LMVD POTAMOLOGY STUDY (T-1)

A Report Submitted to
The Department of the Army
St. Louis District, Corps of Engineers
for the Period of
1 April 1975 to 30 June 1976

by

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Contract No. DACW 43-75-C-0105

30 June 1976

PREFACE

This study, "LMVD Potamology Study (T-1)" was conducted by an interdisciplinary team of engineers and scientists for the U.S. Army Corps of Engineers under Contract No. DACW 43-75-C-0105, dated 21 March 1975. The study was undertaken by the team of specialists for the purpose of assimilating data to be used to obtain a better understanding of the mechanisms and relationships that result in large scale change in the regime of an alluvial river due to man-made modifications. This study was limited to the compilation and processing of data relevant to dikes and revetments, levees, geology, morphology and hydrology.

The study was coordinated under the leadership of Dr. Paul R. Munger, who served as Project Director and editor of the report. Dr. Glendon T. Stevens served as Project Co-Director. Various disciplinary areas of the report were prepared by the following persons:

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Special acknowledgments are due the following persons for their assistance and cooperation during the tenure of this study: Mr. Norbert C. Long, who served as the Contracting Officer's Representative until March 31, 1976; Mr. Jack R. Niemi, the Authorized Representative of the Contracting Officer who served in that capacity from April 1, 1976 through

completion of the study; Mr. C.N. Strauser, Project Administrator; St. Louis District, Corps of Engineers personnel, Miss Jean Finn, Mr. Gary Schwartz, Mr. Michael Dace III and Mr. Gary Dyhouse; Memphis District personnel, Mr. David McNutt, Mr. Bobby Littlejohn; Vicksburg District personnel, Mr. Patrick Harris, Mr. Brian Winkley; LMVD Potamologist, Mr. James Tuttle; New Orleans District personnel, Mr. Billy Garrett, Mr. John Miller, Mrs. Pearl Burke; special consultants who assisted so effectively in locating the necessary data, Mr. Roger Lyon (Memphis), Mrs. Margaret Palmer and Mr. Austin Smith (Vicksburg), and Mr. Robert Turina (St. Louis); Mr. Melvin Jansen and Mr. Howard McCormick of the U.S.G.S. Water Resources Division (Rolla, Missouri); Mr. Robert G. Livingston, U.S. Army Engr. Topo. Labs (Retired), who assisted in obtaining aerial photography and in editing the report; and the many undergraduate and graduate students from the University of Missouri-Rolla, who assisted in the compilation and processing of data, along with students from Memphis State University and Tulane University. In particular, for their contributions in overseeing the work of a number of undergraduate students, the following graduate students deserve being cited: Mr. Leonard Woolsey, Mr. Paul H. Leaver, Mr. Jerry Maurseth and Mr. William Wilkerson, all of the University of Missouri-Rolla.

Without the cooperation and assistance of all of the above named persons, this study would not have been possible. In addition, the excellent cooperation and assistance provided by a number of federal, state and local agencies in obtaining information essential to this study is deeply appreciated.

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I. SUMMARY AND CONCLUSIONS

The primary purpose of this project was to collect and assemble data pertaining to hydrologic, hydraulic, geologic and morphologic factors which relate to the Mississippi River downstream from Alton, Illinois and to present them in a format which would be amenable to detailed analysis at a future time. Except for selected comparisons, analysis of the data was beyond the scope of work.

Hydrologic data are presented in a variety of graphical and tabular formats. Formats were selected such that stage, flow, precipitation, and precipitation-runoff relationships could easily be analyzed for both spatial (according to position in the basin) and temporal comparisons.

Vertical velocity distributions in cross sections at St. Louis and Chester during 1935 and 1973 apparently do not follow the Prandtl-von Karman universal velocity distribution law. This may be a source of error in comparisons of discharges made by different flow-measurement techniques.

In overbank areas near Vicksburg, relationships between river stage and overbank flow rates are well defined. There are no discernible depth-velocity relationships in the same overbank areas.

In the St. Louis district during the period from the late 1800's to the present, the specific effect of levees on scour and deposition on the overbank is indistinguishable from effects of other influencing factors.

There have been no discernible trends in water-surface-profile changes in the Middle Mississippi Reach since 1967.

In the St. Louis Reach, there has been an average bank-width reduction of about 29 percent (3320 feet to 2370 feet) since 1908. This reduction appears to be in response to dike construction. In the Memphis and Vicksburg Reaches, there is no apparent association between average bank width and dike construction.

Occurrences of overbank flow of sufficient magnitude to evaluate the possible influence of levees on stage-discharge relationships were too few.

At Hermann, Missouri, channel conveyance continually decreased from 1930 to 1973 as is reflected by increases in stage of about 2 feet to 3.5 feet for flows ranging from 64,700 cfs to 337,000 cfs, respectively.

For the period, 1881-1934, at St. Louis, stages for flows between 280,900 cfs and 501,300 cfs appear to have increased while stages for flows less than 209,200 cfs appear to have decreased. However, in the period, 1934 to 1973, channel conveyance steadily increased as is reflected by a stage decrease of 1.4 feet or more for all flows less than 501,300 cfs. The trend of increasing channel conveyance began at a time approximately coincident with accelerated dike construction activity which was observed in the period, 1925 to 1940.

Channel conveyance at Chester, Illinois has not changed significantly since 1943.

At Thebes, Illinois, stages for flows between 481,000 cfs and 572,000 cfs did not change significantly from 1934 to 1962, but increased by about 2.5 feet in the period from 1962 to 1974. For the entire 1934 to 1974 period, stages for mid-range flows (about 220,000 cfs) remained essentially unchanged, while stages for flows between 87,000 cfs and 135,000 cfs decreased a total of about 3 feet. Similar stage-discharge

patterns with respect to time are observed at Metropolis, Illinois, on the Ohio River. Backwater from the Ohio River occasionally influences flows in the Mississippi River at Thebes. Therefore, it appears that factors which change stage-discharge relationships in the lower Ohio River may affect stage-discharge relationships in the Mississippi River above their confluence, and vice versa.

At Memphis, Tennessee, channel conveyance increased significantly during the period, 1933 to 1971, as is reflected by stage decreases of 3 feet and 6 feet for flows ranging from 1,070,000 cfs to 260,000 cfs, respectively.

At Vicksburg, Mississippi, channel conveyance increased during the period, 1931 to 1942, as is reflected by stage decreases of about 10 feet for flows of 1,340,000 cfs and less. This trend reversed in 1942 or 1943 so that by 1973 stages had increased by about 5 feet from the 1942 values.

There is no consistent pattern of association between either dike construction or average top-bank width and stage-discharge changes with respect to time. It appears that changes in stage-discharge relationships are primarily influenced by factors, as yet unidentified, in the close vicinity of the respective points of records. Therefore, stage-discharge relationships at any given station do not necessarily reflect conditions for any appreciable distance from the station.

Because of flood protection implications, it is recommended that causal factors relating to stage increases at Thebes and Vicksburg be investigated in detail.

Surficial soils adjacent to the river were classified and mapped through use of infrared color photography. This information is presented on coded overlays.

Lithology of the river bed is presented in cross-section format. Lithologic data were derived from reports of levee borings and borings associated with bridge construction. Data from borings in the channel are few. Due to variable nature of alluvial deposits, extrapolation of information obtained from borings on the floodplain to the channel has a low reliability in terms of precise interpretation.

Contour maps representing the bedrock surface and associated bluff geology have been prepared for the reach from Alton, Illinois to Cairo, Illinois. It appears that the valley is more deeply entrenched than was previously indicated (Fisk, 1944). Control points are too few to permit contouring of the bedrock surface between Thebes Gorge and Cairo.

Morphology data are presented in formats designed to facilitate identification of changes in river features over time. Invert profiles show changes over a period of approximately 40 years for 5 irregularly spaced time intervals which were chosen on the basis of available data and occurrences which may have caused changes in the river morphology. Channel cross sections were prepared at intervals of about 30 miles throughout the study reach for the same approximate time periods chosen for the invert profiles. Changes in 1) bar and chute development, 2) meander pattern, 3) thalweg, and 4) energy dissipaters (within channel confines) are presented on overlays for time periods comparable to those used for profiles and cross sections. Energy dissipaters outside the channel during 1974 are also shown on an overlay.

The morphology overlays clearly show significant changes have occurred at various locations along the river in each of the various time intervals considered. Although no attempt was made to determine causal

factors for the observed changes, it is clear that these data in this format will be useful for such an analysis.

Dike, revetment, and levee information was tabulated showing date of construction, location, construction history (including modifications), materials used, physical characteristics, and current operational status. Levee crevasses were also tabulated, showing date of breach, location, type of breach, flood stage, size and type of area flooded, damage incurred, and physical features of levee and breach. Although information in many cases was missing, these historical presentations are essentially complete.

Information collected, compiled, and presented in this study should prove useful in evaluation of past engineering activities and in anticipation of effects of future engineering activities. These data could readily be used in providing a current base line for the Mississippi River, in which case the data file should be kept updated and developed, particularly in areas where data deficiencies are noted.

II. INTRODUCTION

The Mississippi River, America's greatest river, has the third largest drainage area in the world, exceeded in size only by the watersheds of the Amazon and Congo Rivers. It has, without question, played a major role in the physical and economical growth of the Nation. It is a navigation artery of great importance to the Nation's transportation system as well as a major supplier of water for the cities and industrial communities that have located along its bank. In its lower valley, the Mississippi River flows through one of the most fertile regions on earth. In recent years the recreational worth of this mighty river has increased tremendously. All of these factors point up the importance of the Mississippi River to the Nation's general economy and the economy of the lower valley.

But a river is a greater asset to navigation as a controlled river. Its waters form a navigation artery of tremendous worth as long as the channel is safe and dependable. A river is beneficial to the area through which it flows as long as it is made to work for the region and is not allowed to destroy it.

As a result of many years of work by the U.S. Army Corps of Engineers, the Mississippi River is being controlled in time of high water by levees and floodwalls which are designed to protect the alluvial valley against the project flood, except where it enters the natural backwater areas or is diverted purposely into floodway areas. Revetments have been utilized to minimize meanderings which would cut into and destroy protection levees and to provide a favorable channel alignment for navigation. Dikes

have been built to contract channel crossings, encourage closure of back channels and chutes, and to provide better channel alignment for navigation. Though not complete, these works have generally been very successful. However, the Mississippi River is a dynamic river, continually changing its bed configurations, moving bars and islands, making point bar cutoffs, and generally attempting to break out and resume its cross-valley wanderings.

Two major goals of Corps of Engineers work in the Lower Mississippi Valley are to produce and maintain a stable river system capable of safely conveying project flood flows to the gulf and to develop and maintain a safe and dependable navigation channel of authorized dimensions. Although tremendous strides have been made toward realization of these two goals, considerable work remains to be done.

The major flood on the Mississippi River in the spring of 1973 produced stages which made it apparent that the prevailing stage-discharge relationship was several feet higher than the stage-discharge relationship on which the levee grades and other flood control features had been based in the middle portion of the Lower Mississippi River and in the Atchafalaya Basin Floodway. In addition to deterioration problems observed during the 1973 flood, contraction works, revetments, and other features have aroused the concern of conservation interests and raised questions regarding flood flow capacities in the Upper Mississippi River. Also, excessive shoaling problems are occurring in the Lower Mississippi River below New Orleans, Louisiana. Questions have also been raised regarding the possibility of the deterioration problem migrating downstream and affecting the Baton Rouge to New Orleans reach of the river.

A revitalized and expanded potamology program designed to obtain a

better understanding of the mechanisms and relationships that give rise to large scale change in the regime of an alluvial river as a result of man-made modifications has been formulated, and this study covers a portion of the studies included in the expanded potamology program. This study has resulted in the compilation and processing of a large volume of data. In some cases, lack of data was apparent and in a few others, data were not available until too late to be included. It appears that detailed study of the data assembled would not only be desirable, but might reveal yet a better and clearer understanding of this many-faceted problem.

The areal extent of the study was that portion of the Mississippi River and the adjacent confined floodplain as required between Alton, Illinois and the Gulf of Mexico (Head of Passes), and including the Atchafalaya River. Study sites were selected to include locations of varying physical characteristics to provide sufficient coverages to represent the entire river.

Four districts of the Crops of Engineers were included: St. Louis, Memphis, Vicksburg and New Orleans. The jurisdiction of each district is as follows:

New Orleans--Atchafalaya River and mile 0 to mile 320.5, above Head of Passes

Vicksburg--Mile 320.5 to mile 599.0, above Head of Passes

Memphis--Mile 599.0 to mile 954.0 at Cairo, Illinois*, above Head of Passes

St. Louis--Mile 0 at Cairo, Illinois to mile 202.0 at Alton, Illinois.

*Cairo, Illinois represents mile 0 for the St. Louis District and is at mile 954.0 above Head of Passes. River mileage downstream and upstream from Cairo is with respect to the 1929 datum.

III. HYDROLOGY

Glendon T. Stevens

The drainage basin of the Mississippi River comprises approximately that portion of the United States lying between the Allegheny and the Rocky Mountains, except the Great Lakes and Hudson Bay drainages, with a total area of about 1,240,000 square miles. This covers about two-fifths of the total area of the United States proper. Water from part or all of 31 states, comprising the Mississippi River drainage basin, passes through the Mississippi River into the Gulf of Mexico (Figure 3.1). For convenience in this study, the entire drainage area has been divided into eight sub-areas as follows;

Region No.	Drainage Area	Drainage Area Sq. Mile
I	Upper Missouri	458,000
II	Lower Missouri	62,000
III	Upper Arkansas-Red	153,000
IV	Lower Arkansas-Red-White	117,000
V	Upper Mississippi	182,000
VI	Lower Mississippi	64,000
VII	Ohio	145,000
VIII	Tennessee-Cumberland	59,000

Figure 3.1 and the sub-basin areas were extracted from "Water Atlas of the United States" (Geraghty, et.al., 1973).

The purpose of this section of the LMVD (T-1) contract is to collect and display data that may be utilized in the design and operation of various control structures that will enhance navigation and flood protection on the Mississippi River.

The data needed to complete this section of the study consists of rainfall, mean-daily discharge and mean-daily stage.

The period of record for this section was chosen to be 1930, or the

Figure 3.1 Mississippi River Drainage Basin (from Geraghty, et al., 1973)

first date measurements were made (if after 1930), through the latest published date. This period was selected because the discharge data were thought to be more reliable and consistent. Prior to this time span, measuring equipment and techniques used to determine discharge varied; thus, the mean-daily discharge, mean-daily stage and rainfall data recorded prior to the 1930's were not included.

Rainfall data for the 31 states that make up the Mississippi River Basin were obtained from the National Weather Service for the period of record. These data were utilized in determining the following (*-work required under the contract):

1. The average annual precipitation for each of the eight regions that make up the Mississippi River drainage basin.

2. The average annual rainfall for:

   a. The Missouri River Basin (Reg. 1 and 2)
   b. The Ohio River Basin (Reg. 7 and 8)
   c. The Atchafalaya River Basin (Reg. 3 and 4)
   d. The Mississippi River Basin above St. Louis (Reg. 1, 2, and 5)
   e. The Mississippi River Basin above Memphis (Reg. 1, 2, 5, 7, and 8)
   f. The Mississippi River Basin above New Orleans (Reg. 1, 2, 5, 6, 7, and 8)

3. The average annual precipitation data for each of the eight regions were plotted on extreme value distribution paper, utilizing the Weibull plot position formula.

4. Data from steps 1 and 2 were plotted as a time series.

5* The data from step 2 are presented graphically, along with runoff, as a time series.†

The mean-daily stage and mean-daily discharge data needed to complete this section of the potamology study were obtained for the period of record from the United States Corps of Engineers and the United States Geological Survey, Water Resources Division. Mean-daily stage and mean-discharge data were collected for eighteen locations on the Mississippi

River and its major tributaries. Locations on the Mississippi River were Keokuk, Iowa; Alton, Illinois; St. Louis, Missouri; Chester, Illinois; Thebes, Missouri; Memphis, Tennessee; Vicksburg, Mississippi; Helena, Arkansas; Arkansas City, Arkansas; and Red River Landing, Louisiana. Locations on the major tributaries were Meredosia, Illinois on the Illinois River; Hermann, Missouri on the Missouri River; Metropolis, Illinois on the Ohio River; Paducah, Kentucky on the Tennessee River; Little Rock, Arkansas on the Arkansas River; Clarendon, Arkansas on the White River; Alexandria, Louisiana on the Red River; and Simmesport, Louisiana on the Atchafalaya River. Data collected for each of the above-mentioned stations are displayed in the following forms (*-work required under the contract).

1* For each location, the United States Geological Survey, Water Resource Division eight-digit station number, bankfull stage and drainage area (square miles).

2* The average mean-daily stage and average mean-daily discharge for each year.

3* The average mean-daily stage and average mean-daily discharge for the low-water season (July-November) for each year.

4* The annual volume (inches over drainage basin) of runoff.

5* The number and listing of days per year that the mean-daily stage exceeded bankfull.

6* Percent of time per year mean-daily stage exceeded bankfull.

7* A histogram of mean-daily stage and mean-daily discharge showing the number of times per year that the stage and discharge were within predetermined intervals.

8* The volume (inches over drainage basin) of runoff for each month per year.

9* The average monthly volume of runoff for the period of record.

10* The percent of runoff for each month per year.

11* The average monthly percent runoff for the period of record.

12* The long-term yearly average

a. mean-daily stage (feet)
b. mean-daily flow (1000 cfs)
c. runoff (inches)
d. low-water season mean-daily stage and mean-daily discharge
e. high-water season mean-daily stage and mean-daily discharge

13* The deviation of the yearly averages of items 12, a, b, c, d and e from the long-term averages.

14. A complete listing of recorded mean-daily stages and mean-daily discharges.

15. The yearly minimum and maximum mean-daily stages.

16. The yearly minimum and maximum mean-daily discharges.

17. The long-term average mean-daily stage for each day of a calendar year.

18. The long-term average mean-daily discharge for each day of a calendar year.

19. The average mean-daily stage and average mean-daily discharge for the high-water season (December-June) for each year.

20. The deviation of the monthly average mean-daily stage and monthly average mean-daily discharge from the long-term yearly average for each year.

21. The long-term monthly, average mean-daily stage and mean-daily discharge.

22. The monthly average mean-daily stage and monthly average mean-daily discharge for each year.

23. The deviation of the mean-daily stage and mean-daily discharge

for each month of each year from the long-term mean-daily stage and mean-daily discharge for each month.

24. The mean-daily stage and mean-daily discharge data have been analyzed to determine

    a. the 1, 7, 14, 30, 60, 90 and 120 consecutive day minimums.
    b. the 1, 7, 14, 30, 60, 90, 120, 150 and 180 consecutive maximums

25. Data from 24a and 24b above have been plotted on extreme value distribution paper, utilizing the Weibull plot position formula.††

Data as presented in this section are amenable to making many hydrologic studies and comparisons. The following is a brief list of some of the studies that could and possibly should be undertaken:

A. Using the average annual precipitation data presented, one could study and predict the probability of the joint occurrence of various events. A study of this nature would be worthwhile in the design of flood protection structures and floodplain management.

B. The annual precipitation data, coupled with other pertinent data, could be utilized in studying the wet and dry cycle, thus assisting in developing information that could be utilized in reservoir regulation and operation.

C. Rainfall and runoff data could be utilized in studying the ever-changing land use management practices and possibly the study of the changing sediment (wash) load carried by the Mississippi River and its tributaries.

D. The data presented herein should be coupled with data from Morphology and Geology sections and a study conducted to understand more fully energy dissipaters as to type, location, development, etc.

E. The long-term average mean-daily discharge and mean-daily stage for day of the year would be useful in developing:

a. The long-term average mean-daily discharge hydrograph
b. The long-term average mean-daily stage hydrograph
c. The long-term average mean-daily stage-discharge relationship (rating curve)

The major stumbling block which all encounter who study the response of a river to changes made by man is that of sufficient and adequate data and its acquisition. Therefore, the first two recommendations which this or any other study team could make are:

1. That a comparison of the various types of measuring equipment and techniques used in determining discharge throughout the years in which measurements have been made should be undertaken. Such a study would result in the development of relationships between the various techniques and equipment used to measure discharge. These relationships could then be utilized in adjusting discharge data to the technique and equipment presently used.
2. That a central computerized data bank should be established. This data bank should contain all measured data that have been or will be collected on the Mississippi River and should have the retrieval capability that would make it useful to all.

Note: †A listing of the computer programs used in processing the data can be found in Appendix 3.1. The computer printout of the results obtained from the various calculations made with the rainfall data can be found in Appendix 3.2.

††The computer printout of the results obtained from the various calculations made with the mean-daily discharge and mean-daily stage can be found in Appendix 3.3 which contains four volumes. Tributary data are found in volumes 1 and 2. Volume 3 contains those stations upstream from Memphis. Volume 4 contains Memphis and those stations downstream.

IV. HYDRAULICS

Clifford D. Muir and Jerome A. Westphal

While this section relates directly to contract items specified under hydraulics, the directly related subject of 1) river confinement by levees, 2) comparison of dike construction history with associated changes in top bank width, and 3) comparison of dike construction history with changes in stage-discharge relationship, are included in discussions within this section of the report.

The limited number of overbank velocity measurements obtainable within the time frame of the project proved to be a major constraint on efforts to develop velocity relationships on overbank areas. Most of the available data were flow rates in the overbank area. However, complete flood-stage measurements of velocities and associated depths for the St. Louis and Chester gage locations were obtained for years after 1935. Also, velocities in the overbank area were obtained for the Vicksburg area for the years 1929, 1933, 1935, and 1973.

Estimated velocity distributions at the St. Louis gage are shown on Figures 4.1a and 4.1b for July 1, 1942 and May 1, 1973, respectively. Isovels were contoured on the basis of point velocities taken for discharge measurements. Because of the flood wall at St. Louis, there is relatively little overbank area. There were apparently no striking changes in cross-section shape or velocity distributions at the section between 1942 and 1973.

Estimated velocity distributions at the Chester gage were made in the same manner as at the St. Louis gage and are shown on Figures 4.2a and 4.2b for July 17, 1951 and May 14, 1973, respectively. Although

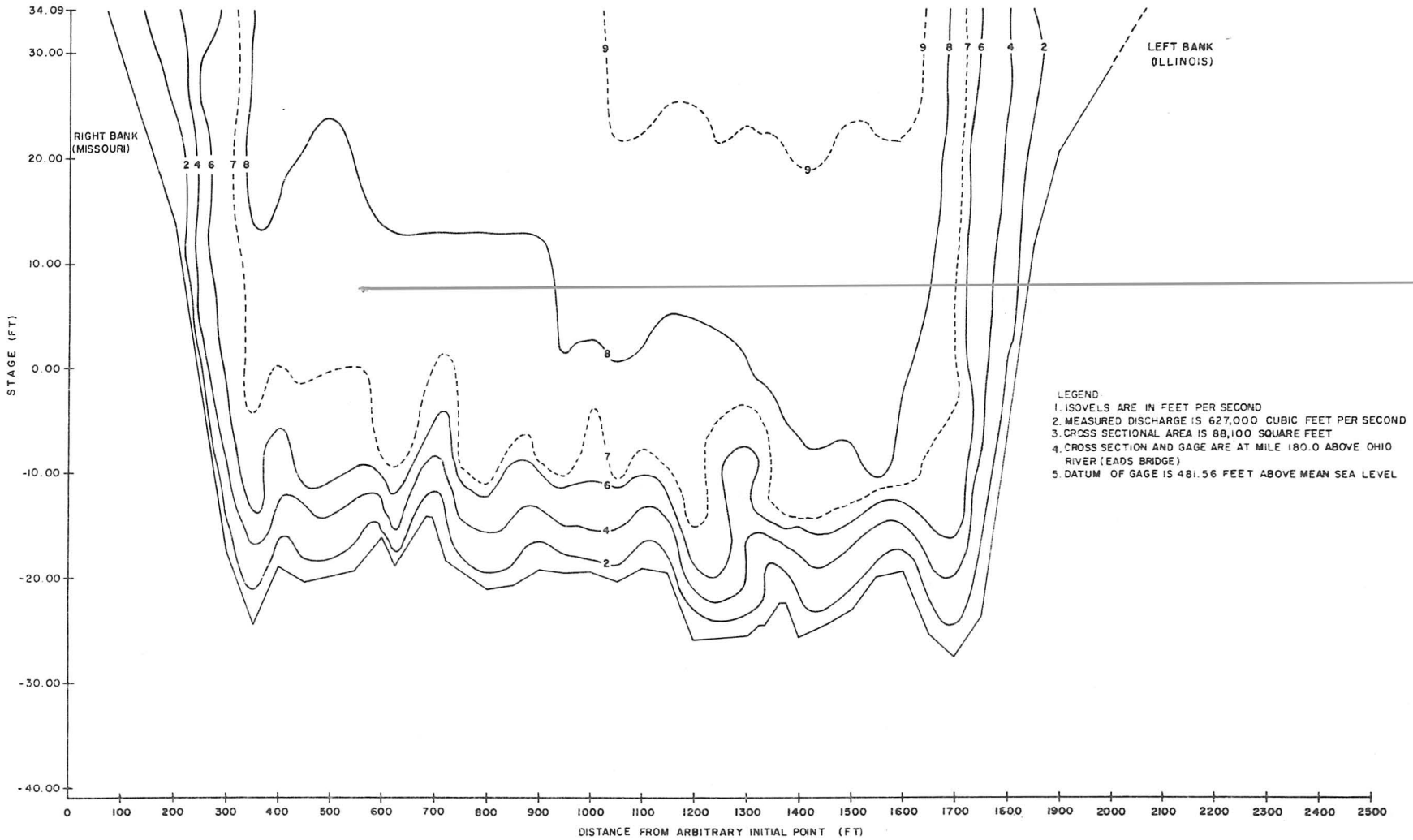

Figure 4.1a Estimated Velocity Distributions in the Mississippi River for July 1, 1942 near St. Louis, Missouri

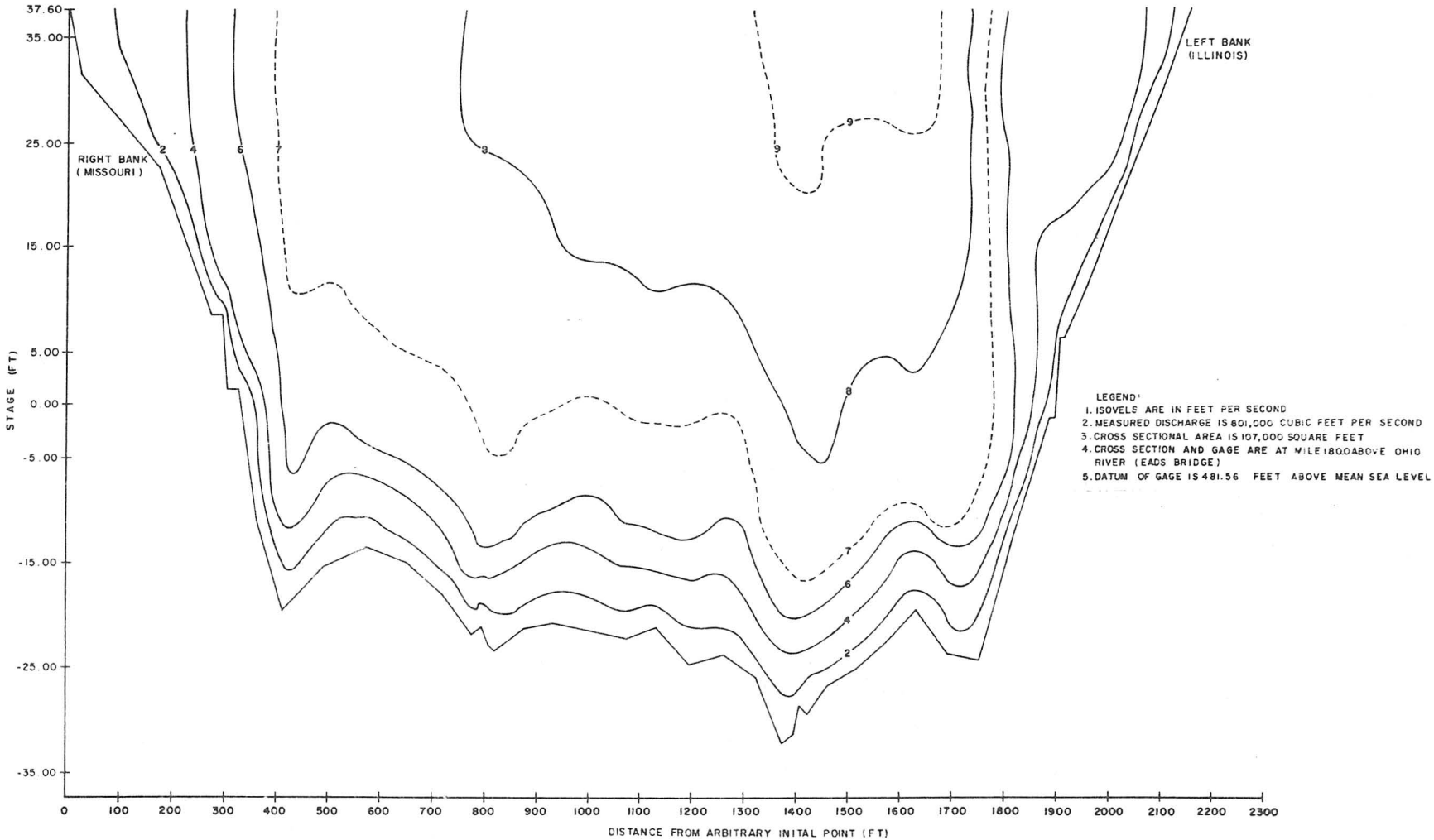

Figure 4.1b Estimated Velocity Distributions in the Mississippi River for May 1, 1973 near St. Louis, Missouri

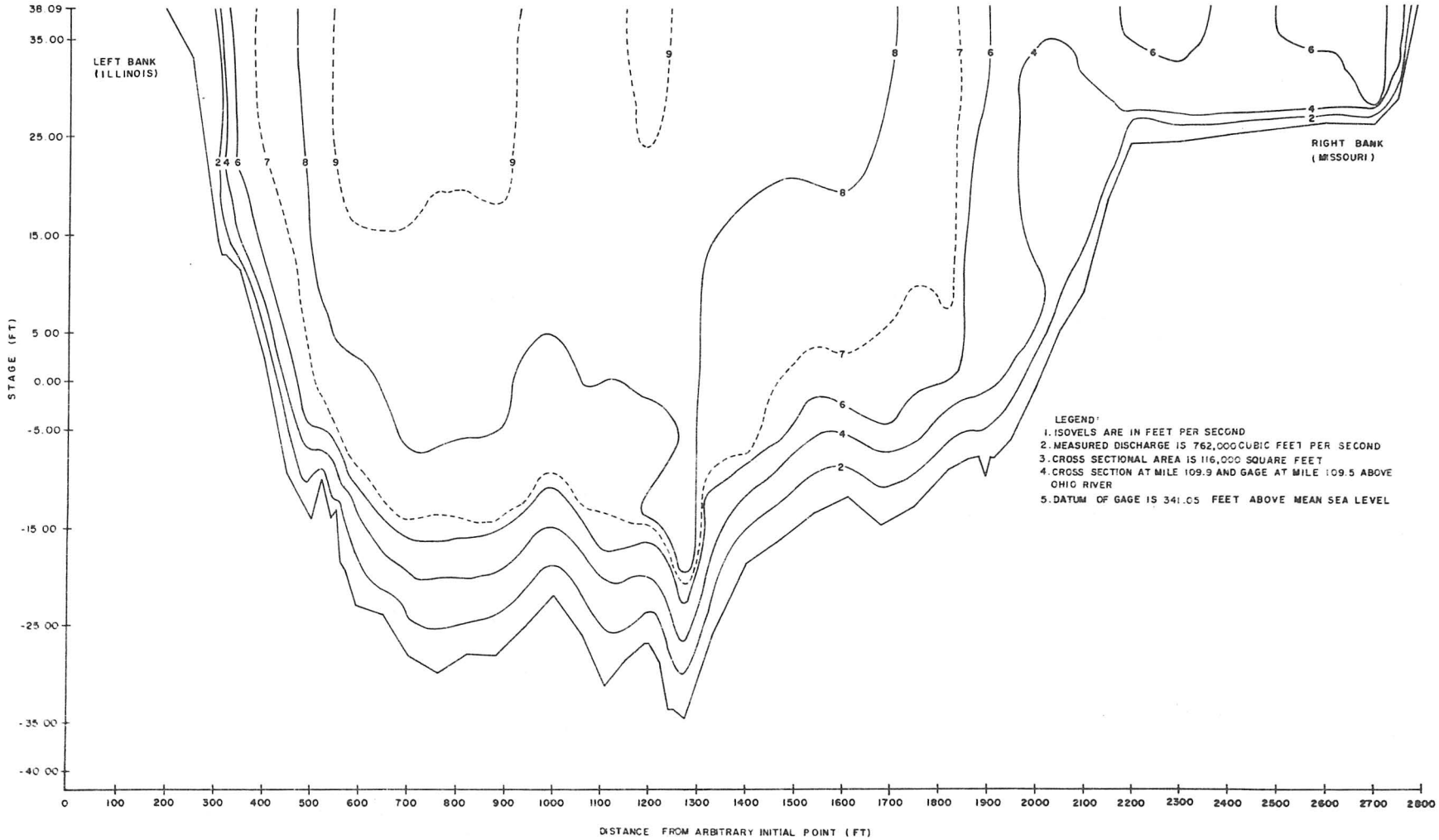

Figure 4.2 a Estimated Velocity Distributions in the Mississippi River for July 17, 1951 near Chester, Illinois

Figure 4.2b Estimated Velocity Distributions in the Mississippi River for May 14, 1973 near Chester, Illinois

the thalweg has apparently migrated toward the left bank (Illinois side) in the period between 1951 and 1973, cross-sectional area and shape are nearly the same for the two dates. The similarity of isovel patterns is probably a reflection of the similarity of cross-section geometry.

Typical overbank velocities for several locations at Vicksburg are included in Tables 4.1, 4.2, and 4.3. Attempts to correlate velocity on the overbank with depth of flow were unproductive. For the range of observations made thus far, there is no discernible functional relationship between depth of flow on the overbank and overbank velocities. Before difinitive statements can be made in this regard, detailed information regarding energy dissipaters are needed. For example, in the St. Louis District where brush had been cleared in front of the levees, a secondary channel developed during flood stages. To overcome overbank velocities and secondary channels, the St. Louis District has constructed spur levees and abatis dikes and has encouraged willow growth in old borrow pits. Therefore, overbank velocities at a given site probably reflect local conditions more than the generalized influence of levees or confinement.

Maximum velocities on the overbank are less than main channel maximum velocities. The 1933 overbank maximum velocity of 2.55 ft/sec at Vicksburg corresponded to a main channel maximum velocity of 7.98 ft/sec. The Chester overbank maximum for May 14, 1973, appeared to be about one-half the main-channel maximum.

A comparison of flow rates in the overbank area and river stage in the vicinity of Vicksburg is presented in Figure 4.3. The 1973 values were for a cross section 0.4 miles downstream from the Vicksburg Bridge. Zero gage at this section was 46.25 feet above mean sea level. The 1929

TABLE 4.1

OVERBANK FLOW

LOWER DELTA POINT, LA. JUNE 7, 1929

STAGE 55.1 FT.

Distance from River Toward Levee (ft)	Depth (ft)	Mean Velocity (fps)
0	6	0.60
500	8	0.37
1000	14	0.47
1500	15	0.43
2000	17	0.13
2500	18	0.13
3000	17	0.43
3500	18	0.47
4000	19	0.64
4500	20	0.80
4600	16	0.53
4700	12	1.01
4800	11	1.07
4900	11	1.21
5000	12	1.34
5100	11	1.53
5200	11	1.64
5300	9	1.81
5400	7	1.14
5500	5	1.58
5700	0	0.00

TABLE 4.2

OVERBANK FLOW
LOWER DELTA POINT, LA.

Date	Stage (ft)	Distance from River Toward Levee (ft)	Depth (ft)	Mean Velocity (fps)
June 5, 1929	55.10	0	4	0.68
		500	6	1.22
		1000	12	0.32
		1500	17	0.58
		2000	19	0.78
		2500	17	0.35
		3000	17	0.65
		3500	19	0.68
		4000	19	0.58
		4500	18	0.55
April 22, 1933	48.03	100	6.5	0.77
		160	5.5	1.50
		250	6.2	2.05
		275	4.7	2.35
		390	5.5	1.88
		450	5.0	2.22
		530	5.3	2.09
		595	5.5	2.40
		620	5.5	2.46
		660	5.5	2.24
		705	5.5	2.55
		750	0.0	0.00

TABLE 4.3

OVERBANK FLOW

VICKSBURG-HIGHWAY BRIDGE; MAY 14, 1973

Distance from River Toward Levee (ft)	Depth (ft)	Mean Velocity (fps)
0	6.0	.216
200	5.4	.320
400	5.6	.560
600	4.9	.720
800	6.2	.510
1000	11.6	.720
1200	12.8	.710
1400	13.2	.560
1600	11.4	.590
1800	12.1	.710
2000	12.2	.640
2200	13.4	.780
2400	15.2	.500
2600	15.5	.590
2800	16.8	.500
3000	16.7	.910
3200	16.0	.820
3400	16.0	1.100
3600	16.7	.930
3800	17.8	.558
4000	18.7	.940
4200	12.1	.880
4400	8.3	1.290
4600	10.1	1.26
4800	8.6	.864
4900	7.6	1.170
5000	4.1	1.188
5050	Close to 0	

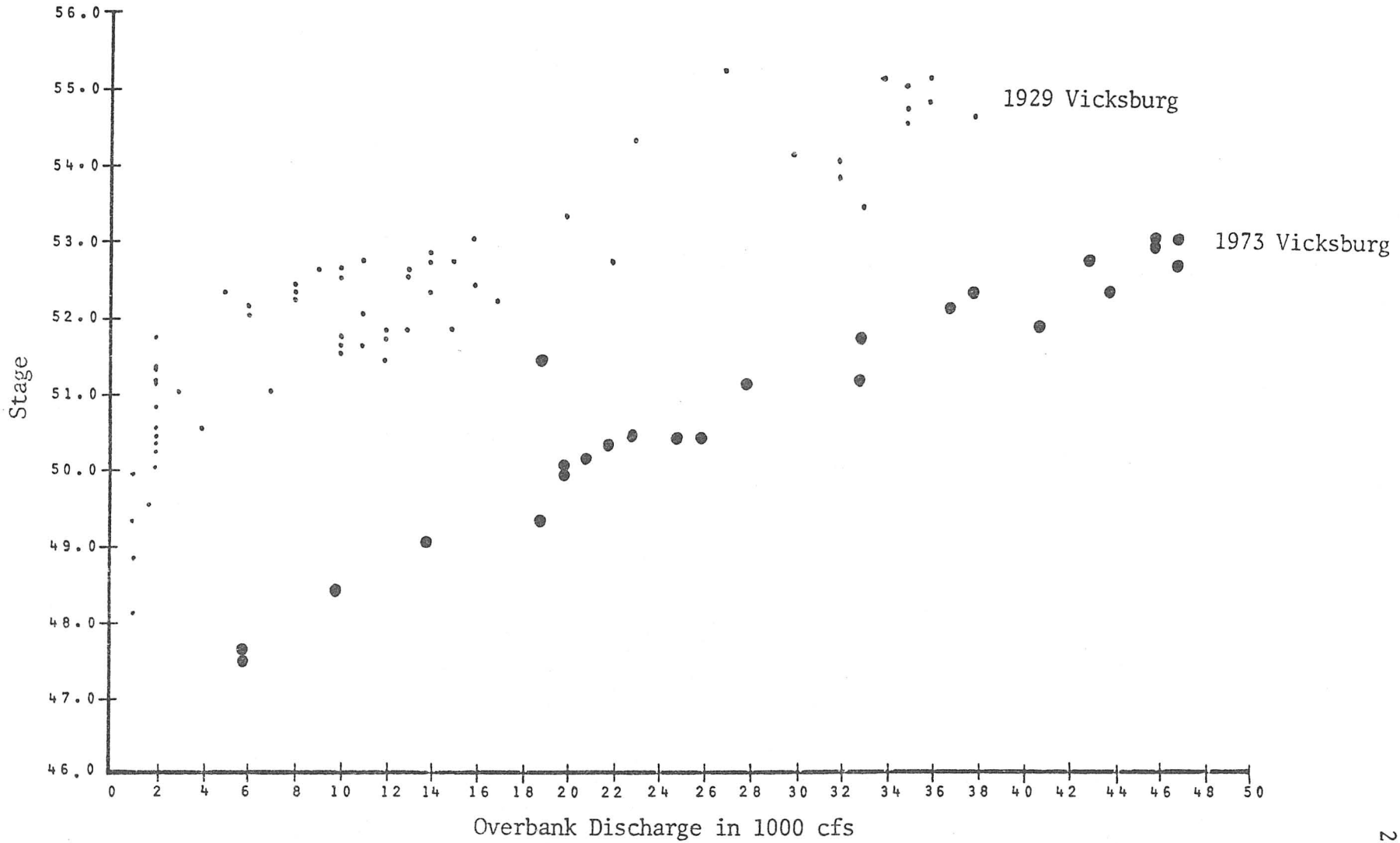

Figure 4.3. River Stage Versus Overbank Discharge Near Vicksburg for 1929 and 1973

values were taken at Lower Delta Point (precise location unknown). Zero gage at Lower Delta Point was 46.16 feet above mean Gulf level. The 1929 data were much more variable than the 1973 data, possibly because of variation of flow-measurement techniques in 1929. However, stage-discharge relationships are reasonably well defined for both periods. Therefore, it appears that on the overbank, localized influences such as vegetation and channelization are sufficient to mask depth-velocity relationships but do not affect the relationship between average flow rate and river stage.

River confinement by levees is both detrimental and beneficial. It prevents the enrichment of agricultural lands which accompanies the deposition of waterborne fine material on protected areas. However, the rather large benefits derived from flood protection cannot be disregarded. According to personal communication from Mr. Michael Dace, III, of the St. Louis District, some farmers whose lands are unprotected by federal levees would accept spur levees over the more expensive closed levees. The purpose would be to allow controlled flooding with the accompanying benefit of deposition of fine materials.

Deposition of coarse materials on agricultural land is detrimental. In the absence of data necessary to evaluate the main channel as a source of coarse sediments which are deposited on overbank areas during floods, a theoretical approach must be used. The problems are 1) to determine the amount of coarse sediments in suspension above banks and 2) to determine if these sediments would be deposited on overbank areas.

An equation commonly used to calculate particle concentrations is:

$$\frac{C}{C_a} = \left\{\frac{(d-y)}{y} \quad \frac{a}{(d-a)}\right\}^z \tag{4.1}$$

where C = the concentration of particles of a given size at a distance y above the bed,

d = mean depth of flow,

y = height of point above river bed,

a = some reference point above the river bed,

C_a = the concentration of particles of a given size at reference point a, and

w = settling velocity of the particles.

The exponent $z = w/ku_*$ (4.2)

where u_* is the shear velocity expressed as $u_* = \sqrt{gRS}$, and (4.3)

S = slope of the hydraulic grade line,

R = hydraulic radius (R = d for rivers),

g = acceleration due to gravity, and

k = von Karman's universal constant.

Toffaleti (1963) developed the following simplified relationships for the lower Mississippi River:

$$C = b(d/y)^z \tag{4.4}$$

where b is a constant equal to the particle concentration when d/y equals unity, and

$$z = \frac{\overline{U}w}{252Sd} \; . \tag{4.5}$$

Toffaleti (1968) states that the distribution of particles less than 0.062 millimeters was fairly uniform at Simmesport on the Atchafalaya River. He listed typical sediment size ranges and classifications as:

Description	Size Range (mm)	Geometric Mean Diameter (mm)	(ft)
Silts and Clays (S&C)	0.062		
Very Fine Sand (VFS)	0.062-0.125	0.0880	0.00029
Fine Sand (FS)	0.125-0.250	0.177	0.00058
Medium Sand (MS)	0.250-0.500	0.354	0.00116
Coarse Sand (CS)	0.500-1.000	0.707	0.00232

Using a typical section at Talbert's Landing on the Mississippi River it was assumed that bed-load gradation would be similar to that at Simmesport shown above. The following table shows the fraction of bed material and settling velocities for particle sizes in the assumed gradation:

Geometric Mean Diameter (ft)	Fraction of Bed Mtl.	Settling Velocity (ft/sec) $70^{o}F$	$80^{o}F$
0.00029	0.071	0.023	0.025
0.00058	0.283	0.069	0.075
0.00116	0.564	0.171	0.183
0.00232	0.078	0.356	0.373

Other assumptions made for calculations were as follows:

	Case I	Case II
Q (main channel)	1,395,800 cfs	1,008,200 cfs
River width	3,830 ft	3,730 ft
Temperature	70°F	80°F
Slope	0.0000382	0.0000382
Area of flow	188,400 ft^2	173,900 ft^2
Ht. of water surface above elevation at which overbank flow begins	10 ft	3.5 ft

The assumed flows and widths correspond to data given for May 5, 1973 and June 12, 1973, respectively. The coefficient b from equation 4.4 was assumed to be the same as that determined for the commensurate particle size by Toffaleti (1968).

The above assumptions were used in equations 4.4 and 4.5 to make determinations of sediment transport. The resulting determinations are given in tabular form. Quantities are given in both tons per day per square foot ($t/d/ft^2$) and parts per million (ppm).

Sediment Transported

Height (ft) Above River Bed	Case I Geometric Mean Diameter (ft) 0.00029		0.00058		Case II Geometric Mean Diameter (ft) 0.00029		0.00058	
	$t/d/ft^2$	ppm	$t/d/ft^2$	ppm	$t/d/ft^2$	ppm	$t/d/ft^2$	ppm
1	7.79	389	7.29	364	4.24	721	4.34	227
10	4.95	274	0.87	43	2.84	181	0.62	30.0
30	3.40	170	0.19	9.5	2.00	128	0.15	9.6
39.2	3.00	150	0.12	6.0				
43.1					1.65	105	0.09	5.8
46.2					1.62	103	0.08	5.10
49.20	2.69	134	0.08	3.99				

To examine the possibility of material being lifted or maintained in suspension, Toffaleti's (1968) version of Einstein's weight-to-lift ratio was used. This equation is

$$\frac{W}{L} = \frac{T}{U^2} 10^4 D \tag{4.6}$$

where W = weight of sediment particle underwater,

L = lift force on bed particle,

T = temperature dependent variable (0.063 for temperature = $70^{o}F$), and

D = particle size diameter.

Assuming a maximum weight-to-lift ratio (W/L) of unity and T equal to 0.063, the minimum velocities required to move sediments on the overbank at a water temperature equal to $70^{o}F$ may be calculated from equation 4.6. For geometric mean diameters of 0.00029 ft. and 0.00058 ft. the minimum velocities are 0.43 ft/sec and 0.60 ft/sec, respectively.

Overbank velocities given for the Vicksburg area in tables 4.1, 4.2, and 4.3, indicate average velocities which are greater than those needed to transport fine sand. Such overbank velocities will not only transport the suspended sediment reaching the overbank but will cause local scour. Deposition will occur only where velocities are less than about 0.43 ft/sec and will be a combination of material supplied by mainstream flow and scour from the overbank. Therefore, deposition of sediment on the overbank will generally be determined by the local configuration (either manmade or natural) of a particular area.

The preceding determinations are very sensitive to assumptions and tend to overestimate the amount of sediment in transport. They indicate the presence of fine sand above bank in the main channel. However, the concentration of fine sand and larger materials at or above the river bank elevation is relatively small. Because it appears from theoretical considerations that concentrations of fine sands are very small near the surface in the main channel, and because velocities on the overbank appear to be greater than the minimum required to keep fine sands in suspension, it is unlikely that the main channel is a significant source of sands deposited on the overbank during floods.

To evaluate effects of confinement and nonconfinement on sediment deposition in overbank areas, 13 cross sections were selected for comparisons of 1974 conditions with those existing during the decade 1879 to 1889. Cross sections were taken at intervals of approximately 10 miles beginning at mile 44.2 and ending at mile 169.8 above Cairo. At most of the cross sections, surveys made during the 1800's did not extend the full width of the flood plain.

At any given cross section, deposition and scour depend on relatively localized factors such as alignment of the river, channel stabilization measures instituted, and development associated with the various forms of land use on the flood plain.

The majority of channel configuration changes are attributable to channel stabilization structures, except at Kaskaskia Island where the Mississippi River captured a part of the Kaskaskia River and formed a cutoff which shortened the Mississippi River by about 11 miles. The old channel subsequently filled with sediment. The majority of changes

on the flood plain are attributable to activities of riparian owners and installation of drainage facilities which were accomplished under authority of the various drainage districts.

The specific effect of levees is indistinguishable from effects of other factors which influence scour and deposition on the overbank. Although the topographic data were insufficient for detailed analysis of change, it appears that in spite of considerable flood-plain development, there has been relatively little change in the valley cross sections during the last 100 years.

Water-surface profiles for annual high and low stages in the Middle Mississippi River for the period, 1967 through 1974, are shown in Figures 4.4 through 4.11. The method used for Figures 4.4 through 4.7 was to select annual maximum and minimum stages at St. Louis gage, then plot corresponding stages at all other stations for that date. For Figures 4.8 through 4.11, annual maximum and minimum stages at each station were plotted irrespective of date of occurrence. Average slope of the annual water-surface profiles shown in Figures 4.4 through 4.7 and the average slope of these for the entire period from 1967 through 1974 is presented in Table 4.4. Also shown in Table 4.4 are slopes of annual maximum and minimum water-surface profiles and the average slope for the period, 1967 through 1974, for selected reaches of the Upper Mississippi, Missouri, and Illinois Rivers.

Although there is considerable variability in slope of the water-surface profile from year to year in each reach, there is no discernible time trend. However, as shown in Figure 4.4, average slope of the high-water profile is steeper than the low-water profile for the Middle

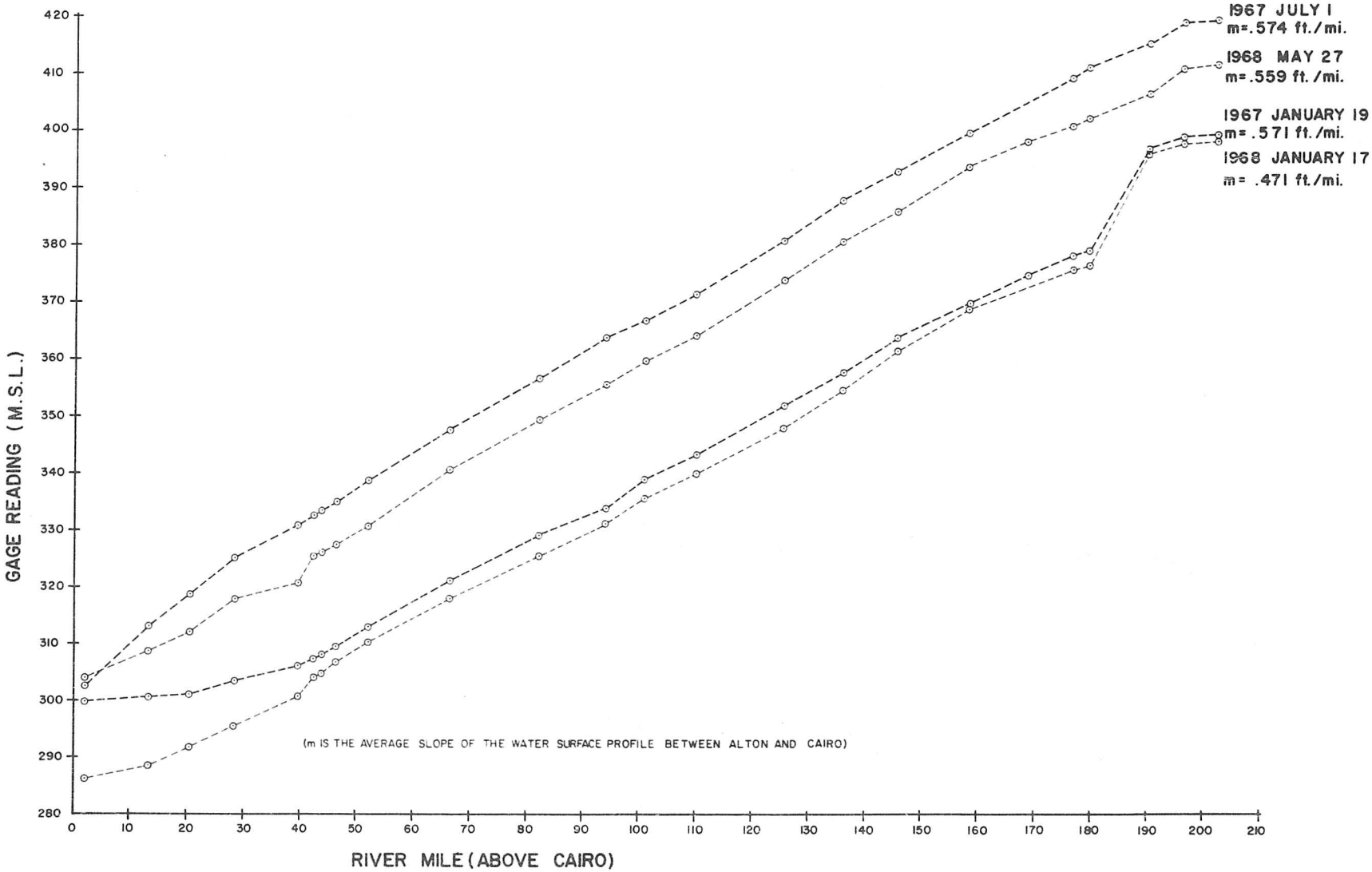

Figure 4.4 Surface Profiles of the Mississippi River Between Alton and Cairo for Maximum and Minimum Stages at St. Louis for 1967 and 1968

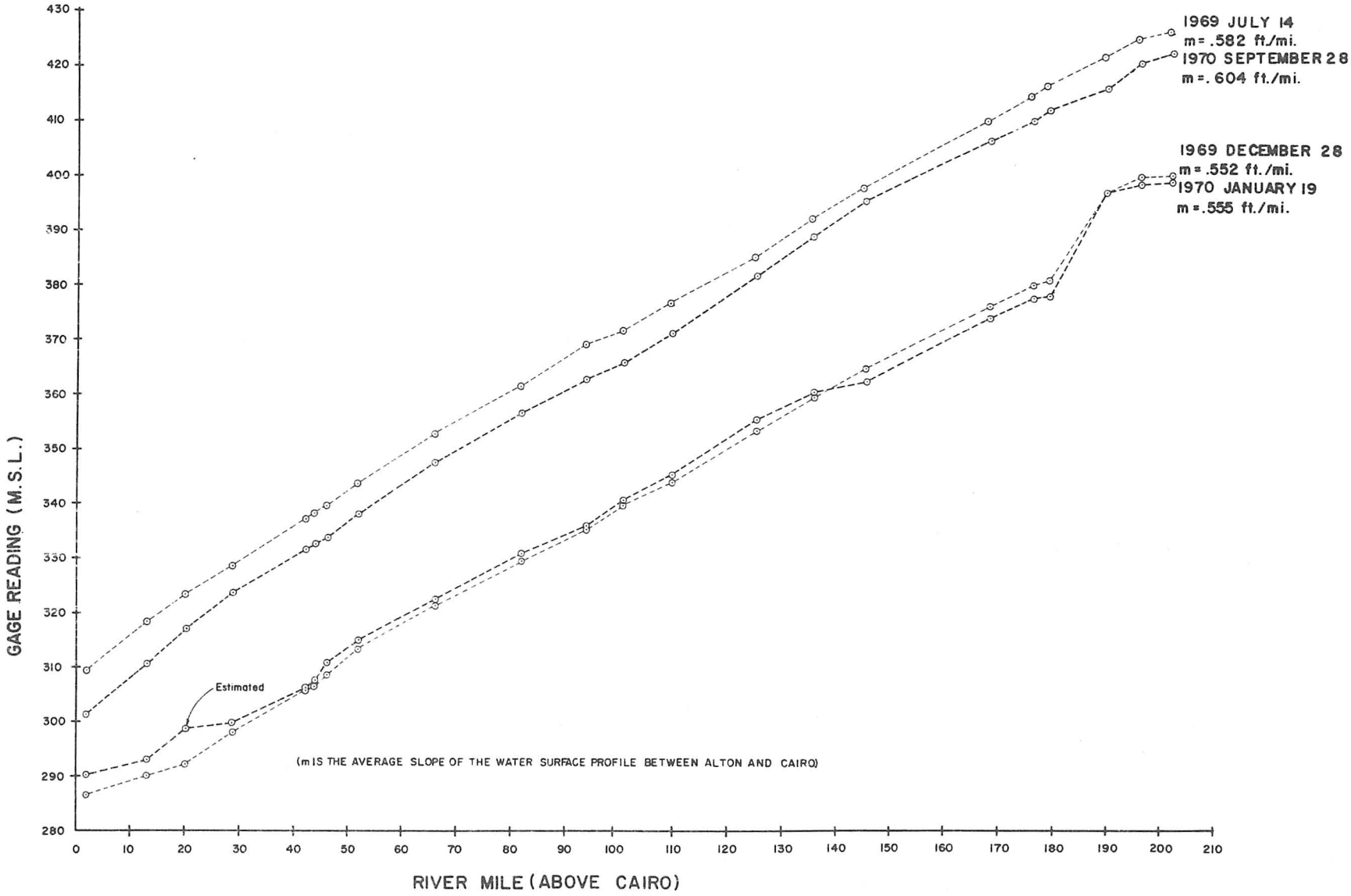

Figure 4.5 Surface Profiles of the Mississippi River Between Alton and Cairo for Maximum and Minimum Stages at St. Louis for 1969 and 1970

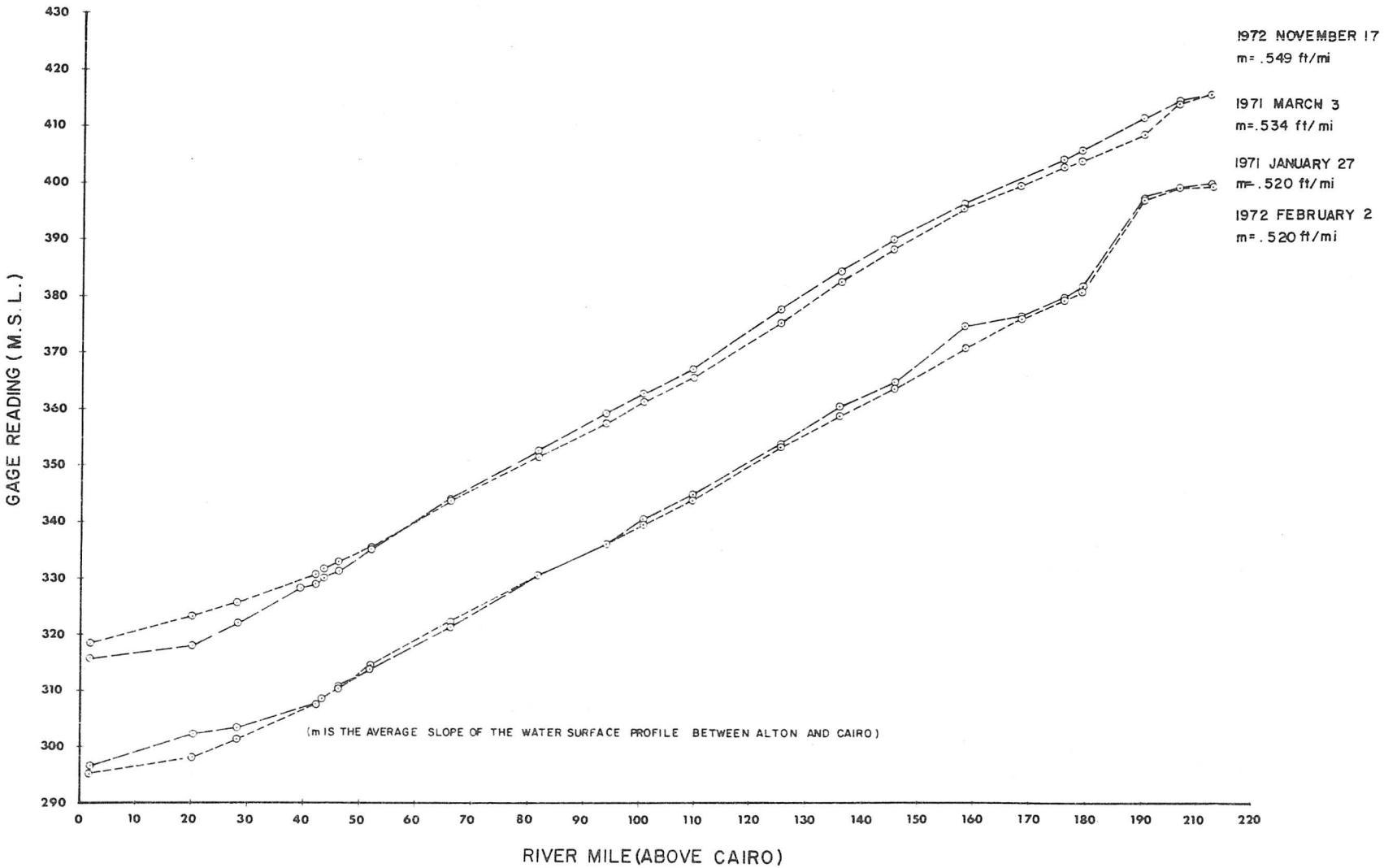

Figure 4.6 Surface Profiles of the Mississippi River Between Alton and Cairo for Maximum and Minimum Stages at St. Louis for 1971 and 1972

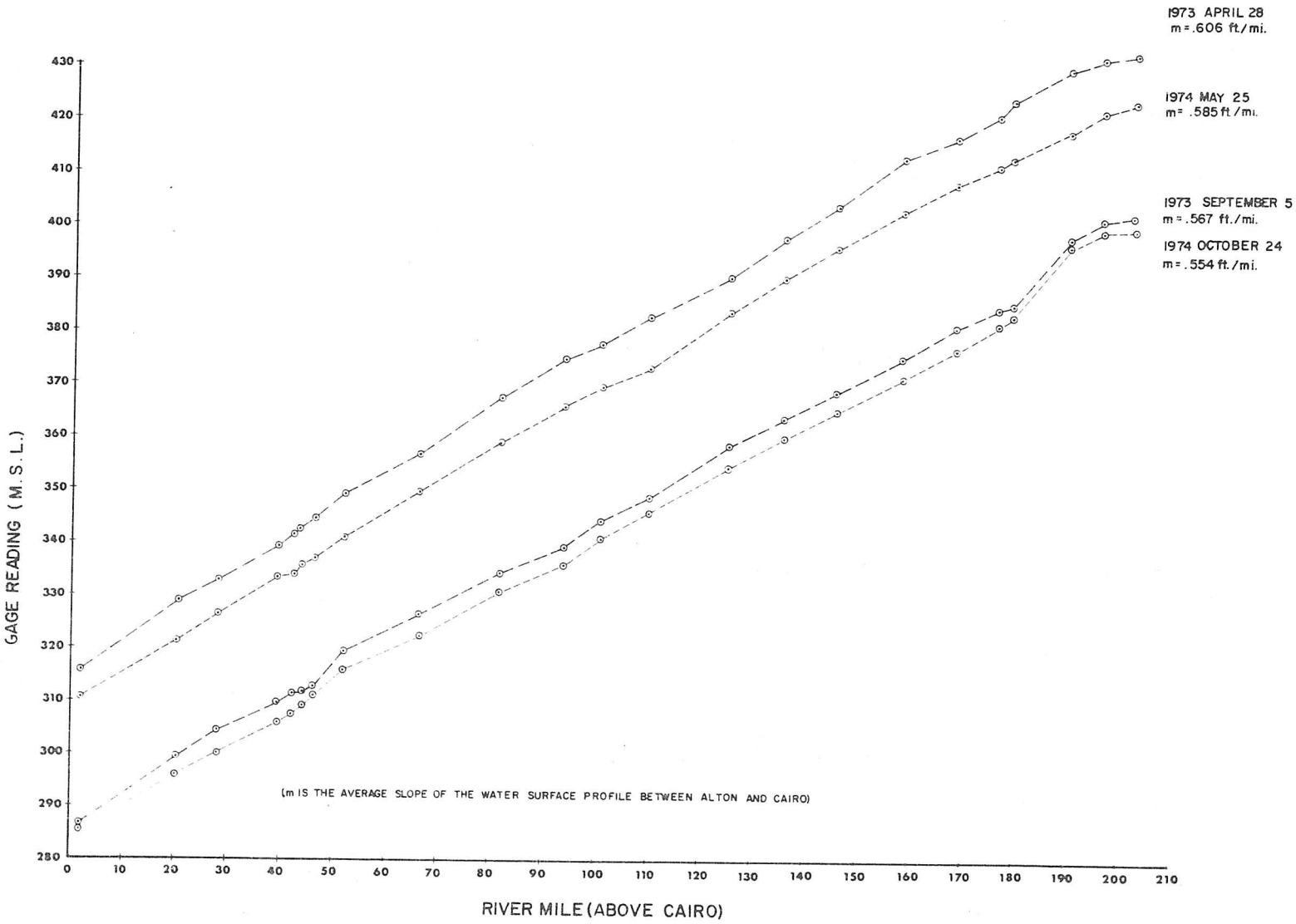

Figure 4.7 Surface Profiles of the Mississippi River Between Alton and Cairo for Maximum and Minimum Stages at St. Louis for 1973 and 1974

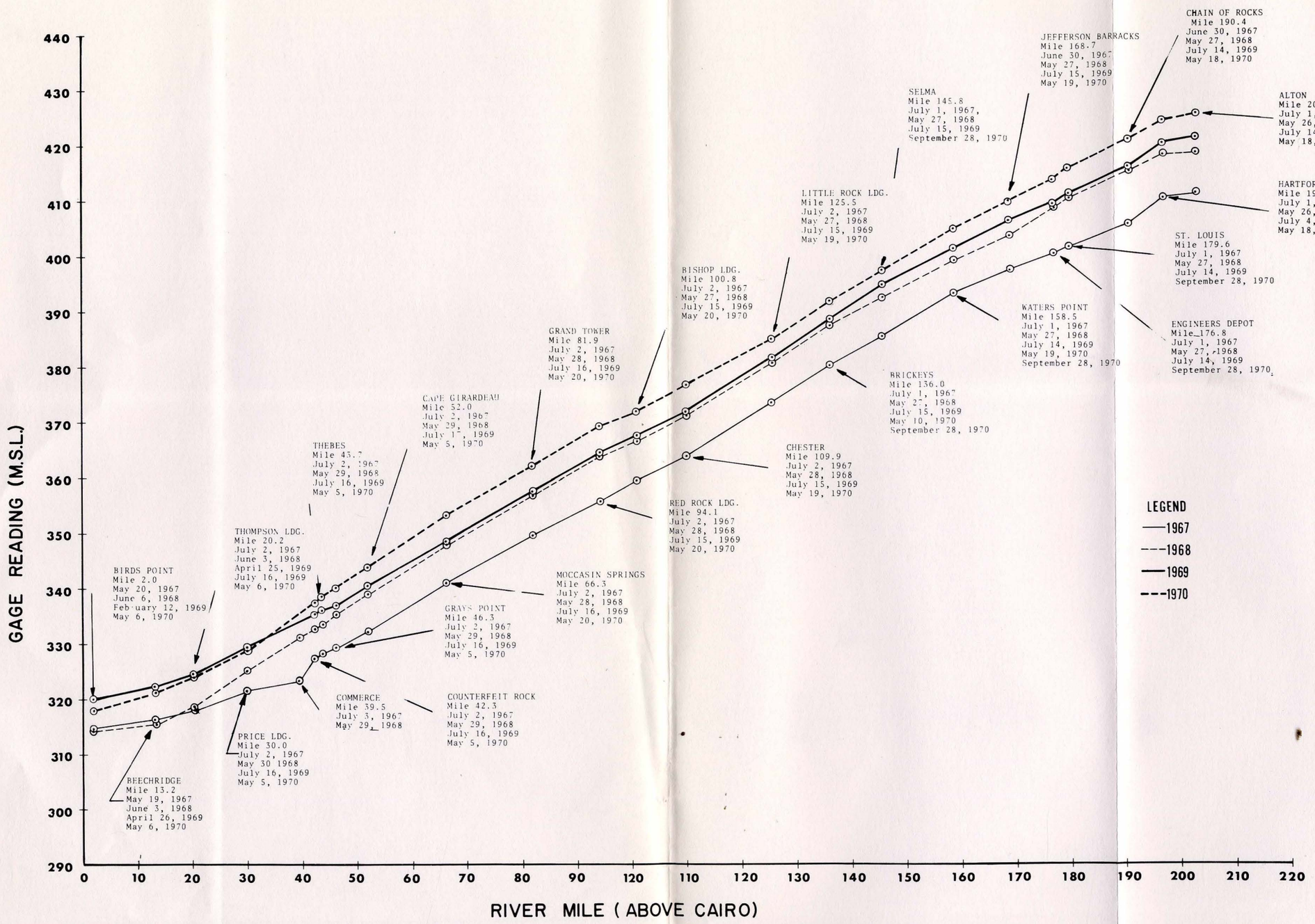

Figure 4.8 Surface Profiles of Mississippi River at Stations Between Alton and Cairo for Maximum Stages for 1967 Through 1970

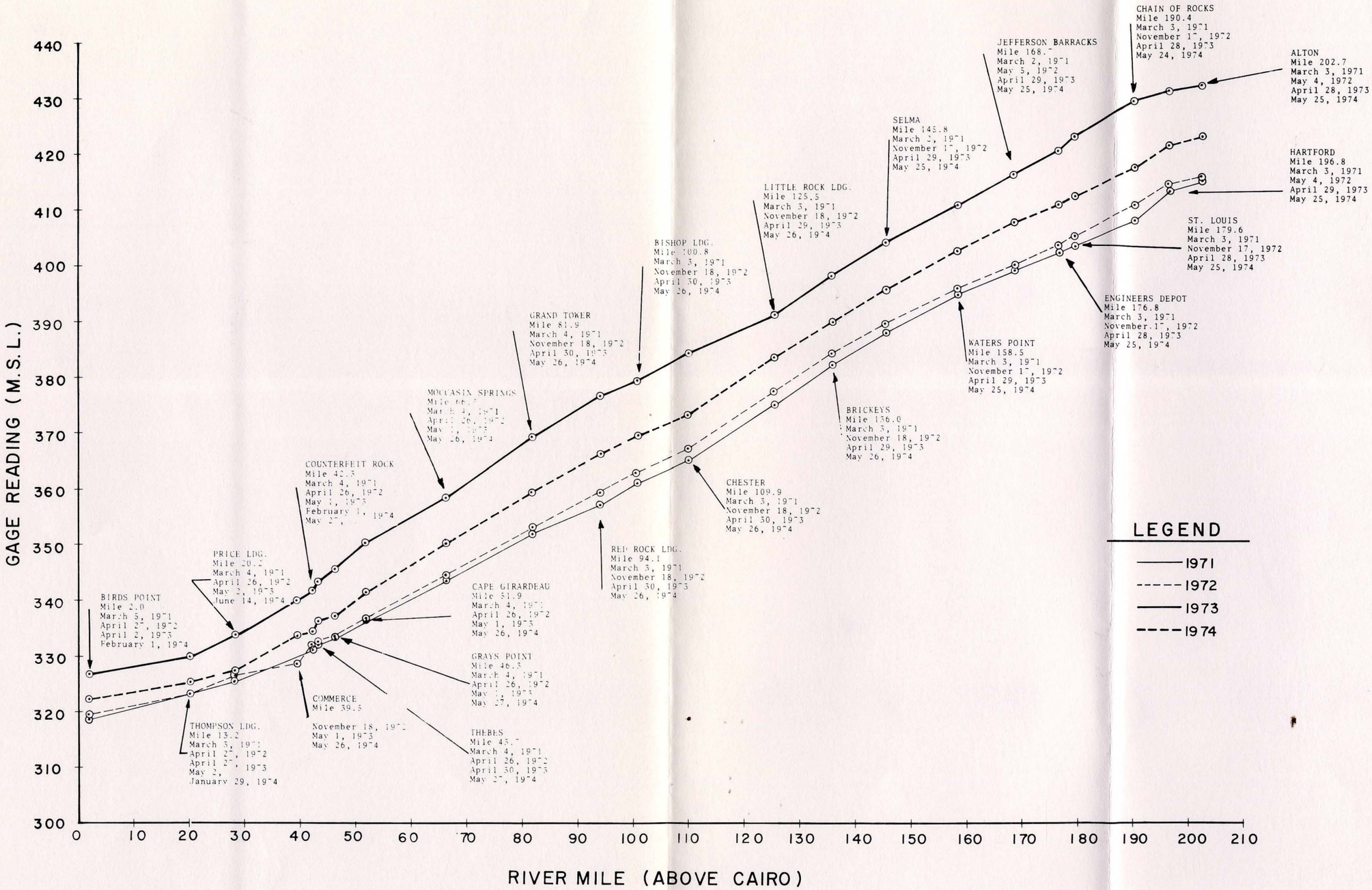

Figure 4.9 Surface Profiles of the Mississippi River at Stations Between Alton and Cairo for Maximum Stages for 1971 Through 1974

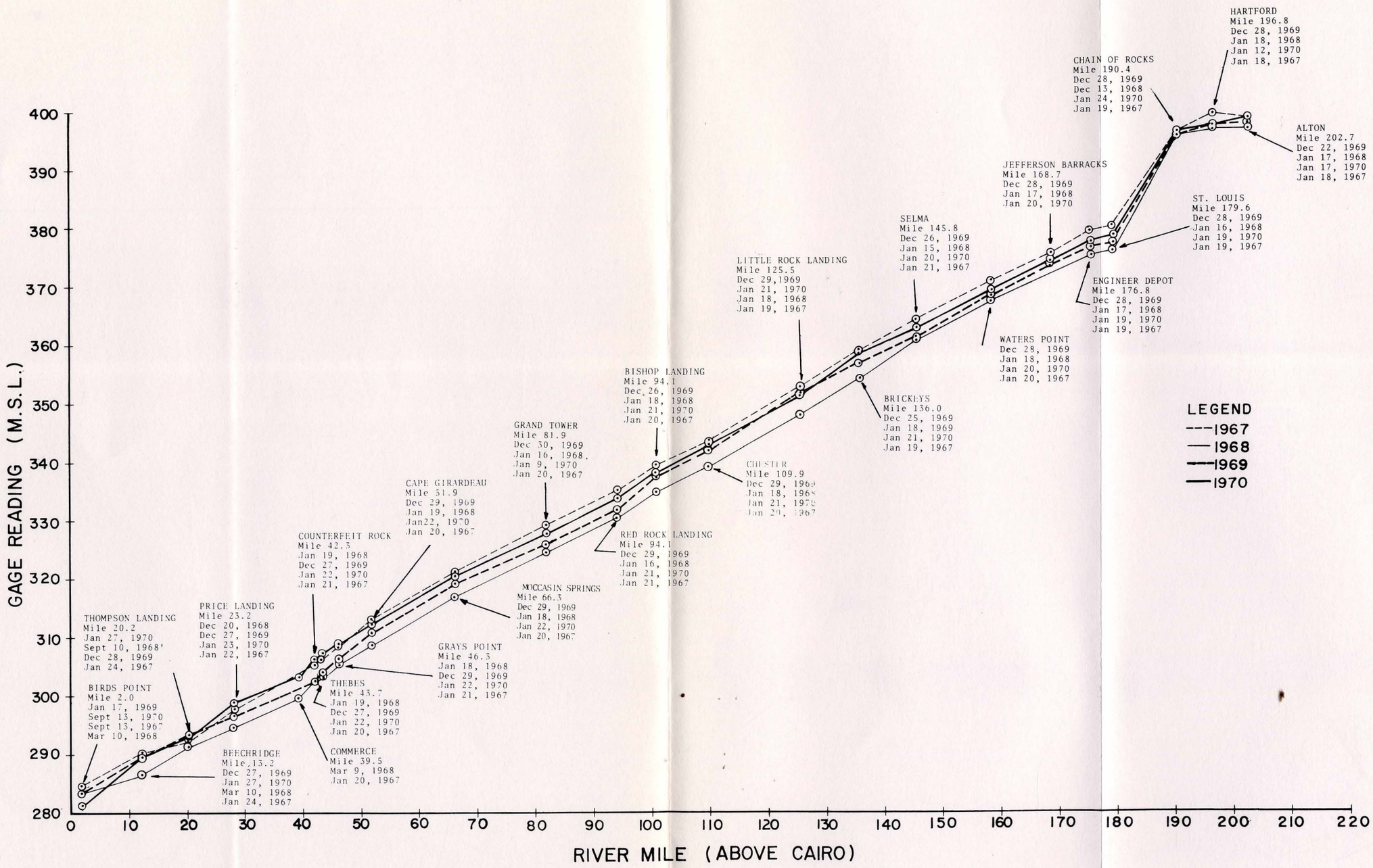

Figure 4.10 Surface Profiles of the Mississippi River at Stations Between Alton and Cairo for Minimum Stages for 1967 Through 1970

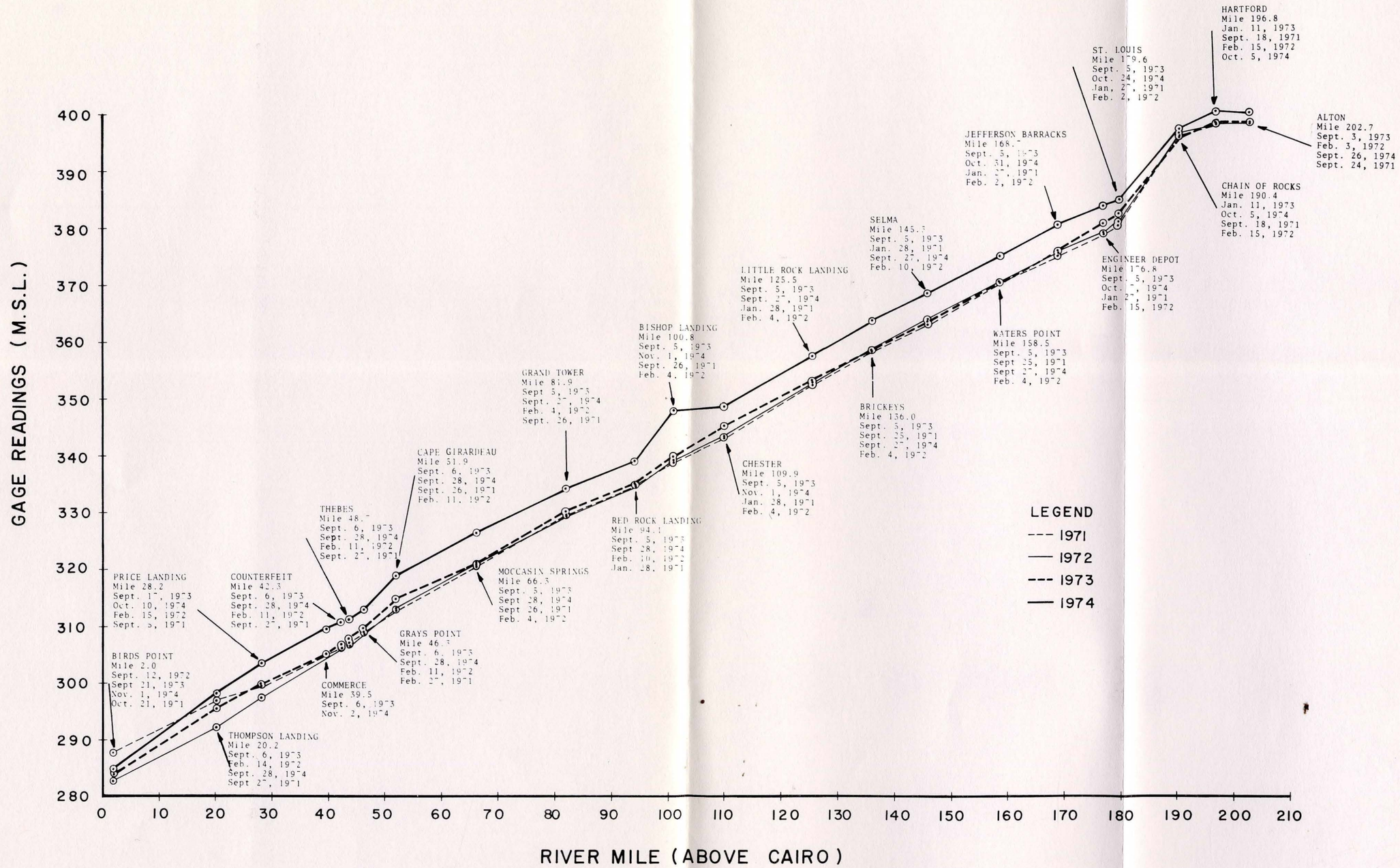

Figure 4.11 Surface Profiles of the Mississippi River at Stations Between Alton and Cairo for Minimum Stages for 1971 Through 1974

Table 4.4 Average Slope of Water-surface Profiles for High- and Low-flow Conditions in Selected River Reaches for Individual Years Between 1967 and 1974 and Average Slope for Each Flow Condition for the Period.

		1967	1968	1969	1970	1971	1972	1973	1974	AVE
Mississippi River from Alton, Illinois to Birds Point, Missouri	High	.574	.559	.582	.604	.519	.531	.588	.566	.565
	Low	.571	.471	.552	.555	.557	.530	.507	.589	.542
From Hermann on the Missouri River to St. Louis on the Mississippi River	High	.880	.878	.811	.830	.851	.866	.781	.810	.838
	Low	.948	.934	.941	.938	.929	.923	.928	.941	.935
Mississippi River from Hannibal, Missouri to Alton, Illinois	High	.317	.456	.402	.415	.473	.467	.402	.432	.421
Mississippi River from Keokuk, Iowa to Alton, Illinois	High	.426	.451	.413	.419	.470	.470	.415	N.A.	.438
From Meredosia on the Illinois River to Alton on the Mississippi River	High	.087	.216	.109	.204	.192	.198	.142	.178	.166

Mississippi Reach (Alton to Birds Point), whereas the situation is reversed in the Missouri River Reach because the St. Louis gage is below the mouth of the Missouri River and the Chain of Rocks topographic control.

The primary function of dikes has been stabilization of the navigation channel. To a lesser extent, they have been used to initiate chute closures and to create storage space for dredging spoils. The earliest record of dike construction in the Mississippi River between Alton, Illinois and Head of Passes, Louisiana is in 1834 when dikes were constructed near river mile 194 above Cairo, Illinois. Most dike construction has occurred between the mouth of the Missouri River and Cairo (St. Louis Reach), a distance of about 195 river miles. Between 1870 and 1900, dike construction in the St. Louis Reach amounted to about 300 dikes with a cumulative length of about 285,000 feet. During this period, most construction took place upstream from Crystal City, Missouri (river mile 149) and tended to concentrate opposite settlements. About 27,000 feet of the total for this period were built upstream from the present Market Street gage (mile 179.7) to keep the river next to St. Louis harbor facilities.

Between 1900 and 1924 only about 60,000 feet of dikes were added to the St. Louis Reach. As shown in Figure 4.12, accelerated construction activity began about 1925 and continued through 1940. During that period, about 469,000 feet were built to bring the cumulative length to about 766,000 feet. From 1940 to 1955, construction activity decreased markedly. Only 85,000 feet were added during this period. Rate of construction increased again between 1955 and 1970. Beginning in 1970, construction of new dikes decreased by more than an order of magnitude from

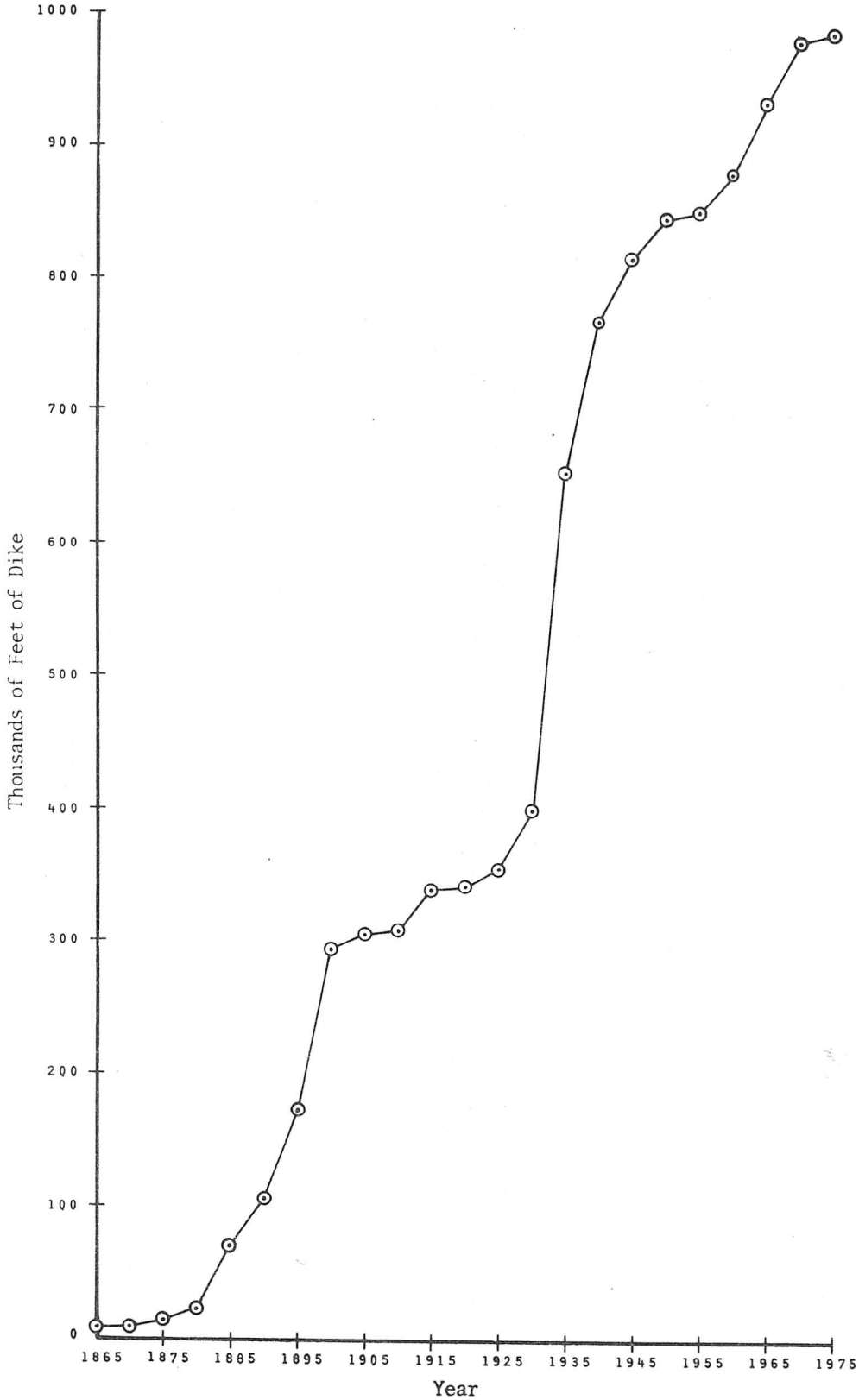

Figure 4.12. Cumulative Length of Dikes Constructed in St. Louis Reach, 1865-1975

the previous 5-year period. By 1975, about 1100 dikes with a cumulative length of 982,000 feet had been built in the St. Louis Reach.

Figure 4.13a shows the average top-bank widths of the St. Louis Reach as scaled from hydrographic surveys and aerial photographs at intervals of about 2 miles for selected years between 1908 and 1974. Top-bank width was taken as the distance between first vegetation. The average width decreased from about 3320 feet in 1908 to about 2370 feet in 1974, a decrease of about 29 percent. However, it should not be inferred from Figure 4.13a that the rate of decrease was necessarily uniform. There are insufficient determinations to describe the time distribution of change. It is clear that river width in this reach has decreased in response to dike construction activity.

As shown in Figure 4.14 for the reach between the White River and Cairo, Illinois (Memphis Reach; river mile 596 to 954 above Head of Passes), although dike construction was initiated in 1900 there was relatively little construction activity until 1956. In 1955 there were about 11,000 feet of dikes. Between 1956 and 1974, approximately 222 dikes with a cumulative total length of 505,000 feet were added. Figure 4.13b shows average top-bank width of the Memphis Reach for selected years. Unlike the St. Louis Reach, there is no obvious relationship between average top-bank width and dike construction for the Memphis Reach.

In the reach between Old River Structure and the White River (Vicksburg Reach; river mile 321 to 596), records show a construction date for only one dike prior to 1962. Hydrographic surveys show a limited number of dikes prior to 1962 but construction dates are not indicated. Between 1962 and 1974, there were 123 dikes built with a cumulative

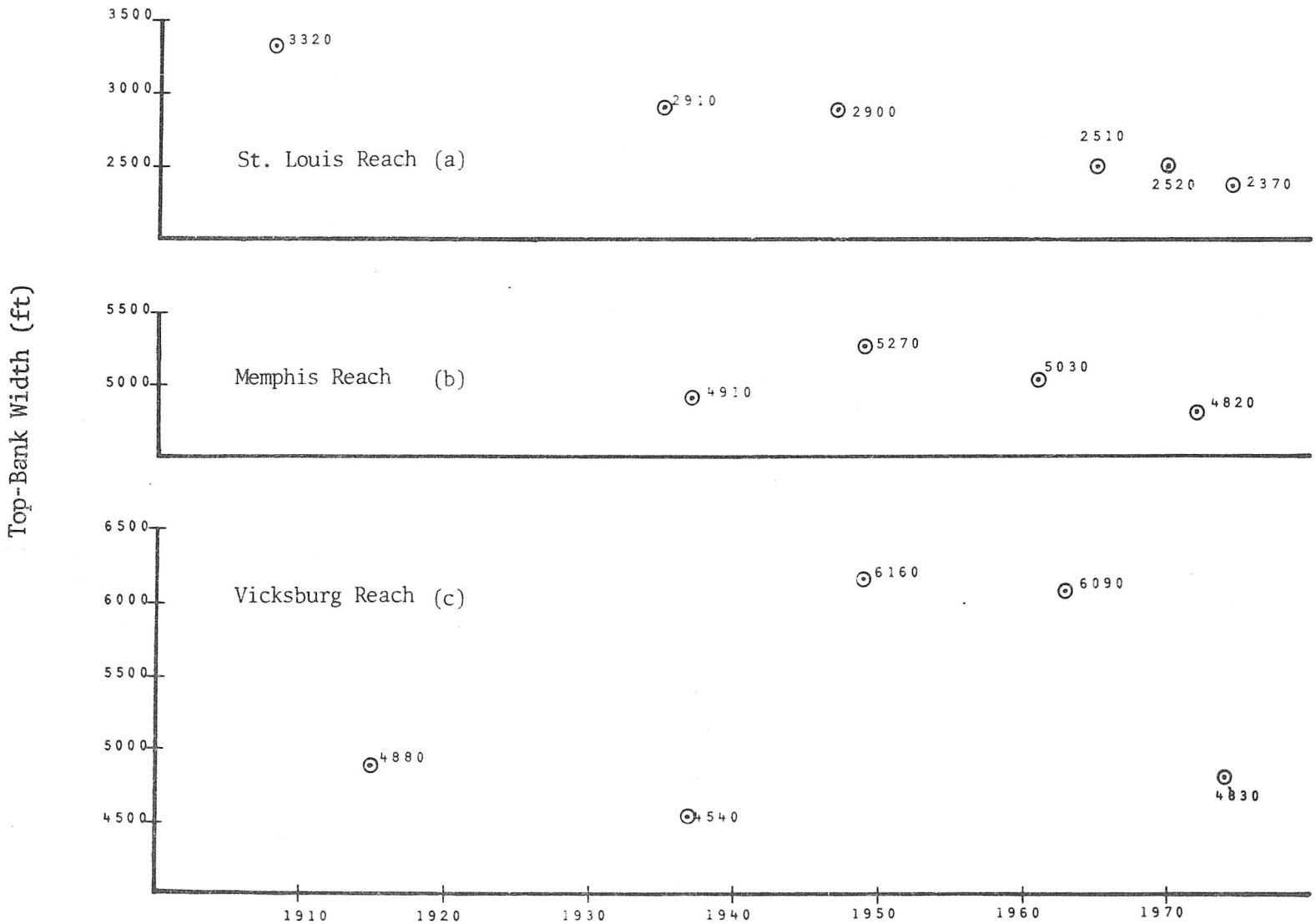

Figure 4.13 - Average Top-Bank Widths of the Mississippi River

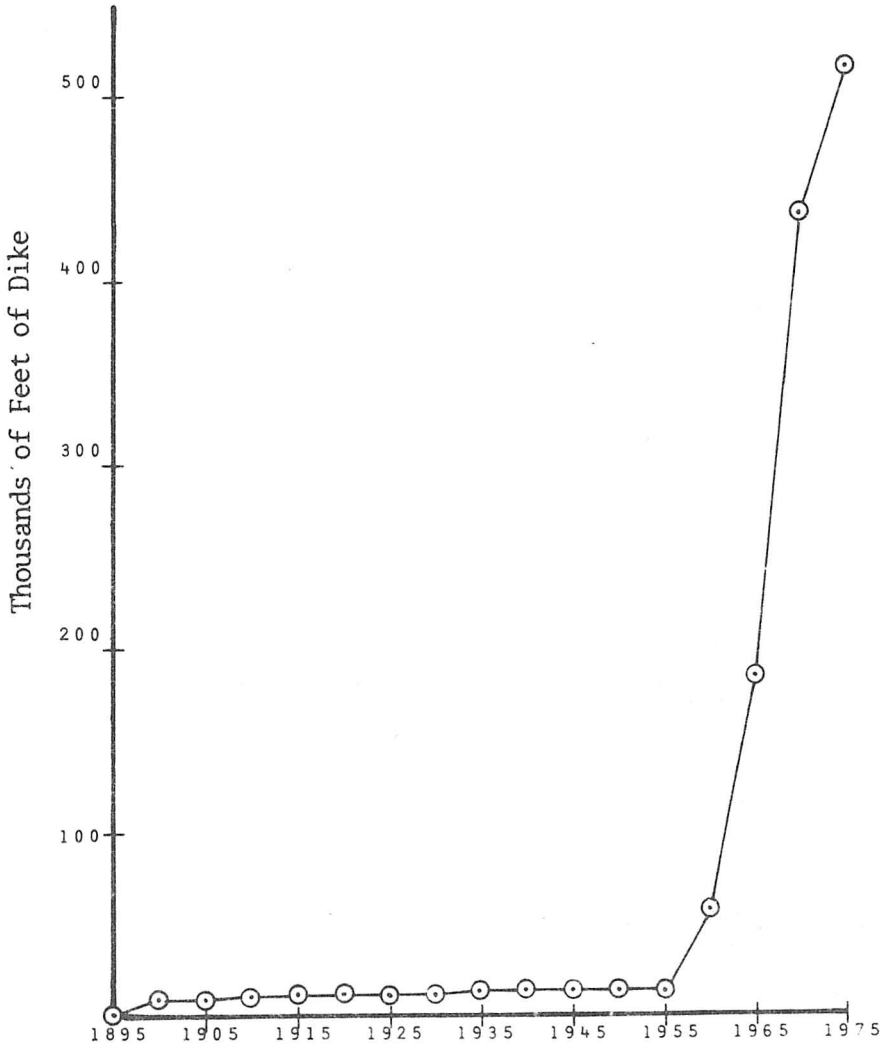

Figure 4.14 - Cumulative Length of Dikes Constructed in the Memphis Reach, 1900-1975

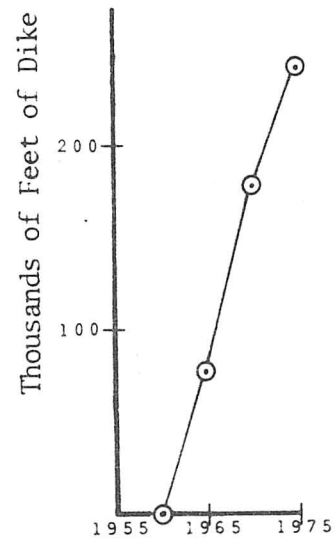

Figure 4.15 - Cumlative Length of Dikes Constructed in the Vicksburg Reach, 1955-1975

length of about 243,000 feet. Figure 4.15 shows the cumulative length of dikes constructed in this reach between 1955 and 1975. Figure 4.13c shows average top-bank widths for Vicksburg Reach for selected years. The decrease between 1963 and 1974 is nearly as large as the increase between 1937 and 1959. Dike construction in the St. Louis Reach was about 5030 feet/mile, in the Memphis Reach about 1400 feet/mile, and in the Vicksburg Reach about 890 feet/mile. In the Memphis Reach, average top-bank width is about twice and dike density in terms of feet per mile is only about one-fifth that in the St. Louis Reach. Therefore, a causal relationship between dike construction and changes in top-bank width in the Memphis Reach should not be inferred without further analysis. Because cumulative length of dikes per mile in the Vicksburg Reach was less than in the Memphis Reach and large bank-width changes have been observed both prior to and after dike construction, any association between dike construction and reduction of top-bank width in the Vicksburg Reach after 1963 is probably unwarranted at this time.

Because dikes constrict the channel (at least in the vicinity of individual dikes) it was necessary to determine if there have been associated changes in stage-discharge relationships over time. The analytical procedure was the same for all stations. The mean-daily discharges were plotted against mean-daily stages for every year of continuous record. Plots were on both arithmetic and logarithmic coordinates. The "average" stage-discharge curve for each year was estimated for fitting a smooth curve to the data by eye. Because the plotted points were approximately linear on the logarithmic coordinates, an estimated straight line of best fit was used. Flow rates were selected to be

representative of low, medium bankfull, and overbank conditions. These fixed flow rates were used in conjunction with the stage-discharge curves to estimate prevailing stage for each flow rate for each year. Stages were then plotted as a function of time.

There was a number of years when bankfull conditions were not realized. Stages for these years were extrapolated from the logarithmic plots for the year in question. This method nearly always resulted in reasonable values of stage for bankfull flow conditions. For flows greater than bankfull, the difference between estimated stages was often 4 feet or more for adjacent years. This was particularly evident when extrapolations were made from rating curves developed for drought years. Because of the extreme variability of above bankfull estimates and because there are relatively few observations above bankfull to use as controls, it was not possible to make an evaluation of changes in flood stages with respect to time as influenced by levee construction. Therefore, no inferences were drawn with respect to effects of levee construction on stage-discharge relationships.

The procedure outlined above was followed for the following stations:

1. Hermann on the Missouri River
2. St. Louis
3. Chester
4. Thebes
5. Metropolis on the Ohio River
6. Memphis
7. Vicksburg

Continuous records were not kept for the above stations until 1930 or later. For that reason and because flow-measuring techniques were not standardized for earlier years, the principal part of the stage-change analysis is based on the post-1930 record.

Figure 4.16 shows stage versus time for selected flows in the Missouri River at Hermann, Missouri. Flood stage at Hermann is 21 feet. Discharge at flood stage is approximately 211,800 cfs. Stages were not estimated for overbank conditions unless an observed flow was greater than 211,800 cfs. For the period, 1930 to 1974, it appears that stages have increased for all flows between 64,700 cfs and 337,000 cfs. Stage increases have ranged from about 2 feet for 64,700 cfs to about 3.5 feet for 337,000 cfs. It was not within the scope of this project to collect data which might relate to stage behavior at this station.

Figure 4.17 shows stage versus time for selected flows in the Mississippi River at St. Louis, Missouri. It is clear from the plotted data that, since 1934, stages have decreased for flows less than 280,900 cfs. With the statistic Z at the 2-percent level of significance as a test criterion, a downtrend with time was also found to exist for bank-full conditions (501,300 cfs). It appears that since 1934, stages have decreased about 1.4 feet for all flows between 501,300 cfs and 154,800 cfs.

Prior to 1934, relatively few flow measurements were made at the St. Louis gage. For those years when there were sufficient data to define a stage-discharge relationship, it was possible to estimate stages corresponding to the same flow rates which were used for the post-1934 stage-change analysis (Fig. 4.17). Those estimates are shown in the following table:

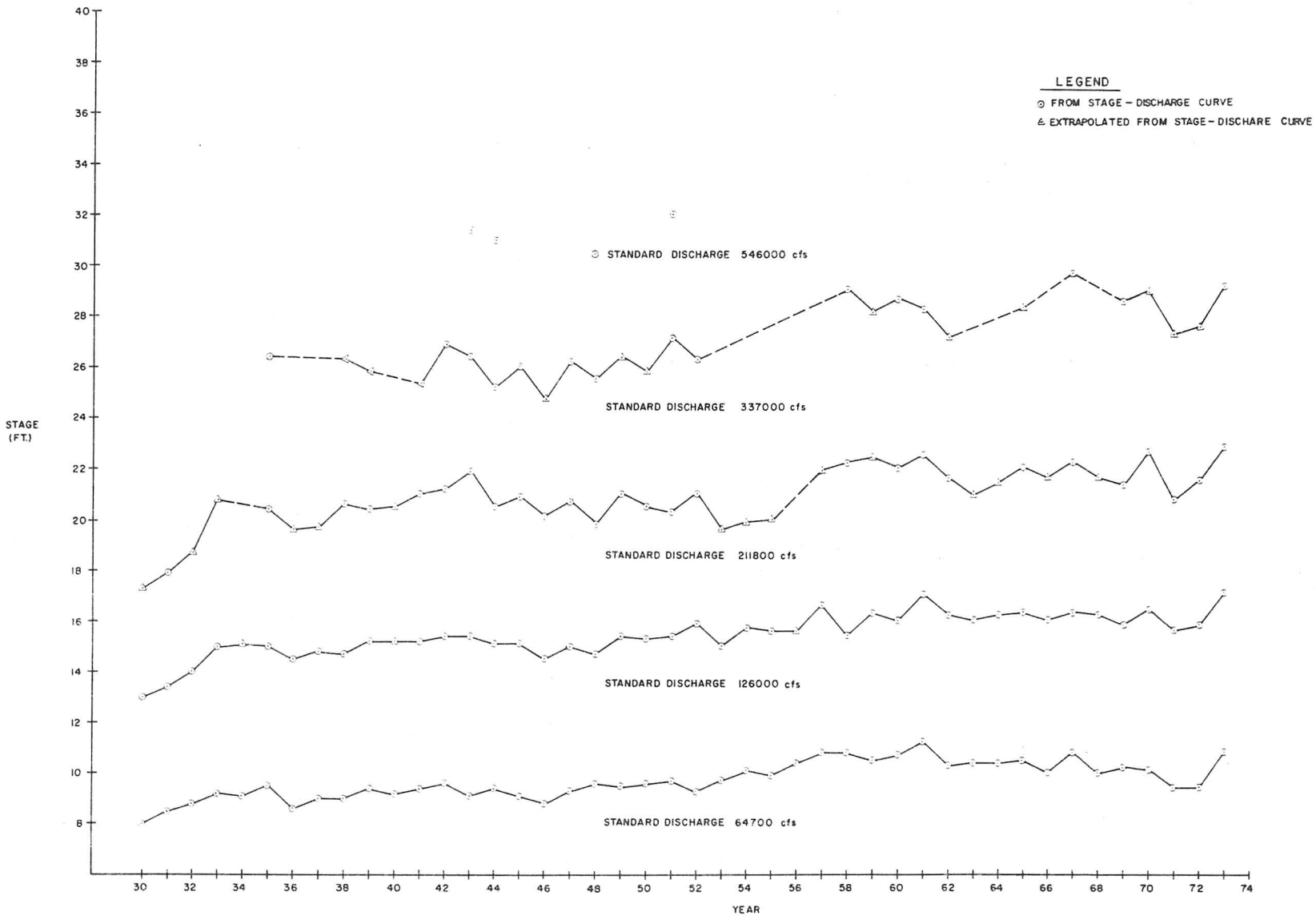

Figure 4 16 Estimated River Stage for Selected Flows in the Missouri River at Hermann Between 1930 and 1974

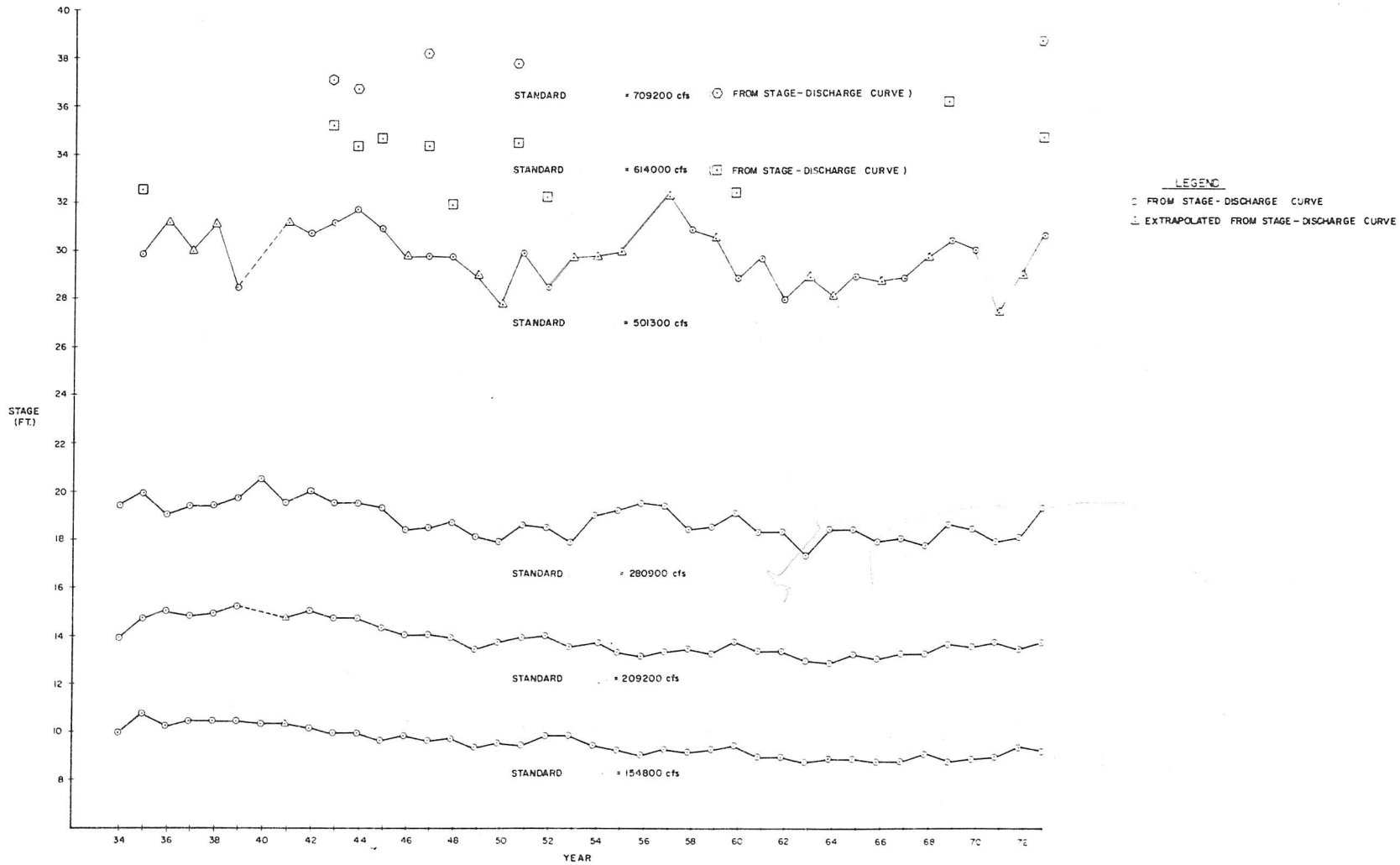

Figure 4.17 Estimated River Stage for Selected Flows in the Mississippi River at St. Louis Between 1934 and 1973

Discharge (cfs)	1881	1900	1903	1904	1934
	(Stages are in feet)				
501,300	25.2	26.5	27.8	28.5	29.9
280,900	18.0	18.7	20.0		20.0
209,200	15.1	15.4	16.0		14.8
154,800	11.5	12.5	12.2		10.8

The indication is that, for flows greater than 280,900 cfs, stages increased during the period from 1881 to 1934. During the same period, flows less than about 209,200 cfs probably passed the St. Louis gage at progressively lower stages.

Prior to about 1930, velocities were measured with a variety of equipment and in accordance with a variety of field procedures. Methods for calculating flow rates from velocity and sounding data were not standardized. Although equipment and field and calculation procedures are now standardized, the U.S. Geological Survey still occasionally describes accuracy of their published mean daily discharge figures at St. Louis as "good". This classification means that 95 percent of the mean daily flows are within 10 percent of the true value. If pre-1934 flows were determined according to contemporary techniques, they would probably differ from those shown in the preceding table. It seems reasonable to expect that the difference could be 20 percent or more.

If flows measured prior to 1934 are in error, then stages which correspond to flows shown in the preceding table are in error. As discussed previously, plots of mean daily stage versus mean daily discharge for each year after 1934 were nearly linear on logarithmic coordinates. A straight line fitted to these data implies an approximate exponential relationship between stage and flow of the form

$$S = MQ^n$$

where S = stage,

Q = flow rate,

n = slope of the straight line fitted to a plot of the logarithm of stage versus the logarithm of discharge, and

M = a constant (the stage which prevails when the flow rate is unity).

If the exponential relationship between stage and discharge is assumed to be a valid approximation, the stage-estimate error resulting from a flow measurement error can be expressed as

$$E_s = \{(\frac{Q_m}{Q_t})^n - 1\} \times 100$$

where E_s = percent error in stage estimate,

Q_m = measured flow rate,

Q_t = true flow rate, and

n = exponent as determined from the stage-discharge relationship.

Exponents (n) for the post-1934 years range from 0.42 to 0.51. If the same general stage-discharge relation is assumed to hold for the pre-1934 period and if the flow measurement error is taken to be 20 percent (Q_m/Q_t = 0.8 or 1.2), then the percent error in stage estimate (E_s) can be shown to be approximately ±10 percent. Therefore, if flow measurements prior to 1934 were systematically 20 percent too high, estimated stages shown in the previous table should be increased by about 10 percent, whereas the converse is true if measured flows were less than the true flows. The following tables show stage-change trends

as they might appear if the pre-1934 measurements were 20 percent too high and 20 percent too low, respectively.

For Q_m/Q_t = 1.2 (Measured flows greater than the true value):

Discharge (cfs)	1881	1900	1903	1904	1934
	(Stages are in feet)				
501,300	27.7	29.2	30.6	31.4	29.9
280,900	19.8	20.6	22.0		20.0
209,200	16.6	16.9	17.6		14.8
154,800	12.7	13.8	13.4		10.8

For Q_m/Q_t = 0.8 (Measured flows less than the true value):

Discharge (cfs)	1881	1900	1903	1904	1934
	(Stages are in feet)				
501,300	22.7	23.8	25.0	25.6	29.9
280,900	16.2	16.9	18.0		20.0
209,200	13.6	13.9	14.4		14.8
154,800	10.3	11.2	11.0		10.8

Figures in the two preceding tables demonstrate that differences between early and contemporary flow determinations (if they exist) would lead to conclusions about direction and rate of change of stage between 1881 and 1934 which are different from those indicated by the existing record. Therefore, unqualified acceptance of pre-1934 stage-change behavior is not justifiable without corroborating evidence that early and contemporary flow determinations are reasonably comparable.

Figure 4.18 shows stage versus time for selected flows in the Mississippi River at Chester, Illinois. Flood stage at Chester is 27.0 feet which corresponds to a flow of about 440,000 cfs. With the statistic Z at the 20-percent level of significance as a test criterion, there

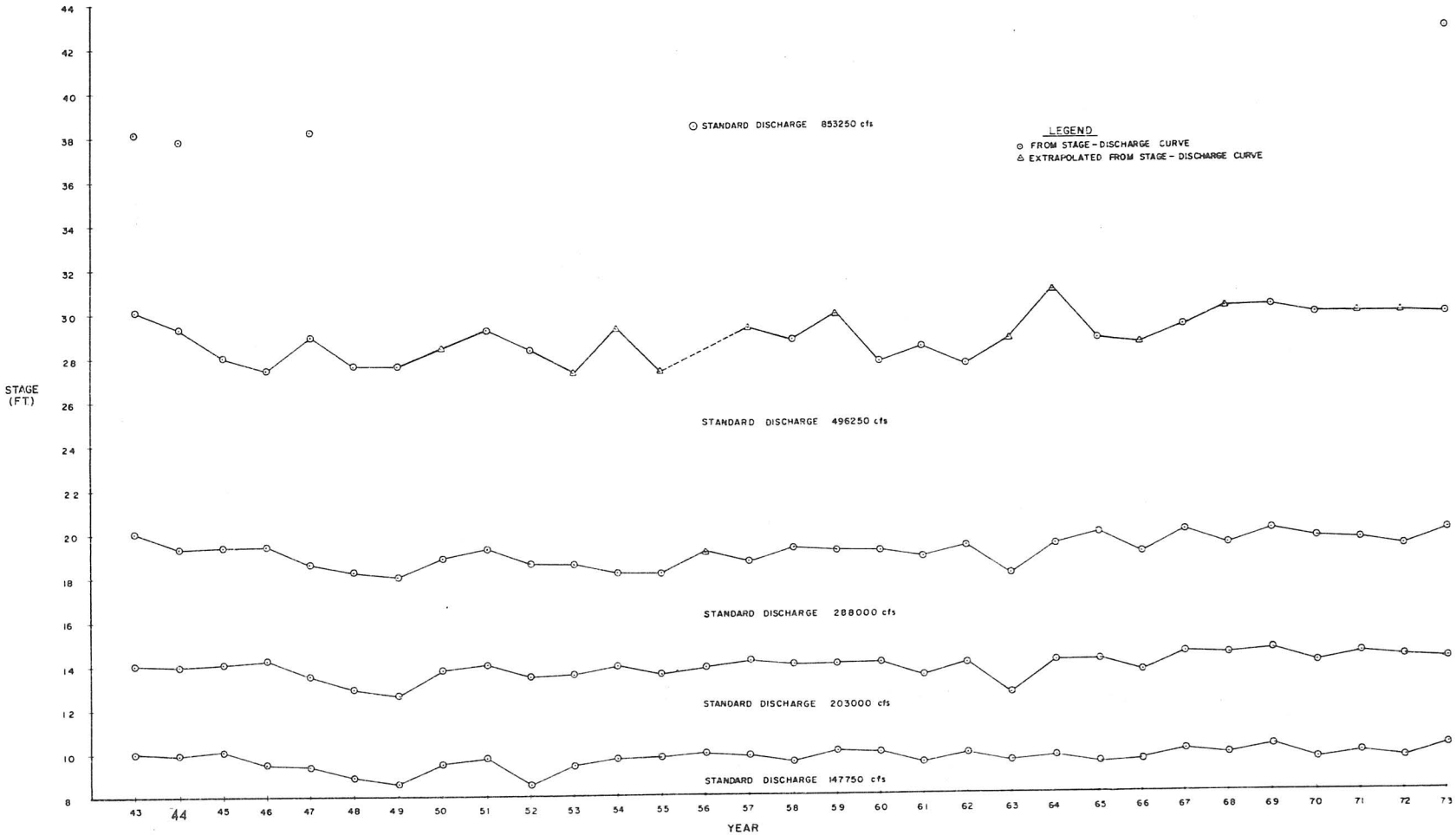

Figure 4-18 Estimated River Stage for Selected Flows in the Mississippi River at Chester Between 1943 and 1973

were no discernible trends in stages for flows less than 496,250 cfs for the period, 1943 through 1973.

Figure 4.19 shows stage versus time for selected flows representing low, mid-range, and bankfull conditions in the Mississippi River at Thebes, Illinois. Stages for low flows appear to have decreased between 1934 and 1973. For instance, with the statistic Z at the 1-percent level of significance as a test criterion, stages for flows between 87,000 cfs and 135,000 cfs decreased about 3 feet and 3.5 feet, respectively, between 1934 and 1973. However, stages for mid-range flows (about 219,000 cfs) remained practically unchanged over the same period.

Flood stage at Thebes is about 33.0 feet. In 1941 a stage of 33.0 feet corresponded to a flow of nearly 572,000 cfs,whereas by 1973 the same stage corresponded to a stage of about 481,000 cfs. Although it appears that the bankfull capacity has decreased since 1934, the decline has not been continuous. From 1934 to about 1963, stages for flows between 481,000 cfs and 572,000 cfs remained essentially unchanged. Because stages for mid-range flows also remained nearly constant, it may be inferred that stages for all intervening flows from about 219,000 cfs to about 572,000 cfs remained relatively stable over the period. However, between 1963 and 1973, stages for the approximate bankfull condition (481,000 cfs to 572,000 cfs) increased steadily to about 2.5 feet over the 1962 condition. Therefore, it appears that for the post-1963 period, stages for mid-range flows remained constant while stages increased for those higher flows near the bankfull condition.

Because of backwater effect from the Ohio River it is common to find 2 feet or more variation in stage at Thebes for discharges greater

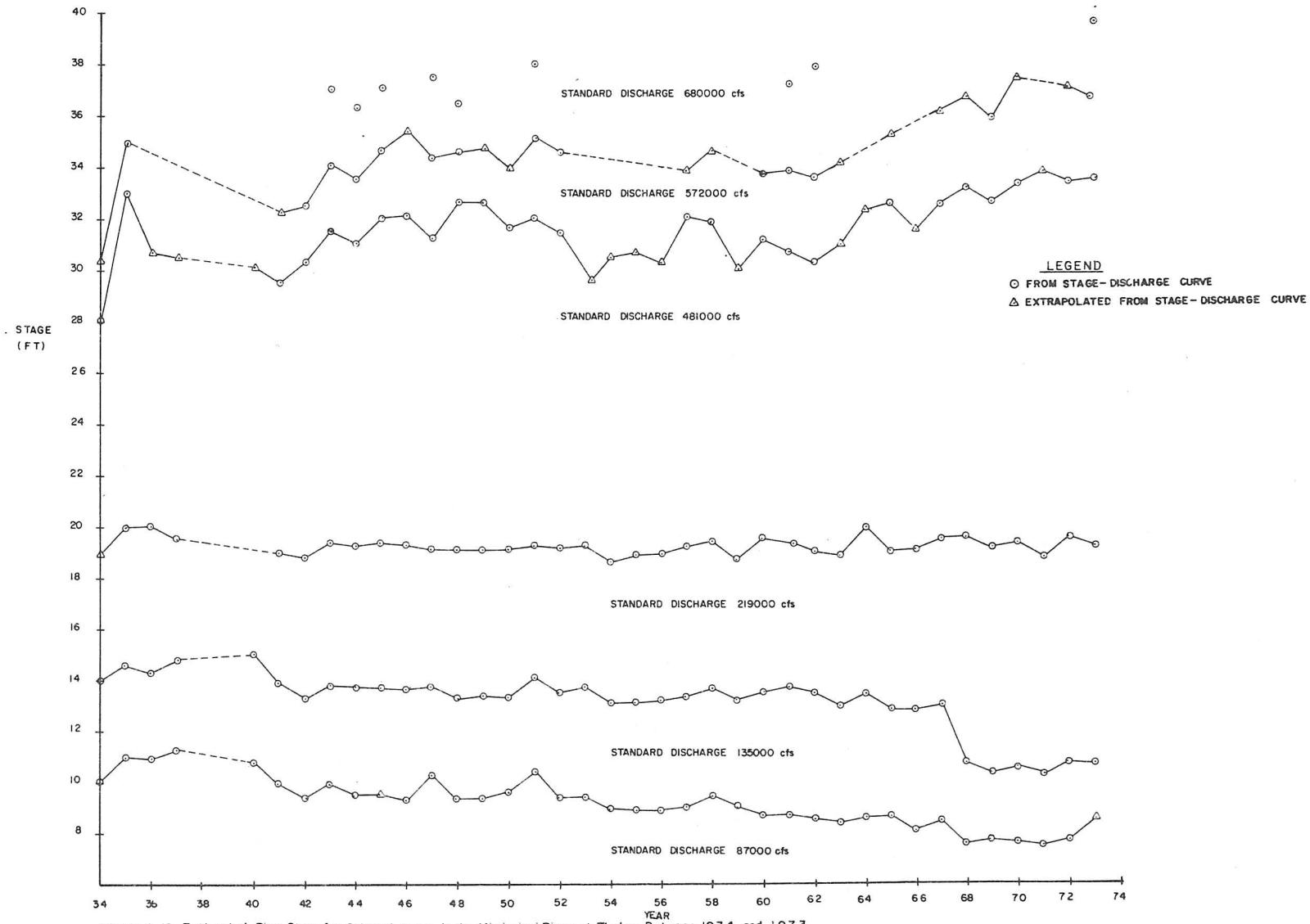

Figure 4.19 Estimated River Stage for Selected Flows in the Mississippi River at Thebes Between 1934 and 1973

than 431,000 cfs. For this reason, and because pertinent flow data for the period prior to 1934 are few, existing stage-discharge data are insufficient for a comparison of pre-1934 and post-1934 stages for flows considered herein.

Figure 4.20 shows stage versus time for selected flows of the Ohio River near Metropolis, Illinois. Flood stage is 43 feet. The pattern is very similar to that of the Mississippi River at Thebes. As at Thebes, stages for low flows (66,600 cfs) have decreased and stages for flows about midway between bankfull and low flows have remained about the same for the period, 1936 through 1973. However, stages for flows near bankfull (774,000 cfs) appear to have increased nearly uniformly since 1936, whereas the corresponding increase at Thebes began about 1963.

Because mean annual flow from the Ohio River is about 46 percent greater than that in the Mississippi River at Thebes, it is possible that those factors which cause changes in the stage-discharge relation in the Ohio River near its confluence with the Mississippi River also will be reflected by similar changes in the stage-discharge relation at Thebes. This speculation is based on similarities in stage-behavior patterns between Metropolis and Thebes. In order to assess the validity of such a possibility, it would be necessary 1) to determine which factors are associated with the stage-behavior pattern at Metropolis, 2) to determine if the observed patterns are representative of the intervening reaches between Metropolis and Thebes and the confluence of the Ohio and Mississippi Rivers, and 3) to analyze the relative time distribution of flow and sediment load from the Mississippi and Ohio Rivers past their confluence.

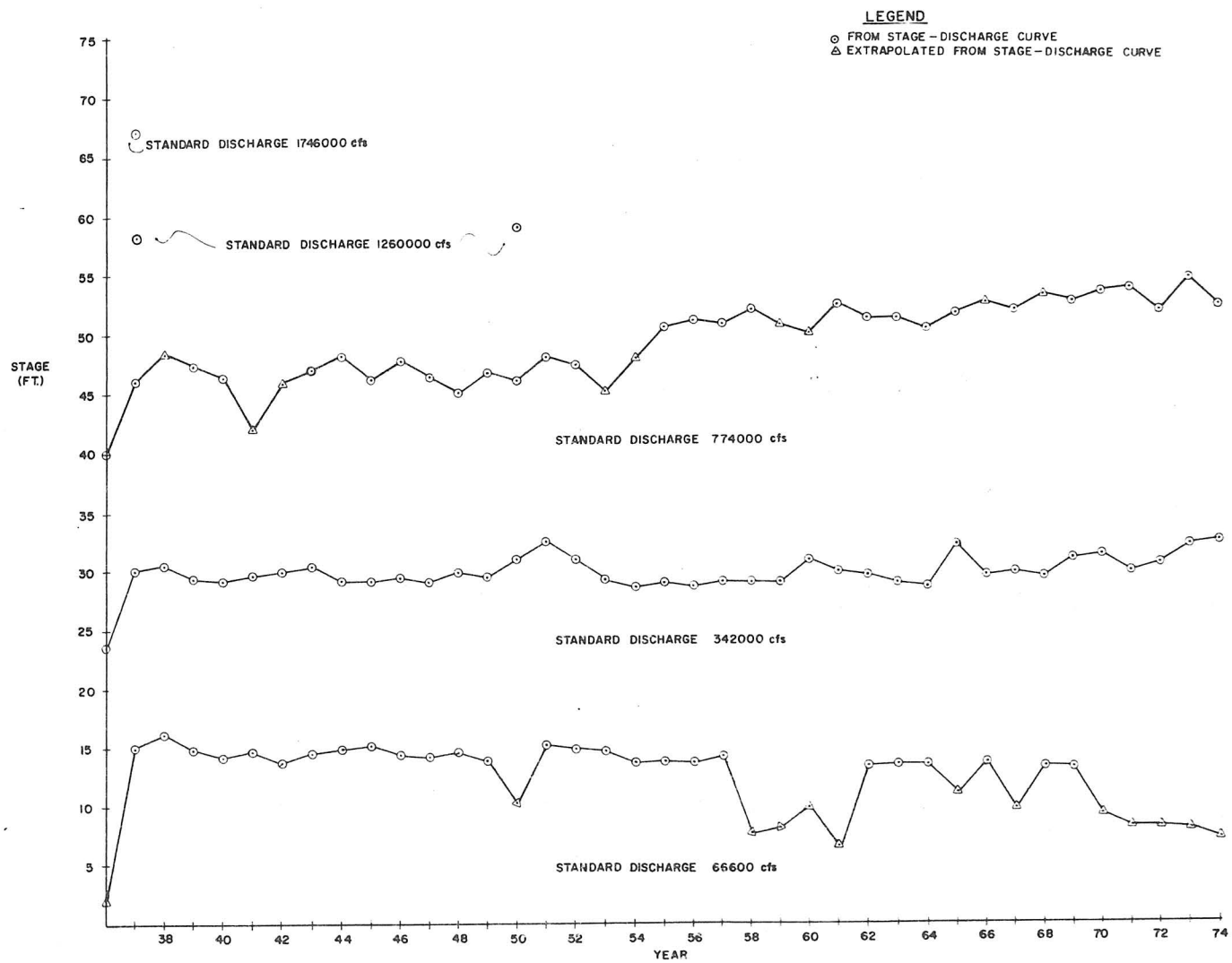

Figure 4.20 Estimated River Stage for Selected Flows in the Ohio River at Metropolis Between 1936 and 1974

Figure 4.21 shows stages versus time for selected flows in the Mississippi River near Memphis, Tennessee during the period, 1933 through 1971. Flood stage at Memphis is 34 feet. Stages for all flows below bankfull have continuously declined since 1933. Stage declines for the period, 1933 through 1971, range from about 3 feet for flows of 1,070,000 cfs to about 6 feet for flows of 260,000 cfs. Records show that in 1890 a flow of 1,070,000 cfs passed Memphis with a stage of 31.5 feet. For that flow rate the 1890 stage was greater than the 1933 stage, but it was less than the 1971 stage. Therefore, it appears that even though there was a decrease in channel capacity, it was temporary. Higher flow rates could be sustained within banks at the Memphis gage in 1971 than could be sustained in 1890.

Figure 4.22 shows stages versus time for selected flows in the Mississippi River near Vicksburg, Mississippi. Flood stage at Vicksburg is 43 feet. All stages for flows near bankfull and below have a similar behavior pattern. In the period, 1931 to about 1942, stages for flows of 1,340,000 cfs or less declined approximately 10 feet. Prior to 1931, stages for 1,340,000 cfs range from estimated extremes of 52 feet in 1858 to 46.9 feet in 1909. In 1913, the stage for 1,340,000 cfs was slightly greater than 48 feet, whereas in 1927 and 1929 it varied between 49 feet and 52 feet. Apparently, whatever factors may have caused the downtrend in stages between 1931 and 1942 manifested themselves after 1931. Beginning in 1942, there was a trend reversal showing steady increase in stages. By 1972 stages for flows near bankfull and below had recovered between 4 feet and 5 feet from the 1942 condition.

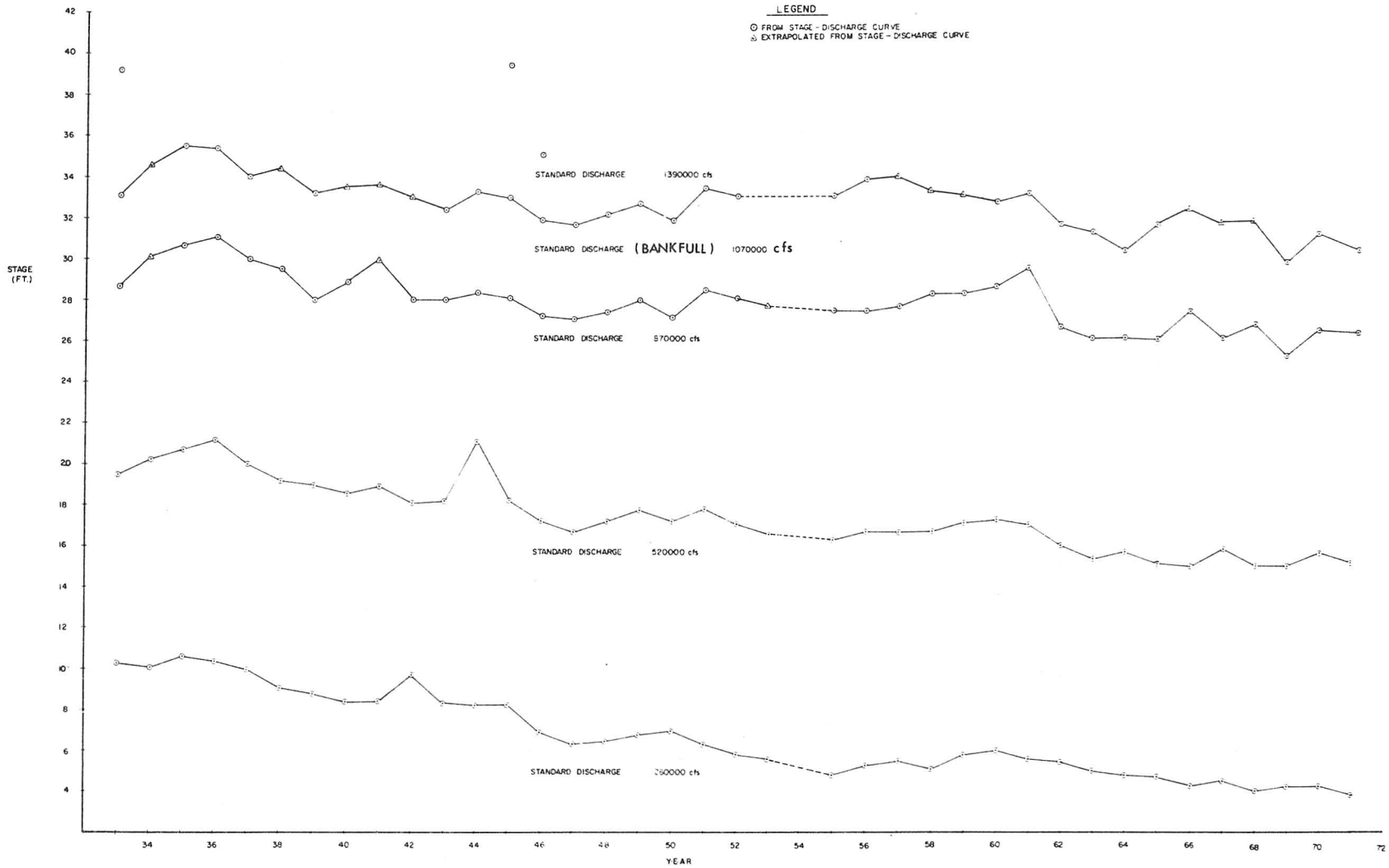

Figure 4.21 Estimated River Stage for Selected Flows in the Mississippi River at Memphis Between 1933 and 1971

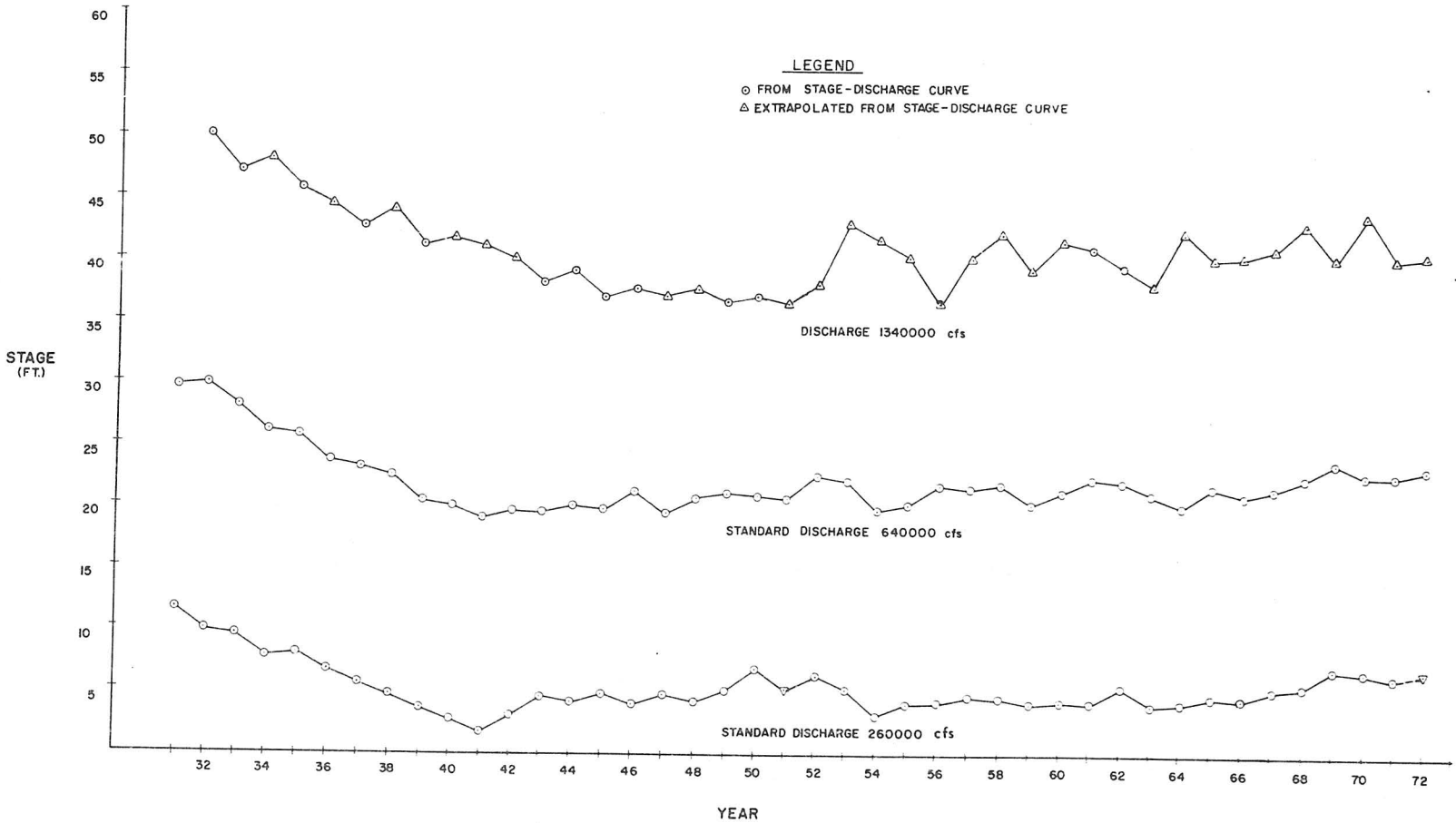

Figure 4.22 Estimated River Stage for Selected Flows in the Mississippi River at Vicksburg Between 1931 and 1972

St. Louis, Chester, and Thebes gages are all in the St. Louis Reach which has received most of the dike construction effort on the Mississippi River. Although average top-bank width appears to have decreased in response to dike construction activity by about 29 percent since 1908, the change in stage-discharge relations with time is different at each of the stations. In the Memphis Reach where dike construction activity has been only about 28 percent of that in the St. Louis Reach (in terms of length of dike per mile), changes in stage-discharge relations with time are similar to those observed at St. Louis but much more pronounced. In the Vicksburg Reach where dike construction was only about 18 percent of that in the St. Louis Reach (in terms of length of dike per mile), changes in stage-discharge relations with respect to time show an initial downtrend in stage for any given flow (less than 1,340,000 cfs) followed by an uptrend or rebound at about one-half the rate of the initial downtrend. Neither trend appears to be associated with dike construction activities in the reach.

The present analysis suggests that generalizations about effect of dikes on stage-discharge relations are not justified. It appears that other important, but so far unidentified, factors also influence stage-discharge relations. The Ohio River-Mississippi River responses near their confluence may be an example of mutual interference wherein modifications in one stream may cause a response in both. Because of the diversity in stage-discharge responses between stations, it appears that localized conditions may be the most important influence on these relations at individual stations. If this is so, then it is unlikely that an individual point of record is representative of conditions throughout a reach.

B. Floodplains

A floodplain is that portion of a river valley adjacent to the river channel which is built of sediments during the present regime of the stream and which is covered with water when the river overflows its banks at flood stages. All floodplains are level, or almost level, plains of low topographic situation existing at the level of (or slightly above or below) an adjacent parent stream. They are characterized by distinctive remnants of erosion and deposition. The type of floodplain is dependent upon the dominant type of flood which the stream experiences and on whether the stream has predominantly single- or multiple-channel flow. Floodplains associated with streams of single-channel flow are known as meander floodplains, covered floodplains, or composite floodplains. Those associated with streams of multiple-channel flow are bar meander or bar plains.

Meander floodplains are created as a result of "bankfull" floods, where there is lateral erosion and lateral deposition of coarse materials. Characteristics of the meander floodplain include well-developed and intricate patterns composed of closely-spaced, often overlapping, series of channel scars, meander scars, oxbow lakes, etc. The surface is at or slightly higher than the present channel.

Vertical deposition of fine materials by "over-the-bank" floods create covered floodplains. Finer sediments (silts and clays) are carried over the plains and dropped only when the stream velocity becomes very low. Considerable thicknesses of fine-grained alluvium may be built up in this way. Natural levees are formed by sudden decrease of stream velocity in the vicinity of the natural banks and resulting deposition of medium (silty) to coarse-grained sediments.

were still unobstructed. Furthermore, according to Fisk, the alluvial deposits below Baton Rouge are mainly clays, as opposed to sandier alluvia above this point. Clay deposits resist erosion much better than sand, so any change in the channel due to levees would be slow and, in the absence of precise observations at the time, impossible to determine. The use of levees by individuals for flood protection continued even after the transfer of Louisiana to the United States and by 1812, the east bank had levees up to Baton Rouge and the west bank forty miles beyond that. By 1844, the west bank levee line was almost continuous up to the Arkansas River, while only a few levees protected the Yazoo Basin on the east side. During most of this period, it appears that there was very little planning done as to the location, dimensions, etc., of the levee system. Some of the largest levees were indeed built with engineering considerations, but this was generally not the case. The levees from the French period and after were constructed by the riparian landowners and supervised by local authorities to ensure adequate strength. What constituted adequate strength seems to have been based on experience rather than on any engineering criteria. As settlement continued, the task of flood control increased and slowly the engineering of levee construction became more important. For example, in 1833 Louisiana created a post for a civil engineer to be a supervisor of public works, including levees, and in 1835 legislatively specified the dimensions of the Concordia Levee. Larger engineering questions of flood control were also discussed. According to Harrison,

> "The long debate on the best plan for protection of the Alluvial Valley bagan in the 1840's. The need for outlets for Mississippi River floodwaters was discussed with attention to their possible location. Reservoirs were mentioned