		Reach Length, miles	Density, acres/mile		
Upstream	Downstream		As of 1974	As of 1990	Difference 1974 - 1990
Islands					
1770.7	1761.7	9.0	23.7	29.9	6.2
1761.7	1749.3	12.4	3.1	4.7	1.6
1674.2	1669.0	5.2	19.6	3.7	-15.9
1669.0	1661.5	7.5	9.7	14.0	4.3
1652.5	1648.5	4.0	9.6	18.0	8.4
1648.5	1643.4	5.1	17.2	9.2	-8.0
1643.4	1638.8	4.6	22.8	24.0	1.2
1631.0	1625.6	5.4	7.6	17.9	10.3
1625.6	1620.7	4.9	22.8	24.1	1.3
1620.7	1603.0	17.7	10.1	8.9	-1.2
1603.0	1599.0	4.0	78.3	3.6	-74.7[1]
1599.0	1596.0	3.0	19.3	35.2	15.9
Entire reach average		82.8	16.4	14.2	-2.2
Reach w/o cutoff average		78.8	13.3	14.7	1.4
Sandbars					
1770.7	1761.7	9.0	0.4	0.5	0.1
1761.7	1749.3	12.4	0.1	0.1	0.0
1674.2	1669.0	5.2	3.7	6.1	2.4
1669.0	1661.5	7.5	2.8	4.7	1.9
1652.5	1648.5	4.0	6.8	8.0	1.2
1648.5	1643.4	5.1	2.4	1.2	-1.2
1643.4	1638.8	4.6	0.0	0.0	0.0
1631.0	1625.6	5.4	1.0	6.8	5.8
1625.6	1620.7	4.9	7.8	4.8	-3.0
1620.7	1603.0	17.7	6.5	11.2	4.7
1603.0	1599.0	4.0	3.0	1.7	-1.3[1]
1599.0	1596.0	3.0	0.9	14.8	13.9
Entire reach average		82.8	3.1	5.1	2.0
Reach w/o cutoff average		78.8	3.2	4.7	1.5

[1] Segment of reach where natural cutoff occurred.

Table 20
Islands and Sandbar Density
Garrison Reach

River Miles		Reach Length, miles	Density, acres/mile		
Upstream	Downstream		As of 1976	As of 1990	Difference 1976 - 1990
Islands					
1389.0	1371.7	17.3	18.5	26.4	7.9
1371.7	1364.0	7.7	58.5	22.5	-36.0
1364.0	1355.3	8.7	0.0	34.4	34.4
1355.3	1346.8	8.5	38.5	45.2	6.7
1346.8	1339.0	7.8	10.2	15.5	5.3
1339.0	1325.2	13.8	16.6	31.5	14.9
1325.2	1315.4	9.8	51.3	51.9	0.6
Entire reach average		73.6	25.9	32.3	6.3
Sandbars					
1389.0	1371.7	17.3	35.5	22.9	-12.6
1371.7	1364.0	7.7	108.1	64.0	-44.1
1364.0	1355.3	8.7	51.9	56.5	4.6
1355.3	1346.8	8.5	40.6	36.1	-4.5
1346.8	1339.0	7.8	18.7	51.3	32.6
1339.0	1325.2	13.8	26.4	45.8	19.4
1325.2	1315.4	9.8	23.9	21.3	-2.6
Entire reach average		73.6	40.6	39.8	-0.8

Table 21
Islands and Sandbar Density
Fort Randall Reach

River Miles		Reach Length, miles	Density, acres/mile		
Upstream	Downstream		As of 1976	As of 1994	Difference 1976 - 1994
Islands					
880.0	877.5	2.5	3.7	7.4	3.7
877.5	874.0	3.5	57.6	48.1	-9.5
874.0	869.7	4.3	45.3	54.1	8.8
869.7	864.5	5.2	0.0	0.0	0.0
864.5	861.9	2.6	255.2	146.1	-109.1
861.9	854.8	7.1	105.1	103.1	-2.0
854.8	850.9	3.9	67.7	39.8	-27.9
850.9	843.5	7.4	117.4	79.9	-37.5
Entire reach average		36.5	80.8	62.4	-18.4
Sandbars					
880.0	877.5	2.5	0.1	0.0	-0.1
877.5	874.0	3.5	18.2	5.9	-12.3
874.0	869.7	4.3	10.0	5.4	-4.6
869.7	864.5	5.2	49.9	4.1	-45.8
864.5	861.9	2.6	125.5	0.4	-125.1
861.9	854.8	7.1	39.9	0.1	-39.8
854.8	850.9	3.9	80.8	3.4	-77.4
850.9	843.5	7.4	52.8	15.7	-37.1
Entire reach average		36.5	46.1	5.4	-40.7

Table 22
Islands and Sandbar Density
Gavins Point Reach

River Miles		Reach Length, miles	Density, acres/mile		
Upstream	Downstream		As of 1981	As of 1994	Difference 1976 - 1994
Islands					
808.5	797.8	10.7	93.9	91.0	-2.9
797.8	787.4	10.4	6.3	6.5	0.2
787.4	776.2	11.2	56.4	55.3	-1.1
776.2	769.0	7.2	29.7	23.7	-6.0
769.0	763.0	6.0	35.4	22.9	-12.5
763.0	753.7	9.3	4.4	9.3	4.9
Entire reach average		54.8	39.6	37.5	-2.1
Sandbars					
808.5	797.8	10.7	11.2	5.0	-6.2
797.8	787.4	10.4	25.0	3.4	-21.6
787.4	776.2	11.2	20.7	6.3	-14.4
776.2	769.0	7.2	16.3	0.2	-16.1
769.0	763.0	6.0	34.5	7.4	-27.1
763.0	753.7	9.3	48.5	82.1	33.9
Entire reach average		54.8	25.3	17.7	-7.6

Table 23
Island Exposure Versus Discharge
Fort Peck Reach

River Mile	Island Area, acres	Discharge, cfs	Date
1766.5	not present	25,000	Sept. 1947 (mosaic)
	present	--[1]	July 1956 (mosaic)
	2.4	12,200	16 Aug 1974
	2.2	22,100	18 June 1976
	6.8	7,900	25 Oct 1990
1762.0	present	25,000	Sept. 1947 (mosaic)
	present	--[1]	July 1956 (mosaic)
	14.6	12,200	16 Aug 1974
	11.2	22,100	18 June 1976
	12.1	7,900	25 Oct 1990
1760.0	2.3	12,200	16 Aug 1974
	4.1	22,100	18 Jun 1976
	5.7	7,900	25 Oct 1990
1756.0	present	25,000	Sep. 1947 (mosaic)
	present	--[1]	July 1956 (mosaic)
	18.1	12,200	16 Aug 1974
	65.9	22,100	18 Jun 1976
	26.6	7,900	25 Oct 1990
1754.5	4.7	12,200	16 Aug 1974
	12.6	22,100	18 Jun 1976
	5.3	7,900	25 Oct 1990
1754.0	3.1	12,200	16 Aug 1974
	7.2	22,100	18 Jun 1976
	5.6	7,900	25 Oct 1990
1751.0	present	25,000	Sep. 1947 (mosaic)
	present	--[1]	July 1956 (mosaic)
	7.3	12,200	16 Aug 1974
	21.7	22,100	18 Jun 1976
	9.0	7,900	25 Oct 1990

(Sheet 1 of 6)

[1] Unknown.

Table 23 (Continued)			
River Mile	**Island Area , acres**	**Discharge, cfs**	**Date**
1674.0	not present	25,000	Sep. 1947 (mosaic)
	present	--[1]	July 1956 (mosaic)
	9.2	12,200	16 Aug 1974
	15.3	22,100	18 Jun 1976
	0.0	7,900	25 Oct 1990
1672.0	present	25,000	Sep. 1947 (mosaic)
	present	--[1]	July 1956 (mosaic)
	82.4	12,200	16 Aug 1974
	87.2	22,100	18 Jun 1976
	To land	7,900	25 Oct 1990
1669.5	present	25,000	Sep. 1947 (mosaic)
	present	--[1]	July 1956 (mosaic)
	10.2	12,200	16 Aug 1974
	5.2	22,100	18 Jun 1976
	7.0	7,900	25 Oct 1990
1668.0	2.6	12,200	16 Aug 1974
	2.6	22,100	18 Jun 1976
	2.6	7,900	25 Oct 1990
1666.0	1.7	12,200	16 Aug 1974
	3.9	22,100	18 Jun 1976
	2.6	7,900	25 Oct 1990
1665.0	5.5	12,200	16 Aug 1974
	0.4	22,100	18 Jun 1976
	5.2	7,900	25 Oct 1990
1662.0	present	25,000	Sep. 1947 (mosaic)
	present	--[1]	July 1956 (mosaic)
	73.2	12,200	16 Aug 1974
	72.7	22,100	18 Jun 1976
	63.1	7,900	25 Oct 1990
			(Sheet 2 of 6)

[1] Unknown.

Table 23 (Continued)

River Mile	Island Area, acres	Discharge, cfs	Date
1660.0	5.1	12,200	16 Aug 1974
	4.4	22,100	18 Jun 1976
	to land	7,900	25 Oct 1990
1653.5	present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	73.2	12,200	16 Aug 1974
	70.5	22,100	18 Jun 1976
	70.0	7,900	25 Oct 1990
1652.0	6.0	12,200	16 Aug 1974
	10.3	22,100	18 Jun 1976
	8.0	7,900	25 Oct 1990
1651.0	present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	14.0	12,200	16 Aug 1974
	5.0	22,100	18 Jun 1976
	8.4	7,900	25 Oct 1990
1649.5	present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	2.0	12,200	16 Aug 1974
	3.5	22,100	18 Jun 1976
	16.0	7,900	25 Oct 1990
1649.0	5.6	12,200	16 Aug 1974
	8.7	22,100	18 Jun 1976
	5.2	7,900	25 Oct 1990
1648.0	not present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	14.0	12,200	16 Aug 1974
	10.3	22,100	18 Jun 1976
	10.4	7,900	25 Oct 1990

(Sheet 3 of 6)

[1] Unknown.

Table 23 (Continued)

River Mile	Island Area, acres	Discharge, cfs	Date
1645.0	25.2	12,200	16 Aug 1974
	22.4	22,100	18 Jun 1976
	15.2	7,900	25 Oct 1990
1643.0	present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	27.2	12,200	16 Aug 1974
	21.4	22,100	18 Jun 1976
	34.8	7,900	25 Oct 1990
1641.0	6.0	12,200	16 Aug 1974
	5.0	22,100	18 Jun 1976
	to land	7,900	25 Oct 1990
1640.5	9.2	12,200	16 Aug 1974
	6.3	22,100	18 Jun 1976
	8.4	7,900	25 Oct 1990
1639.0	present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	61.2	12,200	16 Aug 1974
	59.1	22,100	18 Jun 1976
	67.6	7,900	25 Oct 1990
1638.5	present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	96.0	12,200	16 Aug 1974
	66.5	22,100	18 Jun 1976
	to land	7,900	25 Oct 1990
1638.0	present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	12.4	12,200	16 Aug 1974
	6.9	22,100	18 Jun 1976
	8.4	7,900	25 Oct 1990

(Sheet 4 of 6)

[1] Unknown.

Table 23 (Continued)			
River Mile	**Island Area, acres**	**Discharge, cfs**	**Date**
1633.5	present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	66.5	12,200	16 Aug 1974
	32.5	22,100	18 Jun 1976
	61.2	7,900	25 Oct 1990
1632.0	present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	10.1	12,200	16 Aug 1974
	3.9	22,100	18 Jun 1976
	38.0	7,900	25 Oct 1990
1630.0	not present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	26.4	12,200	16 Aug 1974
	20.2	22,100	18 Jun 1976
	21.7	7,900	25 Oct 1990
1626.5	not present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	9.5	12,200	16 Aug 1974
	2.8	22,100	18 Jun 1976
	8.3	7,900	25 Oct 1990
1625.0	45.9	12,200	16 Aug 1974
	13.3	22,100	18 Jun 1976
	5.5	7,900	25 Oct 1990
1623.5	present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	46.4	12,200	16 Aug 1974
	29.3	22,100	18 Jun 1976
	25.9	7,900	25 Oct 1990
1622.0	not present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	13.9	12,200	16 Aug 1974

(Sheet 5 of 6)

[1] Unknown.

Table 23 (Concluded)

River Mile	Island Area, acres	Discharge, cfs	Date
1622.0 (Cont'd)	8.8	22,100	18 Jun 1976
	22.8	7,900	25 Oct 1990
1614.0	not present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	21.8	12,200	16 Aug 1974
	17.5	22,100	18 Jun 1976
	16.6	7,900	25 Oct 1990
1607.5	present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	11.9	12,200	16 Aug 1974
	10.6	22,100	18 Jun 1976
	20.0	7,900	25 Oct 1990
1604.0	present	25,000	Sep. 1947 (mosaic)
	present	–[1]	July 1956 (mosaic)
	62.7	12,200	16 Aug 1974
	38.6	22,100	18 Jun 1976
	53.4	7,900	25 Oct 1990
1597.5	0.0	12,200	16 Aug 1974
	0.0	22,100	18 Jun 1976
	31.0	7,900	25 Oct 1990

(Sheet 6 of 6)

[1] Unknown.

Table 24
Island Exposure Versus Discharge
Garrison Reach

River Mile	Island Area, acres	Discharge, cfs	Date
1377.5	present (bar)	--[1]	July 1956 (mosaic)
	58.9	13,400	10 Oct 1976
	14.3	24,100	3 June 1981
	76.4	10,300	25 Oct 1990
1376.0	present	--[1]	July 1956 (mosaic)
	150.7	13,400	10 Oct 1976
	104.8	24,100	3 June 1981
	124.3	10,300	25 Oct 1990
1371.0	present	--[1]	July 1956 (mosaic)
	420.3	13,400	10 Oct 1976
	447.6	24,100	3 June 1981
	172.9 (to land)	10,300	25 Oct 1990
1360.5	present (bar)	--[1]	July 1956 (mosaic)
	93.9	13,400	10 Oct 1976
	16.3	24,100	3 June 1981
	150.4	10,300	25 Oct 1990
1353.5	present	--[1]	July 1956 (mosaic)
	127.6	13,400	10 Oct 1976
	50.8	24,100	3 June 1981
	202.8	10,300	25 Oct 1990
1351.0	present (bar)	--[1]	July 1956 (mosaic)
	28.5	13,400	10 Oct 1976
	61.3	24,100	3 June 1981
	49.2	10,300	25 Oct 1990
1348.0	present	--[1]	July 1956 (mosaic)
	131.4	13,400	10 Oct 1976
	75.1	24,100	3 June 1981
	131.8	10,300	25 Oct 1990

(Sheet 1 of 3)

[1] Unknown.

Table 24 (Continued)

River Mile	Island Area, acres	Discharge, cfs	Date
1344.8	present (bar)	–[1]	July 1956 (mosaic)
	10.0	13,400	10 Oct 1976
	7.3	24,100	3 June 1981
	22.9	10,300	25 Oct 1990
1344.0	present (bar)	–[1]	July 1956 (mosaic)
	0.0	13,400	10 Oct 1976
	6.0	24,100	3 June 1981
	17.9	10,300	25 Oct 1990
1340.0	present (bar)	–[1]	July 1956 (mosaic)
	69.3	13,400	10 Oct 1976
	81.9	24,100	3 June 1981
	103.4	10,300	25 Oct 1990
1335.0	present	–[1]	July 1956 (mosaic)
	30.6	13,400	10 Oct 1976
	40.3	24,100	3 June 1981
	84.8	10,300	25 Oct 1990
1331.5	present (bar)	–[1]	July 1956 (mosaic)
	3.9	13,400	10 Oct 1976
	11.7	24,100	3 June 1981
	260.2	10,300	25 Oct 1990
1330.0	present (bar)	–[1]	July 1956 (mosaic)
	89.3	13,400	10 Oct 1976
	83.9	24,100	3 June 1981
	75.2	10,300	25 Oct 1990
1326.0	present (bar)	–[1]	July 1956 (mosaic)
	53.5	13,400	10 Oct 1976
	26.9	24,100	3 June 1981
	99.5	10,300	25 Oct 1990

(Sheet 2 of 3)

[1] Unknown.

Table 24 (Concluded)

River Mile	Island Area, acres	Discharge, cfs	Date
1323.5	present (bar)	--[1]	July 1956 (mosaic)
	212.0	13,400	10 Oct 1976
	195.8	24,100	3 June 1981
	293.9	10,300	25 Oct 1990
1322.0	not present	--[1]	July 1956 (mosaic)
	60.2	13,400	10 Oct 1976
	73.8	24,100	3 June 1981
	84.9	10,300	25 Oct 1990
1320.0	present	--[1]	July 1956 (mosaic)
	0.0	13,400	10 Oct 1976
	7.5	24,100	3 June 1981
	11.2	10,300	25 Oct 1990
1317.5	present	--[1]	July 1956 (mosaic)
	230.4	13,400	10 Oct 1976
	244.7	24,100	3 June 1981
	209.3	10,300	25 Oct 1990

(Sheet 3 of 3)

[1] Unknown.

Table 25
Island Exposure Versus Discharge
Fort Randall Reach

River Mile	Island Area, acres	Discharge, cfs	Date
875.5	95.1	60,000	7 Aug 1975
	201.9	38,000	17 Oct 1976
	147.4	29,500	4 May 1994
871.0	138.0	60,000	7 Aug 1975
	195.2	38,000	17 Oct 1976
	233.1	29,500	4 May 1994
863.5	209.6	60,000	7 Aug 1975
	387.1	38,000	17 Oct 1976
	372.0	29,500	4 May 1994
861.0	12.3	60,000	7 Aug 1975
	12.7	38,000	17 Oct 1976
	12.1	29,500	4 May 1994
857.0	389.0	60,000	7 Aug 1975
	733.9	38,000	17 Oct 1976
	721.0	29,500	4 May 1994
853.0	83.2	60,000	7 Aug 1975
	277.1	38,000	17 Oct 1976
	155.6	29,500	4 May 1994
849.0	101.2	60,000	7 Aug 1975
	188.2	38,000	17 Oct 1976
	133.9	29,500	4 May 1994
845.0	531.7	60,000	7 Aug 1975
	672.5	38,000	17 Oct 1976
	429.3	29,500	4 May 1994

Table 26
Island Exposure Versus Discharge
Gavins Point Reach

River Mile	Island Area, acres	Discharge, cfs	Date
807.0	present	–[1]	July 1956 (mosaic)
	18.1	46,500	29 Aug 1972
	174.8 (ice)	15,000	22 Dec 1977
	127.6	32,000	6 June 1981
	124.5	30,600	5 May 1994
804.0	present	–[1]	July 1956 (mosaic)
	0.0	46,500	29 Aug 1972
	130.0 (ice)	15,000	22 Dec 1977
	64.7	32,000	6 June 1981
	72.5	30,600	5 May 1994
799.5	present	–[1]	July 1956 (mosaic)
	707.5	46,500	29 Aug 1972
	913.5 (ice)	15,000	22 Dec 1977
	777.2	32,000	6 June 1981
	725.5	30,600	5 May 1994
785.0	present	–[1]	July 1956 (mosaic)
	656.1	46,500	29 Aug 1972
	717.0	15,000	22 Dec 1977
	632.4	32,000	6 June 1981
	569.7	30,600	5 May 1994
771.0	present (bar)	–[1]	July 1956 (mosaic)
	164.7	46,500	29 Aug 1972
	243.0	15,000	22 Dec 1977
	171.5	32,000	6 June 1981
	120.5	30,600	5 May 1994
760.0	not present	–[1]	July 1956 (mosaic)
	12.2	46,500	29 Aug 1972
	13.2 (ice)	15,000	22 Dec 1977

(Continued)

[1] Unknown.

Table 26 (Concluded)

River Mile	Island Area, acres	Discharge, cfs	Date
760.0 (Cont'd)	12.5	32,000	6 June 1981
	11.7	30,600	5 May 1994
755.0	present (bar)	–[1]	July 1956 (mosaic)
	29.2	46,500	29 Aug 1972
	43.1 (ice)	15,000	22 Dec 1977
	11.2	32,000	6 June 1981
	11.4	30,600	5 May 1994

[1] Unknown.

Table 27
Suspended Sediment and Turbidity Data Inventory

Location	Record Type	Gage Location		Gage No.	Data Type	Begin	End
Aowa Creek	STORET	Ponca	NE	301202	Turbidity	1973	1977
Apple Creek	STORET	Hwy 1804 Br.	ND	380057	Dissolved solids	1974	1992
Beaver Creek	STORET	Nenominee	NE	301198	Turbidity	1973	1977
Bow Creek	STORET	Wynot	NE	301199	Turbidity	1973	1977
Cannonball River	Water Quality Data	Breien	MT	06354000	Q_s,%<0.062, Turbidity	1972	1991
Ft Peck Lake	STORET	Near Dam	MT	29FPL1	Turbidity	1976	1995
Heart River	Water Quality Data	Mandan	ND	06349000	Q_s,%<0.062, Turbidity	1972	1993
Heart River	Water Quality Data	Mandan	ND	06349000	Dissolved solids, Q_s, %<0.062, JTU/FTU	1972	1993
Heart River	Water Quality Data	Mandan	ND	370008	Dissolved solids, JTU/FTU	1969	1975
James River	STORET	Scotland	SD	06478500	Dissolved solids, Q_s, %<0.062, JTU/FTU	1956	1994
Knife River	Water Quality Data	Hazen	ND	06340500	Q_s,%<0.062, Turbidity	1974	1993
Lake Sakakawea	STORET	Dam Site	ND	381201	Turbidity	1974	
Lake Sakakawea	STORET	Releases	ND	370002	Turbidity	1968	1976
Lake Yankton, Missouri River	STORET	Near Gavins Pt Dam	SD	46CELY	FTU	1957	1975
Milk River	Water Quality Data	Nashua	MT	06174500	Q, Q_s, %<0.062, NTU	1974	1994
Milk River	STORET	Nashua	MT	06174500	Dissolved solids, Turbidity, Q_s, %<0.062	1959	1994
Missouri River	STORET	Bismarck	ND	06342500	Dissolved solids, Q_s, %<0.062, JTU/FTU	1970	1993
Missouri River	STORET	Bismarck	ND	370028	Dissolved solids	1971	1974
Missouri River	Water Quality Data	Bismarck	ND	06342500	Q_s,%<0.062, Turbidity	1970	1991
Missouri River	STORET	Bismarck	ND	370033	Turbidity	1957	1972
Missouri River	Water Quality Data	Culbertson	MT	06185500	Q, Q_s, %<0.062, NTU	1978	1986
Missouri River	Water Quality Data	Culbertson	MT	06185500	Q, Q_s, Bed Material	1971	1978
Missouri River	STORET	Culbertson	MT	06185500	Dissolved solids, Q_s, %<0.062, JTU	1968	1986
Missouri River	STORET	Garrison Dam	ND	06338490	Dissolved solids, Q_s, %<0.062, JTU/FTU	1974	1986

(Continued)

Table 27 (Concluded)

River	Record Type	Gage Location		Gage No.	Data Type	Begin	End
Missouri River	STORET	Garrison Dam	ND	06338490	Dissolved solids, Turbidity	1974	1995
Missouri River	Water Quality Data	Garrison Dam	ND	06338490	Q_s,%<0.062, Turbidity	1974	1995
Missouri River	STORET	Stanton	ND	06340700	Dissolved solids, Q_s, JTU (4)	1988	1991
Missouri River	Water Quality Data	Stanton	ND	06340700	Q_s,%<0.062, Turbidity	1988	1991
Missouri River	STORET	Yankton Water Dpt	SD	460032	Dissolved solids, Q_s, %<0.062, JTU/FTU	1957	1975
Missouri River	STORET	Yankton	SD	06467500	Dissolved solids, Q_s, %<0.062, JTU/FTU	1956	1974
Niobrara River	STORET	Niobrara	NE	300901	Turbidity	1961	1977
Niobrara River	Water Quality Data	Verdel	NE	06465500	Q_s, %<0.062	1971	1994
Niobrara River	Water Quality Data	Verdel	NE	06465500	Dissolved solids, Q_s, %<0.062 JTU/FTU	1972	1994
Niobrara River	Suspended Sediment Concentration	Verdel	NE	06465500	Daily mean concentration, mg/l	1971	1981
Painted Woods Creek	STORET	Wilton	ND	06341800	Dissolved solids, JTU/FTU	1959	1994
Painted Woods Creek	Water Quality Data	Wilton	ND	06341800	Q_s,%<0.062, Turbidity	1970	1995
Ponca Creek	STORET	Verdel	NE	06453600	Dissolved solids, Q_s, %<0.062, JTU/FTU	1975	1980
Ponca Creek	STORET	Verdel	NE	300917	Turbidity	1968	1977
Prairie Elk Creek	Water Quality Data	Oswego	MT	06175540	Q, Q_s, %<0.062	1975	1979
Prairie Elk Creek	STORET	Oswego	MT	06175540	Q_s, Turbidity, %<0.062	1975	1979
Redwater River	STORET	Vida	MT	06177825	Dissolved solids, Q_s, JTU, %<0.062	1975	1985
Redwater River	Water Quality Data	Vida	MT	06177825	Q, Q_s	1975	1985
Sand Creek	Water Quality Data	Wolf Point	MT	06175580	Q, Q_s	1975	1977
Sand Creek	STORET	Wolf Point	MT	06175580	Dissolved solids, Q_s, JTU	1975	1977
Square Butte Creek	STORET	Center	ND	06342260	Dissolved solids	1973	1993
Turtle Creek	STORET	Turtle Lake	ND	06341400	Dissolved solids, JTU (6)	1972	1978
Turtle Creek	Water Quality Data	Washburn	ND	06341410	Q_s,%<0.062, Turbidity	1987	1995
Vermillion River	STORET	Wakonda	SD	06479000	Dissolved solids, Q_s, %<0.062, JTU/FTU	1960	1989

Table 28
Summary of Channel and Catchment Characteristics

Parameter	Fort Peck Reach	Garrison Reach
Date of dam closure	1937	1953
Drainage area above dam, acres	36,800,000	116,100,000
Base flow discharge,[1] cfs	7,300	18,400
Peak annual discharge,[2] cfs	11,100	31,500
Average channel gradient	0.000174	0.000112
Bed material median diameter, mm	0.25-10	0.25-12
Mean bank material shear strength,[3] lbf/ft^2	108.6	108.6

[1] Refers to minimum average monthly discharge (1893-1993) for the CWCP.
[2] Refers to maximum average monthly discharge (1893-1993) for the CWCP.
[3] Refers to conditions measured during field reconnaissance, not worst-case conditions.

Table 29
Comparison of Values of Estimated Geotechnical Characteristics

Parameter	Upper Missouri	Bluff Line Streams[1]	Red River[2]
Field Conditions			
Unit weight, lbf/ft^3	115.9	134.4	not stated
Shear strength, lbf/ft^2	108.6	not stated	not stated
Cohesion, lbf/ft^2	unknown	89.8	not stated
Friction angle, deg	unknown	40	not stated
Worst-Case Conditions			
Unit weight, lbf/ft^3	134.4	140.7	120.0
Cohesion, lbf/ft^2	83.5	77.3	59.9
Friction angle, deg	20	20	27

Note: Comparison of characteristics of estimated values of Missouri River bank materials with measured geotechnical characteristics of streams in the bluff line hills of Northern Mississippi and the Red River, Louisiana.
[1] Data from Table 11 of Thorne, Murphey, and Little (1981).
[2] Data from Table 4.2 of Thorne (1992).

Table 30
Regression Relations Summarizing Temporal Trends of Mean Bed Elevation and Channel Bed Width at Bank Stability Analysis Sites

Site	River Mile	Mean Bed Elevation Z, ft	Channel Bed Width W, ft
Fort Peck Reach (t = years since 1955; n = number of data points used in regressions = 5)			
1	1688	Z = 1947	W = 1227
2	1682.4	Z = 1941+0.57 LOG (t) (r^2= 0.94)	W = 1162 + 29.37 LOG (t) (r^2= 1.00)
3	1674.5	Z = 1936 $10^{-1.11043E-5t}$	W = 1637
4	1669	Z = 1926	W = 801
5	1647.5	Z = 1910	W = 1050
6	1642.5	Z = 1909	W = 1339
7	1638.2	Z = 1902	W = 1155
8	1631.3	Z = 1897 - 0.63 LOG(t) (r^2 = 0.77)	W = 1483 + 56.18 LOG (t) (r^2= 0.94)
9	1619.9	Z = 1887 - 0.82 LOG(t) (r^2 = 0.94)	W = 1063
10	1616.5	Z = 1884	W = 1159 + 109.83 LOG (t) (r^2= 0.49)
11	1612.8	Z = 1881	W = 1158 + 115.51 LOG (t) (r^2= 0.49)
12	1608.4	Z = 1878	W = 1387
13	1604	Z = 1875 - 0.0416 LOG(t) (r^2 = 0.94)	W = 1310
Garrison Reach (t = years since 1953; n = number of data points used in regressions = 8)			
14	1386	Z = 1671 - 7.57 LOG(t) (r^2 = 0.93)	W = 1074 + 174.10 LOG (t) (r^2= 0.84)
15	1379.8	Z = 1669 - 3.74 LOG(t) (r^2 = 0.63)	W = 1864
16	1376.6	Z = 1667 - 4.63 LOG(t) (r^2 = 0.95)	W = 2965 + 121.06 LOG (t) (r^2= 0.79)
17	1358.9	Z = 1656 - 4.46 LOG(t) (r^2 = 0.84)	W = 1660
18	1346.4	Z = 1644 - 1.89 LOG(t) (r^2 = 0.62)	W = 1339

Note: Regression relations for Fort Peck and Garrison Reaches are based on data for periods 1955-1978 and 1953-1985, respectively.

Table 31
Cumulative Values of Future Bed Degradation and Bank Toe Erosion For CWCP and Best-Case PA Compared To Present (1995) Estimated Conditions

Site	River Mile	Cumulative Bed Degradation ΔZ, ft					Cumulative Bank Toe Erosion $\Delta W/2$, ft				
		1996	2000	2005	2015	2045	1996	2000	2005	2015	2045
Fort Peck Reach											
1	1688.0	0	0	0	0	0	0	0	0	0	0
2	1682.4	0±0.07	-0.03±0.2	-0.03±0.36	-0.10±0.62	-0.20±1.25	0.16±0.07	0.79±0.20	1.48±0.36	2.66±0.62	5.28±1.21
3	1674.5	0.03±0.26	0.26±0.26	0.49±0.43	0.98±0.72	2.46±1.18	0	0	0	0	0
4	1669.0	0	0	0	0	0	0	0	0	0	0
5	1647.5	0	0	0	0	0	0	0	0	0	0
6	1642.5	0	0	0	0	0	0	0	0	0	0
7	1638.2	0	0	0	0	0	0	0	0	0	0
8	1631.3	0±0.33	0.03±1.54	0.10±2.92	0.13±5.25	0.23±10.3	0.29±0.07	1.48±0.26	2.79±0.49	5.05±0.85	10.07±1.71
9	1619.9	0±0.03	0.03±0.16	0.07±0.33	0.10±0.59	0.26±1.15	0	0	0	0	0
10	1616.5	0	0	0	0	0	0.59±0.85	2.89±4.17	5.45±7.84	9.88±14.17	19.65±27.86
11	1612.8	0	0	0	0	0	0.59±0.20	2.95±0.98	5.68±1.87	10.33±3.45	20.64±6.59
12	1608.4	0	0	0	0	0	0	0	0	0	0
13	1604.0	0±0.33	0.0±.10	0±0.20	0.03±0.33	0.03±0.62	0	0	0	0	0
Garrison Reach											
14	1386.0	0.07±0.03	0.36±0.13	0.69±0.23	1.28±0.39	2.56±0.79	0.89±0.03	4.26±0.59	8.07±1.16	14.73±2.00	29.66±3.97
15	1379.8	0.03±0.03	0.20±0.13	0.36±0.23	0.62±0.43	1.28±0.85	0	0	0	0	0
16	1376.6	0.03±0.20	0.23±0.49	0.42±0.85	0.79±1.48	1.57±2.82	0.62±0.10	2.95±0.49	5.61±0.89	10.24±1.64	20.60±3.22
17	1358.9	0.03±0.03	0.23±0.13	0.39±0.26	0.75±0.53	1.51±1.01	0	0	0	0	0
18	1346.4	0±0.03	0.10±0.13	0.16±0.23	0.30±0.39	0.66±0.79	0	0	0	0	0

Note: 1995 values are extrapolated beyond last survey date of 1978 and 1985 for Fort Peck and Garrison Reaches, respectively. Error estimates are obtained from uncertainty of extrapolated values of 95 percent confidence intervals. Negative values indicate aggradation.

Table 32
Annual Bed Material Load for CWCP and PA from Flow Duration Curves

Period	Percent Exceeded	Discharge, cfs		Bed Material, tons	
		CWCP	PA	CWCP	PA
Fort Peck Reach					
1	5	15,900	16,500	80,920	82,676
2	10	14,100	14,800	150,447	155,042
3	20	12,100	12,400	135,942	138,264
4	30	11,100	10,500	127,762	122,493
5	40	9,700	9,000	114,978	107,875
6	50	8,700	8,200	104,661	99,048
7	60	7,700	7,700	93,082	93,082
8	70	6,700	6,000	79,891	69,427
9	80	5,500	5,000	61,176	52,138
10	90	3,900	3,600	28,578	20,988
11	95	3,600	3,100	10,494	3,404
			Total	987,931	944,438
Garrison Reach					
1	5	41,700	42,100	329,869	332,047
2	10	37,200	39,600	607,630	636,159
3	20	29,100	32,200	495,574	541,766
4	30	25,300	26,700	431,721	456,298
5	40	22,600	21,700	380,225	361,682
6	50	21,000	20,000	346,719	324,456
7	60	18,700	16,300	293,788	231,110
8	70	16,600	15,500	239,432	208,147
9	80	15,500	12,200	208,147	98,904
10	90	12,400	11,400	106,324	67,956
11	95	11,700	10,800	39,905	21,643
			Total	3,479,335	3,280,167

Table 33
Annual Bed Material Load for CWCP and PA from Plates 2 and 3

Month	Days	Discharge, cfs		Bed Material, tons	
		CWCP	PA	CWCP	PA
Fort Peck Reach					
January	31	10,090	9,430	100,827	95,379
February	28	10,130	9,420	91,357	86,072
March	31	8,110	8,510	83,234	87,112
April	30	7,280	7,700	72,134	76,506
May	31	8,960	9,950	91,261	99,702
June	30	8,140	14,090	80,837	123,600
July	31	7,660	8,270	78,637	84,808
August	31	7,710	7,360	79,161	75,149
September	30	10,620	8,800	101,565	86,913
October	31	11,100	8,650	108,510	88,426
November	30	9,080	6,910	89,354	68,069
December	31	8,990	8,610	91,531	88,052
			Total	1,068,409	1,060,057
Garrison Reach					
January	31	21,730	21,030	303,717	295,027
February	28	21,870	20,080	280,186	250,295
March	31	20,920	23,260	292,995	334,087
April	30	21,930	26,740	301,227	375,601
May	31	22,240	28,310	316,708	410,232
June	30	21,730	29,050	297,791	406,676
July	31	23,230	23,730	333,586	341,839
August	31	21,620	20,270	305,750	280,762
September	30	31,490	25,680	436,925	360,431
October	31	26,110	20,810	378,881	290,952
November	30	18,360	12,720	234,588	96,946
December	31	19,480	18,010	265,356	234,948
			Total	3,751,709	3,677,797

Table 34
Accuracy of Selected Riverbank Stability Analysis

Analysis	Mean Predicted Factor of Safety	Mean Observed Factor of Safety
Darby and Thorne (in preparation)	1.43	1.0
Osman and Thorne (1988)	1.82	1.0
Lohnes and Handy (1968)	1.83	1.0
Huang (1983)	3.26	1.0

Table 35
Lengths of Unstable and Stable Bank Lines and Number of Study Sites in Each Bank Failure Category

Category	Fort Peck Reach miles	Garrison Reach miles	Number of Study Sites
Study reach length	179.0	69.6	Not applicable
Sampled bank lines	116.0	26.8	18
Stable bank line	50.1	15.9	9
Unstable bank line	66.2	10.9	9
Planar failure	29.5 (45%)[1]	6.5 (59%)[1]	18
Pop-out failure	22.1 (33%)[1]	1.5 (14%)[1]	0
Cantilever failure	12.9 (19%)[1]	3.0 (27%)[1]	0
Rotational failure	1.7 (3%)[1]	0.0 (0%)[1]	0

Note: Lengths based on September 1995 field reconnaissance.
[1] Percentage based on length of unstable banks, and the numbers of stable and unstable sites are based on the predictions resulting from the study analysis.

Table 36
Darby-Thorne Stability Analysis for CWCP

Site	RM	H, ft	H^1, ft	α, deg	K, ft	K_h, ft	γ, lb/ft^2	c, lb/ft^2	ϕ, deg	WSE, ft	GWSE, ft	FS	BW, ft	β, deg	V, ft^3/ft	Predicted	Observed
Fort Peck Reach																	
1	1688.0	9.55	9.55	65	0	3.51	134.4	83.5	20	0.75H-0.98	0.75H	1.02	2.76	52.9	4.95	Upper Bank	Upper Bank
2	1682.4	12.00	12.00	78	0	0.00	134.4	83.5	20	0.75H-0.98	0.75H	0.59	8.17	49.0	46.82	Unstable	Unstable
3	1674.5	14.50	14.50	60	0	6.50	134.4	83.5	20	0.75H-0.98	0.75H	0.63	5.05	54.5	17.98	Upper Bank	Unstable
4	1669.0	10.99	10.99	50	0	0.00	134.4	83.5	20	0.75H-0.98	0.75H	1.13	6.14	35.0	32.08	Marginal	Marginal
5	1647.5	16.01	16.01	45	0	8.56	134.4	83.5	20	0.75H-0.98	0.75H	1.19	2.17	52.5	3.44	Marginal	Stable
6	1642.5	11.12	11.12	80	0	2.69	134.4	83.5	20	0.75H-0.98	0.75H	0.59	7.35	50.0	40.58	Unstable	Unstable
7	1638.2	19.03	19.03	60	0	1.51	134.4	83.5	20	0.75H-0.98	0.75H	0.54	6.63	54.6	30.89	Upper Bank	Unstable
8	1631.3	10.01	10.01	58	0	2.69	134.4	83.5	20	0.75H-0.98	0.75H	1.30	2.20	51.5	3.01	Stable	Marginal
9	1619.9	4.99	4.99	33	0	1.51	134.4	83.5	20	0.75H-0.98	0.75H	2.22	1.02	52.5	0.75	Stable	Stable
10	1616.5	6.00	6.00	40	0	0.98	134.4	83.5	20	0.75H-0.98	0.75H	3.51	0.62	52.5	0.32	Stable	Stable
11	1612.8	14.99	14.99	78	0	5.25	134.4	83.5	20	0.75H-0.98	0.75H	0.47	10.47	52.8	75.89	Unstable	Unstable
12	1608.4	25.98	25.98	40	0	7.81	134.4	83.5	20	0.75H-0.98	0.75H	0.56	6.33	54.5	27.88	Upper Bank	Upper Bank
13	1604.0	8.01	8.01	55	0	0.98	134.4	83.5	20	0.75H-0.98	0.75H	1.19	5.18	37.7	17.44	Marginal	Stable
Garrison Reach																	
14	1386.0	18.01	18.01	45	0	2.99	134.4	83.5	20	0.75H-2.1	0.75H	1.36	1.84	52.5	2.48	Stable	No photo
15	1379.8	18.01	18.01	38	0	4.49	134.4	83.5	20	0.75H-2.1	0.75H	1.09	2.46	52.5	4.41	Upper Bank	No photo
16	1376.6	19.00	18.01	60	0	8.99	134.4	83.5	20	0.75H-2.1	0.75H	0.52	7.28	54.4	37.24	Upper Bank	No photo
17	1358.0	8.01	8.01	50	0	2.00	134.4	83.5	20	0.75H-2.1	0.75H	1.73	1.35	52.5	1.40	Stable	No photo
18	1346.4	12.40	12.40	50	0	1.51	134.4	83.5	20	0.75H-2.1	0.75H	2.28	1.44	47.0	1.08	Stable	No photo

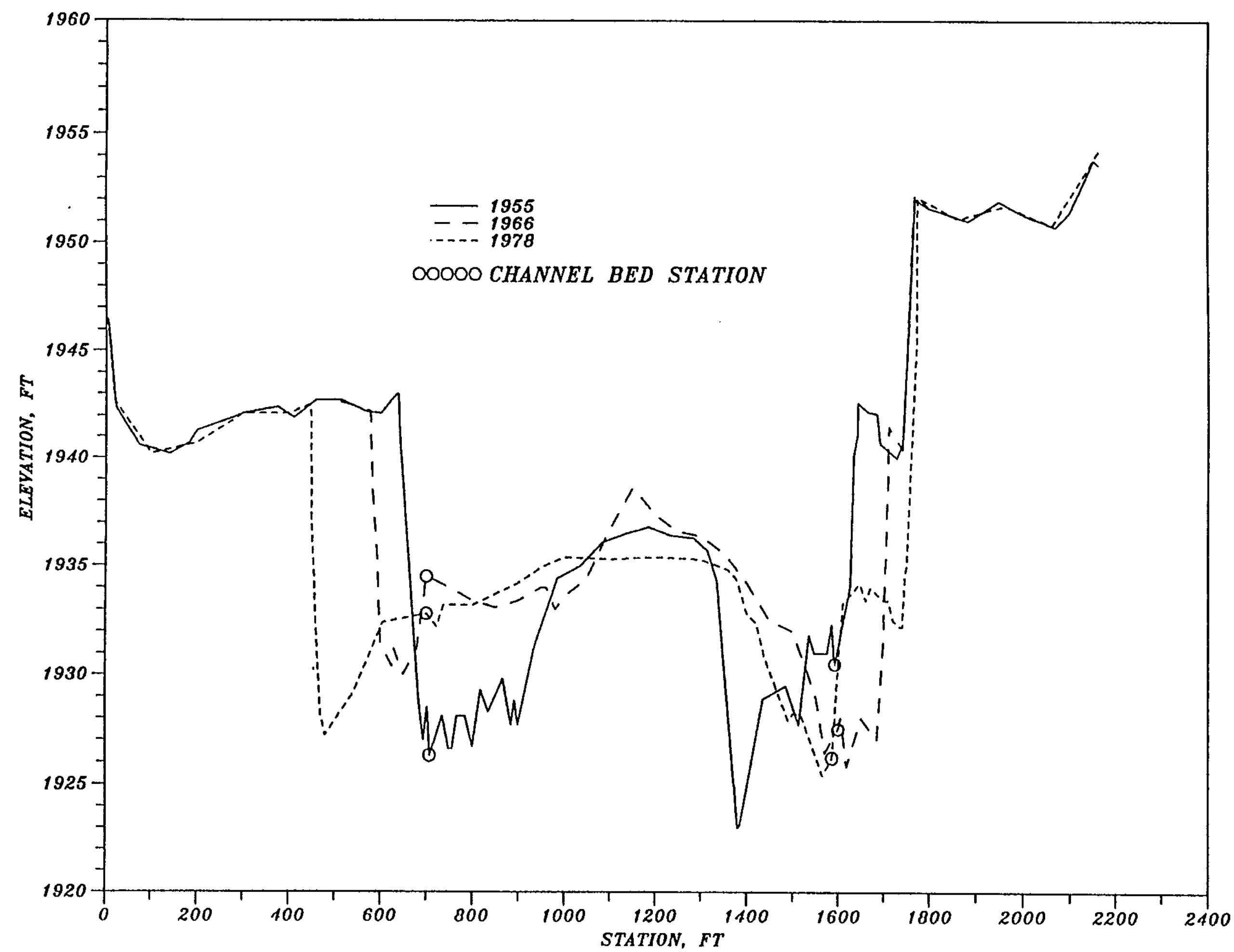
Note: Input data parameters were based on field measured values during September 1995 field reconnaissance. They are shown in Plate 84 and defined as follows: RM = river mile; H = bank height; H^1 = bank height above failure plane; α = bank slope; K = tension crack depth; K_h - relic tension crack depth; γ = specific weight; c = soil cohesion; ϕ = friction angle; WSE = water surface elevation; GWSE = ground water surface elevation; FS = factor of safety; BW - top bank failure block width; β = failure plane angle; V = velocity.

Table 37
Number of Sites in Each Stability Category

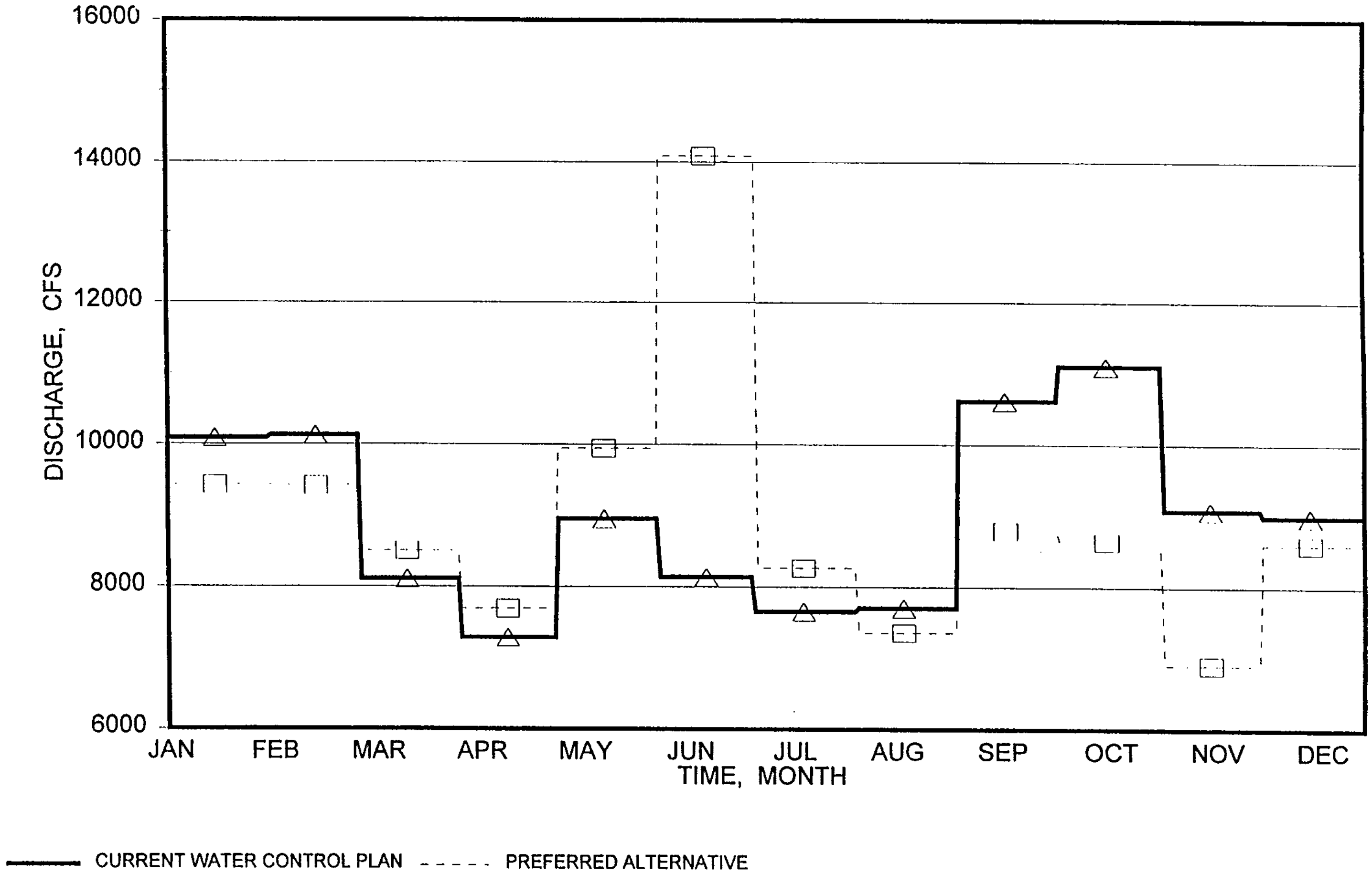
Category	1 Year (1996)	5 years (2000)	10 years (2005)	20 Years (2015)	50 years (2045)
Unstable	3 (17%)	3-4 (17-22%)	4-7 (22-38%)	5-7 (28-39%)	5-7 (28-38%)
Upper bank	6 (33%)	6 (33%)	5 (28%)	6 (33%)	5 (28%)
Marginal	4 (22%)	4-5 (22-28%)	3-4 (17-22%)	4 (22%)	3-4 (16-22%)
Stable	5 (28%)	3-5 (17-28%)	3-5 (17-28%)	1-3 (6-17%)	2-5 (12-28%)

Table 38
Average Annual Bank Material Eroded and Existing Bank Protection

Period	Bank Volume, acre-ft/year	Cumulative Bank Protection, %
Fort Peck Reach		
1955-1966	1,639	0
1966-1978	870	0
Garrison Reach		
1956-1976	1,422	7
1976-1985	772	21
Fort Randall Reach		
1954-1975	642	0
1975-1985	310	9
Gavins Point Reach		
1960-1974	3,919	5
1974-1986	3,161	27

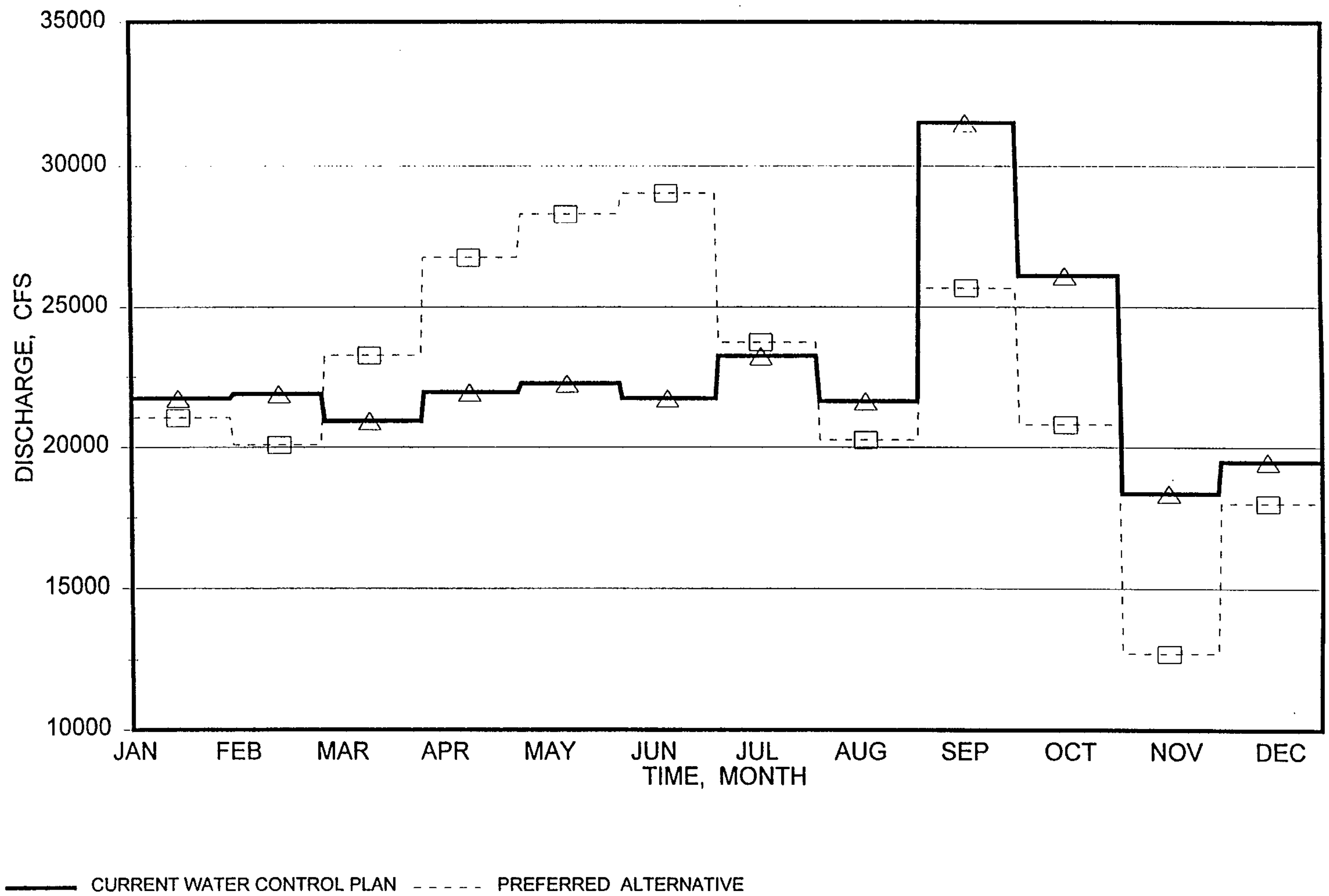


EXAMPLE OF CHANNEL CROSS SECTION
FORT PECK REACH



1898 - 1993 AVERAGE MONTHLY DAM RELEASE
FORT PECK REACH

Plate 2



1898 - 1993 AVERAGE MONTHLY DAM RELEASE
GARRISON REACH

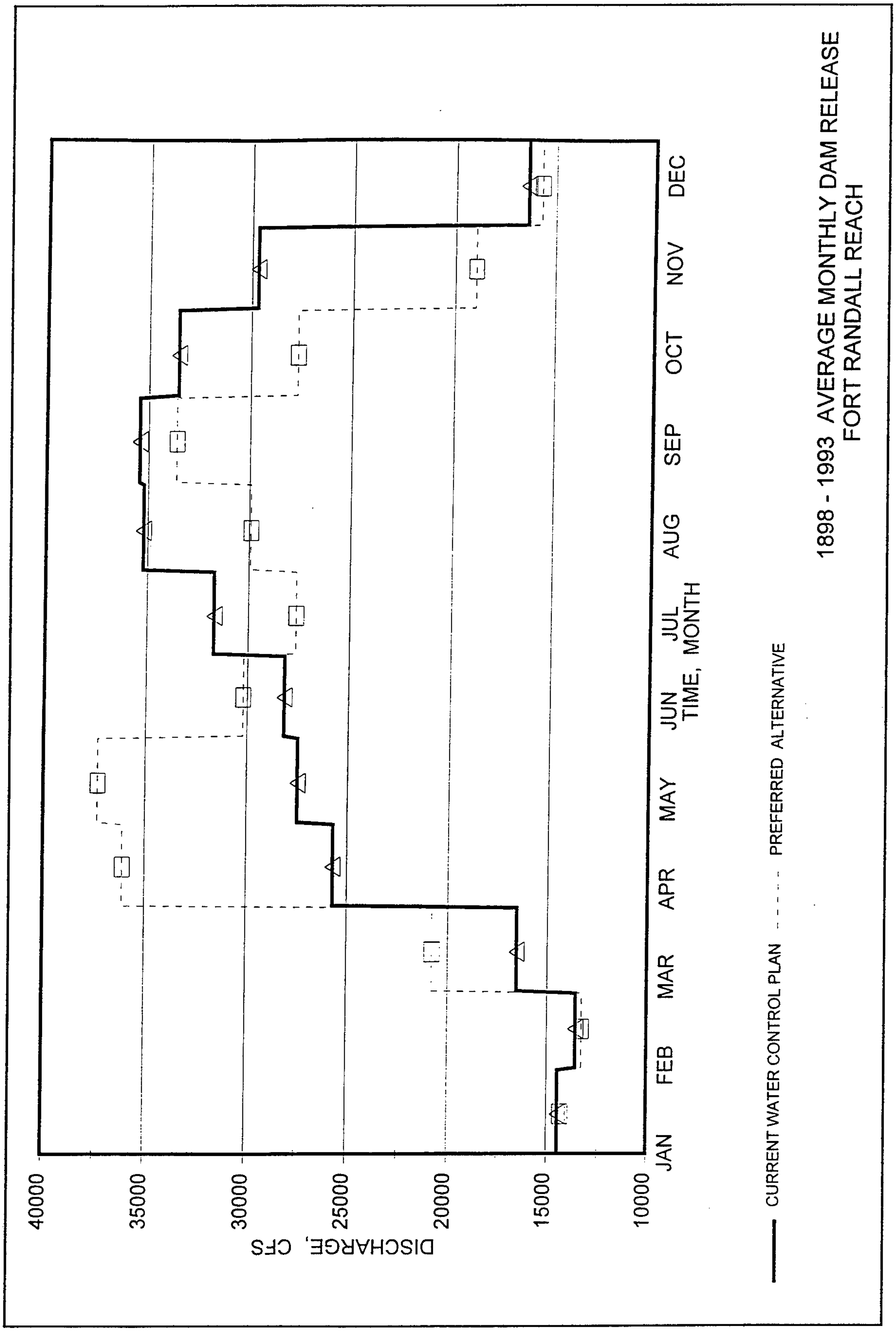
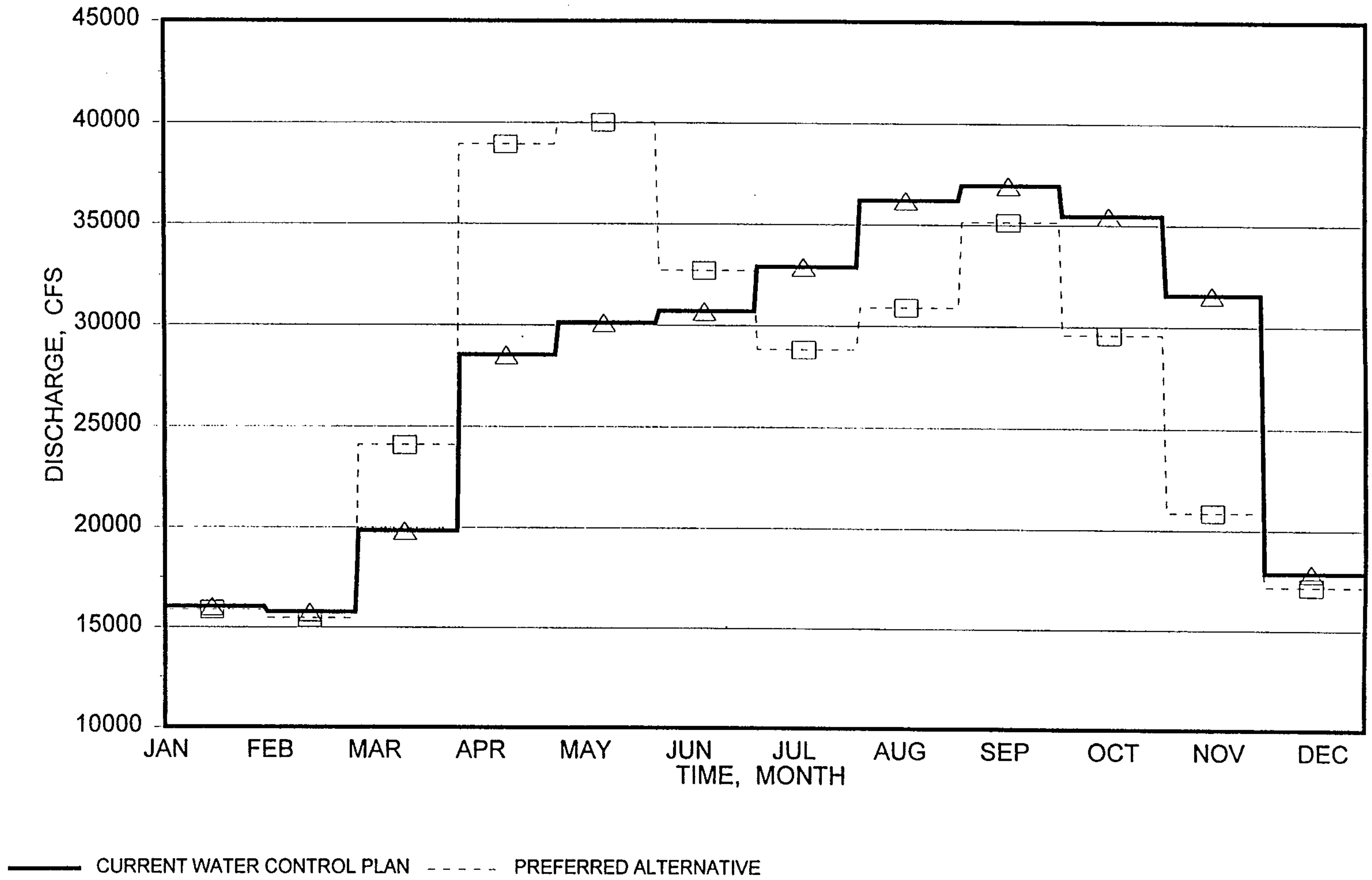

1898 - 1993 AVERAGE MONTHLY DAM RELEASE
FORT RANDALL REACH
DISCHARGE, CFS
40000
35000
30000
25000
20000
15000
10000
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC
TIME, MONTH
CURRENT WATER CONTROL PLAN
PREFERRED ALTERNATIVE

Plate 4



1898 - 1993 AVERAGE MONTHLY DAM RELEASE
GAVINS POINT REACH

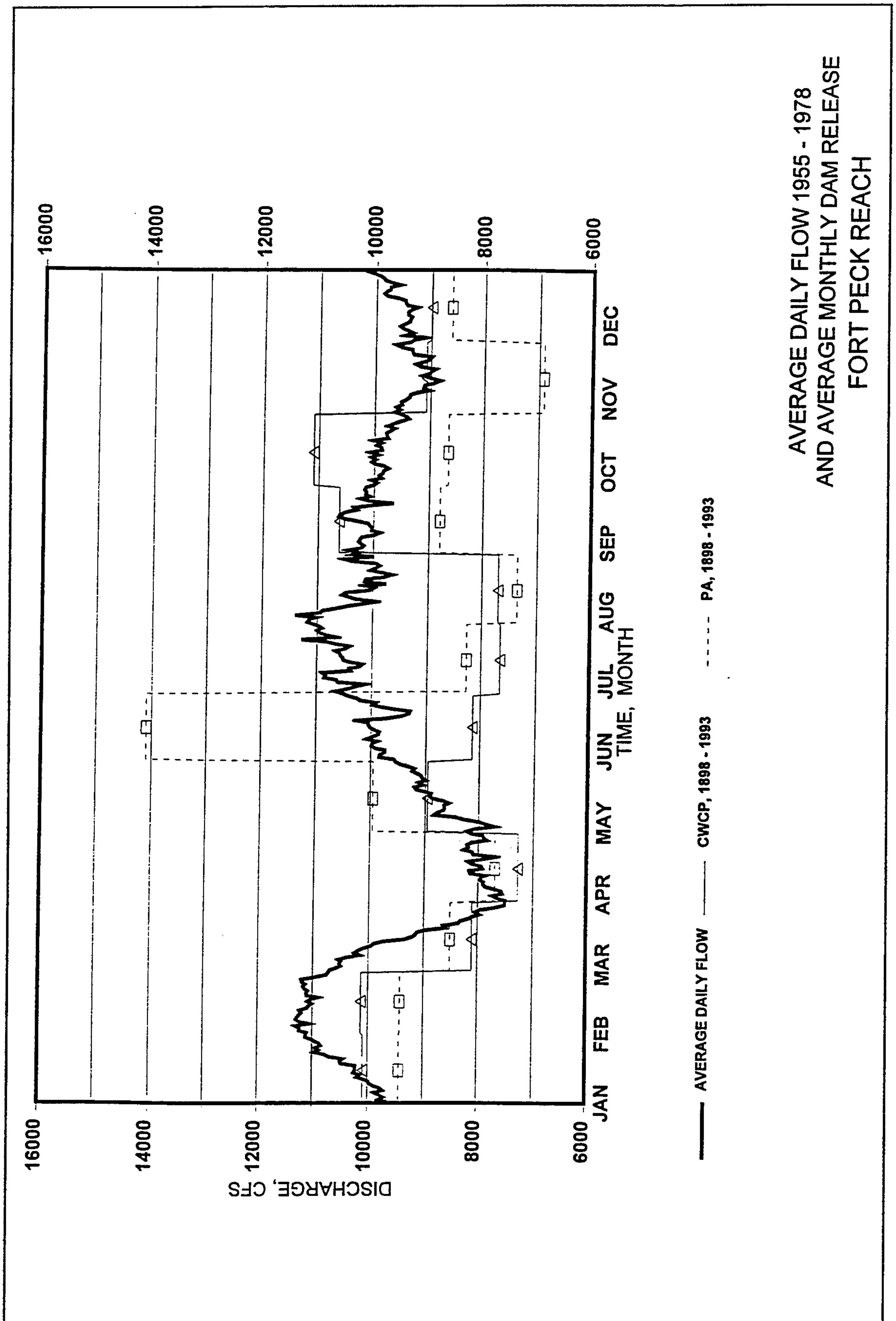
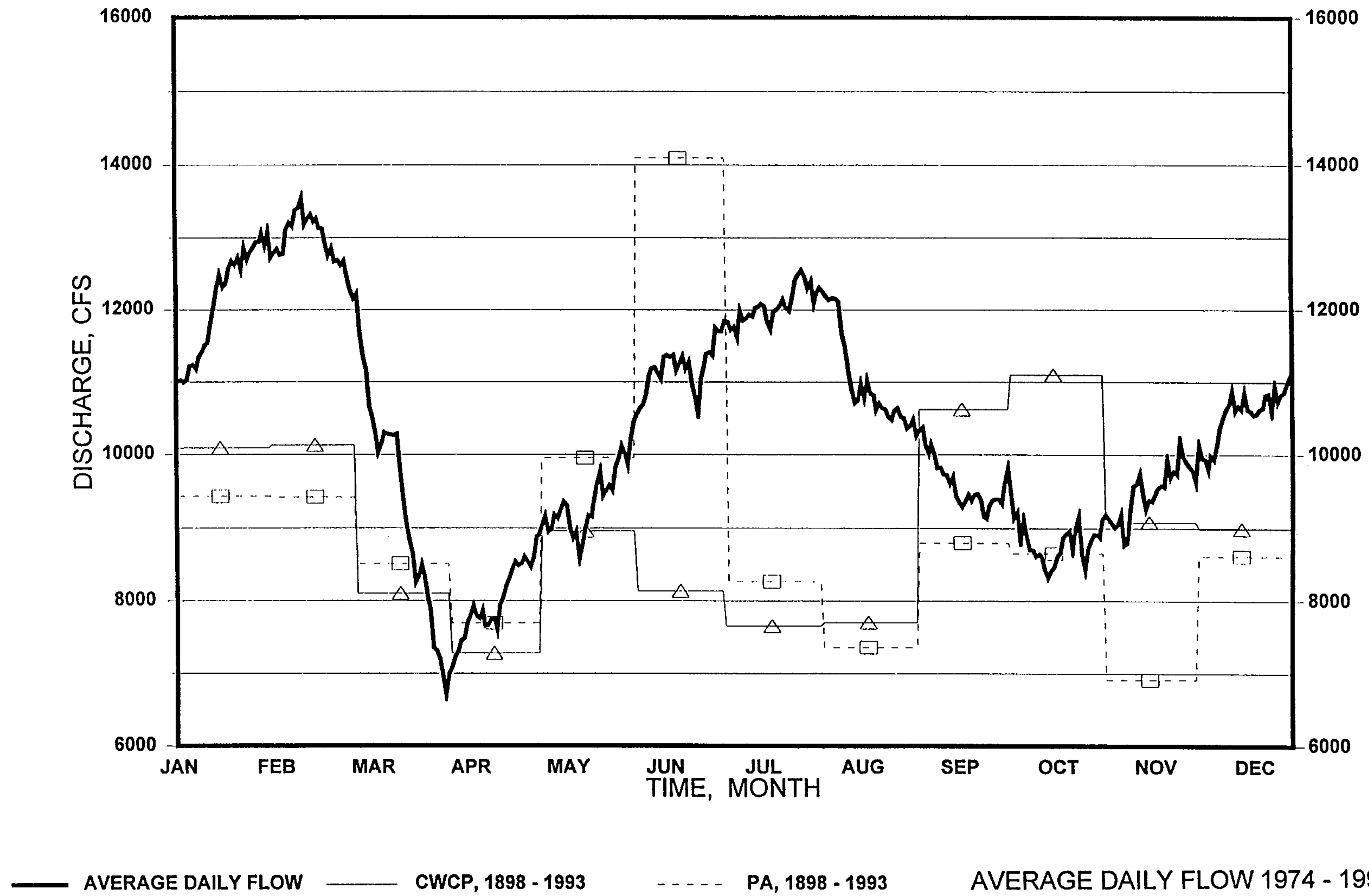
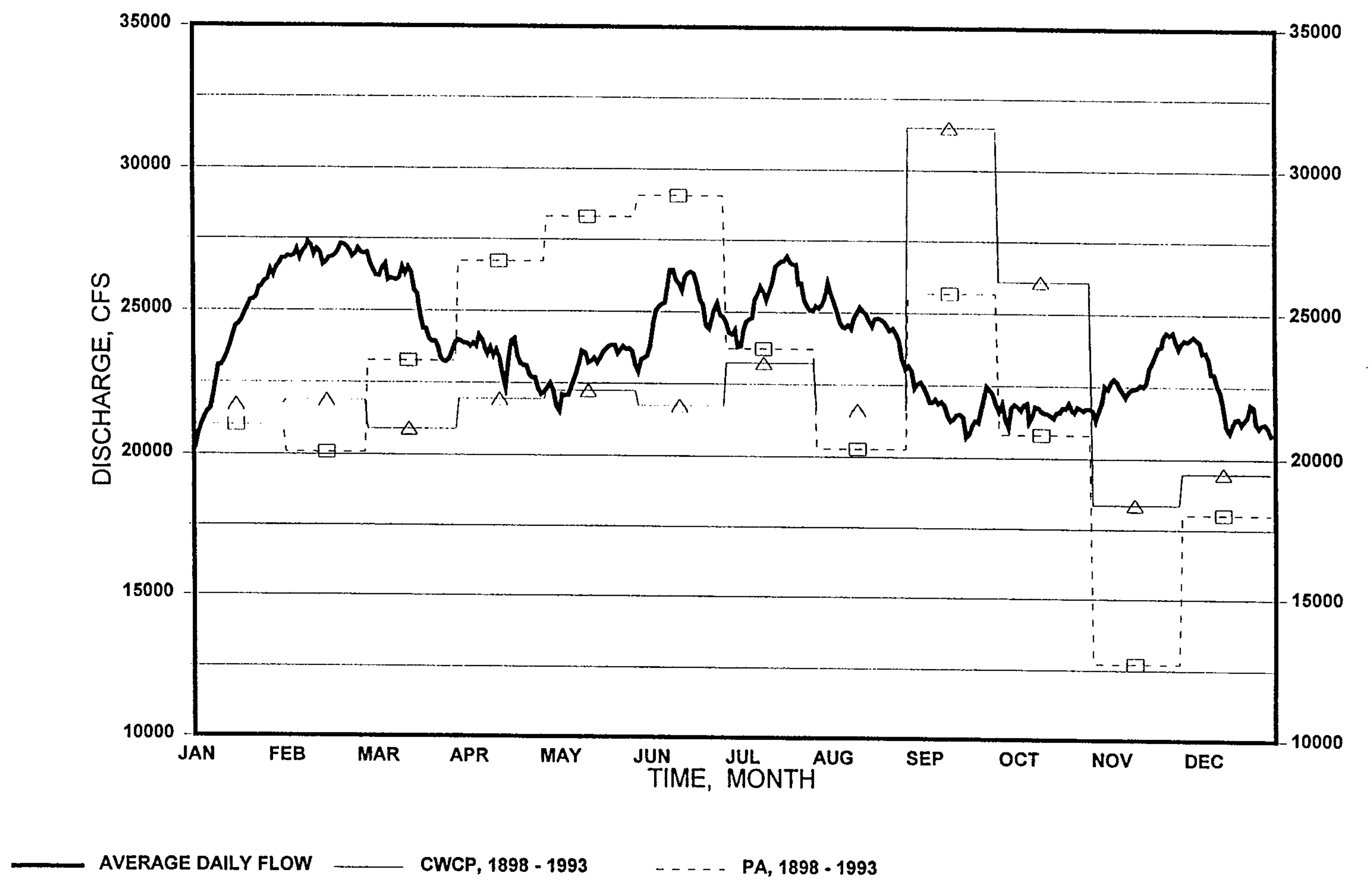

AVERAGE DAILY FLOW 1955 - 1978
AND AVERAGE MONTHLY DAM RELEASE
FORT PECK REACH
DISCHARGE, CFS
16000
14000
12000
10000
8000
6000
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC
TIME, MONTH
AVERAGE DAILY FLOW
CWCP, 1898 - 1993
PA, 1898 - 1993

Plate 6

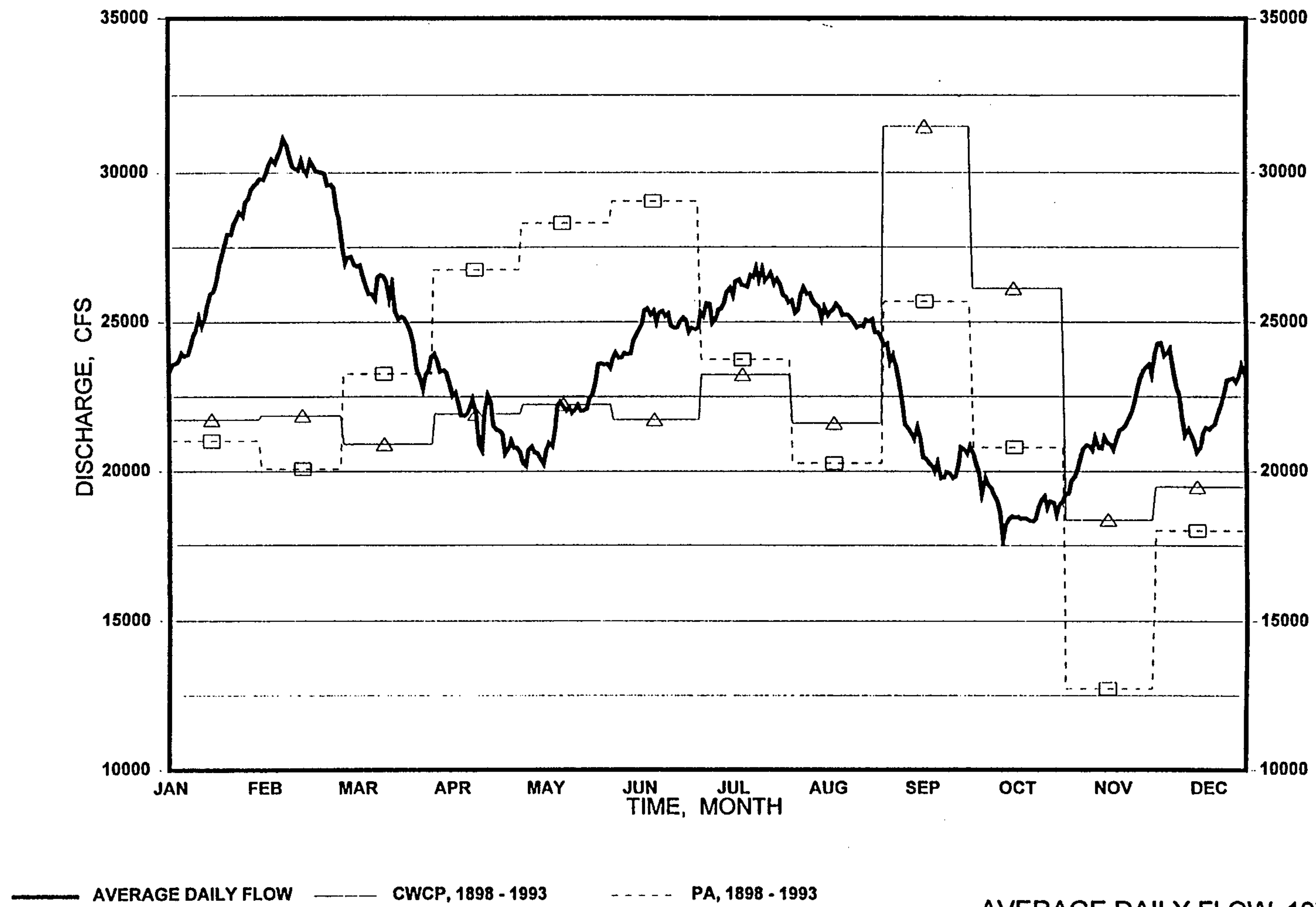


AVERAGE DAILY FLOW 1974 - 1990
AND AVERAGE MONTHLY DAM RELEASE
FORT PECK REACH

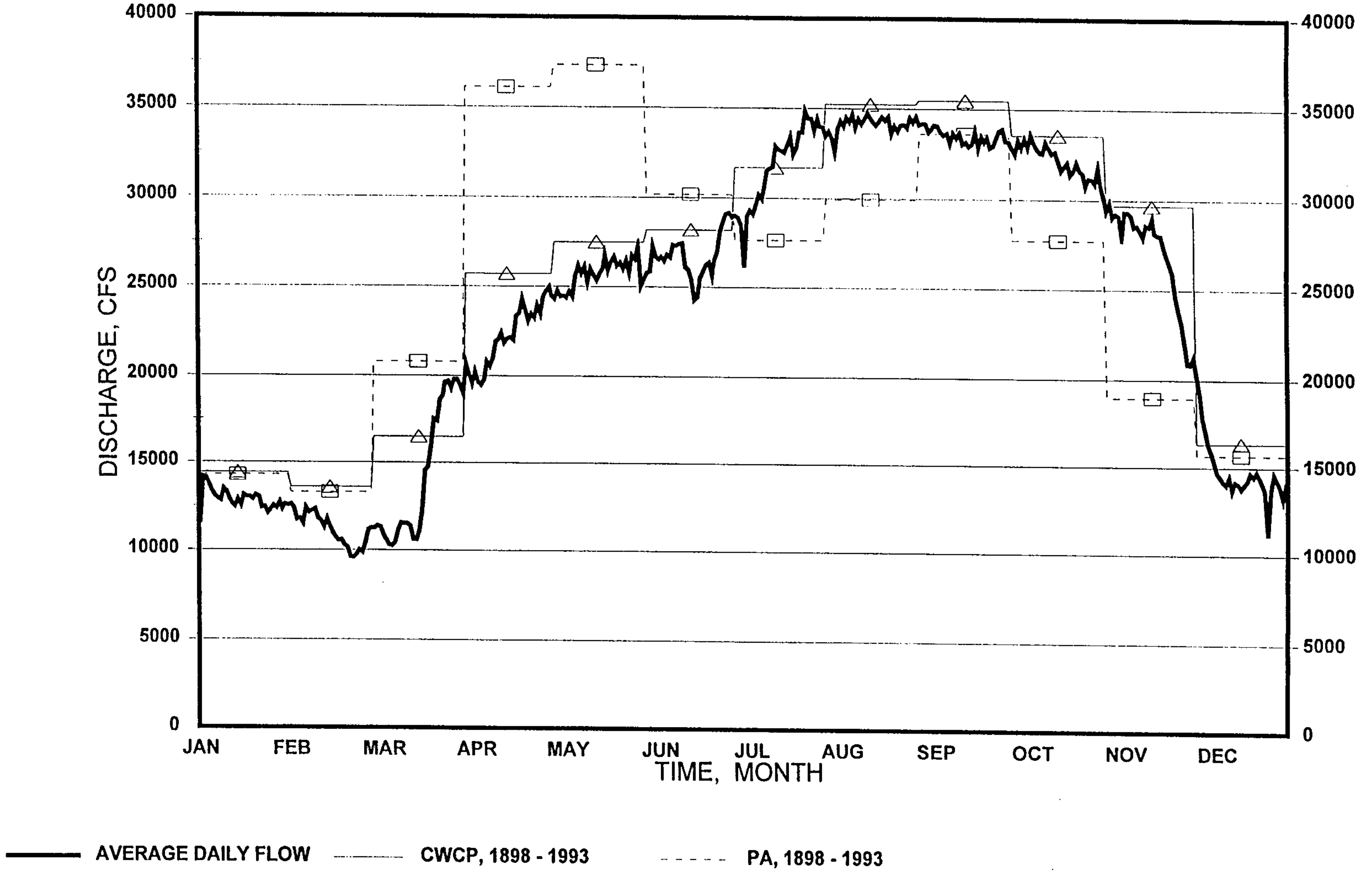


AVERAGE DAILY FLOW 1956 - 1985
AND AVERAGE MONTHLY DAM RELEASE
GARRISON REACH

Plate 8

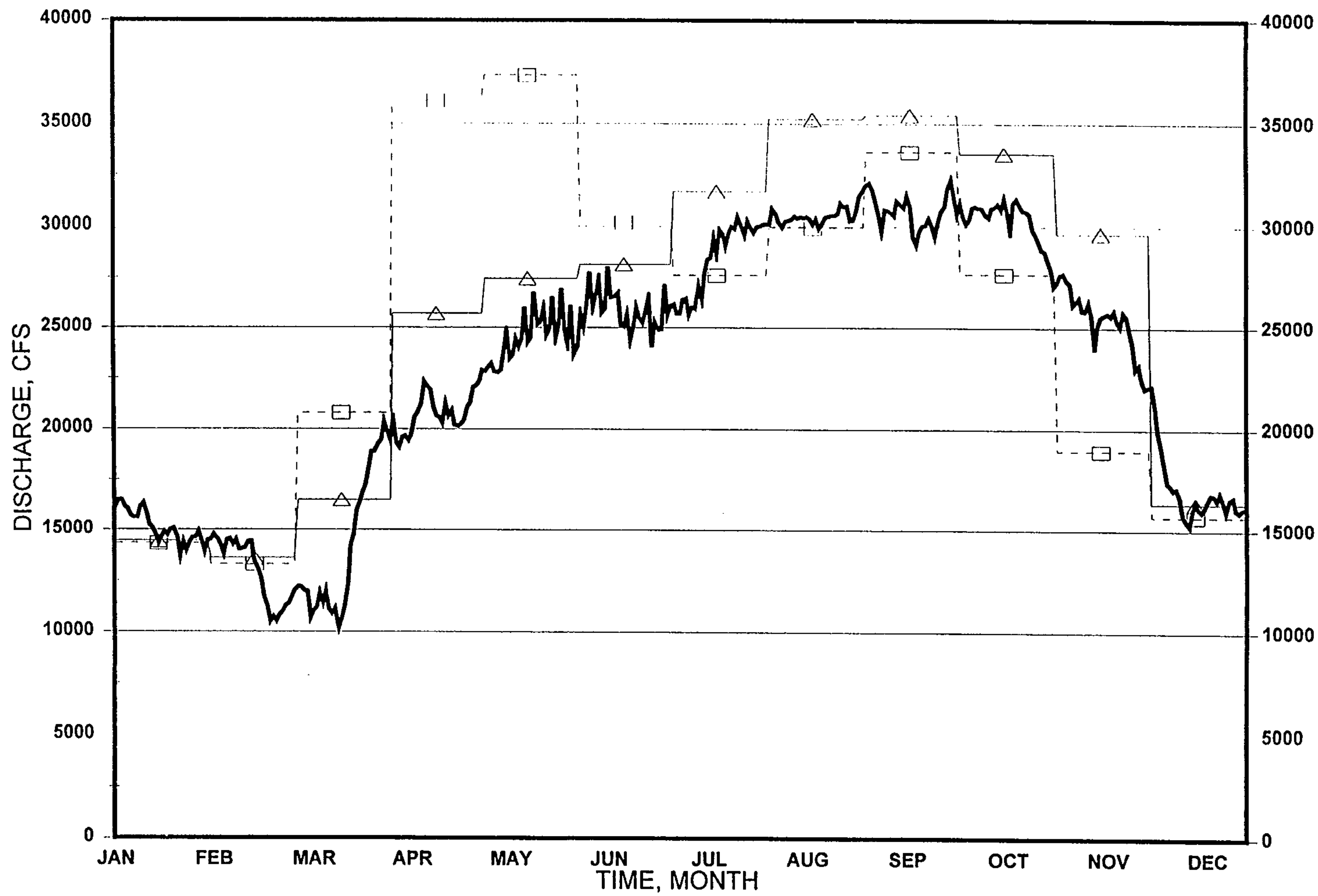


AVERAGE DAILY FLOW, 1976 - 1990
AND AVERAGE MONTHLY DAM RELEASE
GARRISON REACH



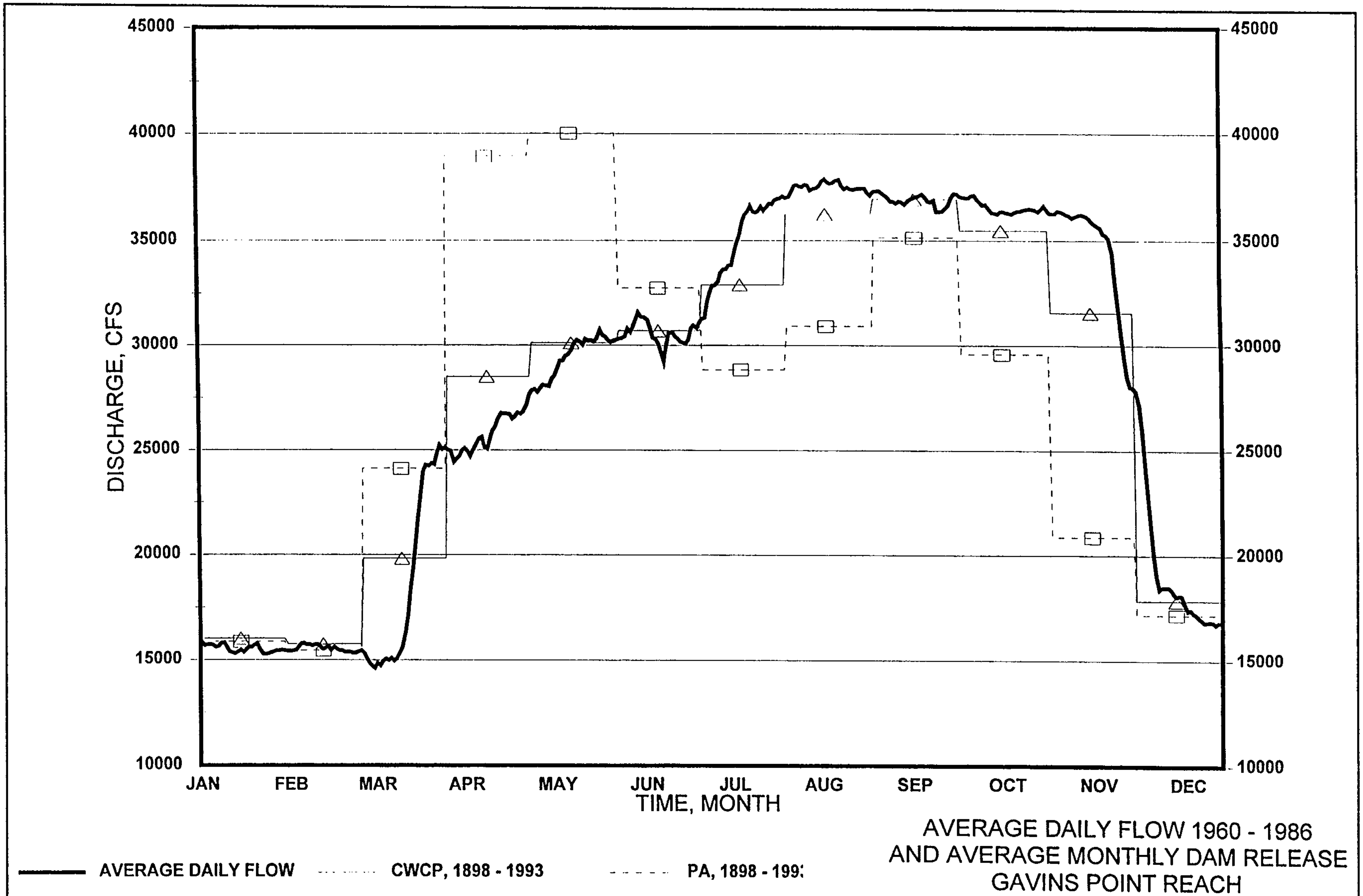
AVERAGE DAILY FLOW 1954 - 1985
AND AVERAGE MONTHLY DAM RELEASE
FORT RANDALL REACH

Plate 10



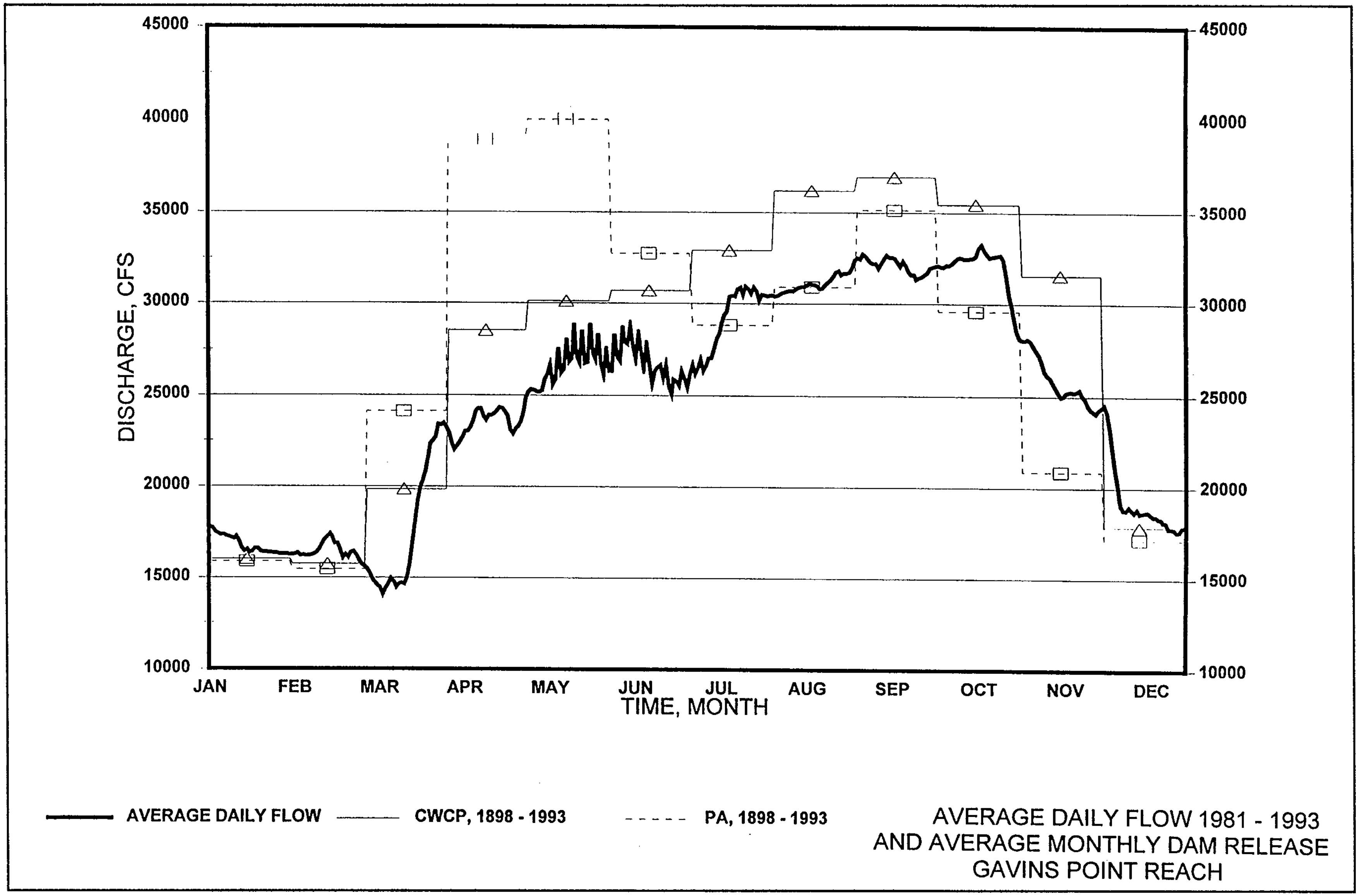
AVERAGE DAILY FLOW — CWCP, 1898 - 1993 - - - - PA, 1898 - 1993

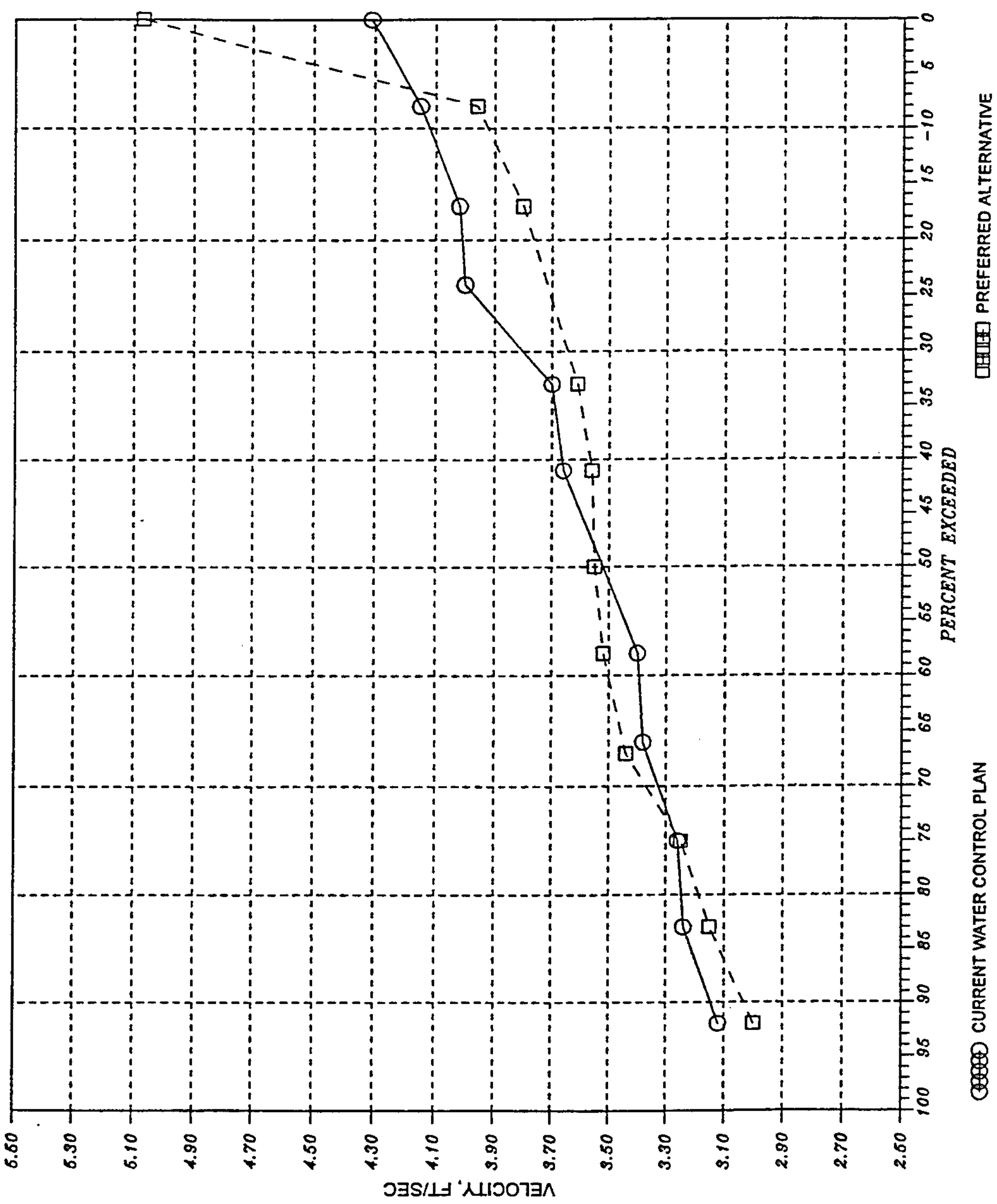
AVERAGE DAILY FLOW 1976 - 1994
AND AVERAGE MONTHLY DAM RELEASE
FORT RANDALL REACH



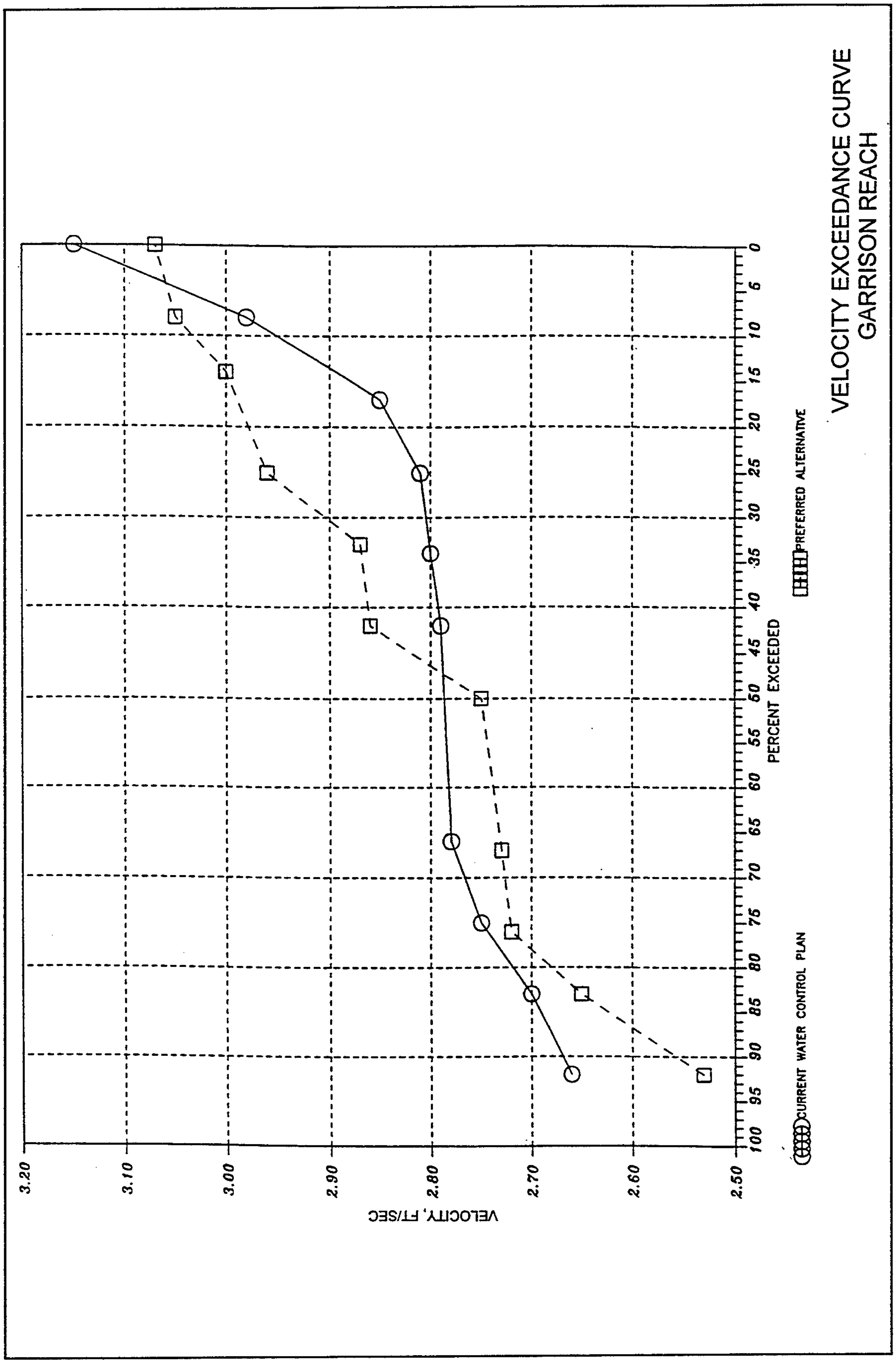
AVERAGE DAILY FLOW 1960 - 1986
AND AVERAGE MONTHLY DAM RELEASE
GAVINS POINT REACH

Plate 12


45000
40000
35000
30000
25000
20000
15000
10000
DISCHARGE, CFS
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC
TIME, MONTH
AVERAGE DAILY FLOW
CWCP, 1898 - 1993
PA, 1898 - 1993
AVERAGE DAILY FLOW 1981 - 1993
AND AVERAGE MONTHLY DAM RELEASE
GAVINS POINT REACH

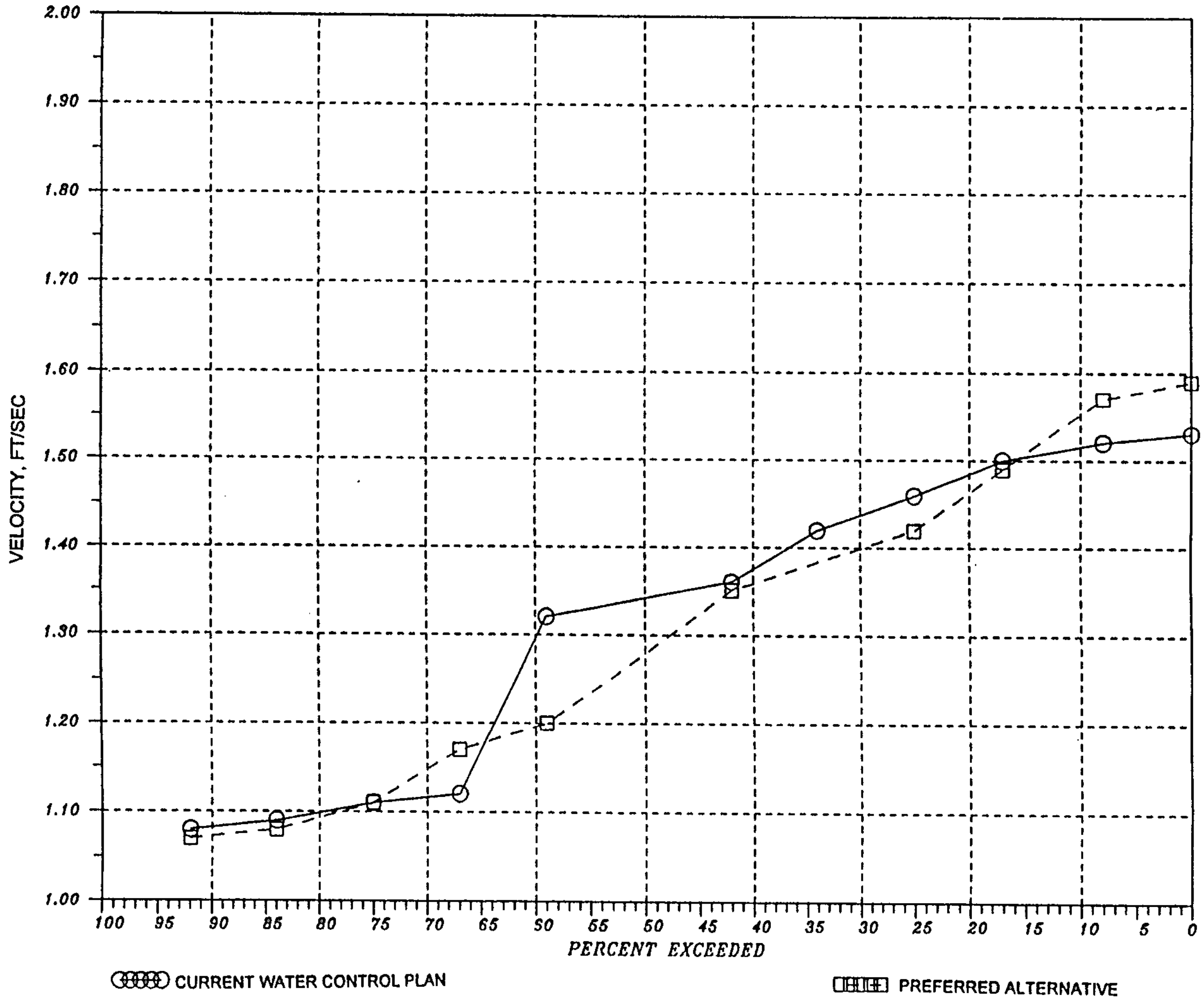


VELOCITY EXCEEDANCE CURVE
FORT PECK REACH

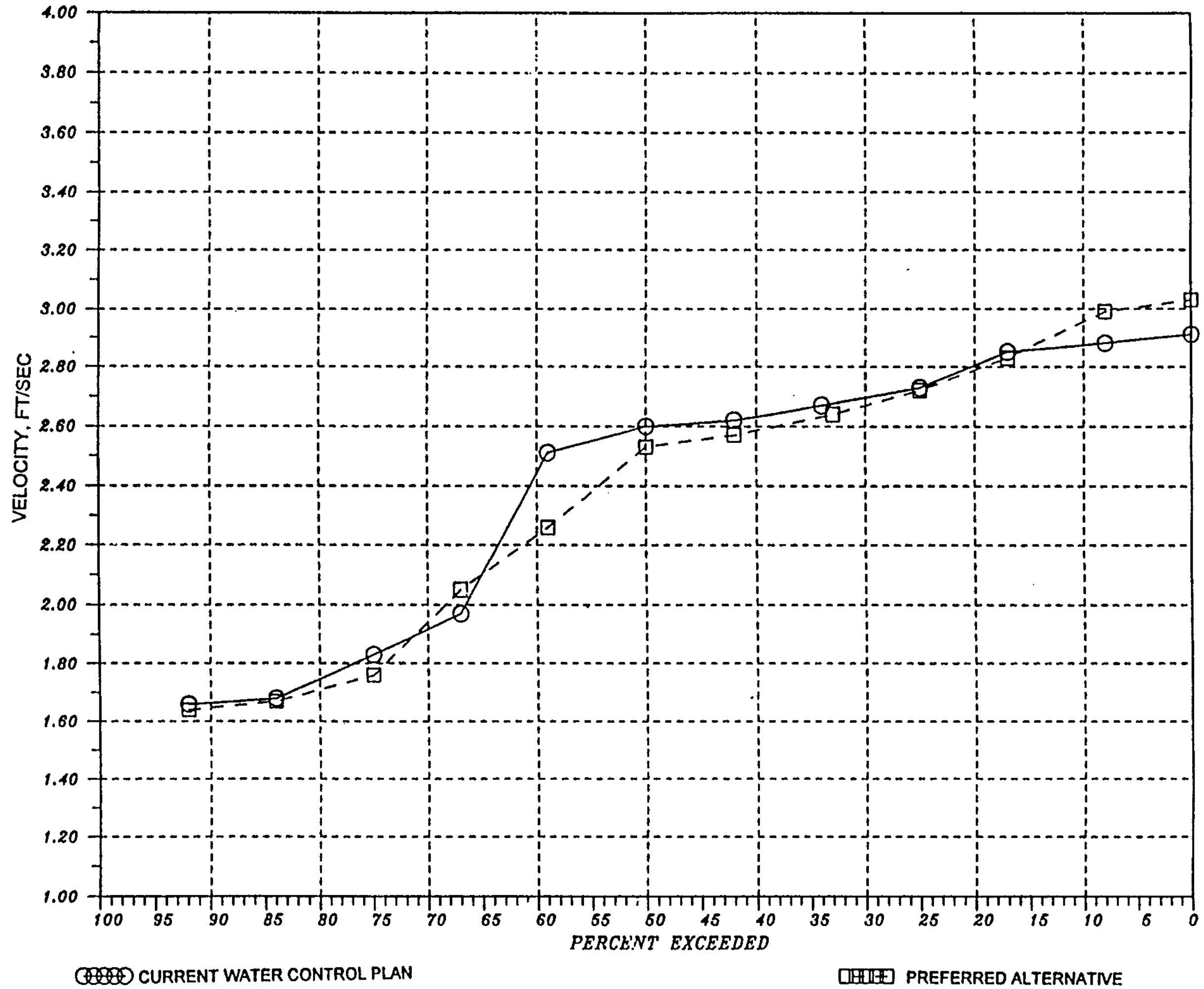


VELOCITY EXCEEDANCE CURVE
GARRISON REACH
VELOCITY, FT/SEC
PERCENT EXCEEDED
CURRENT WATER CONTROL PLAN
PREFERRED ALTERNATIVE
3.20
3.10
3.00
2.90
2.80
2.70
2.60
2.50
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
95
100

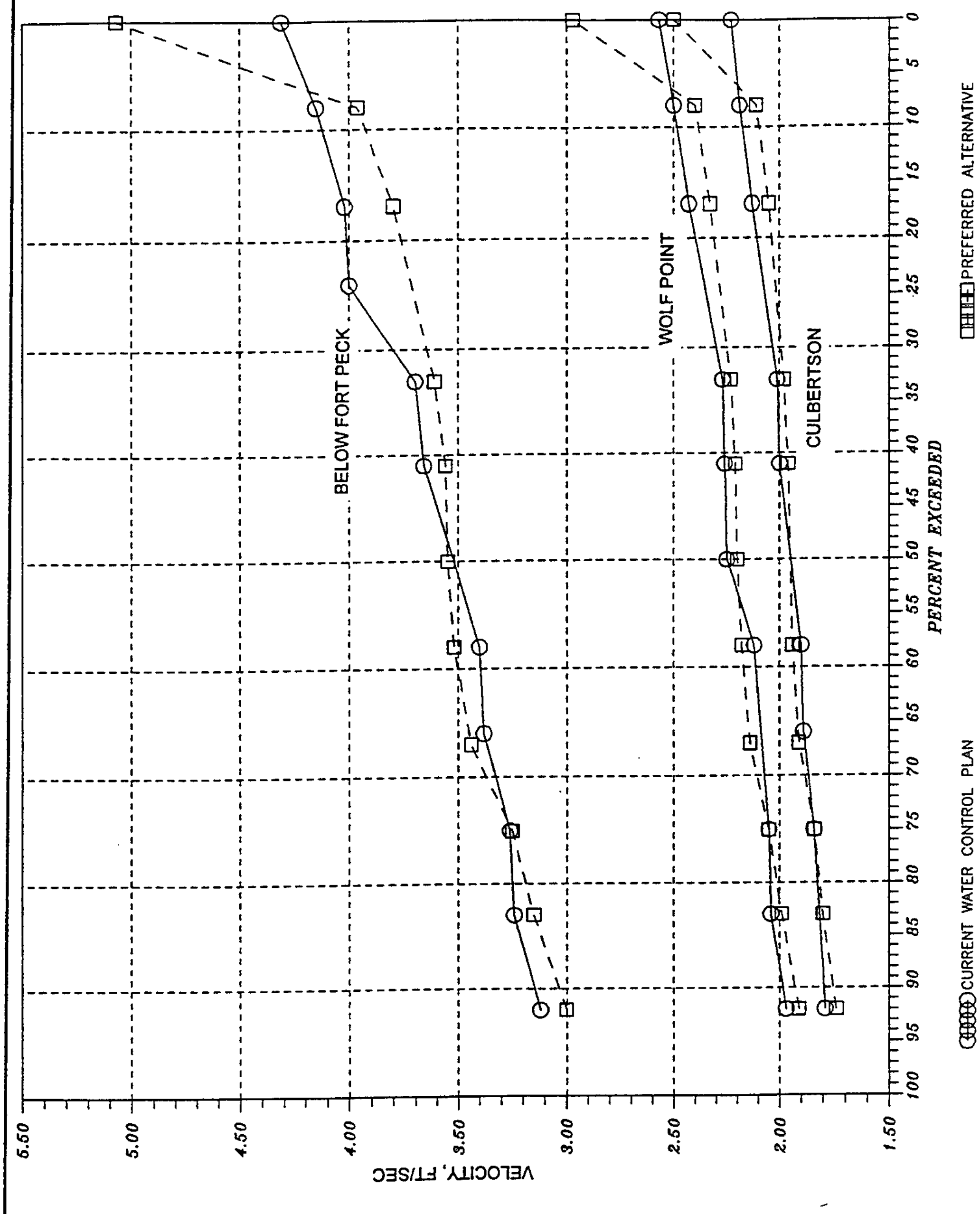
Plate 15



VELOCITY EXCEEDANCE CURVE
FORT RANDALL REACH

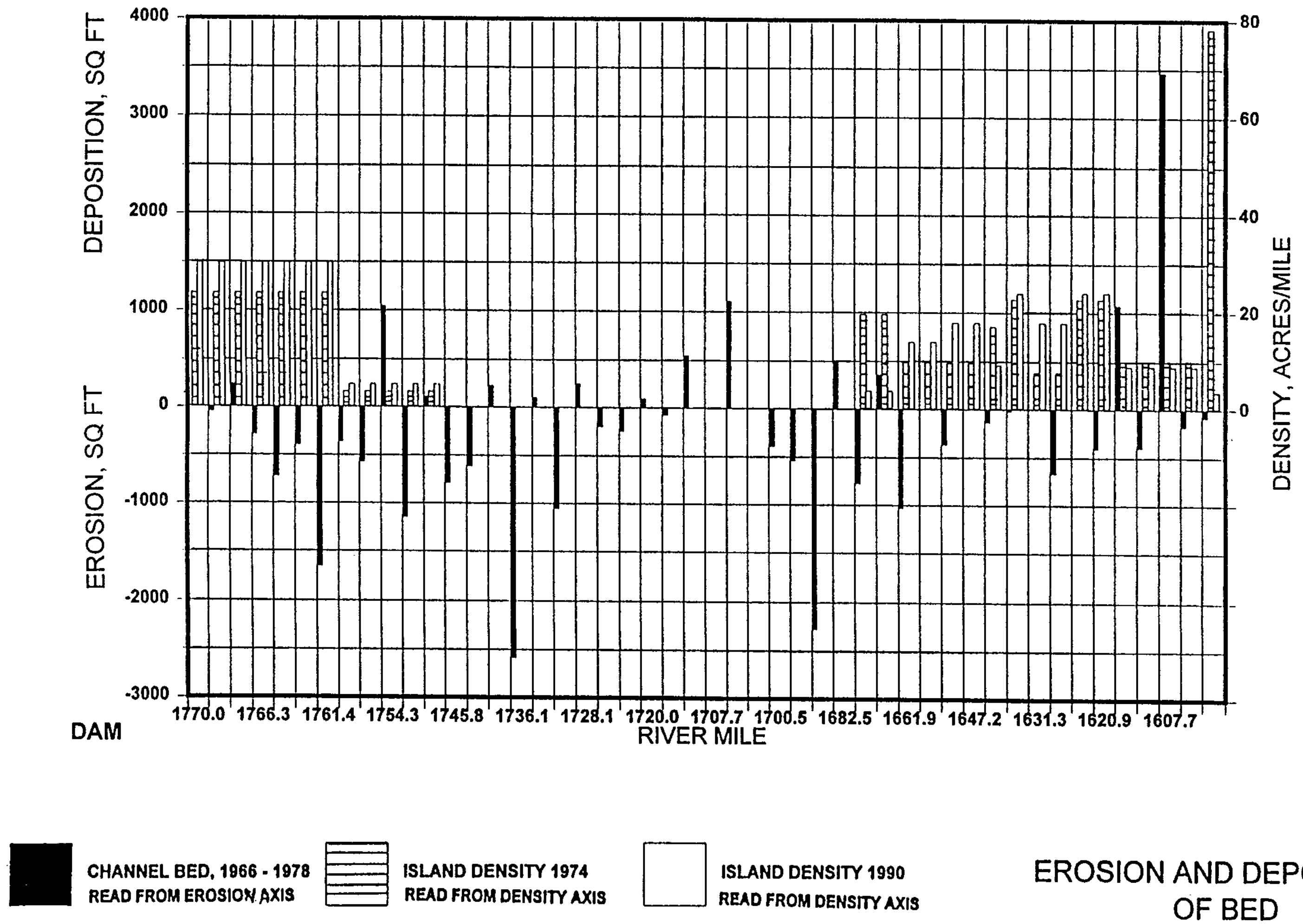


VELOCITY EXCEEDANCE CURVE
GAVINS POINT REACH

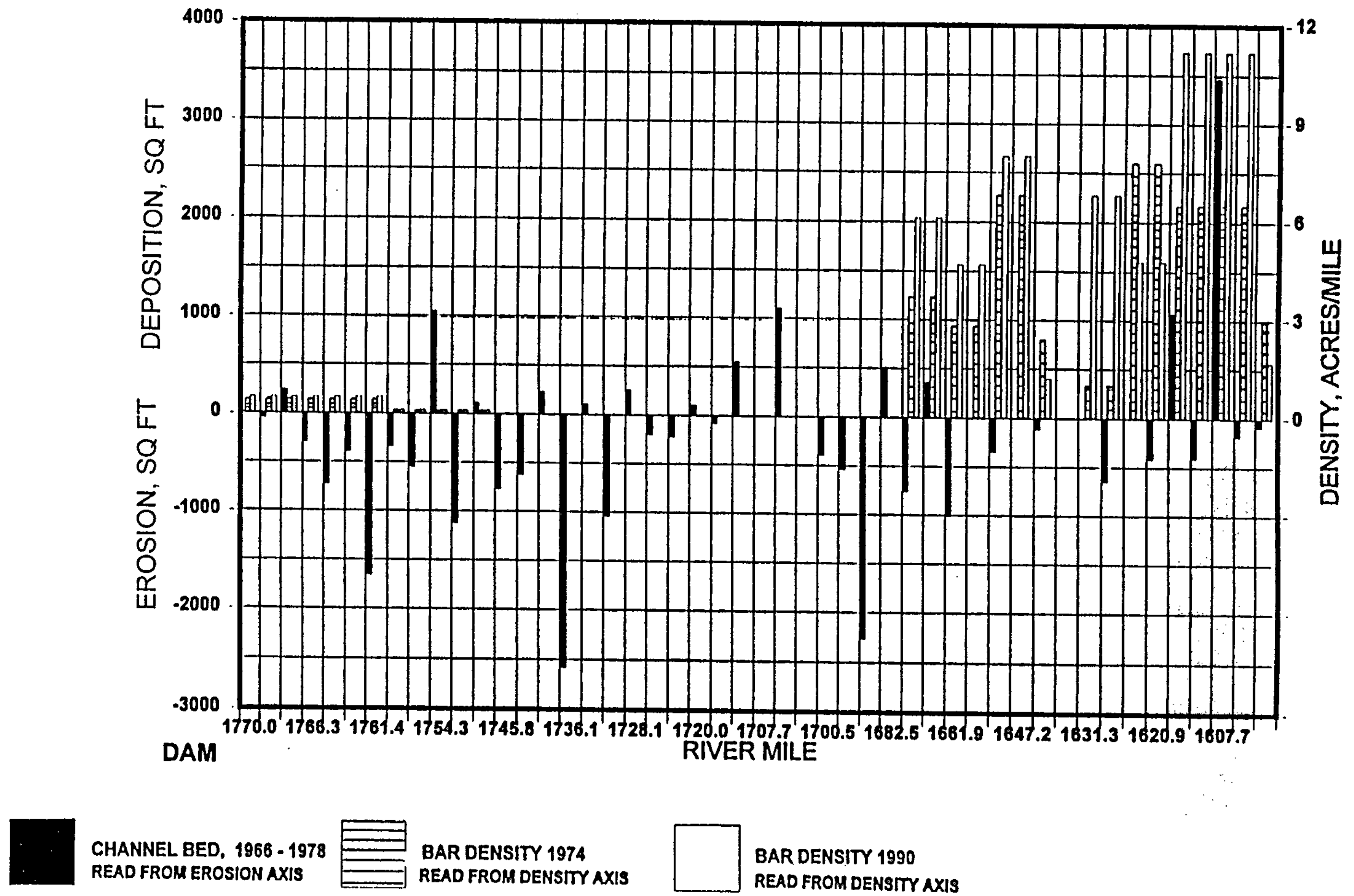


VELOCITY EXCEEDANCE CURVES
AT THREE LOCATIONS
FORT PECK REACH

Plate 18

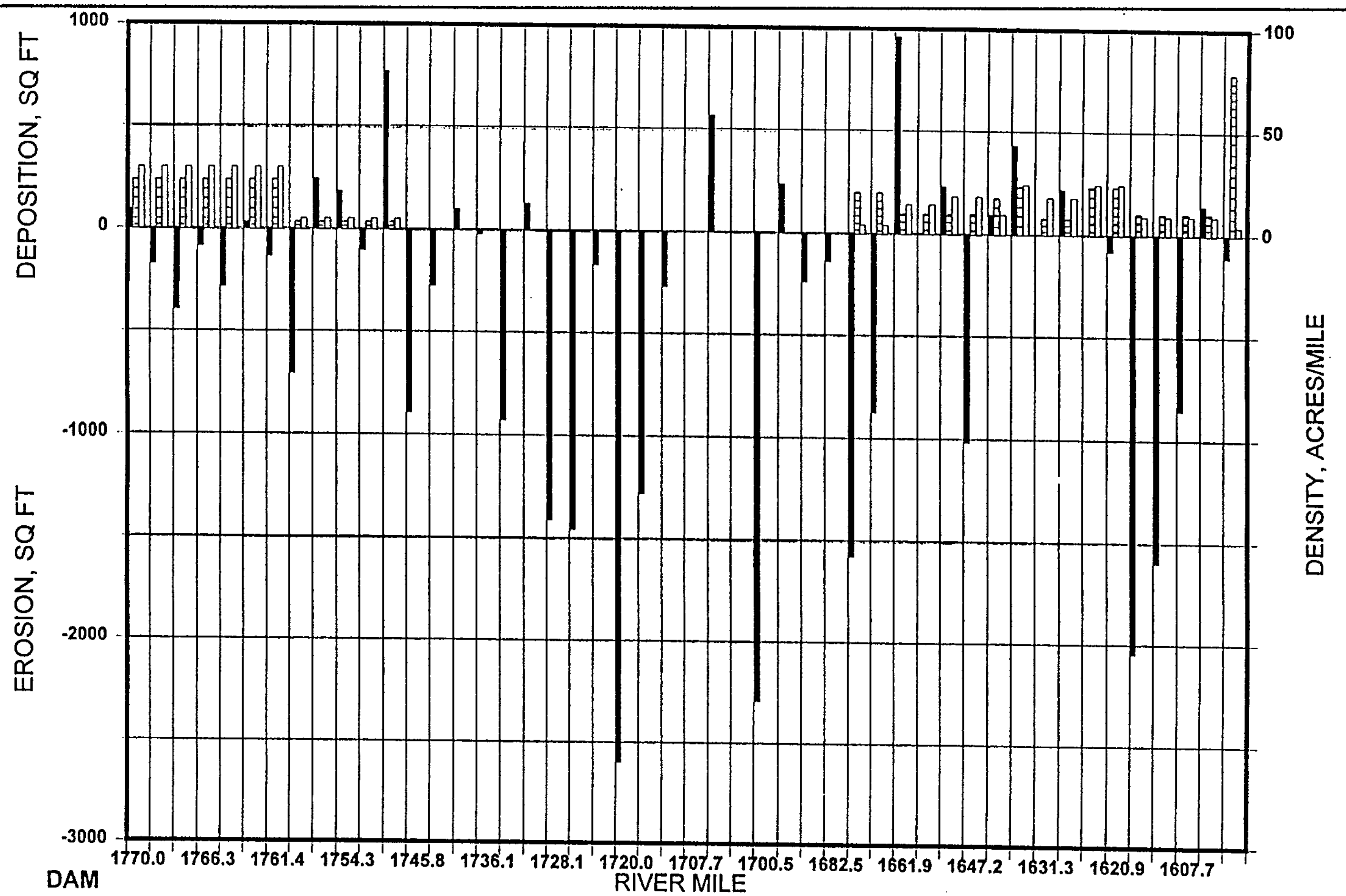


EROSION AND DEPOSITION
OF BED
COMPARED TO ISLAND DENSITY
FORT PECK REACH



EROSION AND DEPOSITION
OF BED
COMPARED TO BAR DENSITY
FORT PECK REACH

Plate 20



CHANNEL BANKS, 1966 - 1978
READ FROM EROSION AXIS

ISLAND DENSITY 1974
READ FROM DENSITY AXIS

ISLAND DENSITY 1990
READ FROM DENSITY AXIS

EROSION AND DEPOSITION
OF BANKS
COMPARED TO ISLAND DENSITY
FORT PECK REACH

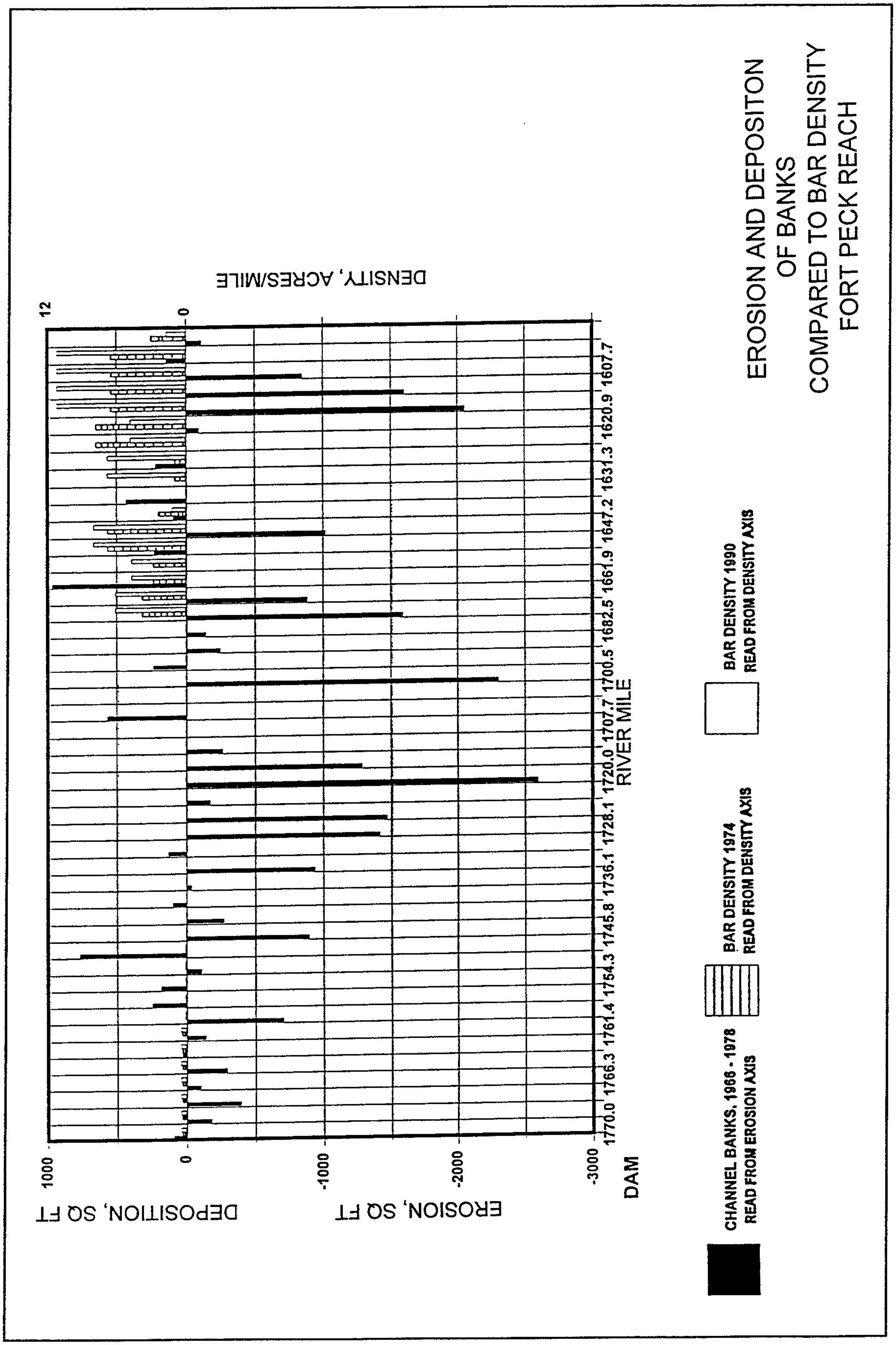
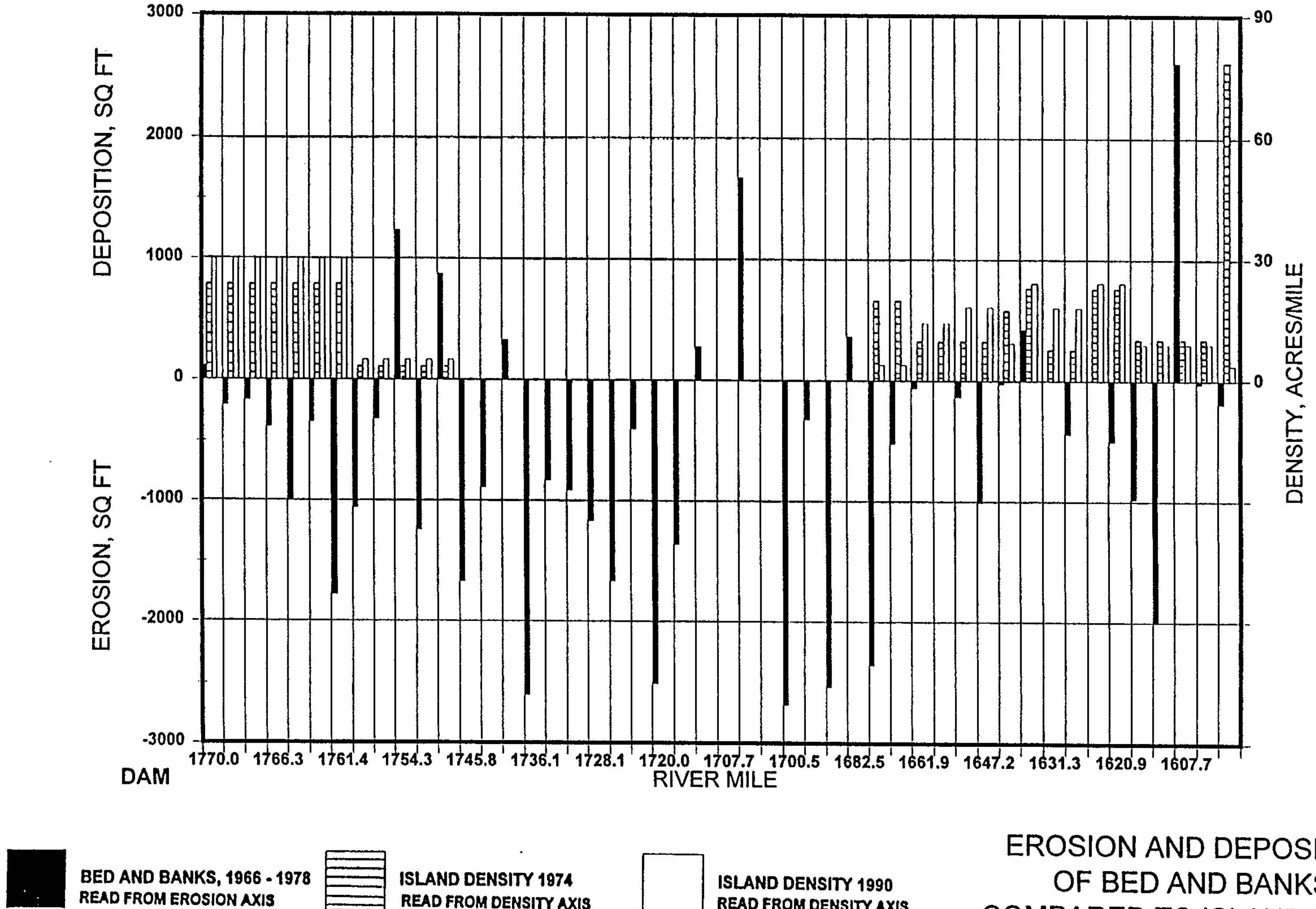

EROSION AND DEPOSITON
OF BANKS
COMPARED TO BAR DENSITY
FORT PECK REACH
DENSITY, ACRES/MILE
12
0
DEPOSITION, SQ FT
EROSION, SQ FT
1000
0
-1000
-2000
-3000
DAM
1770.0
1766.3
1761.4
1754.3
1745.8
1736.1
1728.1
1720.0
1707.7
1700.5
1682.5
1661.9
1647.2
1631.3
1620.9
1607.7
RIVER MILE
CHANNEL BANKS, 1966 - 1978
READ FROM EROSION AXIS
BAR DENSITY 1974
READ FROM DENSITY AXIS
BAR DENSITY 1990
READ FROM DENSITY AXIS

Plate 22



EROSION AND DEPOSITION
OF BED AND BANKS
COMPARED TO ISLAND DENSITY
FORT PECK REACH

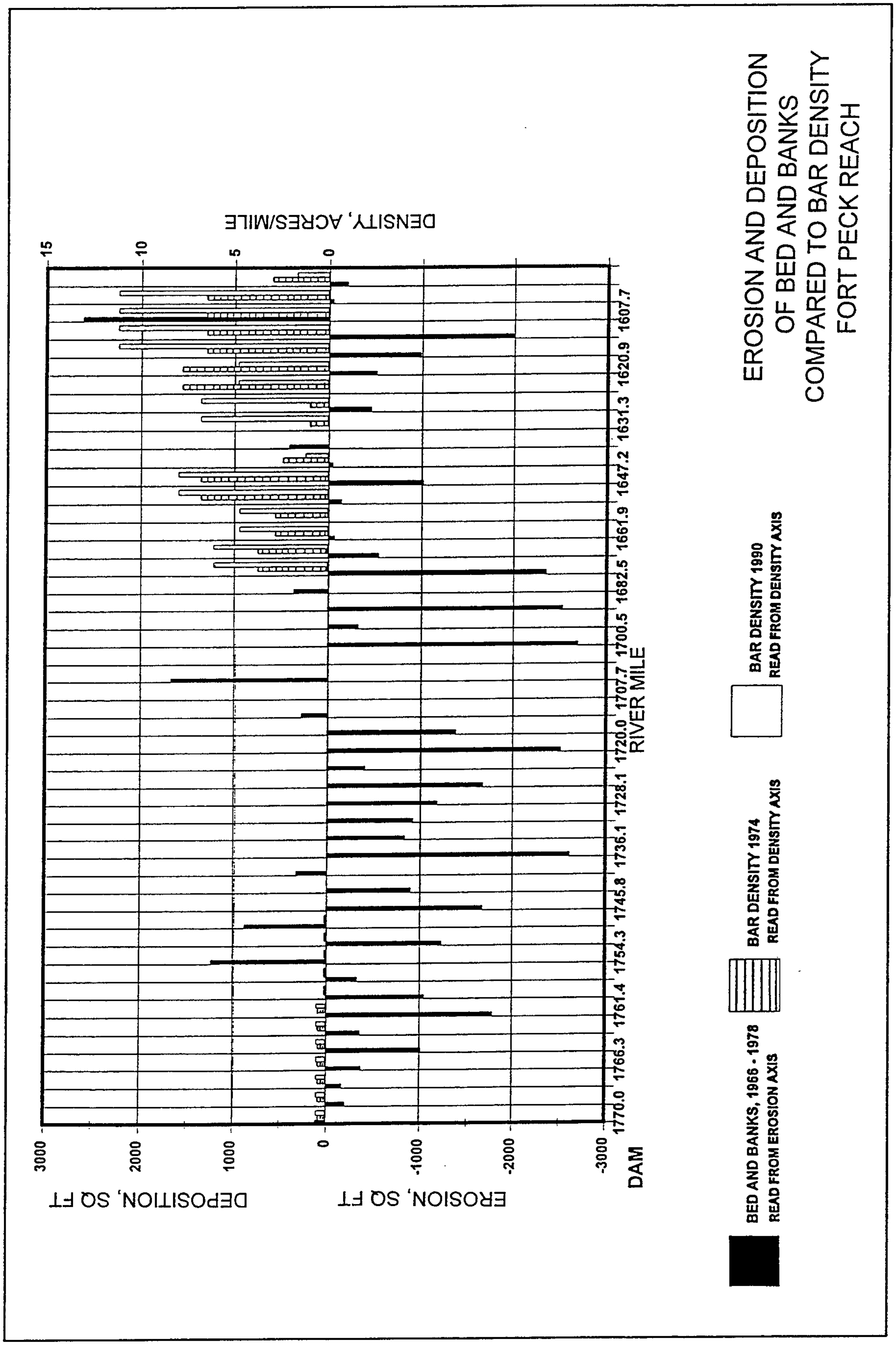
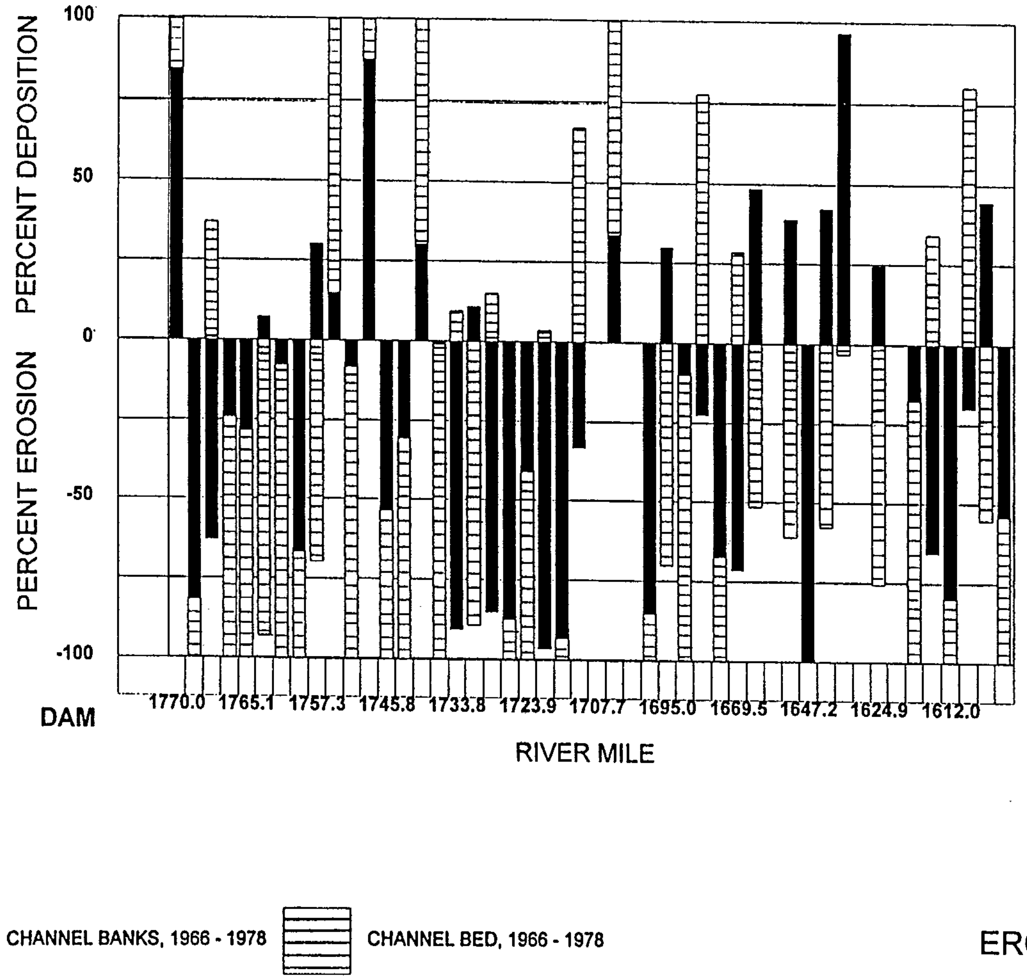
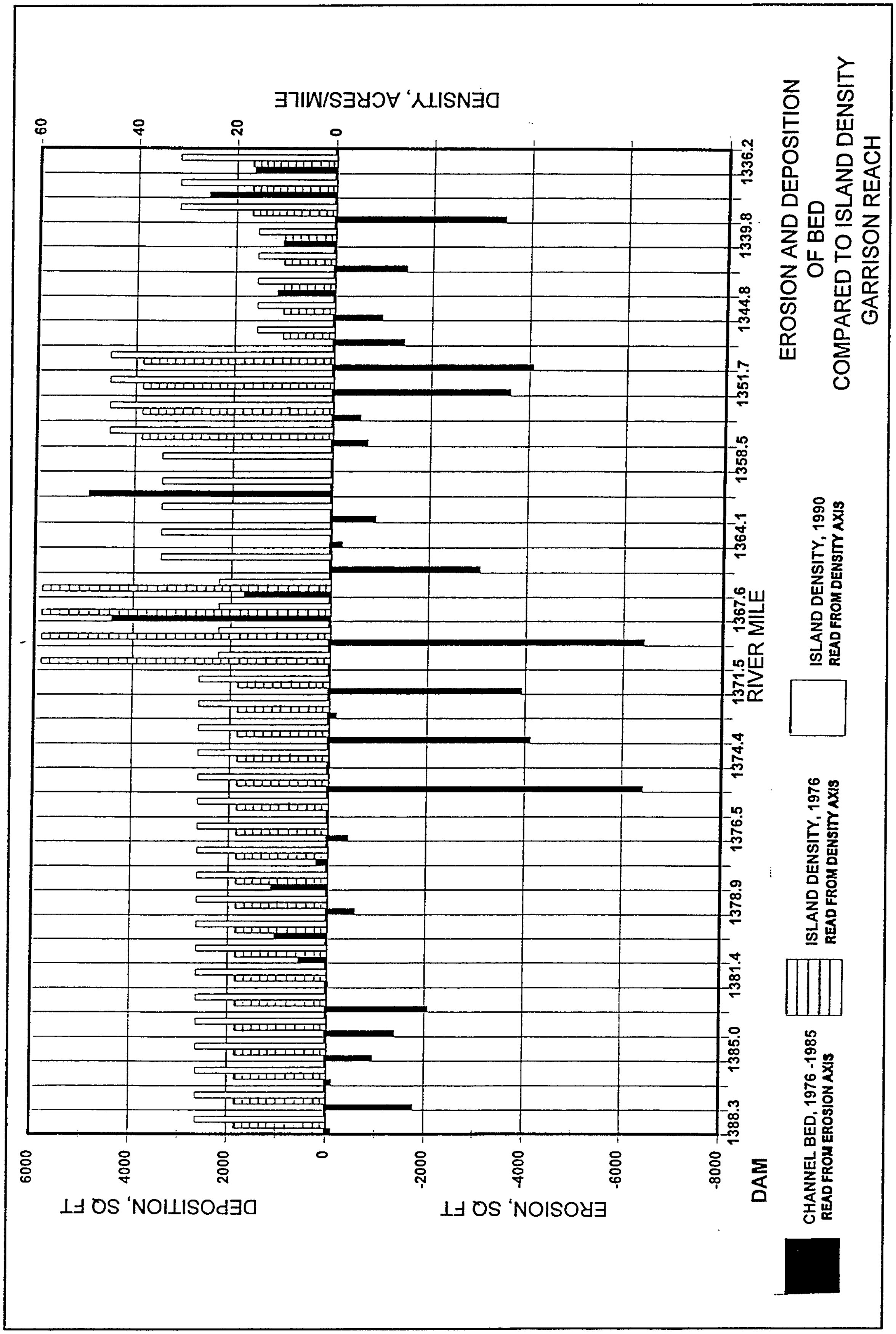

EROSION AND DEPOSITION
OF BED AND BANKS
COMPARED TO BAR DENSITY
FORT PECK REACH
DENSITY, ACRES/MILE
15
10
5
0
DEPOSITION, SQ FT
EROSION, SQ FT
3000
2000
1000
0
-1000
-2000
-3000
DAM
RIVER MILE
1770.0
1766.3
1761.4
1754.3
1745.8
1736.1
1728.1
1720.0
1707.7
1700.5
1682.5
1661.9
1647.9
1647.2
1631.3
1620.9
1607.7
BED AND BANKS, 1966 - 1978
READ FROM EROSION AXIS
BAR DENSITY 1974
READ FROM DENSITY AXIS
BAR DENSITY 1990
READ FROM DENSITY AXIS

Plate 24

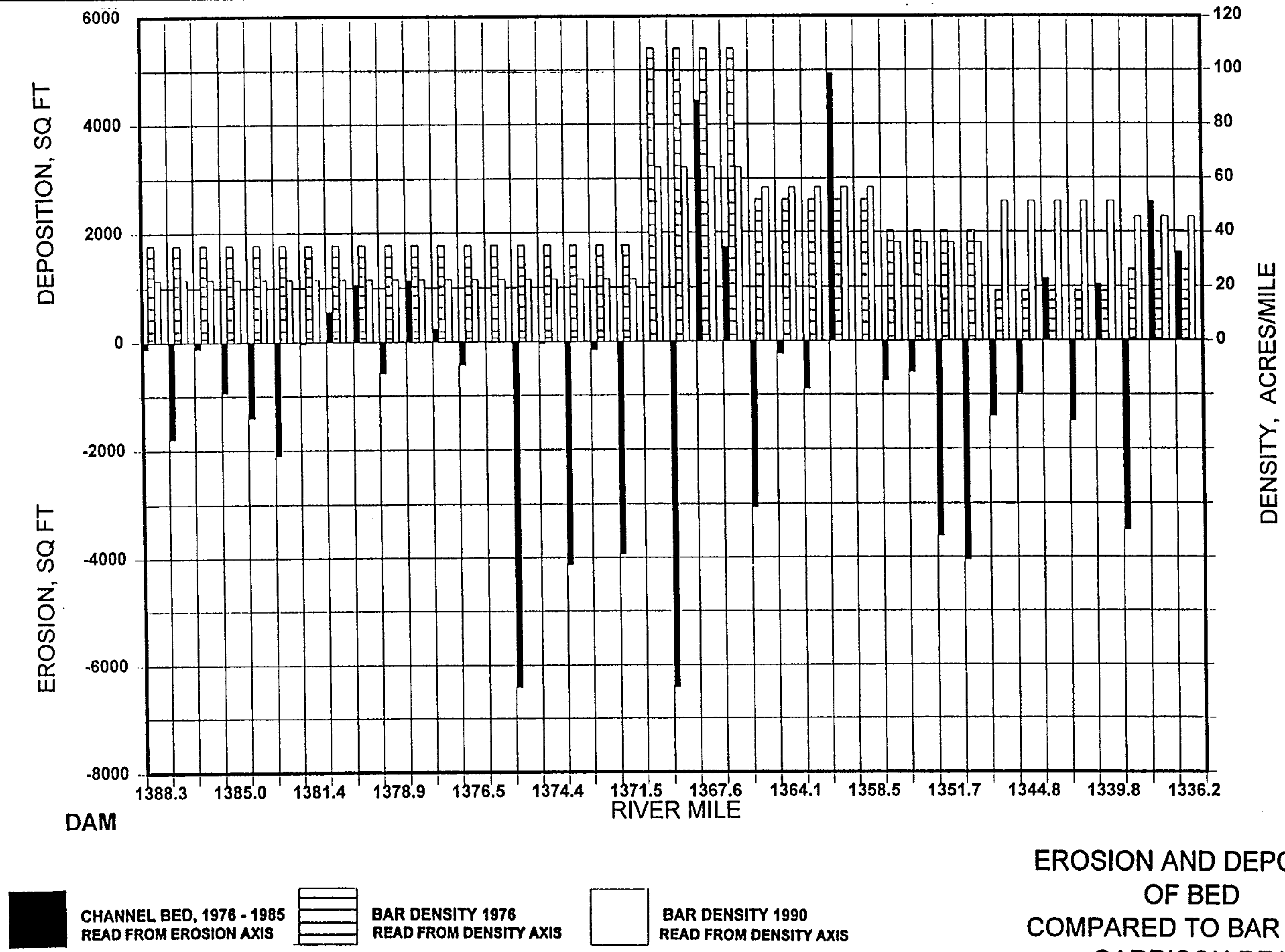


EROSION AND DEPOSITION
OF BED AND BANKS
FORT PECK REACH

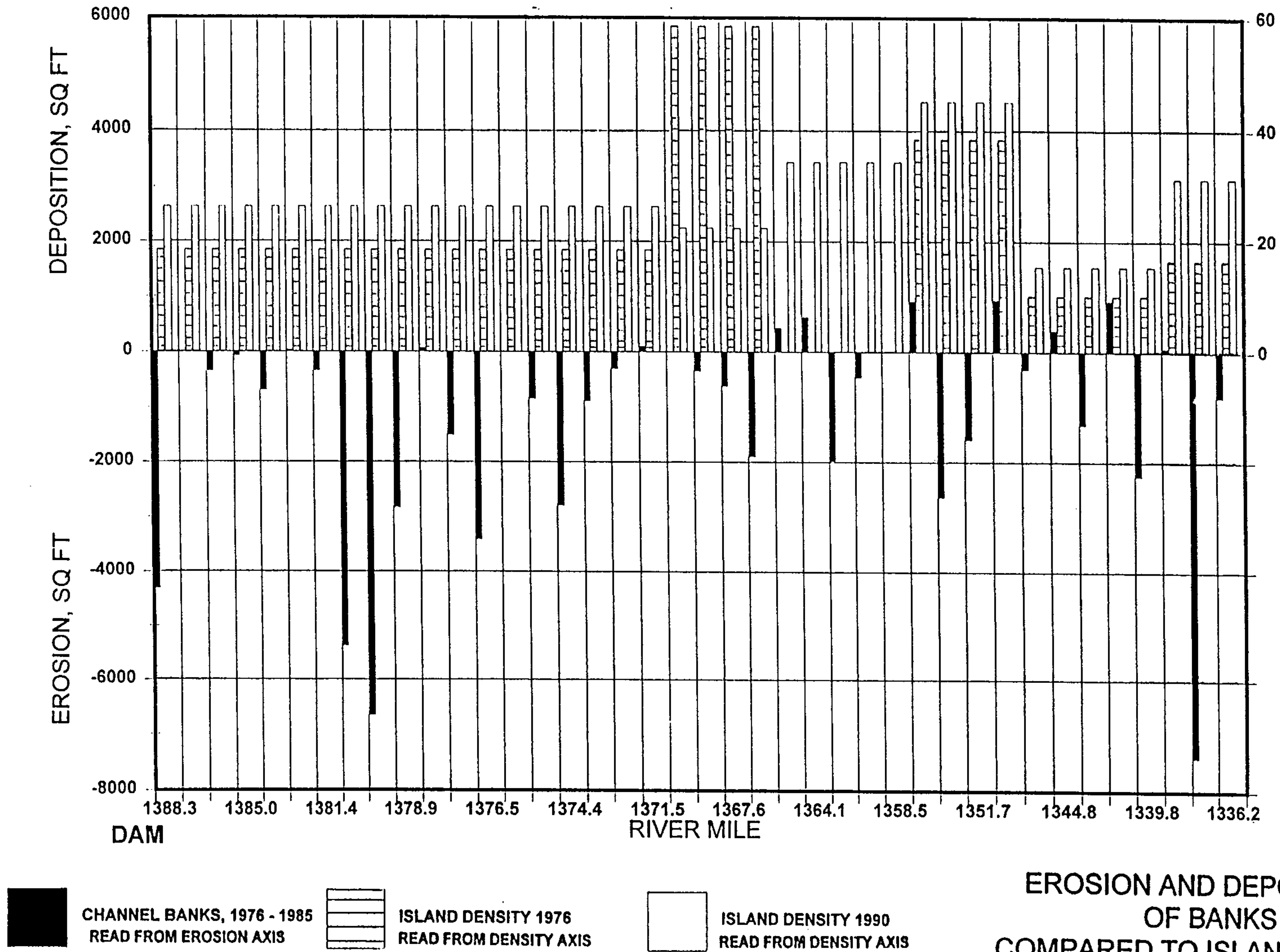


EROSION AND DEPOSITION
OF BED
COMPARED TO ISLAND DENSITY
GARRISON REACH
DENSITY, ACRES/MILE
60
40
20
0
DEPOSITION, SQ FT
EROSION, SQ FT
6000
4000
2000
0
-2000
-4000
-6000
-8000
DAM
RIVER MILE
1336.2
1339.8
1344.8
1351.7
1358.5
1364.1
1367.6
1371.5
1374.4
1376.5
1378.9
1381.4
1385.0
1388.3
CHANNEL BED, 1976 -1985
READ FROM EROSION AXIS
ISLAND DENSITY, 1976
READ FROM DENSITY AXIS
ISLAND DENSITY, 1990
READ FROM DENSITY AXIS

Plate 26

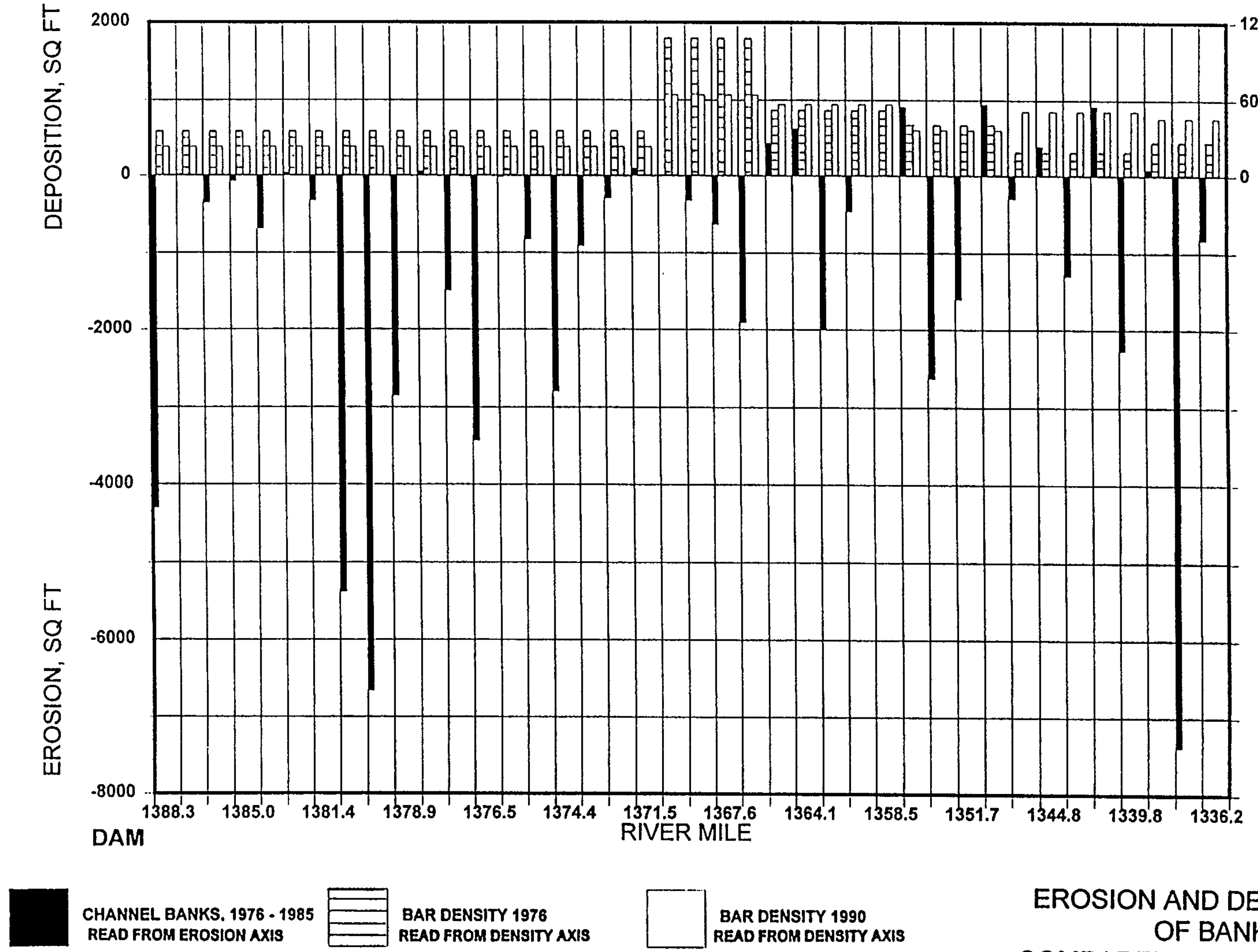


EROSION AND DEPOSITION
OF BED
COMPARED TO BAR DENSITY
GARRISON REACH

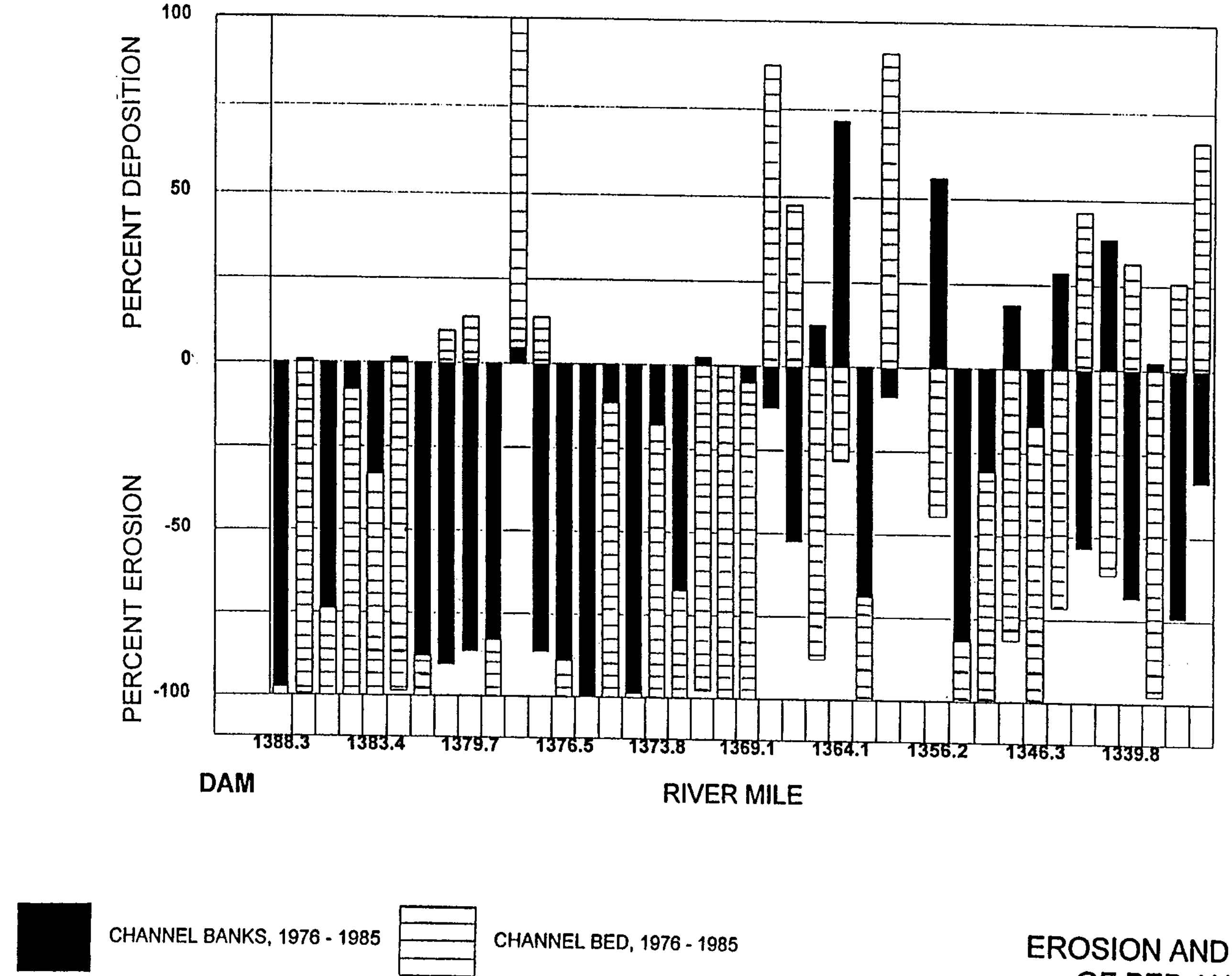


EROSION AND DEPOSITION
OF BANKS
COMPARED TO ISLAND DENSITY
GARRISON REACH

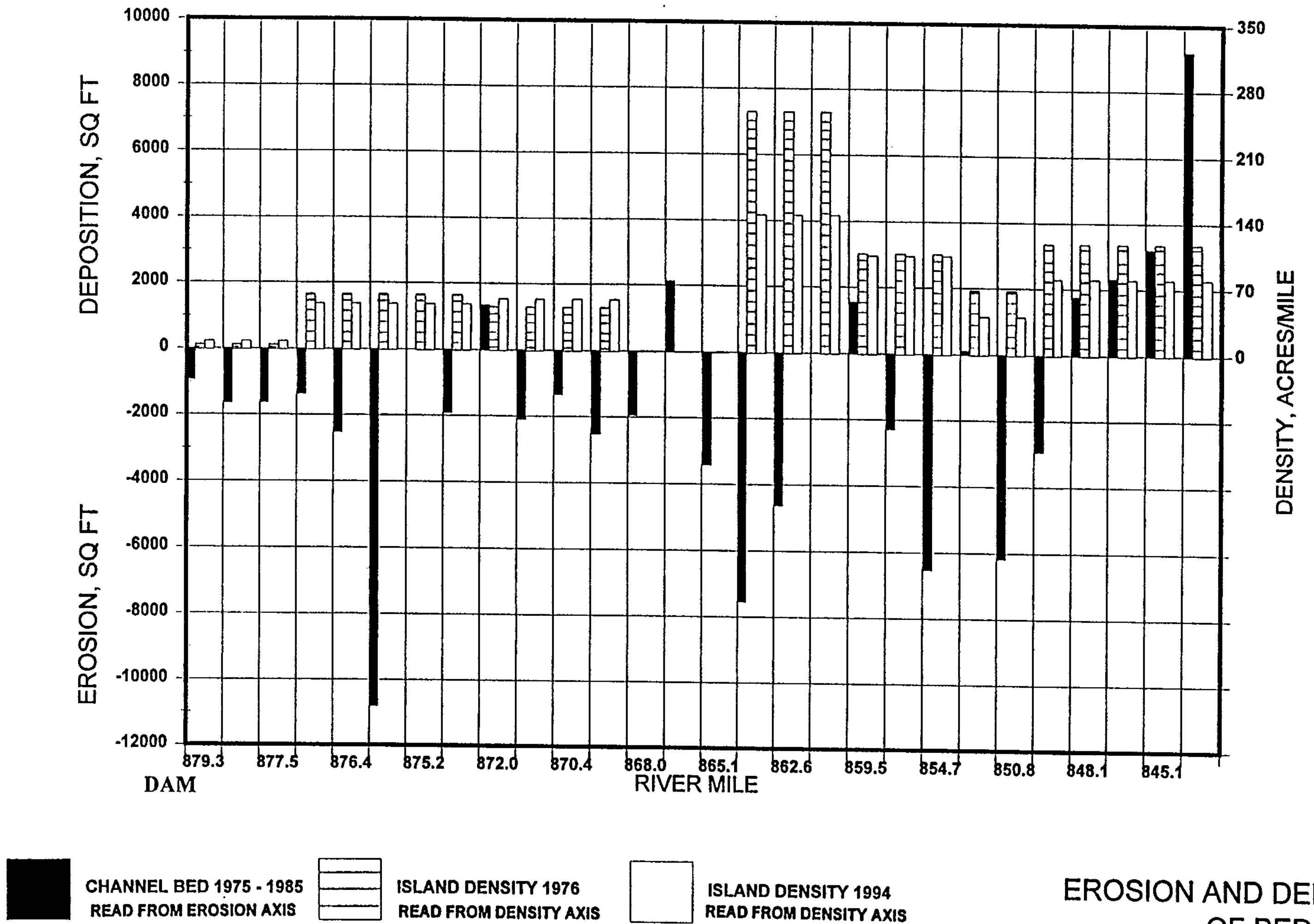
Plate 28



EROSION AND DEPOSITION
OF BANKS
COMPARED TO BAR DENSITY
GARRISON REACH



EROSION AND DEPOSITION
OF BED AND BANKS
GARRISON REACH



EROSION AND DEPOSITION
OF BED
COMPARED TO ISLAND DENSITY
FORT RANDALL REACH

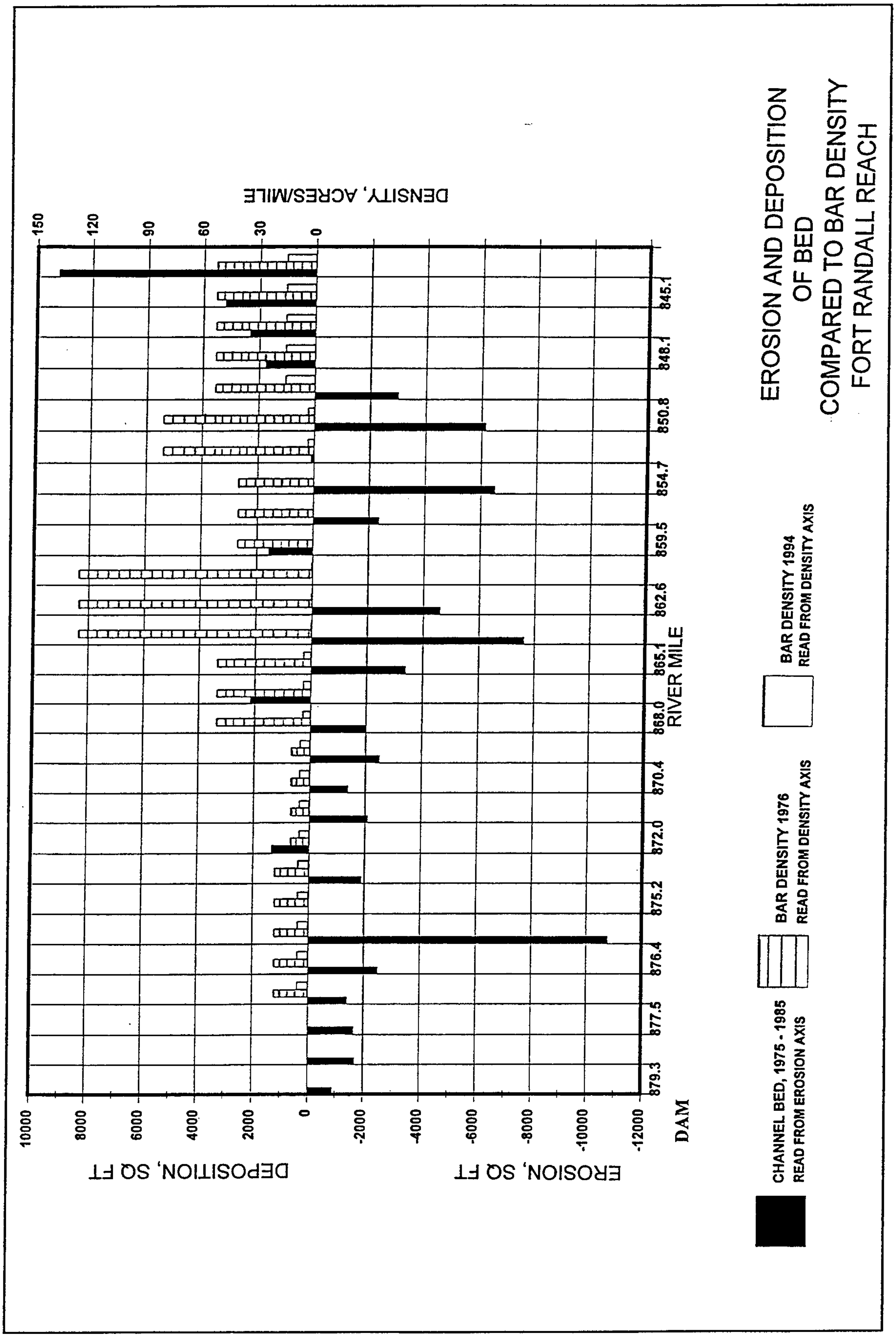
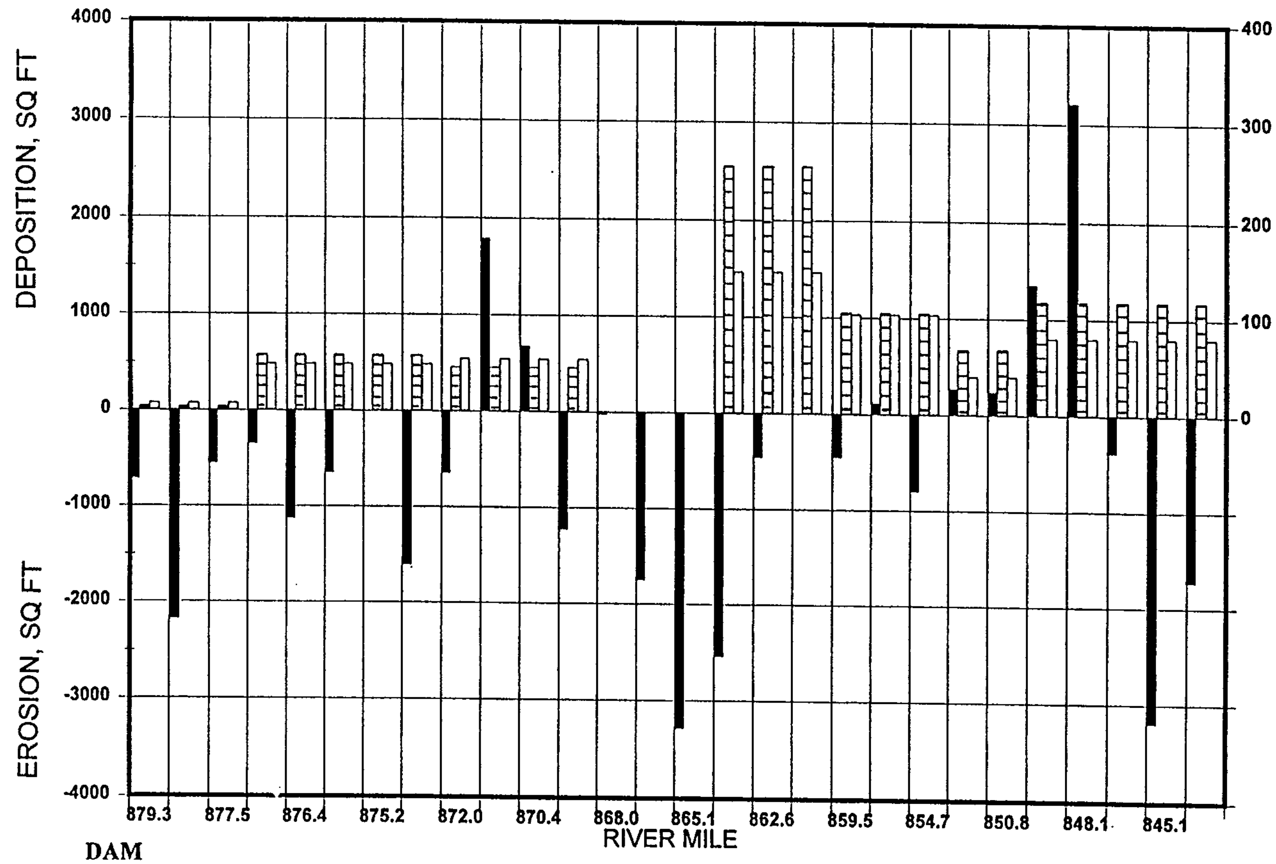

EROSION AND DEPOSITION
OF BED
COMPARED TO BAR DENSITY
FORT RANDALL REACH
DENSITY, ACRES/MILE
DEPOSITION, SQ FT
EROSION, SQ FT
RIVER MILE
DAM
879.3
877.5
876.4
875.2
872.0
870.4
868.0
865.1
862.6
859.5
854.7
850.8
848.1
845.1
10000
8000
6000
4000
2000
0
-2000
-4000
-6000
-8000
-10000
-12000
150
120
90
60
30
0
CHANNEL BED, 1975 - 1985
READ FROM EROSION AXIS
BAR DENSITY 1976
READ FROM DENSITY AXIS
BAR DENSITY 1994
READ FROM DENSITY AXIS

Plate 32

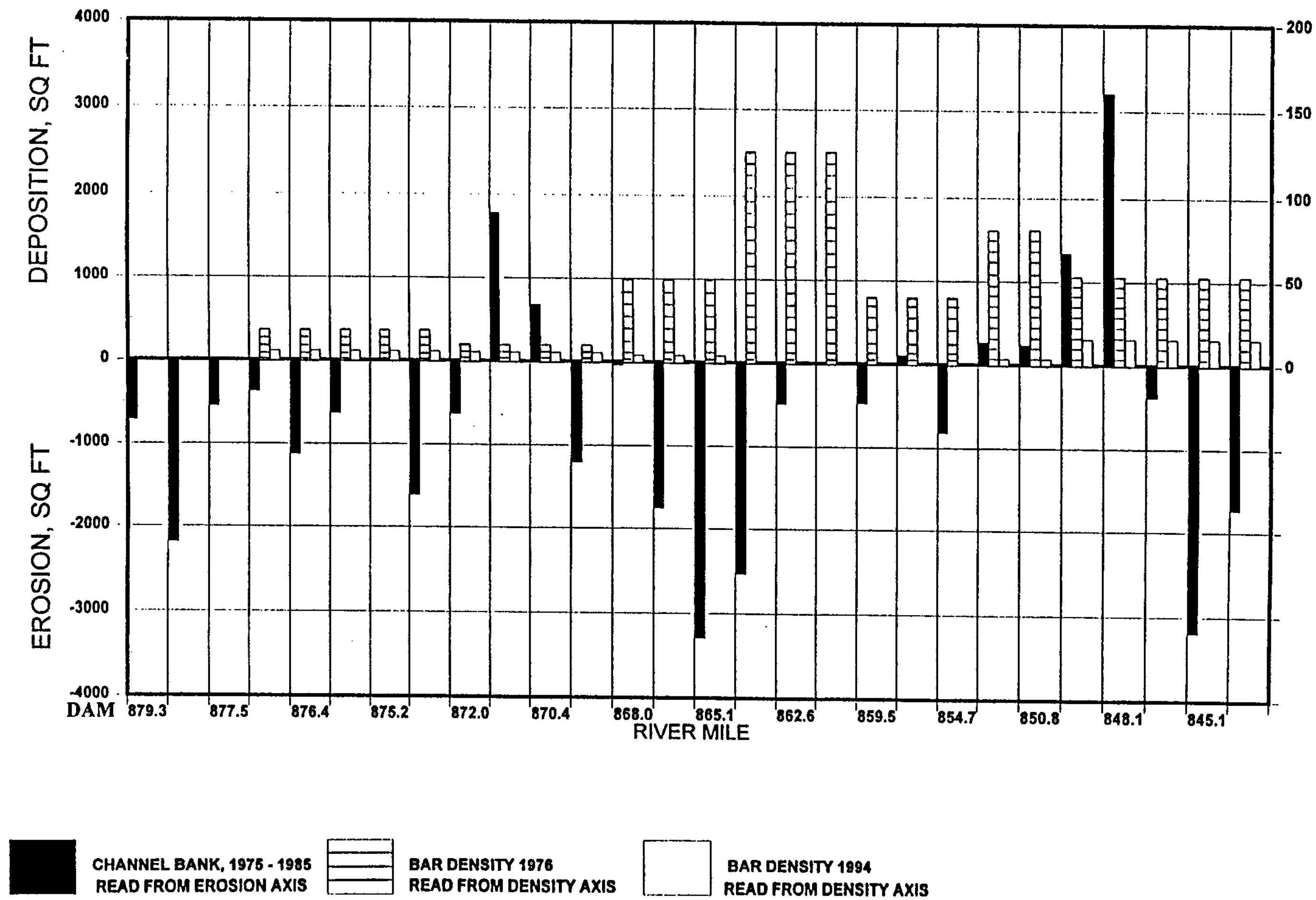


CHANNEL BANK, 1975 - 1985
READ FROM EROSION AXIS

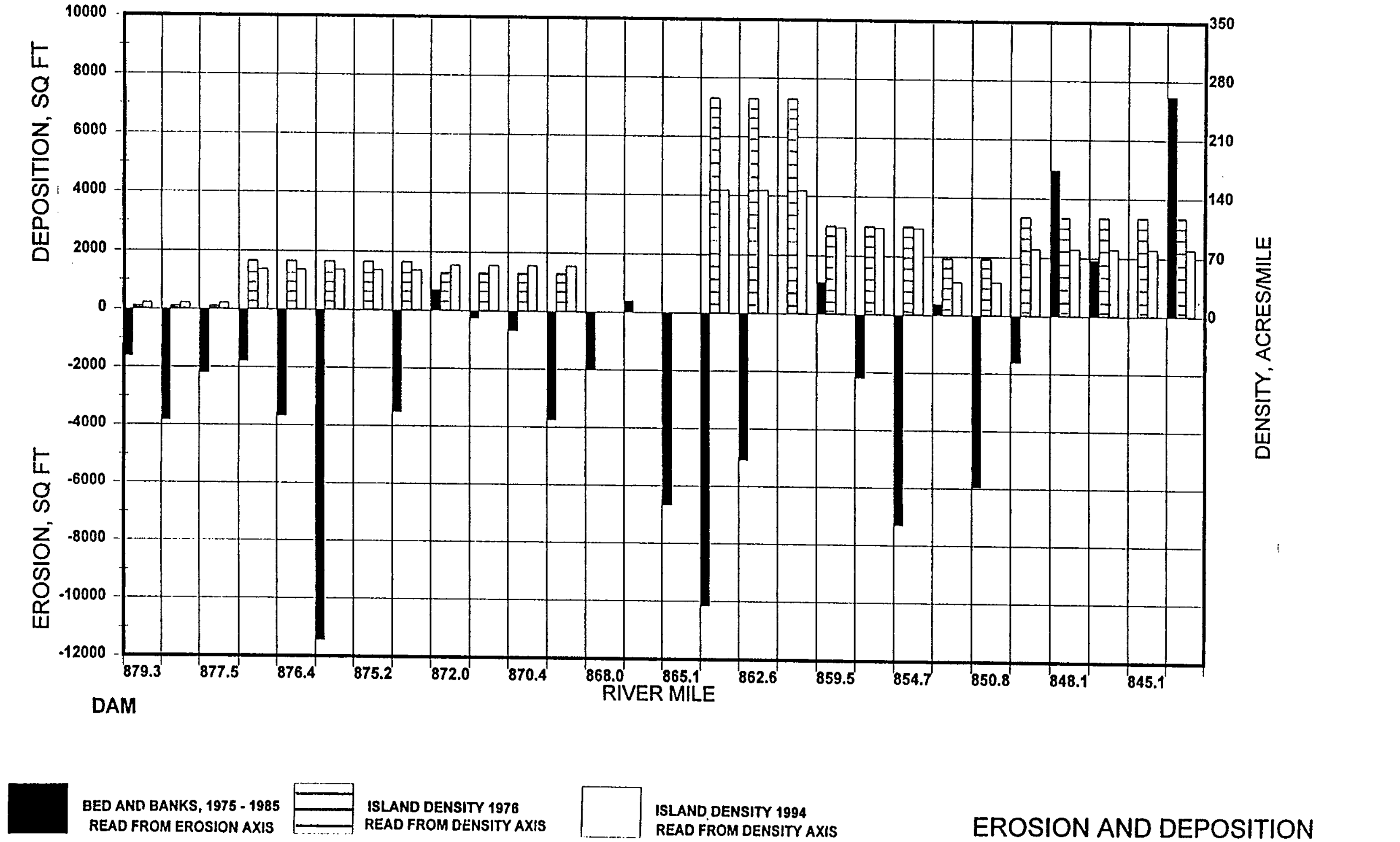
ISLAND DENSITY 1976
READ FROM DENSITY AXIS

ISLAND DENSITY 1994
READ FROM DENSITY AXIS

EROSION AND DEPOSITION
OF BANKS
COMPARED TO ISLAND DENSITY
FORT RANDALL REACH



EROSION AND DEPOSITION
OF BANKS
COMPARED TO BAR DENSITY
FORT RANDALL REACH



EROSION AND DEPOSITION
OF BED AND BANKS
COMPARED TO ISLAND DENSITY
FORT RANDALL REACH

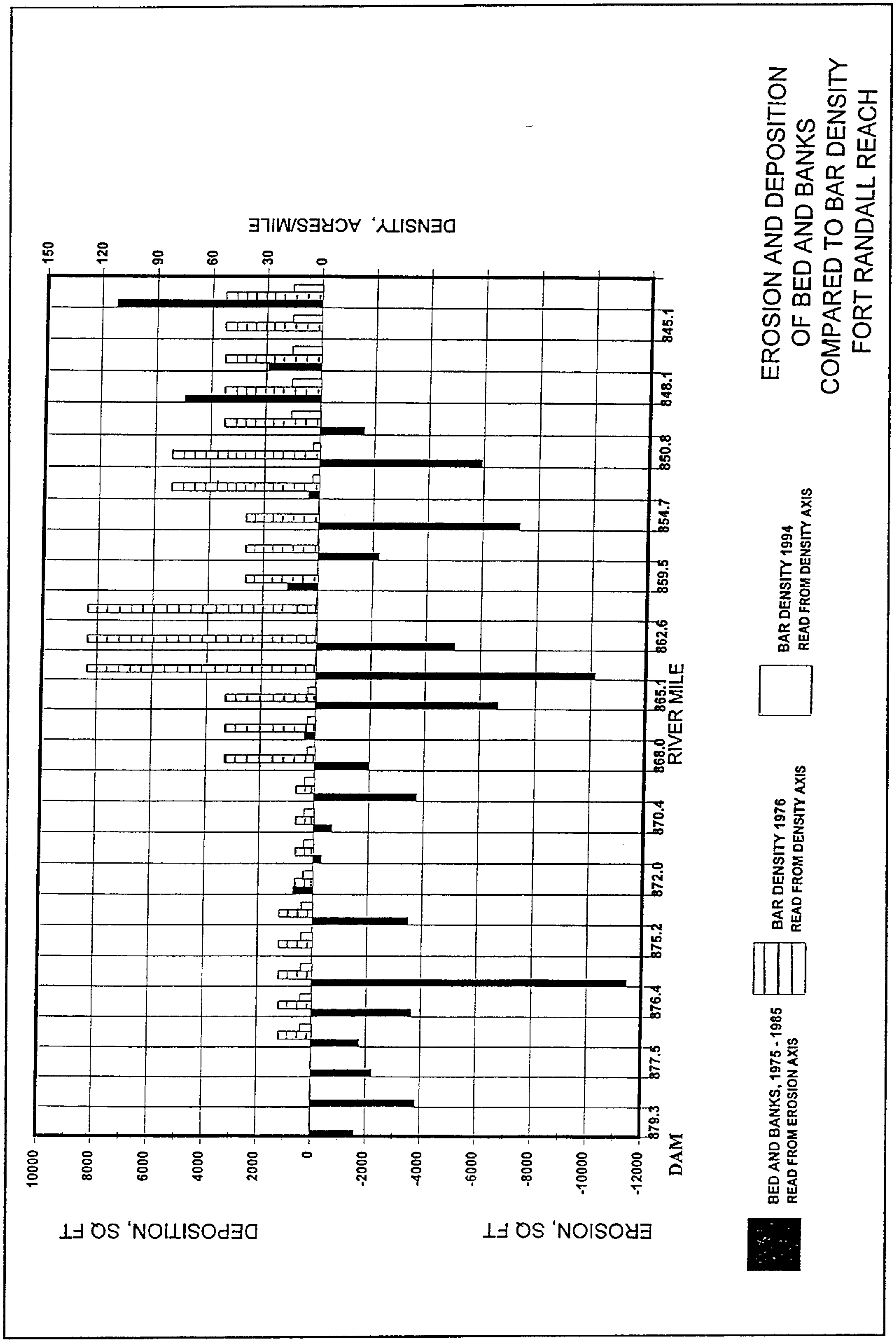
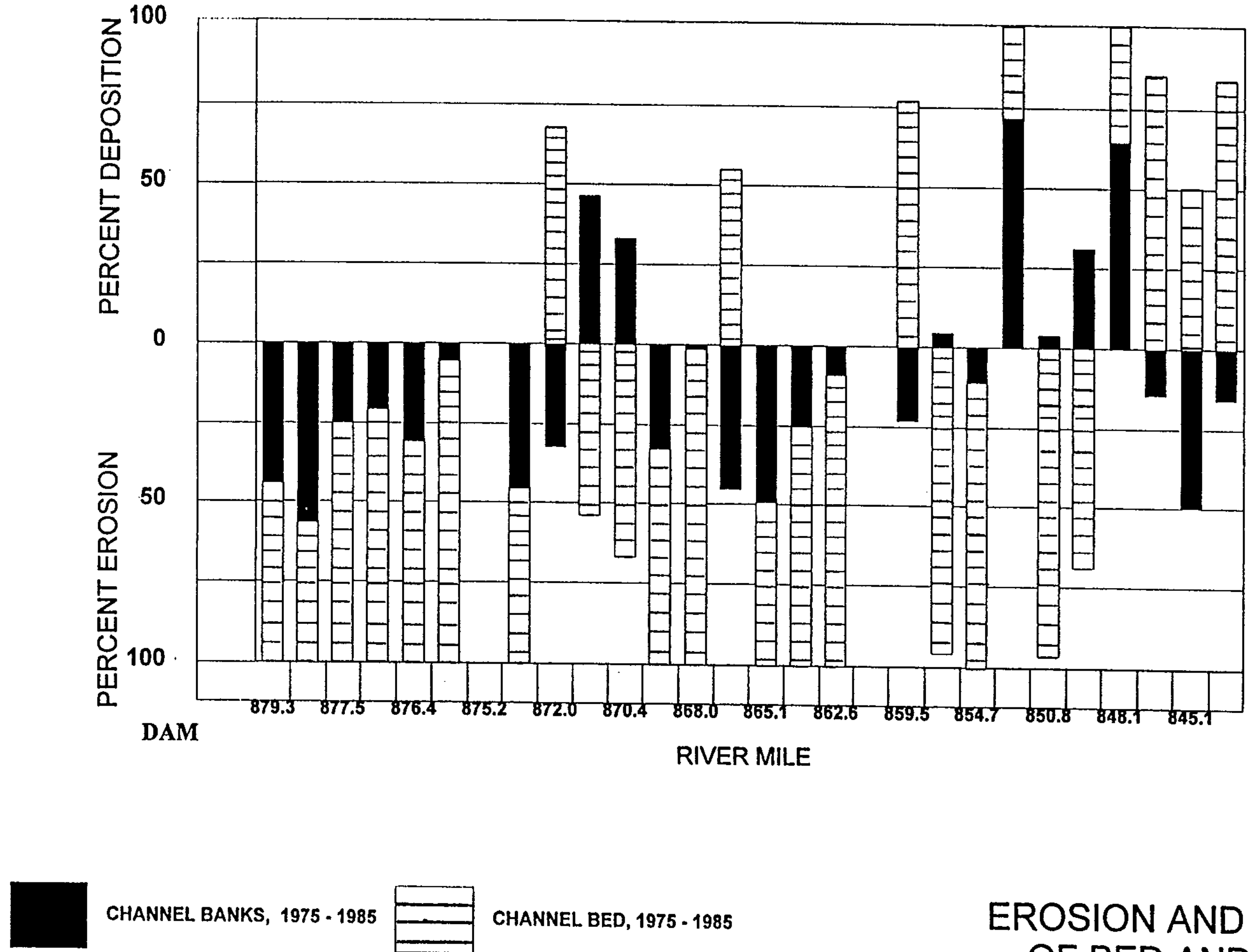
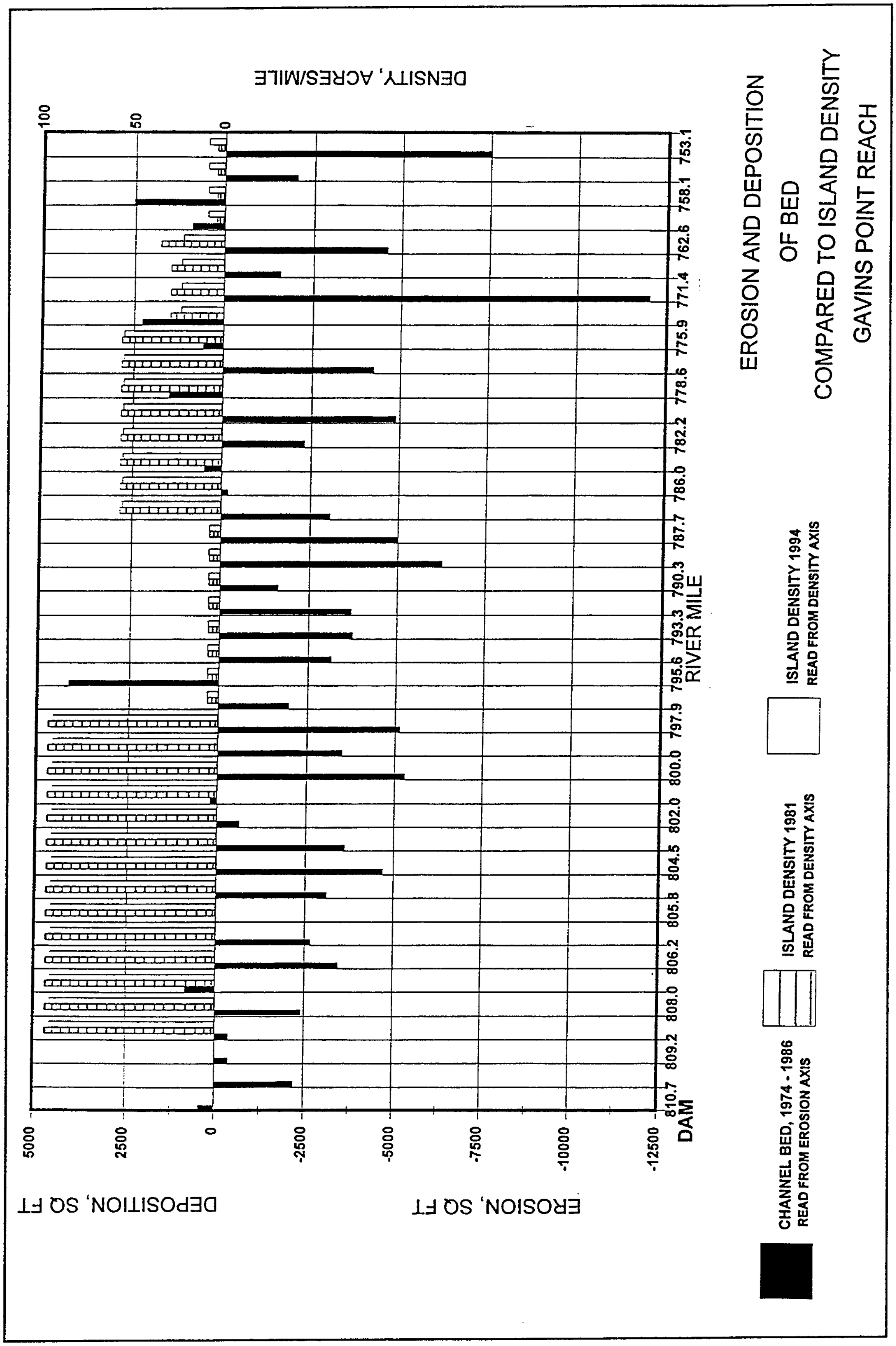

EROSION AND DEPOSITION
OF BED AND BANKS
COMPARED TO BAR DENSITY
FORT RANDALL REACH
DENSITY, ACRES/MILE
DEPOSITION, SQ FT
EROSION, SQ FT
RIVER MILE
DAM
879.3
877.5
876.4
875.2
872.0
870.4
868.0
865.1
862.6
859.5
854.7
850.8
848.1
845.1
BED AND BANKS, 1975 - 1985
READ FROM EROSION AXIS
BAR DENSITY 1976
READ FROM DENSITY AXIS
BAR DENSITY 1994
READ FROM DENSITY AXIS

Plate 36

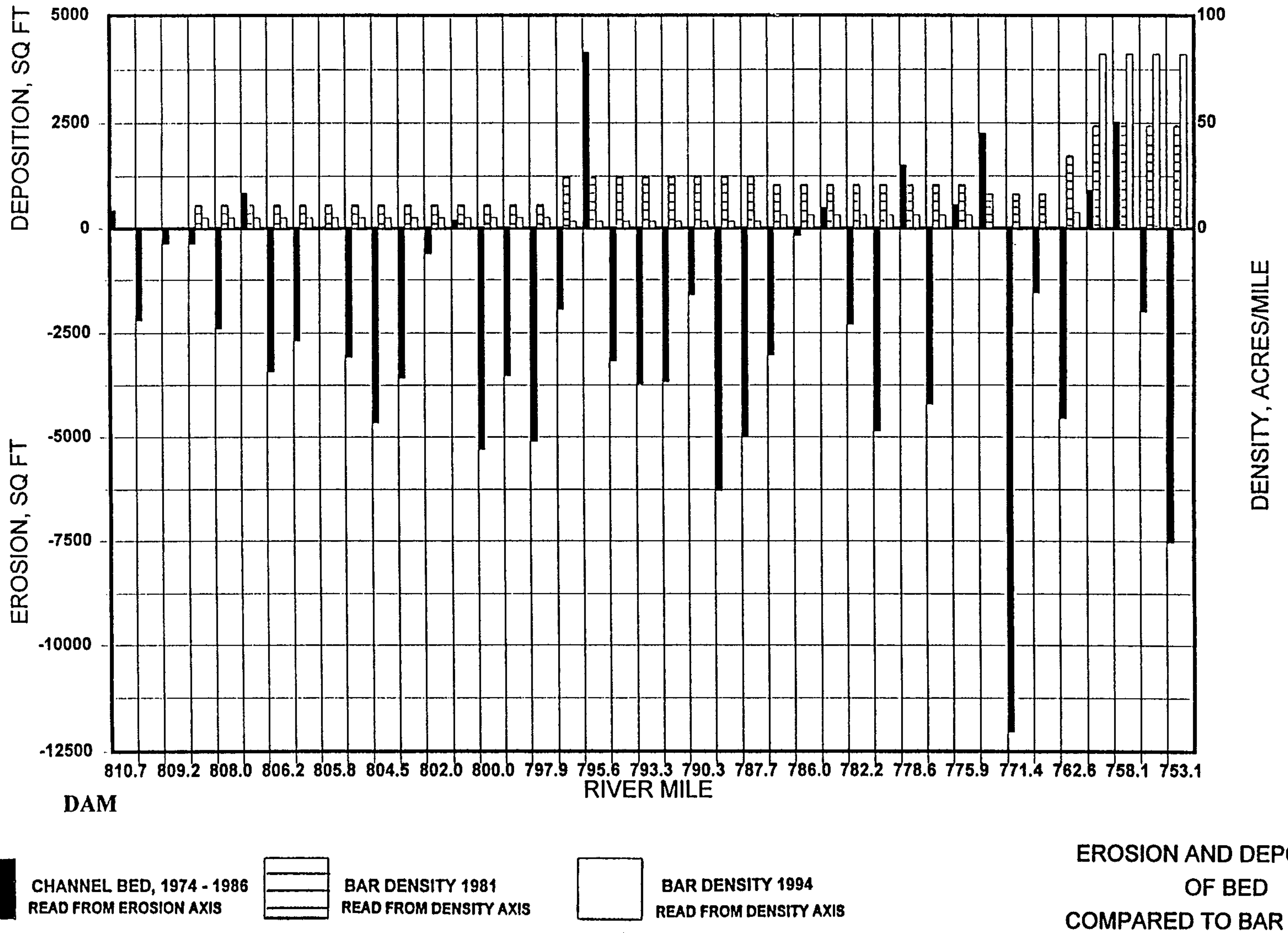


EROSION AND DEPOSITION
OF BED AND BANKS
FORT RANDALL REACH



EROSION AND DEPOSITION
OF BED
COMPARED TO ISLAND DENSITY
GAVINS POINT REACH
DENSITY, ACRES/MILE
100
50
0
DEPOSITION, SQ FT
EROSION, SQ FT
5000
2500
0
-2500
-5000
-7500
-10000
-12500
RIVER MILE
DAM
810.7
809.2
808.0
806.2
805.8
804.5
802.0
800.0
797.9
795.6
793.3
790.3
787.7
786.0
782.2
778.6
775.9
771.4
762.6
758.1
753.1
CHANNEL BED, 1974 - 1986
READ FROM EROSION AXIS
ISLAND DENSITY 1981
READ FROM DENSITY AXIS
ISLAND DENSITY 1994
READ FROM DENSITY AXIS

Plate 38



EROSION AND DEPOSITION
OF BED
COMPARED TO BAR DENSITY
GAVINS POINT REACH

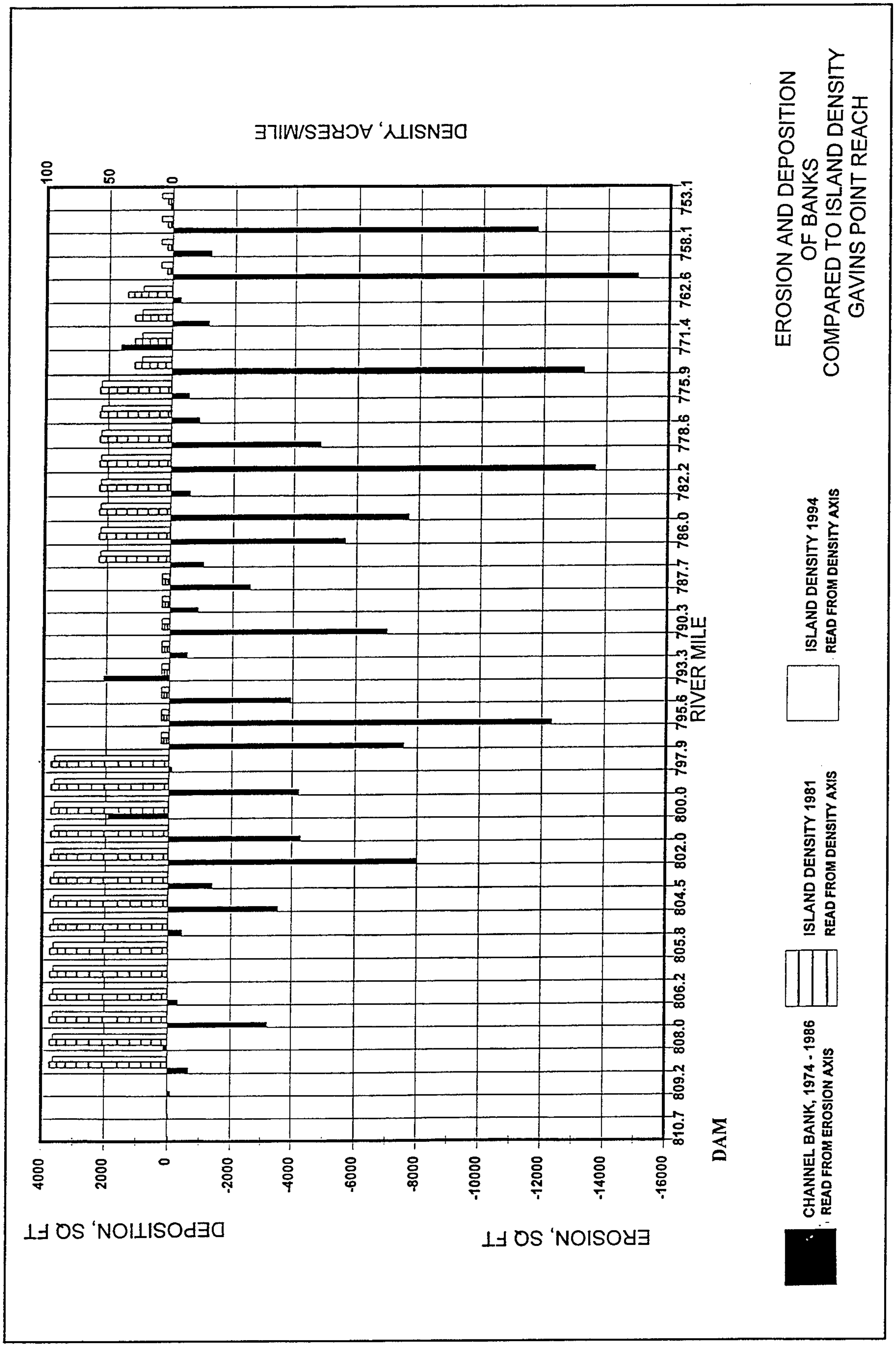
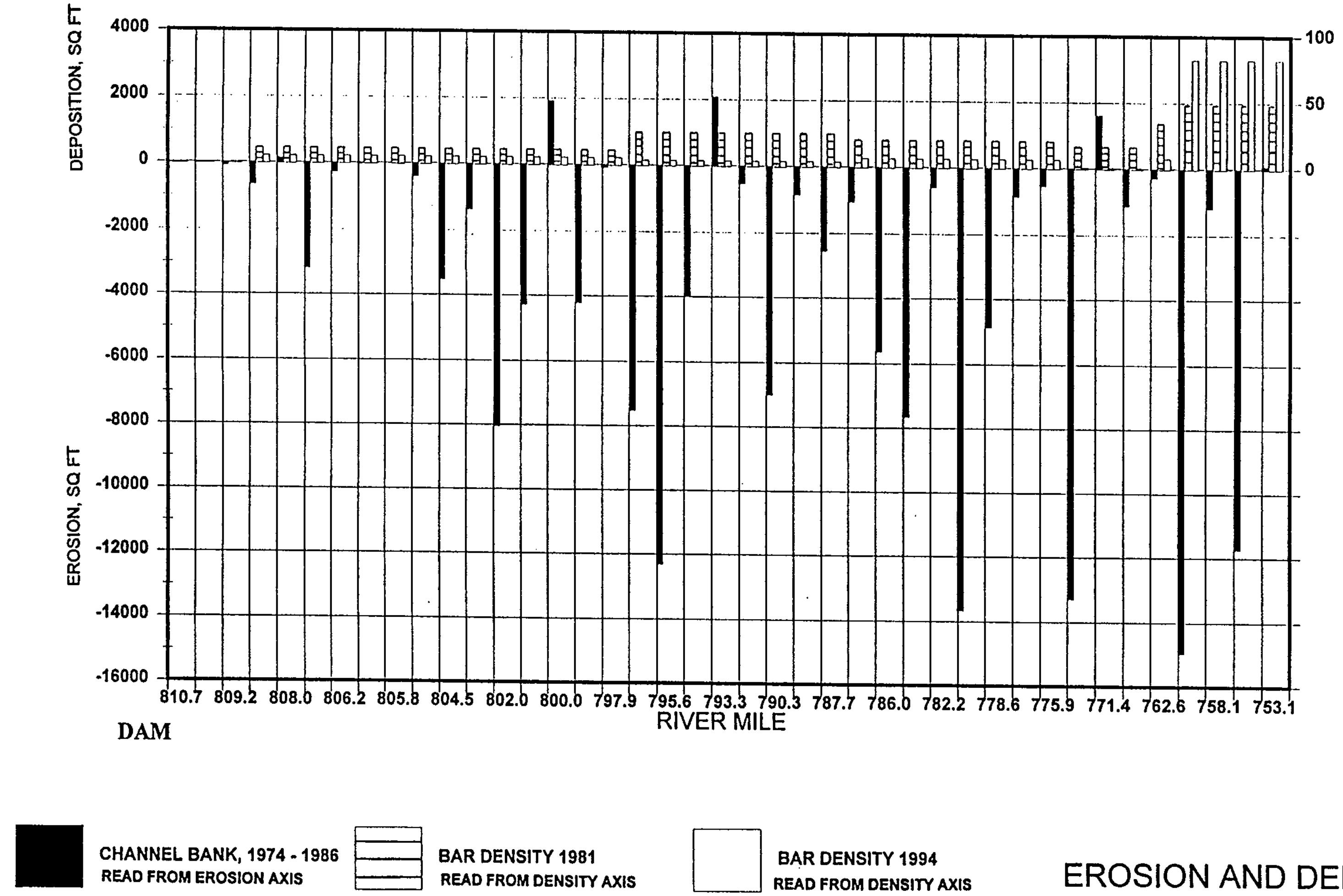

EROSION AND DEPOSITION
OF BANKS
COMPARED TO ISLAND DENSITY
GAVINS POINT REACH
DENSITY, ACRES/MILE
100
50
0
DEPOSITION, SQ FT
EROSION, SQ FT
4000
2000
0
-2000
-4000
-6000
-8000
-10000
-12000
-14000
-16000
RIVER MILE
DAM
810.7
809.2
808.0
806.2
805.8
804.5
802.0
800.0
797.9
795.6
793.3
790.3
787.7
786.0
782.2
778.6
775.9
771.4
762.6
758.1
753.1
CHANNEL BANK, 1974 - 1986
READ FROM EROSION AXIS
ISLAND DENSITY 1981
READ FROM DENSITY AXIS
ISLAND DENSITY 1994
READ FROM DENSITY AXIS

Plate 40



EROSION AND DEPOSITION
OF BANKS
COMPARED TO BAR DENSITY
GAVINS POINT REACH

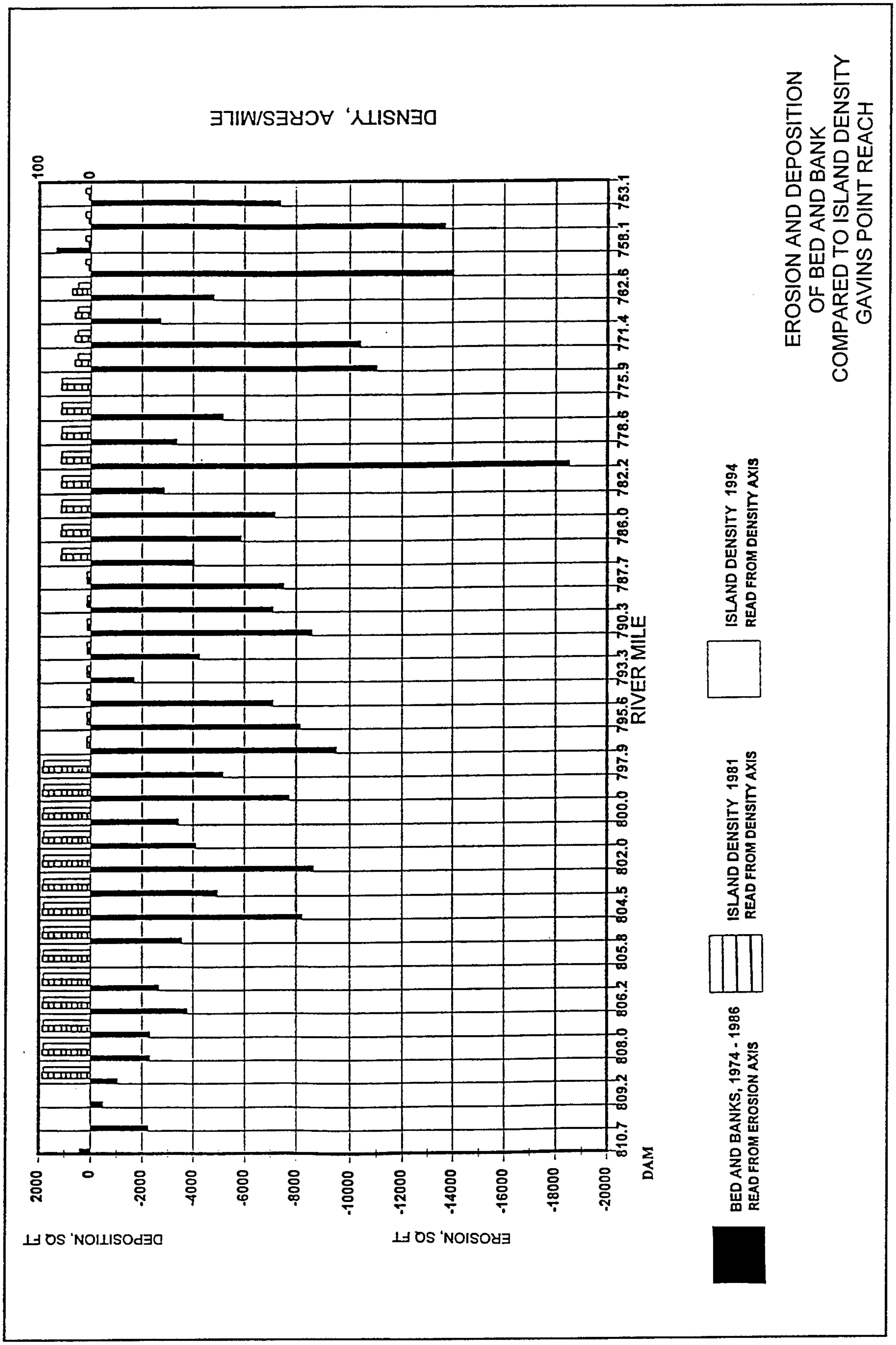
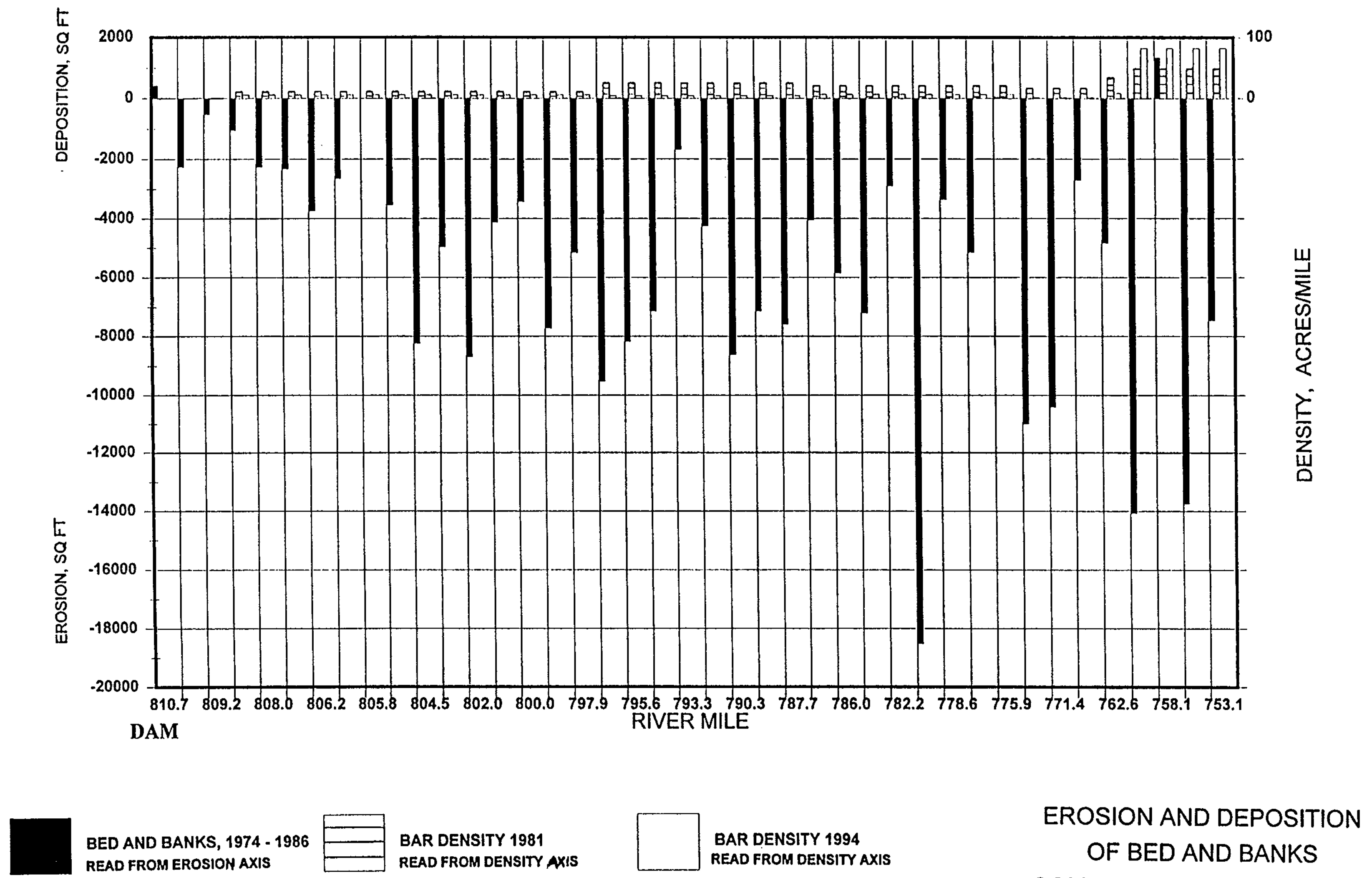

EROSION AND DEPOSITION
OF BED AND BANK
COMPARED TO ISLAND DENSITY
GAVINS POINT REACH
DENSITY, ACRES/MILE
DEPOSITION, SQ FT
EROSION, SQ FT
RIVER MILE
DAM
810.7 809.2 808.0 806.2 805.8 804.5 802.0 800.0 797.9 795.6 793.3 790.3 787.7 786.0 782.2 778.6 775.9 771.4 762.6 758.1 753.1
2000 0 -2000 -4000 -6000 -8000 -10000 -12000 -14000 -16000 -18000 -20000
0 100
BED AND BANKS, 1974 - 1986
READ FROM EROSION AXIS
ISLAND DENSITY 1981
READ FROM DENSITY AXIS
ISLAND DENSITY 1994
READ FROM DENSITY AXIS

Plate 42



EROSION AND DEPOSITION
OF BED AND BANKS
COMPARED TO BAR DENSITY
GAVINS POINT REACH

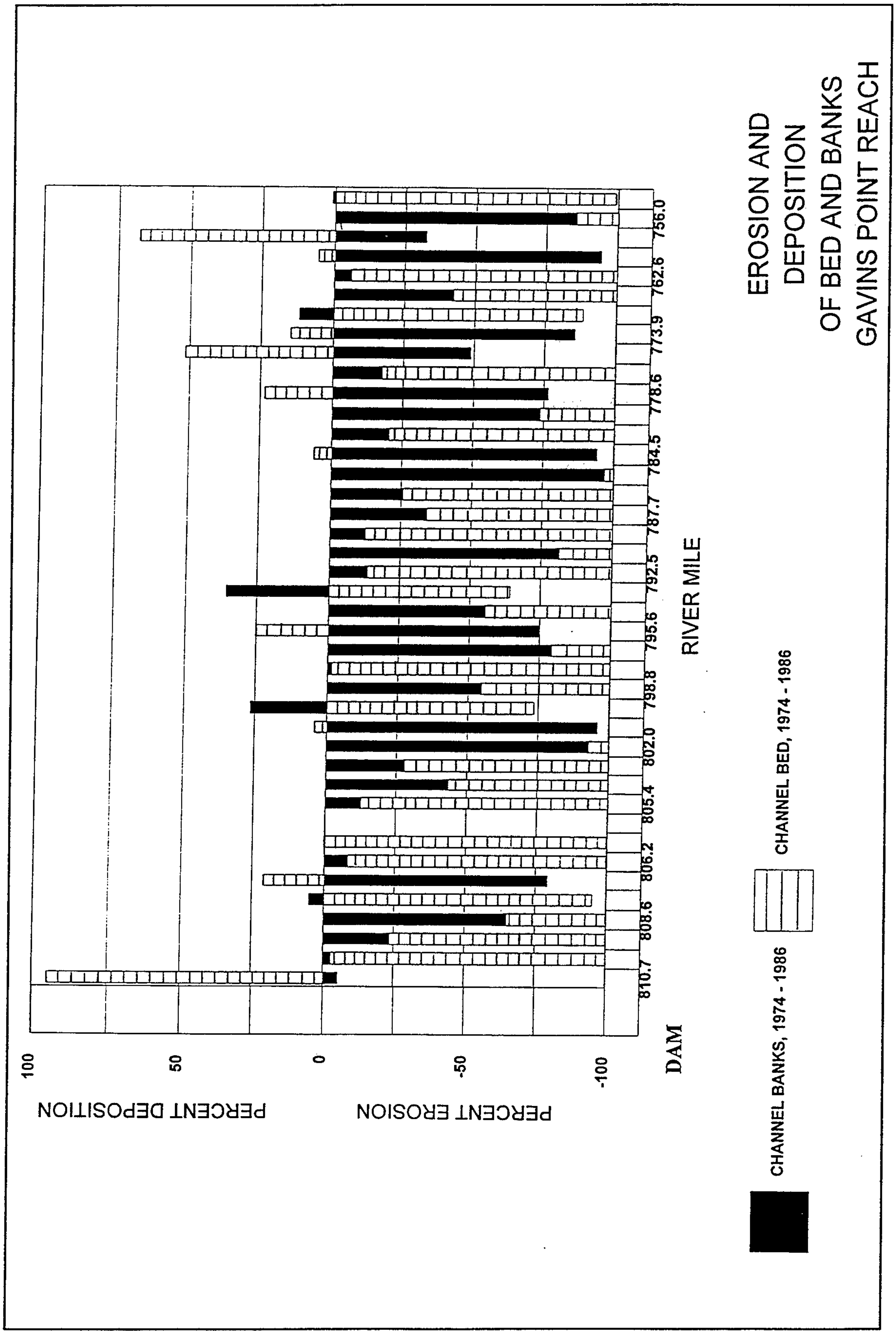
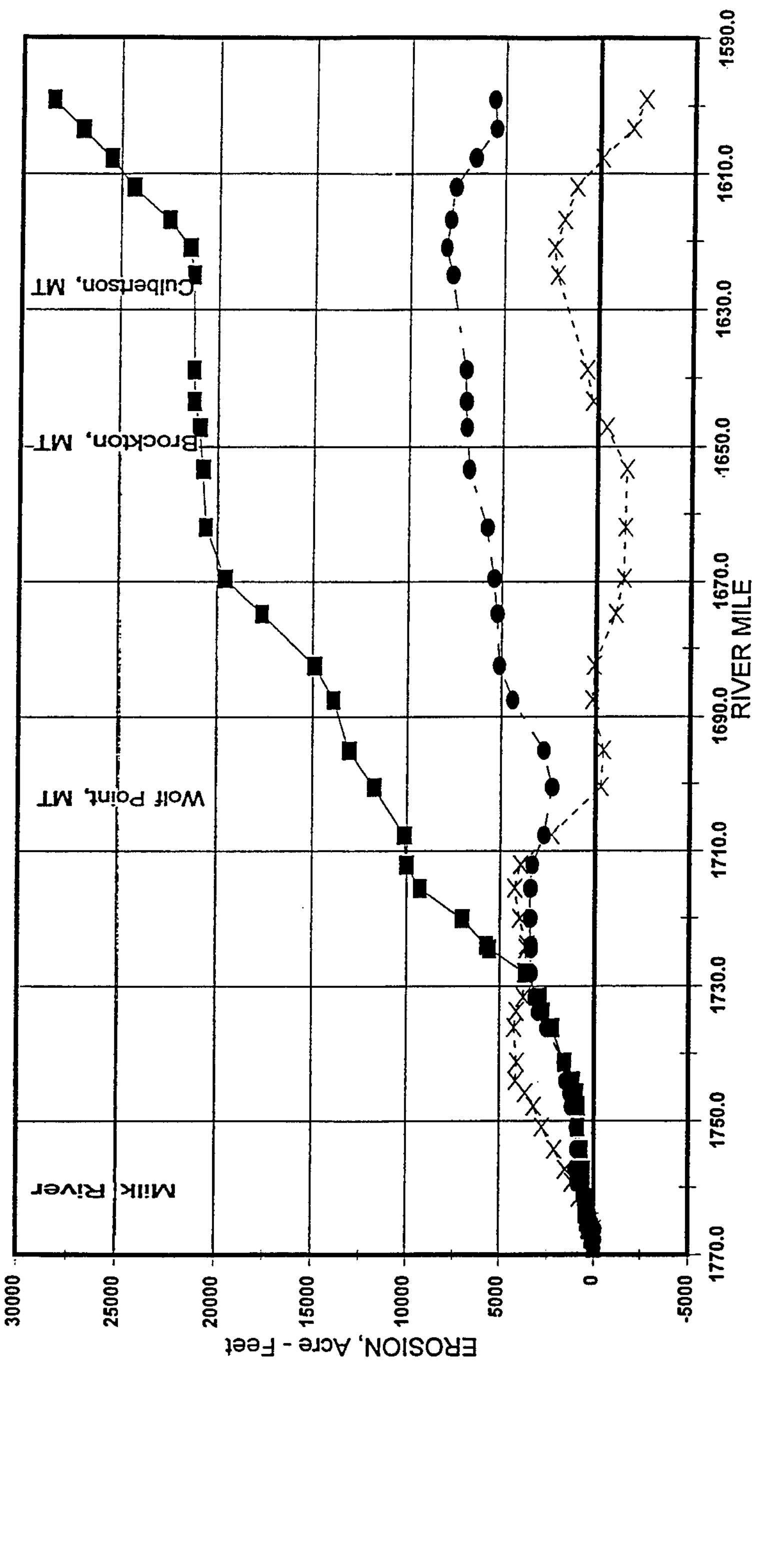
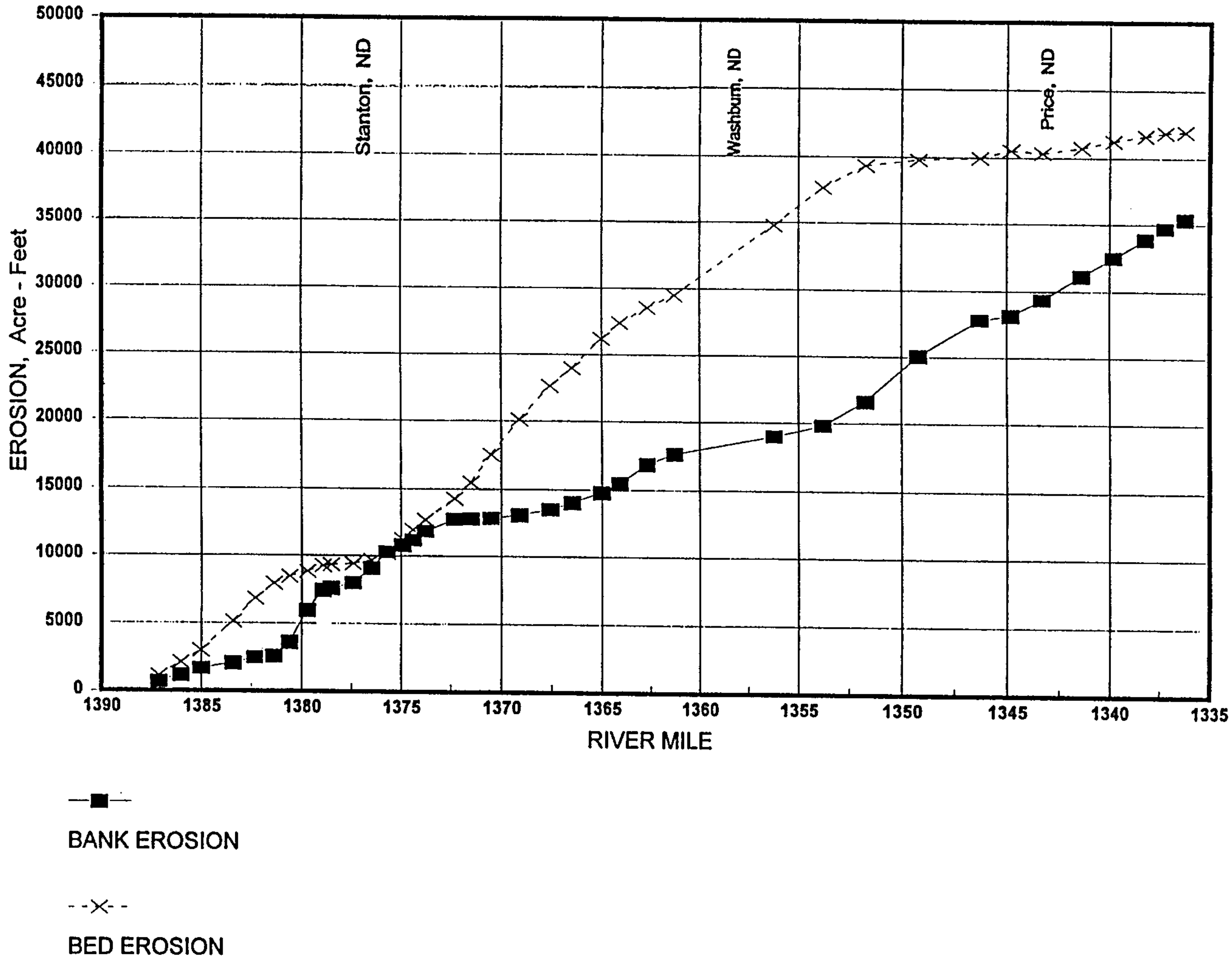

EROSION AND DEPOSITION OF BED AND BANKS GAVINS POINT REACH
PERCENT DEPOSITION
PERCENT EROSION
100
50
0
-50
-100
DAM
810.7
808.6
806.2
805.4
802.0
798.8
795.6
792.5
787.7
784.5
778.6
773.9
762.6
756.0
RIVER MILE
CHANNEL BANKS, 1974 - 1986
CHANNEL BED, 1974 - 1986

Plate 44

CUMULATIVE BED AND BANK EROSION
FORT PECK REACH

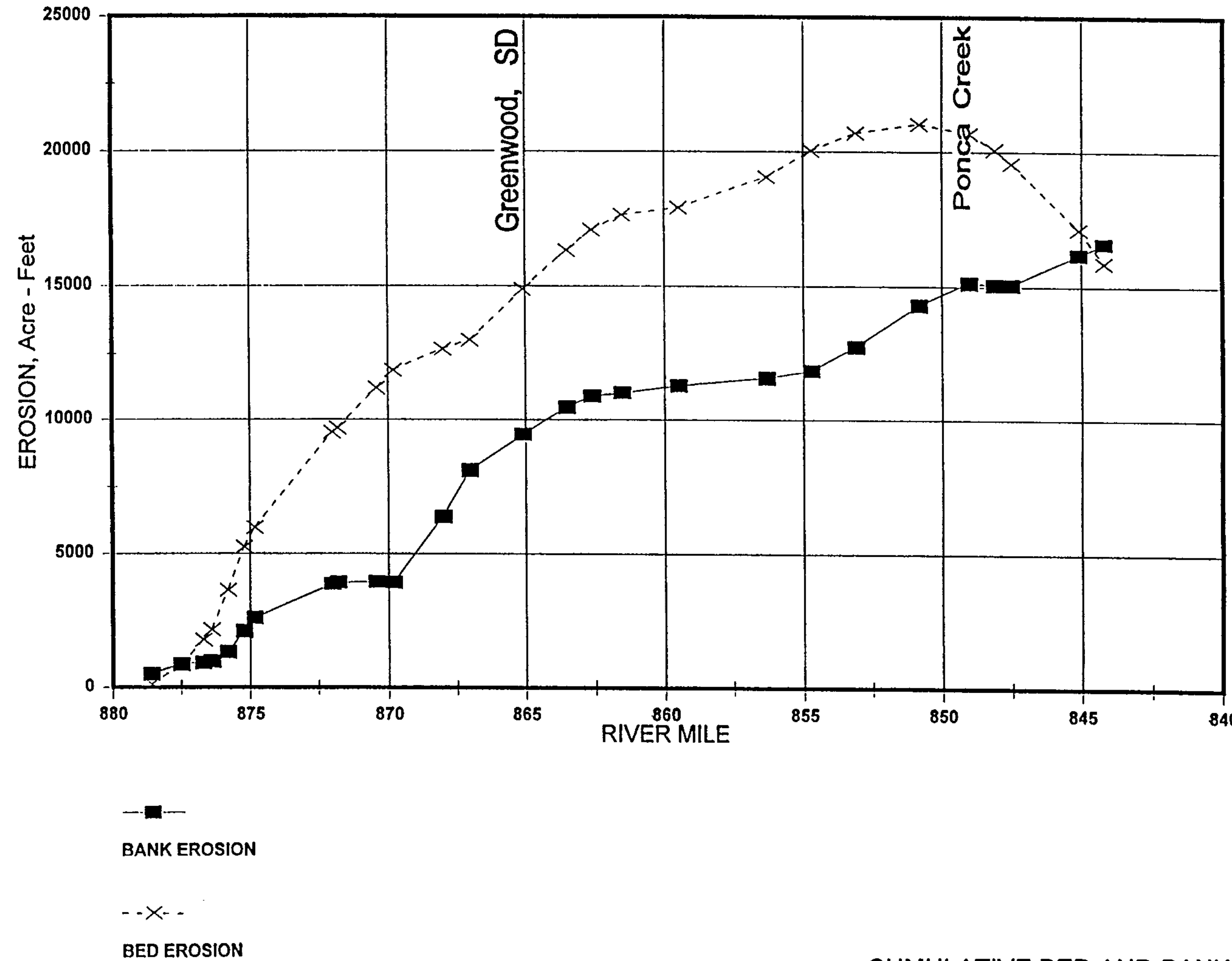


BANK EROSION 1955 - 1978 — BED EROSION, 1955 - 1978 — BED EROSION, 1966 - 1978

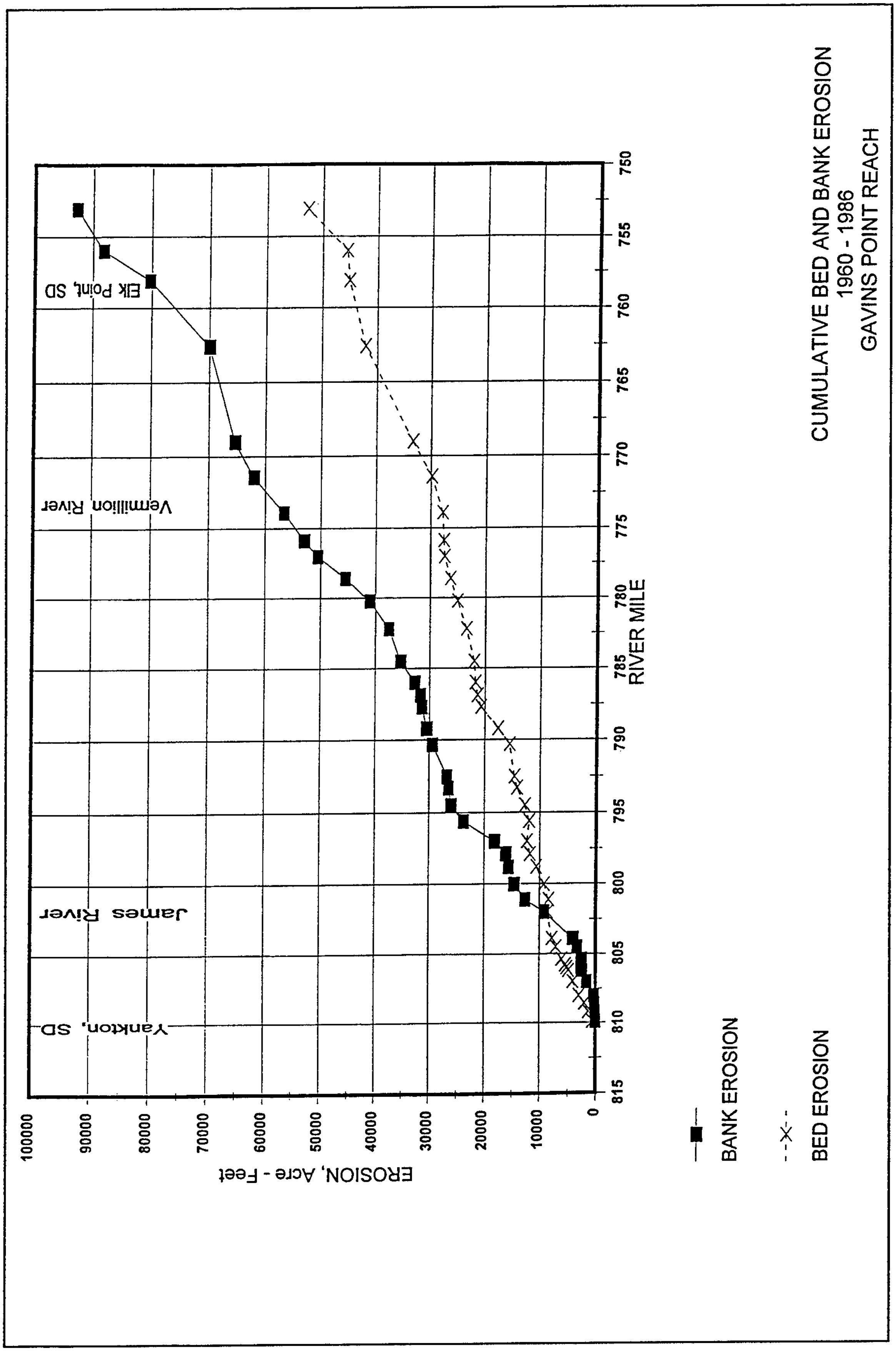


CUMULATIVE BED AND BANK EROSION
1956 - 1985
GARRISON REACH

Plate 46

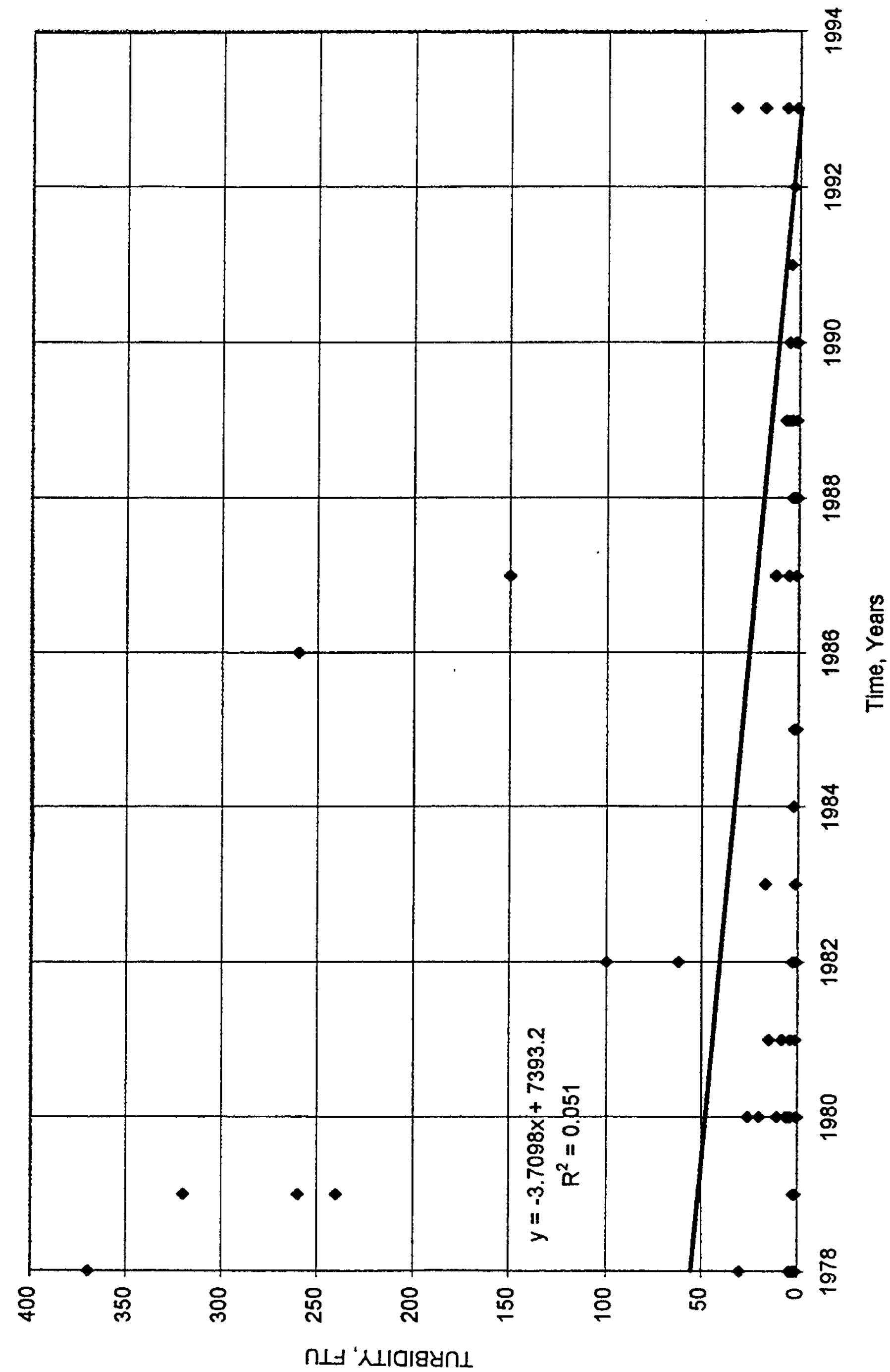


CUMULATIVE BED AND BANK EROSION
1954 - 1985
FORT RANDALL REACH

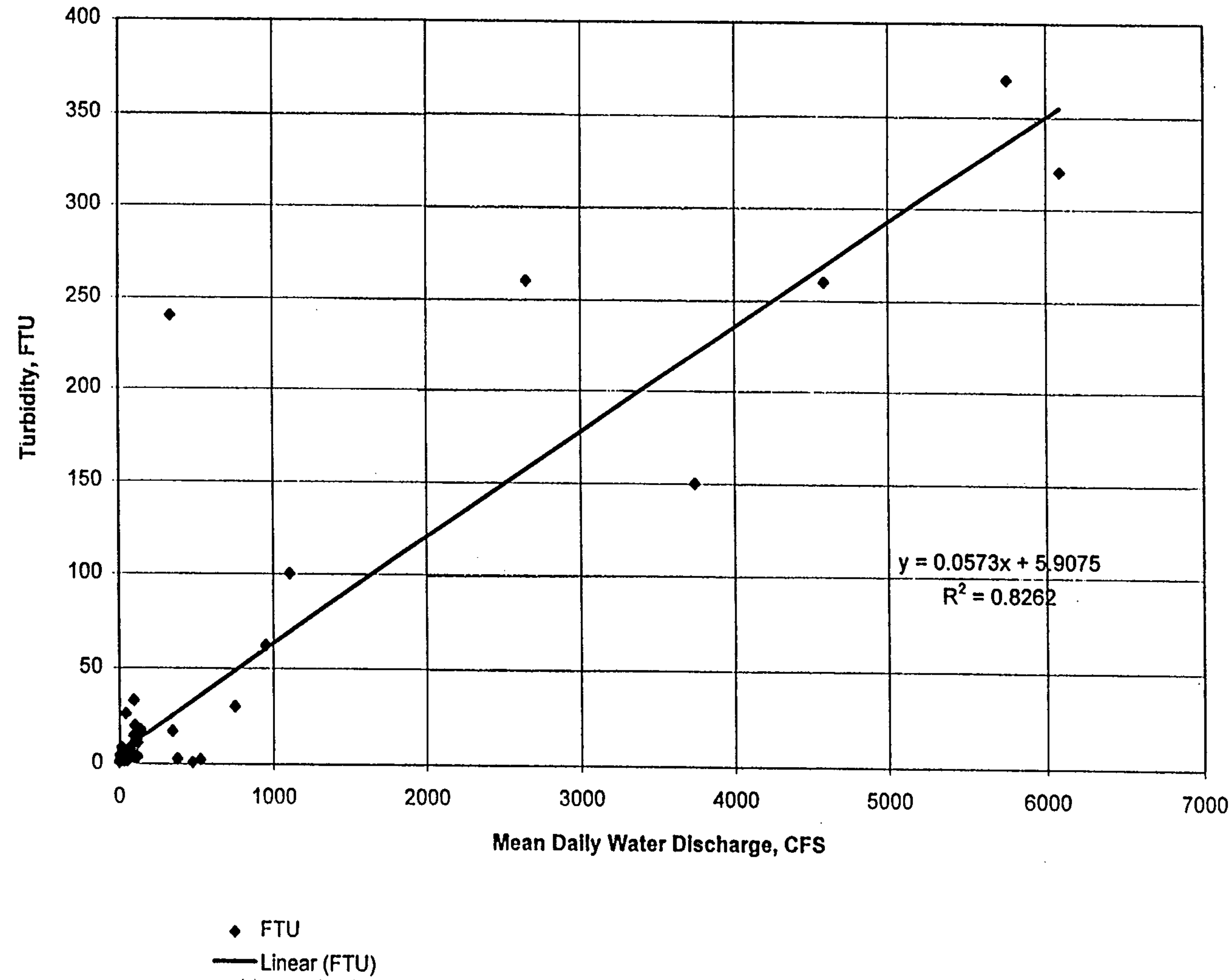


CUMULATIVE BED AND BANK EROSION
1960 - 1986
GAVINS POINT REACH

Plate 48

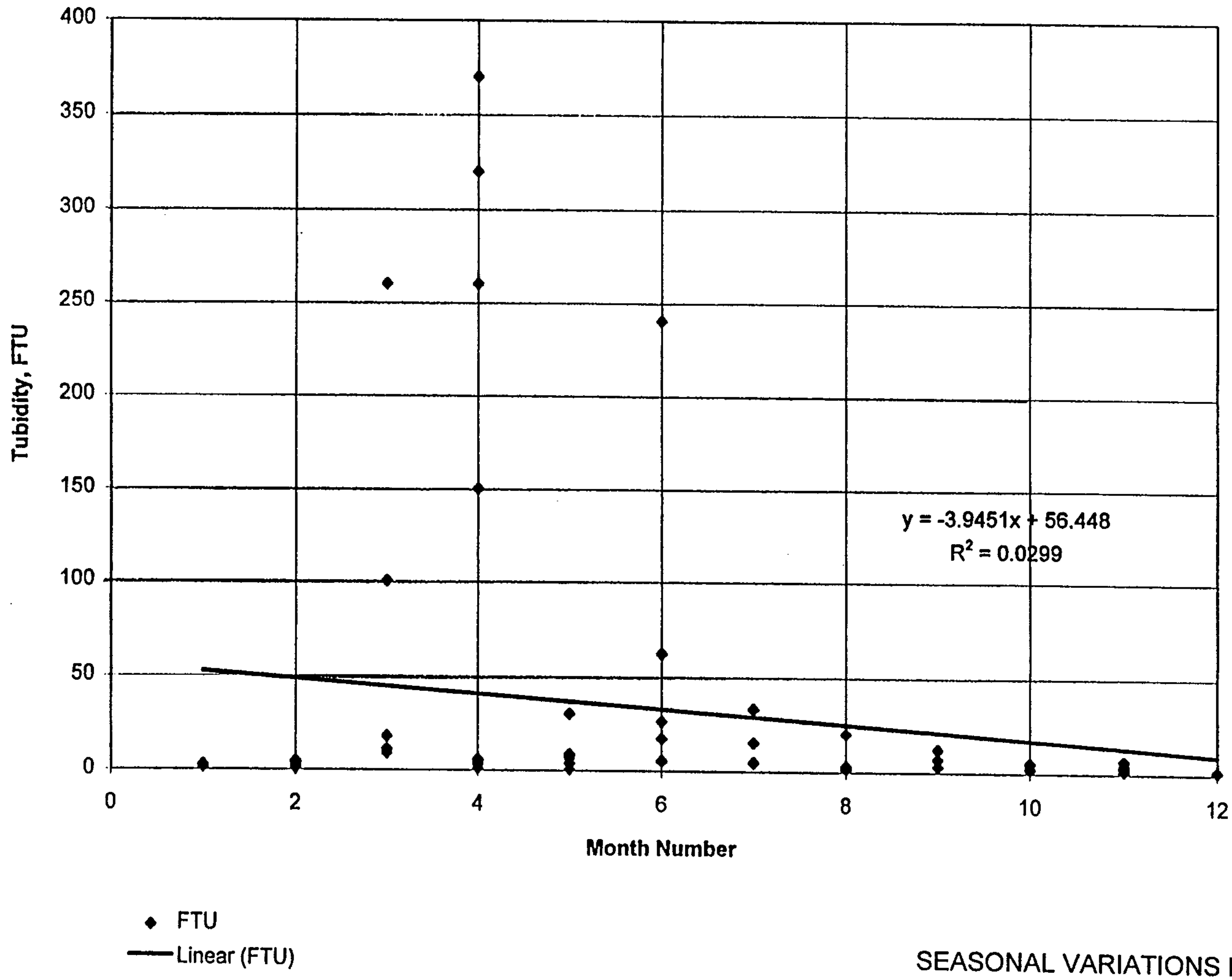


TURBIDITY MEASUREMENTS
HEART RIVER
MANDAN, ND



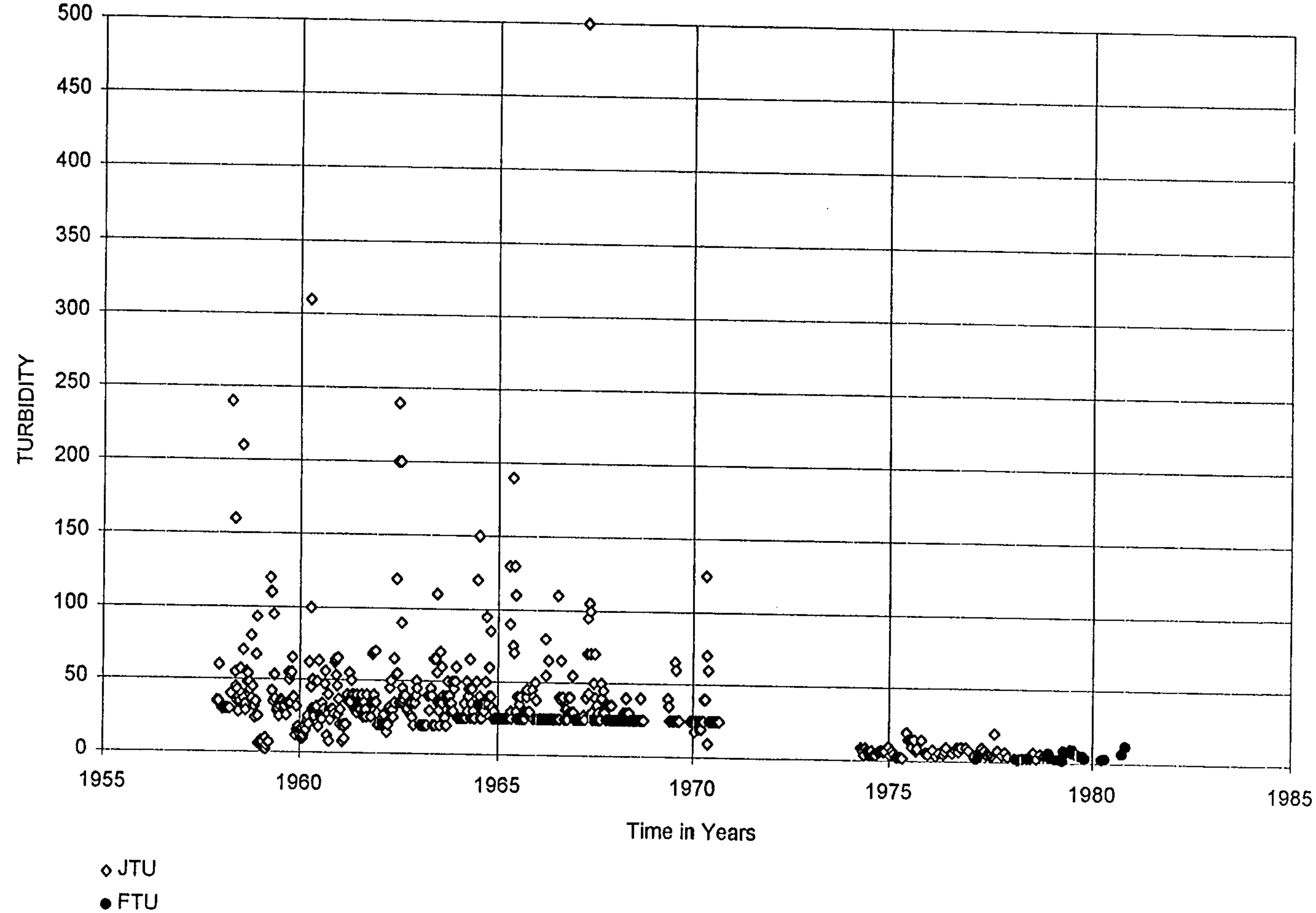
TURBIDITY VERSUS WATER DISCHARGE
HEART RIVER
MANDAN, ND

Plate 50

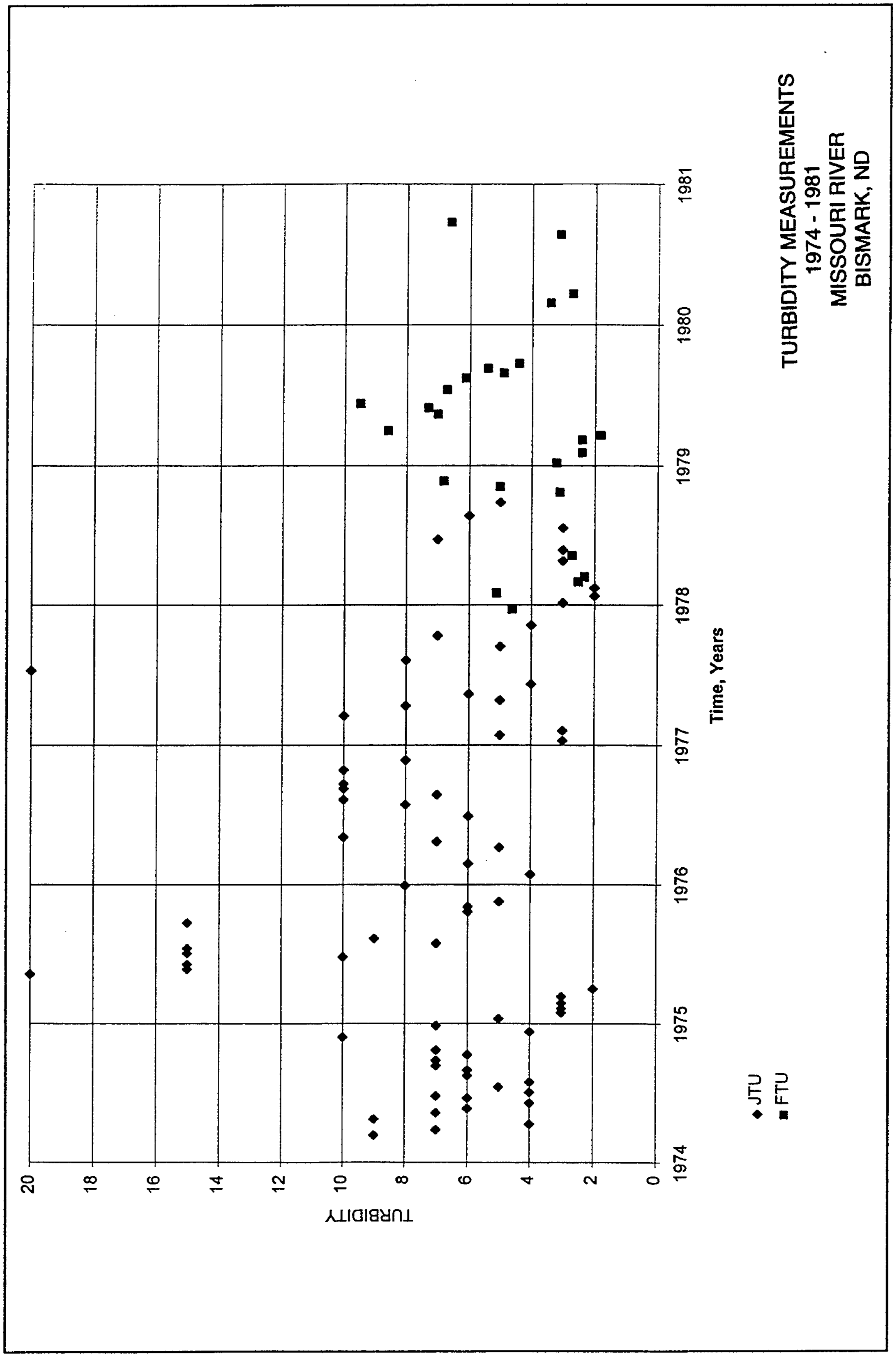


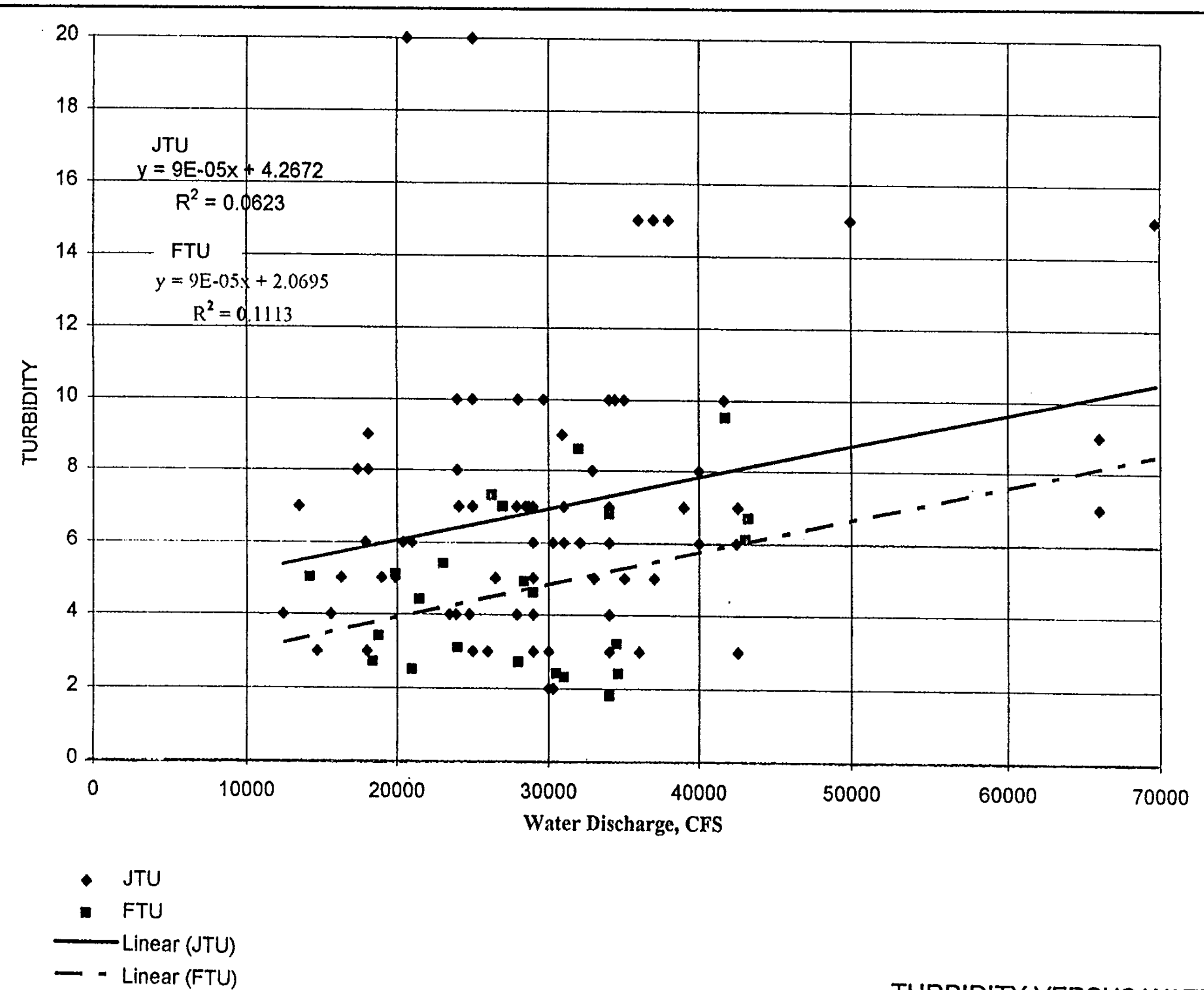
SEASONAL VARIATIONS IN TURBIDITY
HEART RIVER
MANDAN, ND

Plate 52

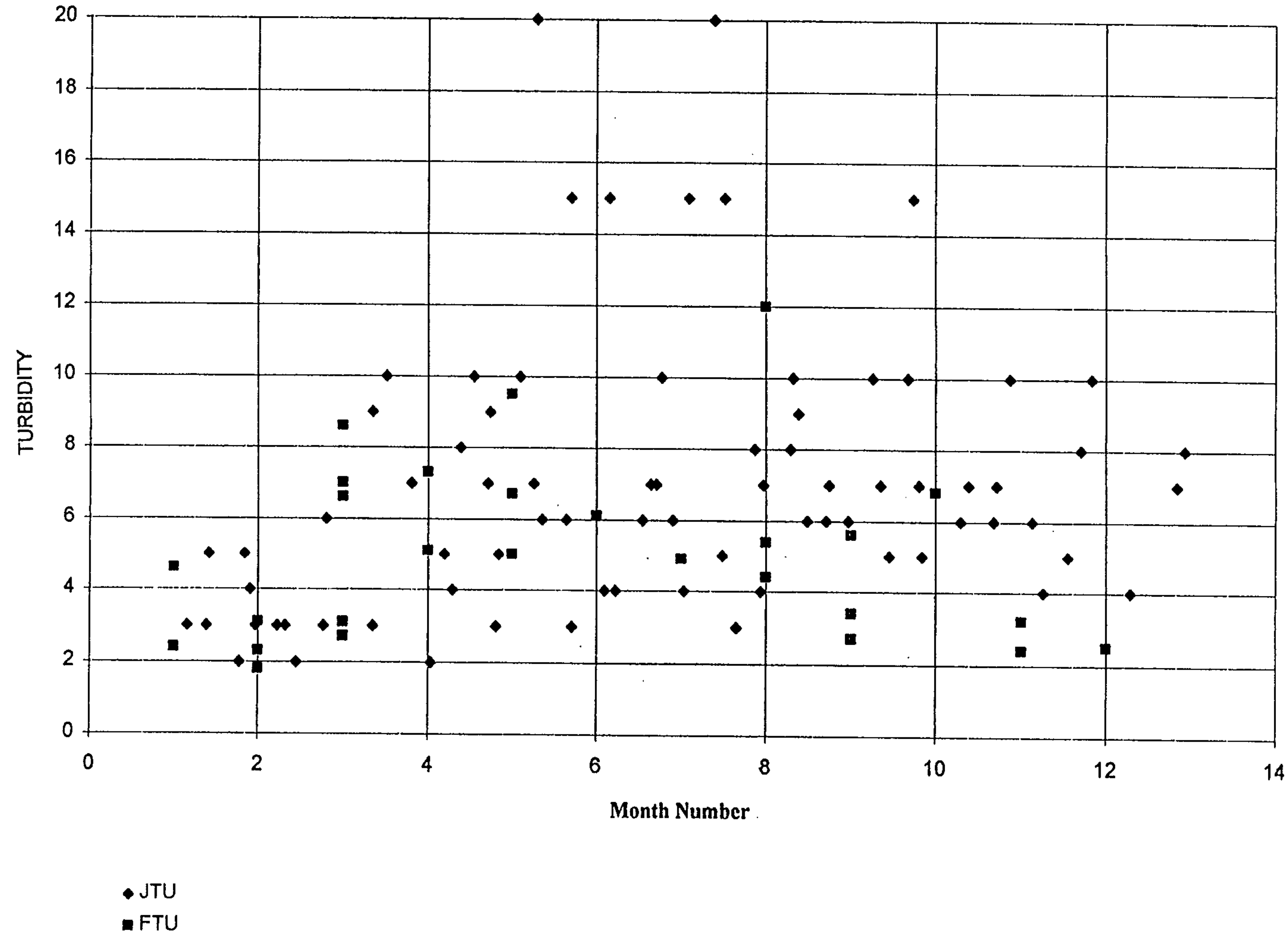


TURBIDITY MEASUREMENTS
1955 - 1985
MISSOURI RIVER
BISMARK, ND


TURBIDITY MEASUREMENTS
1974 - 1981
MISSOURI RIVER
BISMARK, ND
TURBIDITY
20
18
16
14
12
10
8
6
4
2
0
1974
1975
1976
1977
1978
1979
1980
1981
Time, Years
JTU
FTU



TURBIDITY VERSUS WATER DISCHARGE
MISSOURI RIVER
BISMARCK, ND



SEASONAL TREND IN TURBIDITY
MISSOURI RIVER
BISMARCK, ND

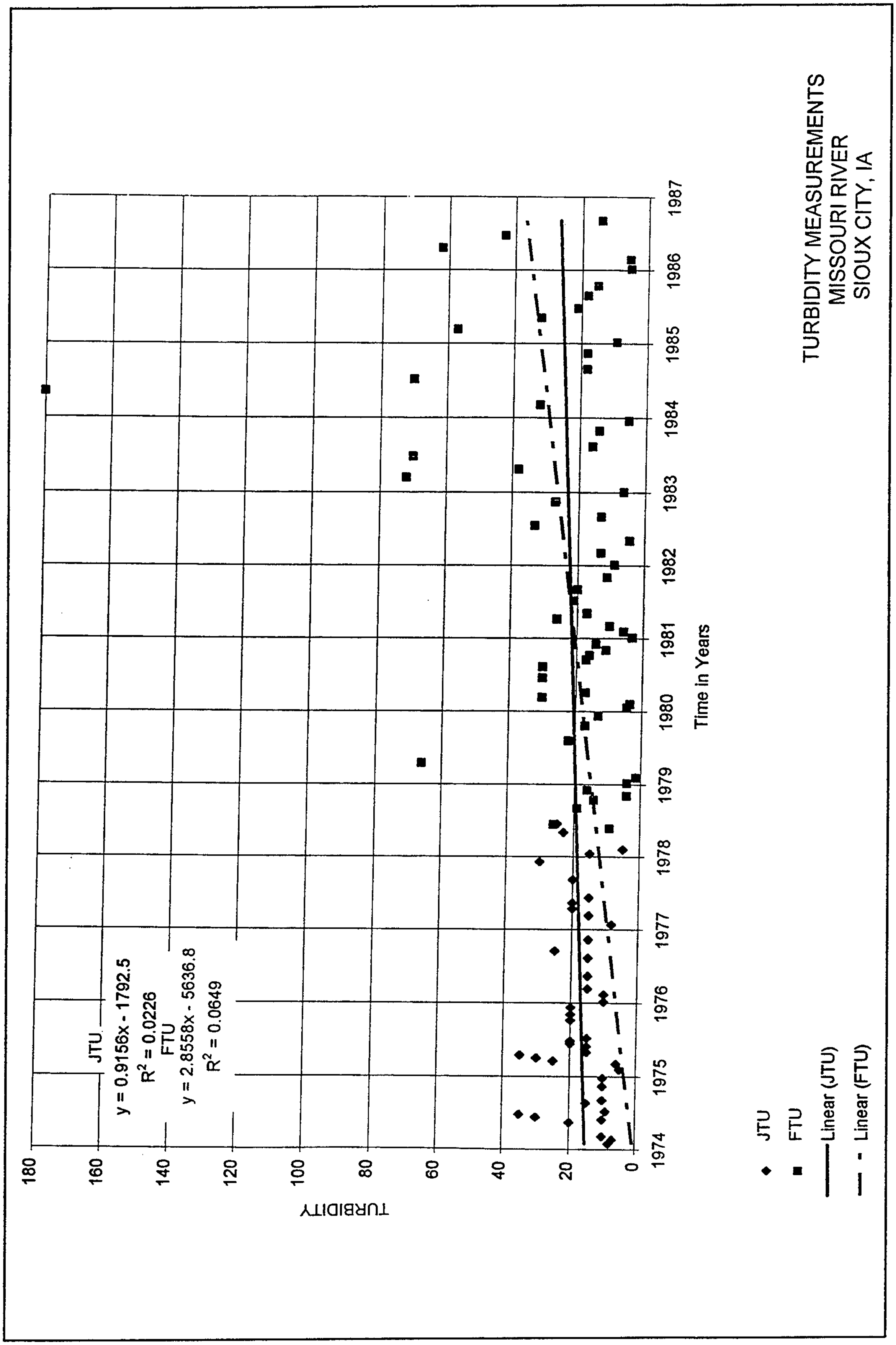
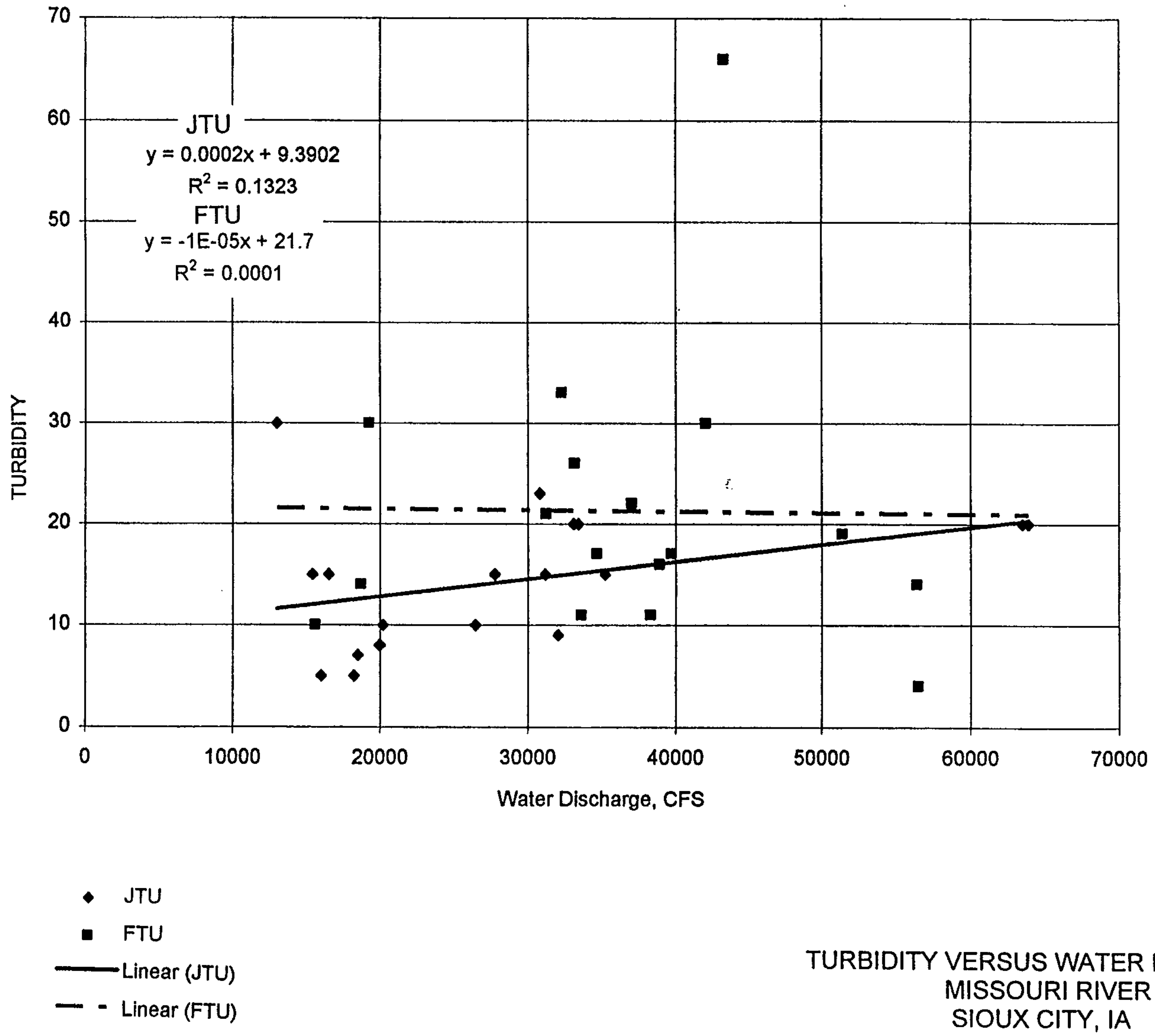

TURBIDITY MEASUREMENTS
MISSOURI RIVER
SIOUX CITY, IA
TURBIDITY
Time in Years
180
160
140
120
100
80
60
40
20
0
1974
1975
1976
1977
1978
1979
1980
1981
1982
1983
1984
1985
1986
1987
JTU
y = 0.9156x - 1792.5
R^2 = 0.0226
FTU
y = 2.8558x - 5636.8
R^2 = 0.0649
JTU
FTU
Linear (JTU)
Linear (FTU)

Plate 56



TURBIDITY VERSUS WATER DISCHARGE
MISSOURI RIVER
SIOUX CITY, IA

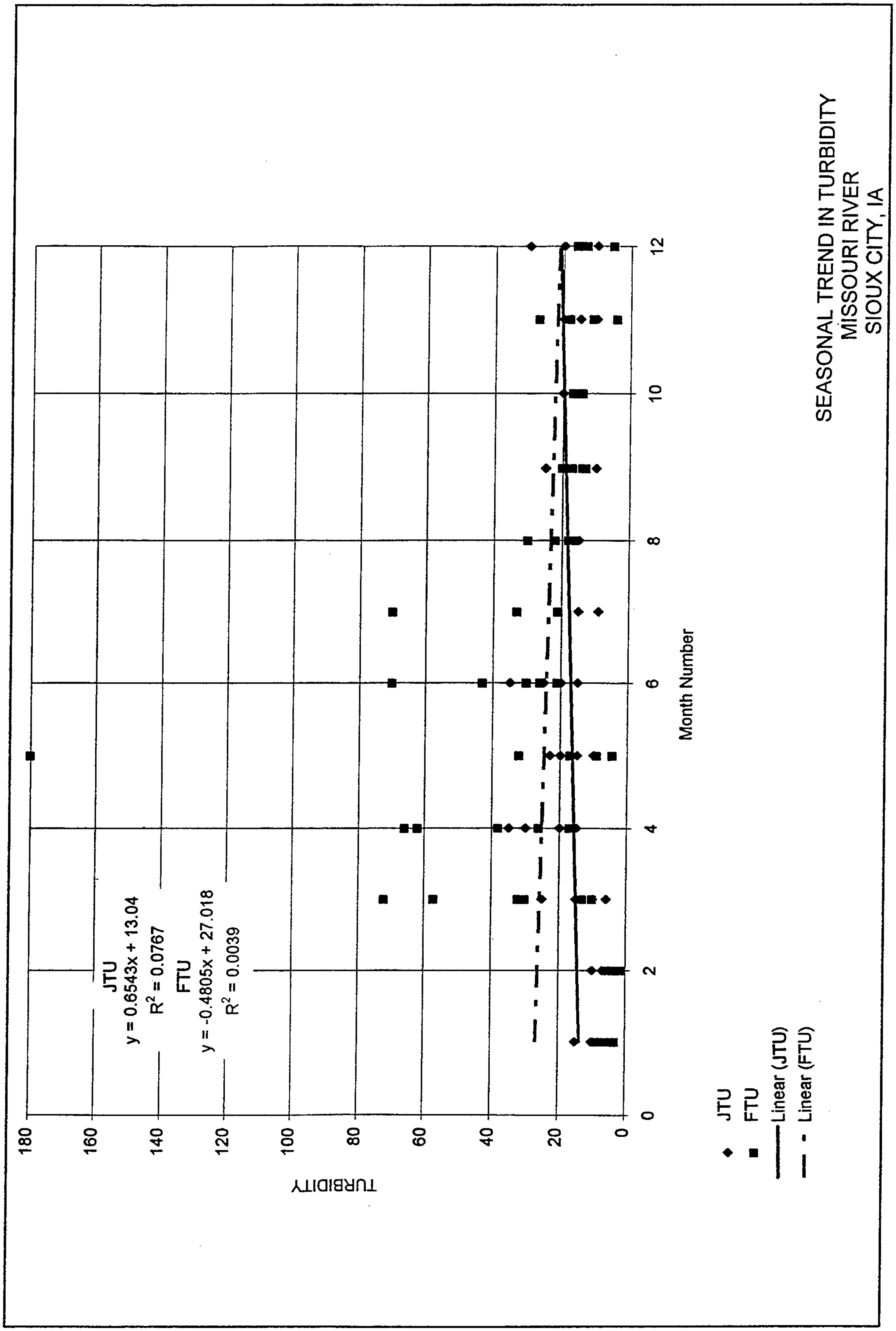
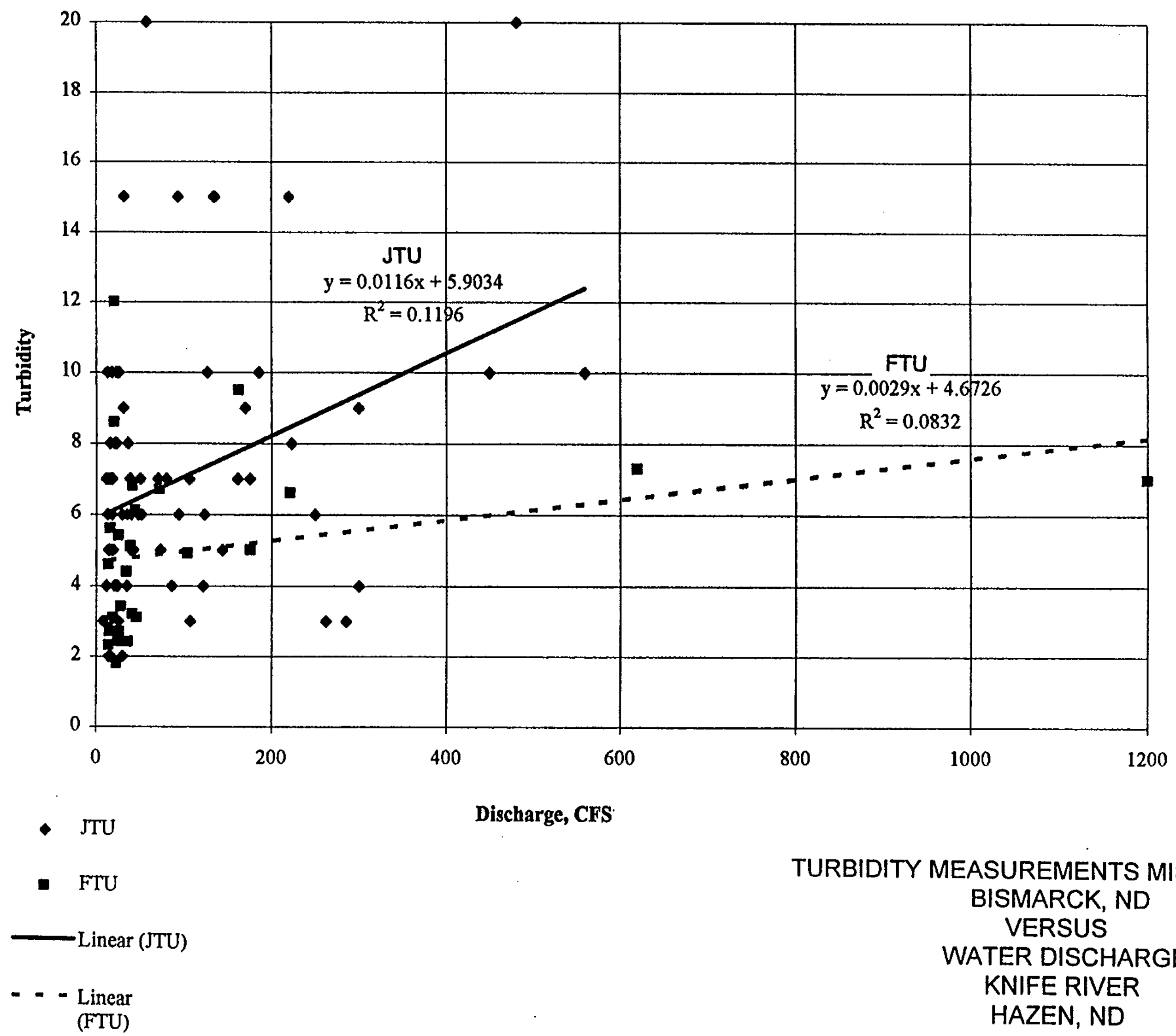
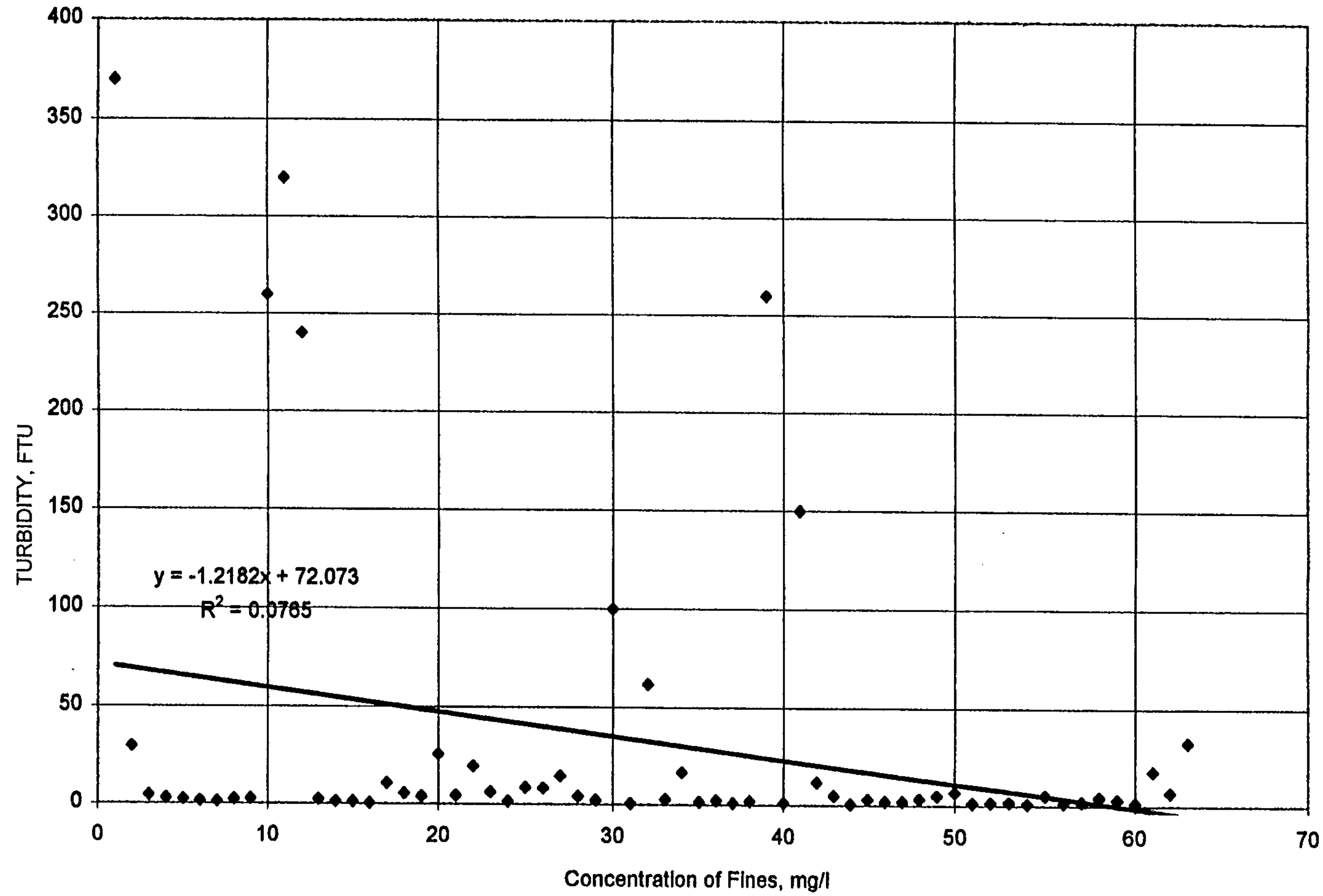

SEASONAL TREND IN TURBIDITY
MISSOURI RIVER
SIOUX CITY, IA
TURBIDITY
Month Number
JTU
y = 0.6543x + 13.04
R2 = 0.0767
FTU
y = -0.4805x + 27.018
R2 = 0.0039
JTU
FTU
Linear (JTU)
Linear (FTU)
0
20
40
60
80
100
120
140
160
180
0
2
4
6
8
10
12

Plate 58

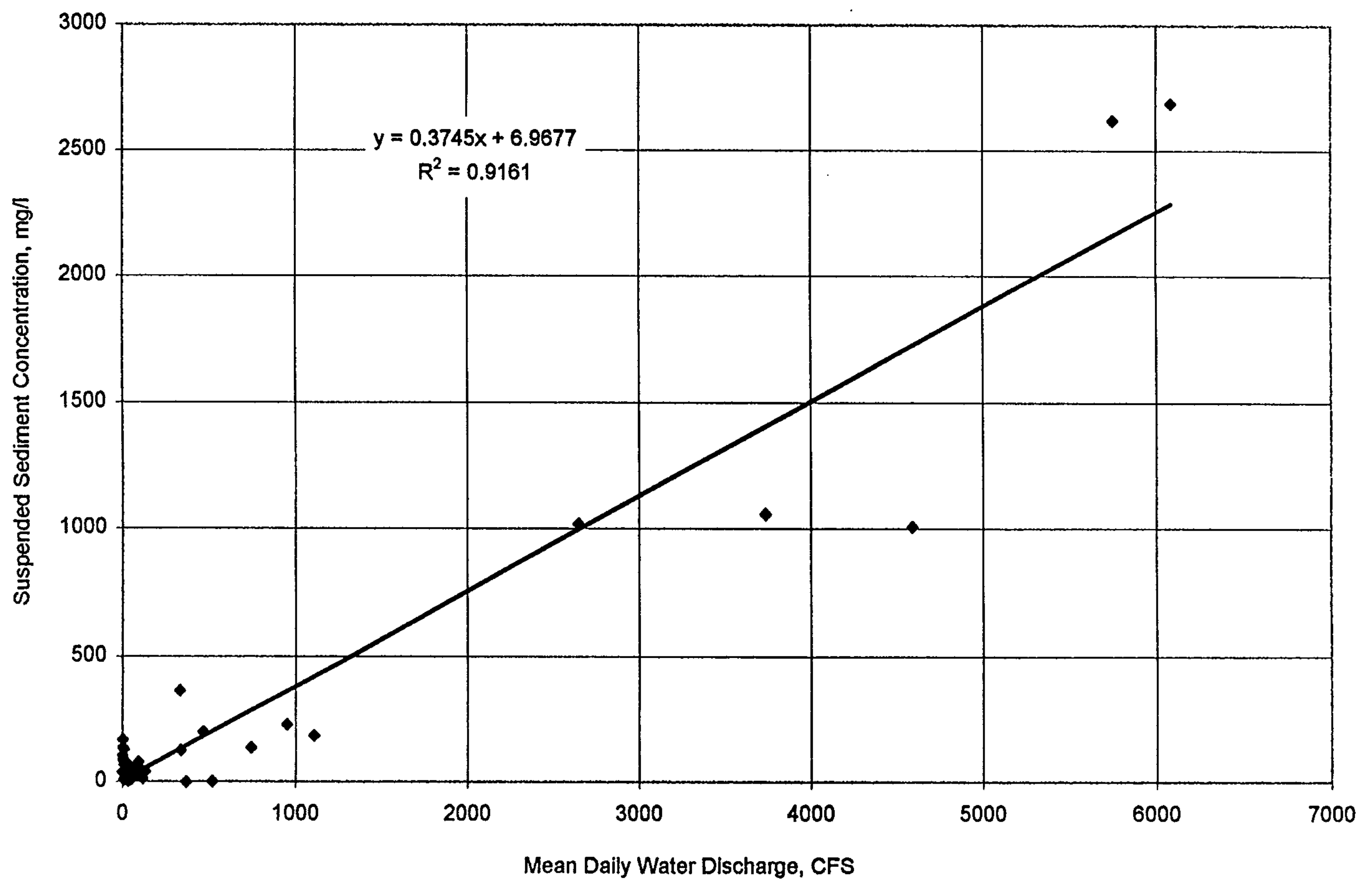


TURBIDITY MEASUREMENTS MISSOURI RIVER
BISMARCK, ND
VERSUS
WATER DISCHARGE
KNIFE RIVER
HAZEN, ND

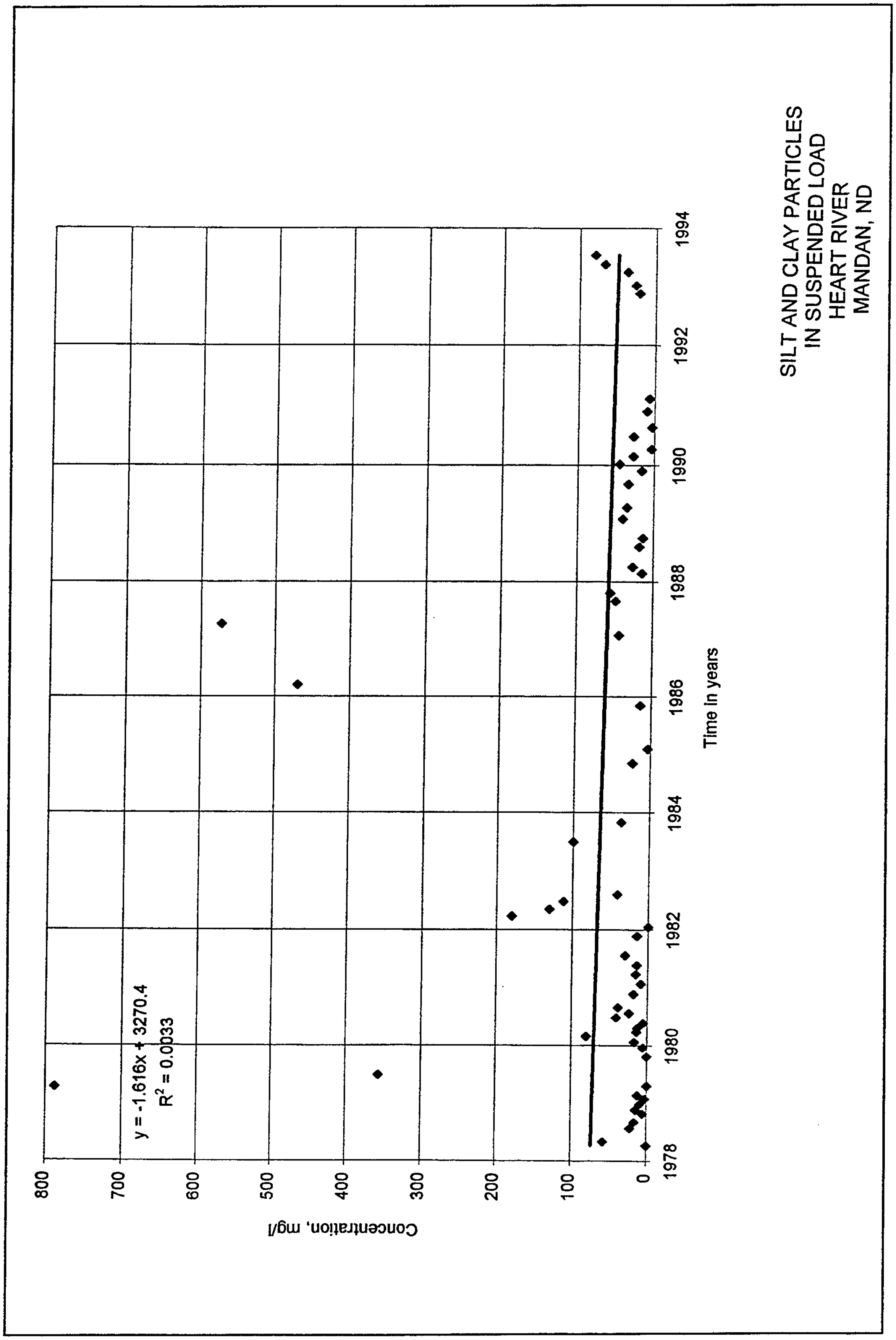


TURBIDITY VERSUS CONCENTRATION
OF FINE SEDIMENTS
HEART RIVER
MANDAN, ND

Plate 60

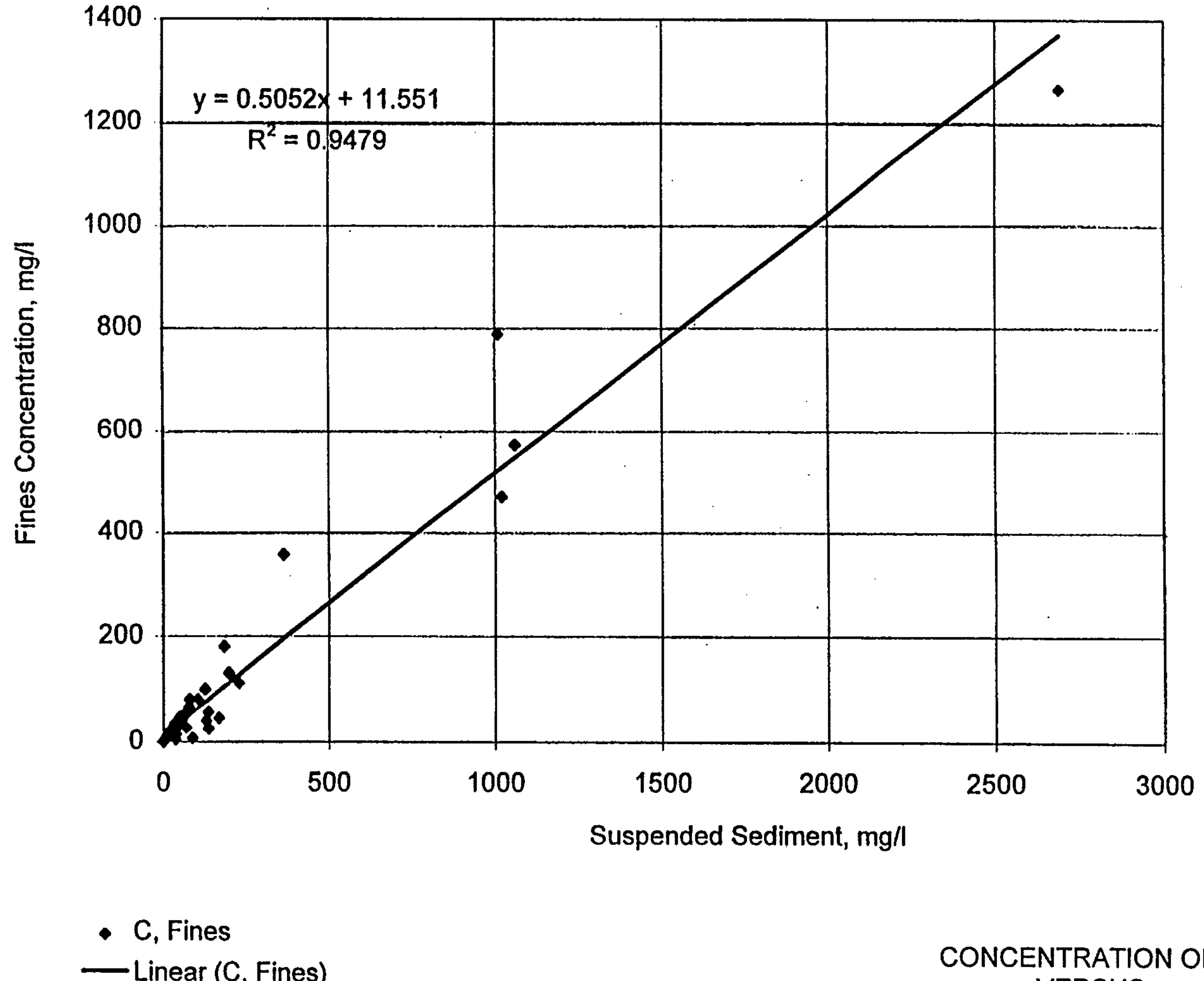


SUSPENDED SEDIMENT CONCENTRATION
VERSUS
WATER DISCHARGE
HEART RIVER
MANDAN, ND



SILT AND CLAY PARTICLES
IN SUSPENDED LOAD
HEART RIVER
MANDAN, ND
y = -1.616x + 3270.4
R^2 = 0.0033
Concentration, mg/l
Time in years
0
100
200
300
400
500
600
700
800
1978
1980
1982
1984
1986
1988
1990
1992
1994

Plate 62



CONCENTRATION OF FINES
VERSUS
SUSPENDED SEDIMENT CONCENTRATION
HEART RIVER
MANDAN, ND

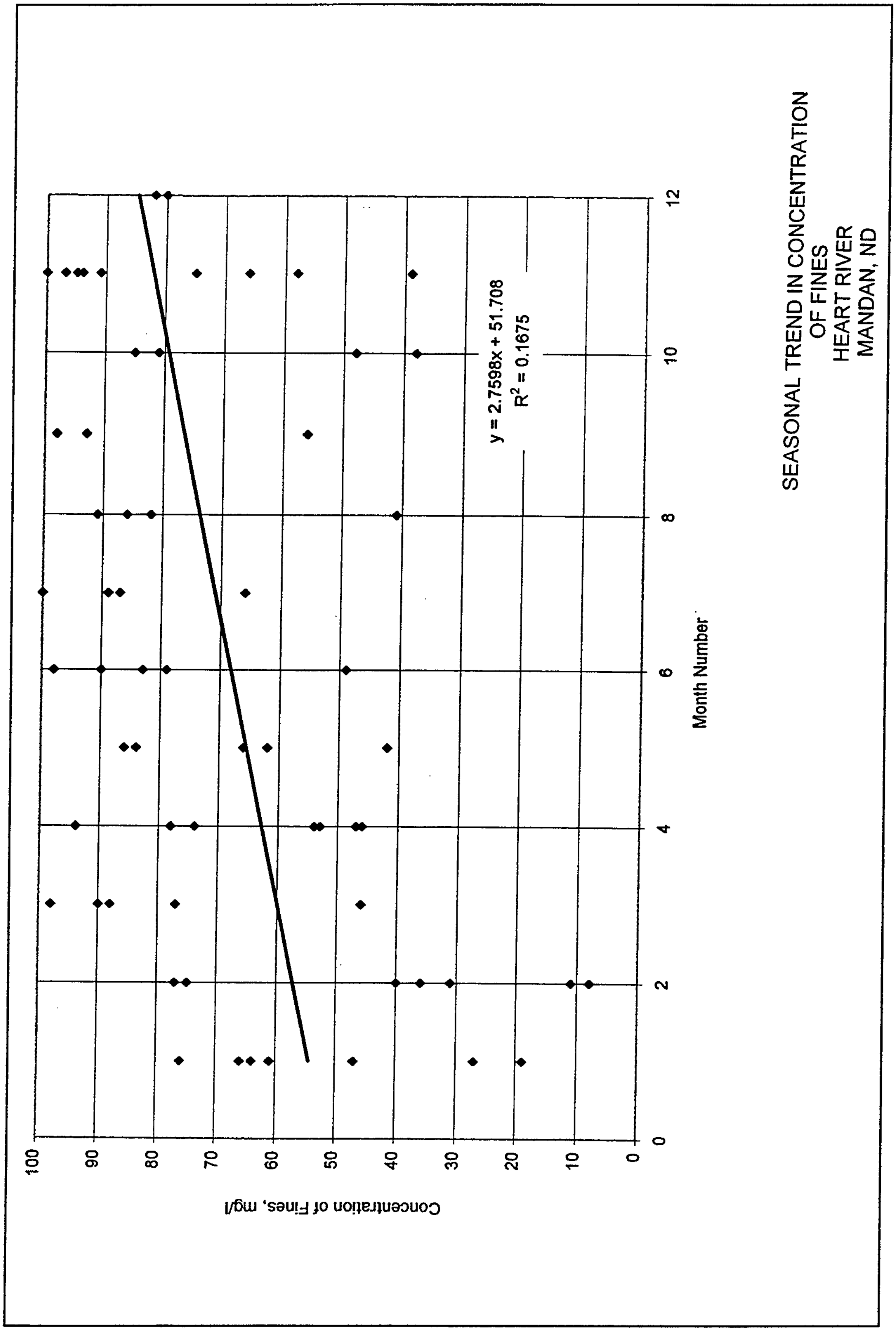
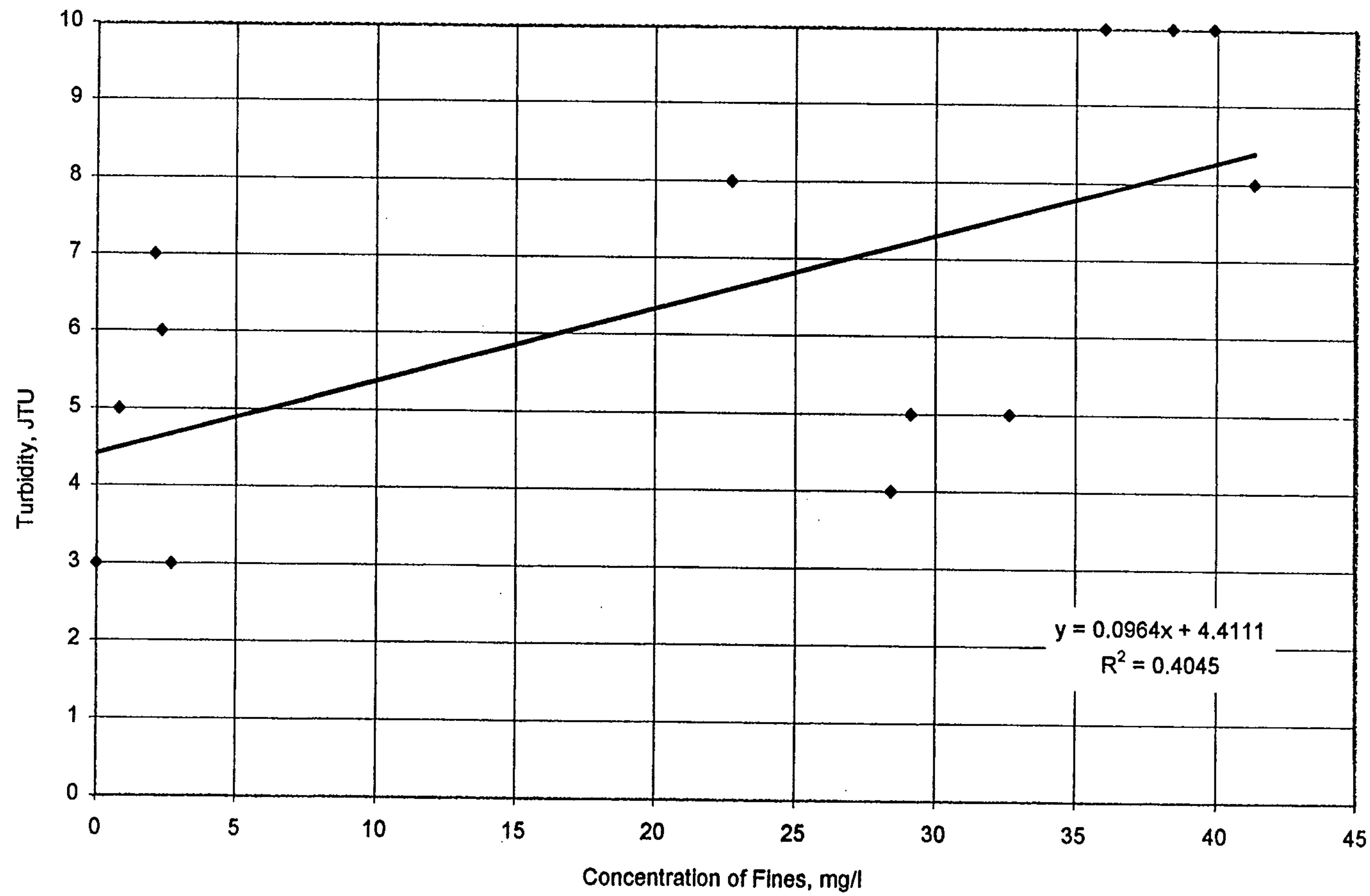
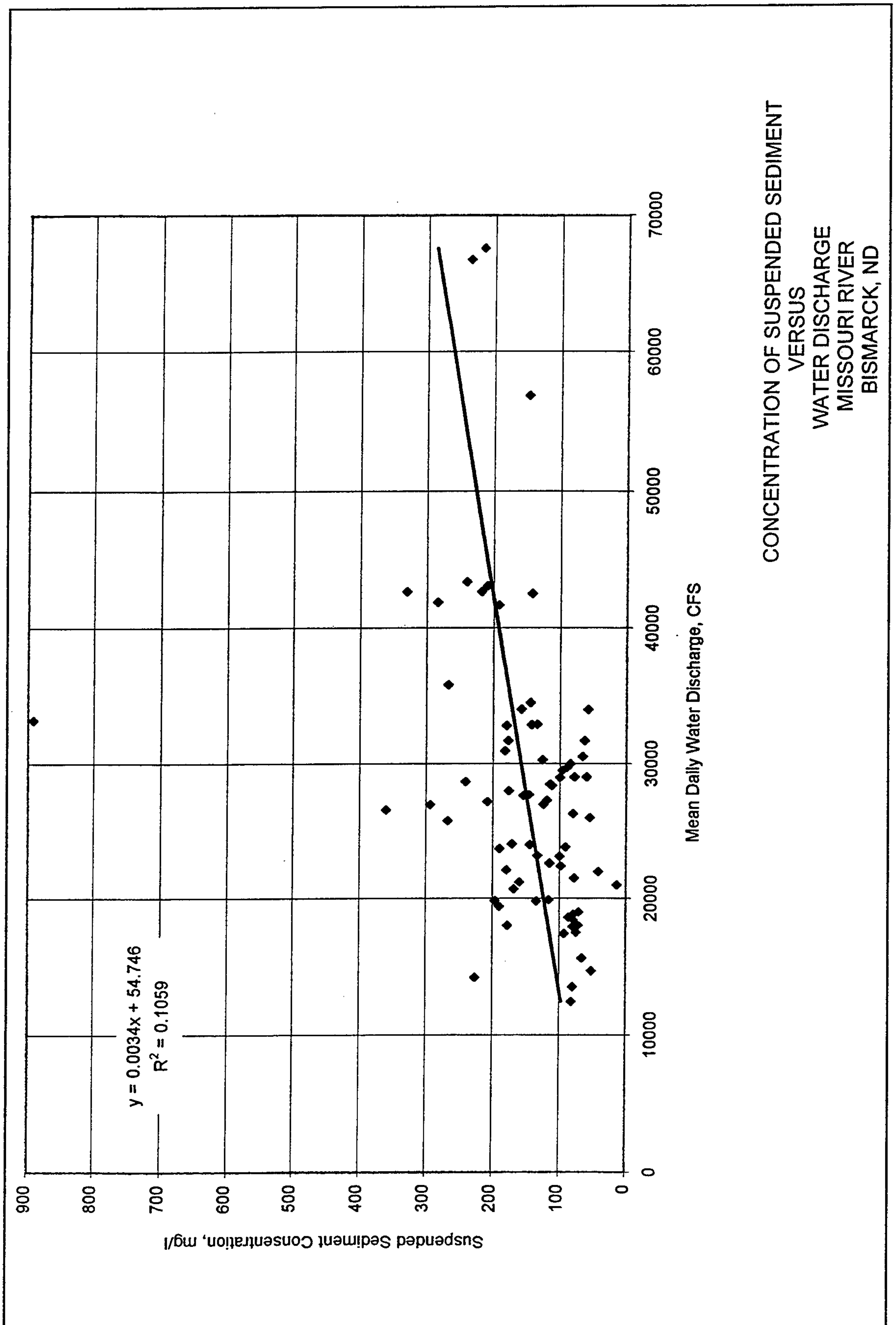
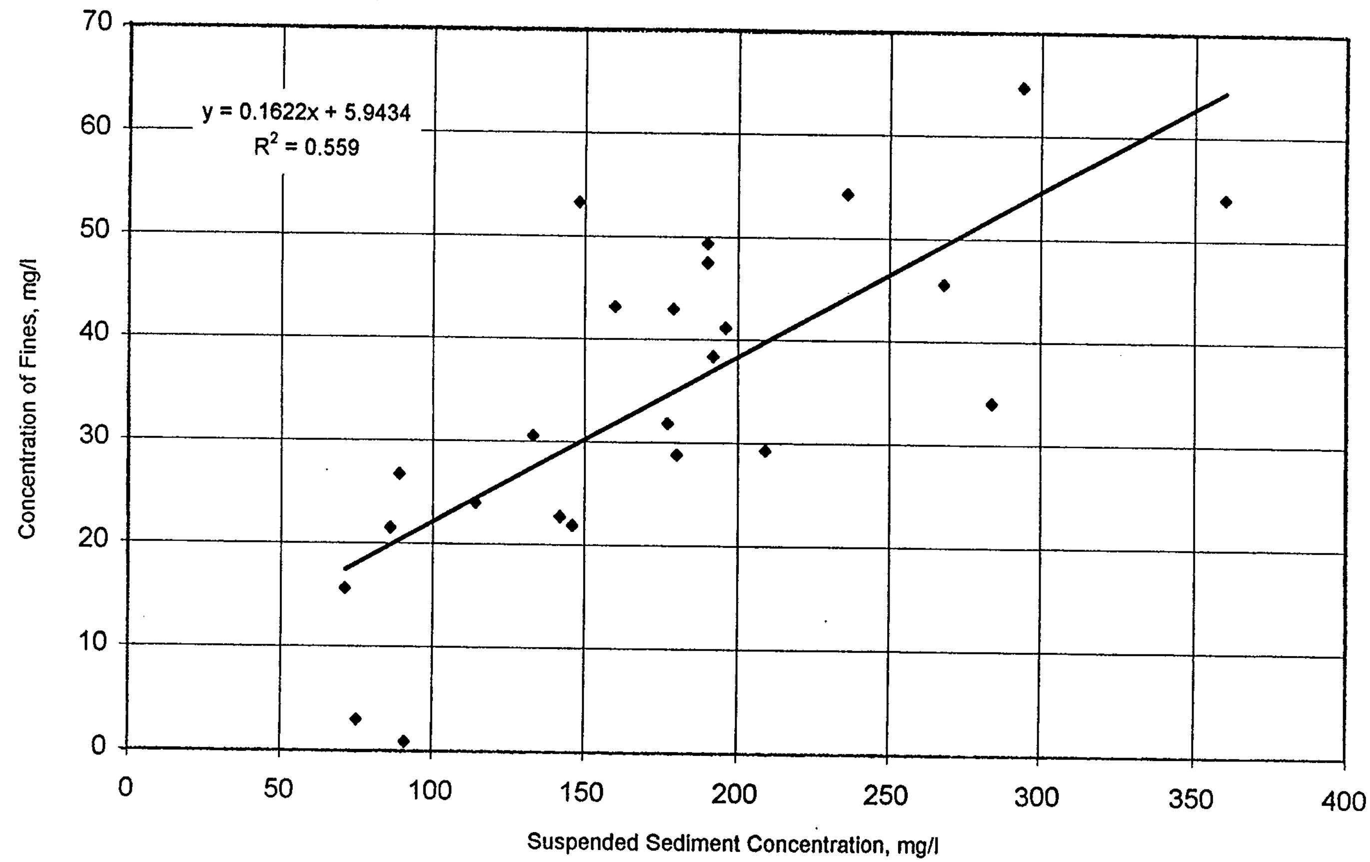

SEASONAL TREND IN CONCENTRATION
OF FINES
HEART RIVER
MANDAN, ND
y = 2.7598x + 51.708
R^2 = 0.1675
Concentration of Fines, mg/l
Month Number
0
10
20
30
40
50
60
70
80
90
100
0
2
4
6
8
10
12

Plate 64



TURBIDITY VERSUS
CONCENTRATION OF FINES
MISSOURI RIVER
BISMARCK, ND


CONCENTRATION OF SUSPENDED SEDIMENT
VERSUS
WATER DISCHARGE
MISSOURI RIVER
BISMARCK, ND
y = 0.0034x + 54.746
R2 = 0.1059
Suspended Sediment Consentration, mg/l
Mean Daily Water Discharge, CFS
0
100
200
300
400
500
600
700
800
900
0
10000
20000
30000
40000
50000
60000
70000



CONCENTRATION OF FINES
VERSUS
SUSPENDED SEDIMENT CONCENTRATION
MISSOURI RIVER
BISMARCK, ND

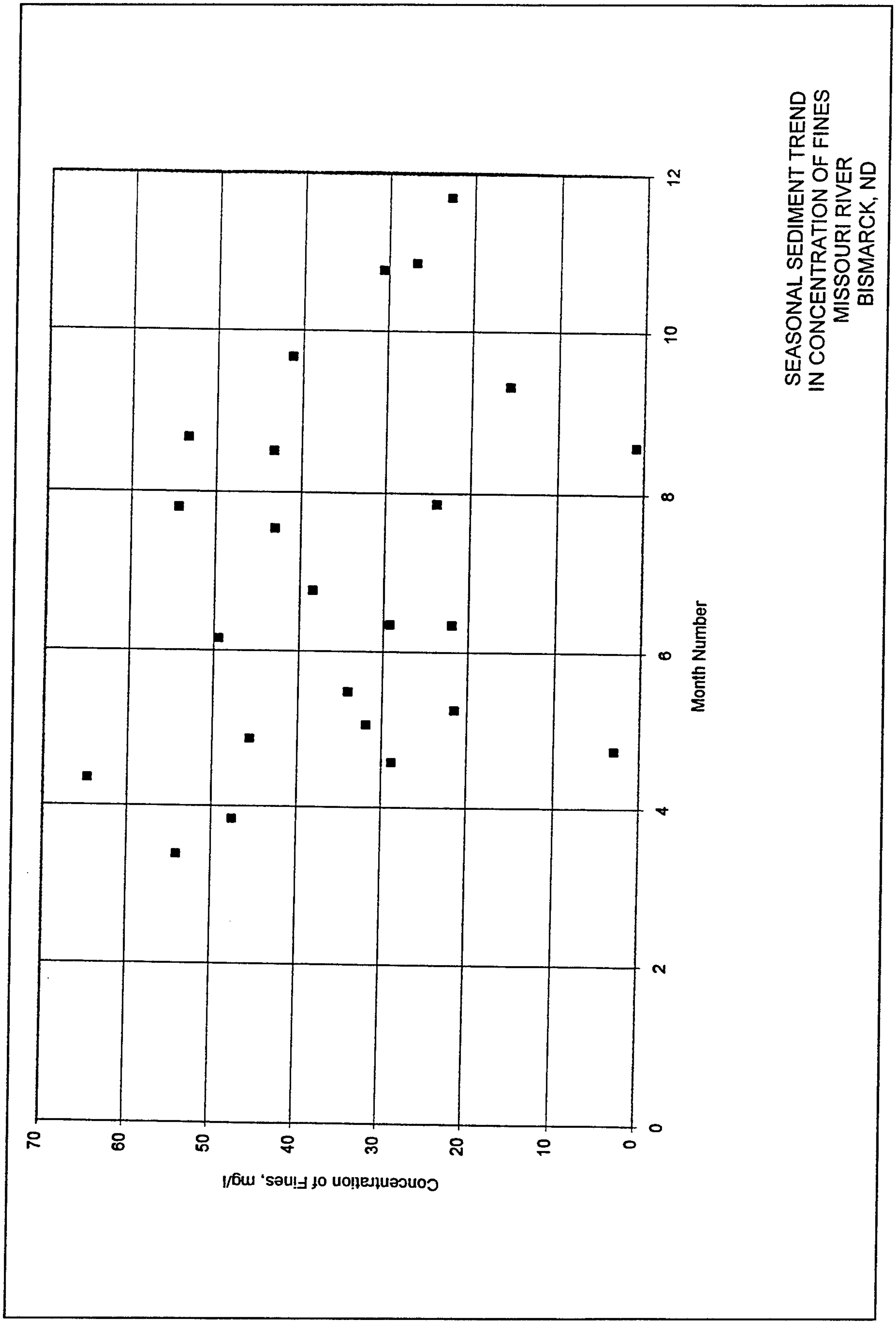
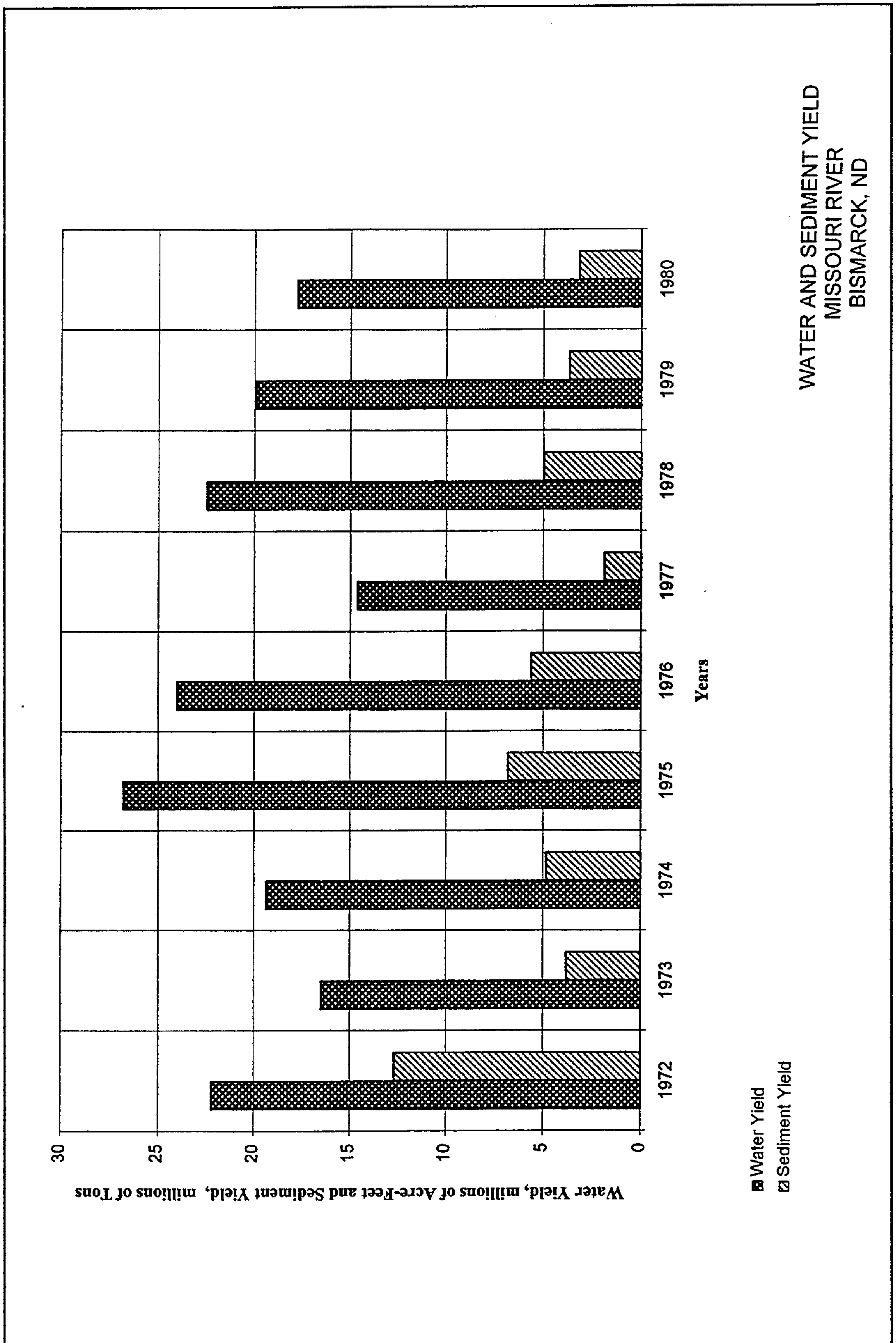

SEASONAL SEDIMENT TREND
IN CONCENTRATION OF FINES
MISSOURI RIVER
BISMARCK, ND
Concentration of Fines, mg/l
Month Number
0
10
20
30
40
50
60
70
0
2
4
6
8
10
12

Plate 68


WATER AND SEDIMENT YIELD
MISSOURI RIVER
BISMARCK, ND
Water Yield, millions of Acre-Feet and Sediment Yield, millions of Tons
30
25
20
15
10
5
0
1972
1973
1974
1975
1976
1977
1978
1979
1980
Years
Water Yield
Sediment Yield

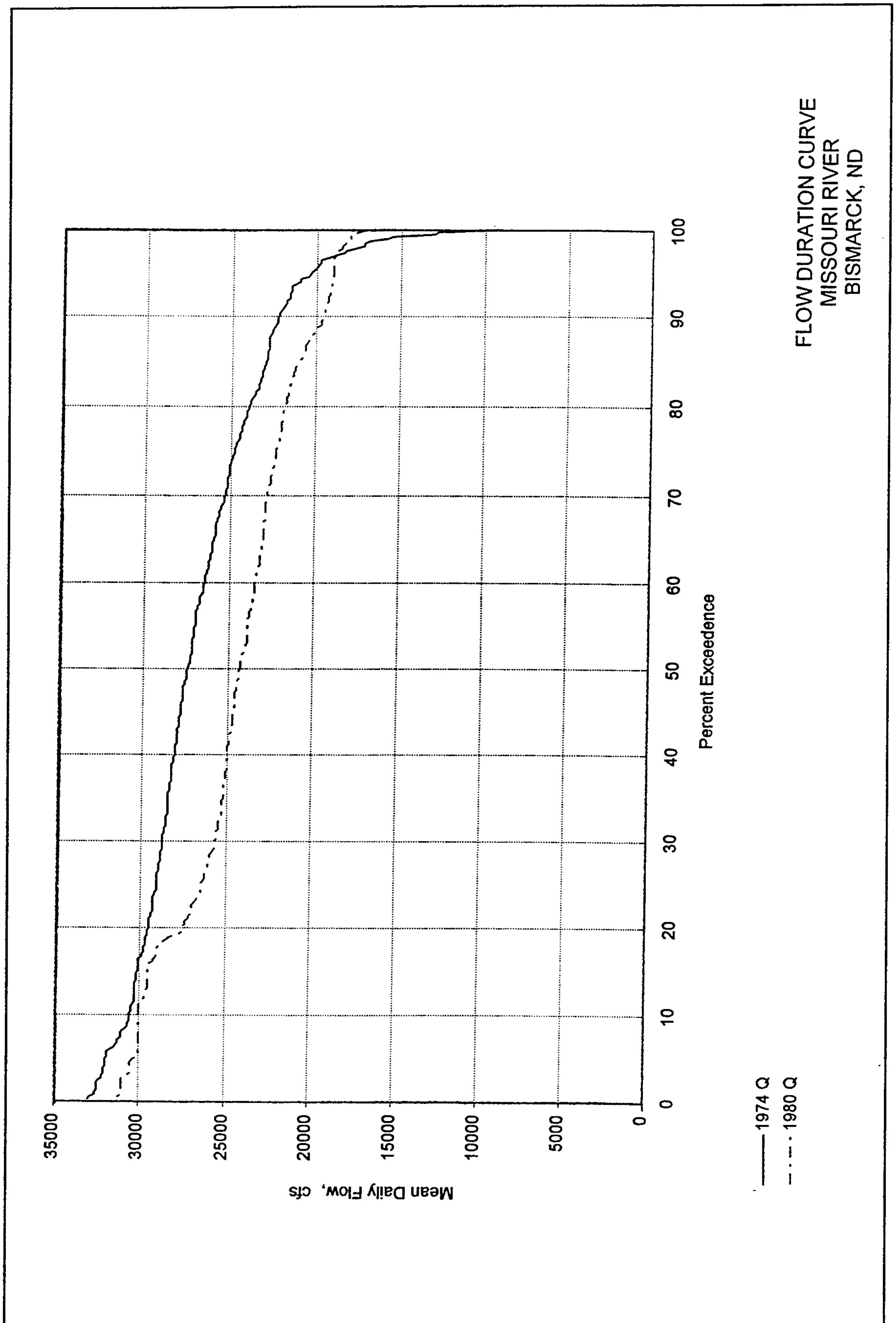
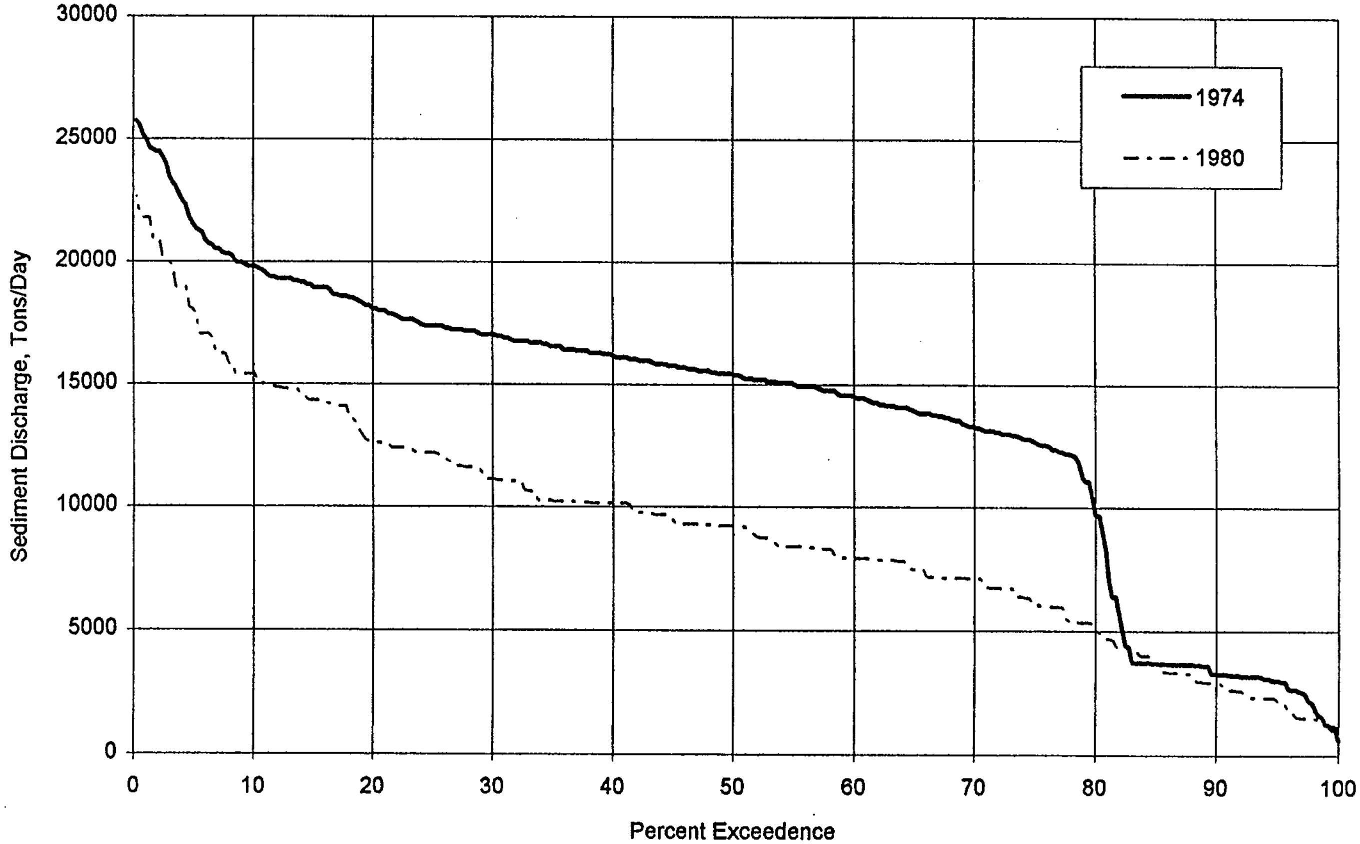
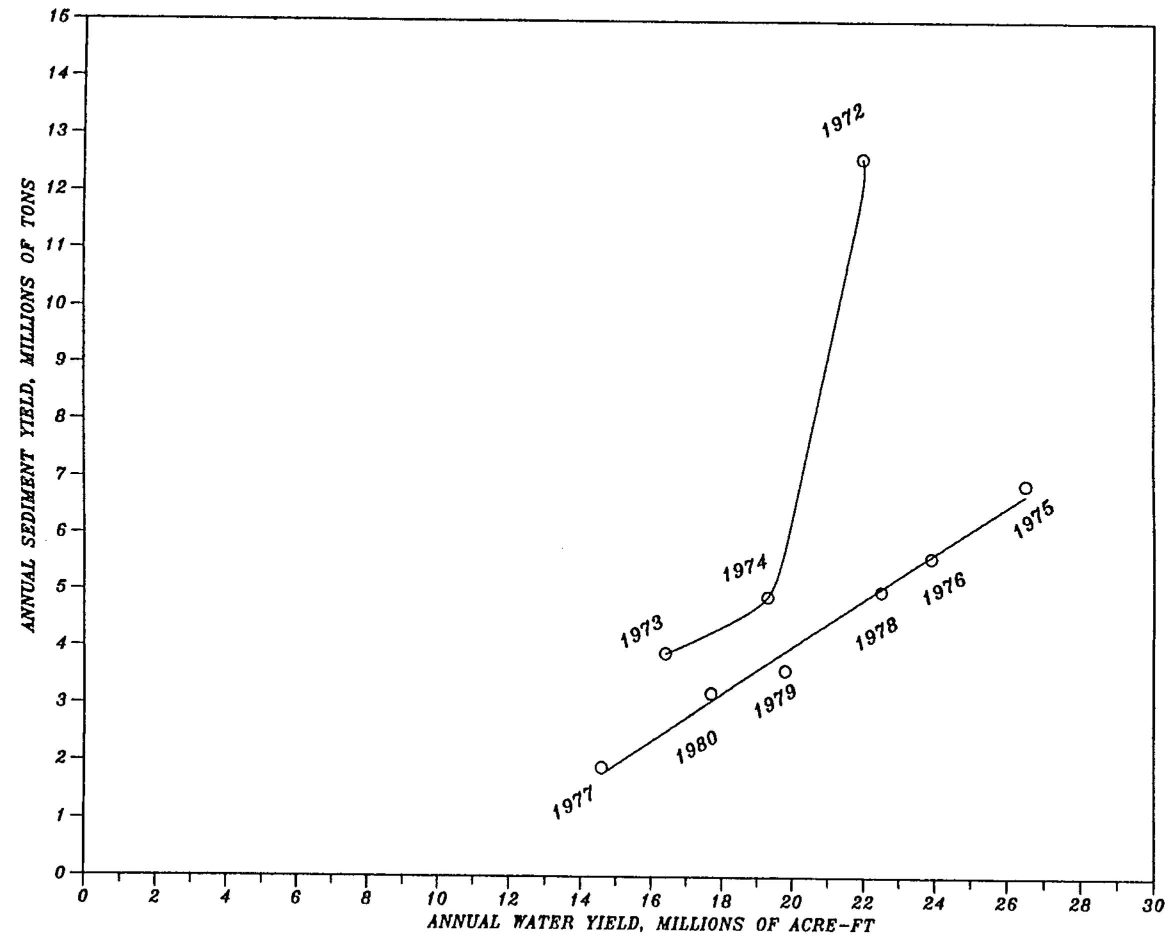

FLOW DURATION CURVE
MISSOURI RIVER
BISMARCK, ND
Mean Daily Flow, cfs
Percent Exceedence
35000
30000
25000
20000
15000
10000
5000
0
0
10
20
30
40
50
60
70
80
90
100
1974 Q
1980 Q

Plate 70

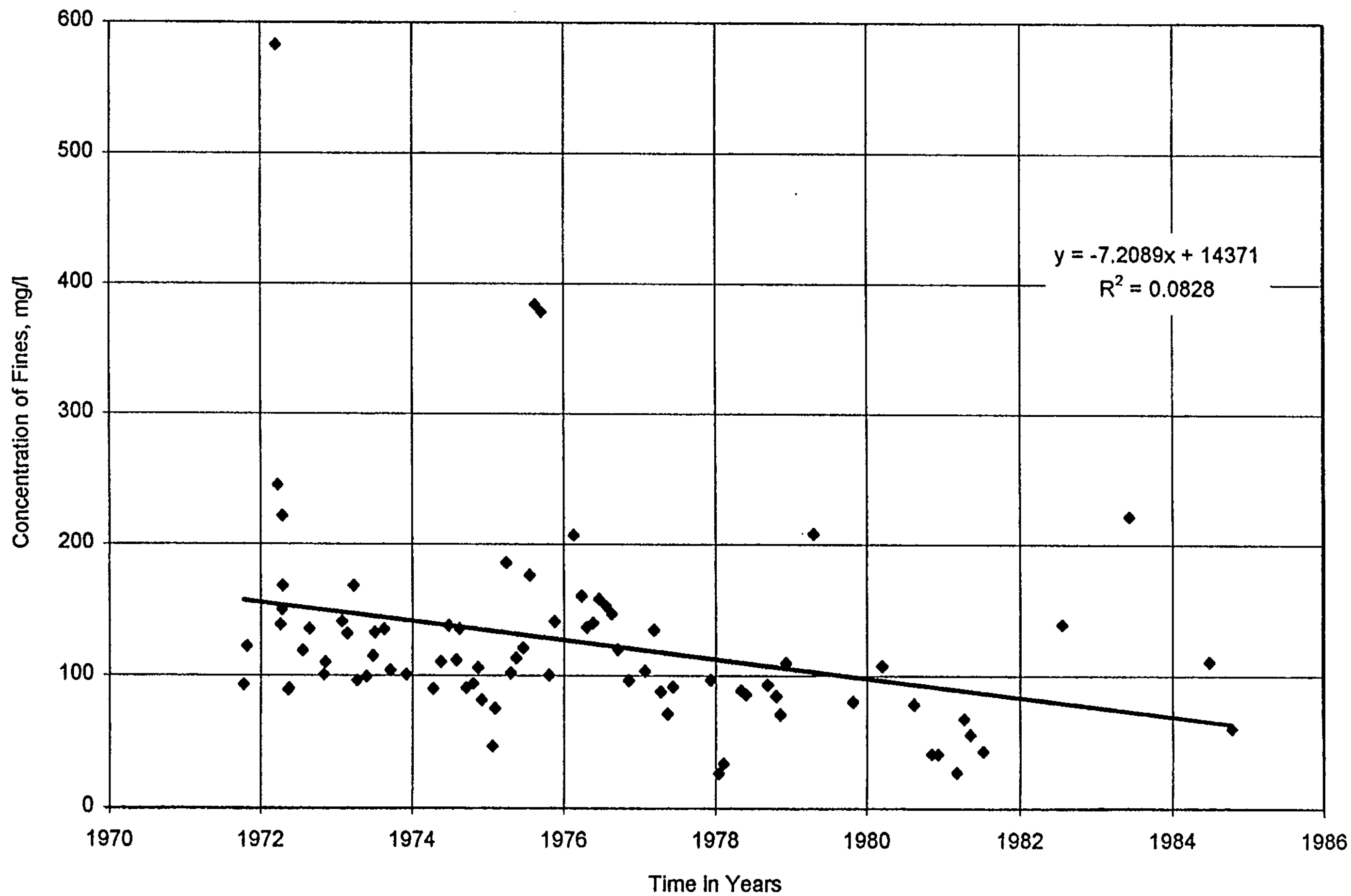


SEDIMENT DISCHARGE DURATION CURVE
MISSOURI RIVER
BISMARCK, ND

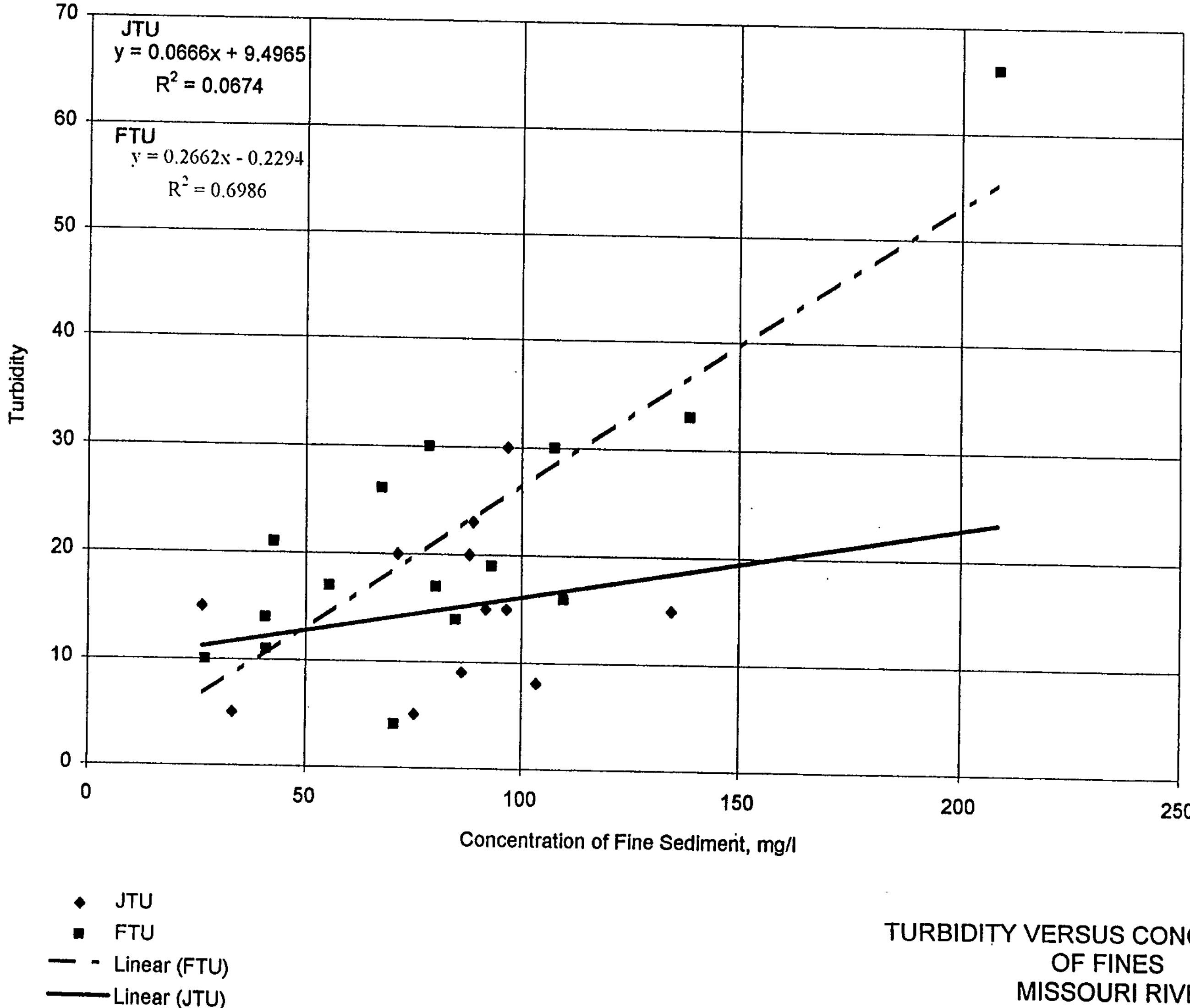


ANNUAL SEDIMENT YIELD
VERSUS
ANNUAL WATER YIELD
AT BISMARCK, ND

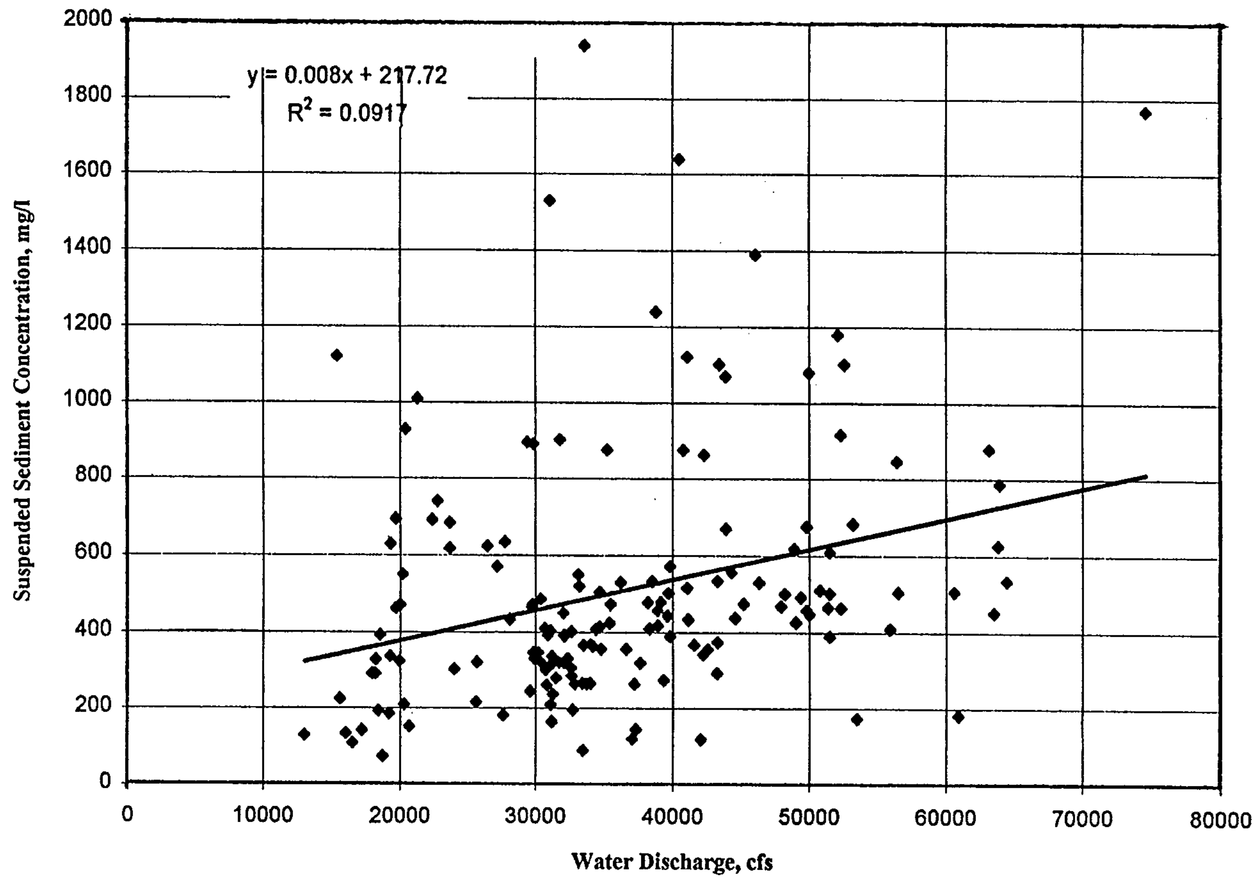
Plate 72



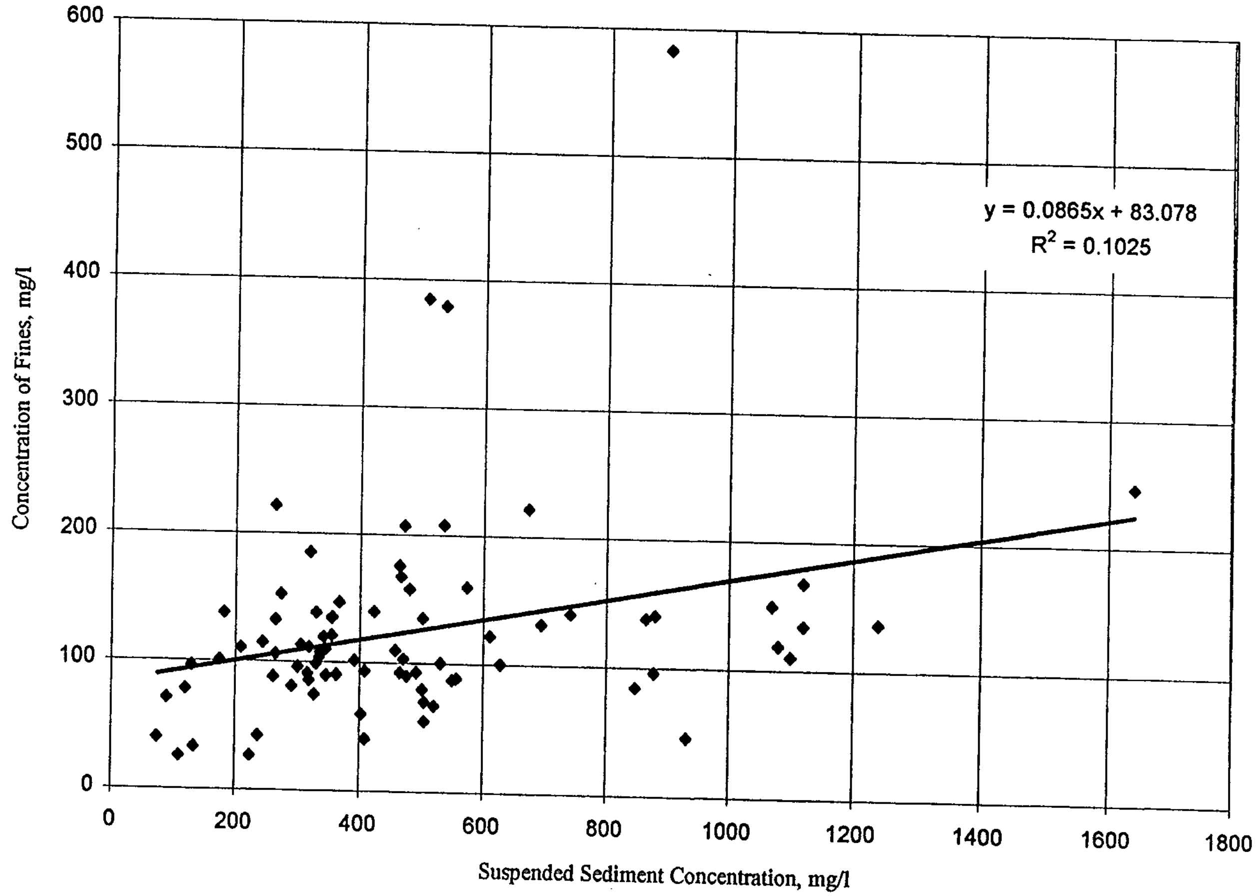
CONCENTRATION OF FINES MEASUREMENT
MISSOURI RIVER
SIOUX CITY, IA



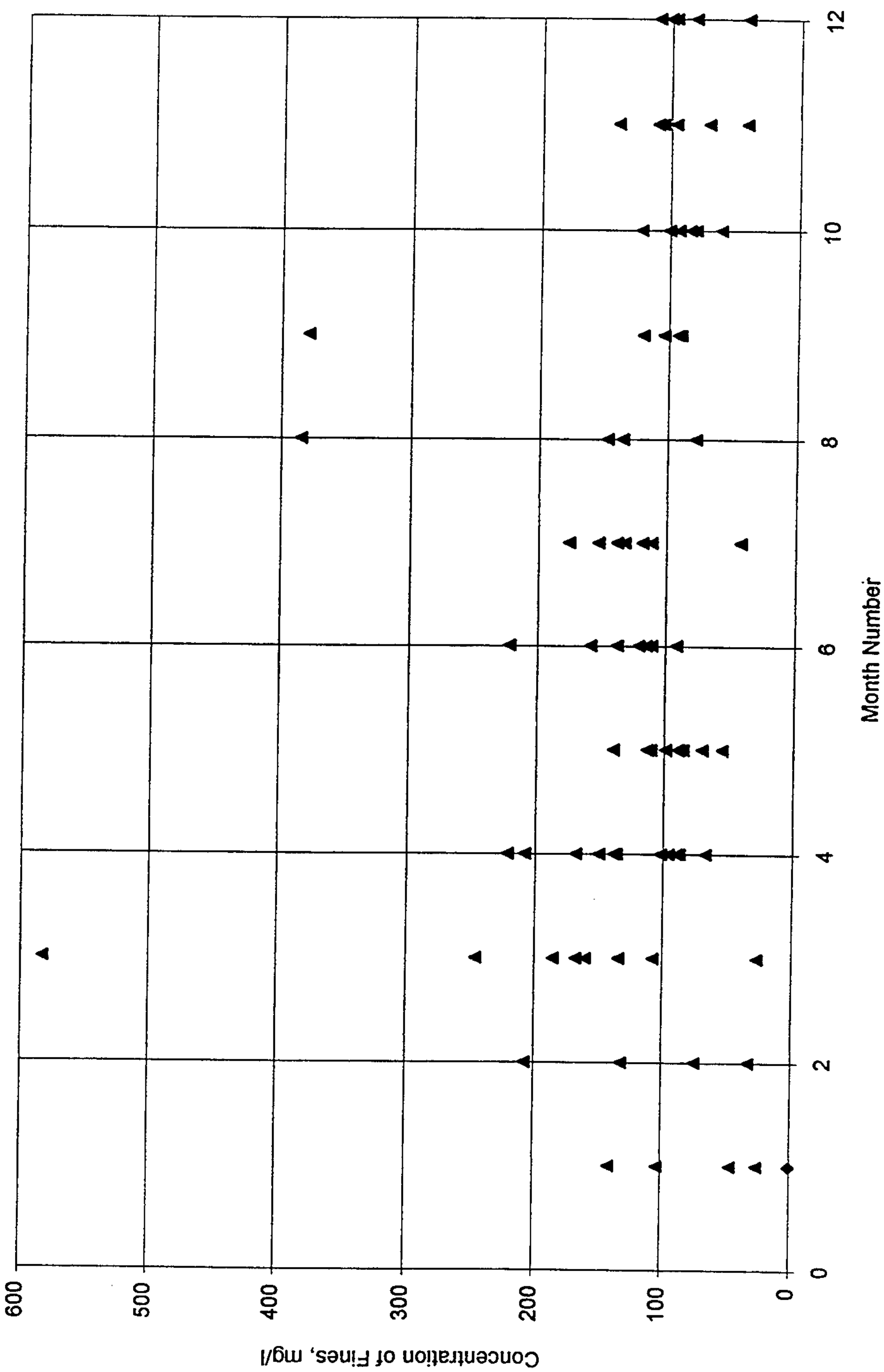
TURBIDITY VERSUS CONCENTRATION
OF FINES
MISSOURI RIVER
SIOUX CITY, IA



SUSPENDED SEDIMENT CONCENTRATION
VERSUS
WATER DISCHARGE
MISSOURI RIVER
SIOUX CITY, IA



CONCENTRATION OF FINES
VERSUS
SUSPENDED SEDIMENT CONCENTRATION
MISSOURI RIVER
SIOUX CITY, IA



SEASONAL TREND IN CONCENTRATION
OF FINES
MISSOURI RIVER
SIOUX CITY, IA

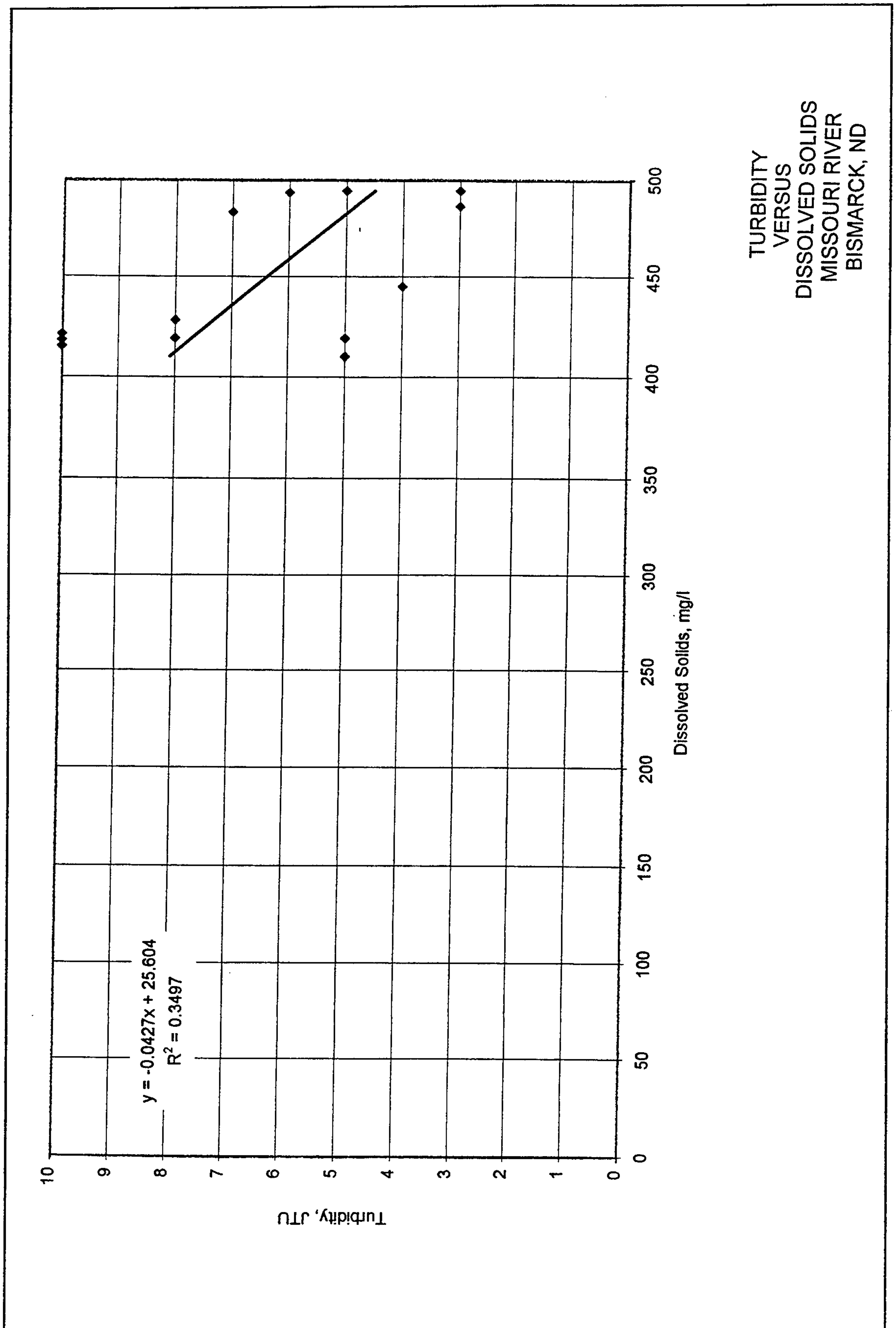
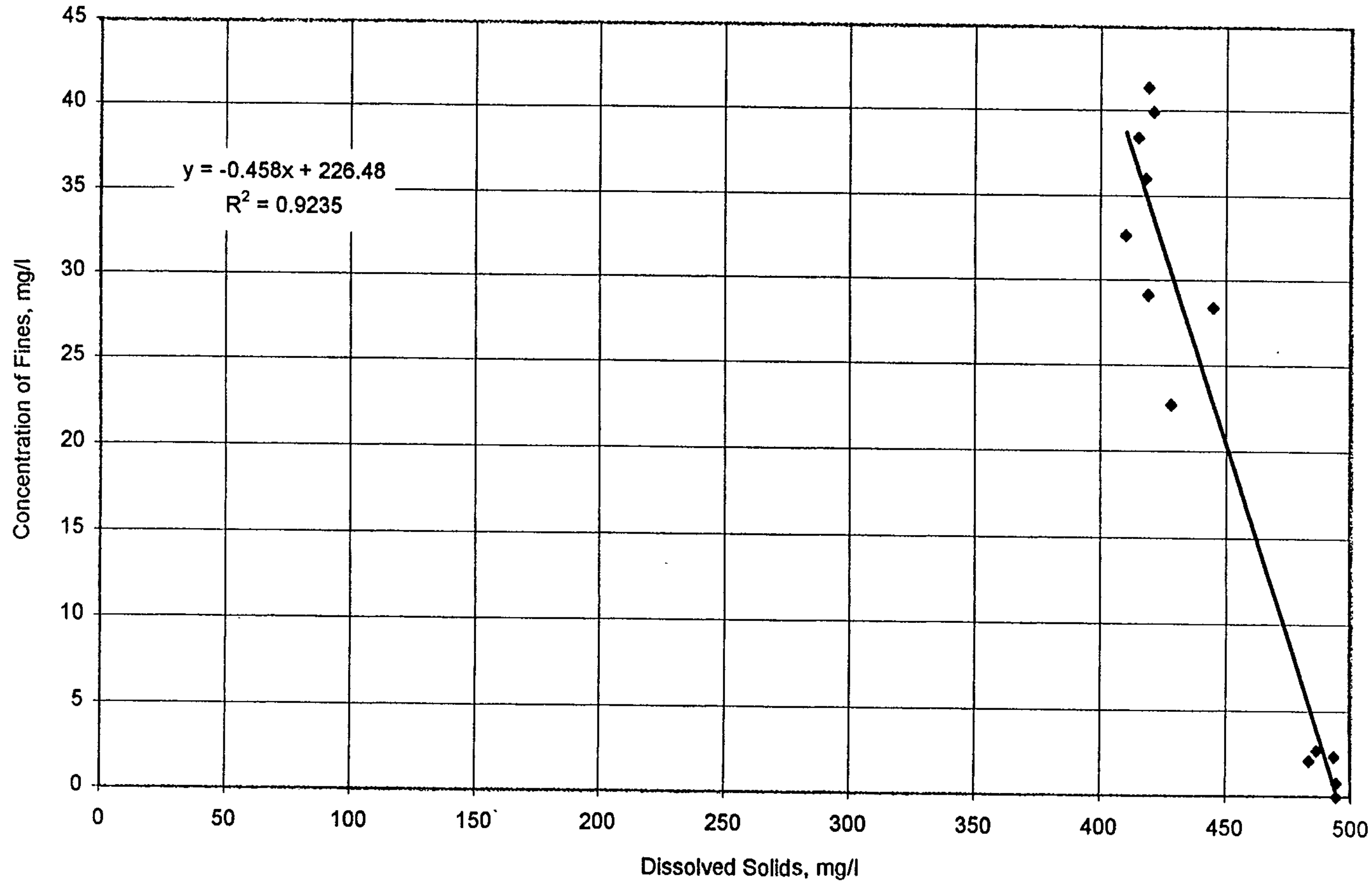
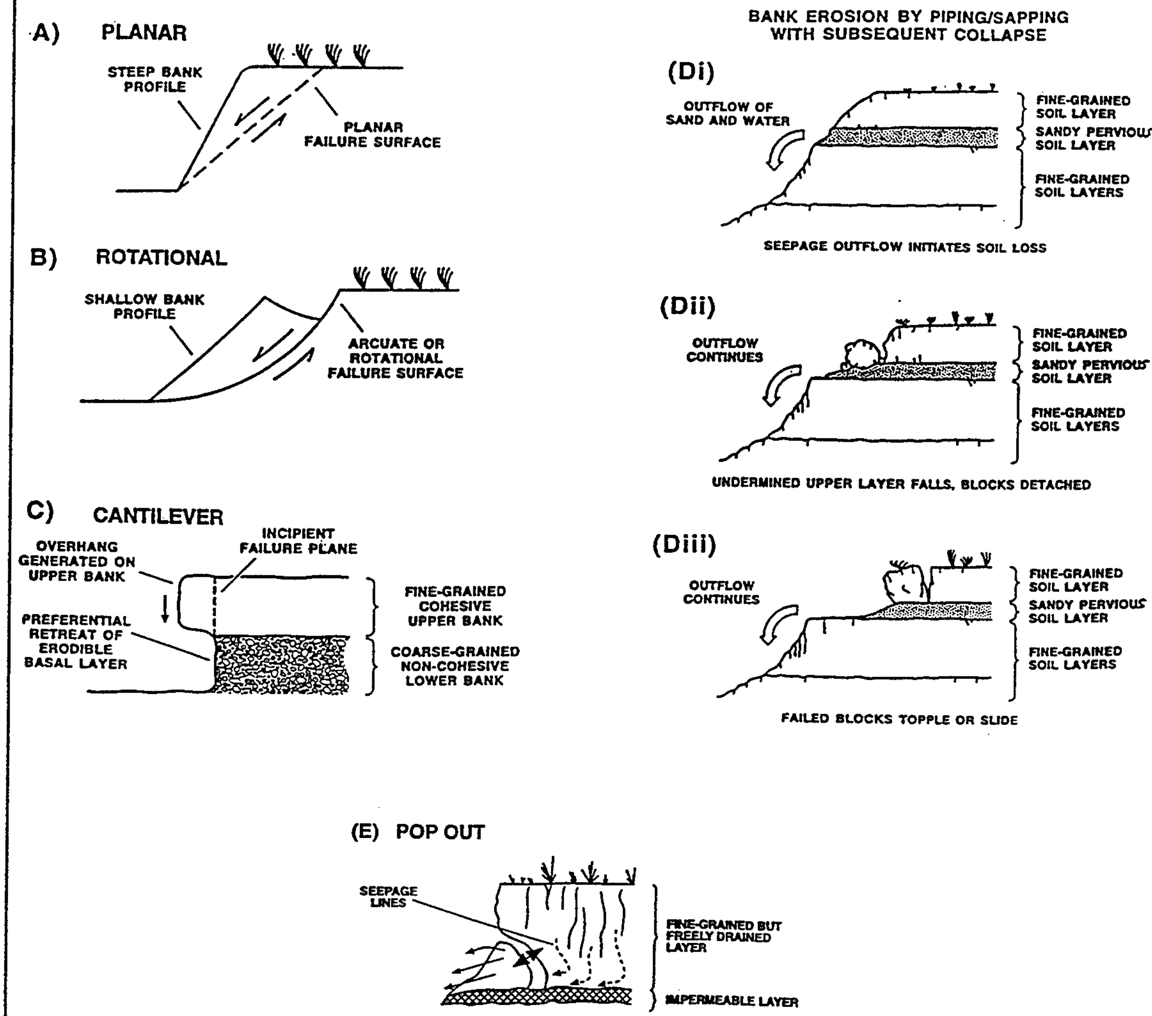

TURBIDITY
VERSUS
DISSOLVED SOLIDS
MISSOURI RIVER
BISMARCK, ND
Turbidity, JTU
Dissolved Solids, mg/l
y = -0.0427x + 25.604
R2 = 0.3497
0
1
2
3
4
5
6
7
8
9
10
0
50
100
150
200
250
300
350
400
450
500

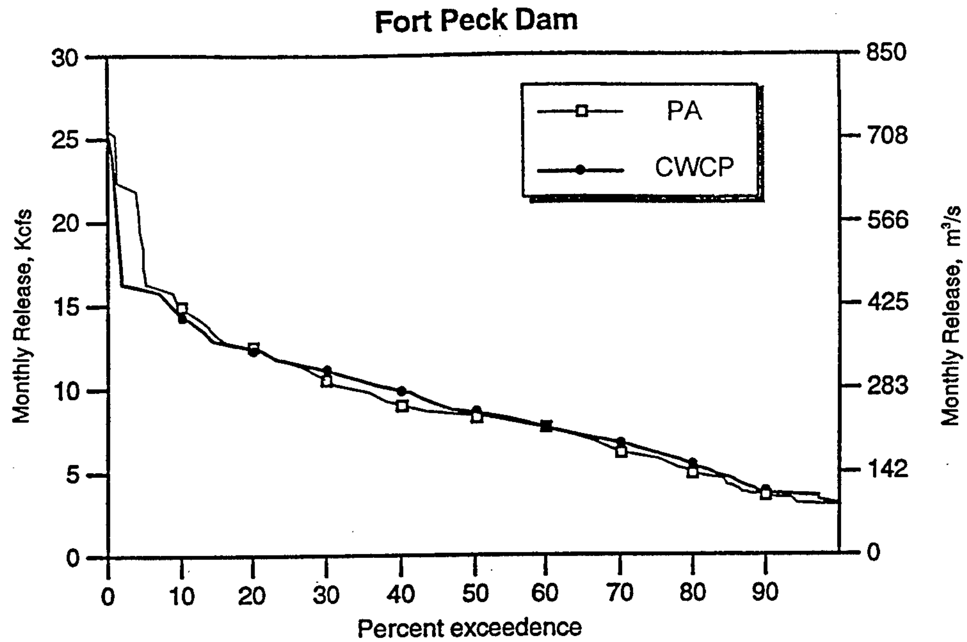
Plate 78

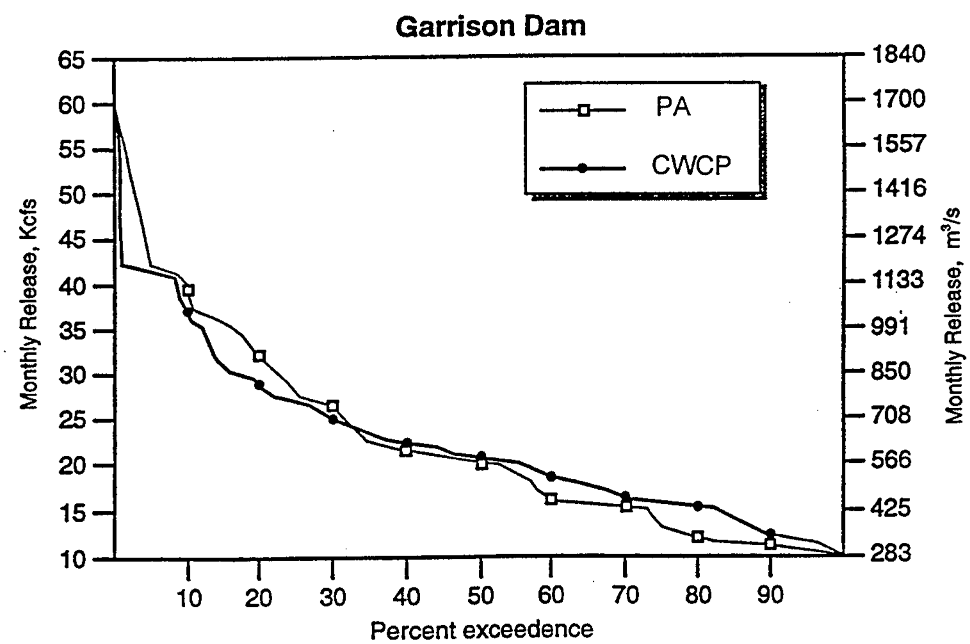


CONCENTRATION OF FINES
VERSUS
DISSOLVED SOLIDS
MISSOURI RIVER
BISMARCK, ND

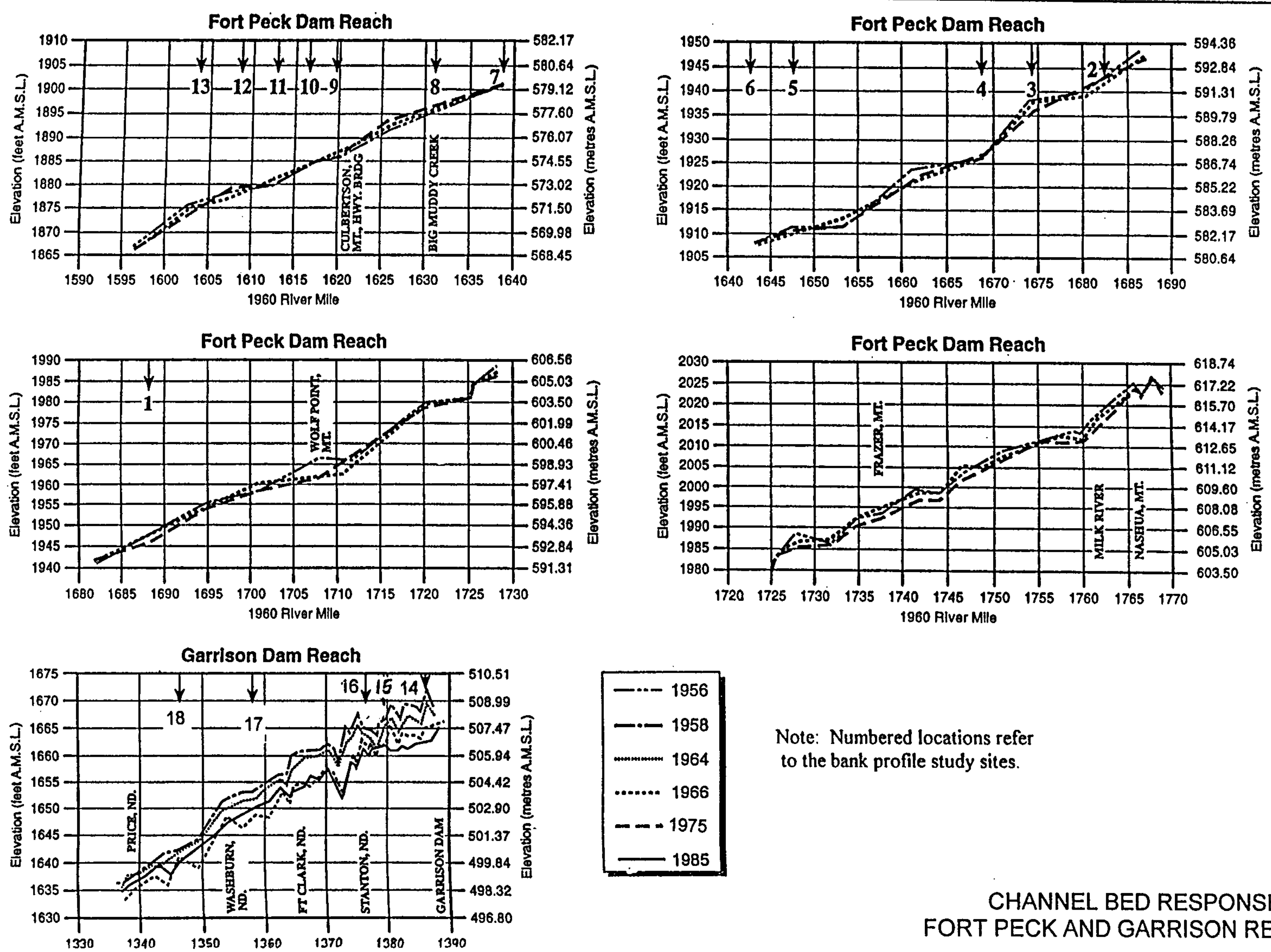


BANK FAILURE MECHANISM



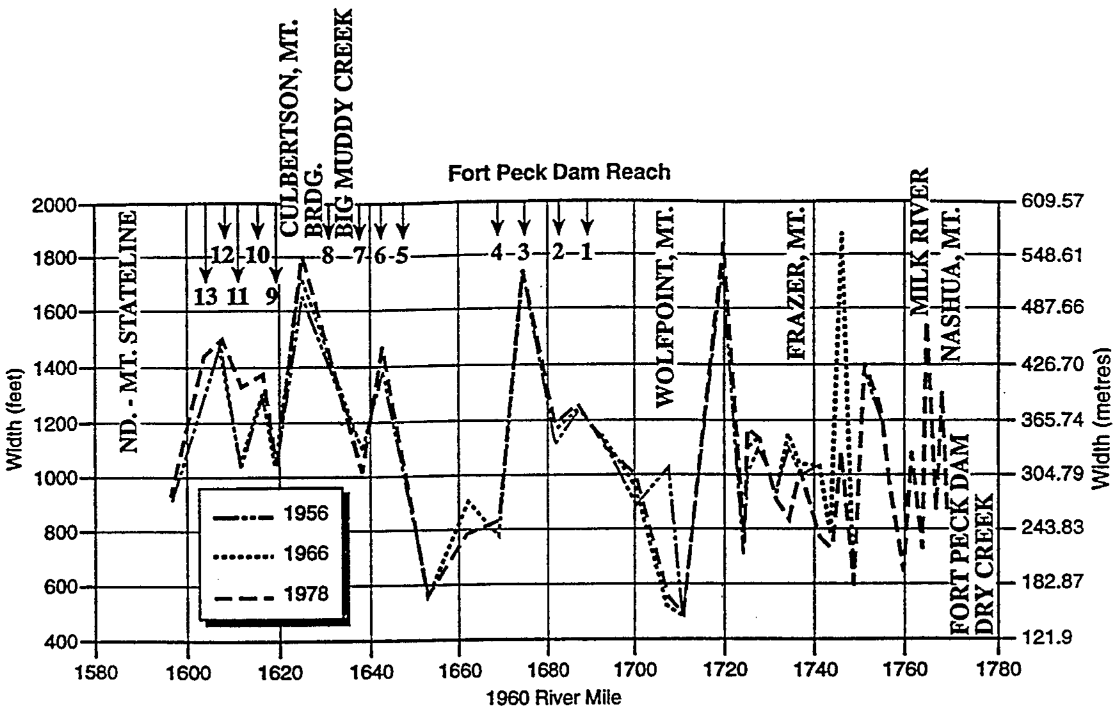


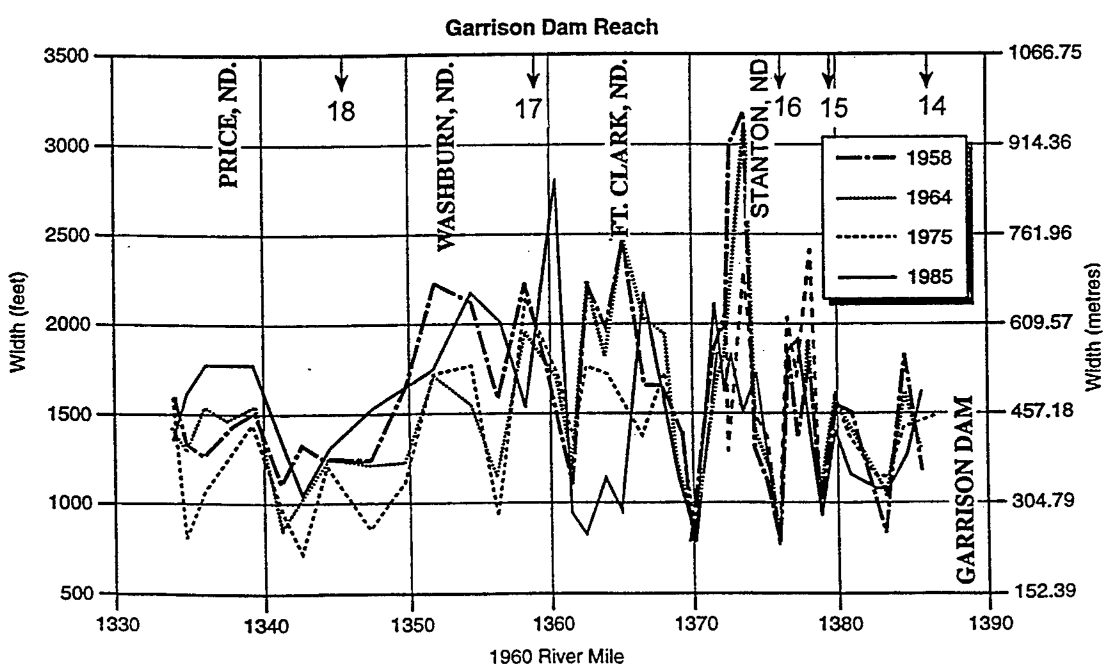
FLOW DURATION CURVES
AT FORT PECK AND GARRISON DAMS



Note: Numbered locations refer to the bank profile study sites.

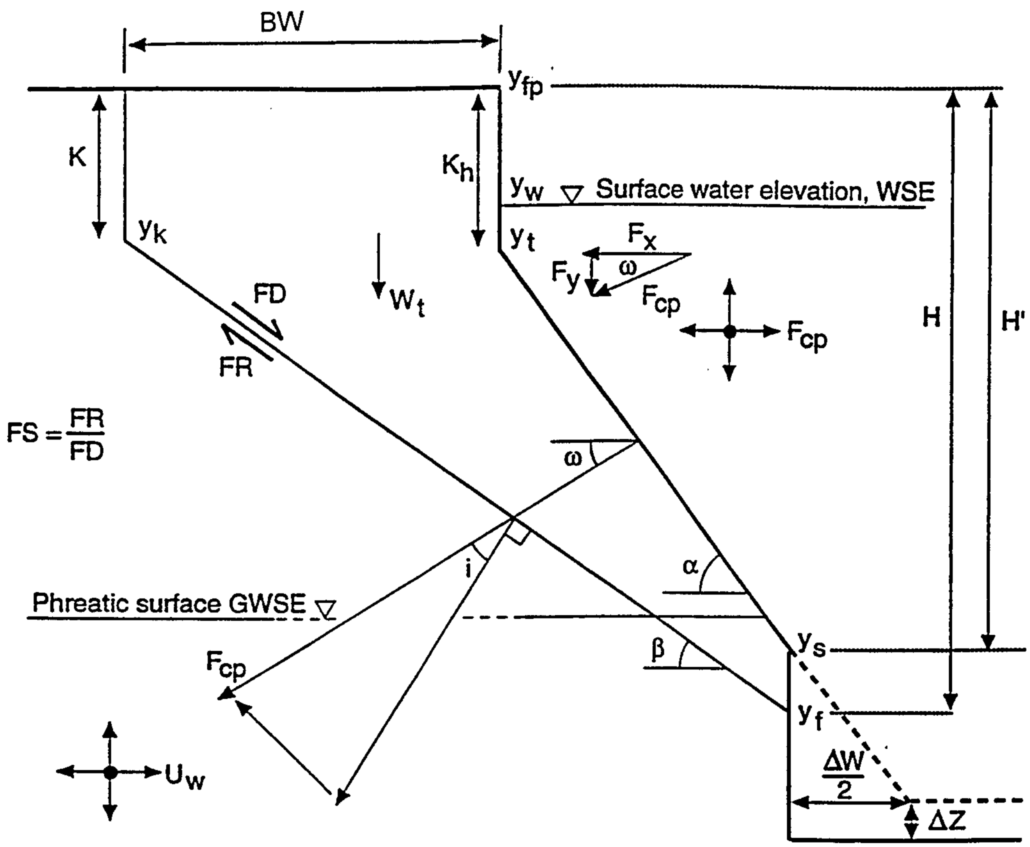
CHANNEL BED RESPONSE
FORT PECK AND GARRISON REACHES



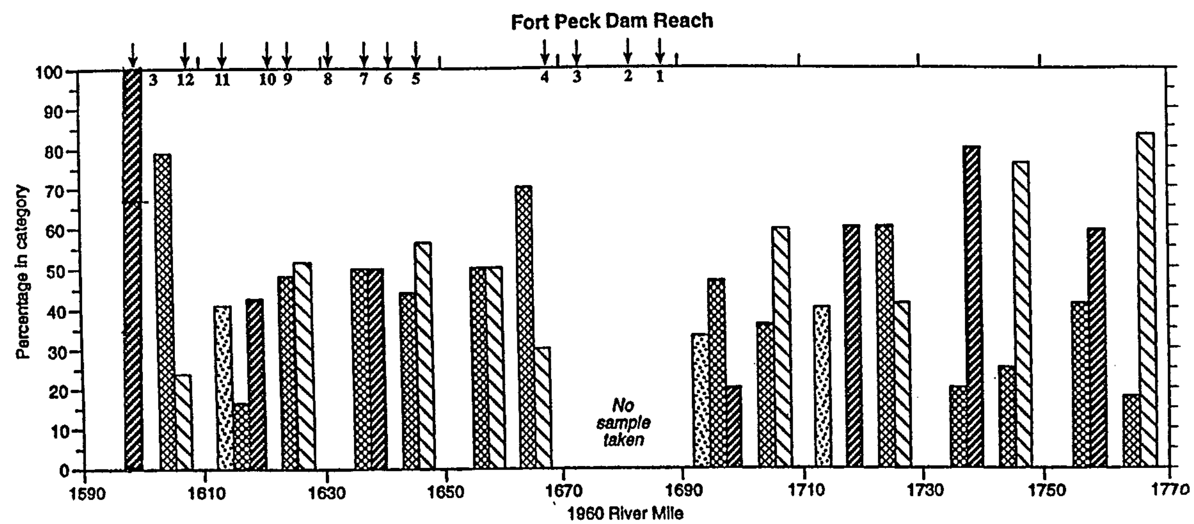


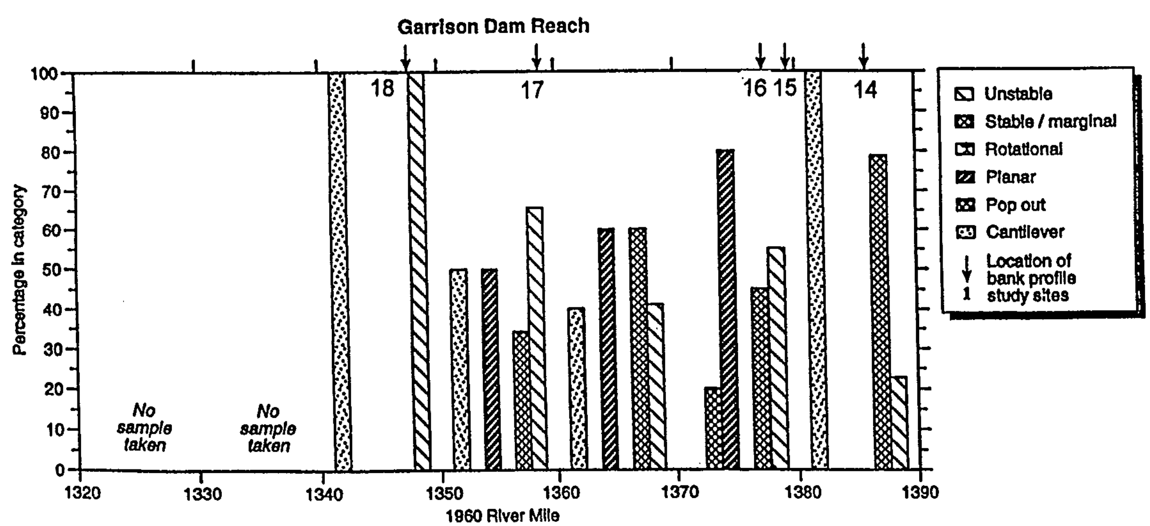
Note: Numbered locations refer to the bank profile study sites.

CHANNEL BED WIDTH
FORT PECK AND GARRISON REACHES

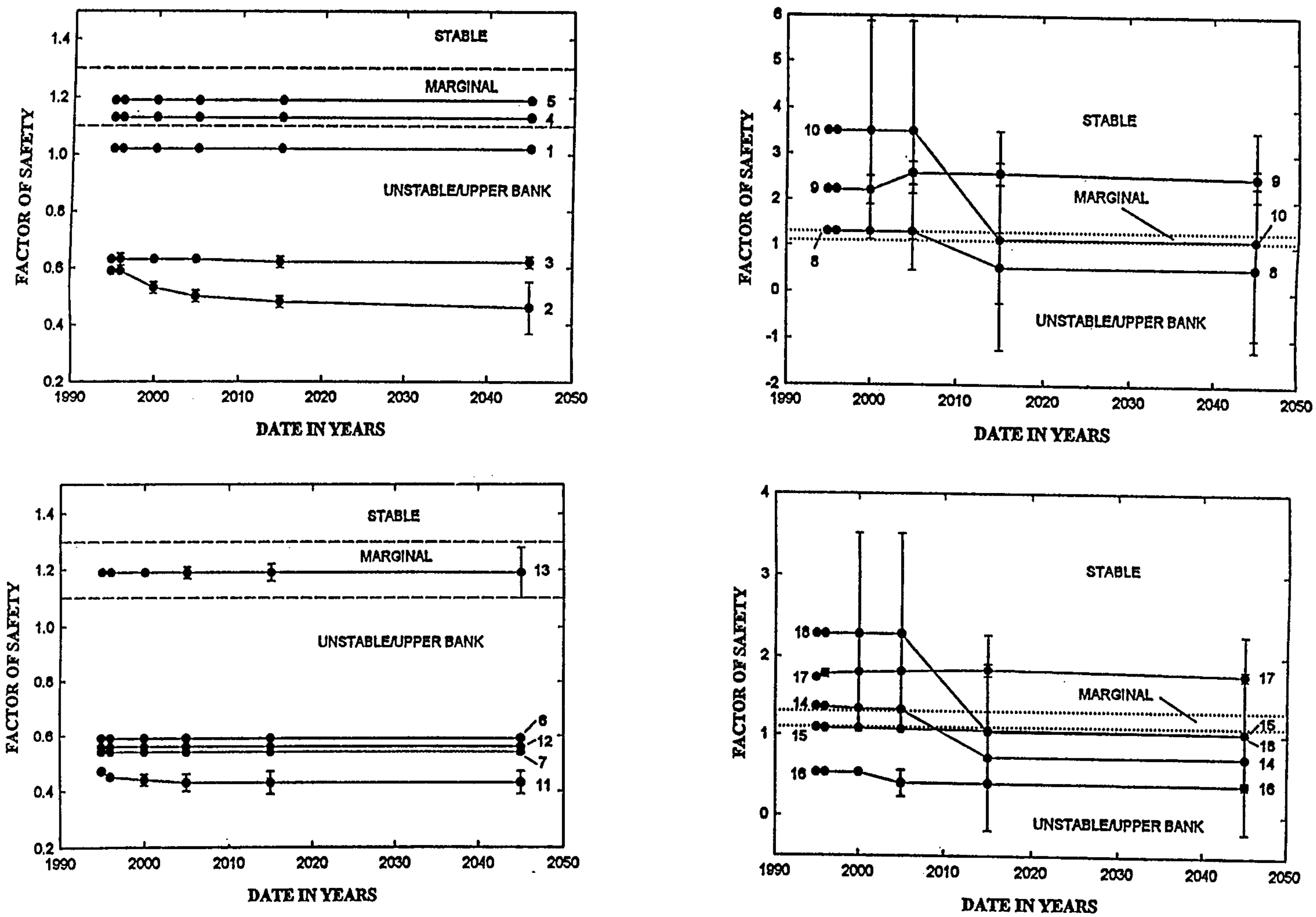


BANK PROFILE DEFORMATION PARAMETERS
DARBY - THORNE ANALYSIS



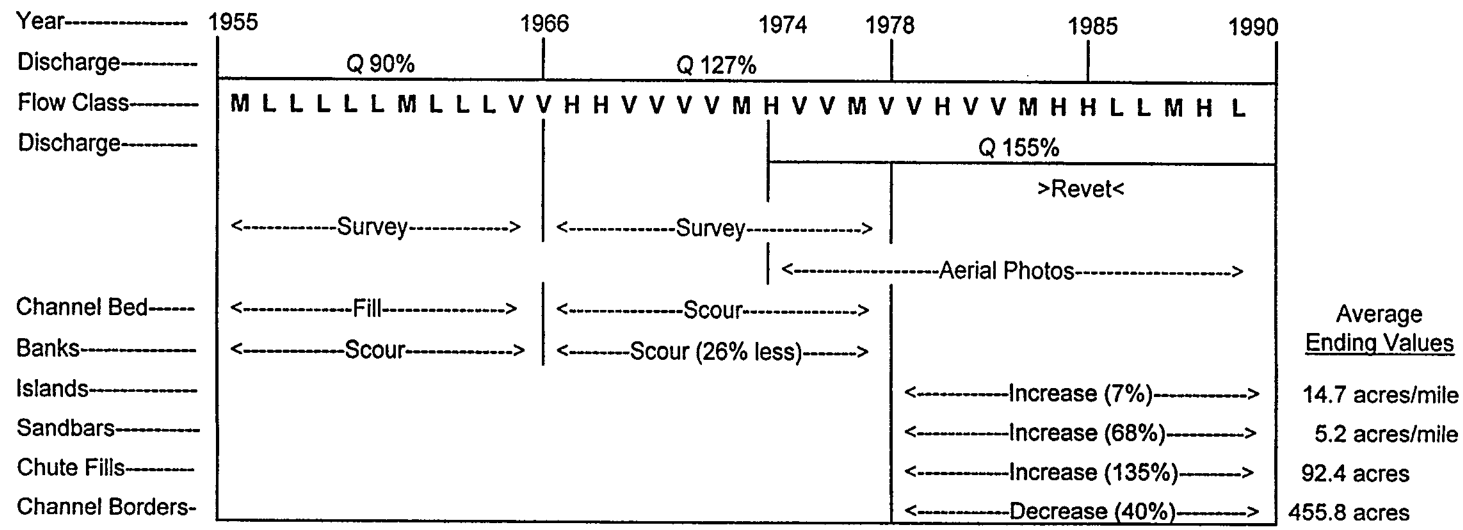


OBSERVED BANK INSTABILITY 1995



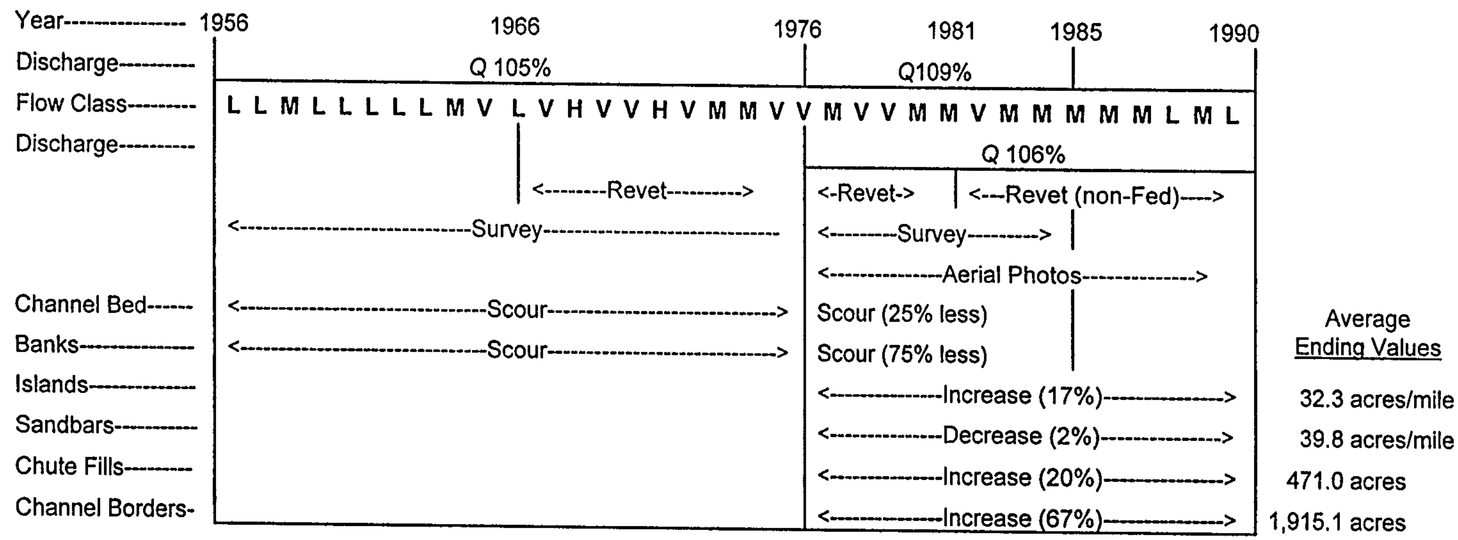
Note: Numbered locations refer to the bank profile study sites.

SITE FACTORS OF SAFETY FOR
OBSERVED / ANALYZED EROSION SITES
IN THE FORT PECK AND GARRISON REACHES



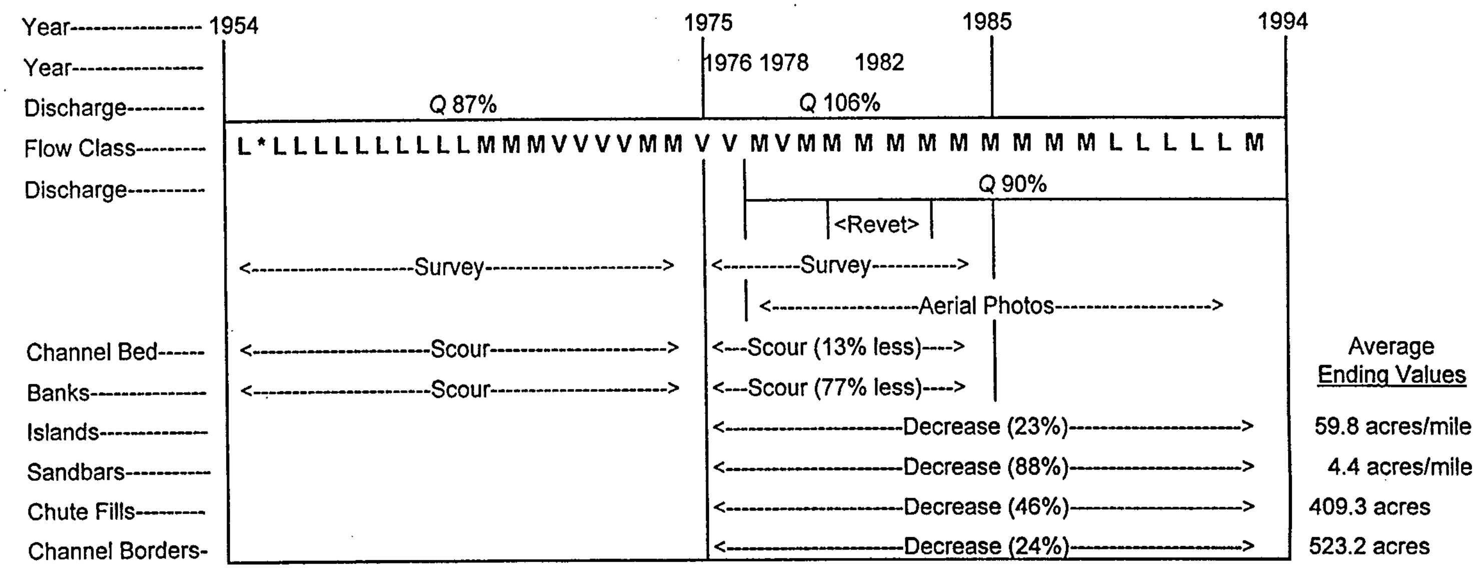
Note: Discharges presented are average yearly volumes relative to the 96-year average of the CWCP and PA.
Flow classifications are based on Chapter 2 values with the following symbols:
L = low flow conditions.
M = medium flow conditions.
H = high flow conditions.
V = very high flow conditions.
Changes in channel bed and banks are based on five segments and not entire reach.
Changes in island and sandbar densities exclude cutoff near RM 1600 that occurred between 1974 and 1990.
Decrease in channel border fills was partially a result of several of these areas which were apparent in 1974 and became attached to the bankline by 1990.

OVERALL RESPONSE IN THE FORT PECK REACH



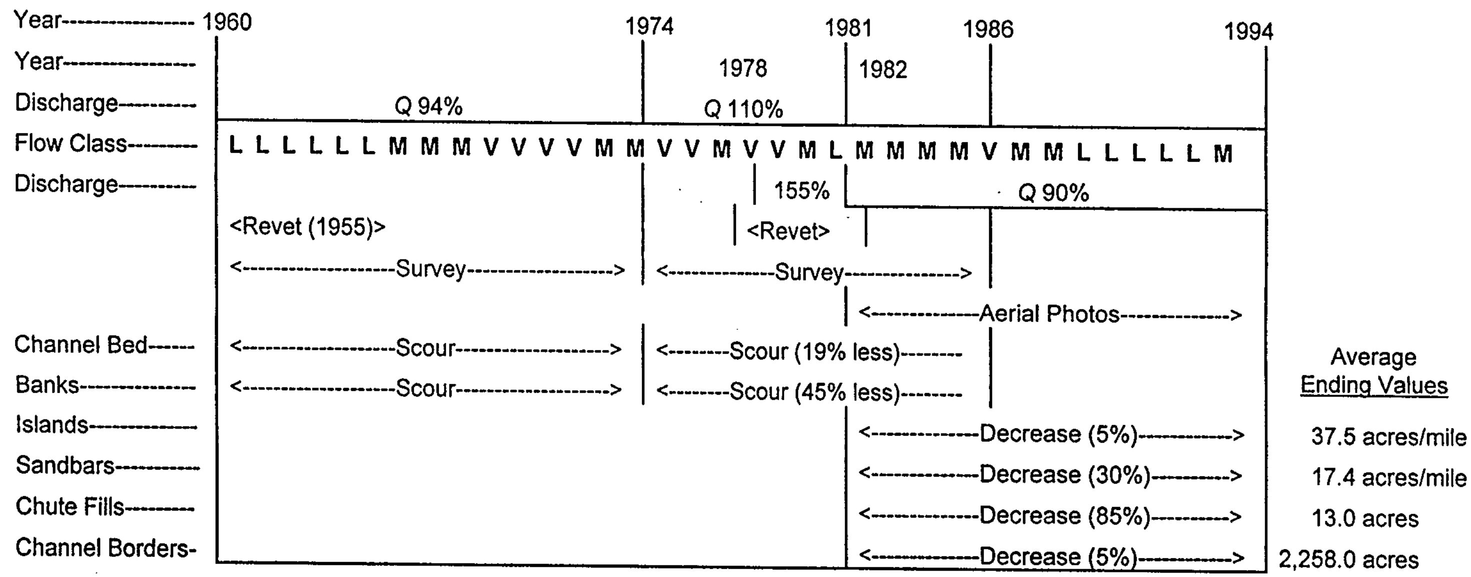
Note: Discharges presented are average yearly volumes relative to the 96-year average of the CWCP and PA.
Flow classifications are based on Chapter 2 values with the following symbols:
L = low flow conditions.
M = medium flow conditions.
H = high flow conditions.
V = very high flow conditions.
Decrease in sandbars was partially a result of several of these features being converted to channel border fills between 1976 and 1990.

OVERALL RESPONSE IN
THE GARRISON REACH



Note: Discharges presented are average yearly volumes relative to the 96-year average of the CWCP and PA.
Flow classifications are based on Chapter 2 values with the following symbols:
L = low flow conditions.
M = medium flow conditions.
H = high flow conditions.
V = very high flow conditions.
* = no discharge data available.

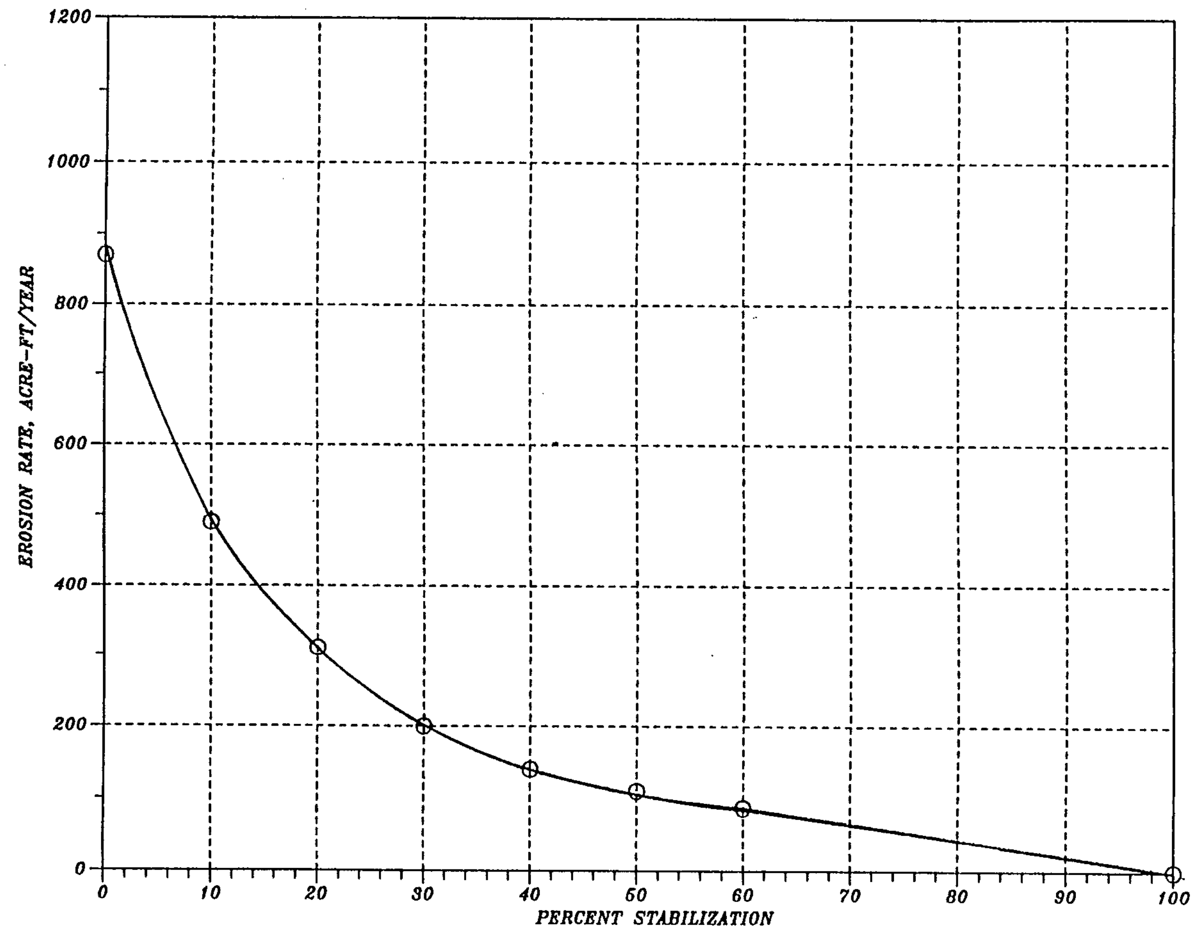
OVERALL RESPONSE IN THE FORT RANDALL REACH



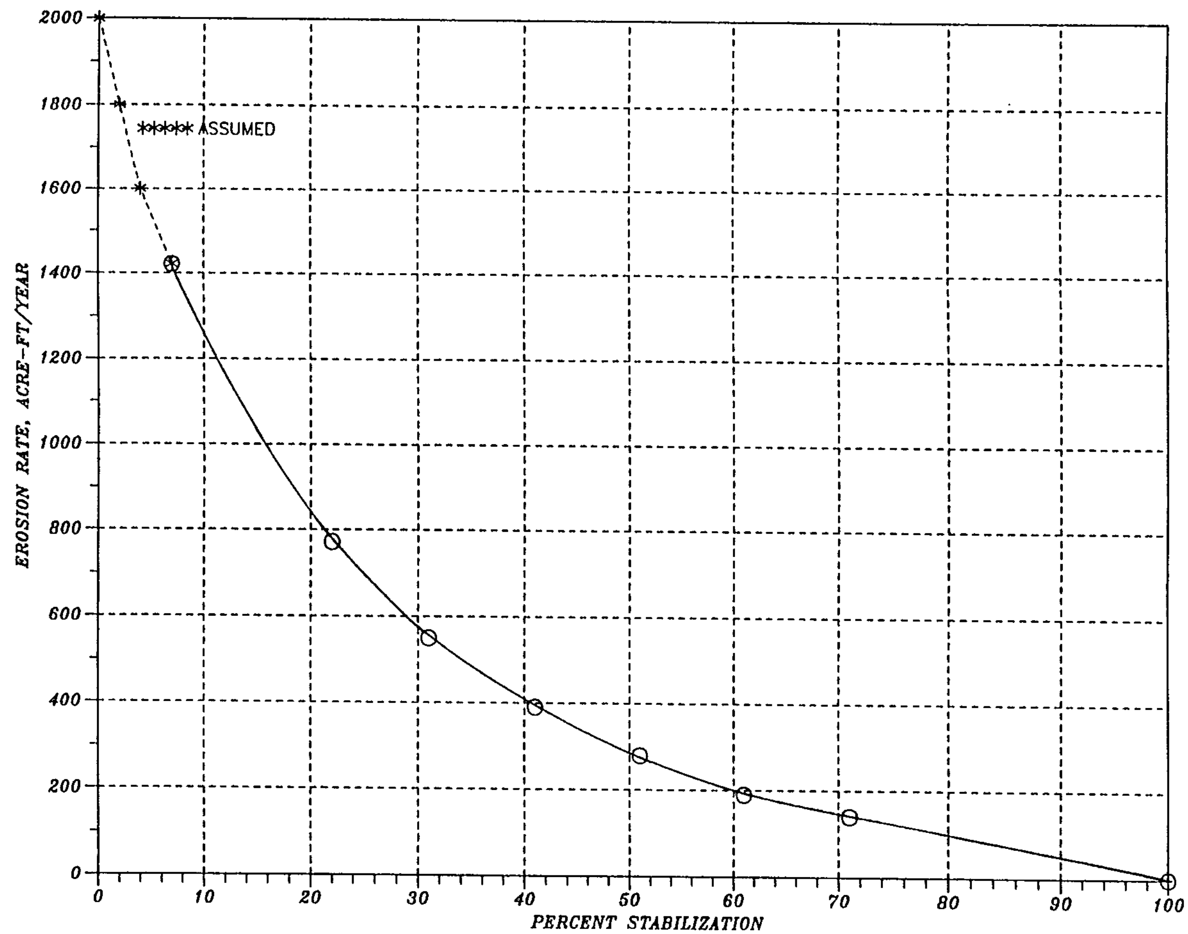
Note: Discharges presented are average yearly volumes relative to the 96-year average of the CWCP and PA.
Flow classifications are based on Chapter 2 values with the following symbols:
L = low flow conditions.
M = medium flow conditions.
H = high flow conditions.
V = very high flow conditions.

OVERALL RESPONSE IN THE
GAVINS POINT REACH

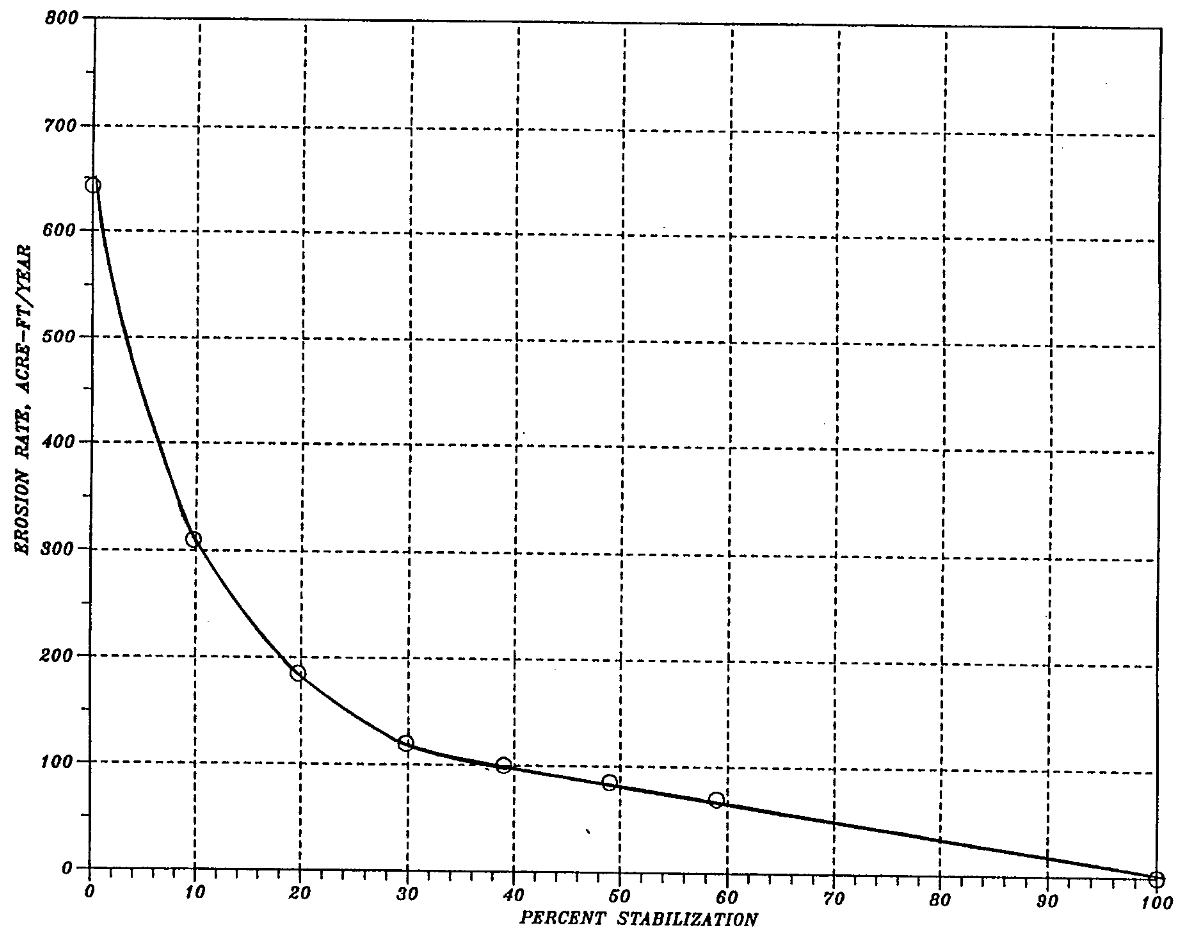
Plate 90



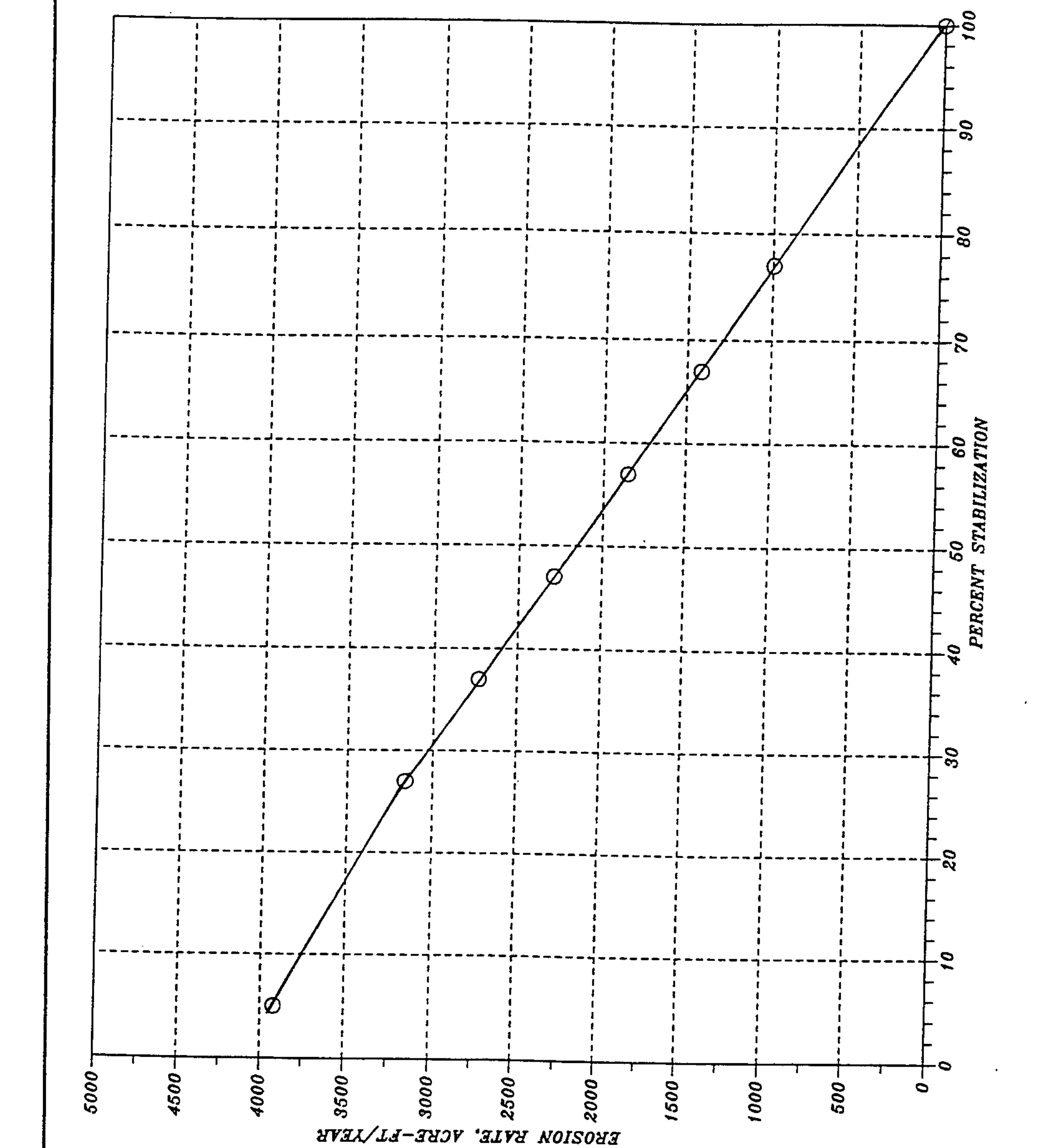
EROSION RATE VERSUS BANK STABILIZATION
FORT PECK REACH



EROSION RATE VERSUS BANK STABILIZATION
GARRISON REACH



EROSION RATE VERSUS BANK STABILIZATION
FORT RANDALL REACH



EROSION RATE VERSUS BANK STABILIZATION
GAVINS POINT REACH

Plate 94

REPORT DOCUMENTATION PAGE

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Thomas J. Pokrefke, David A. Abraham, Peggy H. Hoffman,
William A. Thomas, Stephen E. Darby, Colin R. Thorne

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13. ABSTRACT ***(Maximum 200 words)***

This study was undertaken to assist the Missouri River Region, U.S. Army Engineer Division, Northwestern, in determining if the Current Water Control Plan (CWCP) or an alternative plan best meets the current needs of the Missouri River. Initially it was determined that streambank erosion rates would increase resulting from a change in operation from the CWCP to the alternative identified as the Preferred Alternative (PA). These potential impacts, however, were not quantified; therefore, this study was undertaken to address the cumulative erosion impacts of changing the operation of the main stem dams and adding additional streambank erosion control measures. Also some resource agencies requested that an assessment of the effects of additional streambank erosion control be addressed. Four reaches totaling 362 miles downstream from Fort Peck (189 miles), Garrison (79 miles), Fort Randall (36 miles), and Gavins Point (58 miles) Dams were included in this evaluation. As part of a habitat evaluation effort, environmental features such as islands, sandbars, and backwater and chute habitats were analyzed. Other factors such as channel degradation, channel geometry, and turbidity were addressed to the extent possible.

14. SUBJECT TERMS

Bank stability	Islands
Bank stabilization	Sandbars
Habitats	Water control

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U.S. Army Engineer Waterways Experiment Station
3909 Halls Ferry Road, Vicksburg, MS 39180-6199;
Mobile Boundary Hydraulics, 606 Highway 80 West
Clinton, MS 39056-4183;
University of Nottingham
University Park, Nottingham NG7 2RD
United Kingdom