Glen Brown

SEDIMENTATION STUDY OF THE YAZOO RIVER BASIN

PHASE I GENERAL REPORT

CONTRACT NO. DACW 38-76-C-0193

Prepared for

U. S. ARMY CORPS OF ENGINEERS
VICKSBURG DISTRICT
Vicksburg, Mississippi

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AUTHORIZATION

This investigation was conducted for the U.S. Army Corps of Engineers, Vicksburg District, Lower Mississippi Division, under Contract No. DACW38-76-C-0193. Larry Banks and Larry Eckenrod were the authorized Project Managers for the Vicksburg District and Daryl B. Simons and Ruh-Ming Li were the Principal Investigators for Colorado State University. The purpose of this investigation was to determine the extent of sediment problems in the main stem Yazoo-Tallahatchie-Coldwater River System and principal tributaries excluding the Sunflower River Basin. In addition, this study recommends ways to control these sedimentation problems and others that may be encountered with the proposed Upper Auxiliary Channel Alternative Project in operation.

In accordance with the contract, the general report which describes the findings of the investigation is submitted. Detailed descriptions of data and mathematical models used in the analysis are presented in the following separate reports:

1. "User's Manual for Known Discharge Sedimentation Model," by D. B. Simons, R. M. Li, and G. O. Brown.
2. "User's Manual for Unsteady Flow Water and Sediment Routing Model," by D. B. Simons, Y. H. Chen, and R. M. Li.
3. "Cross Sectional Data," by D. B. Simons, R. M. Li, and G. O. Brown.
4. "User's Manual for Program CHANSEC," by D. B. Simons, R. M. Li, G. O. Brown, and L. A. Barkau.
5. "Temporal Design," by D. B. Simons, R. M. Li, T. J. Ward, and N. Duong.

ACKNOWLEDGMENTS

Many people provided important and capable assistance to complete this study. The writers would like to extend their appreciation to all those who assisted in this effort and mention a few who were particularly helpful.

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Technical staff of great service were Gary Edelen and Karen Olson, who handled many computer system problems, Arlene Nelson, Head of Technical Typing, Hanae Akari, Head of Drafting, and finally, Jan Wilson, Margaret Reuss, and Carol Stafford, the three secretaries closest to the everyday problems confronting this study. Water and Environment Consultants, Inc., at Fort Collins, Colorado was the subcontractor that collected and collated the necessary data for the analysis, and are acknowledged for their service.

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I. EXECUTIVE SUMMARY AND CONCLUSION

1.1 General

The Yazoo River Basin covers approximately 13,400 square miles in the northwest portion of Mississippi. About 6,600 square miles are in the alluvial valley of the Mississippi River, while the remaining 6,800 square miles are hill watersheds. According to overflow characteristics, the Yazoo Basin is divided into backwater and headwater areas. The Yazoo headwater area is the portion above Yazoo City comprising about 2,300 square miles of alluvial lands and 6,600 square miles of rolling and rugged hill watersheds (Figure 1). The Upper Yazoo Project is a complex flood control system located upstream from the Will M. Whittington Auxiliary Channel. The project provides for approximately 178 miles of channel enlargement of the main rivers, about 203 miles of levees, and 109 floodgate structures.

The Yazoo Basin Sedimentation Study involved a system analysis of the main channel and its tributaries from which water and sediment is routed through the main channel. The purpose of this analysis was to determine the effectiveness of the proposed system considering flood control, navigation, and the location of aggradation and degradation problems in the main channel and its tributaries. Methods of minimizing operation and maintenance problems were also evaluated. This analysis provided a method for evaluating the Upper Yazoo Project system and the various design alternatives outlined by the U.S. Army Corps of Engineers (Design Memorandum No. 41).

In the Phase I study the emphasis was to evaluate the river response to the various design alternatives on the main stem Yazoo-Tallahatchie-Coldwater River system and principal tributaries such as the Little

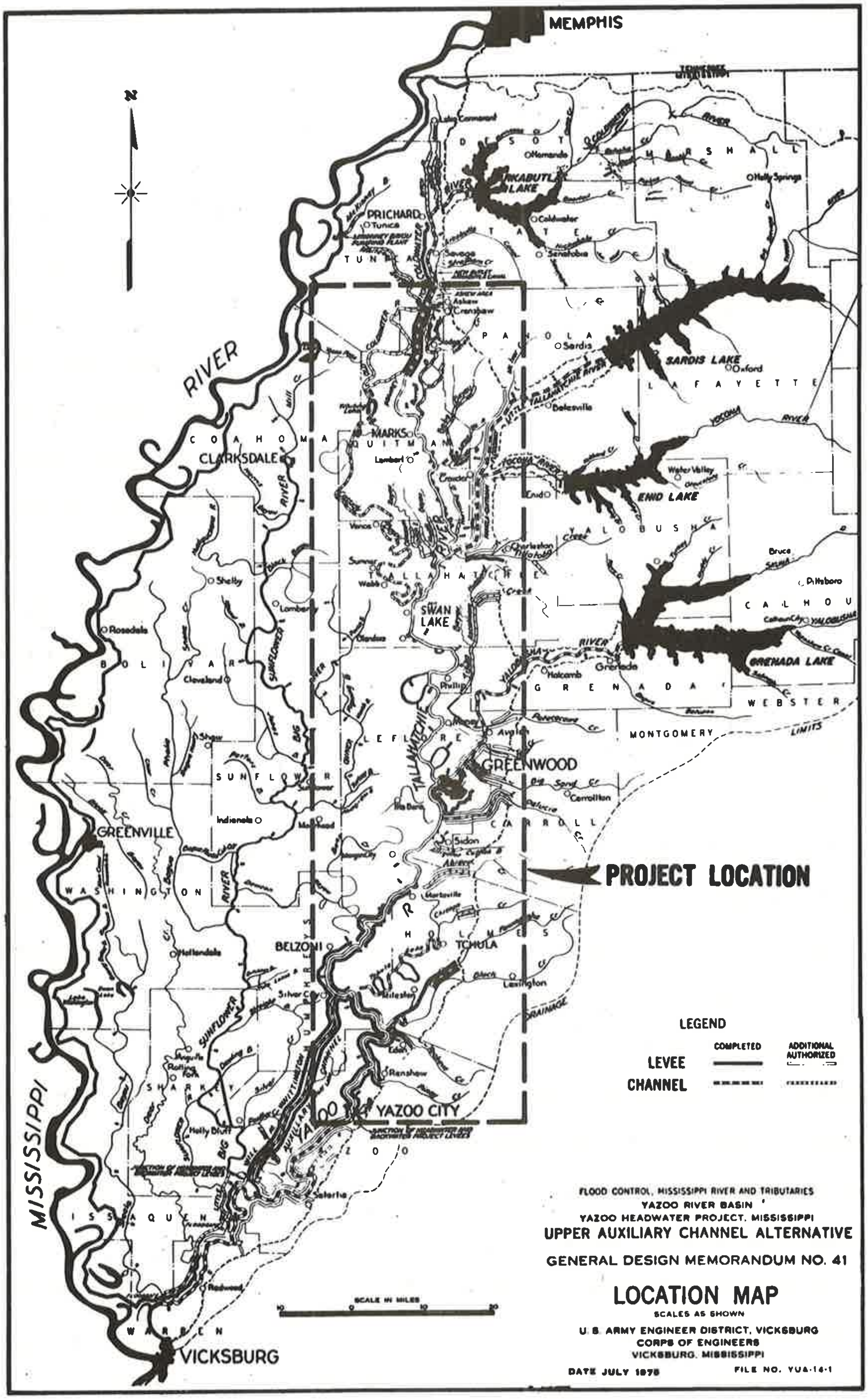


Figure 1. Project location map.

Tallahatchie, Yocona, and Yalobusha Rivers. The Sunflower River Basin was excluded from the analysis. Two mathematical models for routing water and sediment through the system were developed. Utilizing the models, the effects of channel enlargement on flowline, reduced velocities, sediment depositional rates, and other aspects of river response were evaluated. Initial results indicate the hill tributaries and watersheds significantly effect the main stem. The Phase II study is recommended to provide a detailed analysis of tributaries and watersheds and their effects on the main stem. This report describes the general findings pertaining to the Phase I study and the recommendations for Phase II.

1.2 Design Alternative Plans

According to General Design Memorandum No. 41, Upper Auxiliary Channel Alternative, Yazoo River Basin Headwater Project by the U.S. Army Corps of Engineers, Vicksburg District, the following structural alternatives were considered:

Plan A (Cross country). Major features of this plan include an overland auxiliary channel and channel modifications on the Yazoo-Tallahatchie-Coldwater River system. This plan considers channel modification or enlargement on 177.7 miles of river, 72.3 miles of new channel and 233 miles of levees.

Channel size on the main stem varies as follows: Yazoo City to Greenwood, 150-foot bottom; Greenwood to Swan Lake, 110-foot bottom; Swan Lake to the mouth of the P-Q Floodway, 100-foot bottom; from the mouth of the P-Q to the mouth of the Old Little Tallahatchie, 85-foot bottom; and from the Old Little Tallahatchie to the mouth of the Old Coldwater, a 50-foot bottom. On the auxiliary channel, bottom width is

100 feet from Swan Lake to Greenwood where it increases to 160 feet, and remains the same to its confluence with the Yazoo at Tchula Lake.

Plan B (Big channel). This plan omits the overland auxiliary channel and provides the same flood reduction as Plan A. Flood protection is achieved through increasing channel capacity by enlarging the Yazoo-Tallahatchie-Coldwater River system. This plan includes 177.7 miles of channel enlargement, 1.3 miles of new channel, 142 miles of levees, and a structure to allow the controlled diversion of some flows through Tchula Lake.

Channel size on the main stem differs from Plan A due to the 150-foot bottom from Yazoo City to Tchula Lake, a 310-foot bottom from Tchula Lake to Greenwood, and a 210-foot bottom from Greenwood to Swan Lake. Above Swan Lake, channel dimensions are similar to Plan A.

Plan C (Small channel). This plan follows the same channel alignment as Plan B but utilizes a smaller channel and higher levees. It includes 177.7 miles of channel enlargement, 1.3 miles of new channel, 202.8 miles of levees, 0.2 mile of floodwall, and a diversion of some flows through Tchula Lake. Channel size is the same as Plan A except there is no auxiliary channel.

Plan D (Low levee). This alternative utilizes the same channel alignment as Plans B and C but would allow overtopping of the levees along the main stem by floods of greater than a 10-year frequency. The physical features include 177.7 miles of channel enlargement, 1.3 miles of new channel, 156 miles of levees, and diversion of flows through Tchula Lake. Channel size in Plan D is identical to Plan C.

Plan E (Recommended). The recommended plan is a combination of features of Plans B and C. It includes a larger channel north of

Greenwood than Plan C. Physical features include 177.7 miles of channel enlargement, 1.3 miles of new channel, 202.8 miles of levees, 0.2 mile of floodwall, and diversion of flows through Tchula Lake. Channel bottom width is 150 feet between Yazoo City and Greenwood, 130 feet between Greenwood to the mouth of the P-Q Floodway, 100 feet from the mouth of the P-Q Floodway to the mouth of the Little Tallahatchie River, and 75 feet from the Old Little Tallahatchie to the mouth of the Old Coldwater River.

Plan E was recommended after evaluation of different alternatives with respect to environmental impacts, social aspects, economic constraints, engineering problems, and trade-off analysis. The layout is shown in Figure 2. The potential sedimentation problems and possible remedies associated with this alternative plan were analyzed in this study.

Other structural alternatives. Several other structural alternatives that were considered include: the diversion of Coldwater River flows to the Mississippi River, various alignments for a cross-country channel, a Quiver River diversion, other combinations of auxiliary channel and river enlargement, additional reservoirs, and a floodway alternative. These alternatives were eliminated by the U.S. Army Corps of Engineers (Design Memorandum No. 41) due to excessive costs or inability to provide the necessary channel capacity to accomodate reservoir emptying without channel modifications.

1.3 Models for System Analysis

For analyzing a complicated system like the Yazoo River Basin, different levels of resolution in the analysis are needed to reach different objectives. Two mathematical models were developed for

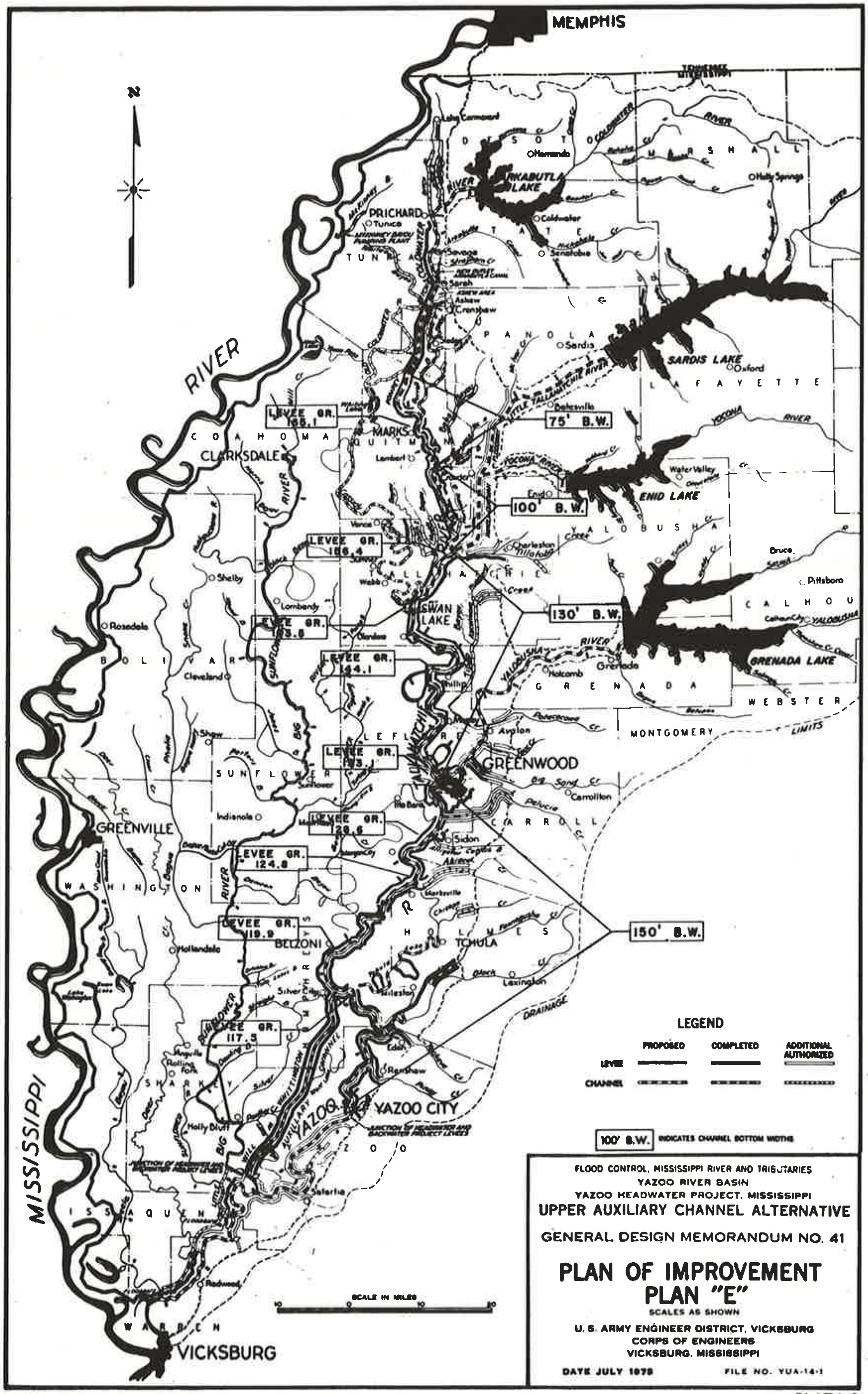


Figure 2. Layout of Plan E.

different levels of resolutions. One is for simulating the flood and sediment movement in a short time resolution (less than a day) which involves the use of an unsteady flow model. This model is particularly useful in predicting flood movement in the Yazoo River system. Another model simulates the sediment movement only and assumes the water hydrographs are known. This simplified model involves use of the discharge synthesized technique and the known discharge uncoupled model for sediment routing. This model can be utilized to evaluate the response of the system using a larger time increment such as one or two days, or even longer. In this study a seven-day time increment for simulation was used since this is the approximate time required for water to travel through the system being analyzed. The closeness of weekly and daily flow statistics further validate the selection of a seven-day time increment for simulation. The primary objective of this study was to identify and analyze the sedimentation problems. Consequently, the known discharge model was used extensively in the analysis.

1.4 Alternative Study Runs

A large portion of the Yazoo River Basin delta will receive substantial flood control benefits from construction of the proposed project as a result of the increased flow-carrying capacity of the main stem river system. Major problems currently existing in the Yazoo Basin include not only flooding but deposition and erosion of sediment that affects many of the tributary streams and the main stem river system.

During the design of the proposed project, emphasis was placed on creating a channel system that would minimize effects of the project work on sedimentation problems in the Yazoo Basin. It is recognized the proposed work will not alleviate all sedimentation problems in the basin streams, and in some cases existing problems could be increased. It is anticipated that developing the channel as proposed will lower the flowline and may cause additional sediment flow to the main channel from the tributary system. Consequently, a plan should be studied that utilizes control structures on the tributaries just upstream of the point where major tributaries enter the Yazoo main stem. These structures would prevent head cutting and the discharge of excess sediments into the main channel.

Another alternative considers utilizing other techniques to further minimize sediment contributed from the watershed to the main channel. For example, control structures such as check dams could be designed, spaced and located on major tributaries to further reduce sediment inflow to the main channel. Also, an analysis could be executed to determine the optimum design utilizing control structures and dredging to minimize maintenance problems in the main channel after it has been modified to meet the requirements dictated by flood control needs and navigation requirements.

In order to identify the extent of sedimentation problems in the main stem and principal tributaries, and the possible remedies, 15 alternative study runs were made. These alternative study runs were conducted by routing water and sediment through the tributaries and the Yazoo River main stem for a selected hydrograph utilizing the various alternative plans. In accordance with letters from the U.S. Army Corps of Engineers, dated March 9, 1978 and June 19, 1978, the alternative study runs described below were completed.

Run No. 1. Simulation run utilizing a 50-year synthetic hydrograph (11 years of recorded data and 39 years of generated data) with natural (existing) river conditions. Greenwood Cutoff was assumed closed for flows less than 25,000 cfs and open for flows greater than 25,000 cfs. It was further assumed that Abiaca Creek would only deliver about 20 percent of the sediment inflow at the hill line to the Yazoo River.

Run No. 2. Simulation run utilizing a 50-year synthetic hydrograph with Plan E conditions and Greenwood Cutoff operated as in Run No. 1. It was assumed that Abiaca Creek would be a leveed floodway that would likely deliver most of the sediment contributed at the hill line to the Yazoo River.

Run No. 3. Simulation run utilizing a 50-year synthetic hydrograph with Plan E conditions, and the Greenwood channel operated as in Run No. 1. The sediment contributing to the main stem from selected tributaries was reduced to near that observed considering the existing conditions identified in Run No. 1.

Run No. 4. Simulation run the same as Run No. 2 but with a reduction of sediment load for the tributaries determined after results were obtained from Run No. 3. This determined whether an additional reduction in sediment load to the main stem could be justifiable in lieu of maintenance dredging.

Run No. 5. Simulation run the same as Run No. 2 except all peak flows were assumed increased 15 percent on both the tributaries and the main stem. This determined the effect of channelization on the tributaries and the main channel.

Run No. 6. Simulation run the same as Run No. 5 except peak flows were increased on the tributaries by an additional 15 percent;. however, flows on the main stem were determined by the model. This evaluated the effect of future land use changes on the system.

Run No. 7. Simulation run the same as Run No. 2 except it was assumed the river was dredged to the original Plan E channel configuration after each year. This run determined the practicality of annual dredging. Additionally, the amount of deposition with annual dredging was compared with allowing sediment to accumulate during the 50-year life of the project.

Run No. 8. Simulation run of a 50-year synthetic hydrograph with completion of the construction schedule for the Upper Yazoo Project. The purpose of the run was to identify problems that could occur in the future as construction continues and dredging proceeds upstream.

Run No. 9. Simulation run assuming natural conditions and utilizing the unsteady flow model for 1973 and 1974 with the Greenwood Cutoff open.

Run No. 10. Simulation run of Plan E using the unsteady flow models for 1973 and 1974 assuming the Greenwood Cutoff is open. This run determined increases in peak flow rates on the main stem.

Run No. 11. Simulation run determining the sediment deposition rate on the lower reach of the Yazoo River from Belzoni to Vicksburg for the identical conditions in Run No. 2.

Run No. 12. Simulation run the same as Run No. 2 except it was assumed the river was dredged to the original Plan E channel every five years. The purpose of this run was to determine if a five-year dredging interval is practical and to compare it with the amount of sediment deposition if sediment is allowed to accumulate for the 50-year life of the project.

Run No. 13. Simulation run similar to Run No. 3 except the main stem channel design was modified (Section 2.2).

Run No. 14. Simulation run that combines Runs No. 4 and 13. This was the best alternative design considered without maintenance dredging.

Run No. 15. Simulation run the same as Run No. 11 except a minimum pool with a water surface elevation of 65 feet was assumed maintained at the proposed lock and dam near Vicksburg. Results of this evaluation indicated whether or not required nine-foot navigation depths could be maintained with the proposed Upper Auxiliary Alternative Plan and the Yazoo River Naviagation Project in operation. The design channel dimensions were modified from the original Plan E to accommodate navigation (Section 2.3).

1.5 Outline of the Study Results

Both unsteady flow and known discharge models were calibrated and verified for two separate reaches of the river with good results. The two reaches of the mainstream river that were tested included the Ft. Pemberton Cutoff at Greenwood Bendway, and a reach on the Tallahatchie from Swan Lake to Locopolis. Results of both models are acceptable and comparable. This verifies that the known discharge model can be used to analyze the Yazoo River system. Results of the analysis in this study are primarily based on the known discharge model. For the main stem, the estimated rate of net filling (considering degradation and aggradation) is about 210,000 cubic yards per year under the natural condition, as shown in Table 7, page 92. The rate of net filling with Plan E conditions increased deposition 400 percent, or the net filling rate in the main stem is about 840,400 cubic yards per year. If all peak flows are increased 15 percent on the tributaries and main stem (effect of channelization), the rate of net filling in the main stem would increase an additional 13 percent. If an additional 15 percent increase in peak flows is imposed on the tributaries to account for future land use changes, an additional 19 percent increase in deposition would result in the main stem.

Control of sediment inflow from the tributaries is an important issue significantly affecting maintenance of the main stem. With a 50 percent reduction in sediment inflows from Abiaca, Pelucia, Big Sand and Tillatoba Creeks, a 31 percent reduction in net deposition along the main stem is achieved even while sediment inflows from the other tributaries was unchanged. The impact of construction scheduling does not significantly change the overall net filling rate in the main stem. A refined analysis is needed to minimize construction effects on main stem channel response.

If annual dredging is implemented to maintain the main stem channel system, the total amount of maintenance dredging for 50 years is 71,346,000 cubic yards (1,426,000 cubic yards annually). If every five years the total amount of sediment dredged is 63,608,000 cubic yards (1,272,000 cubic yards annually), the dredging quantity decreases 11 percent when compared with the volume resulting from annual dredging. It is essential to note the quantity of maintenance dredging is not the only consideration that should be evaluated when selecting the best alternative. Flood levels are also of primary importance.

Modification of the main stem design channel dimensions allows use of a higher base level from Greenwood Bendway to Darling, and an increase in sediment transporting capacity from Belzoni to Greenwood Bendway. The latter condition is achieved by narrowing channel width, an improvement over the original Plan E design. The best alternative plan without maintenance dredging determined from analysis consists of channel modification and control of sediment inflow from Abiaca, Pelucia, Big Sand, and Tillatoba. This plan would reduce the net filling rate in the main stem about 37 percent. In addition, the proposed nine-foot navigation channel would increase the net filling rate in the lower reach (from the mouth of Big Sunflower to Belzoni) about 78 percent over the original Plan E channel. Further examination of the minimum thalweg depth for the 50-year simulation period indicated the proposed nine-foot navigation channel is a feasible development plan if the Plan E channel from Belzoni to Greenwood is maintained.

It is important to mention that if the Plan E channel or its modified channel is allowed to accumulate sediment without maintenance for the 50 years of the project lifetime, the efficiency of flood stage reduction would be significantly decreased. Thus, the proposed plan can only be effective if maintenance measures such as dredging and/or control of sediment inflows from major sediment contributing tributaries are implemented. A refined analysis of the tributaries and scheduling of maintenance dredging is recommended for the future study.

II. PROPOSED UPPER AUXILIARY ALTERNATIVE PLAN

2.1 General Description of Plan E

The proposed Upper Auxiliary Alternative Plan, as recommended by the Vicksburg District, U.S. Army Corps of Engineers, is Plan E (refer to Figure 2). Design Memorandum No. 41 by the U.S. Army Corps of Engineers provides detailed information of this alternative plan. For convenience, some important features of the plan are presented in this report. Plan E includes channel improvement and levees along 179 miles of the main stem of the Yazoo, Tallahatchie, and Coldwater Rivers. This channel improvement is required to (a) provide additional channel capacity to reduce the frequency and duration of flooding caused by the rapid concentration of runoff from the hill tributaries into the existing main stem channel; (b) increase channel capacity so the emptying period flows from the four headwater reservoirs can be accomplished without causing overbank flooding or drainage problems in the delta area adjacent to the main stem; and (c) improve interior drainage of the tributary delta streams. Details of channel improvements and levees for the proposed plan are shown in Figure 3 and are discussed by reaches in the subsequent paragraphs.

2.1.1 Yazoo River--Yazoo City to Greenwood

Major features of the flood control plan in this area consist of a 150-foot bottom width channel along the main stem, and construction of levees and appurtenant drainage works in reaches where levees have not been constructed. Beginning at Yazoo City, the levee will be continuous along the west bank of the river except for the opening at the head of the Will M. Whittington Auxiliary Channel. The east bank levee system will consist of a series of loop levees that tie to the hills

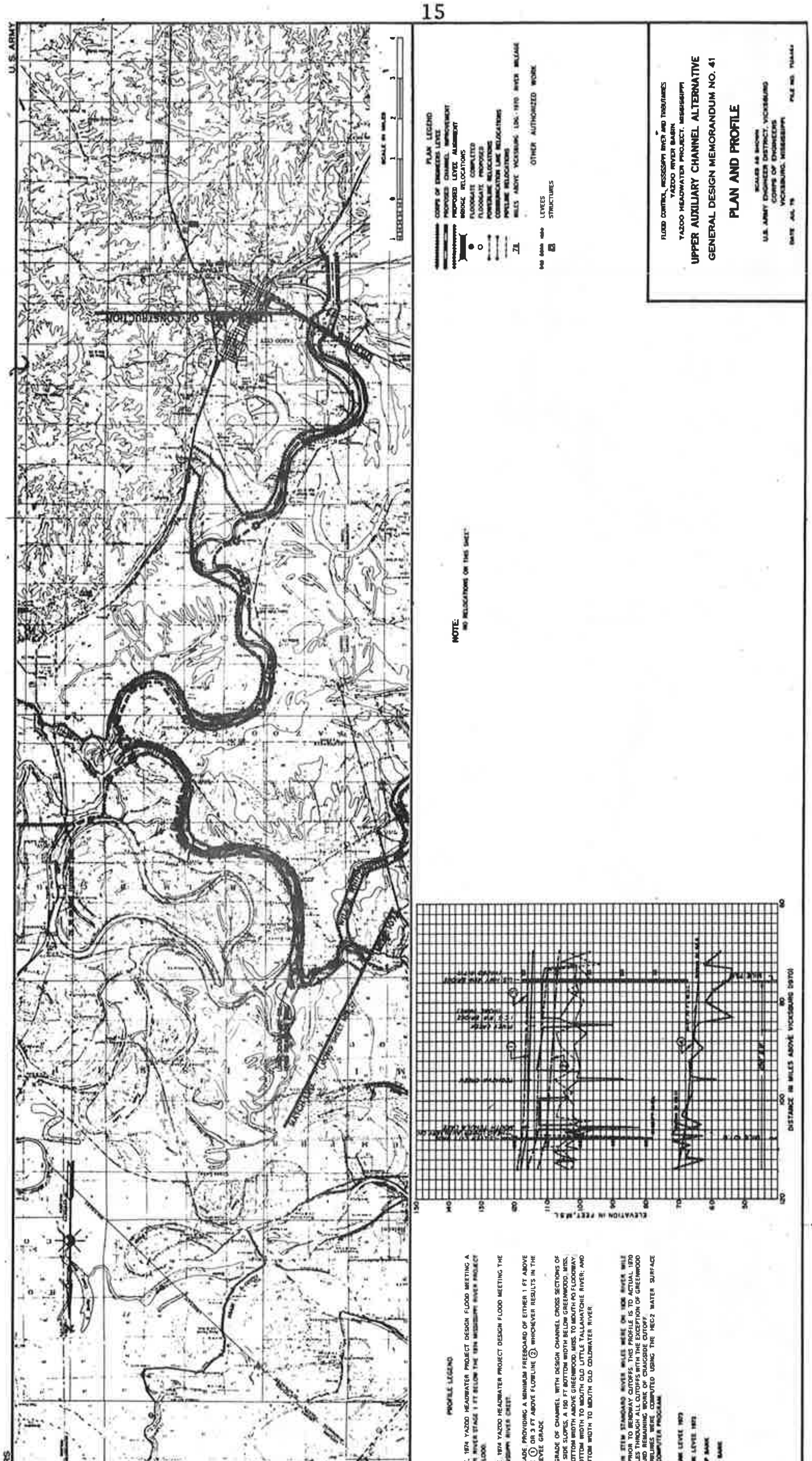


Figure 3. Detailed layout of Plan E.

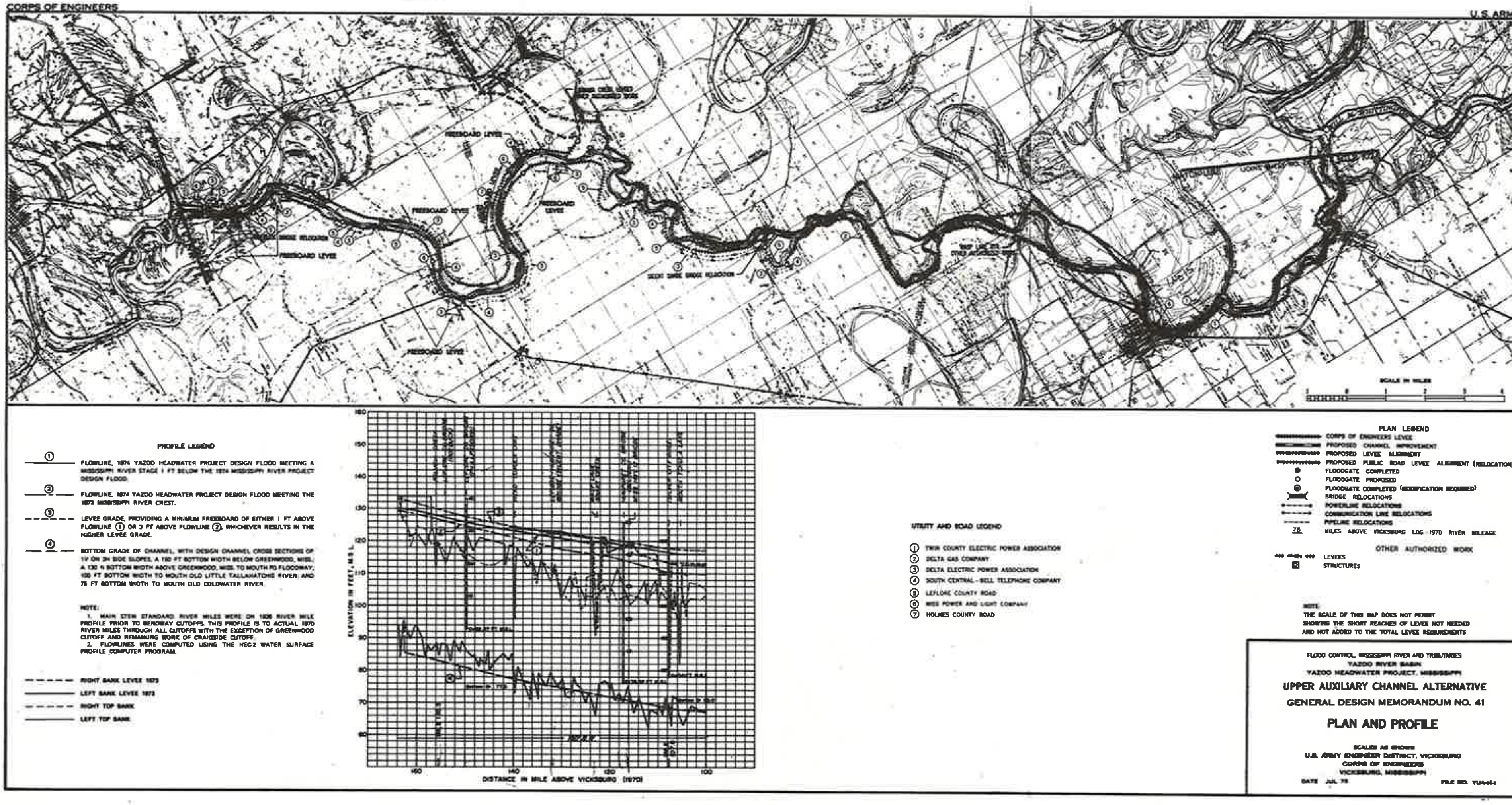
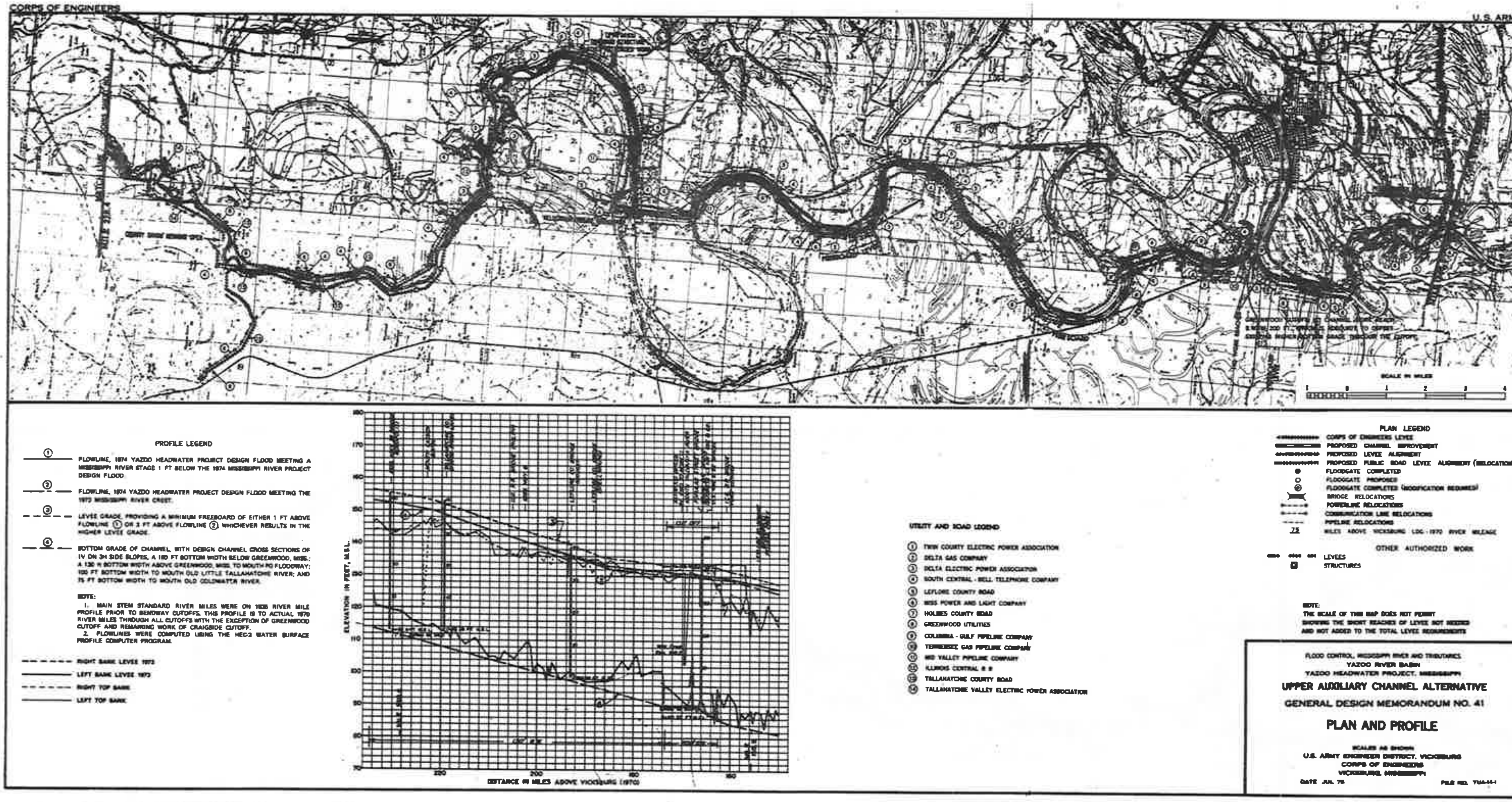


Figure 3. (continued)



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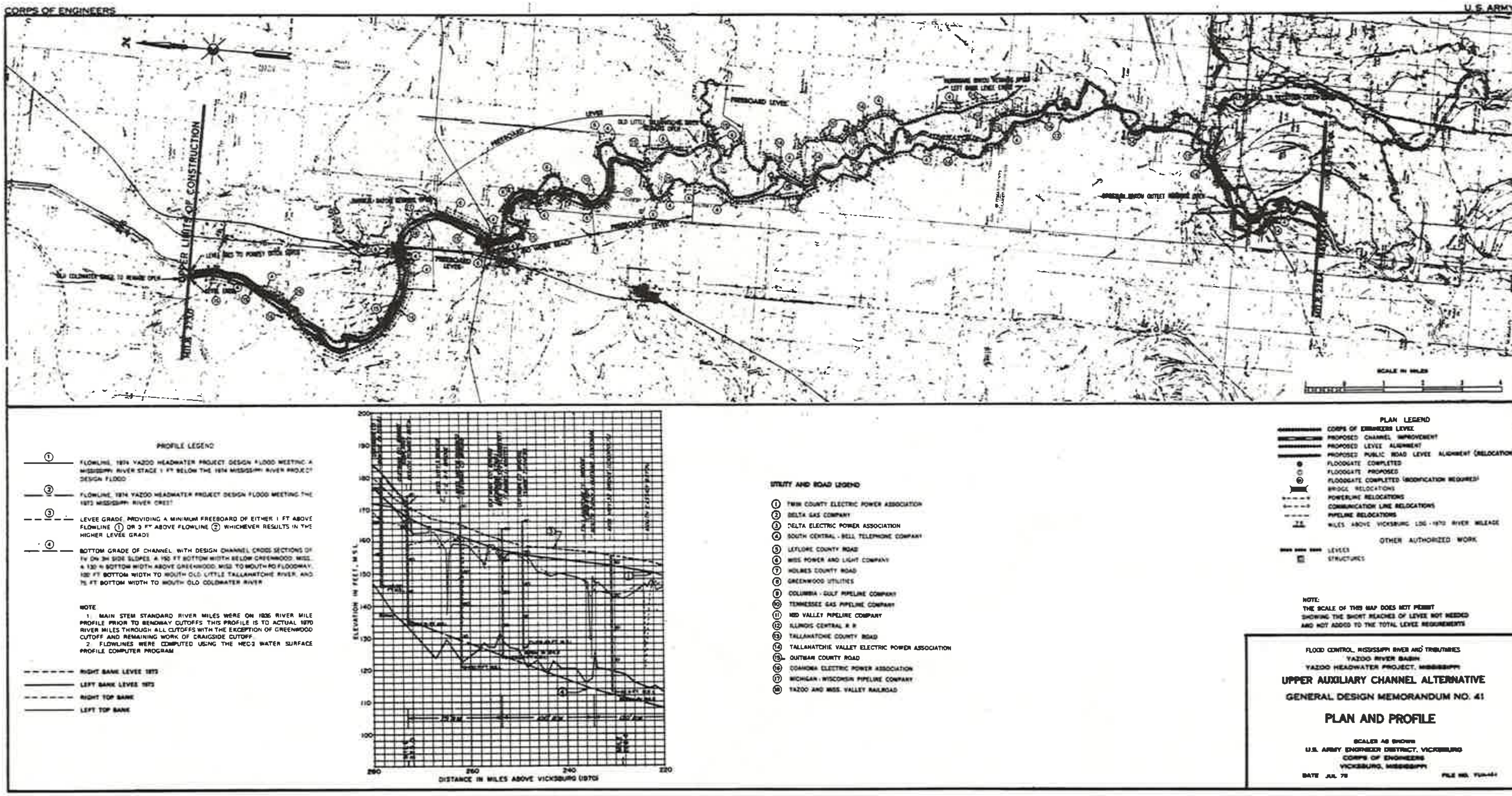


Figure 3. (continued)

beginning with the area below Yazoo City and extending to Piney Creek (the Renshaw area). The next three upstream loop levees will provide protection for the area from Piney Creek to Techeva Creek (the Eden area), the Techeva Creek to Marksville area, and the Abiaca Creek to Pelucia Creek area. The Techeva Creek to Marksville area levees will be tied to the hills along the Hillside Floodway (Black and Fannegusha Creeks). Drainage for the Tchula Lake area will be provided by a floodgate near the lake's mouth.

The proposed structure at the head of Tchula Lake near Marksville will divert flows from the Yazoo River through Tchula Lake during periods when stages at the lower end of the lake permit. During Yazoo River floods with an approximate 10-year or less frequency, and with little backwater effect from the Mississippi River, up to 6,000 cfs may be diverted through the structure without causing flooding in the lower end of the lake. This will result in lower stages and reduced duration of flooding on the main stem, thereby allowing areas protected by levees with floodgates to drain at an earlier date. For floods of greater magnitude than a 10-year frequency, stages at the lower end of the lake are likely to be such that if additional water is diverted into the lake, appreciable overbank flooding will result at the lower end. Hence, no diversion of flows through Tchula Lake during the project flood or other large floods were further considered in this study.

At Greenwood, flows from the Tallahatchie and Yalobusha Rivers were tentatively assumed to be carried through the Greenwood (Fort Pemberton) Cutoff when the discharge at Greenwood exceeded 25,000 cfs. Future studies of the Greenwood Protection Works plan will dictate the appropriate distribution of flows between the cutoff and bendway.

2.1.2 Tallahatchie River--Greenwood to Mouth of the Panola-Quitman Floodway

In this reach, major features of the project consist of a 130-foot bottom width channel, levees, and appurtenant drainage facilities. The east bank levee will begin at the hills near Charleston and follow the south bank of Tillatoba Creek and the east bank of the Tallahatchie River to Greenwood. The west bank levee will be continuous throughout the reach except for openings at Opossum and Cassidy Bayous. Openings in the main stem levee are essential at these locations during extreme floods to permit water to enter the low-lying storage areas, thereby reducing peak flows and river stages downstream. These areas will benefit from the project during the more frequent floods due to the increased channel capacity, lowering of stages on the main stem, and elimination of overbank flooding of the area from overtopping of levees or natural overbank flow.

The existing opening adjacent to Cassidy Bayou above Swan Lake, commonly referred to as "Chute Bridge," will be closed as part of this plan. Another major feature of the proposed plan in this reach is the construction of the authorized Craigside Cutoff between miles 175.4 and 185.6 on the Tallahatchie River.

2.1.3 Tallahatchie River--Mouth of the Panola-Quitman Floodway to Mouth of the Old Little Tallahatchie River

In this reach, the proposed plan consists of a 100-foot bottom width channel on the main stem from the mouth of the Panola-Quitman (P-Q) Floodway to the mouth of the Old Little Tallahatchie River, and a continuous levee along the west bank of the Tallahatchie River. The east bank levee will have an opening from just above the Panola-Quitman Floodway to above Hurricane Bayou. Another opening will be located near

Lambert where flows from the Old Little Tallahatchie River and Bobo Bayou enter the main stem. These openings are required so that low-lying areas adjacent to the river can be effectively used as storage distributaries during extreme floods to reduce peak flows and stages at downstream locations. These areas will benefit during more frequent floods due to the lowering of stages on the main stem at these openings.

2.1.4 Coldwater River--Mouth of Bobo Bayou to Mouth of Old Coldwater River

A 75-foot bottom width channel and a continuous levee along the west bank of the river are proposed for this reach. Along the east bank, the levee will be continuous except for an opening near the mouth of Burrell Bayou. This opening is required to permit floodwaters to enter the low-lying areas and reduce peak flows and stages downstream.

2.1.5 Coldwater River (Pompey Ditch)--Mouth of Old Coldwater River to Prichard

Based on backwater computations by the U.S. Army Corps of Engineers and recent flowline observations for this reach of the Coldwater River, it was determined that no channel work or additional levees are required on the Coldwater River above the mouth of the Old Coldwater River. The opening at the mouth of the Old Coldwater River will be left open to allow backwater storage areas to be used to lower flows and stages downstream on the main stem. Stages on the lower end of the Old Coldwater River will be lowered by the project due to the increased channel capacity on the main stem. However, improvement of the Old Coldwater River channel will be required to appreciably lower stages in the vicininty of Yazoo Pass and in the upper reaches of the Old Coldwater River and Whiteoak Bayou.

2.2 First Modified Plan E Channel Geometry

This modification to the Plan E channel design would provide a higher base level from Greenwood Bendway to Darling, and an increase in sediment transporting capacity from Belzoni to Greenwood. The latter condition is achieved by narrowing the channel. Such a modified channel would improve the original Plan E design reducing the net filling or depositional rate in the main stem. Detailed modifications of Plan E are explained in the following sections.

2.2.1 Yazoo River--Belzoni to Greenwood

Channel design was changed from a 150-foot bottom width in Plan E to a 130-foot bottom width. Initially, the bottom elevation at Belzoni was assumed to drop about two feet; however, it was finally determined not to make this change after verifying that the sediment transport capacity is decreased by deepening the main stem channel.

2.2.2 Tallahatchie River--Greenwood to Mouth of the Panola-Quitman Floodway

Channel widths upstream of Greenwood were unchanged as compared to Plan E. However, the bottom elevation at Money was raised to 98.0 feet and the elevation at the mouth of the P-Q Floodway was raised to 117.5 feet. This modification raises the water surface level near the mouth of the P-Q Floodway and also reduces degradation in the floodway.

2.2.3 Tallahatchie River--Mouth of the Panola-Quitman Floodway to Mouth of Old Coldwater River

Channel widths in this reach were kept the same as those identified in Plan E. The bottom elevation at Darling was raised to 137.7 feet. Again, raising the bottom elevation in this reach reduced degradation in the P-Q Floodway.

2.3 Second Modified Plan E Channel Geometry

This modification was made to determine the sensitivity of the lower Yazoo system to increases in channel width that would be required should the Yazoo Navigation Project be implemented. A 200-foot bottom width channel was selected for the reach below Greenwood to evaluate sediment transport capacities and dredging requirements for a larger and wider channel. Pertinent criteria for the navigation project had not been determined, therefore, the 200-foot channel was selected to represent an average of the effect of bend widenings that would likely be required in most of the bendways on the lower Yazoo.

III. SYSTEM DESIGN

3.1 Overall System Design

As mentioned previously, the Yazoo River Basin is composed of a main stem and numerous tributaries (refer to Figure 1). Analysis of the Basin simulates the river system as a whole rather than only selected areas.

In the Yazoo River Basin, tributaries to the main stem were divided into controlled, uncontrolled, and point source type streams. The controlled tributaries, Coldwater, Little Tallahatchie, Yacona and Yalobusha Rivers, are regulated by the Arkabutla, Sardis, Enid, and Grenada Reservoirs. The controlled tributaries significantly effect the response of the Basin. Uncontrolled tributaries are generally smaller than controlled ones and do not have large storage reservoirs; however, they are important in analysis. An example of this type of tributary is Big Sand Creek. The third type of tributary is referred to as a point source and is generally smaller than the other two types. Point source tributaries are considered as a single point input. The potential impact of sediment inflow from point source tributaries to the main stem is small. However, in order to conserve flow continuity, these tributaries were included in the analysis of the Yazoo system as point source inputs. An example of this type of tributary is Piney Creek.

Classification of tributaries is based upon examining system response which is described in the next section. Controlled, uncontrolled, and point source tributaries are shown schematically in Figure 4. This figure also shows the relation of the tributaries to the main stem system. In addition, the nonpoint source contribution from additional

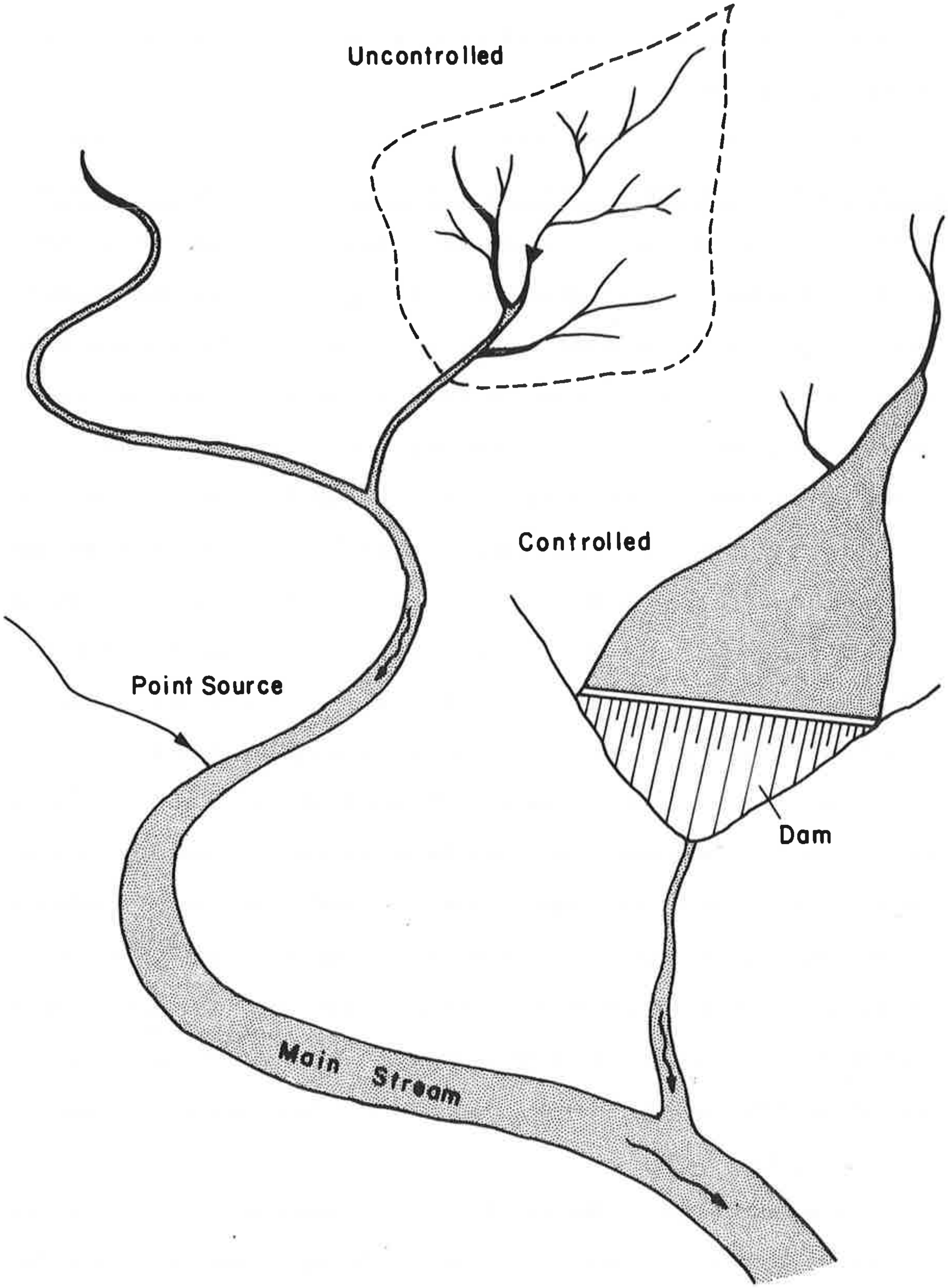


Figure 4. Schematic of river system.

uncontrolled areas was considered in order to conserve flow continuity in the total system.

The computer model developed to analyze the system is a physical process model based on the physical environment. Information provided by the model for the main stem includes: water discharge, water level, sediment discharge, and aggradation or degradation of the channel. These outputs were simulated for different alternatives and operating conditions of the river system development as previously identified. The input required was provided from output of the controlled and uncontrolled tributaries, and other point or nonpoint contributions from tributaries. Input for these types of models consists of water and sediment discharges from primary input sources, and lateral inflows or outflows from land surfaces. Reliable methods are needed to synthesize these required input data if measured data are not available. Because of a limited data base for the analysis, discharges from ungaged tributaries and nonpoint sources were synthesized utilizing watershed area and continuity. Sediment versus discharge rating relations for tributaries were estimated by assuming that a dynamic equilibrium condition is reached on the natural channel utilizing a 50-year hydrograph. Subsequent problems encountered in the conceptual system design included determination of locations of main tributaries and other sources, and generation of hydrographs for the simulation of future channel response.

3.2 Spatial Design

Spatial and temporal designs of the Yazoo River system are necessary to provide a realistic representation of the space-time structure for accurate simulation of the system. The location of rivers and tributaries and the location of all pertinent gaging stations, structures, and confluences are included in the spatial design. The spatial design was based on the potential contribution of all sediment sources to the bed elevation changes.

By applying the sediment continuity equation, a sediment transport equation, and a set of typical flow conditions, the bed elevation changes along the main stem between each two neighboring confluences were determined. Also, the percentage changes contributed by the tributaries were determined considering the ratio of sediment transport rates between the main stem and its tributaries. By summing all changes in the sediment storage volume (product of bed elevation change, wetted perimeter, and the space increment between two neighboring confluences) and relating this total change to the individual change, the percentage of sediment from each tributary contributing to changes in bed elevations in the main stem was determined.

After ranking the potential contributions of sediment according to the computed percentages, the determination of important tributaries (either controlled or uncontrolled) and point source inputs were delineated. The abstracted system of the Yazoo River system is given in Figure 5. There are 31 tributaries to the Yazoo Basin. Only 12 of the tributaries were identified important for the analysis: Arkabutla Creek, Little Tallahatchie River, P-Q Floodway, Yocona River, Tillatoba Creek, Yalobusha River, Potococowa Creek, Teoc Creek, Big Sand Creek, Abiaca Creek, Pelucia Creek, and Techeva Creek. Other inputs could be considered as point source inputs. Old Coldwater River, Bobo Bayou, Burell Bayou, and all the tributaries in the Sunflower Basin such as Big Sunflower River, Little Sunflower River, Deer Creek, and Steele Bayou were excluded from analysis. Since the primary objective of the Phase I

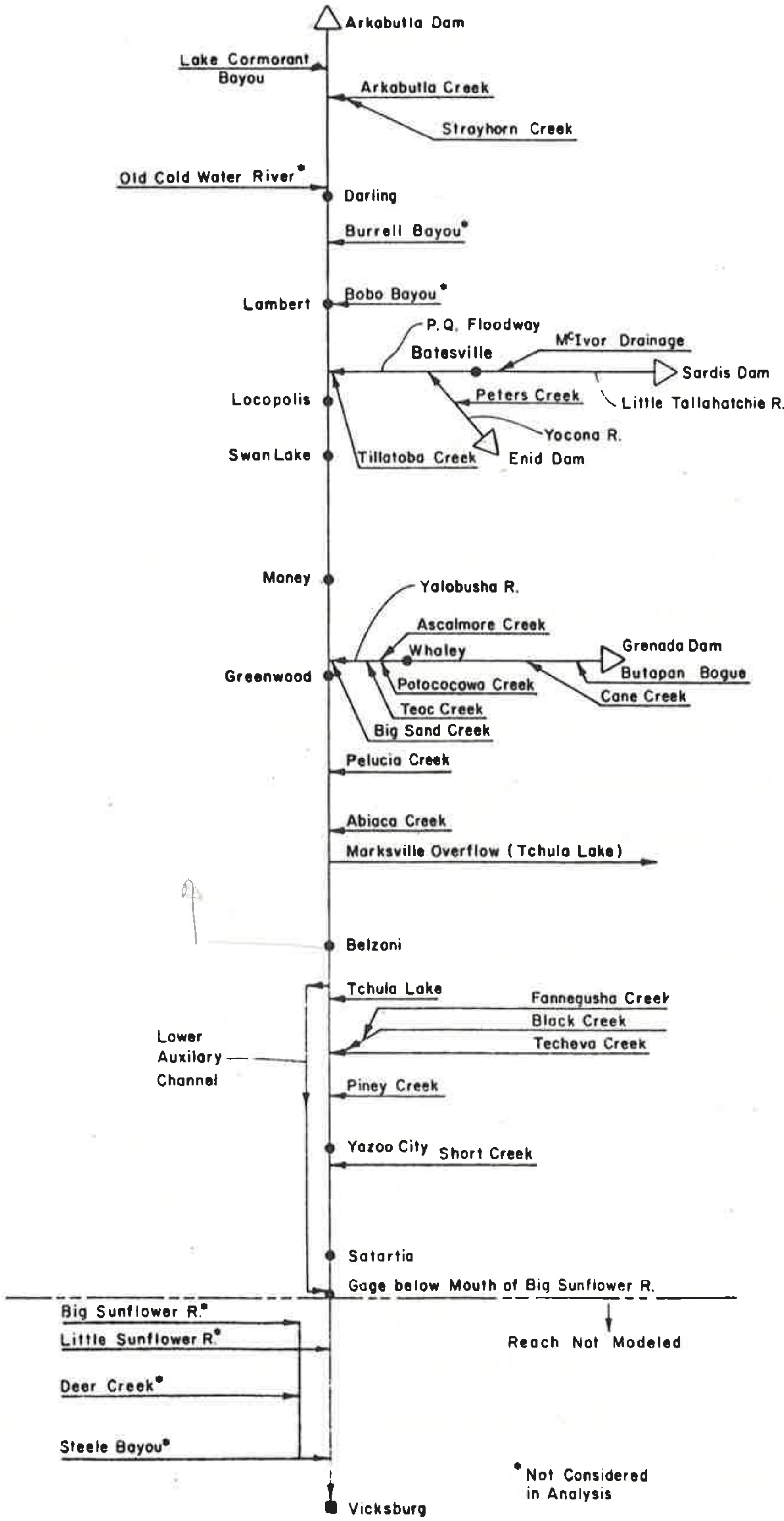


Figure 5. Spatial representation of the existing conditions (Run No. 1).

study was to evaluate the response of the main stem and major tributaries such as the Little Tallahatchie River, P-Q Floodway, Yocona River, and Yalobusha River, spatial representation of Phase I only covered these four. Other tributaries were considered as point sources, some of which will be studied in more detail during Phase II.

Spatial representation of Alternative Run No. 1 was the same as shown in Figure 5. The proposed Plan E is the basic spatial design for other alternative runs. Run No. 2 is Plan E as proposed by the U.S. Army Corps of Engineers (Figure 3) and represented in Figure 6. A detailed index map of the cross sections and the associated bed profiles utilized in the model is shown in Appendix A. Alternative study Runs No. 3, 4, 5, 6, 7, 11, and 12 have the same spatial design as Run No. 2 except channel cross-section data for Runs No. 7 and 12 were changed to conform with the original cross sections after a specified period in lieu of maintenance dredging.

For simplicity, Plan E and its modified channels would have channel bottom elevations as specified in the design, even when the original bed elevation is lower than the design bed. It was tentatively determined that the construction schedule proceed upstream in two-year intervals. At the end of the eighth year, construction would proceed from Yazoo City to Greenwood, and construction would be complete to the mouth of the P-Q Floodway at the end of the 14th year. The total Upper Yazoo project would be completed at the end of the 18th year after start of construction.

Run No. 8 considers the effect of construction and its associated spatial representation is shown in Figure 7. Spatial representation for study runs (No. 9 and 10) utilizing the unsteady flow water and sediment

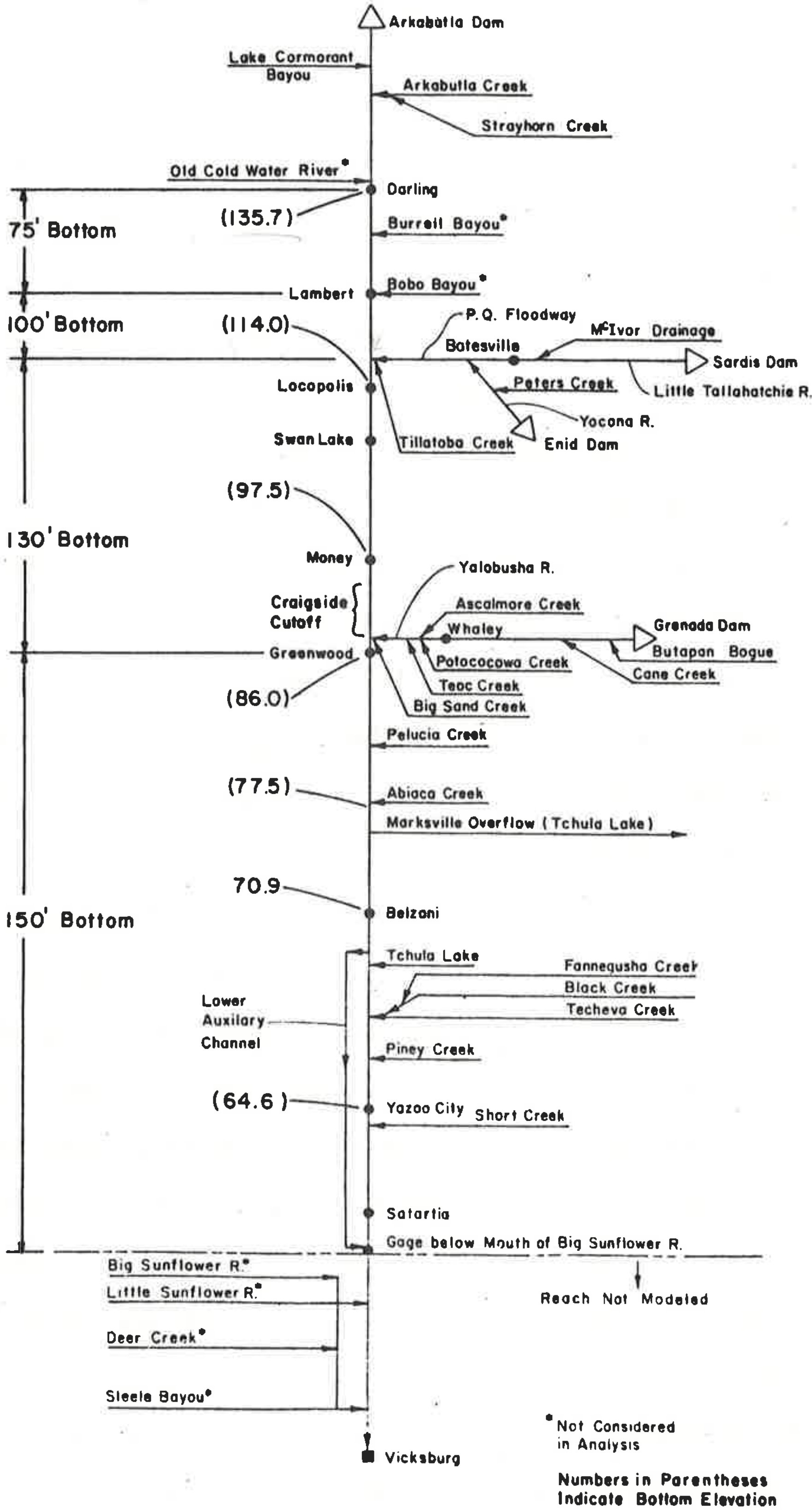


Figure 6. Spatial representation of Plan E conditions (Runs No. 2, 3, 4, 5, 6, 7, 11, and 12).

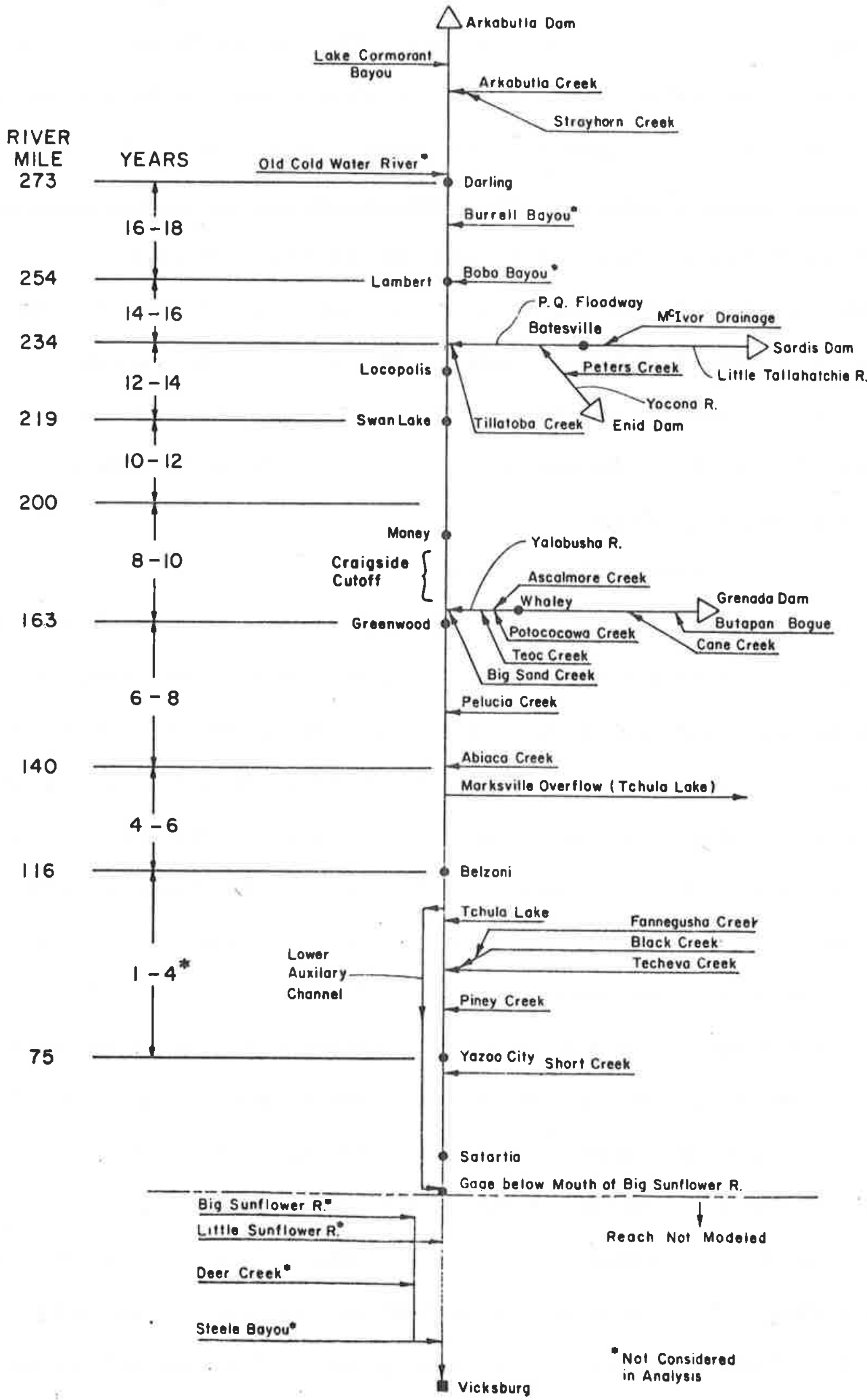


Figure 7. Spatial representation of construction scheduling (Run No. 8).

routing model includes a smaller area. The spatial design is shown in Figure 8. The first modifications to Plan E were evaluated in Runs No. 13 and 14 and are spatially represented in Figure 9. The impact of the Upper Yazoo Project on the lower reach and the feasibility of a nine-foot navigable channel with lock and dam near Vicksburg were studied in Runs No. 11 and 15. The spatial representation of these two runs is shown in Figures 6 and 10, respectively. Detailed descriptions of the layout of the spatial design and cross-sectional data used are given in "Cross Sectional Data--Sedimentation Study of the Yazoo River Basin."

3.3 Temporal Design

3.3.1 General Description

A primary component of the analysis of the Yazoo River Basin is realistic water discharge records. These records were developed from existing discharge and water stage data. Sixty-two sets of discharge records were developed for this study including 30 sites with existing discharge or stage records, 14 ungaged tributary sites, and 18 nonpoint sources as outlined in Figure 11. Two sets of discharge records were developed for each site. The first set was composed of 4,018 average daily flow values for January 1, 1964 to December 31, 1974. This period was used because of the availability of discharge or water stage measurements. The second set was composed of 2,609 weekly average flow values, i.e., the average daily flow for a seven-day time span. This time period included 574 discharge values for the 11 years from 1964 through 1974 plus 2,035 synthesized values of discharge for the next 39 years. Weekly flow records were developed from pre-existing average daily flow records. Tasks involved in the development of the temporal design are

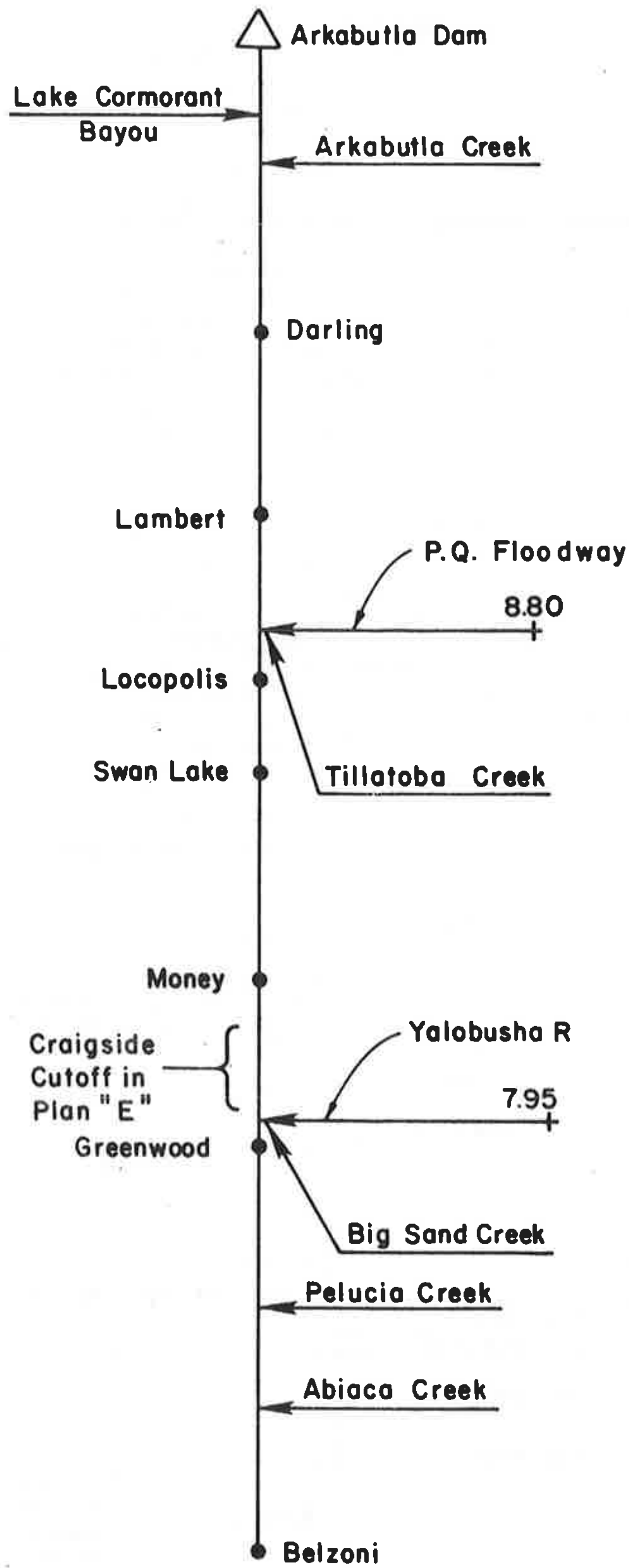


Figure 8. Spatial representation of the unsteady flow model (Runs No. 9 and 10).

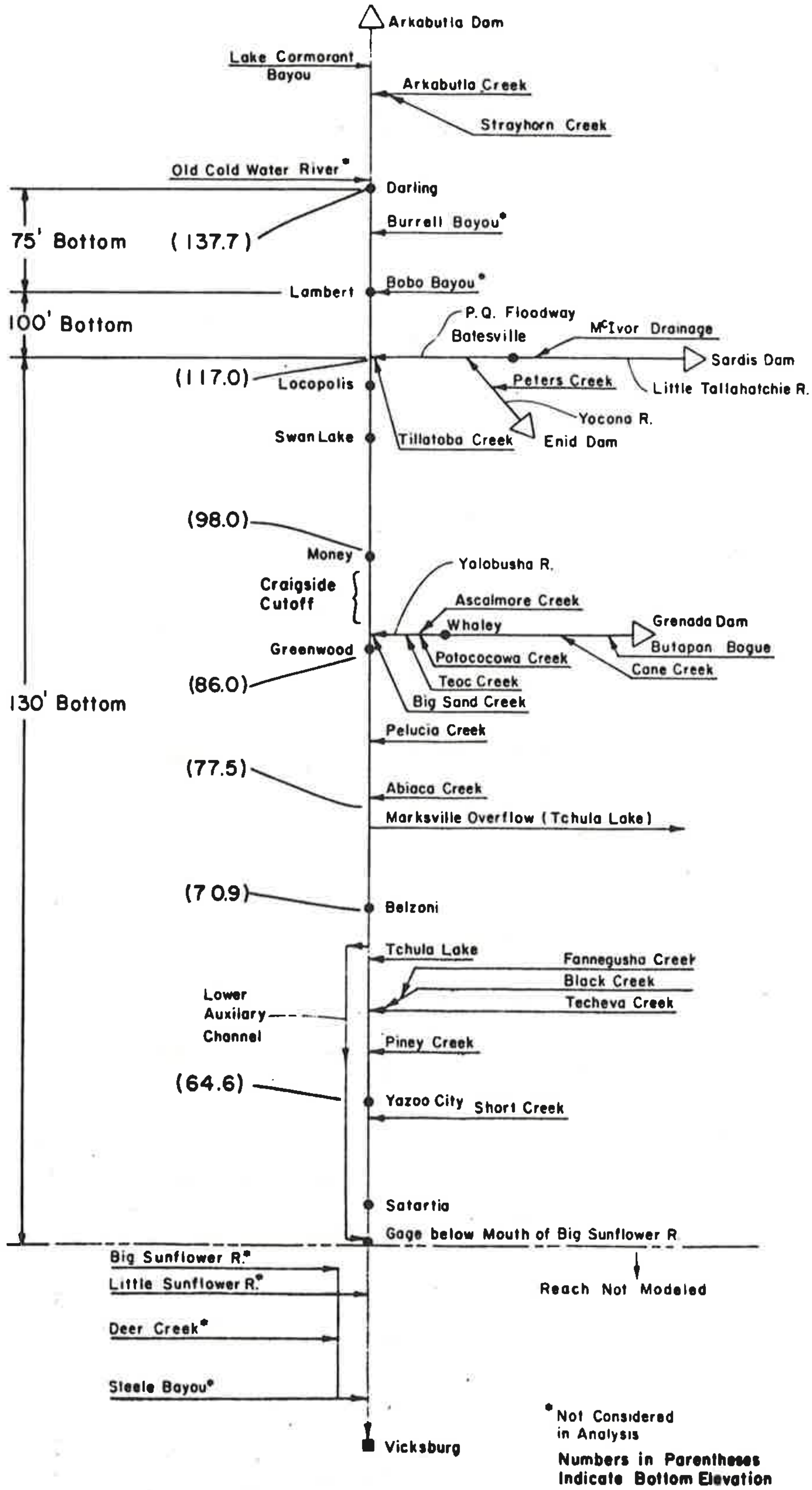


Figure 9. Spatial representation of the first modifications for Plan E channel design (Run No. 13).

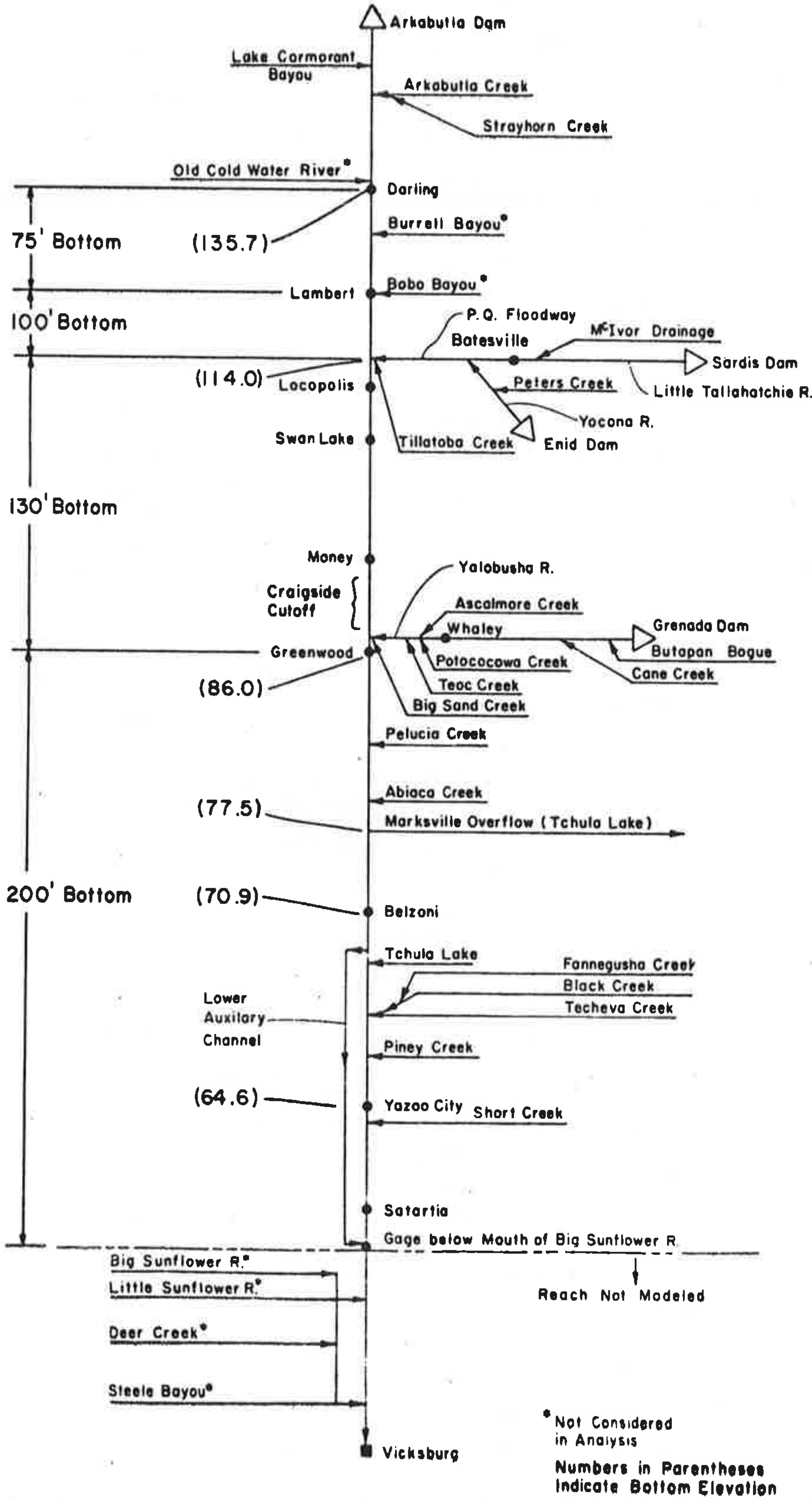


Figure 10. Spatial representation of the second modification for Plan E channel design for navigation (Run No. 15).

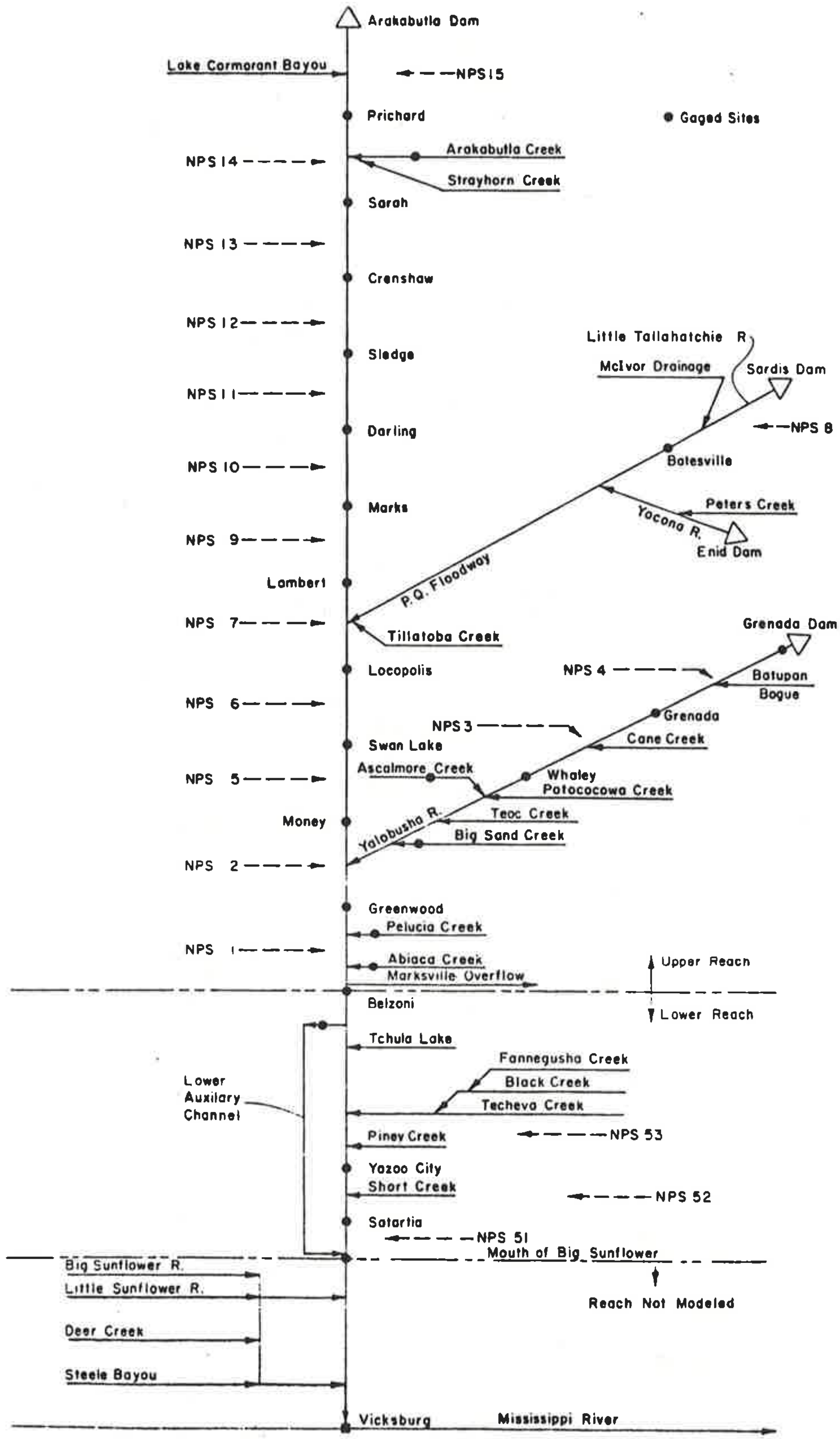


Figure 11. Spatial representation of gaged sites, ungaged sources, and nonpoint sources.

shown in Figure 12 and described in "Temporal Design--Sedimentation Study of Yazoo River Basin."

3.3.2 Development of Average Daily Flow Records

Average daily flow records for seven sites were obtained from the U.S. Geological Survey Water Resources Division. These sites included outflows from the four major reservoirs and the three river sites of Lambert, Swan Lake, and Greenwood. Records for the other 23 gaged sites were developed from U.S. Army Corps of Engineers stage records supplied on magnetic tape. Stage-discharge rating curves were developed either from observed stages and discharges or supplied by personnel from the U.S. Army Corps of Engineers.

Stage readings taken at 8 A.M. were converted to discharges and were used to develop average daily discharges by linear interpolation. Existing average daily discharges and the interpolated discharges provided the flow records for gaged sites. Ungaged sites such as Teoc Creek required data synthesis using nearby sites. The approach used in this synthesis involved computed representative discharge per square mile for a nearby similar site or pair of sites. The area of the synthesized site was multiplied by discharge per square mile. For example, if a site had an area of 100 square miles and a nearby gaged tributary yielded a flow of two cfs per square mile on a particular day, then the flow at the synthesized site was set at 200 cfs. When applicable, an average discharge per unit area for two nearby gaged stations was used for synthesis of discharge. For some sites, the discharge per unit area was found as the difference in flow between two river or major tributary sites. Flow from the McIvor Drainage site, for example, was computed by

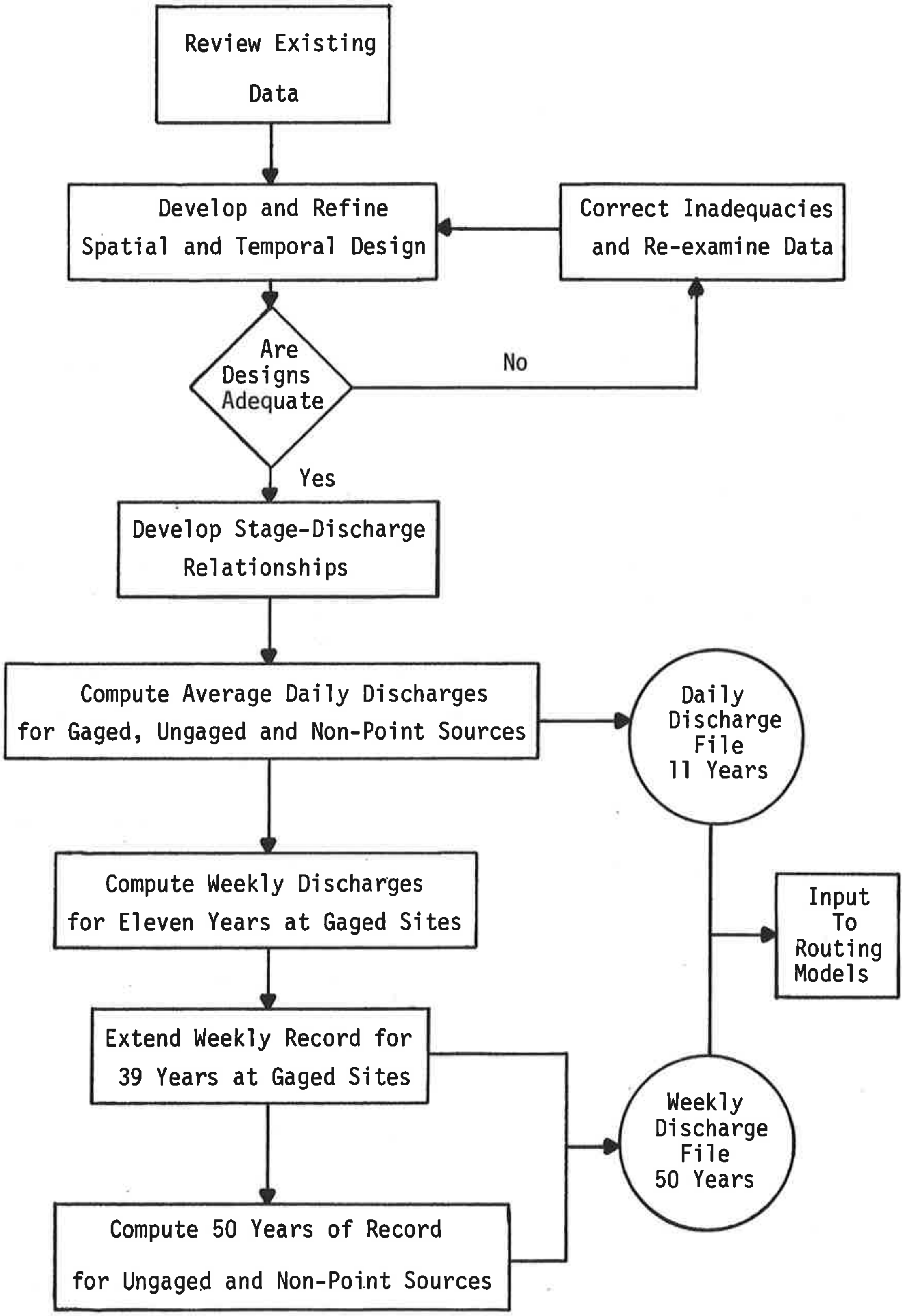


Figure 12. Sequence of discharge record development.

using the difference in flow between the P-Q Floodway near Batesville and outflow from Sardis Dam.

After all gaged and ungaged sites were determined, nonpoint sources were computed. Nonpoint sources for a river reach were computed as the difference between outflow and inflow of the reach. The reach between Prichard and Sarah contains inflow at Prichard, Arkabutla Creek, and Strayhorn Creek, and outflow at Sarah. The nonpoint source was computed as Prichard discharge minus the three inflows. Using this flow continuity approach, all water is accounted for by gaged, ungaged, and nonpoint sources. Use of daily flow continuity is reasonable since flow travel times between any two stations in a reach is usually less than one day.

3.3.3 Development of Weekly Flow Values

Weekly flow values were developed using an approach similar to that used to estimate daily flow values but includes additional simulation of extended records. Generation of synthetic streamflow sequences that are sufficiently long so they can be expected to simulate typical flow conditions was an important task. This record allowed realistic input to the mathematical simulation models of the Yazoo River Basin system to evaluate effects of various flows on structures and environmental conditions along the river. To accomplish this task, several stochastic models were constructed to preserve the mean, variance, and autocorrelation structure of the historical flow record.

Time series models are often fitted to the autocorrelation function or equivalently its Fourier transform. The stationary component of the time series is then removed. Research has shown the best method is to synthesize logarithms of the flows using an Auto-Regressive Moving Average Scheme (ARMA) with time varying auto-regressive (AR) coefficients that preserve long range dependence of the hydrologic series. This approach employs Kalman filtering to improve the fitting of an AR (2) (auto-regressive lagged two-time periods) model to the historic data. This model was then used to extend the 11-year record an additional 39 years to a total of 50 years.

Trend elements were approximated by polynomial equations, while a cyclical component with variations resulting from seasonal flow patterns was represented by harmonic functions. Removal of the trend and cyclical components facilitated ARMA simulation of the time series. In this application, an AR (2) model was fitted to the standardized time series. A sixth order harmonic model was used to fit the time variant AR (2) coefficients. The harmonic model was then used to extend the AR (2) coefficients forward in time. These coefficients were used to create a new standardized series. The operations are summarized as follows:

(a) Compute the average discharge for each seven-day period.

(b) Take the logarithm of the average discharge data.

(c) Detect the trend and cyclical components of the discharge series and subtract them from the historical data.

(d) Standardize the residual time-series.

(e) Fit an AR (2) model to this series by Kalman filtering.

(f) Fit a sixth order harmonic model to the coefficients of the AR (2) model.

(g) Calculate the mean and standard deviation of the fitting errors of the AR (2) model. These statistics generated random noise which was added to the generated data from the AR (2) model when synthesized data was desired.

Synthesized streamflow data was obtained through the following inverse operations:

(a) Generate the coefficients for the AR (2) model with the sixth order harmonic model.

(b) Generate the residual time-series by adding in normally distributed random errors.

(c) De-standardize the data to return to a residual time-series.

(d) Add the trend and cyclical components to the residual time-series.

(e) Take the exponential of the generated data.

(f) Check with the lower and upper bounds on the generated values.

(g) Check the simulated with the historical flow characteristics, such as frequency of peak flow and flow volume distribution.

If the flows were realistic and consistent the flow series was accepted for use.

After the weekly flow values for the gaged sites were extended to 50 years, ungaged and nonpoint sources were computed the same as daily flow values. A seven-day average was used because flow rates indicate that water can move through the system from Arkabutla Dam to Vicksburg in about seven days.

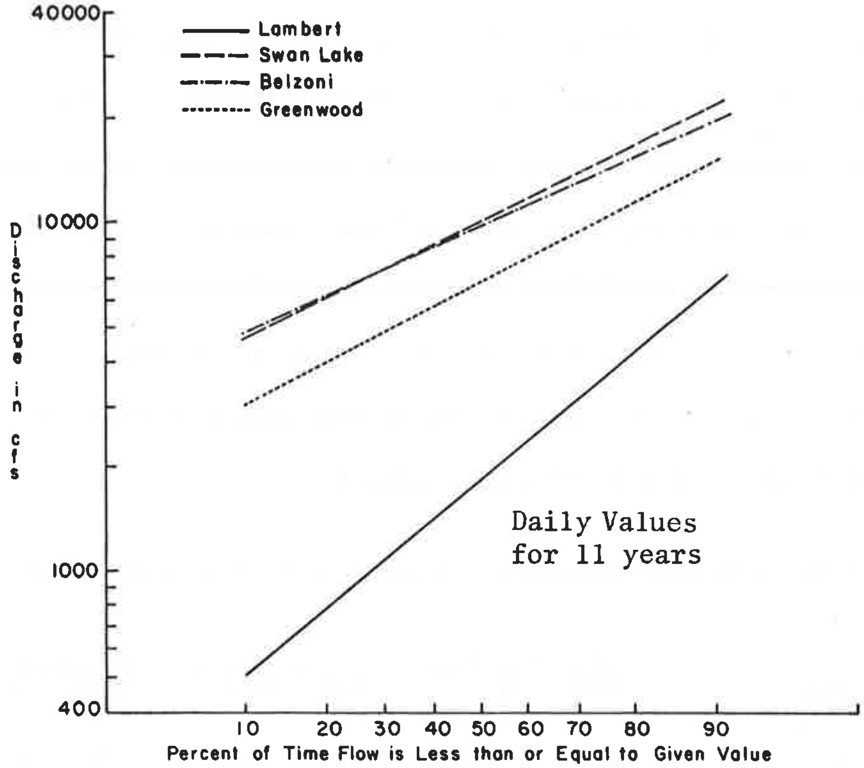
3.4 Discussions of the Results

A summary of daily and weekly flow values at key gaging stations is given in Table 1. As Table 1 shows, the simulated record retains the characteristics of the actual record particularly for the gaged stations. Flow frequency for four key main stem gaging stations are shown in Figure 13. The shapes of the frequency curves are very similar. However, the 50-year synthesized hydrograph has lower frequencies than

Table 1. Comparison of flow statistics for 11 years daily, 11 years weekly, and 50 years of weekly flows.

Upper River

Station	Statistic	11 Years Daily	11 Years Weekly	50 Years Weekly
Belzoni	Max	28158.76	28114.91	28114.91
	Min	1339.75	1466.92	1466.92
	Mean	11335.60	11335.60	12183.22
	Std.Dev.	5777.74	5734.42	4436.44
Greenwood	Max	43800.00	40857.14	40857.14
	Min	971.00	1065.71	1065.71
	Mean	11727.19	11727.19	12674.57
	Std.Dev.	6513.29	6426.53	4924.74
Money	Max	22830.88	22419.91	22419.91
	Min	516.73	561.00	561.57
	Mean	8145.13	8145.13	9183.14
	Std.Dev.	4785.18	4731.65	3860.66
Swan Lake	Max	44900.00	36428.57	36428.57
	Min	612.00	774.14	774.14
	Mean	7929.28	7929.28	8917.06
	Std.Dev.	4857.45	4750.23	3842.77
Locopolis	Max	33753.37	29802.49	29802.49
	Min	650.53	713.15	713.15
	Mean	7291.47	7291.47	8260.64
	Std.Dev.	4767.61	4685.40	3641.99
Lambert	Max	15100.00	14571.43	14571.43
	Min	85.00	116.43	116.43
	Mean	2843.12	2843.12	3400.68
	Std.Dev.	2785.66	2670.86	2681.66
Yazoo City	Max	19656.32	19605.04	19605.04
	Min	2000.00	2086.55	2086.55
	Mean	9229.60	9229.60	9709.58
	Std.Dev.	3929.38	3895.01	3078.39
Satartia	Max	20142.66	20122.68	20122.68
	Min	2000.00	2000.00	2000.00
	Mean	9410.73	9410.73	9891.98
	Std.Dev.	4218.92	4184.54	3499.55
Yazoo River at Mouth Big Sunflower	Max	44171.82	44155.87	44155.87
	Min	2245.98	2411.04	2411.04
	Mean	18816.81	18816.81	19723.91
	Std.Dev.	9563.75	9496.20	8098.45



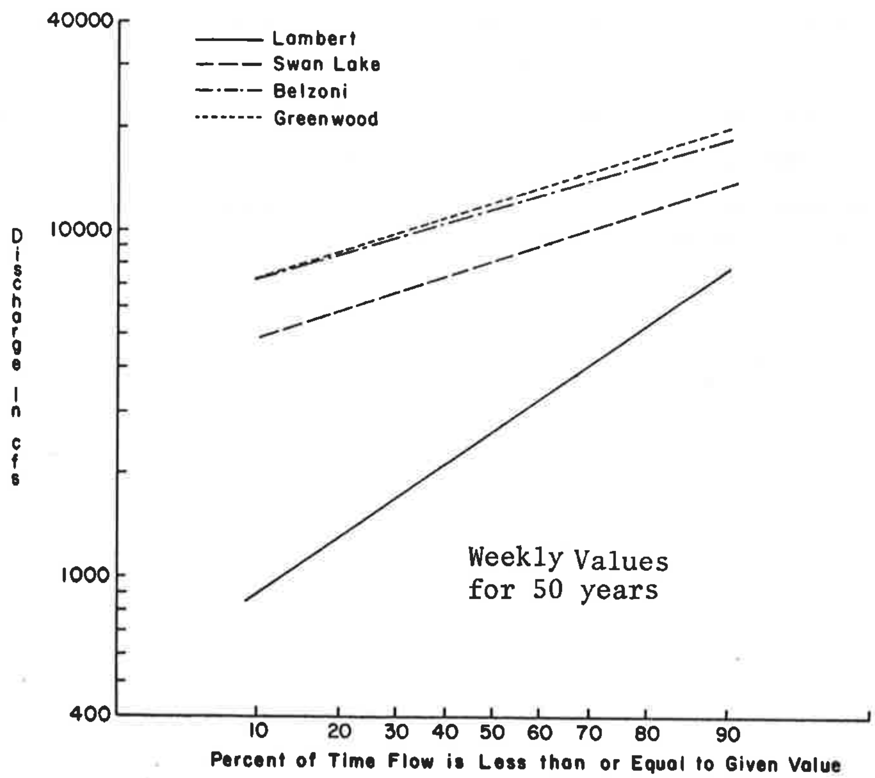


Figure 13. Flow duration for four main stem gages.

the 11-year observed hydrograph for similar discharges. In addition, the comparison of flows larger than 25,000 cfs at some key stations is given in Table 2. This shows that the 50-year synthesized hydrograph is statistically comparable to the original statistics. This similarity can be seen by comparing the observed and synthesized hydrographs as shown in Figures 14a and 14b. These figures indicate the time series analysis retains the cyclical nature and relative magnitude of the observed data. The 50-year synthesized hydrographs at Greenwood, Belzoni, Swan Lake, and Lambert are given in Appendix B.

Table 2. Main stem and tributary stations with flows over 25,000 cfs.

Station	11 years Daily Flow		11 years Weekly		50 years Weekly	
	Number	Percent	Number	Percent	Number	Percent
Yazoo at Mouth of Big Sunflower	1129	28.10	163	28.40	776	29.74
Yazoo at Belzoni	120	2.99	16	2.79	23	0.88
Yazoo at Greenwood	157	3.91	21	3.66	44	1.69
Yalobusha at Grenada Town	2	0.05	-0-	-0-	-0-	-0-
Tallahatchie at Swan Lake	39	0.97	6	1.05	10	0.38
Tallahatchie near Lambert	-0-	-0-	-0-	-0-	-0-	-0-
Tallahatchie at Locopolis	30	0.75	3	0.52	4	0.15

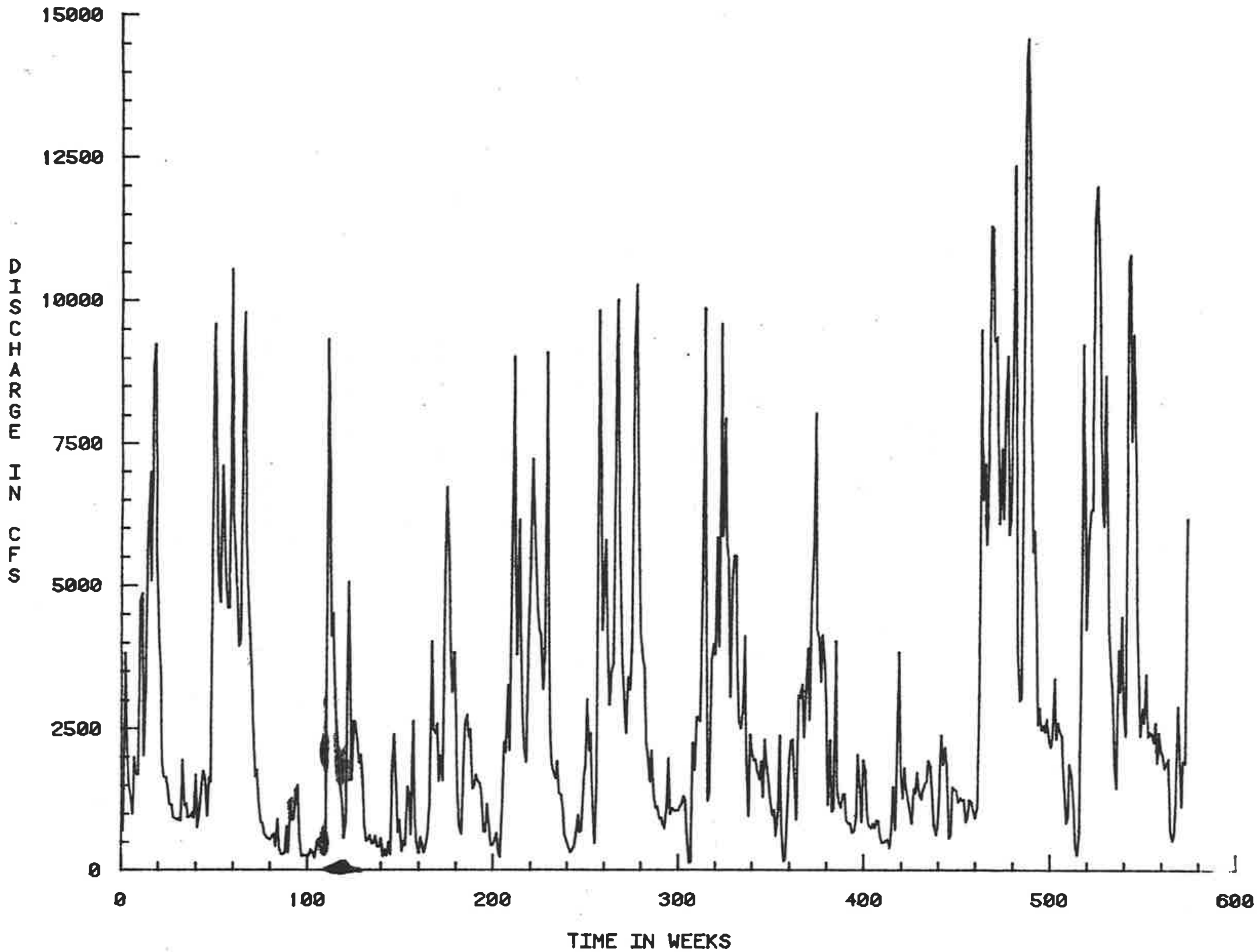


Figure 14a. Seven-day average discharge for Tallahatchie River near Lambert observed at station No. 405.

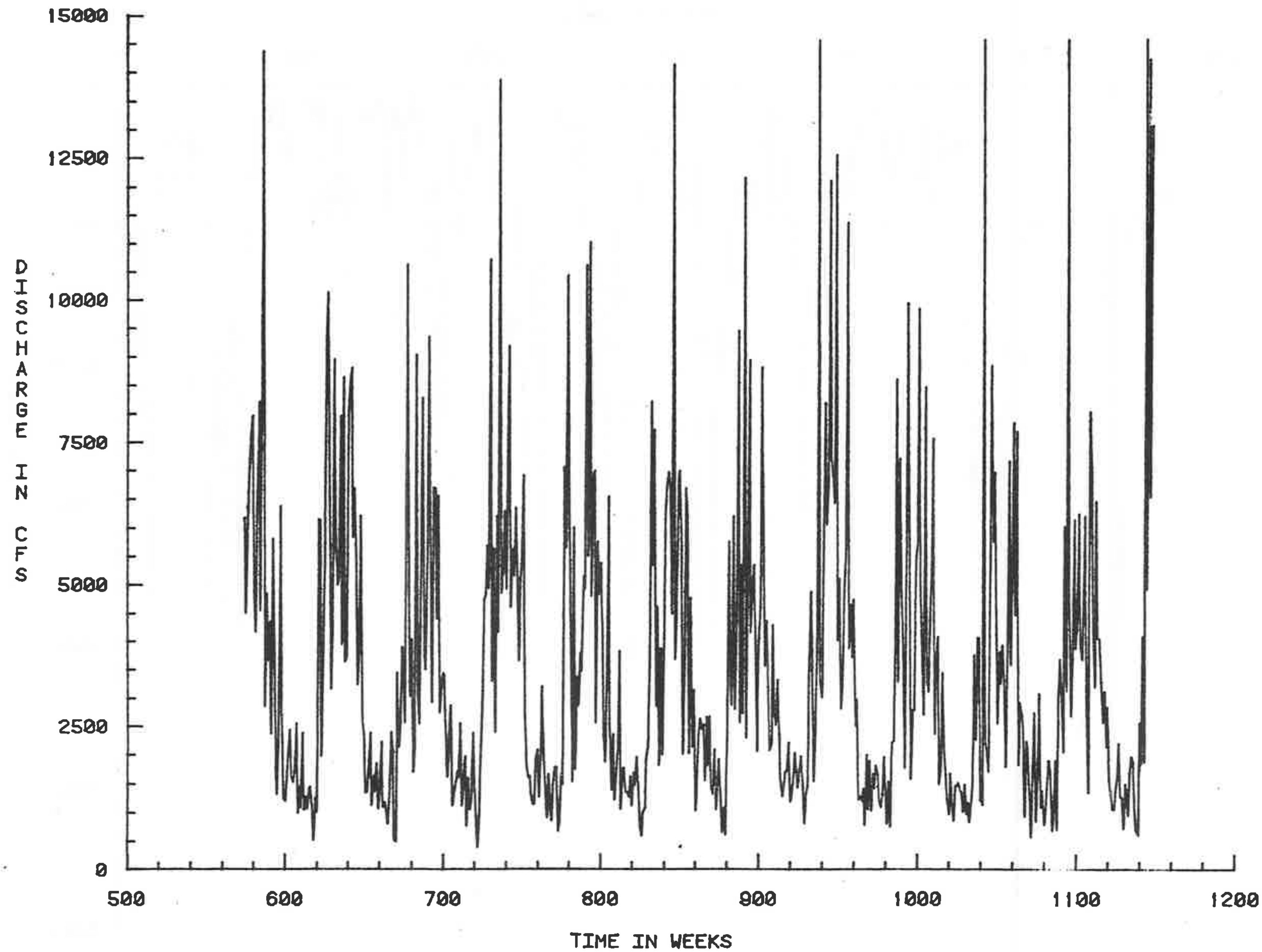


Figure 14b. Seven-day average discharge for Tallahatchie River near Lambert generated for station No. 405.

IV. UNSTEADY FLOW WATER AND SEDIMENT ROUTING MODEL

4.1 General Features of the Unsteady Flow Model

Several alternative study runs were identified and utilized to determine the most feasible approach for improving the Yazoo River Basin. The major problems involved with river development are generally derived from changes in river hydraulics and geomorphology. Since a local change in a river reach will affect a large river area, and since induced impacts (especially the flood wave movement) will change with time, it was necessary to develop a mathematical model capable of evaluating the unsteady flow water and sediment related problems in a large river basin.

A one-dimensional mathematical model capable of routing water and sediment through the river system was developed to study the short- and long-term response of the river to natural and man-induced activities in the Yazoo River main stem system. This unsteady flow model was utilized to examine the applicability of the known discharge model, also developed in this study. The unsteady flow model was formulated with one-dimensional partial differential equations representing the conservation of mass for sediment, and the conservation of mass and momentum for sediment-laden water. Effects of the hydraulic structures, interactions between the Yazoo main stem and its main tributaries, and flow and sediment distributions at Greenwood Bendway and Ft. Pemberton Cutoff were studied. Partial differential equations were solved by an implicit finite-difference method using a digital computer.

The information in this chapter describes the construction, calibration, and application of the unsteady flow water and sediment routing model for the Yazoo main stem system. Detailed information concerning the unsteady flow model is given in the "User's Manual for Unsteady Flow Water and Sediment Routing Model."

4.2 Construction and Calibration of the Mathematical Model

The unsteady flow model of the Yazoo River system covers the Yazoo River main stem from Arkabutla (river mile 307.5) to Belzoni (river mile 116.2), the P-Q Floodway from mile 8.8 to the mouth, and the Yalobusha River from mile 7.95 to the mouth. The bifurcation of the Greenwood Bendway and Ft. Pemberton Cutoff is also modeled. There are numerous tributaries, creeks, and bayous flowing into the Yazoo River main stem. Based on an order of magnitude analysis, two major tributaries (P-Q Floodway and Yalobusha River) are included in the unsteady flow main stem model.

Creeks carrying significant water and sediment into the main stem model include Lake Cormorant Bayou, Arkabutla Creek, Tillatoba Creek, Big Sand Creek, Pelucia Creek, and Abiaca Creek. Nonpoint water source inputs are also considered in the model. An index diagram of the modeled Yazoo River system is shown in Figure 15. The river reach was divided into 79 sections with space increments ranging from 0.1 mile at the confluence to 9.0 miles in the Coldwater River. Available field data used to construct and calibrate the mathematical model are described below. These data are almost identical to that used in the known discharge model (Chapter V).

1. The cross-sectional geometries at the selected 79 cross sections were obtained from periodic cross-sectional surveys by the U.S.

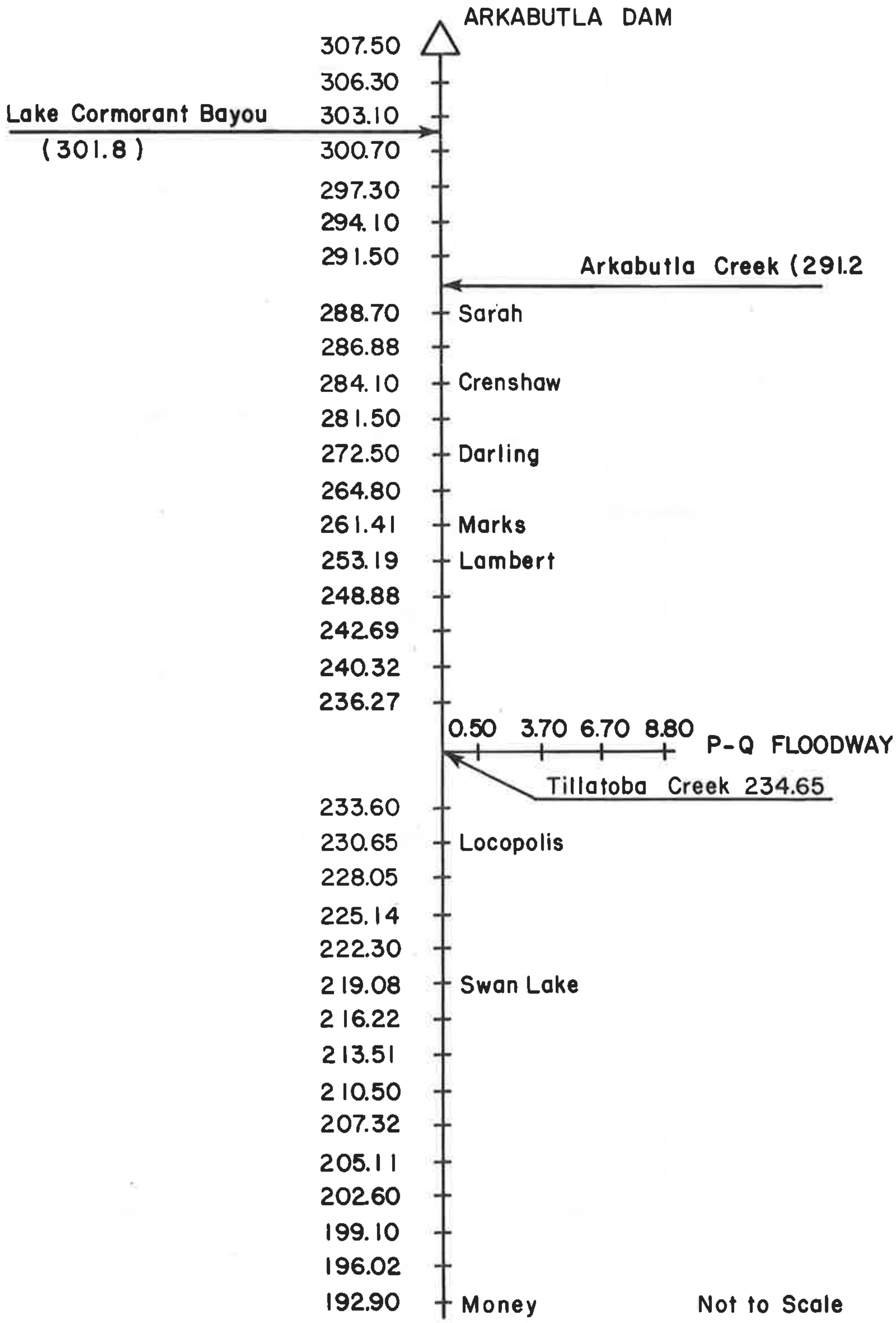


Figure 15. Index diagram of the Yazoo River main stem model.

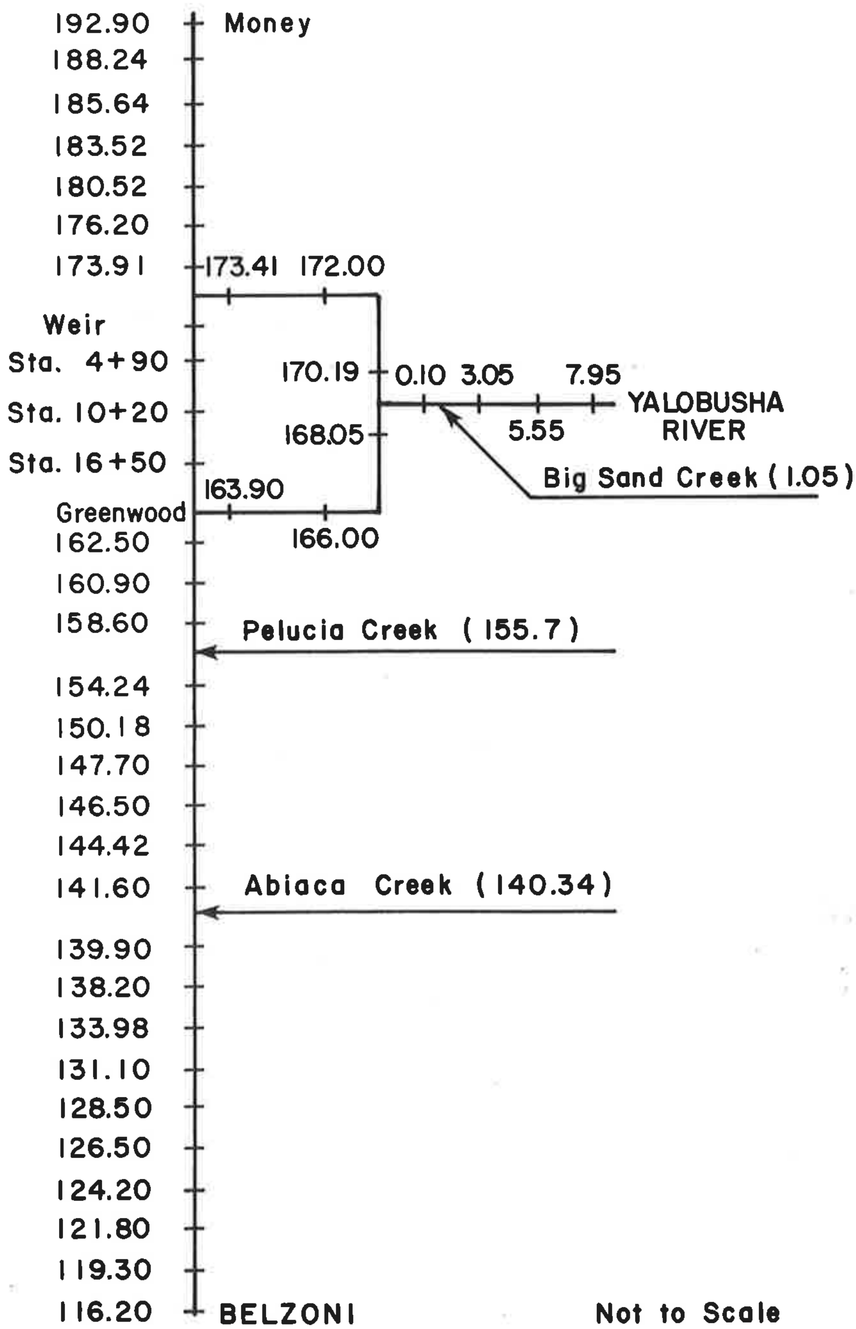


Figure 15. (continued)

Army Corps of Engineers. The most recent data were used except for the Ft. Pemberton Cutoff where cross-sectional geometries from 1973 and 1974 were used in calibrating the sediment routing model.

2. The 1973 and 1974 discharge and stage hydrographs at Belzoni, Greenwood, Money, Swan Lake, Locopolis, Lambert, Marks, Darling, Sledge, Crenshaw, Sarah, Prichard, and Arkabutla Dam in the Yazoo main stem and gaging stations were used to determine the point and nonpoint source input based on conservation of water volume in unit time. The discharge and stage hydrographs were also used to evaluate resistance to flow in the rivers.

Using this information, the following relations were evaluated at all 79 sections in the Yazoo main stem model:

1. Geometric properties of the river sections including cross-sectional areas, top widths, and hydraulic radius were estimated from the cross-sectional geometries. By using the cross-sectional coordinates input to the computer program these variables were directly calculated for a given stage.

2. The Manning equation was employed to relate friction slope to the flow and channel characteristics. The Manning roughness coefficients were determined from the steady water surface profiles for given discharges, where stage-discharge relationships were assessed from the stage and discharge hydrographs measured at the gaging stations. The Manning roughness coefficients were expressed as a function of discharge. Typical values vary from 0.017 to 0.042 in the main channel, and 0.08 to 0.20 on the floodplains.

3. Sediment discharge was related to the flow and channel characteristics by a sediment transport function. Using the Modified

Einstein procedure to compute the total sediment load of suspended sediment discharge data separating the bed-material load and wash load, the following relation was established in the Yazoo main stem:

$$C_s = 4.48 \times 10^{-6} V^{2.16} D^{-0.06} \qquad (1)$$

where C_s is the mean concentration of bed-material load on a volume basis, V is the mean velocity in fps, and D is the hydraulic radius in feet. Porosity was estimated to be 0.3.

4. Sediment discharges (in cfs) contributed from the point sources were computed from the relation:

$$Q_{s_\ell} = a\, Q_\ell^b \qquad (2)$$

where Q_ℓ is the lateral point water discharge lateral inflow, and a and b are empirical coefficients listed in Table 3.

If Q_ℓ is positive (indicating inflow), Equation 2 is used to compute Q_s; otherwise, Q_s is assumed negligible. Furthermore, sediment carried by the nonpoint source flow is assumed negligible.

It was desired to reproduce the flow characteristics and geomorphic changes of the study reach from 1973 to 1974. Three 1973-1974 flow discharge hydrographs were used as upstream boundary conditions for the modeled reaches of the Yazoo main stem, P-Q Floodway, and Yalobusha River. The discharge hydrographs were measured at Arkabutla Dam on the Yazoo main stem; and synthesized by adding flow discharge data from Batesville, Peters Creek, and Enid Dam to represent the discharge on the P-Q Floodway and by adding flow discharge from Whaley, Potococowa Creek, Ascalmore Creek, and Teoc Creek to account for the flow on the Yalobusha River. The downstream boundary condition utilized was the stage-discharge rating curve at Belzoni.

Table 3. Coefficients for calculating sediment discharge in tributaries for the unsteady flow model.

POINT SOURCE	a	b
Arkabutla Creek	0.15×10^{-8}	2.40
Pelucia Creek	0.75×10^{-7}	2.30
Abiaca Creek (Natural condition)	0.15×10^{-7}	2.40
Tillatoba Creek	0.40×10^{-8}	2.40
Big Sand Creek	0.10×10^{-7}	2.40

When the discharge hydrographs were routed through the modeled river reach, the water discharge, velocity, water surface elevation, sediment discharge, erosion and deposition on cross sections, and thalweg elevation changes were calculated at each step. Size of the time step varied from three hours to two days depending on the rate of change in flow discharge. A larger time step was used when the rate of change was relatively smaller.

Calculated discharge and stage hydrographs from April 12, 1973 to February 23, 1974 at Swan Lake and at the Ft. Pemberton Cutoff are compared with the measured stages in Figures 16 to 19. In general, these figures show an acceptable agreement between the measured and calculated values. Deviation in the computed and measured discharges at Swan Lake could be improved if the lateral flows from the point and nonpoint sources were more accurately evaluated. Discharge through the Ft. Pemberton Cutoff is significantly affected by channel characteristics in the Greenwood Bendway, where the Yalobusha River and Big Sand Creek are influential. Because the 1977 cross sections in the Greenwood Bendway area were used in the model to route the 1973 hydrograph, and

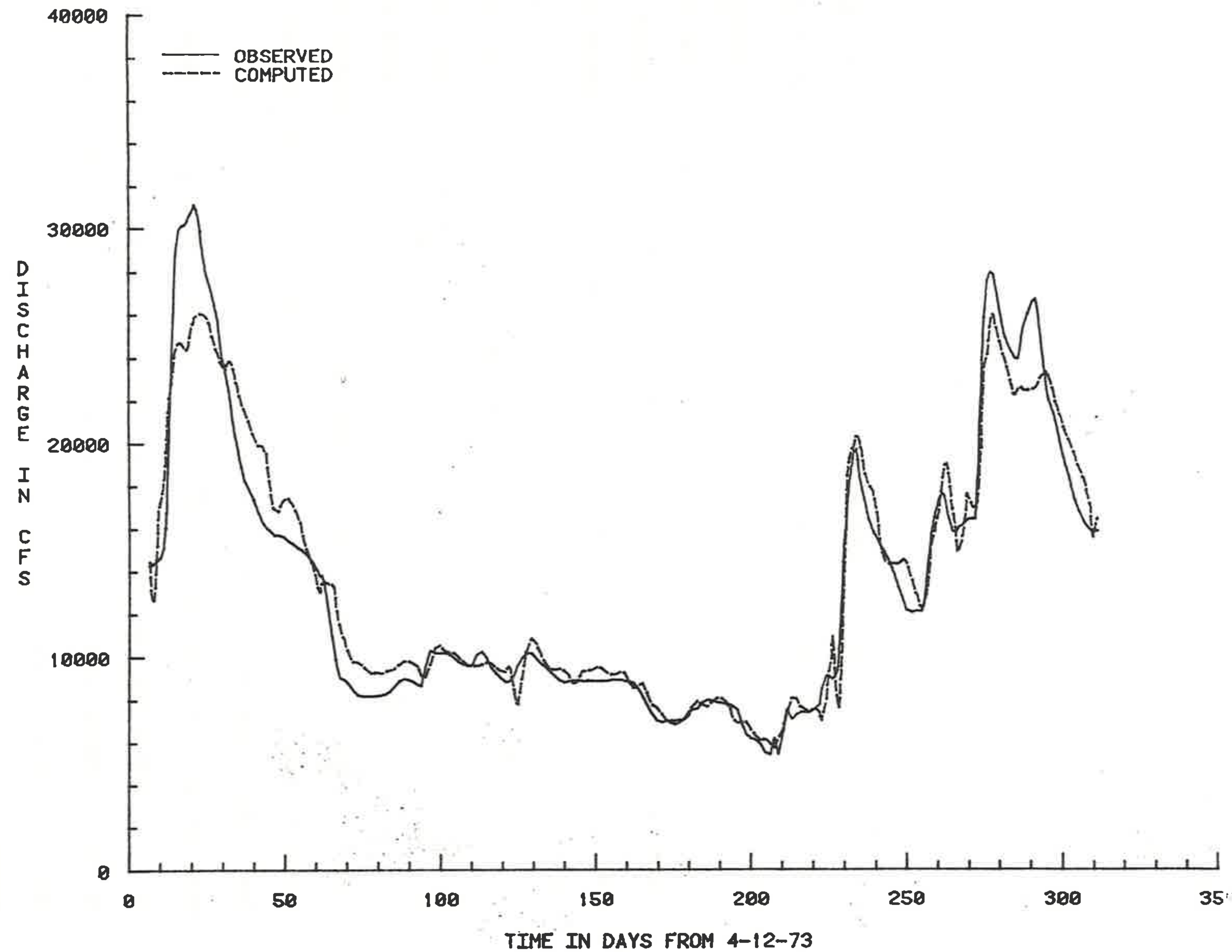


Figure 16. Computed and measured discharge hydrographs at Swan Lake utilizing the unsteady flow model (started April 12, 1973).

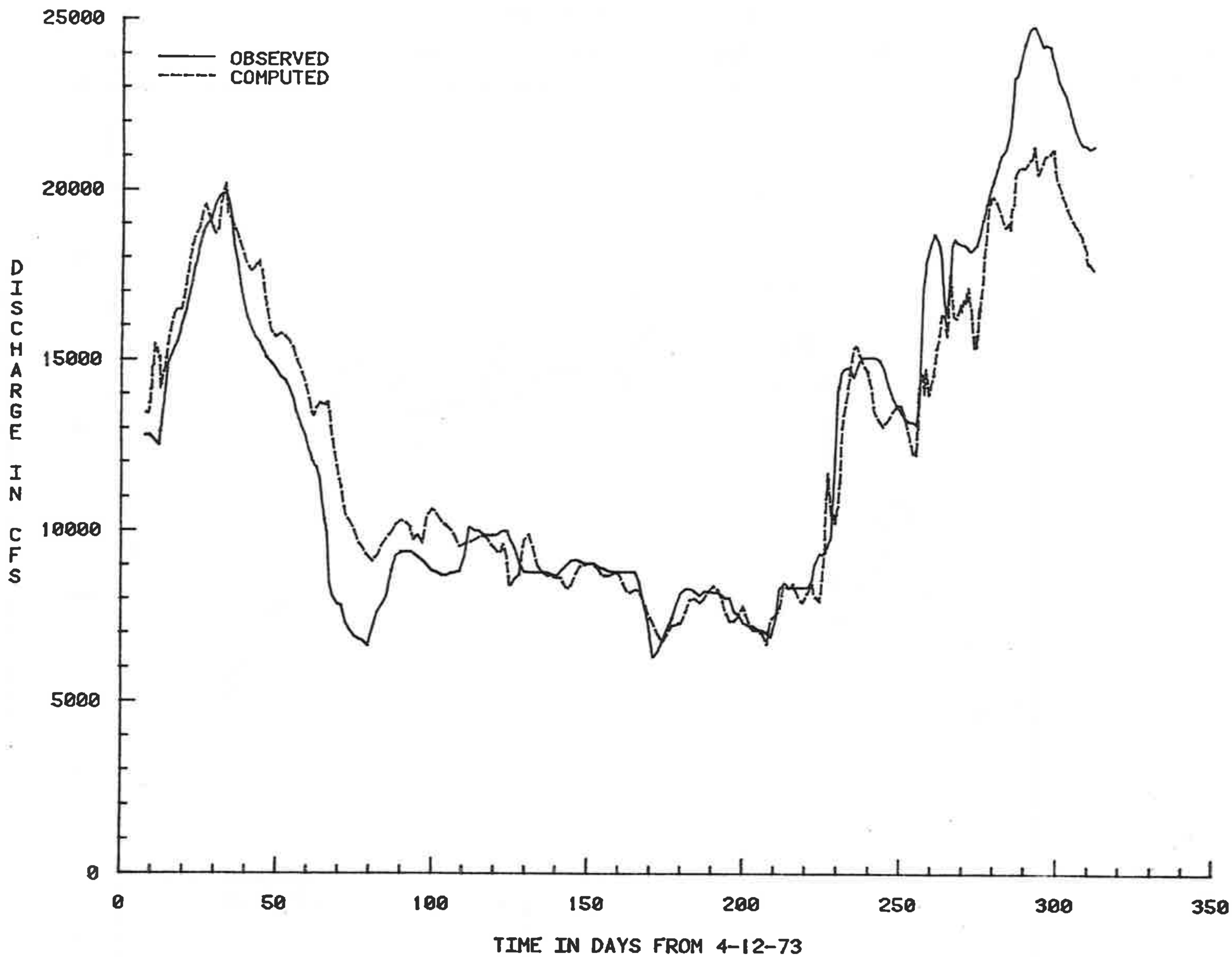


Figure 17. Computed and measured discharge hydrographs at Ft. Pemberton utilizing the unsteady flow model (started April 12, 1973).

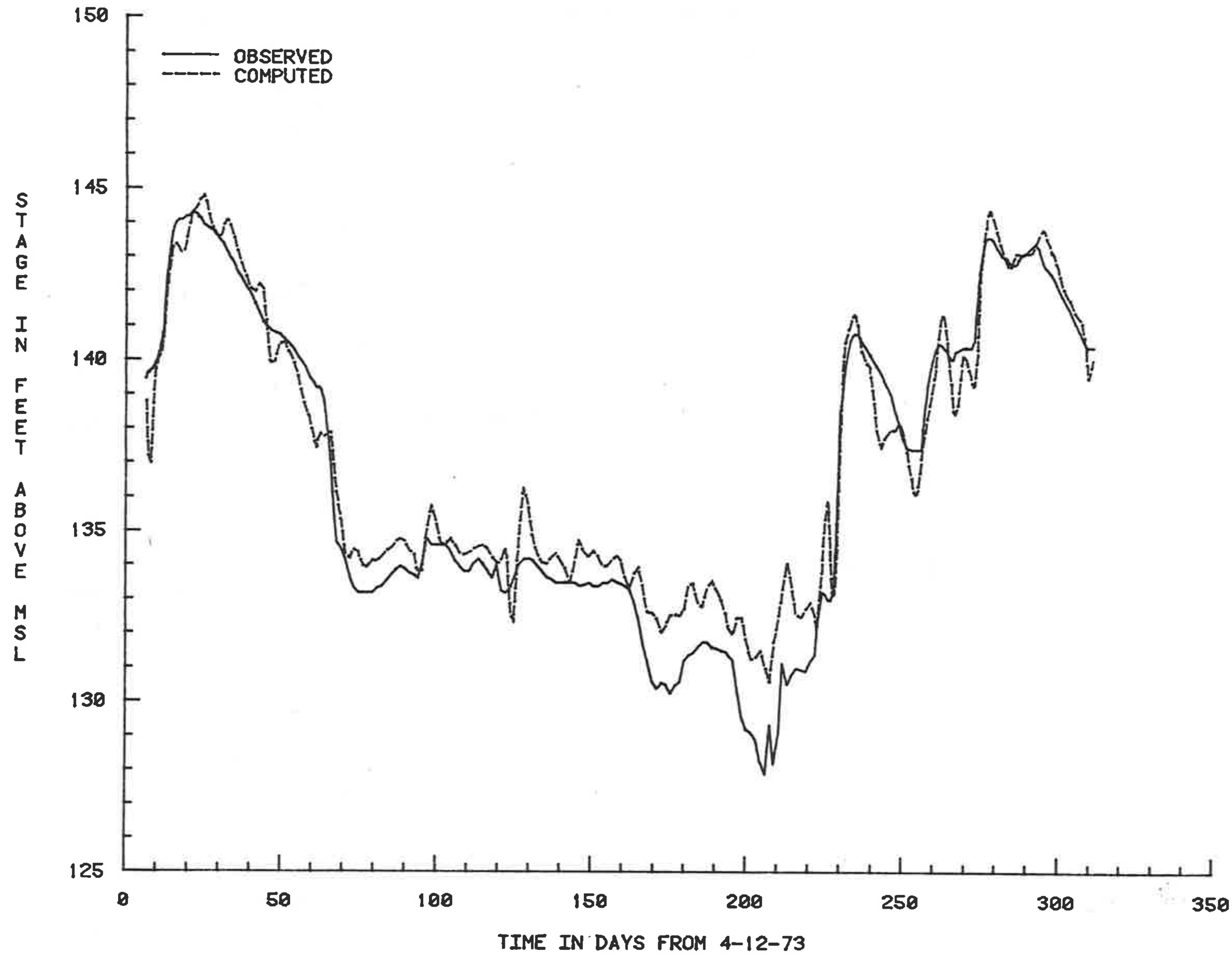


Figure 18. Computed and measured stage hydrographs at Swan Lake utilizing the unsteady flow model (started April 12, 1973).

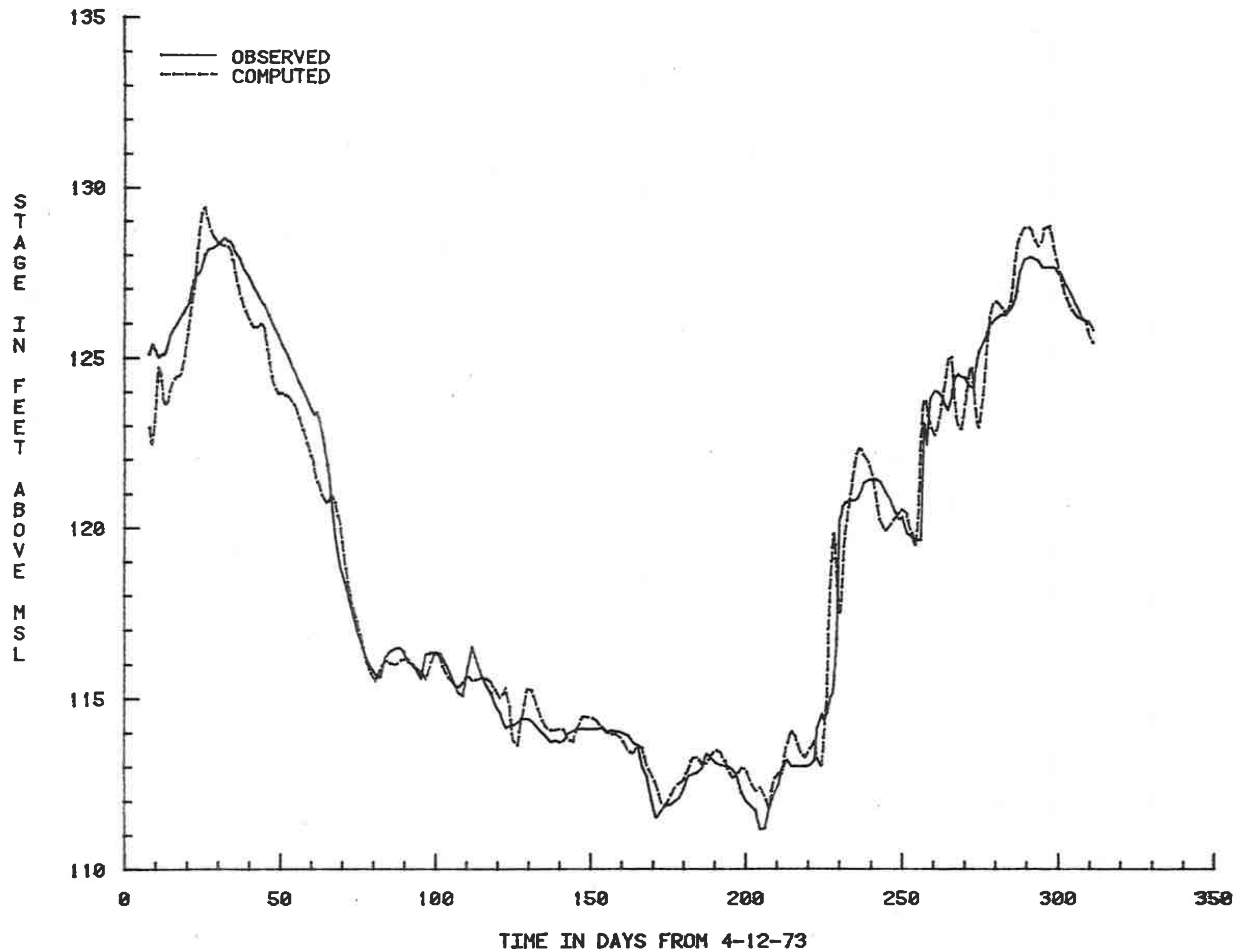


Figure 19. Computed and measured stage hydrographs at Ft. Pemberton utilizing the unsteady flow model (started April 12, 1973).

because this river reach was very active, it was expected that the flow discharge distribution between the Ft. Pemberton Cutoff and Greenwood Bendway would differ from that which actually occurred. This model reach can be updated and improved by using recently collected river data near Greenwood.

During the same routing time period, computed changes in the cross-sectional geometries and thalweg elevations at the Ft. Pemberton Cutoff are compared with the measured data in Figures 20 through 23. Both the measured and computed results show general erosion in the cutoff, but the computed erosion was smaller. This could result from smaller computed water discharges compared to those measured through the cutoff. Changes in cross-sectional shapes were assumed to be proportional to the relative magnitude of conveyance at each subsection of a cross section. As shown in Figures 20 through 23, the agreement between the measured and computed cross-sectional changes is good, indicating the assumed sedimentation distribution utilizing conveyance is valid.

To further verify the mathematical model, the 1974 discharge hydrographs were routed through the calibrated mathematical model. It was found that the mathematical model also simulated the 1974 flow conditions reasonably well (Figures 24 and 25). Since the calculated results generally agreed with the measured data, it was concluded that the mathematical model as calibrated and verified was as reliable as the available field information, and could be employed to estimate the river's response to future development. Applicability and reliability of the model can be improved by critical evaluation of the point and nonpoint sources with improved knowledge of flow distribution between Ft. Pemberton and Greenwood Bendway, and with further examination of the

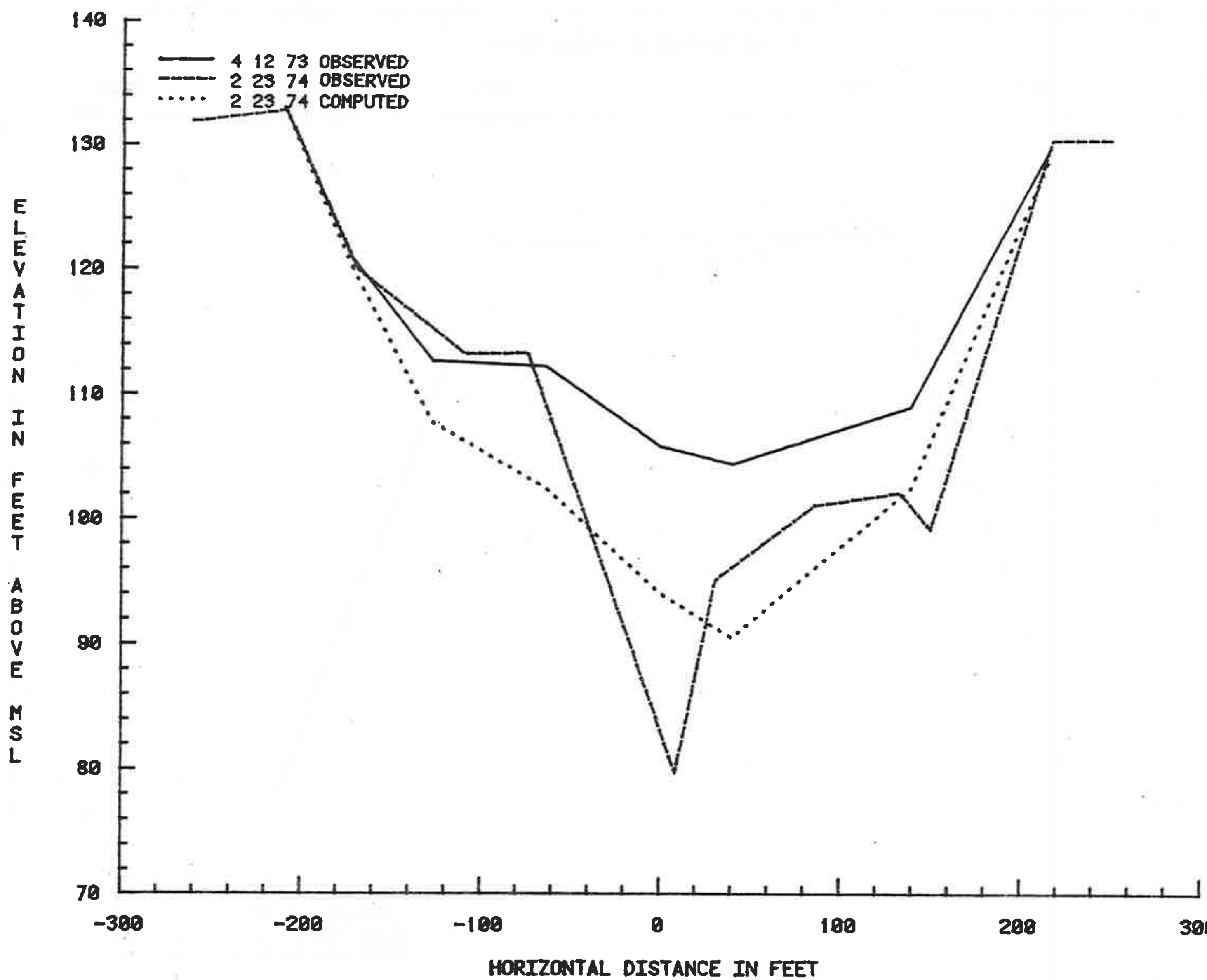


Figure 20. Comparison between computed and measured cross-sectional changes at Ft. Pemberton utilizing the unsteady flow model (Station 4+90).

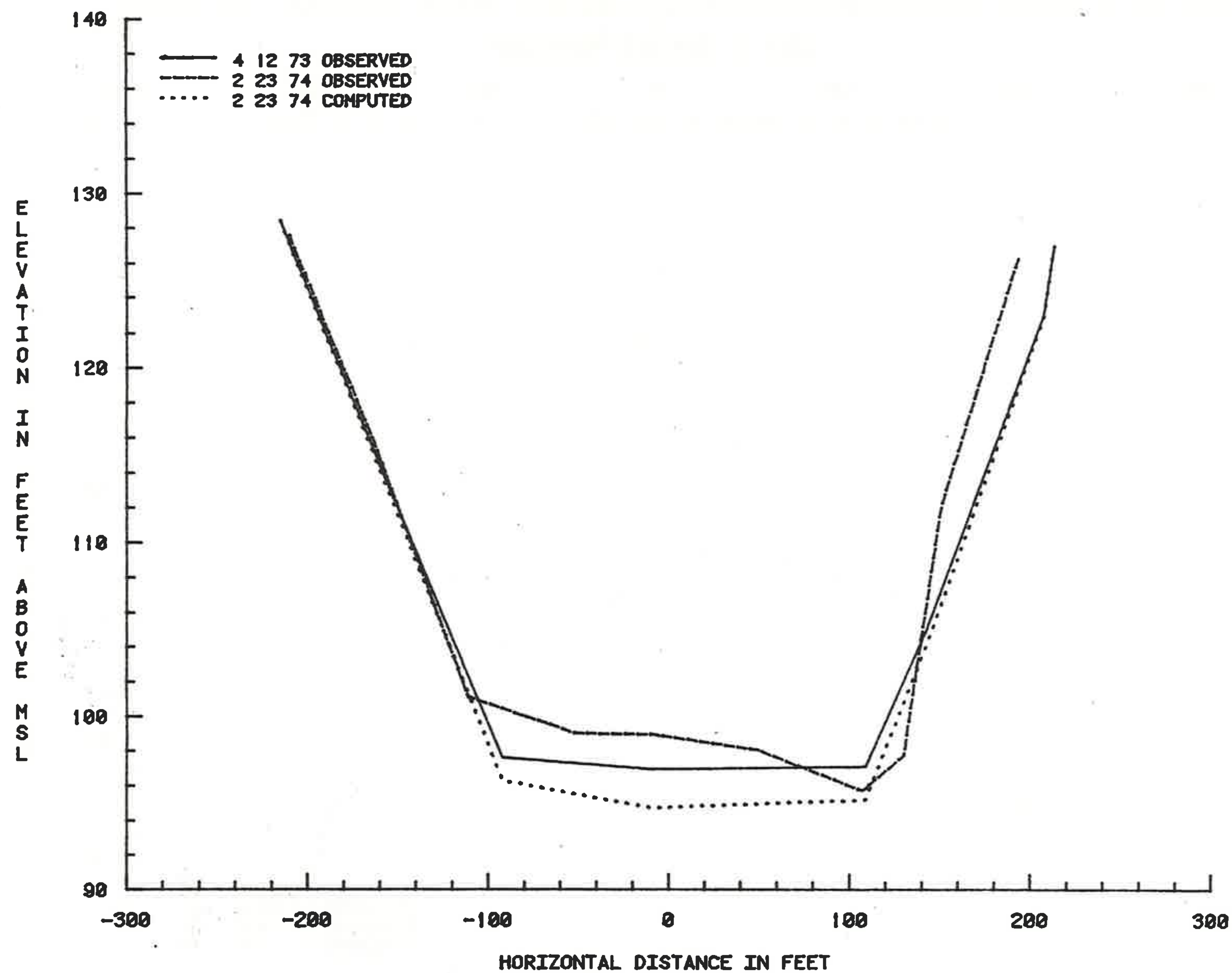


Figure 21. Comparison between computed and measured cross-sectional changes at Ft. Pemberton utilizing the unsteady flow model (Station 10+20).

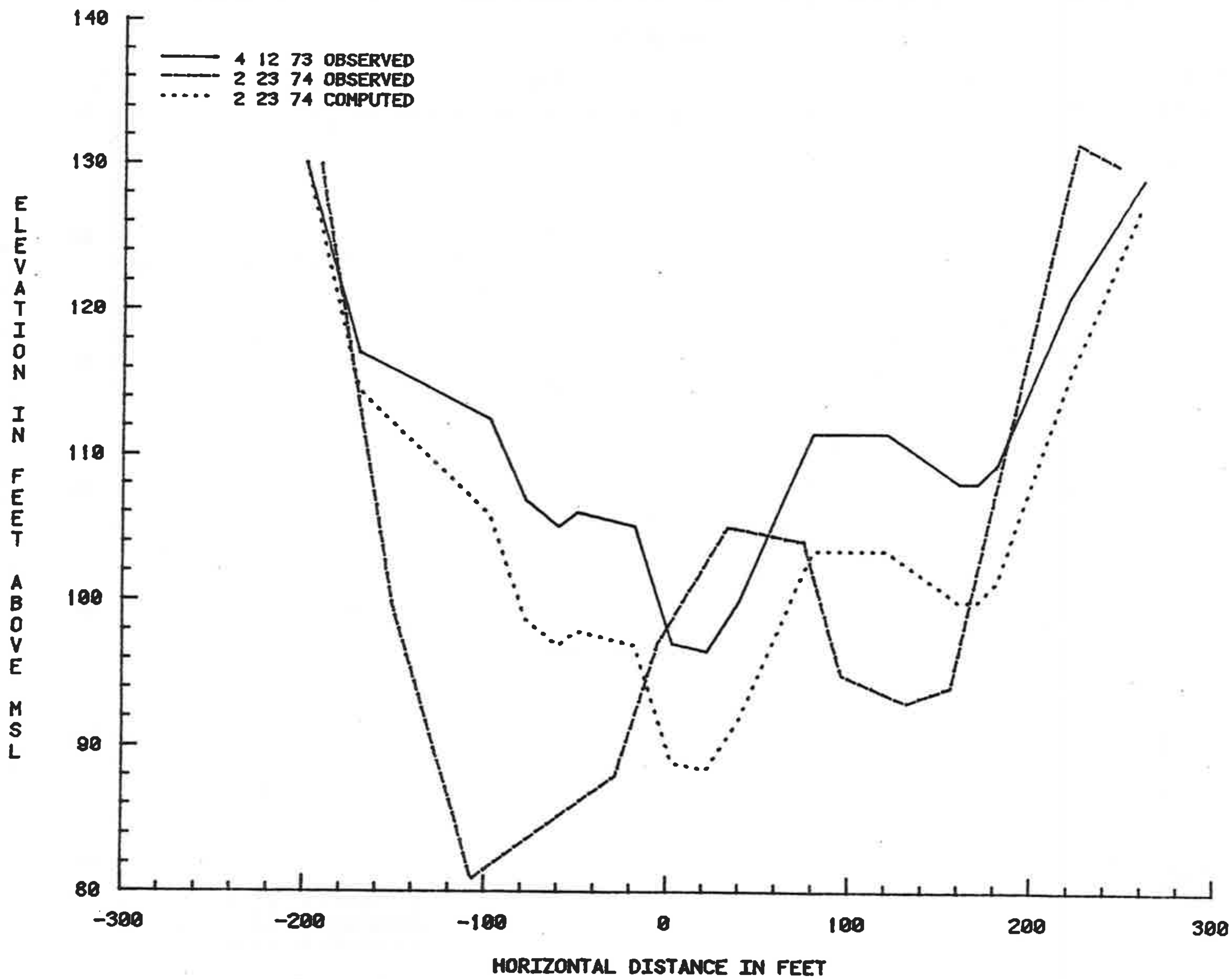


Figure 22. Comparison between computed and measured cross-sectional changes at Ft. Pemberton utilizing the unsteady flow model (Station 16+50).

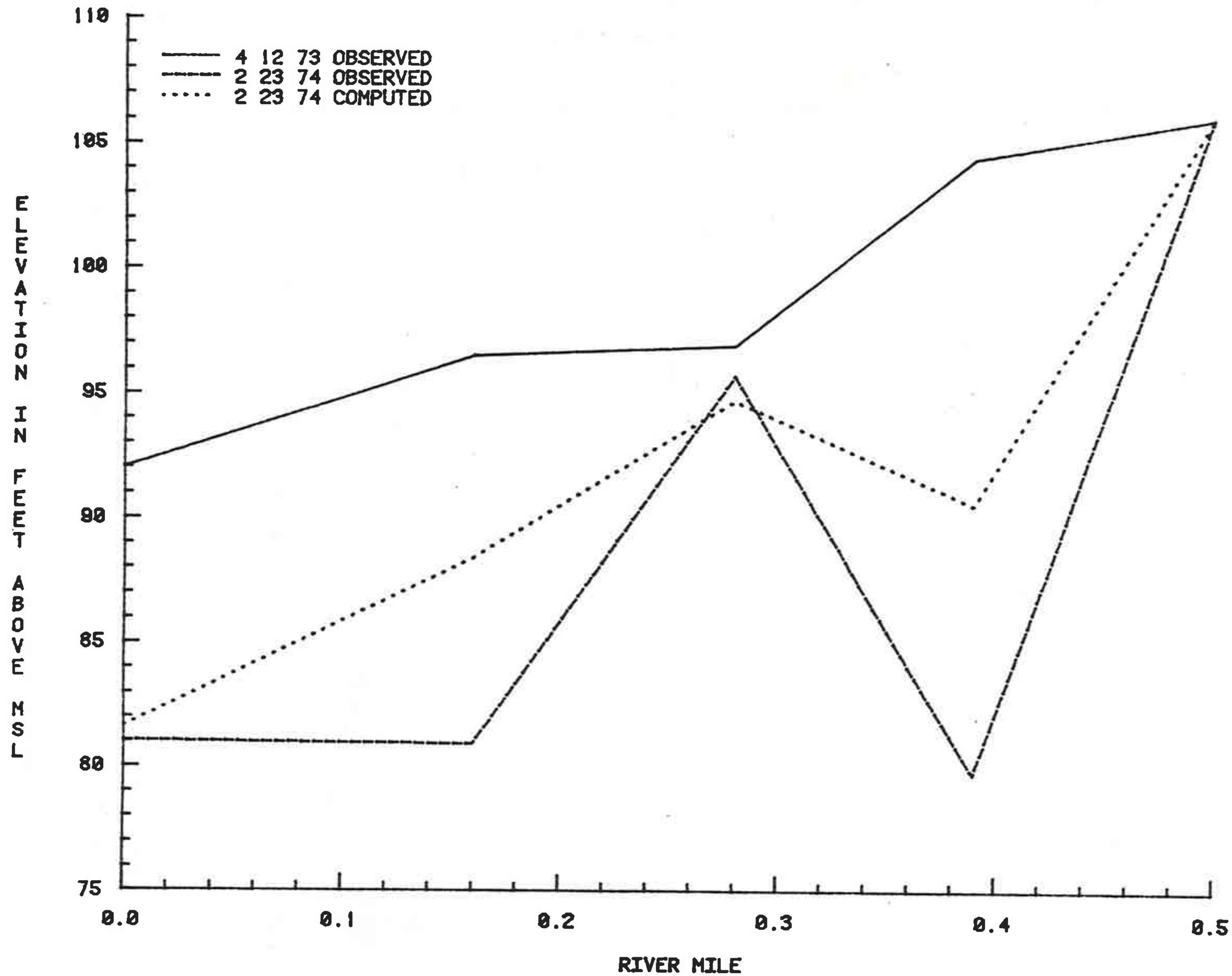


Figure 23. Unsteady flow model calibration of bed elevation at Ft. Pemberton.

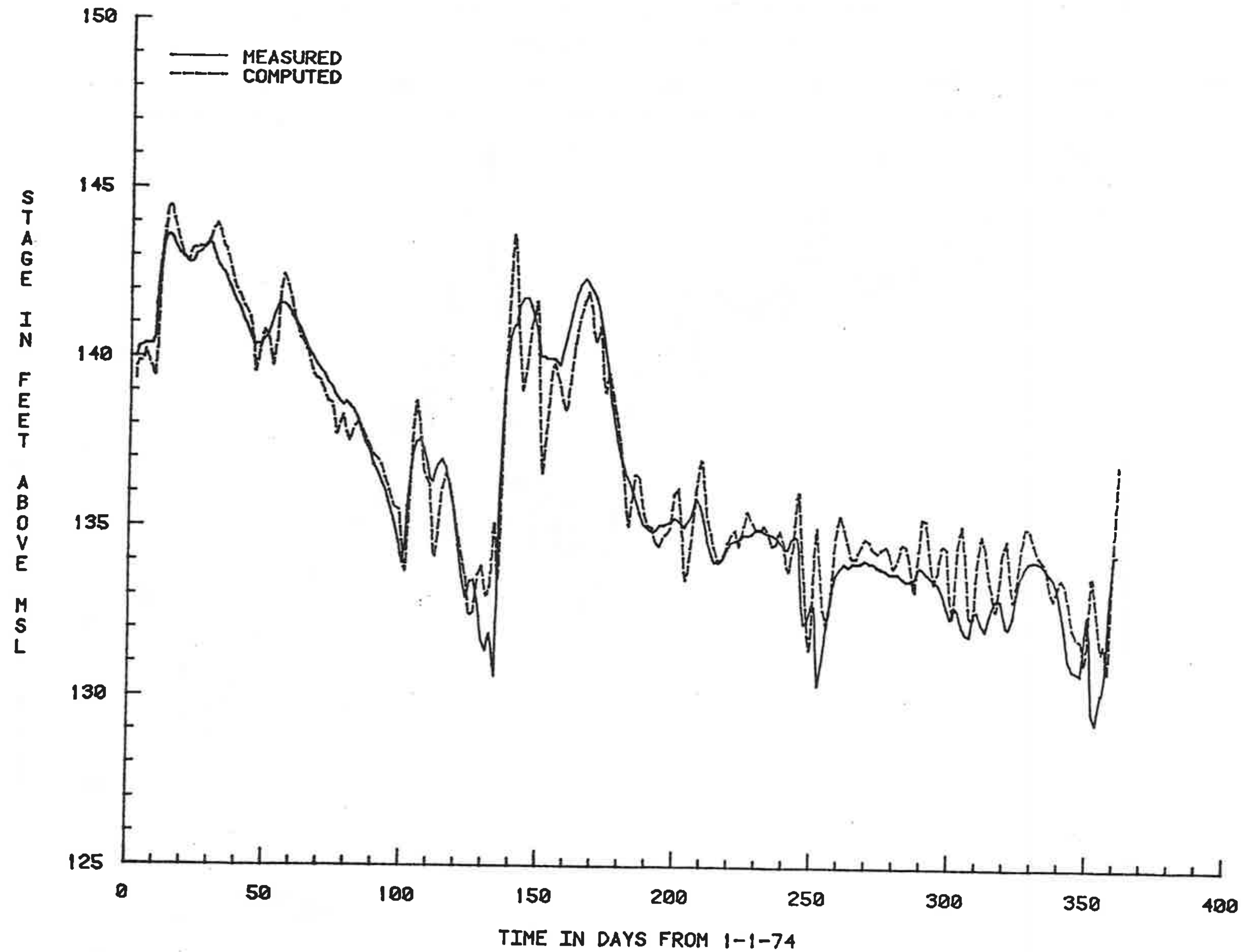


Figure 24. Computed and measured stage hydrographs at Swan Lake utilizing the unsteady flow model (started January 1, 1974).

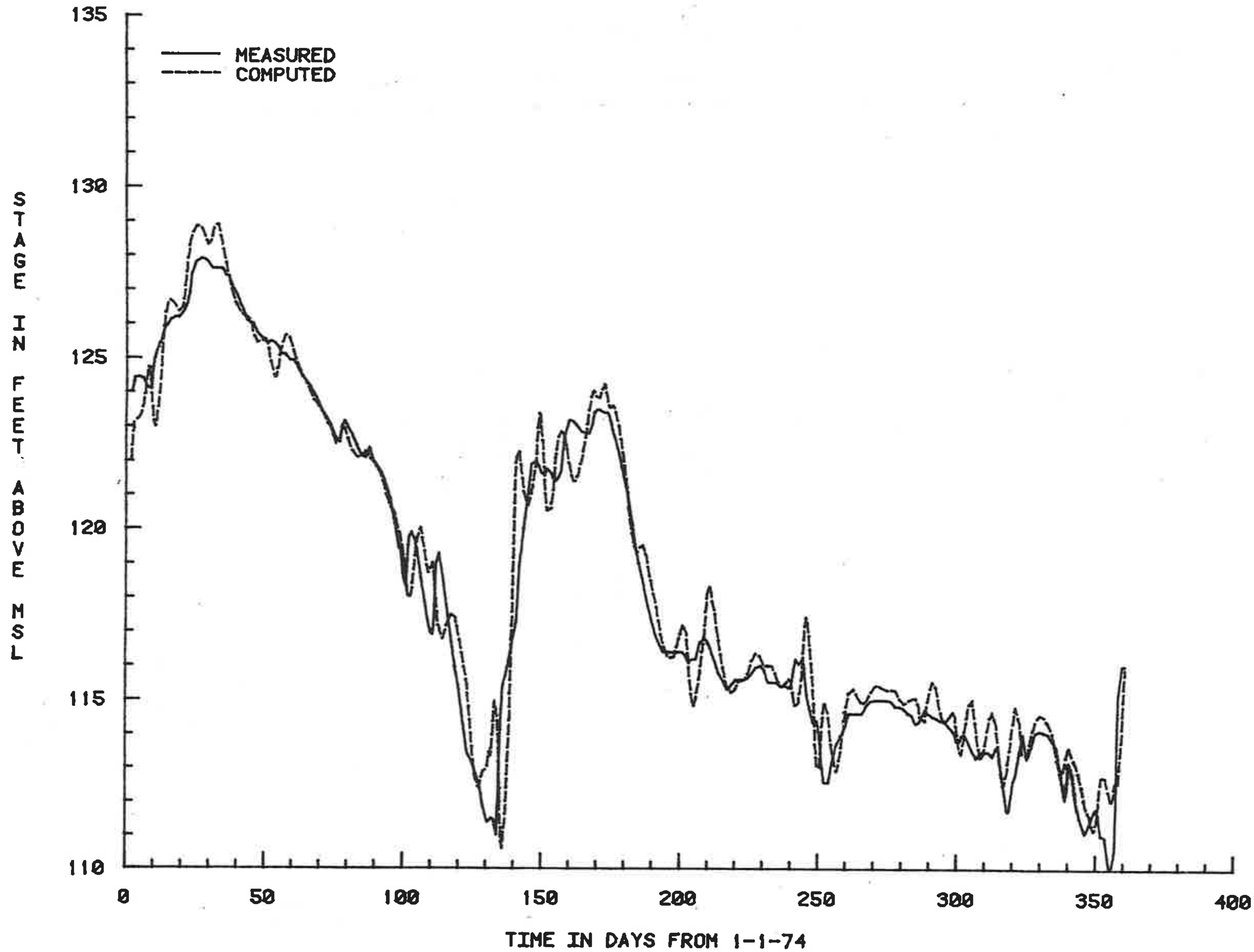


Figure 25. Computed and measured stage hydrographs at Ft. Pemberton utilizing the unsteady flow model (started January 1, 1974).

geomorphic changes in the Yazoo main stem. The current Yazoo Basin data base is not adequate for modeling the entire system utilizing the unsteady flow model. The known discharge model was used more extensively than the unsteady flow model. The unsteady flow model was used to evaluate Runs No. 9 and 10, and to examine the applicability of the known discharge model.

V. KNOWN DISCHARGE MODEL

A known discharge, uncoupled, sediment routing model was developed and used in this study. The model was calibrated and verified for two separate reaches of the river with good results. The following section includes a brief description of the model, the two reaches that were calibrated, the calibration procedure, and the verification of results. Detailed information concerning the known discharge model is given in the "User's Manual for Known Discharge Sedimentation Model."

5.1 Model Features

5.1.1 Known Discharge Assumption

This program is a known discharge or steady flow model. It assumes that during any one time period, water discharge is constant along a reach of river, except where lateral inflows occur. Although a model of this type cannot predict the dynamic effects that an unsteady model can, it requires considerably less computer time. However, the model is still able to calculate flood stages and provide a practical method to model sediment movement over long time periods. The time increment on the input hydrographs may vary from a few hours to a month or longer depending on the flow conditions and the required accuracy of the results. In the model calibration, a time increment of one day was used.

5.1.2 Uncoupled Routing Procedure

Water and sediment routing are uncoupled. This makes the bed profile constant during any one time increment with changes in the bed profile due to the sediment movement introduced at the end of each time increment.

5.1.3 Sediment Transport Equations

The bed material sediment transport at each cross section was calculated by a transport equation that was derived from the Yazoo River sediment discharge measurements. The equation is:

$$Q_s = 4.48 \times 10^{-6} \, W \, V^{3.16} D^{0.94} \quad (3)$$

where Q_s is the sediment transport in cfs, W is the average channel width in feet, V is the average water velocity in feet per second, and D is the average channel depth in feet. While this equation is not applicable to other rivers, it fits sediment data from the Yazoo River.

Sediment discharges contributed from the point inputs were computed from Equation 2. Coefficients used in the known discharge model are listed in Table 4.

5.1.4 Channel Geometry

River geometry data required by the model includes digitized channel cross sections, river distance between cross sections, horizontal location of the banks, and the Manning's n value for the main channel and overbank areas. The model uses this digitized geometry data to determine geometry equations for each cross section. These geometry equations define conveyance, width, depth, and area of the channel as a function of the maximum depth. The program uses these channel geometry equations for backwater calculations.

5.1.5 Backwater Calculations

The backwater curve is calculated by using an iterative first order Newton-Raphson approximation to solve the total head equation at each cross section. Since the channel geometry equations are used to describe the hydraulic properties, the first derivative of the total

Table 4. Coefficients for calculating sediment discharge in tributaries for the known discharge model.

Point source	a	b
Short Creek	1.2×10^{-8}	2.4
Piney Creek	5.0×10^{-9}	2.4
Fannegusha Creek	1.0×10^{-8}	2.4
Black Creek	1.0×10^{-8}	2.4
Techeva Creek	1.0×10^{-8}	2.4
Tchula Lake	1.5×10^{-9}	2.2
Abiaca Creek*	1.5×10^{-8}	2.4
Pelucia	7.5×10^{-8}	2.3
Tillatoba Creek	4.0×10^{-9}	2.4
Arkabutla Creek	1.5×10^{-9}	2.4
Strayhorn Creek	1.5×10^{-9}	2.4
Lake Cormorant Bayou	4.0×10^{-9}	2.1
Big Sand Creek	1.0×10^{-8}	2.4
Potococowa Creek	2.0×10^{-8}	2.4
Teoc Creek	1.0×10^{-8}	2.4
Ascalmore Creek	1.0×10^{-8}	2.4
Cane Creek	4.0×10^{-8}	2.4
Batupan Bogue	3.0×10^{-8}	2.1
McIvor Ditch	5.0×10^{-8}	2.4
Peters Creek	1.0×10^{-9}	2.4

*Natural conditions

head equation can be evaluated analytically. This makes the backwater calculations up to 10 times faster than most trial and error methods.

5.1.6 One-Dimensional Simulation

This program is one-dimensional. This means that it can only model water and sediment in the longitudinal direction along the river. It cannot precisely model lateral phenomenon such as meandering or sediment distribution across the river cross section. While this is a limitation, there are no practical methods presently available to model multidimensional flow. However, in order to account for the lateral changes in the cross section, the degradation and aggradation were assumed to be distributed according to the relative magnitude of conveyance at a subsection in the cross section.

5.1.7 Simulation Procedure

Figure 26 shows a short flow diagram of the program operation. The program is set up in modular forms for easy updating, correction, and revision.

5.2 Description of Calibration Reaches

Two reaches of the main stem river that were calibrated were the Ft. Pemberton cutoff at Greenwood Bendway and a reach on the Tallahatchie from Swan Lake to upstream of Locopolis. These two reaches and the cross section used in the model are shown in Figures 27 and 28.

These two reaches are quite different since the Swan Lake-Locopolis reach is quite stable and experiences little bed profile change. The Ft. Pemberton reach, however, is quite unstable and experienced a 20-foot degradation during 1973 and 1974.

For the Ft. Pemberton reach, backwater computations were carried upstream, starting with a known stage-discharge relationship at Belzoni

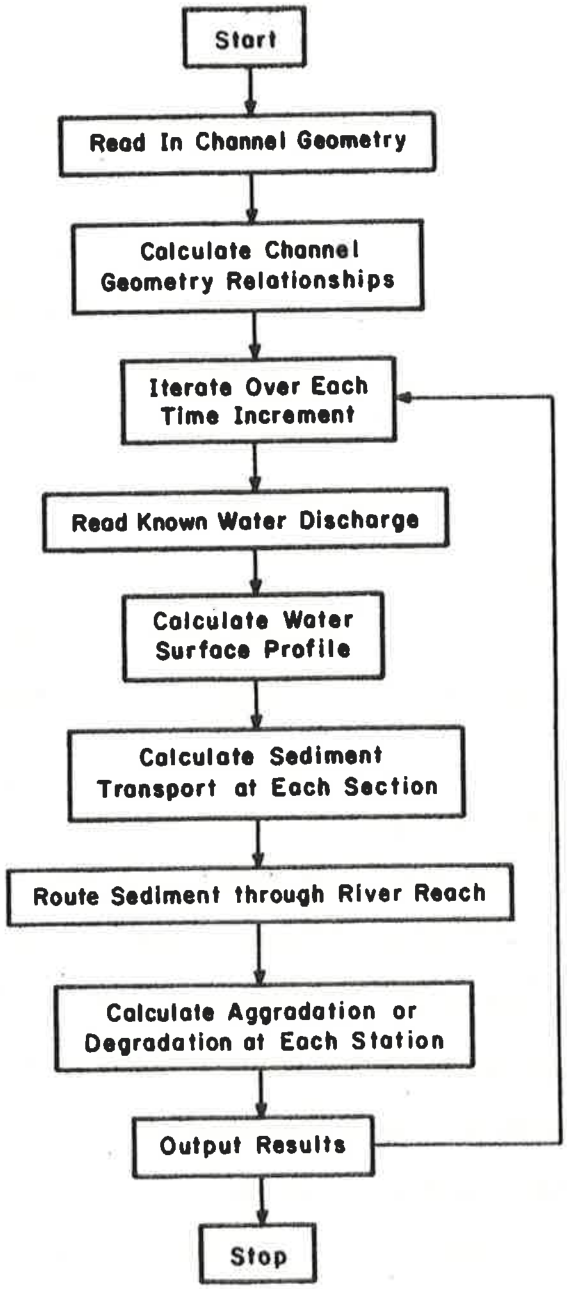


Figure 26. Flow chart illustrating program operation of the known discharge model.

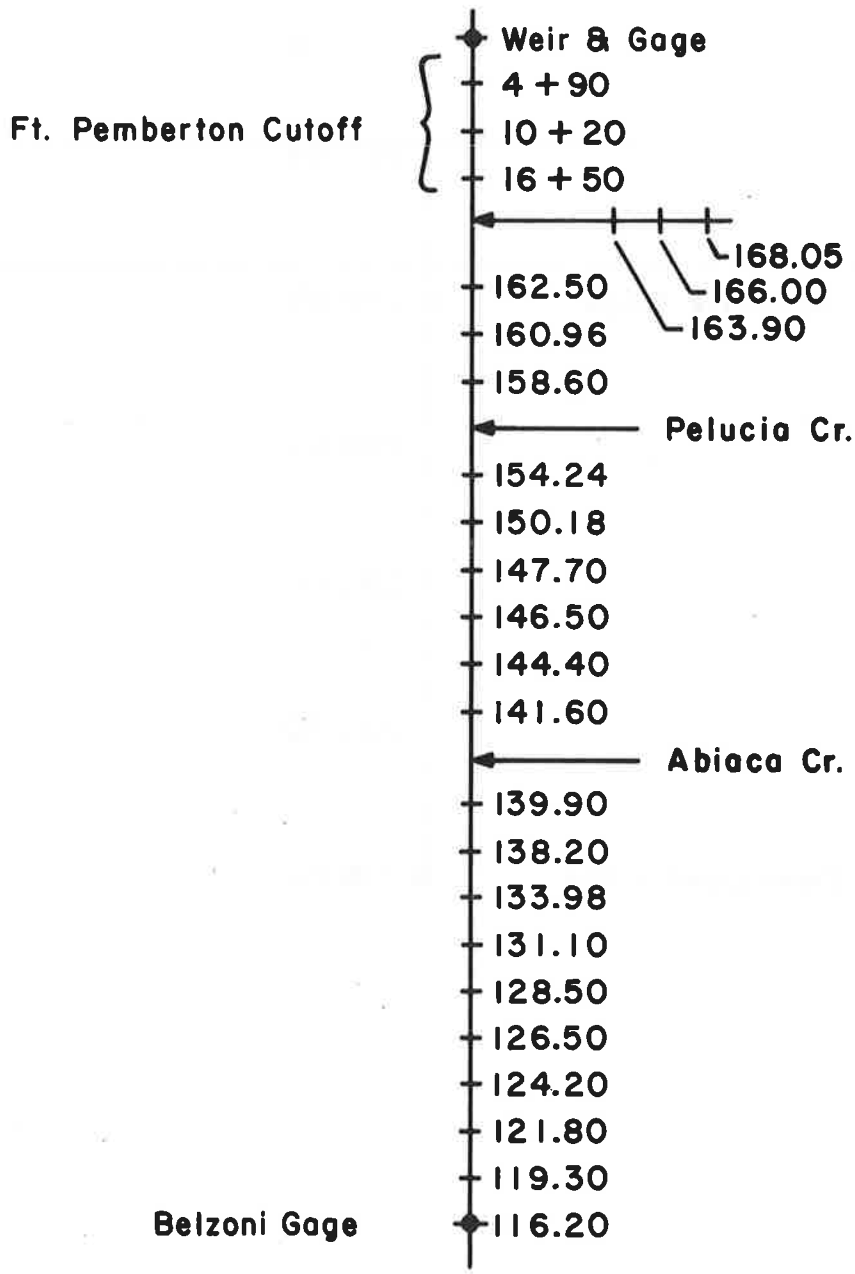


Figure 27. Calibration reach No. 1--Ft. Pemberton Reach (not to scale).

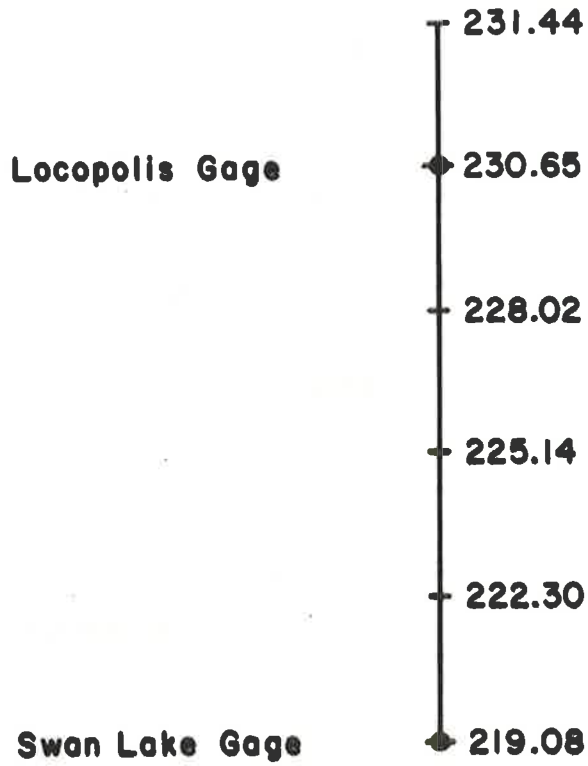


Figure 28. Calibration reach No. 2--Swan Lake-Locopolis Reach (not to scale).

Gage continuing to the Ft. Pemberton Gage. It was also necessary to carry computations a short distance up to Greenwood Bendway to obtain the correct sediment input from the bendway. For the Swan Lake-Locopolis reach, backwater computations were carried upstream starting with a known stage-discharge relationship at the Swan Lake Gage to upstream of Locopolis Gage (river mile 231.44).

5.3 Model Calibration

5.3.1 Ft. Pemberton

In the cutoff there were two sets of measurements. The model was calibrated to reproduce these values. These measurements were the stage at the Ft. Pemberton Gage (Yazoo-Tallahatchie) and the bed elevation at four sections directly downstream of the weir in the cutoff.

Bed elevation was calibrated by adjusting the sediment transport over the weir at the upstream section, and the water surface elevation was calibrated by adjusting Manning's n. Since the two quantities are not independent, an iterative method was used where only one of the quantities was adjusted at a time.

First, Manning's n was calibrated by multiplying the estimated n value for the main channel and overbanks by a constant. Then, the model was run with the flows from April 12, 1973 to February 23, 1974 (318 days), and the error between the observed and computed water surfaces was minimized until no further reduction in error could be obtained. During these runs the sediment transport was set to zero, therefore, the bed was assumed fixed.

The model was then run for the same time period with the sediment transport in the river set to its normal value. With the initial bed profile of April 12, 1973, the sediment transport over the weir, which was computed with the same relationship as the river's, was adjusted by changing a constant until the computed bed at the end of the time period was matched as closely as possible with the observed value. Manning's n was then readjusted.

Results of the calibration suggested the Manning's n values of 0.030 for the main channel and 0.150 for the overbank flows. With these values of n, the average error between observed and computed stages at Ft. Pemberton for the calibration period was 0.87 foot with a maximum error of 3.72 feet. Figure 29 shows a plot of the observed and computed stage hydrographs for the calibration. Figure 30 shows a plot of observed and computed bed elevations at the four cross sections downstream of the weir. As indicated, the computed bed profile for February 23, 1974 closely matches the observed bed profile. Even though this was only calibration, the model was able to simulate the large degradation at Permanent Range (P.R.) 162.50, Station 16+50 and Station 4+90, and the hump that was formed at Station 10+20 by adjusting only one parameter describing sediment input to the cutoff. Sediment input to the cutoff should be modified because of the presence of the weir. In addition, simulated changes in cross-sectional shapes are compared with measured changes (Figures 31 to 34). It is apparent the model simulated the changes in the cross sections very well.

5.3.2 Swan Lake-Locopolis

In the Swan Lake-Locopolis reach there was not enough cross-sectional data to calibrate sediment transport, therefore, only Manning's n was calibrated and sediment transport was assumed equal to its normal value. In this reach it was necessary to allow Manning's n to vary with discharge since there appears to be large differences in the head

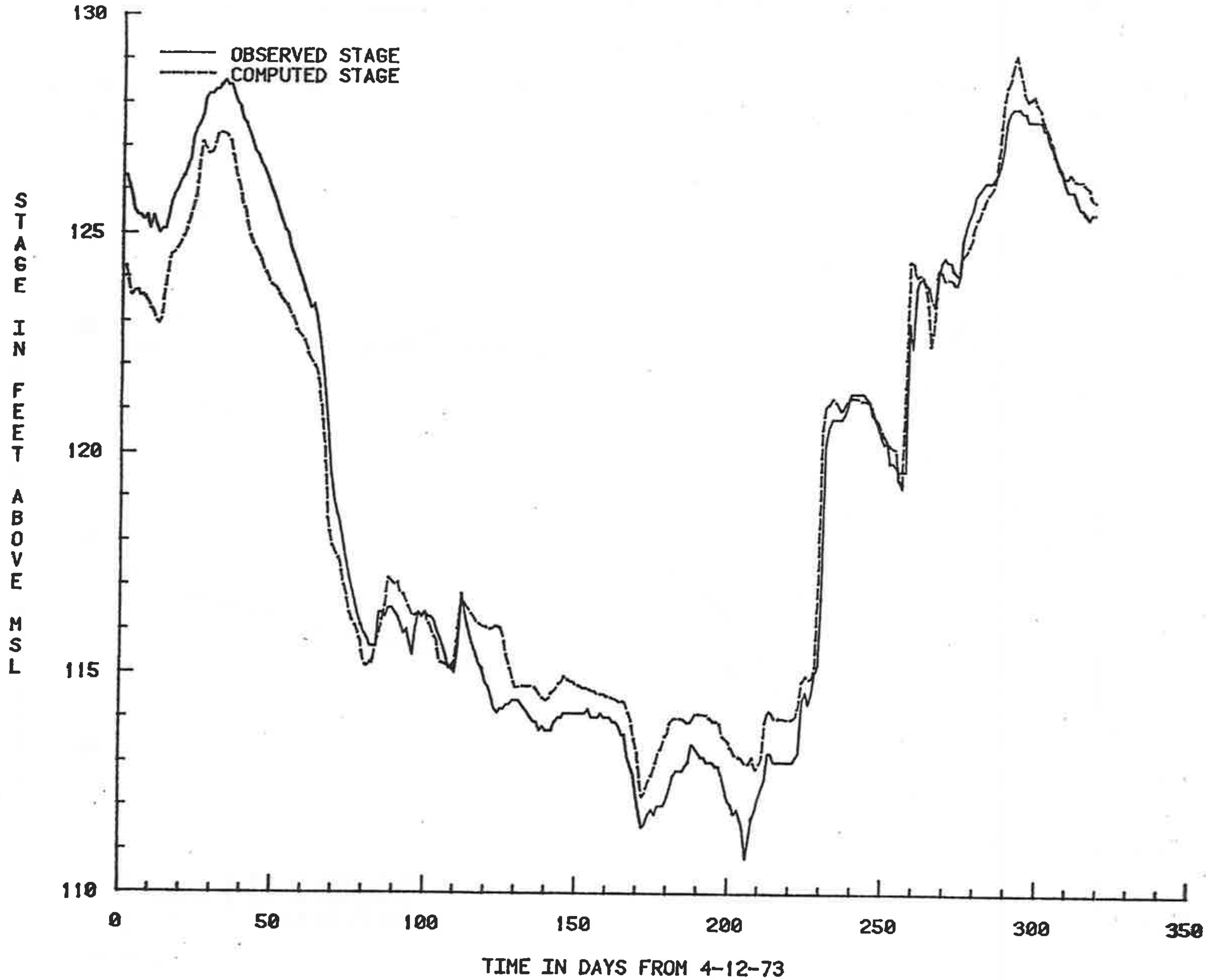


Figure 29. Known discharge model calibration at Ft. Pemberton.

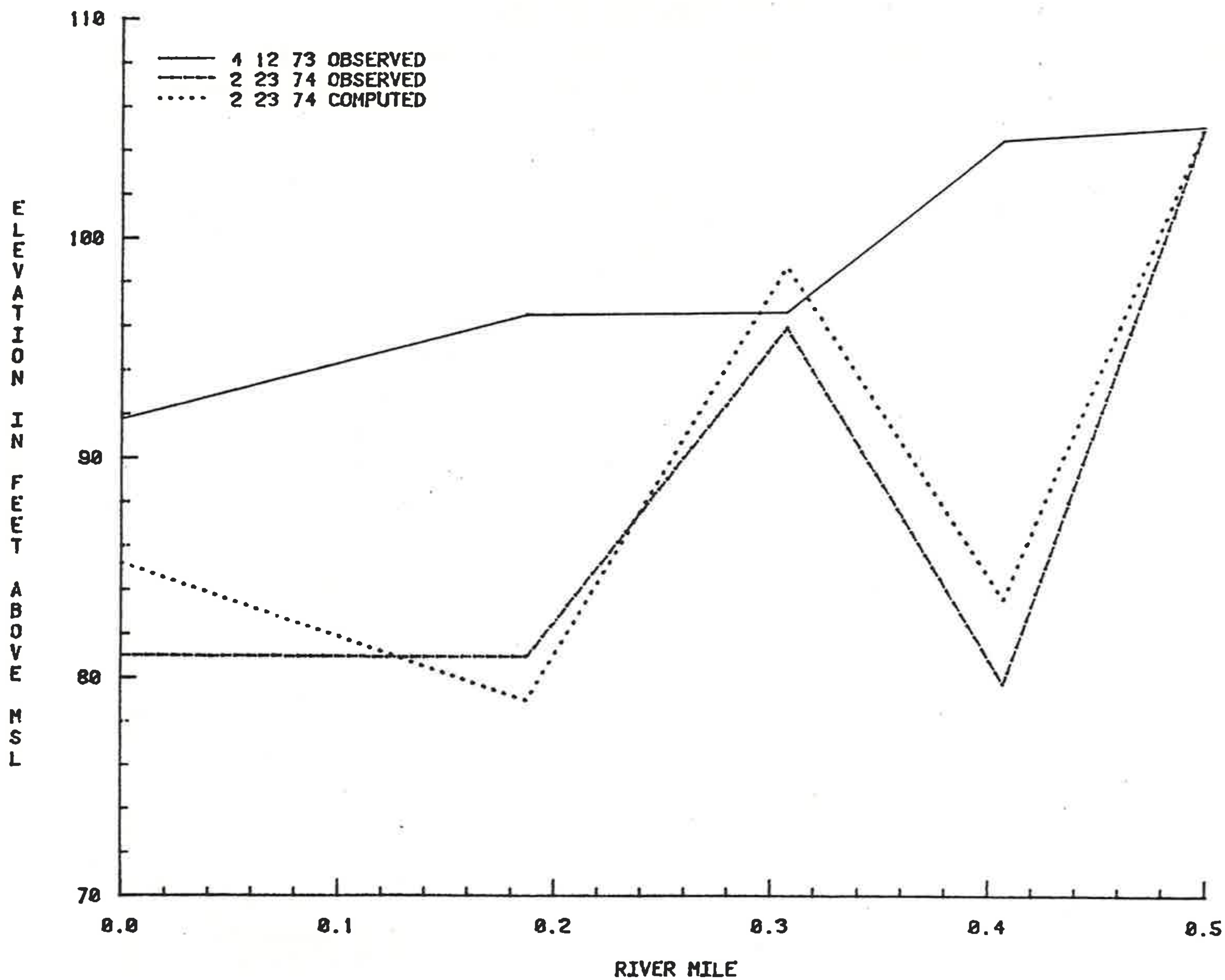


Figure 30. Known discharge model calibration of bed elevation at Ft. Pemberton.

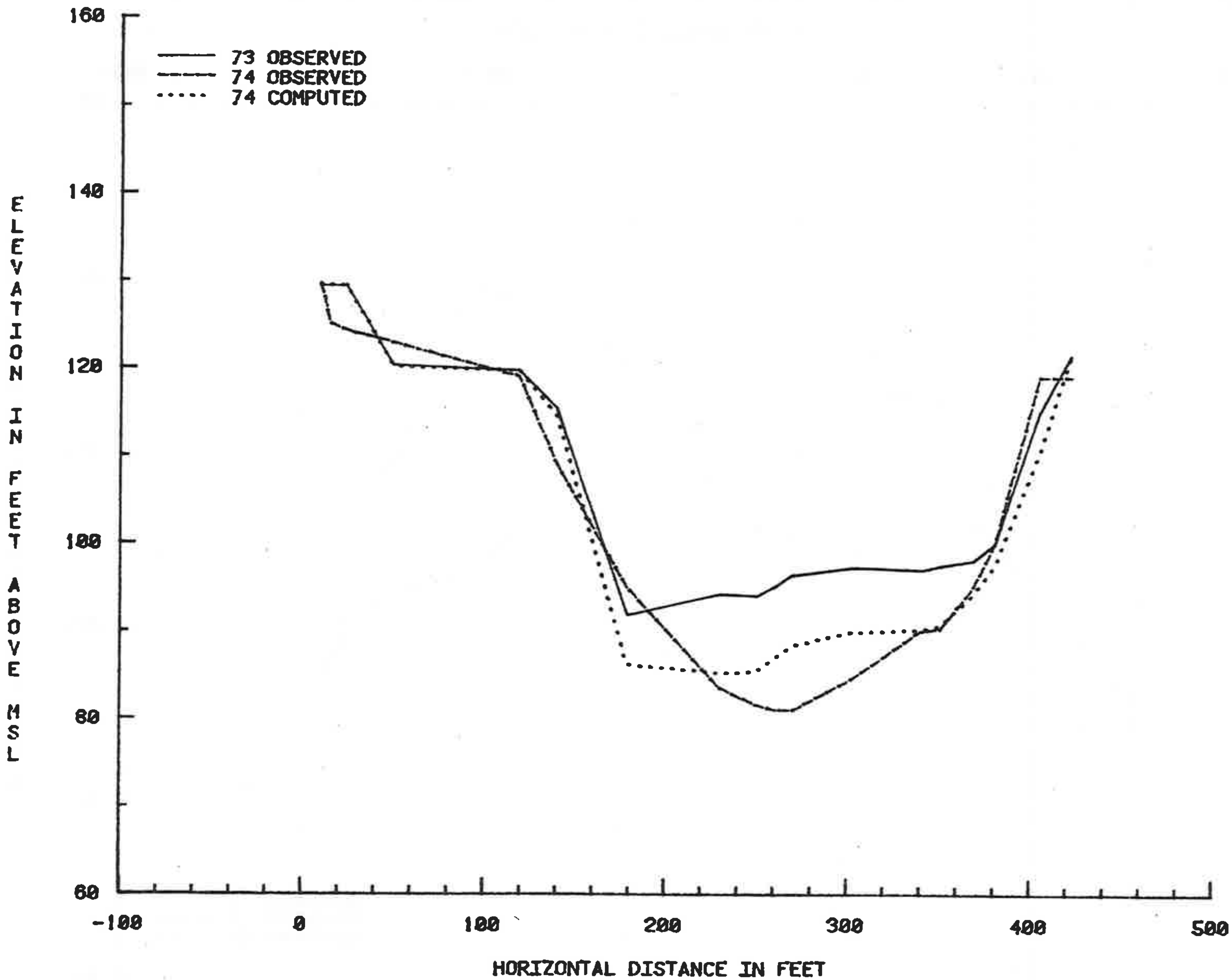


Figure 31. Comparison between computed and measured cross sections at Ft. Pemberton using the known discharge model (P.R. 162.5).

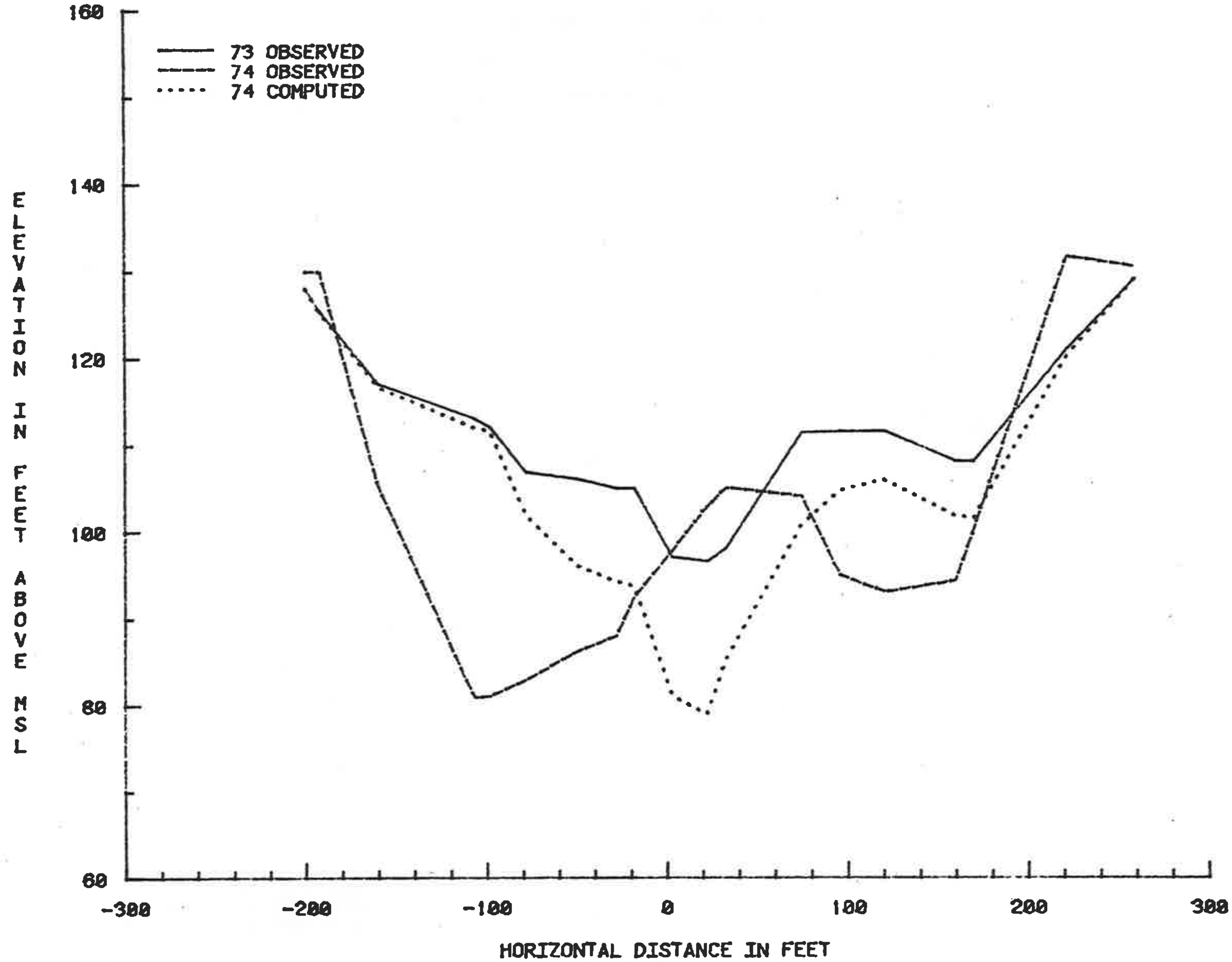


Figure 32. Comparison between computed and measured cross sections at Ft. Pemberton using the known discharge model (Station 16+50).

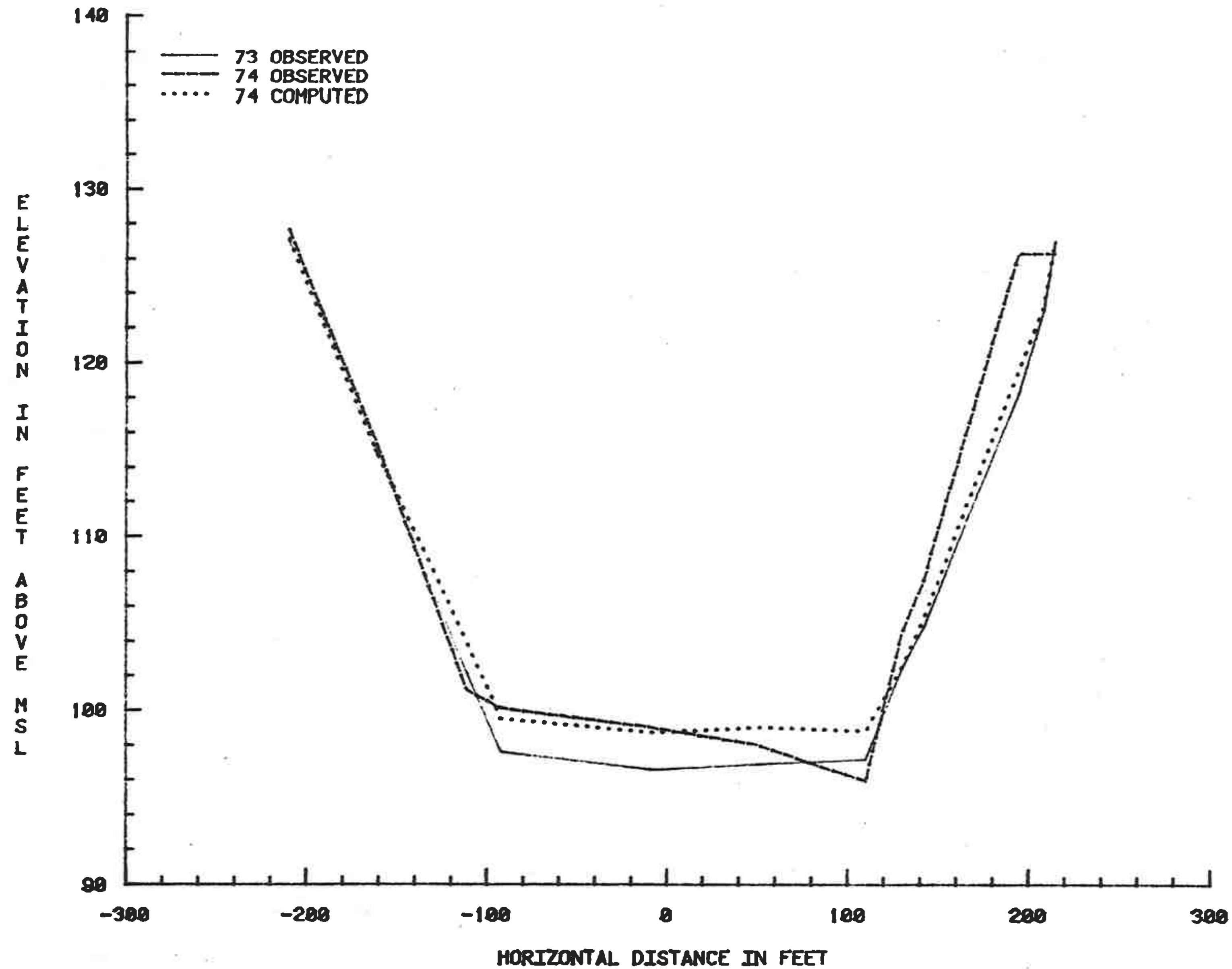


Figure 33. Comparison between computed and measured cross sections at Ft. Pemberton using the known discharge model (Station 10+20).

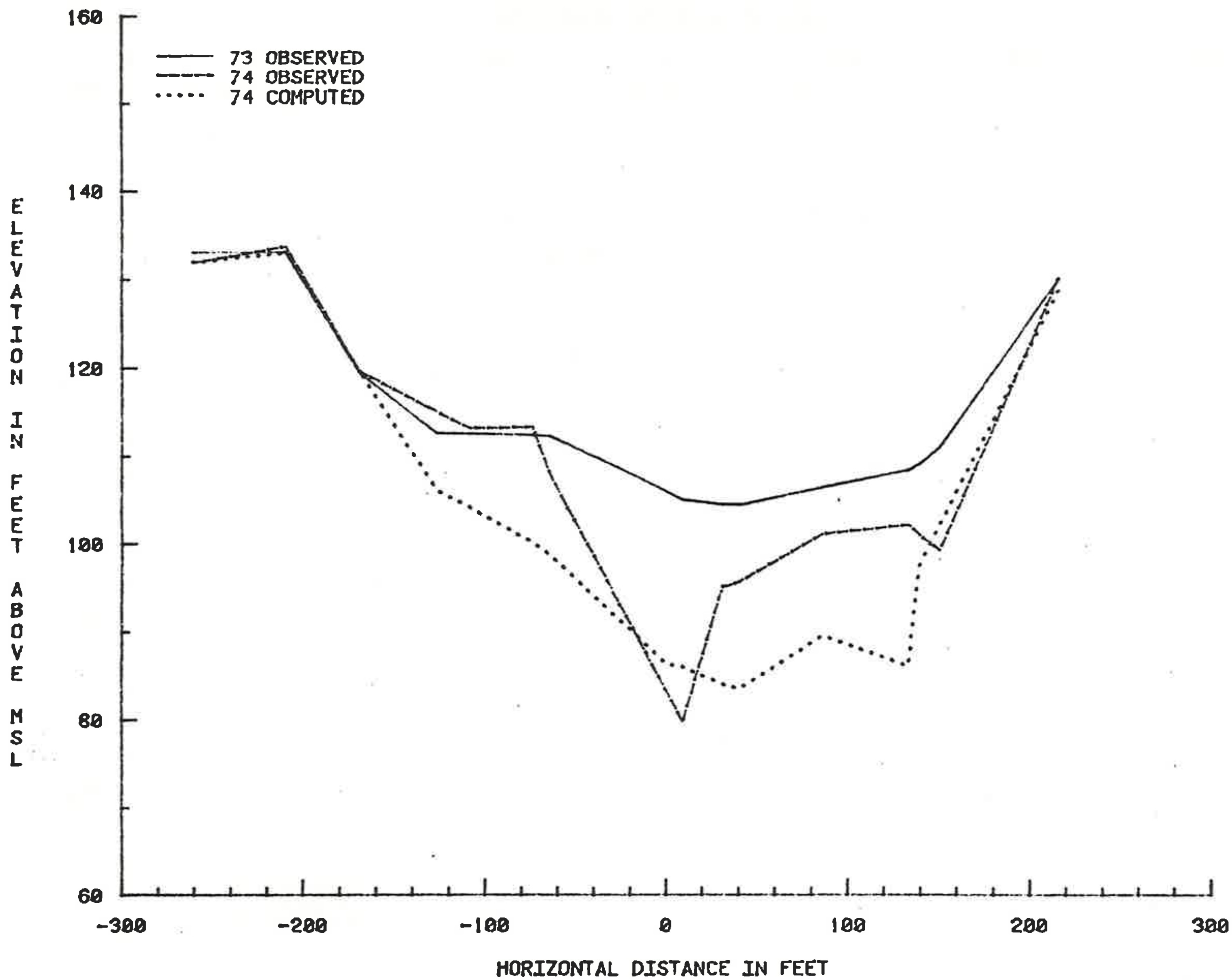


Figure 34. Comparison between computed and measured cross sections at Ft. Pemberton using the known discharge model (Station 4+90).

loss from Swan Lake to Locopolis for different flow levels. The period used for calibration was Januay 1, 1971 to December 31, 1972 (731 days). Results of the calibration are shown in Table 5. Manning's n varies as a power function of discharge. As indicated in the table, Manning's n is about 0.02 for the extreme high flow and approximately 0.042 for low flows. These values are very reasonable considering the hydraulics of the fluvial system. With these values of n, the average error between observed and computed stages at Locopolis for the calibration time period was 0.34 foot, with a maximum error of 2.12 feet. Figure 35 shows the observed and computed stage hydrographs for the calibration. As indicated, the error between the two is quite small.

5.4 Model Verification

To verify the applicability of the model, an additional run was made for each reach using the calibrated values of Manning's n and sediment transport equations. These runs were made for time periods immediately following calibration. At Ft. Pemberton the verification was from February 24, 1974 to December 31, 1974 (311 days). At Locopolis the period used was from January 1, 1973 to December 31, 1974 (730 days). At Ft. Pemberton the average error between the computed and

Table 5. Manning's n calibration for Swan Lake.

Discharge at Swan Lake	Manning's n	
	Main Channel	Overbank
2,000	.042	.210
10,000	.035	.175
35,000	.020	.100

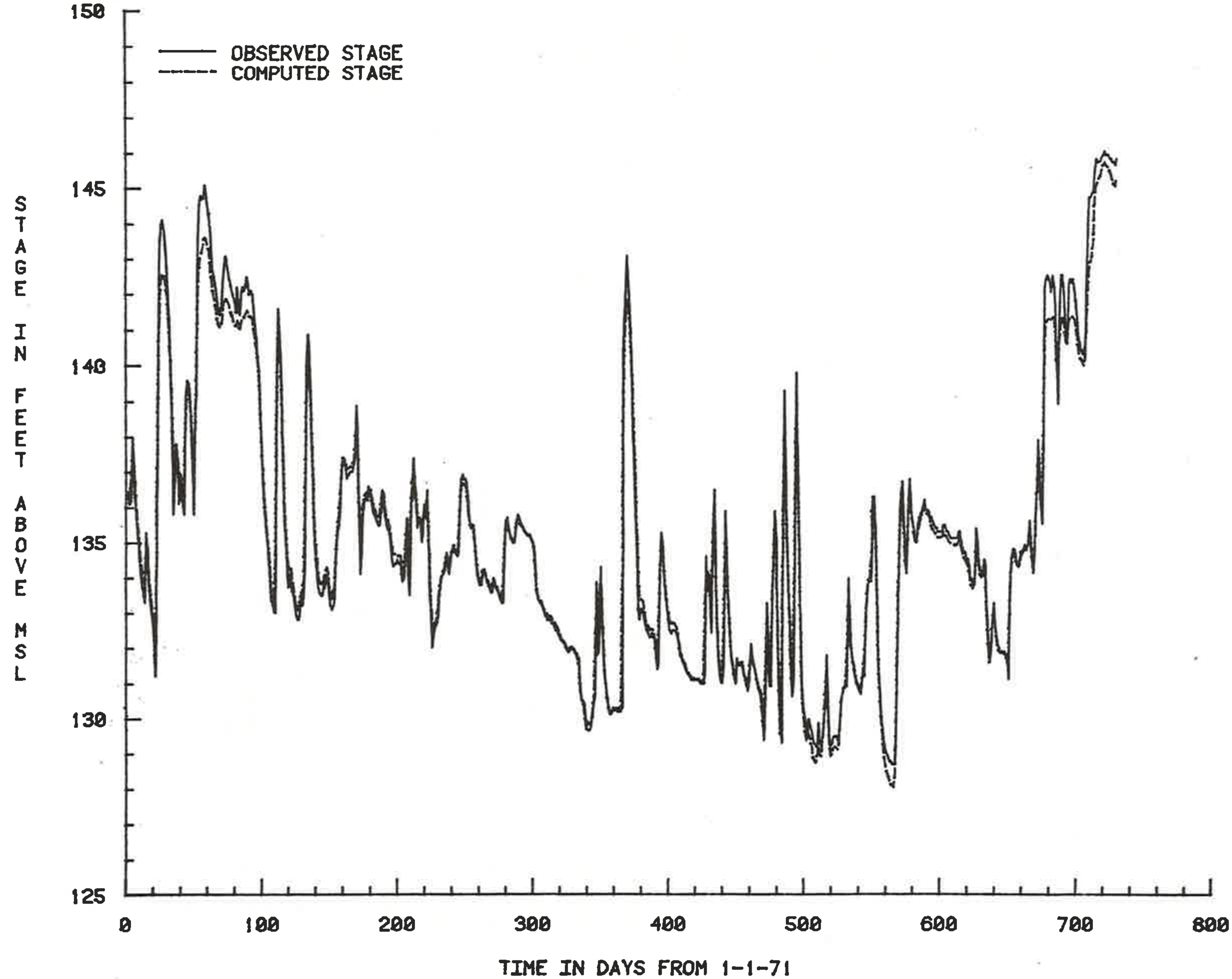


Figure 35. Known discharge model calibration at Locopolis.

measured stage for the verification period was 1.10 feet with a maximum error of 2.48 feet; and at Locopolis, the average was 0.74 foot with a maximum error of 2.21 feet. Figures 36 and 37 show plots of the measured and computed stages at Ft. Pemberton and Locopolis. In addition, Figure 38 shows measured and computed bed elevations for June 1, 1974 between Swan Lake and Locopolis. As seen in Figure 38, the two bed profiles closely match. The prediction period is January 1, 1971 to December 31, 1974.

A summary of the calibration and verification results from the known discharge model is given in Table 6. Comparison of the unsteady flow model with the known discharge model presented in this section indicates that the known discharge model is adequate for study. Results of the known discharge model are generally better than those obtained from the unstady flow model because of the accuracy of discharge. The simplicity of application and the tremendous savings in computer time plus the accuracy obtained proves the known discharge model is very useful for analyzing a complicated river system like the Yazoo River Basin.

Table 6. Calibration and verification for the known discharge model.

Reach	Calibration Error (ft)		Verification Error (ft)	
	Mean	Max	Mean	Max
Fort Pemberton	0.87	3.72	1.10	2.48
Locopolis	0.34	2.12	0.74	2.21

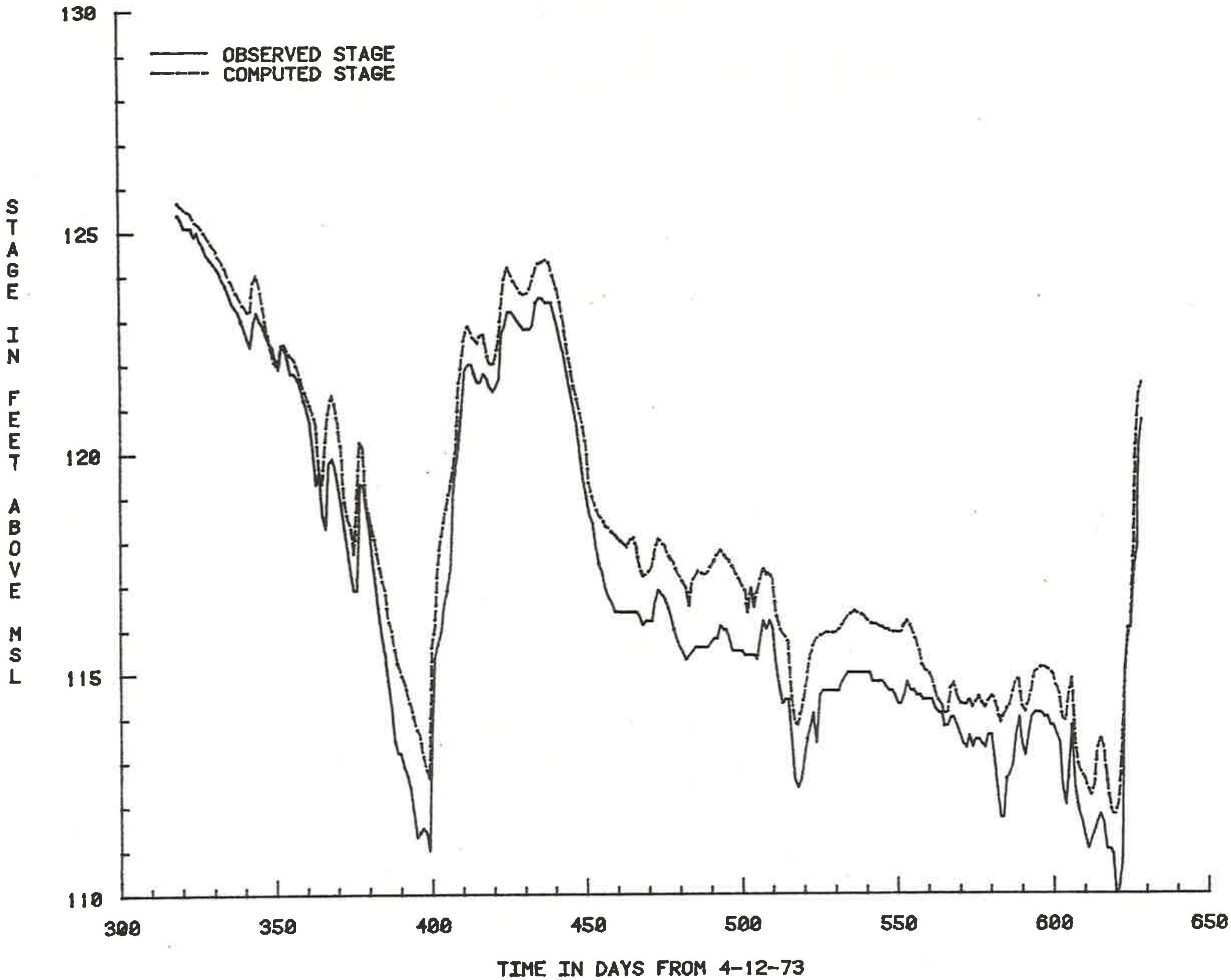


Figure 36. Known discharge model verification at Ft. Pemberton.

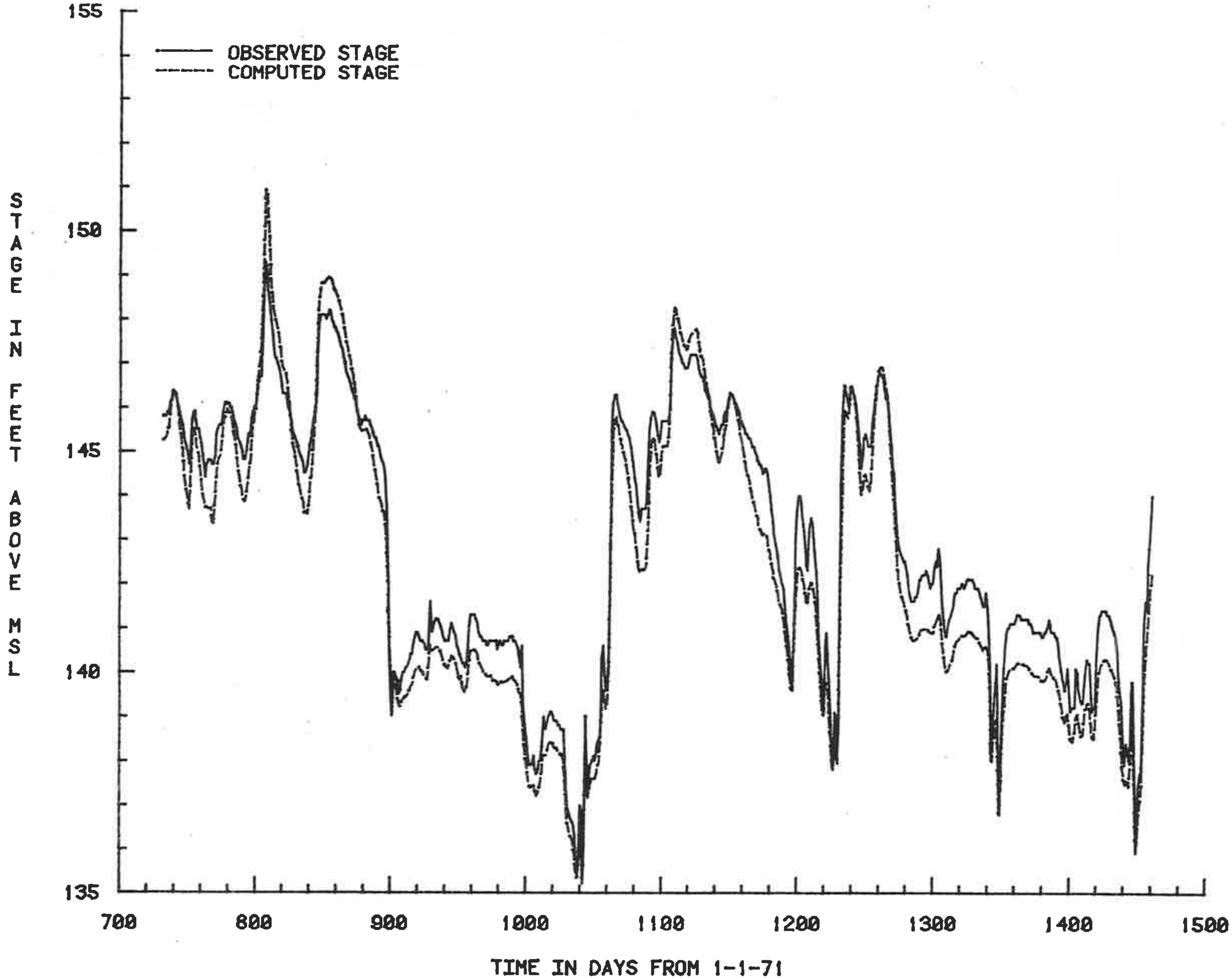


Figure 37. Known discharge model verification at Locopolis.

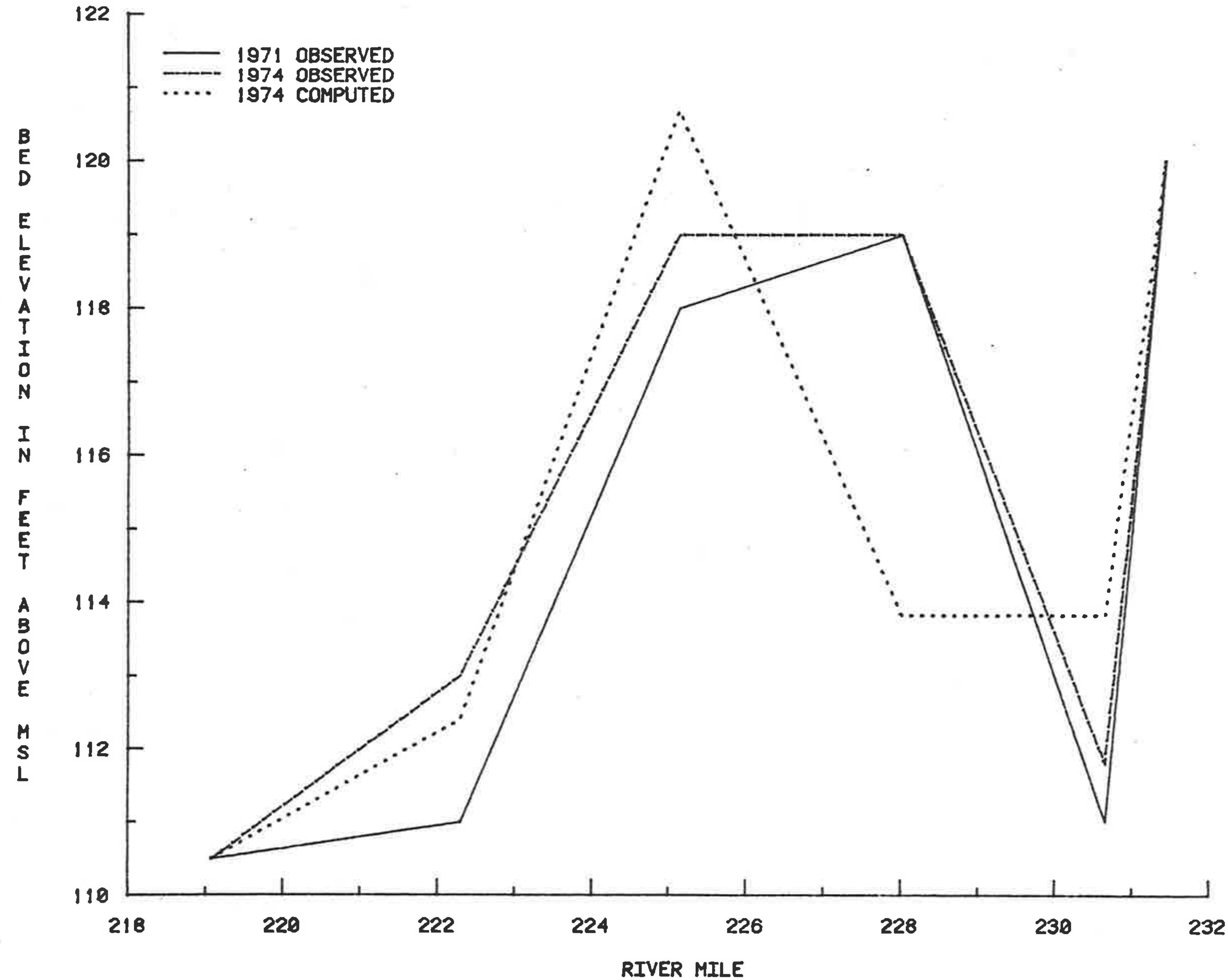


Figure 38. Known discharge model verification of bed elevation in Swan Lake-Locopolis Reach.

VI. RESULTS OF ANALYSIS

Both the unsteady flow and the known discharge models were applied to evaluate river response to specified design alternatives. The alternative study runs were specified by the U.S. Army Corps of Engineers and were described in Section 1.4. Runs No. 9 and 10 were analyzed by applying the unsteady flow model (Chapter IV). The other 13 runs were completed utilizing the known discharge model (Chapter V). Spatial representations of each study run were given in Section 3.2.

6.1 Application of the Unsteady Flow Model

To evaluate the effectiveness of Plan E in passing the flood flow, a two-year hydrograph (1973 and 1974) was routed through the natural Yazoo River and the proposed Plan E channel system utilizing the unsteady flow model (Runs No. 9 and 10, respectively). The Greenwood Cutoff was assumed open for these two runs. Discharges, stages, and bed elevations were computed and compared to evaluate the impact of Plan E. The enlargement of channel cross sections and the introduction of the Craigside Cutoff affects the propagation of the flood wave. Comparison between the flood discharges of Runs No. 9 and 10 shows that the peak discharge after implementing Plan E increased less than two percent at Greenwood and two percent at Belzoni for floods similar to the one in 1973. Also, due to the cutoff upstream of Ft. Pemberton, the flood wave from the Tallahatchie River peaked near Greenwood about two hours earlier. This alteration may or may not help reduce the flood potential in the Greenwood area. The river response depends on the flood wave introduced by the Yalobusha River. An assumption of 15 percent increase in discharge due to channelization adopted by the U.S. Army Corps of Engineers (Run No. 5) is certainly conservative.

Generally, for the two-year simulation period, Plan E would lower the local river stages. Effects would be more significant at low flow (11,000 cfs at Belzoni) than high flow (28,000 cfs at Belzoni) as shown in Figure 39. In this figure, changes in stages in the Yazoo main stem at low flow and high flow after implementation of Plan E are plotted. Negative values indicate that the stage in the Plan E channel is lower than in the natural channel. Maximum decreases in stage would occur near the Craigside Cutoff and between Swan Lake and Locopolis where the bankfill cross-sectional area was increased significantly for Plan E. Some increase in stage along the mainstem due to deposition from the tributary sediment inflows would also occur after implementing the recommended plan. The efficiency in flood stage reduction would be greatly impaired if the Plan E channel is not maintained for a long period, as described later in this chapter (Section 6.2.3).

Geomorphic changes computed by the unsteady model indicates there would be more sediment deposition in the Yazoo River main stem two years after implementing Plan E than in the natural channel (Figure 40). The deposited sediment volume in the Plan E channel from Belzoni to just below the Greenwood Bendway would be 3,055,000 cubic yards, compared to 1,399,000 cubic yards in the natural channel. This results from the enlargement of the channel section for Plan E plus the increase of sediment inflow from Abiaca Creek. The Greenwood Bendway downstream of the mouth of the Yalobusha River would continue filling if the Ft. Pemberton Cutoff is opened. The estimated deposition volume in the Greenwood Bendway is 838,000 and 784,000 cubic yards in the Plan E channel and the natural channel, respectively. Because of the enlargement of the channel in the Tallahatchie River for Plan E, more sediment would deposit in the Tallahatchie and less sediment would be transported

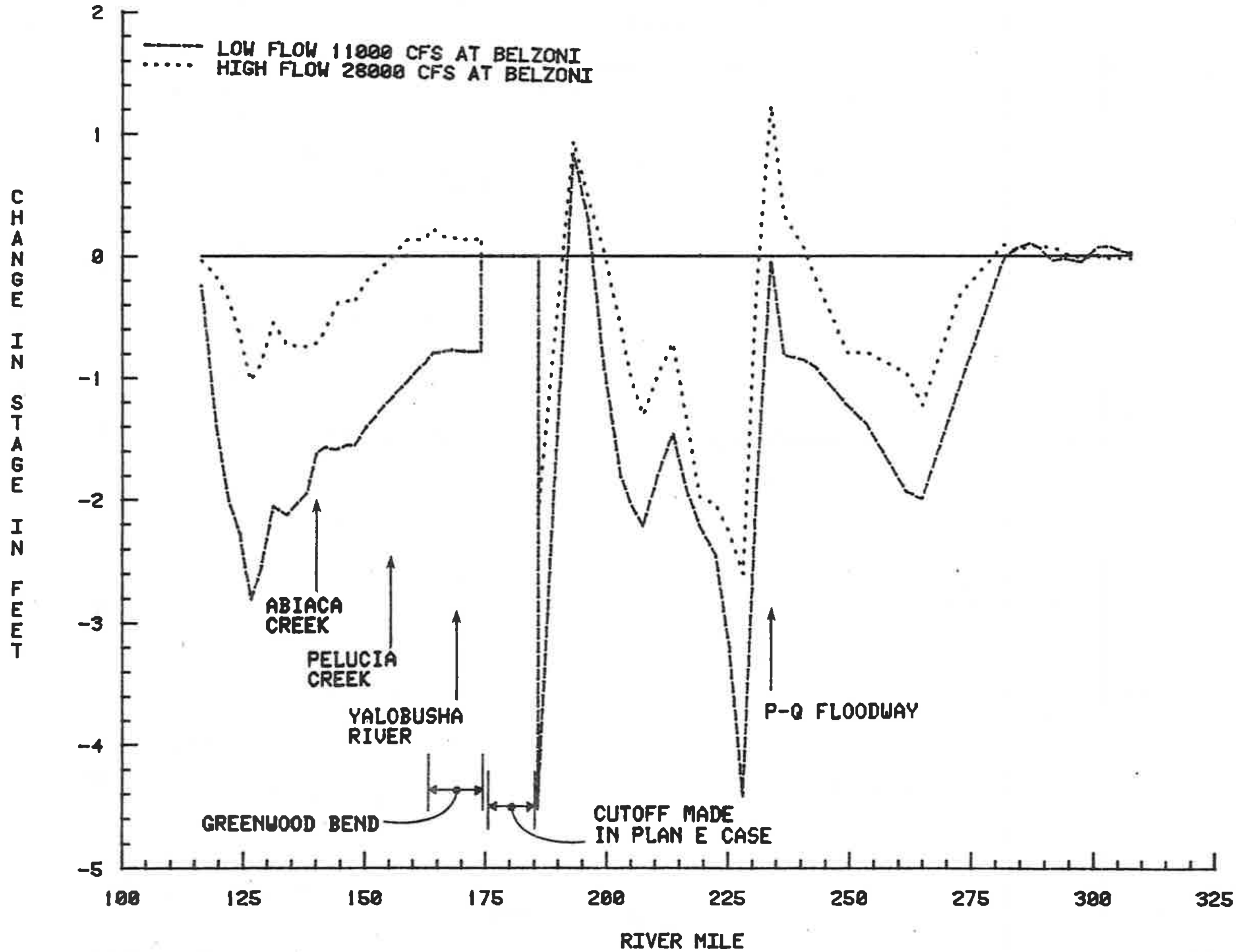


Figure 39. Stage changes from the natural channel case to the Plan E case (unsteady flow model).

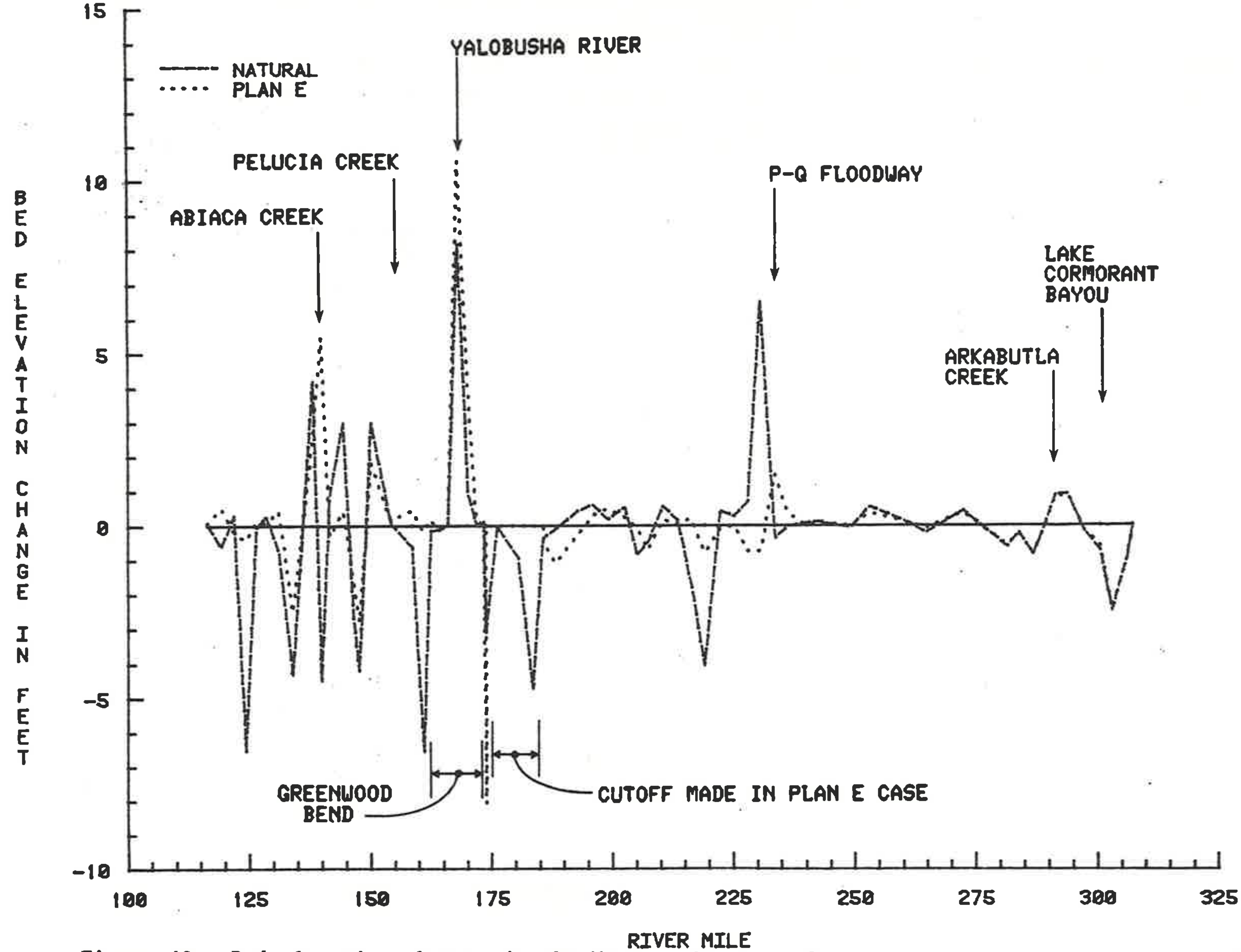


Figure 40. Bed elevation changes in the Yazoo main stem after two years (unsteady flow model).

downstream. This would result in erosion immediately upstream of and at the Ft. Pemberton Cutoff as shown in Figure 40.

6.2 Application of the Known Discharge Model

6.2.1 General

Applicability of the known discharge model in the Yazoo River System was examined and established by comparing it to the unsteady flow model. As mentioned earlier, the primary purpose of this study was to identify sedimentation problems and evaluate possible remedies. The known discharge simulation model is adequate for this purpose. The simplicity in application, the tremendous savings in computer time, plus the accuracy achieved verifies the known discharge model as an excellent tool for evaluating alternatives.

As mentioned in Section 3.2, the spatial representation of the known discharge model covered a much larger study area. The spatial designs for various study runs differed. Nevertheless, it is worthwhile to identify the river segments utilizing the same designation system. Hence, referring back to Figure 5, River Segment No. 1 extends from the mouth of Big Sunflower to Belzoni. River Segment No. 2 extends from Belzoni to just below the Greenwood Bendway. River Segment No. 3 includes the Greenwood Bendway, and River Segment No. 4 extends from immediately upstream of the Greenwood Bendway to Arkabutla Dam. The Yalobusha River is identified as River Segment No. 5. River Segment No. 6 includes the Little Tallahatchie and the P-Q Floodway. The Yocona River is defined as River Segment No. 7 and the Greenwood Cutoff is identified as River Segment No. 8. Simulated results for different river conditions for a 50-year synthetic hydrograph are summarized in Table 7. The summary of dredging volume for Runs No. 7 and 12 are given in Table 8.

Table 7. Summary of net degradation and aggradation for 50 years under different design conditions.

Alternative Runs No.	Net Degradation and Aggradation in 10^3 cubic yards Reaches							
	1	2	3	4	5	6	7	8
1	--	1,941	445	8,097	1,242	-2,078	908	- 48
2	--	22,075	2,846	16,909	4,181	-2,809	658	3
3	--	14,683	2,112	15,186	3,226	-3,750	518	- 14
4	--	13,422	1,323	14,038	2,142	-4,002	454	- 22
5	--	25,209	3,484	18,717	7,023	-1,405	859	-109
6	--	30,829	5,143	20,319	11,171	-275	955	-128
7	--	2,944	-1,278	-7,212	-121	-10,613	-890	- 81
8	--	19,832	2,250	12,127	3,157	-3,893	608	- 21
11	5,313	23,228	2,742	16,665	3,990	-2,863	233	- 39
12	--	6,464	-992	-2,849	457	-9,203	-625	- 76
13	--	15,889	1,488	12,385	2,418	-3,121	587	- 36
14	--	14,510	726	11,117	1,408	-3,359	613	- 49
15	9,446	26,388	3,960	19,520	6,048	-2,709	222	-15

Table 8. Summary of total dredging volumes.

Alternative Runs No.	Cumulative Dredging Volumes for 50 Years in 103 Cubic Yards Reach					
	1	2	3	4	8	Total
7		36,568	4,509	29,582	687	71,346
12	--	35,100	2,199	25,910	399	63,608

The net degradation and aggradation volumes in the main stem above Belzoni (sum of River Segments No. 2, 3, 4, and 8) are given in Table 9. For Runs No. 7 and 12, total dredging volumes were included in Table 9.

6.2.2 Run No. 1 (Natural Conditions)

For the main stem from Belzoni to Arkabutla Dam (River Segments No. 2, 3, 4, and 8), the estimated rate of net filling (net degradation and aggradation) is about 210,000 cubic yards per year with natural conditions. Figure 41 shows the beginning and final bed profiles after 50 years under natural conditions.

The maximum water surface elevations at each cross section are also plotted in Figure 41. These maximum vwater. surface elevations are the maximum values at each cross section considering the 50-year simulation period. These values do not necessarily occur at the same time for all of the cross sections and may not take place during the period of maximum discharge. The downstream water surface elevations and the long-term sediment movement in the system will dictate local water surface elevations.

The cumulative net aggradation volumes in the main stem for the 50 years of simulation are shown in Figure 42. Generally, the main stem

Table 9. Summary of net degradation and aggradation in the main stem above Belzoni.

Alternative Runs No.	Total Degradation and Aggradation Volume in 50 years 10^3 Cubic Yards	Ratio to Natural Condition	Ratio to Plan E Condition
1	10,435	1.0	0.25
2	41,833	4.01	1.0
3	31,967	3.06	0.76
4	28,761	2.76	0.69
5	47,301	4.53	1.13
6	56,163	5.38	1.34
7*	65,719	6.30	1.57
8	34,188	3.28	0.82
12*	57,981	5.56	1.39
13	29,726	2.85	0.71
14	26,304	2.52	0.63

*Dredging volumes are included.

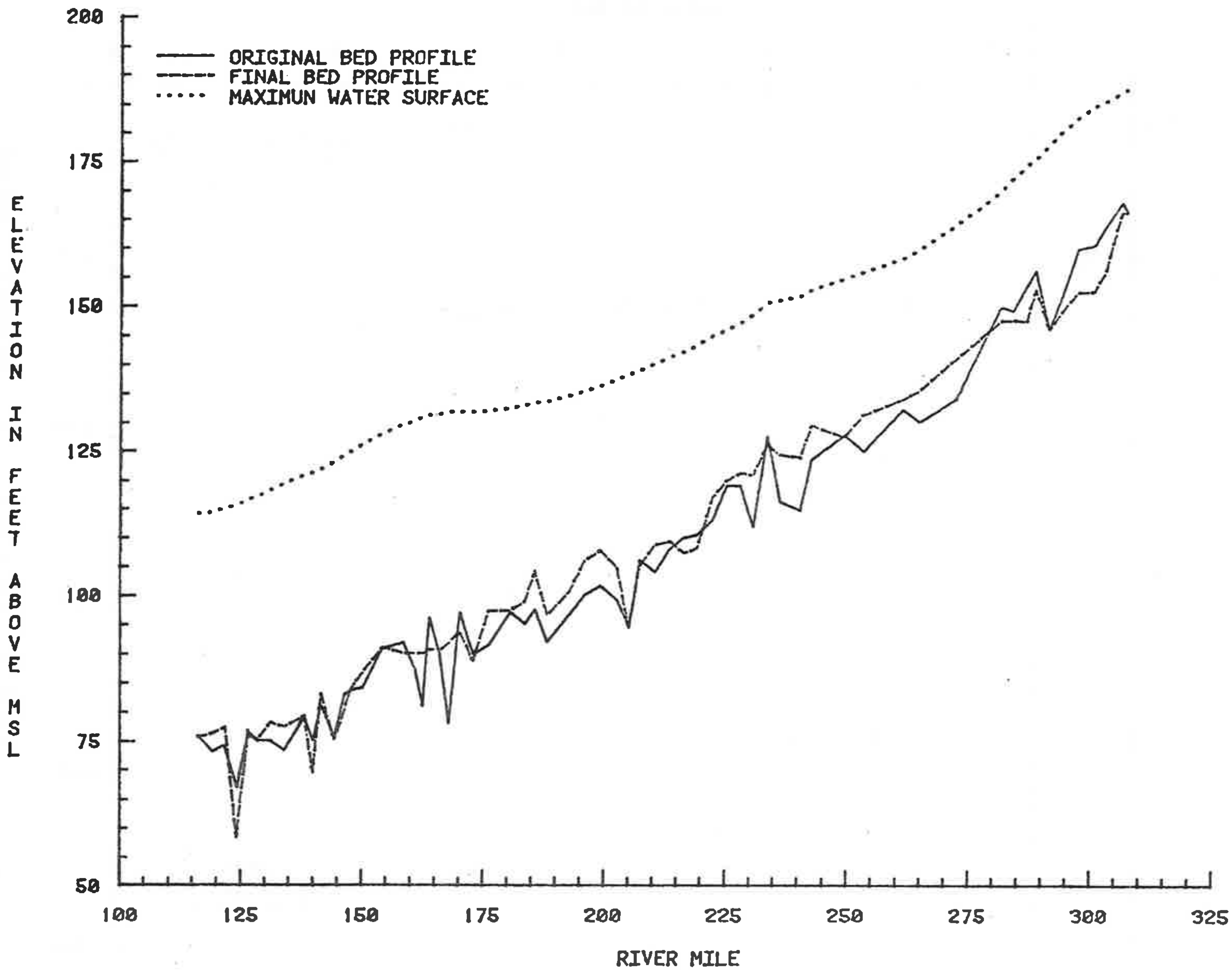


Figure 41. Beginning and final bed profiles and maximum water surface elevations for natural conditions (Run No. 1).

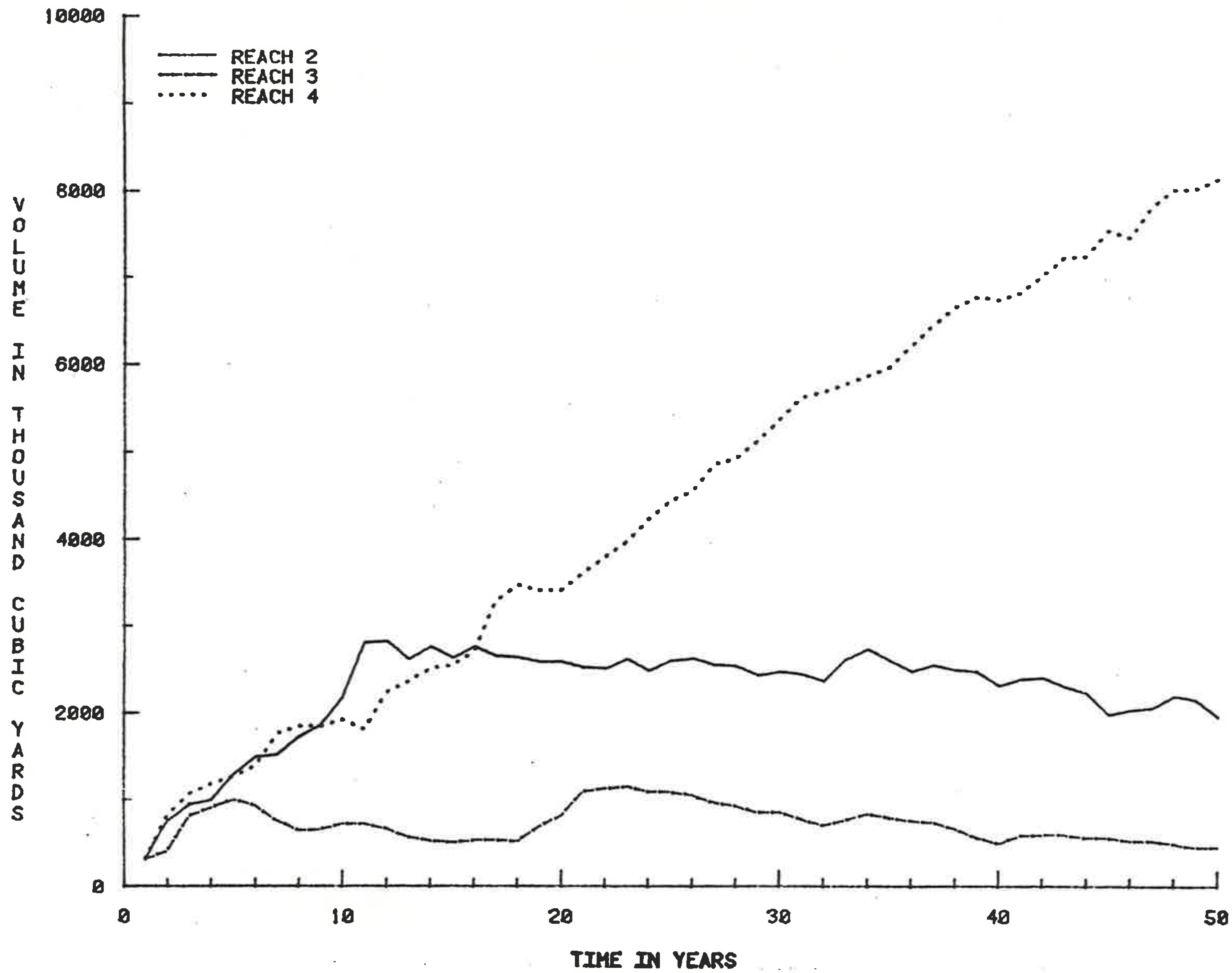


Figure 42. Cumulative aggradation (positive) volumes in the main stem for natural conditions (Run No. 1).

segments, except the Greenwood Cutoff, are depositing for the 50-year simulation cycle. River Segment No. 4 (from Greenwood to Arkabutla Dam) is filling, particularly below the P-Q Floodway. This is due to severe degradation in the P-Q Floodway.

The cumulative degradation and aggradation in the major tributaries (River Segments No. 5, 6, and 7) is shown in Figure 43. As indicated by model results, the Yalobusha River (River Segment No. 5) is aggrading which agrees with observations by the U.S. Army Corps of Engineers (Design Memorandum No. 41). This river has been filling with sediment since the enlargement of the channel cross section in 1954. River Segment No. 6 (P-Q Floodway and the Little Tallahatchie River) is degrading according to model calculations. This also agrees with observations by the U.S. Army Corps of Engineers. The river bed gradient of the Little Tallahatchie River is about four times that of the main stem river due to construction of the P-Q Floodway. Degradation in the P-Q Floodway was so severe that extensive emergency dredging was required to maintain the Tallahatchie River below the mouth of the P-Q Floodway. The Yocona River (River Segment No. 7) is sensitive to the water surface in the P-Q Floodway. The simulated results show that the Yocona River is almost in a state of equilibrium.

Examples of the stage-discharge relationship just below Greenwood Bendway (river mile 162.5) and at Swan Lake (river mile 219.08) during the first year, 10th year, 30th year, and 47th year are displayed in Figures 44 and 45. The 47th year was selected because it closely approximated the 1973 flood. These figures indicate the stage-discharge relationship at both stations are consistent with time for the natural condition. Additional plots of the change in the stage-discharge

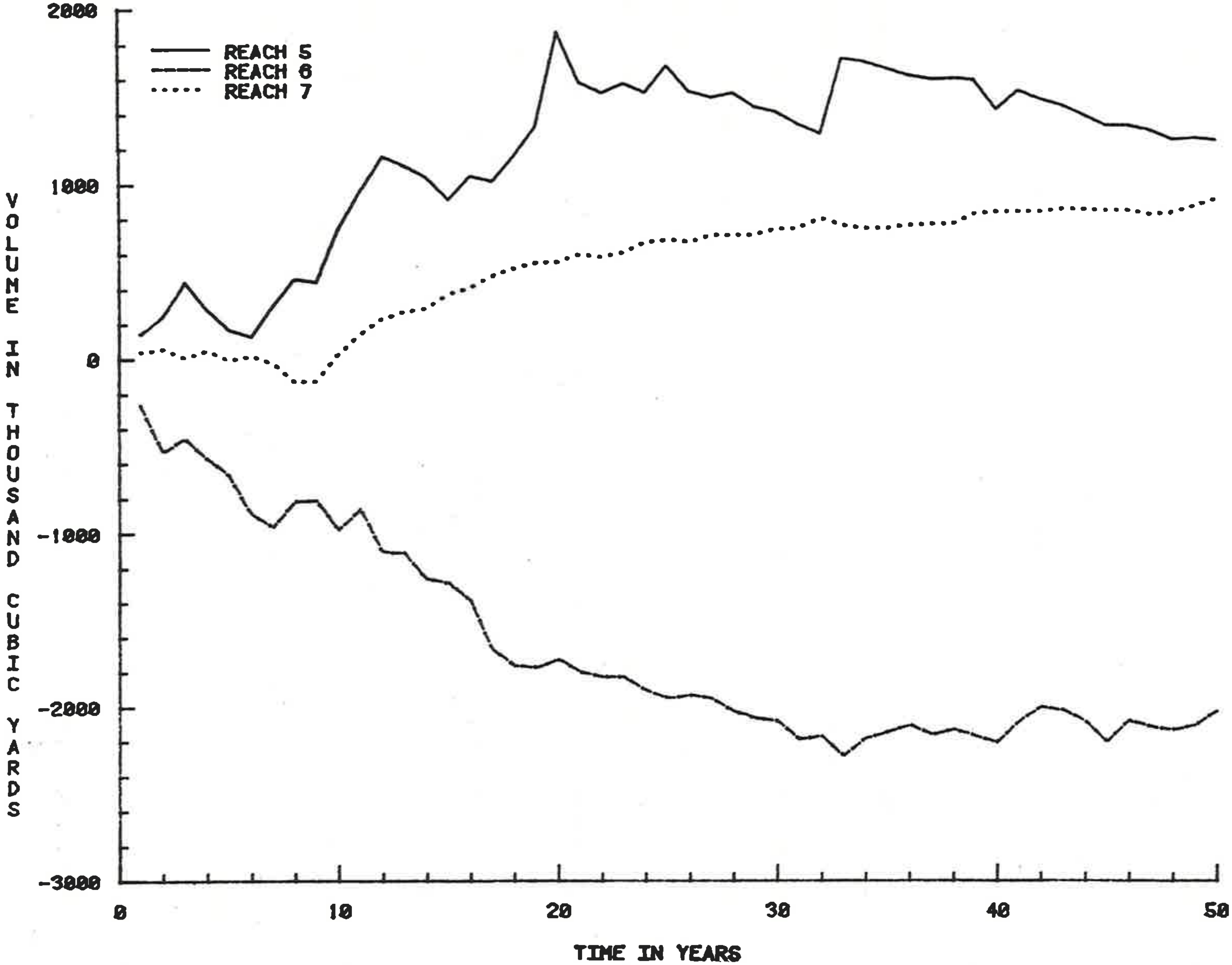


Figure 43. Cumulative degradation (negative) and aggradation (positive) volumes in the major tributaries for natural conditions (Run No. 1).

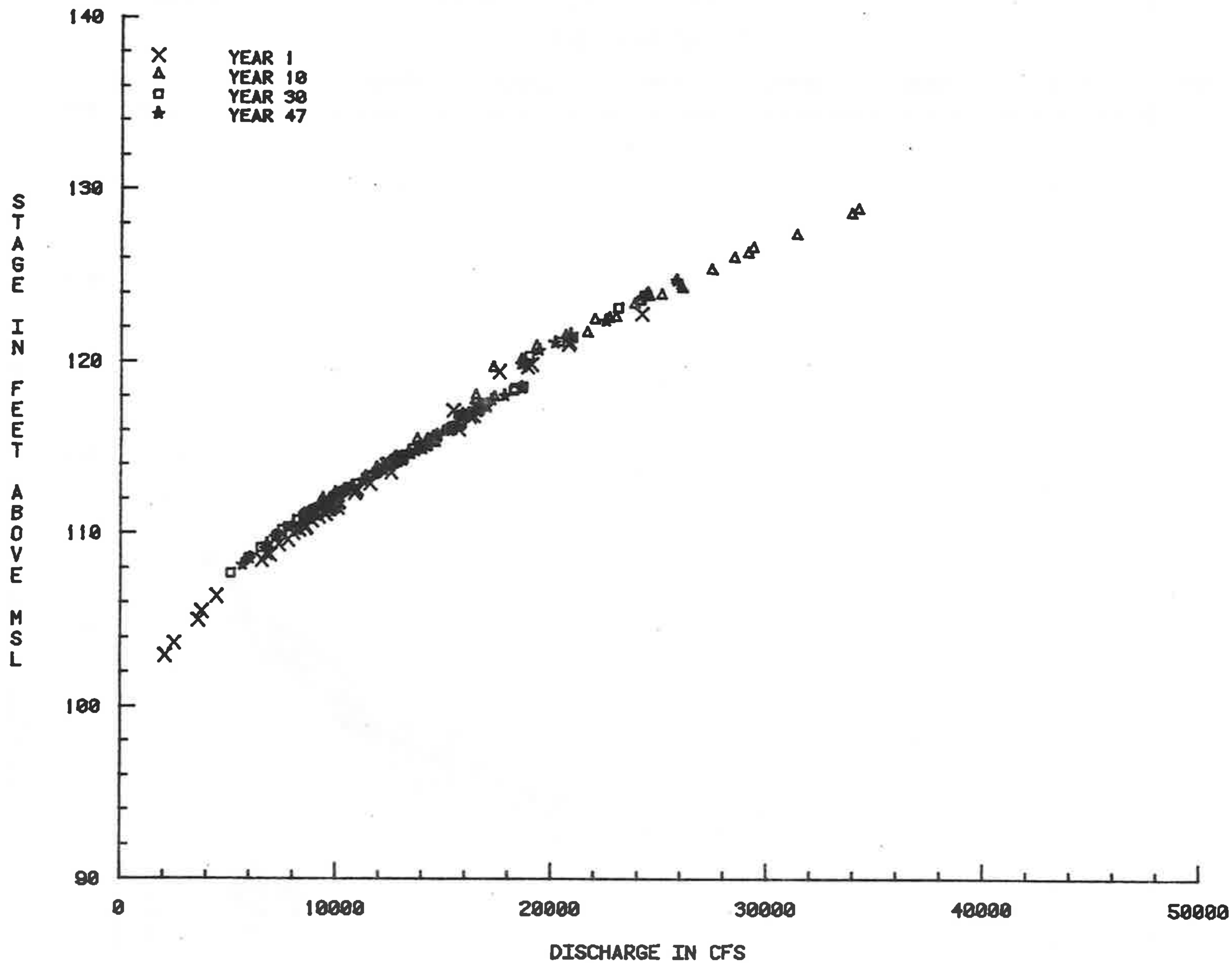


Figure 44. Stage discharge relationship downstream of the Greenwood Bendway (river mile 162.5) for natural conditions.

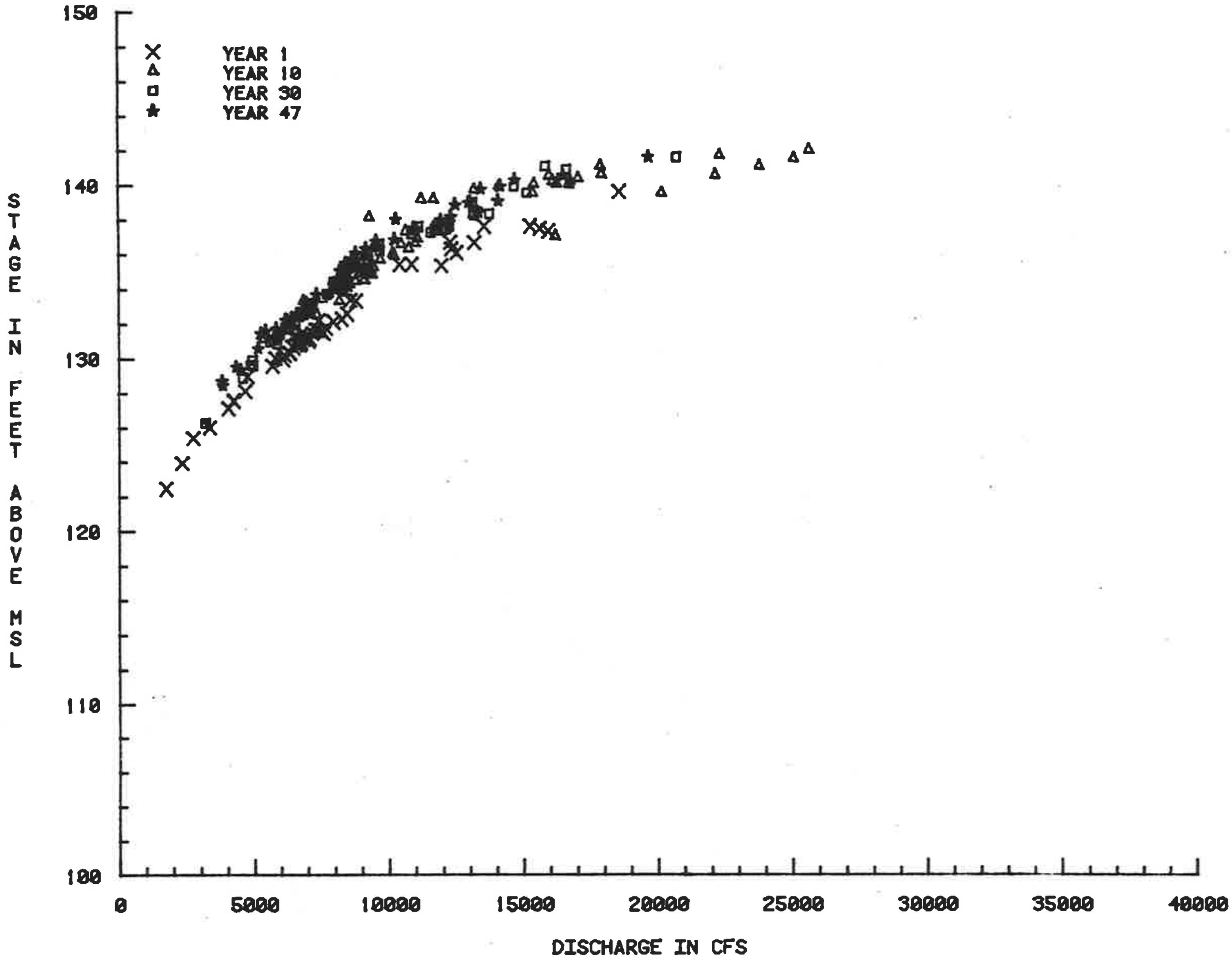


Figure 45. Stage discharge relationship at Swan Lake (river mile 219.08) for natural conditions.

relationship and comparisons of the stage-discharge relationships for different conditions are given in Appendix C.

6.2.3 Run No. 2 (Plan E)

The net rate of sediment deposition in the main stem utilizing Plan E conditions increased approximately 400 percent. That is, the net depositional rate in the main stem is about 840,000 cubic yards per year. This is due to the enlargement of the channel, the increase in the sediment supply from Abiaca Creek (about 500 percent), and the increase in the degradation along the P-Q Floodway (about 40 percent). Lowering the water surface levels in the Tallahatchie enhances degradation in the P-Q Floodway.

Figure 46 shows the beginning and final bed profiles as well as the maximum water surface elevations along the main stem under Plan E conditions considering 50 years of simulation. This figure clearly indicates that the most important areas causing maintenance problems are the reaches below Abiaca Creek and below the mouth of the P-Q Floodway. The maximum water surface levels are generally higher than those under natural conditions for reaches downstream of Money. Cumulative aggradation volumes in the main stem for 50 years is shown in Figure 47. The study shows that deposition rates of Plan E are much larger than those for natural river conditions. The computed average rate of deposition for Plan E decreases with time, which is consistent with changes in hydraulic conditions.

For a detailed examination of Plan E, Figure 48 provides the maximum water surface elevations under both the natural and Plan E conditions for the 50-year simulation period. Generally, the maximum water surface elevations are higher for Plan E in the main stem except in the reach

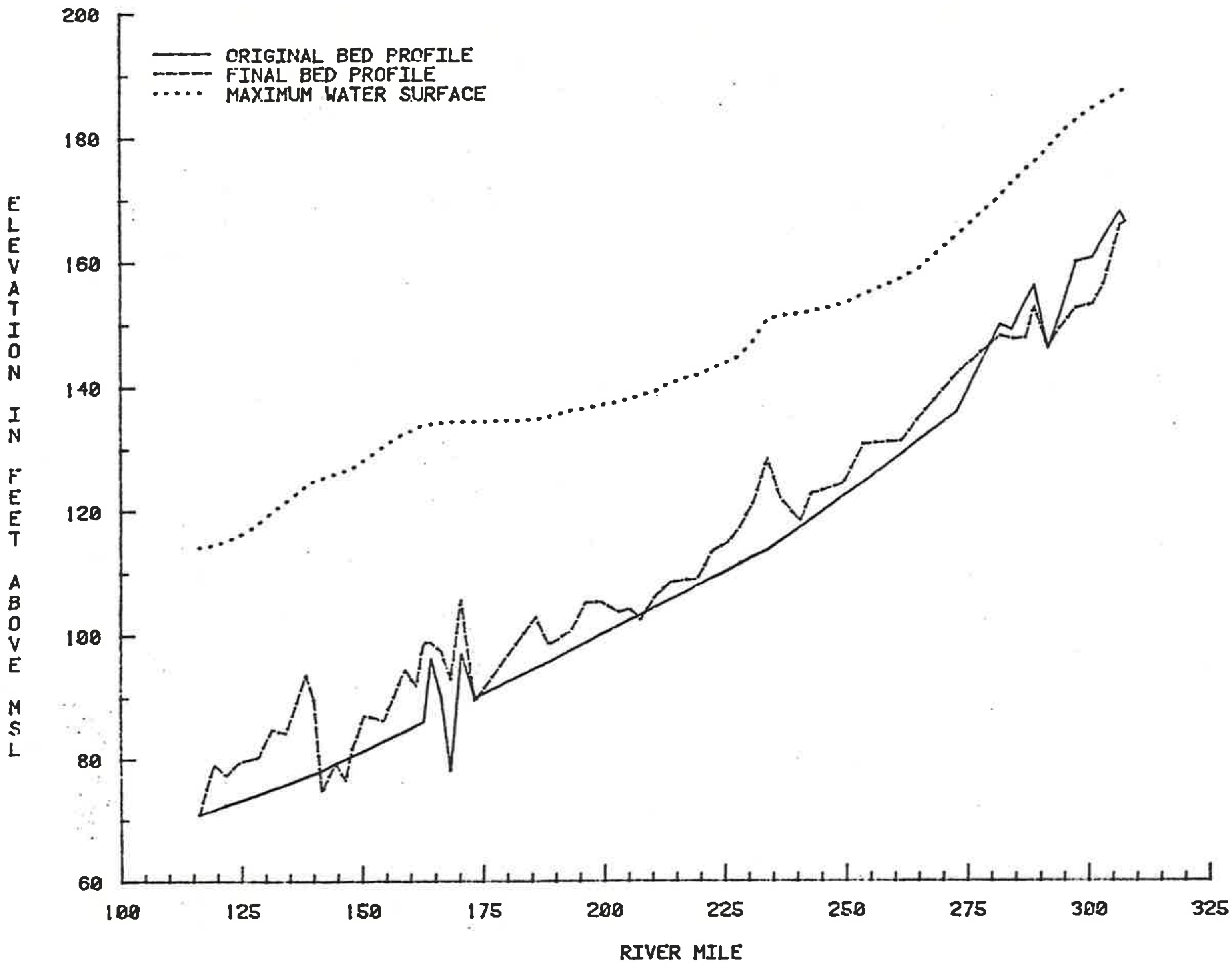


Figure 46. Beginning and final bed profiles and maximum water surface elevations for Plan E conditions (Run No. 2).

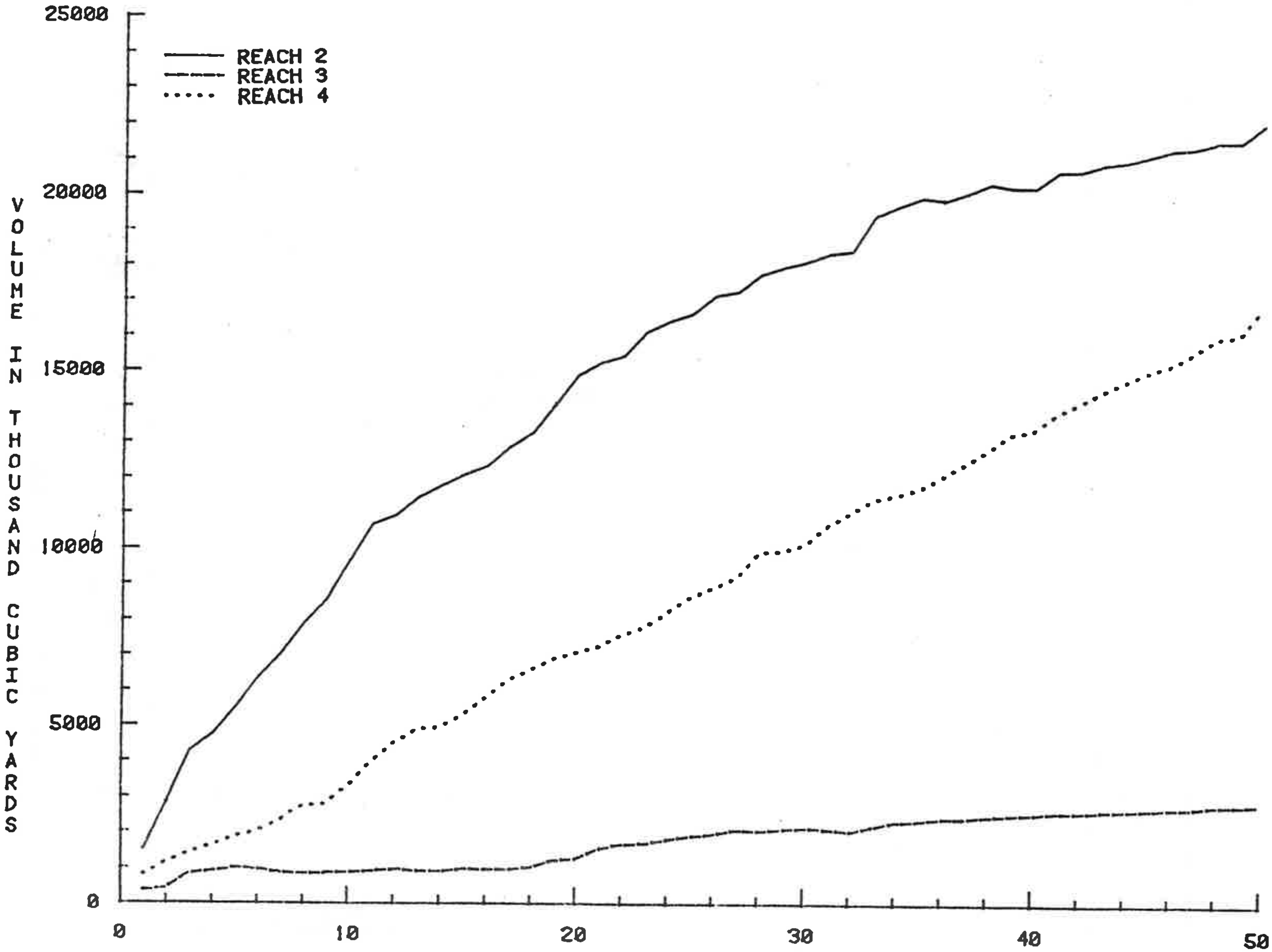


Figure 47. Cumulative aggradation (positive) volumes in the main stem for Plan E conditions (Run No. 2).

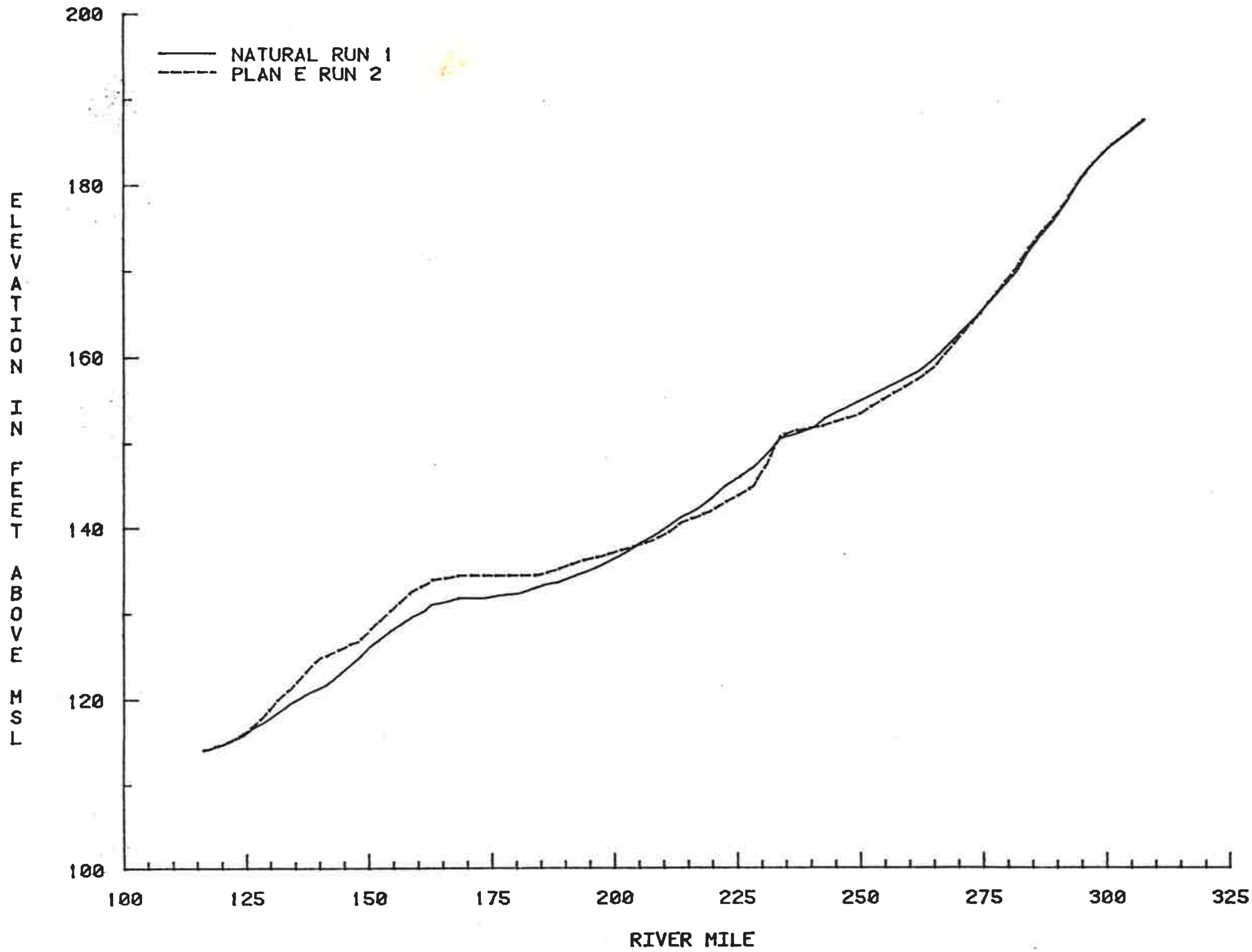


Figure 48. Comparison of maximum water surface elevations between natural and Plan E conditions (Runs No. 1 and 2).

near and upstream of Swan Lake (river mile 219.08). The higher water surface elevations in the reach downstream of river mile 200.0 are due to accumulation of sediment for 50 years without maintenance, and the significant increase of sediment supply to the main stem from Abiaca and the P-Q Floodway. The reduction in stage near and upstream of Swan Lake is primarily the result of the Craigside Cutoff. Figure 49 reestablishes that the computed maximum water surface elevations for the Plan E channel without maintenance are higher than the computed flow line for the reach downstream of Money (river mile 192.6), and are lower for the reach upstream of Money. These comparisons are based on the 1974 Yazoo Headwater Project Design Flood meeting the 1973 Mississippi crest as reported by the U.S. Army Corps of Engineers.

It is important to mention that if the Plan E channel is allowed to accumulate sediment for 50 years without maintenance, the efficiency of flood stage reduction of Plan E would be significantly decreased. This is further demonstrated in Figures 50 and 51. In these figures, the annual maximum water surface elevations just below the Greenwood Bendway (river mile 162.5) and at Swan Lake (river mile 219.08) for both natural and Plan E conditions are plotted. The river stage reduction by implementation of Plan E is only good in the first two years at river mile 162.5, but is always effective at Swan Lake. After the channel fills with sufficient sediment, river stage reduction is significantly decreased and finally terminated in some reaches. The stage frequency curves just below the Greenwood Bendway and at Swan Lake (river mile 219.08) for natural and Plan E conditions are shown in Figures 52 and 53, respectively. These curves indicate that the proposed Plan E can only be effective if maintenance measures such as dredging and/or control of

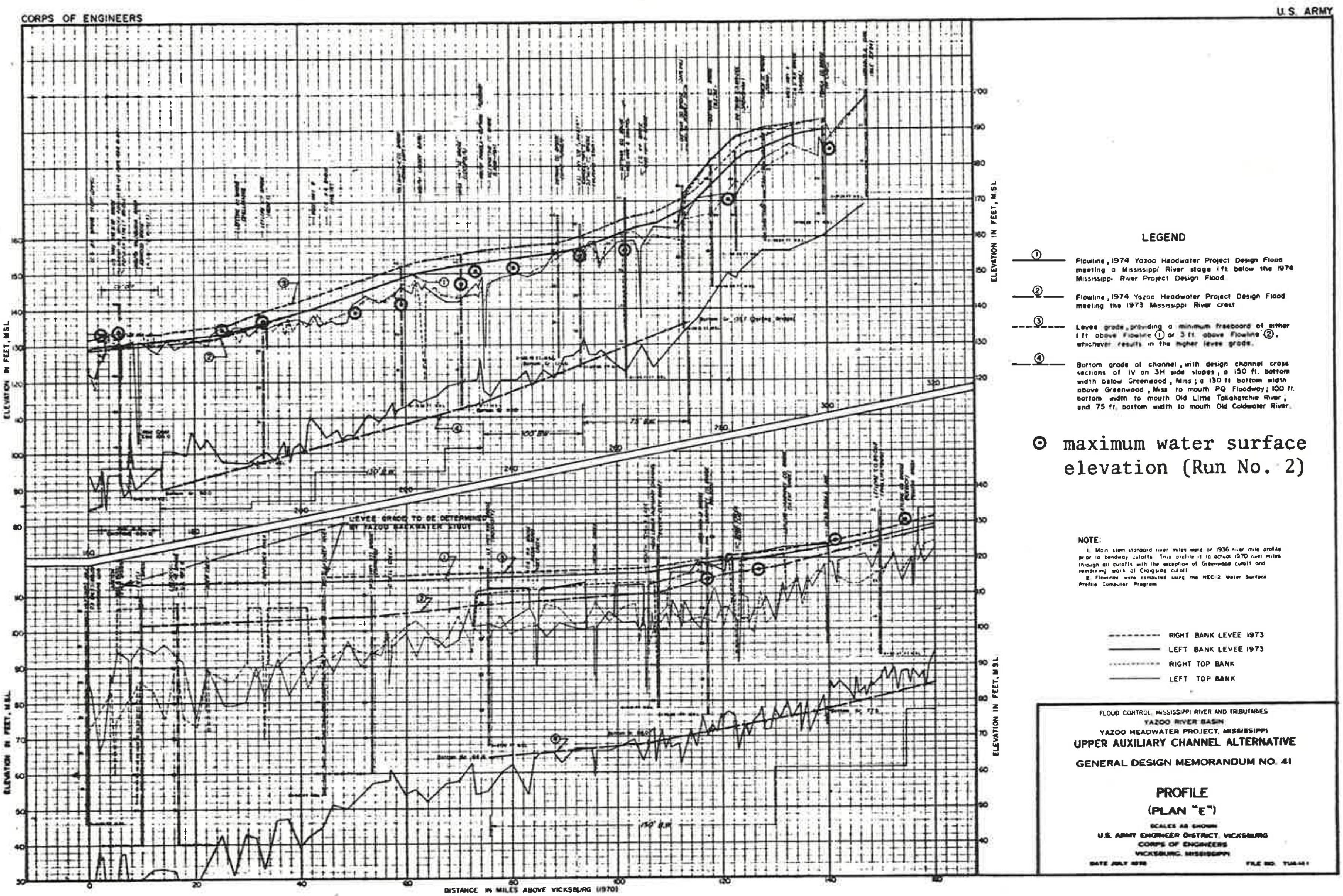


Figure 49. Maximum water surface elevations for Plan E (Run No. 2) and flowlines of project design flood.

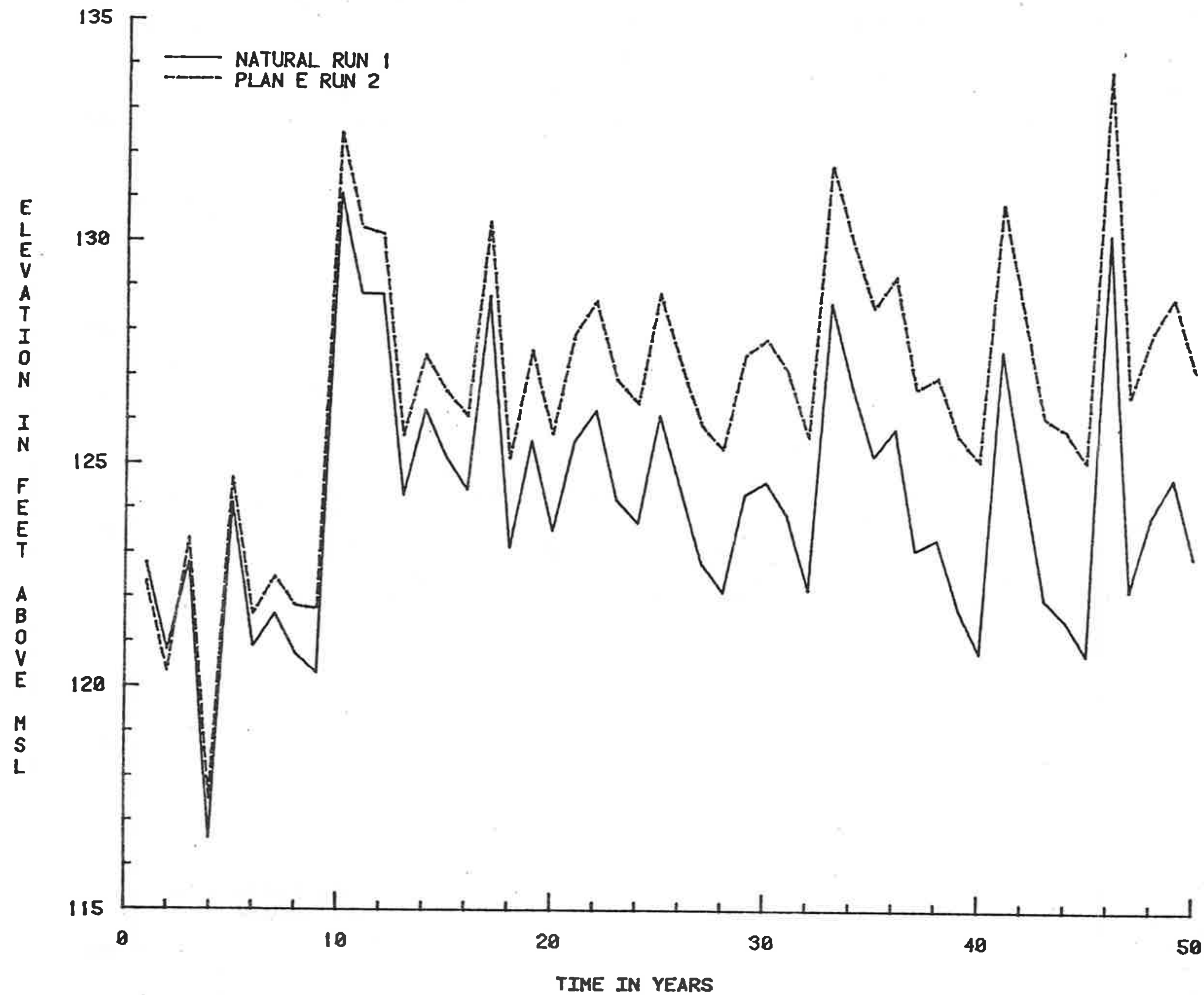


Figure 50. Annual maximum water surface elevations just downstream of the Greenwood Bendway (river mile 162.5) for natural and Plan E conditions.

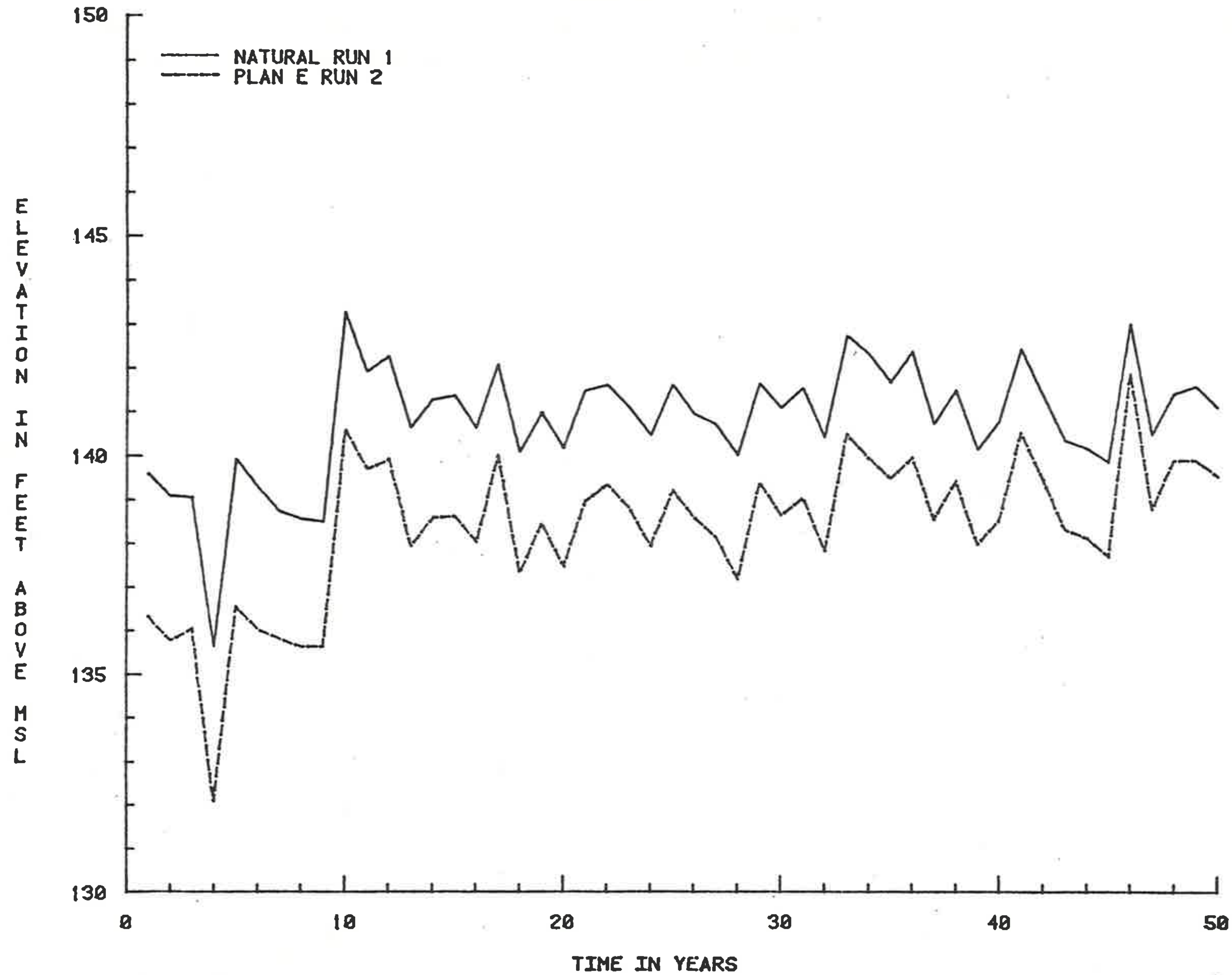


Figure 51. Annual maximum water surface elevations at Swan Lake (river mile 219.08) for natural and Plan E conditions.

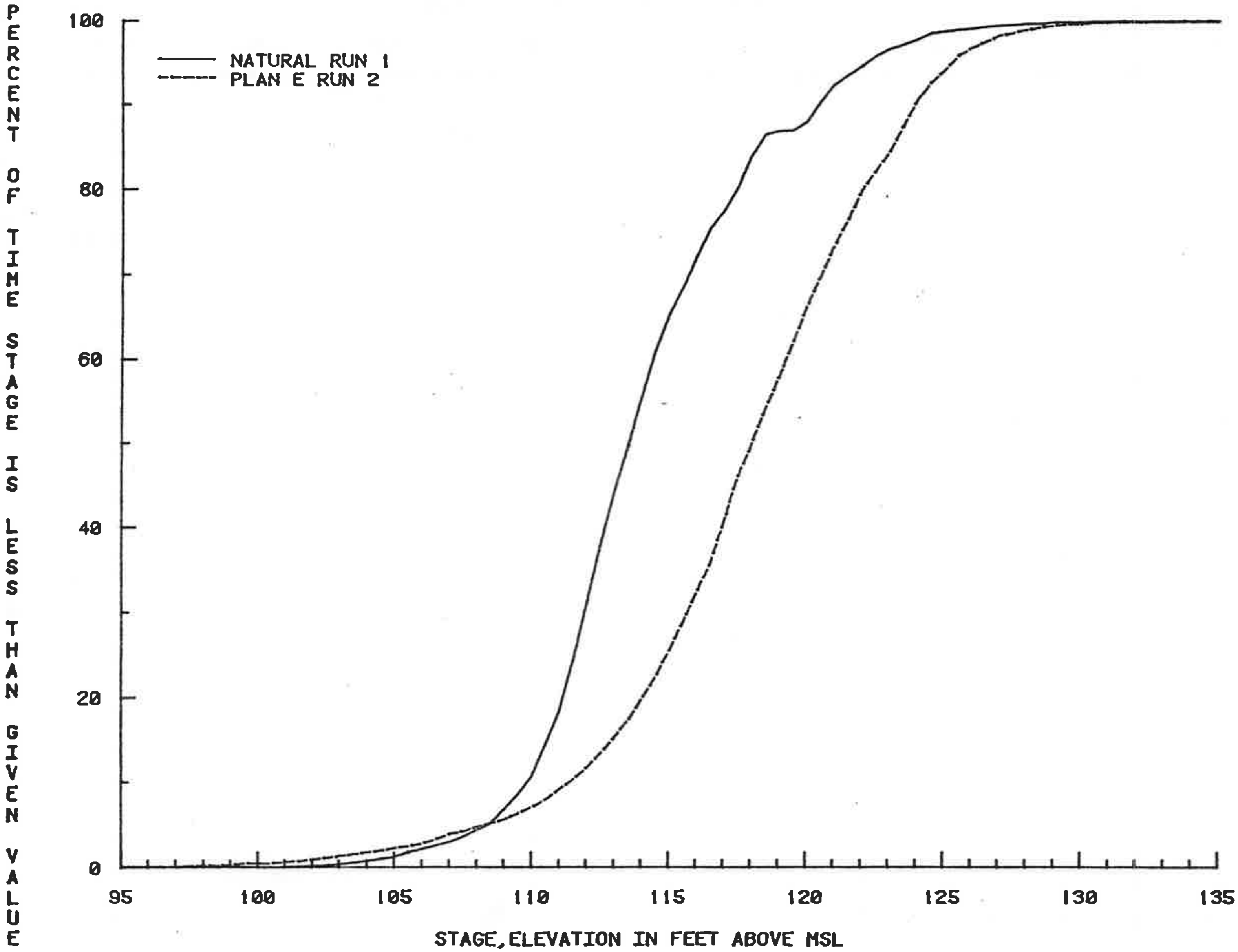


Figure 52. Stage frequency curve just below the Greenwood Bendway (river mile 162.5) for natural and Plan E conditions.

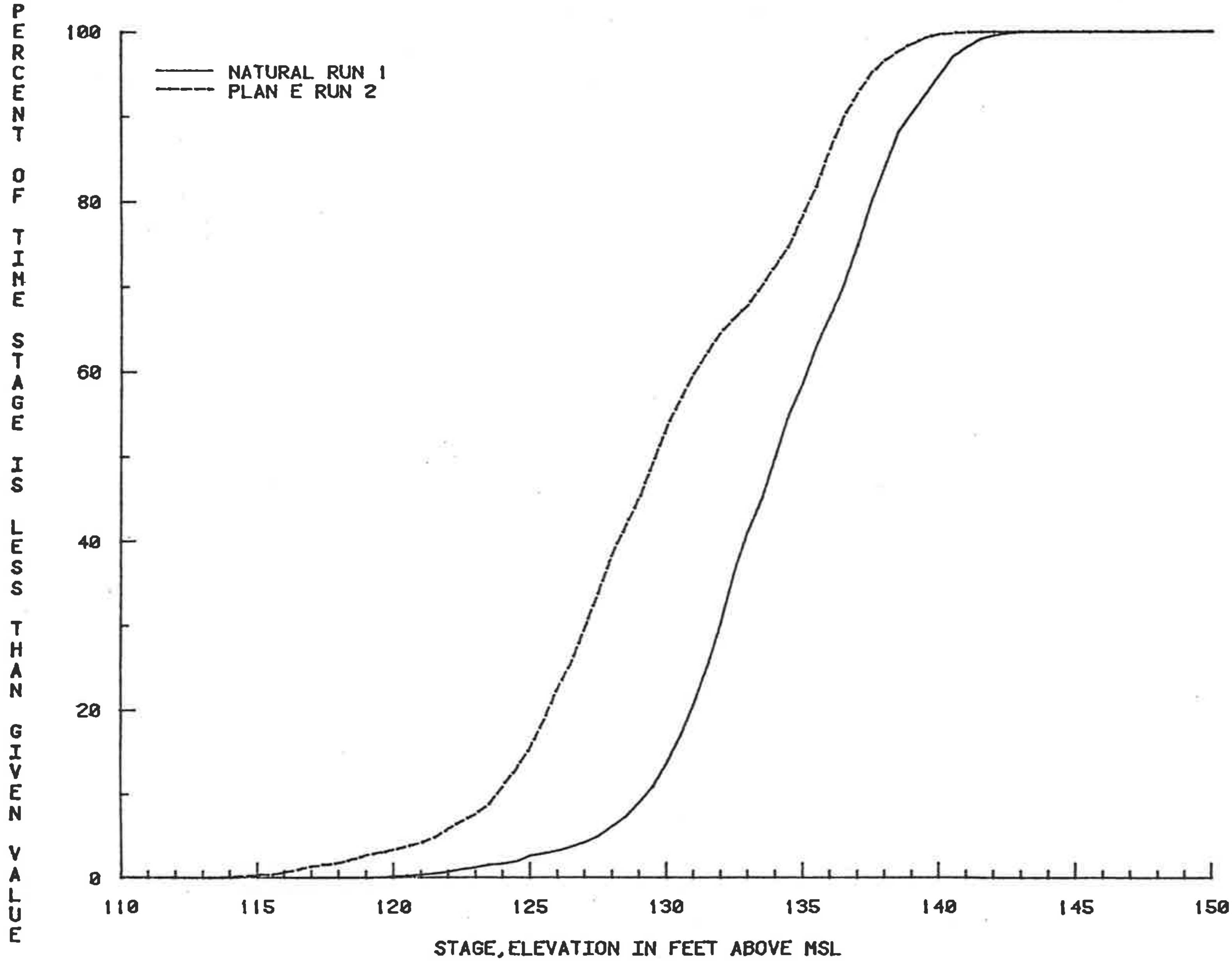


Figure 53. Stage frequency curve at Swan Lake (river mile 219.08) for natural and Plan E conditions.

sediment inflows from major sediment contributing tributaries are implemented. This is particularly true for the reach downstream of Money.

The stage-discharge relationship below Greenwood Bendway (river mile 162.5) and at Swan Lake (river mile 219.08) for different time periods are shown in Figures 54 and 55, respectively. Again, these figures indicate that the proposed Plan E can only be effective for flood control if maintenance dredging, control of sediment supply from tributaries, or other means of maintenance are implemented.

6.2.4 Runs No. 3 and 4 (Control of the Sediment from Tributaries)

Comparison of the responses of Plan E and natural conditions show that reduction of sediment from Abiaca and Tillatoba is necessary. With a 50 percent reduction in sediment inflow from Abiaca and Tillatoba (Run No. 3), a 24 percent reduction in net filling rate along the main stem is achieved. A further control in tributary sediment inflow with a 50 percent reduction of sediment from Abiaca, Pelucia, Big Sand, and Tillatoba Creeks (Run No. 4), results in a 31 percent reduction in the net filling rate along the main stem. Figure 56 shows the cumulative aggradation volumes in the main stem (sum of River Segments No. 2, 3, 4, and 8) for Runs No. 2, 3, and 4. Figure 57 demonstrates that the maximum water surface elevations are smaller for Run No. 4 compared to Run No. 2. These figures illustrate the effectiveness of maintaining the main stem by controlling the sediment from major contributing tributaries. In the proposed Phase II study, the necessary works in these tributaries utilizing grade control structures or a series of check dams to reach the desired level of sediment inflow reduction will be made.

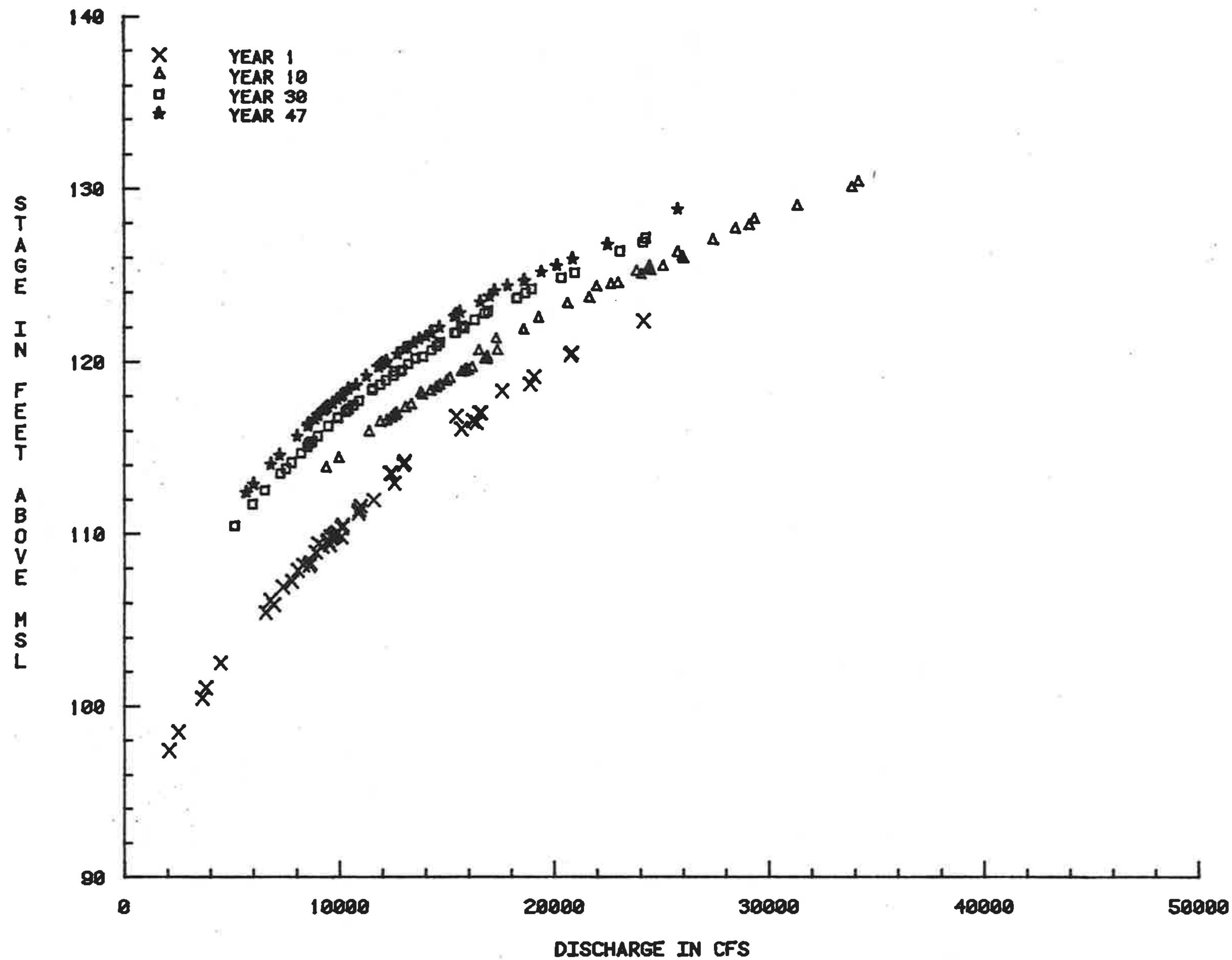


Figure 54. Stage discharge relationship at Greenwood Bendway (river mile 162.5) for Plan E.

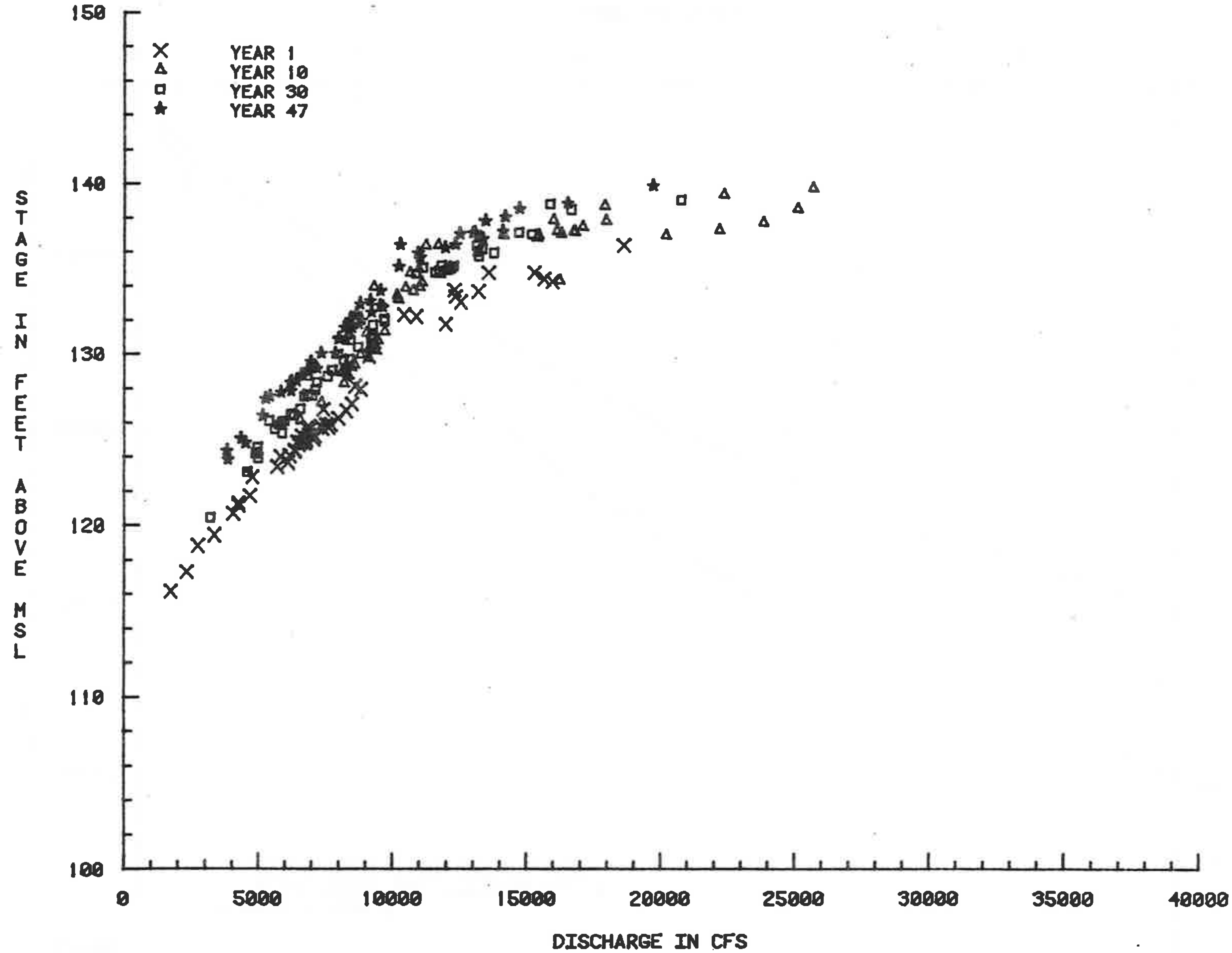


Figure 55. Stage discharge relationship at Swan Lake (river mile 219.08) for Plan E.

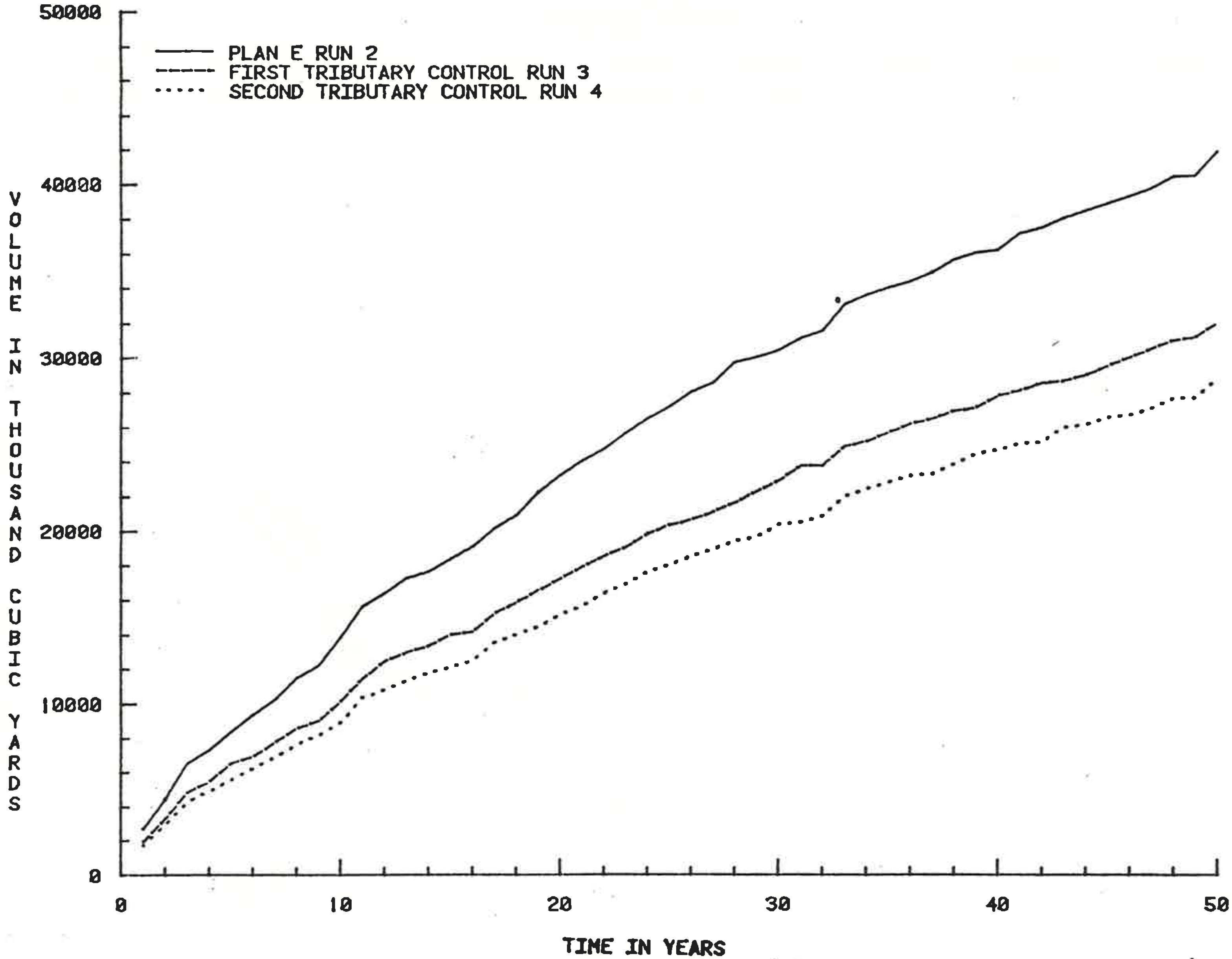


Figure 56. Cumulative aggradation (positive) volumes in the main stem for Runs No. 2, 3, and 4 (control of tributary sediment inflow).

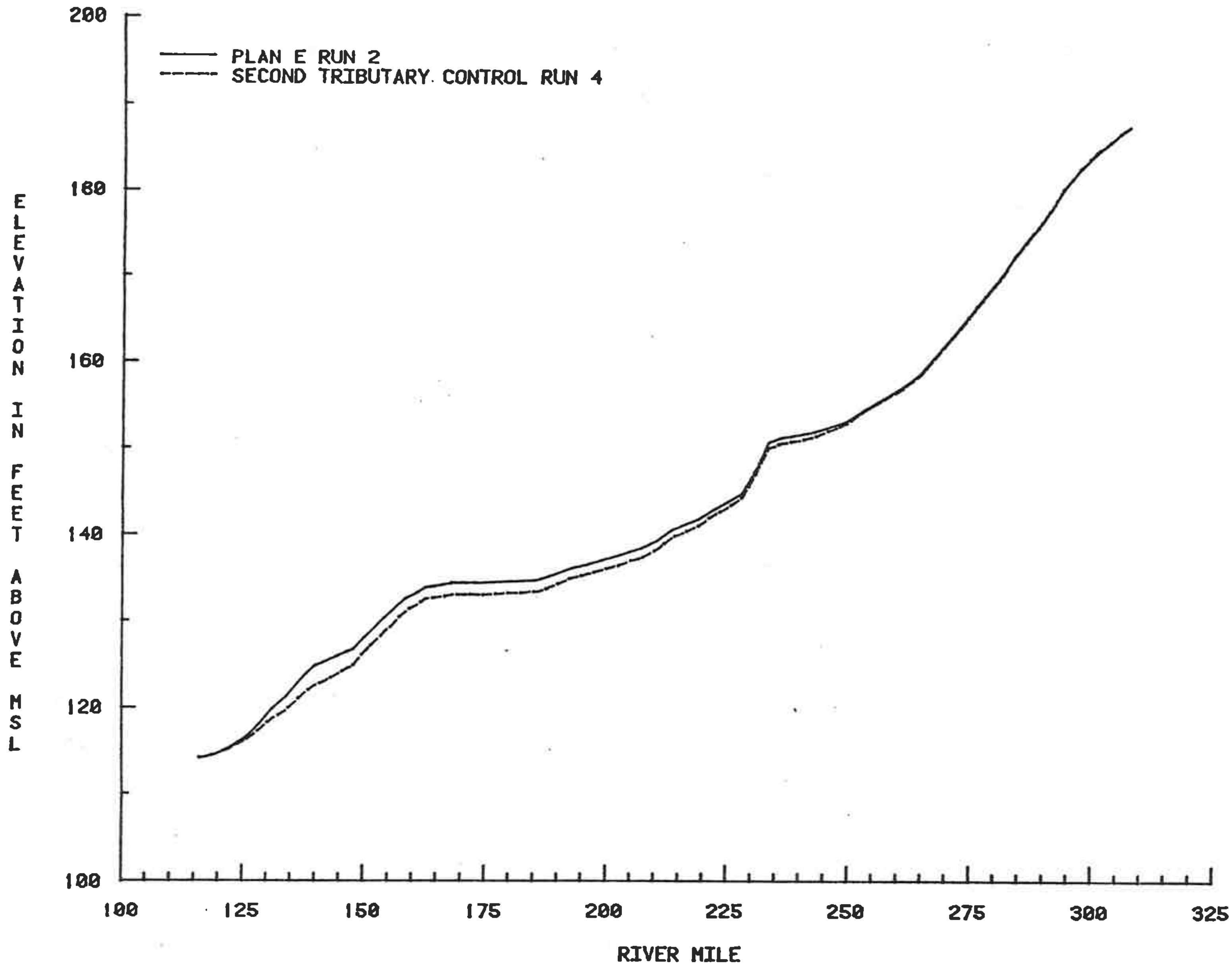


Figure 57. Maximum water surface elevations for Runs No. 2 and 4 (control of tributary sediment inflow).

6.2.5 Runs No. 5 and 6 (Effect of Channelization and/or Land Use)

The effect of channelization on the system response was evaluated by assuming a 15 percent increase in peak flows on tributaries and the main stem (Run No. 5). In addition, the effect of future land use changes on the river system was evaluated by assuming that peak flows would be increased on tributaries by an additional 15 percent. Flows on the main stem were determined by the model (Run No. 6). Results of evaluation are presented in Figure 58 with regards to cumulative net filling volumes. This figure shows that if all peak flows are increased 15 percent on both tributaries and the main stem, the rate of net filling in the main stem would increase about 13 percent. If an additional 15 percent increase in peak flows on the tributaries is imposed, an additional 19 percent increase in the net filling rate in the main stem would result.

The maximum water surface elevations along the main stem for the 50-year simulation under Runs No. 2, 5, and 6 are shown in Figure 59. In general, both the channelization and the land use change would induce the increase in the river stage.

6.2.6 Runs No. 7 and 12 (Annual and Every 5-year Dredging)

In addition to controlling the inflow of the tributary sediment, another effective means of maintaining the main stem is to utilize maintenance dredging. In this study, two time intervals (annual and every five years) of dredging were examined. The cumulative maintenance dredging volume of the two time intervals, listed in Table 8, is shown in Figure 60. If annual dredging is implemented, the total amount of maintenance dredging for 50 years is 71,346,000 cubic yards (or 1,426,000

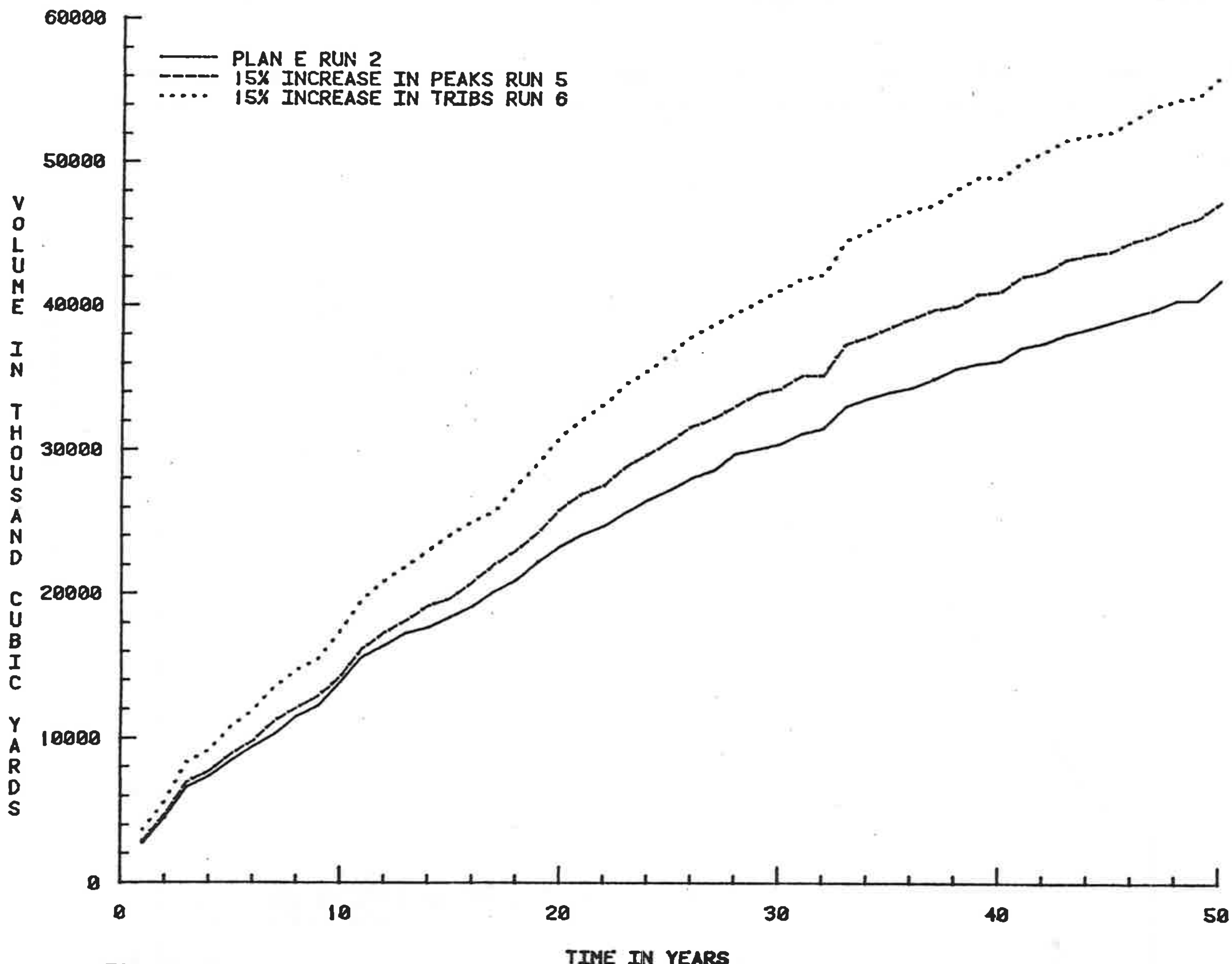


Figure 58. Cumulative aggradation (positive) volumes in the main stem for Runs No. 2, 5, and 6 (effect of channelization and land use changes).

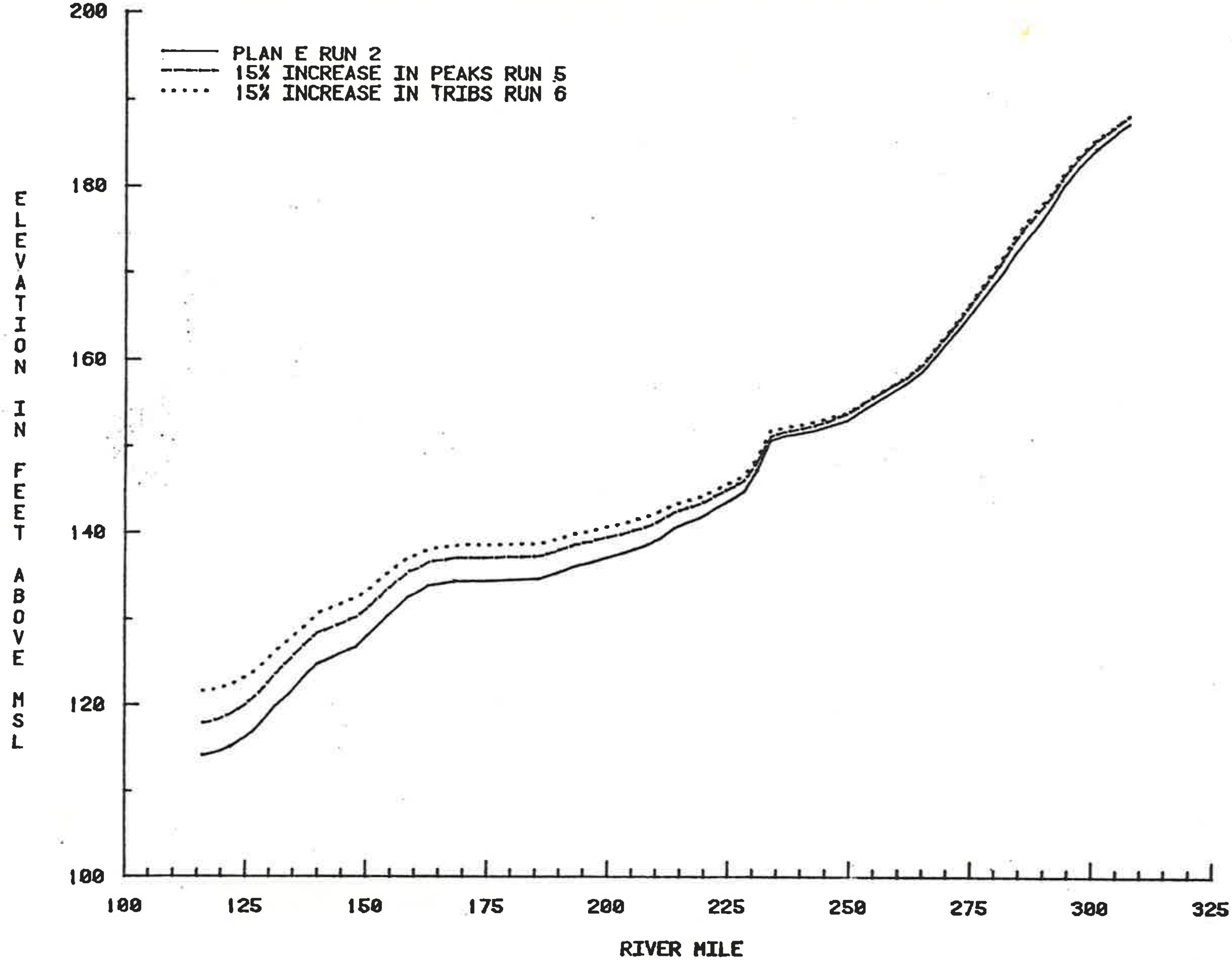


Figure 59. Maximum water surface elevations for Runs No. 2, 5, and 6 (effect of channelization and land use changes).

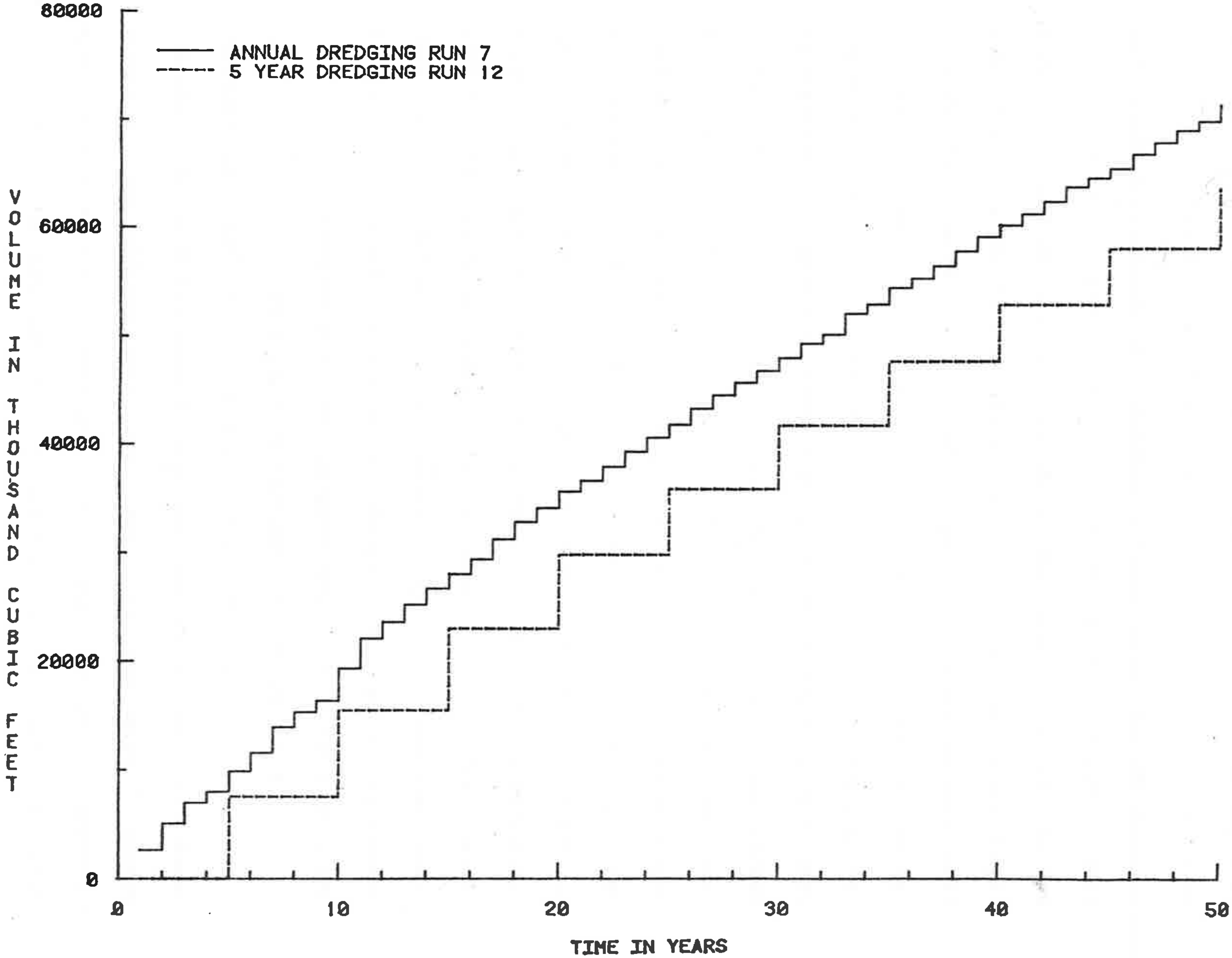


Figure 60. Cumulative dredging volumes in the main stem for annual dredging and every 5-year dredging (Runs No. 7 and 12).

cubic yards annually). If every five years the total amount of dredging is 63,608,000 cubic yards (or 1,272,000 cubic yards annually), the dredging quantity decreases 11 percent over the annual dredging.

The maximum water surface elevations along the main stem under Plan E, annual dredging, and every five year dredging (Runs No. 2, 7, and 12) for the 50-year simulation are presented in Figure 61. This figure shows that the river stages with dredging are much lower than those with no maintenance dredging. The proposed plan of flood reduction can only be effective with maintenance in the main stem. In addition, the maintenance dredging every five years would practically provide the same level of flood reduction as with annual dredging. A comparison of maximum water surface elevations for both natural and Plan E with every five-year dredging (Runs No. 1 and 12) is given in Figure 62. This figure shows that the Plan E channel with a dredging interval every five years will provide good flood protection. This is further demonstrated in Figures 63 and 64. These figures show that the stage-discharge relationships at river miles 162.5 and 219.08 are not significantly changing with time. In addition, it is important to mention that both annual and every five-year dredging, as shown in Table 7, would substantially increase the degradation in the P-Q Floodway. A refined analysis to obtain an optimum dredging schedule is suggested.

6.2.7 Run No. 8 (Construction Scheduling)

In reality, the construction of Plan E cannot be completed immediately. On the average, the rate of construction would be approximately 10 miles per year. The construction schedule was tentatively determined by the U.S. Army Corps of Engineers to proceed upstream from Yazoo City in two-year increments. At the end of eight years,

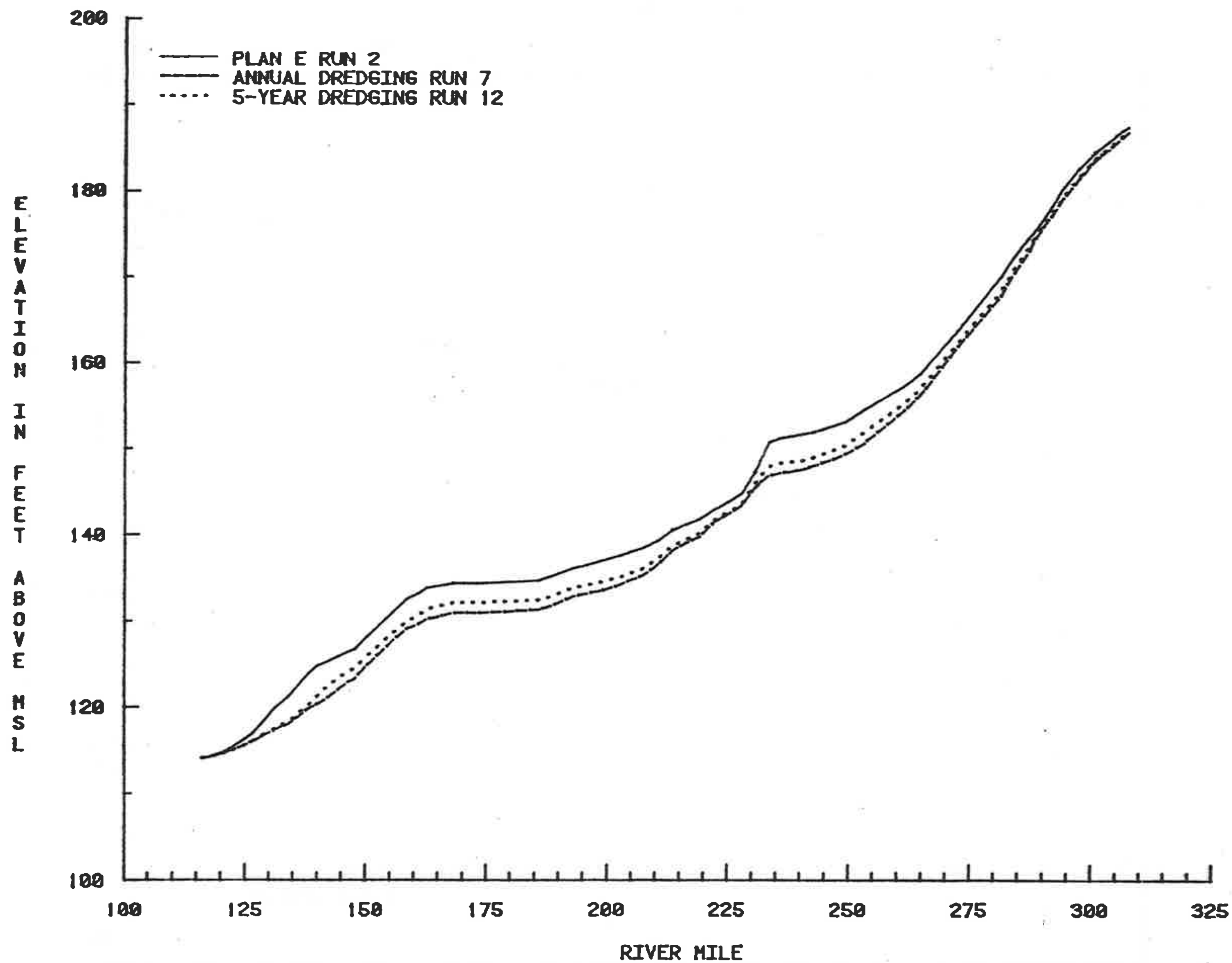


Figure 61. Maximum water surface elevations for Runs No. 2, 7, and 12 (effectiveness of dredging).

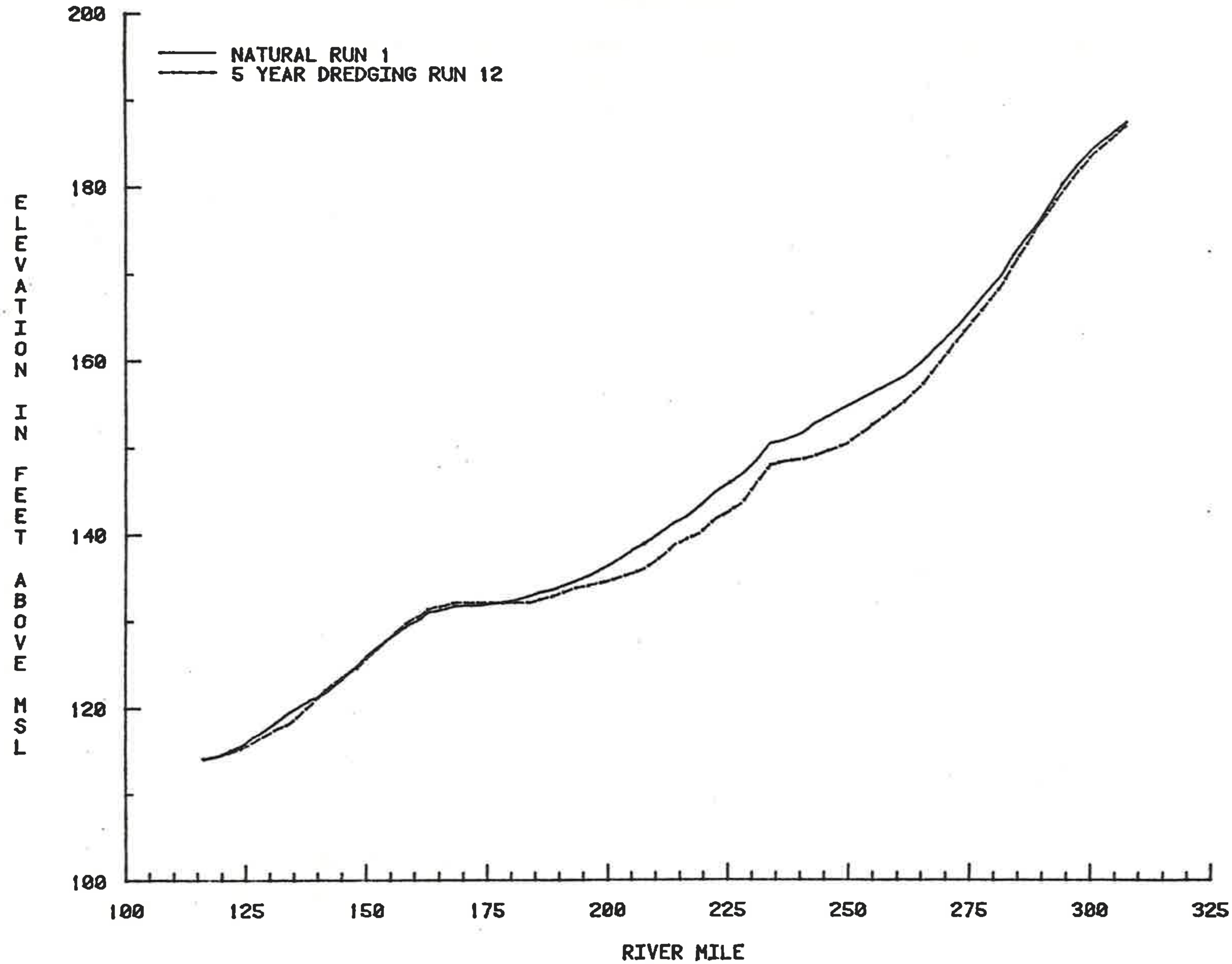


Figure 62. Maximum water surface elevations for Runs No. 1 and 12.

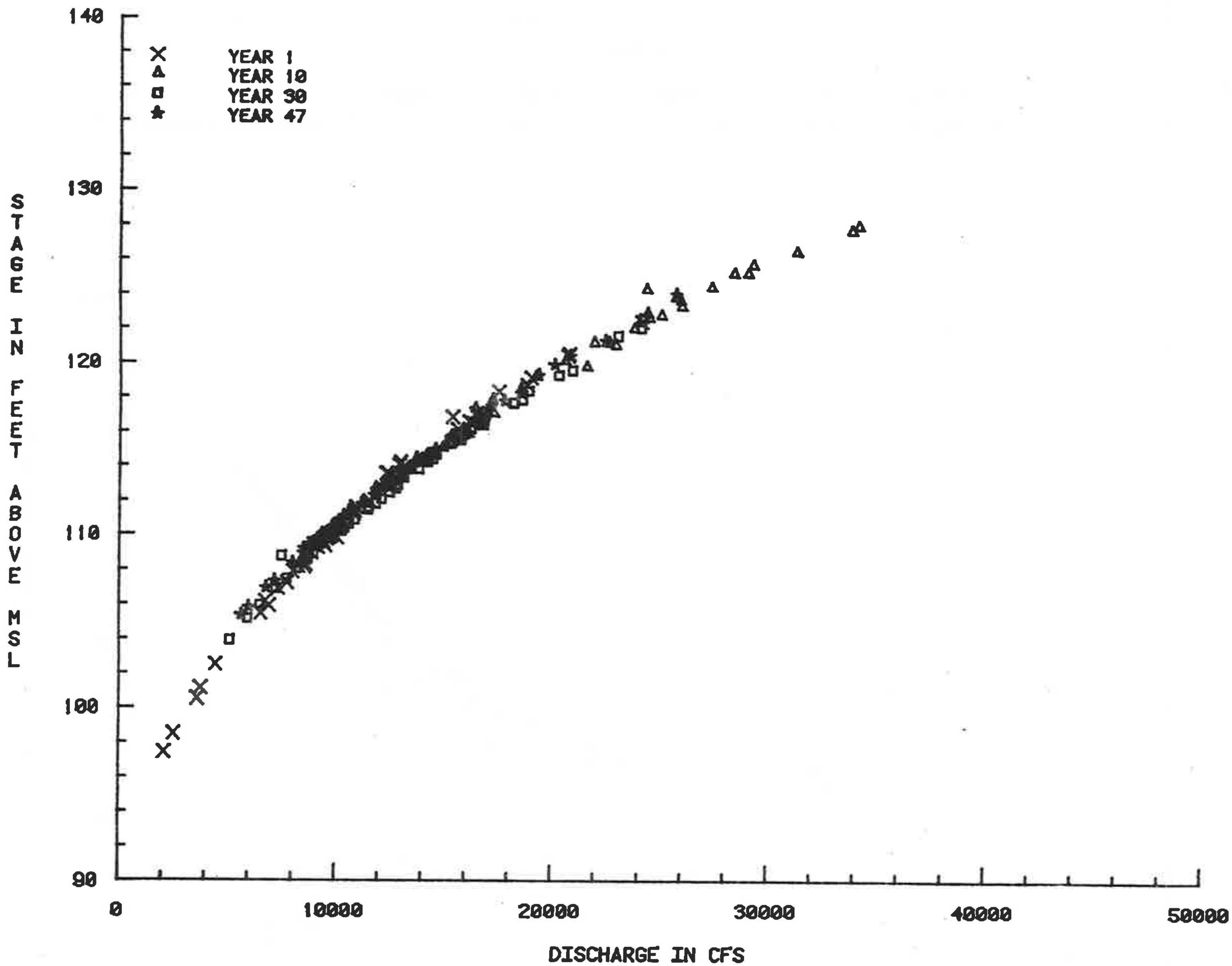


Figure 63. Stage discharge relationship at Greenwood Bendway (river mile 162.5) for every 5-year dredging.

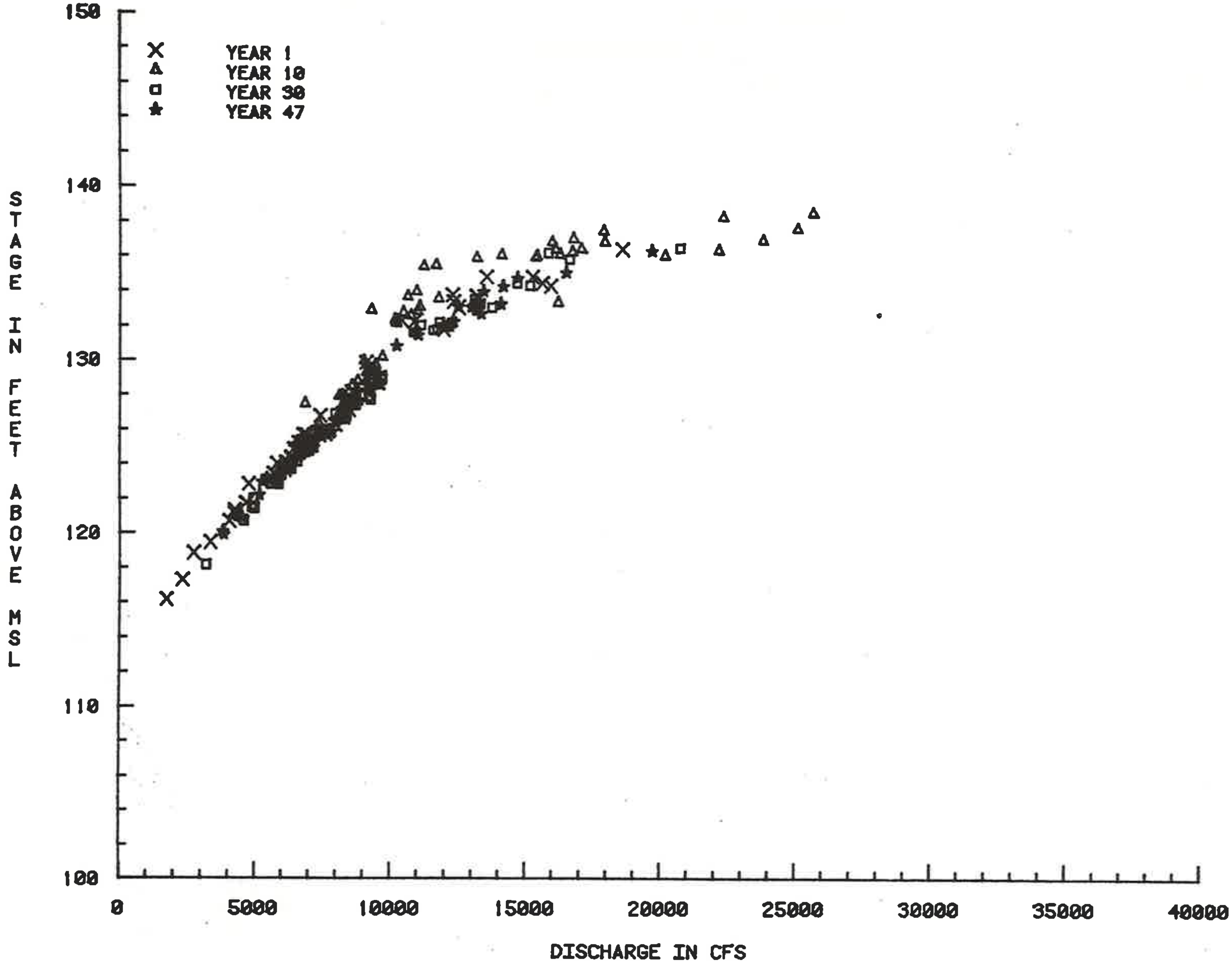


Figure 64. Stage discharge relationship at Swan Lake (river mile 219.08) for every 5-year dredging.

construction would have proceeded from Yazoo City to Greenwood. The construction would be completed to the mouth of the P-Q Floodway at the end of the 14th year. The total Upper Yazoo Project would be completed at the end of the 18th year after initiation of construction.

Run No. 8 studied the effect of construction on the system. Results of cumulative aggradation in the main stem are shown in Figure 65. This figure shows that at the beginning of construction, the net depositional rate in the main stem is smaller than those for Plan E. The rates of filling are close to one another after approximately 20 years. The total volume of degradation and aggradation is smaller for Run No. 8 as compared to the original Plan E (Run No. 2) because of the gradual rate of channelization. This justifies the assumption that immediate channelization is more conservative than construction scheduling if sedimentation is the major concern. However, considering the flooding potential, the assumption of immediate completion of Plan E is less conservative than the case with construction scheduling. Figure 66 gives the annual maximum stages at Swan Lake (river mile 219.08) for Runs No. 2 and 8. This figure shows that the flood stages, with the proposed construction schedule, are generally higher during the first 12 years. A refined analysis considering construction scheduling is important for the implementation of the Plan E channel.

6.2.8 Runs No. 13 and 14 (Modification of the Plan E Channel)

Because of severe degradation in the P-Q Floodway, it is desirable to raise the base level near the mouth of the P-Q Floodway. The large amount of deposition in the reach between Belzoni and Greenwood indicates that improved sediment transporting capacity is needed in this reach. The modification of the Plan E channel design as assumed in Run

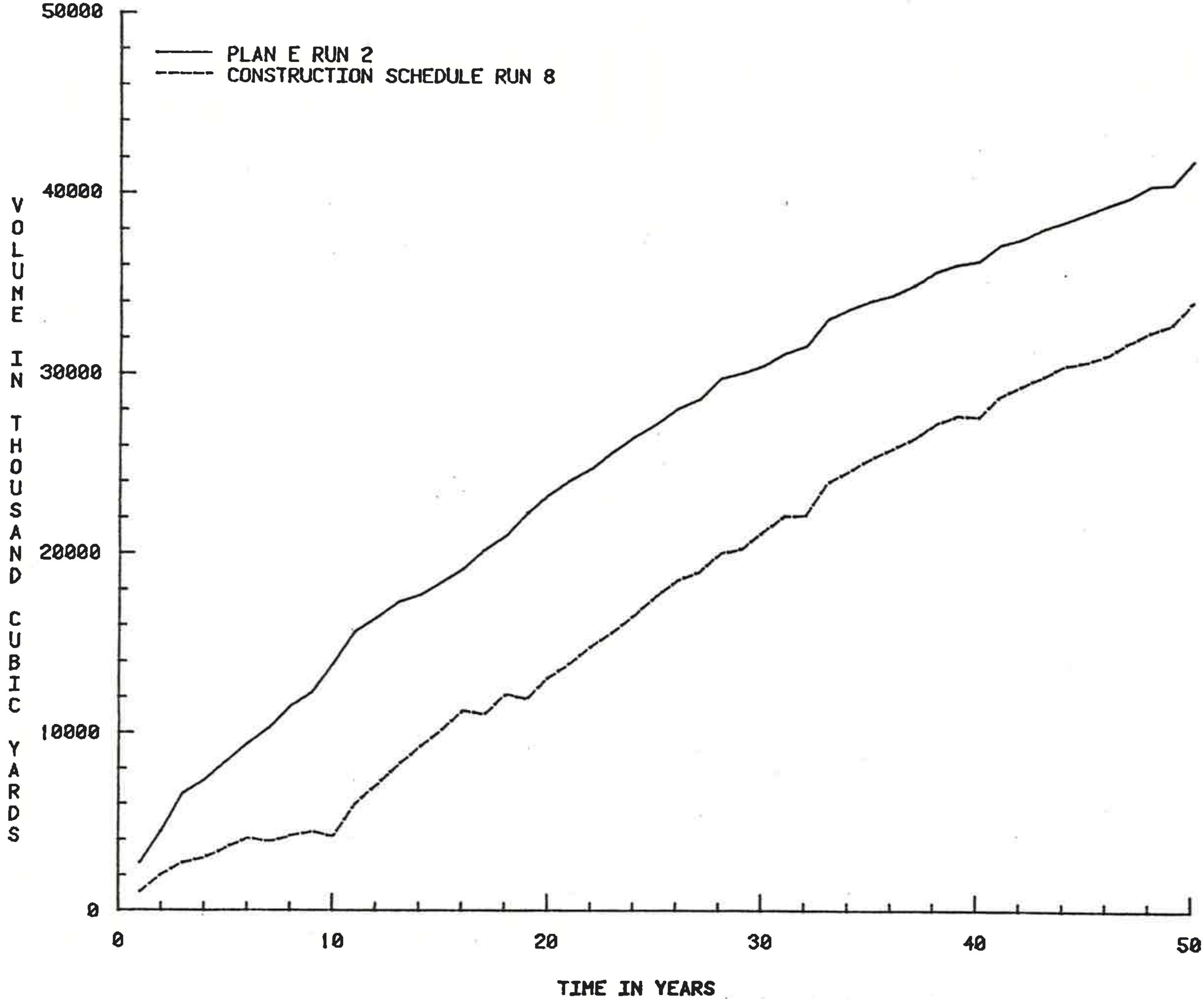


Figure 65. Cumulative aggradation (positive) volumes in the main stem for Runs No. 2 and 8 (effect of construction scheduling).

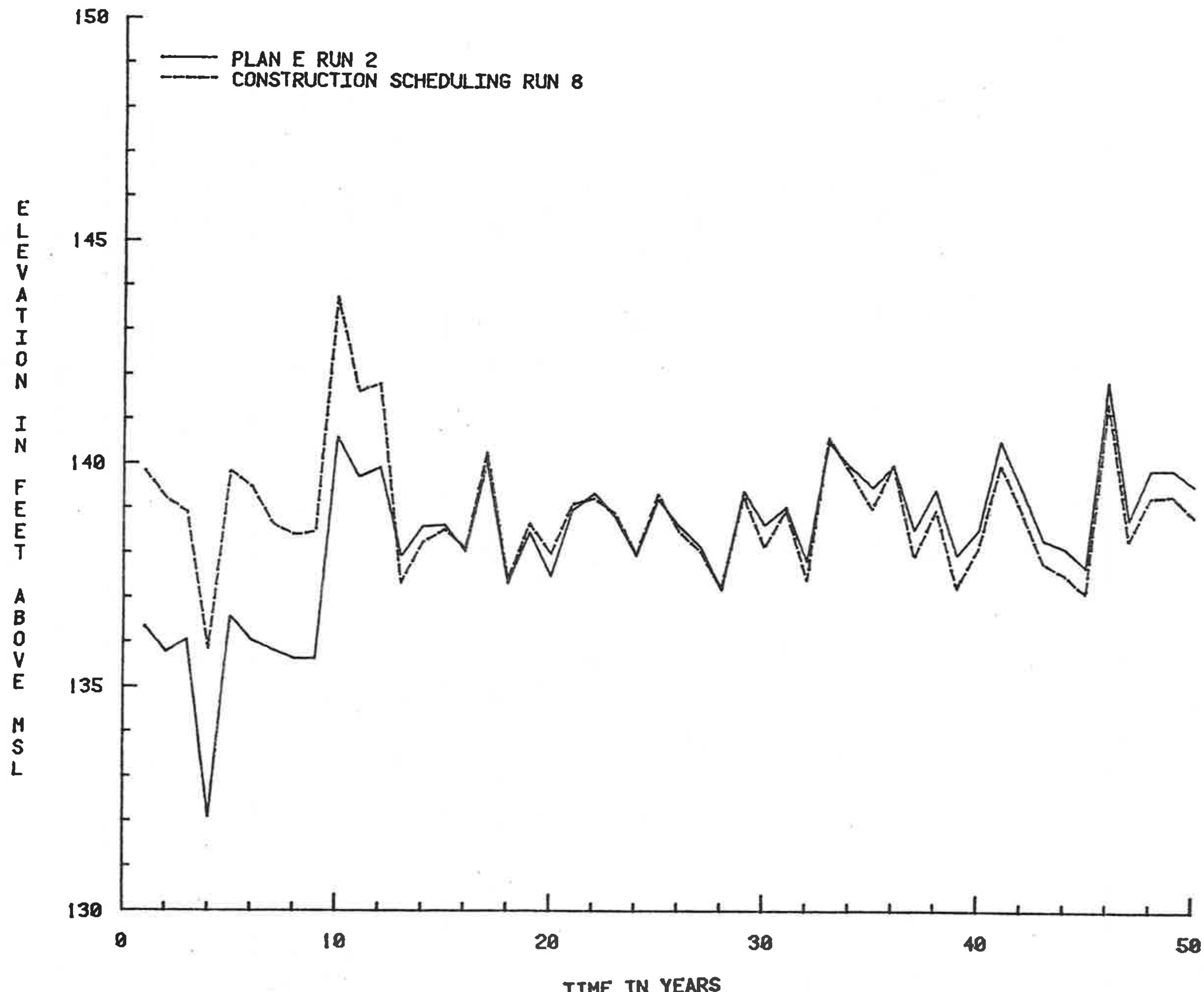


Figure 66. Annual maximum water surface elevations at Swan Lake (river mile 219.08) for Plan E and construction scheduling conditions.

No. 13 would cause a higher base level from Greenwood to Darling, and an increase in the sediment transporting capacity from Belzoni to Greenwood. This would be achieved by narrowing the channel width. The original plan for modification in the reach from Belzoni to Greenwood considers the lowering of the channel bottom at Belzoni about two feet. However, it was decided not to make this change after it was identified that the sediment transport capacity is decreased by deepening the channel.

Figure 67 compares the cumulative aggradation volumes in the main stem for Runs No. 3 and 13. This comparison indicates that channel modification will reduce the net filling rate by approximately seven percent. Reduction of the net degradation and aggradation compared to the original Plan E is about 29 percent.

The best alternative plan without maintenance dredging is assumed to be a combination of controlling tributary sediment (Run No. 4) and the channel modification (Run No. 13). This alternative study run is called Run No. 14. Figure 68 shows the cumulative aggradation volume in the main stem for Runs No. 2 (original Plan E) and 14 (the best alternative). As shown in Table 7 and Figure 68, the best alternative plan without maintenance dredging would reduce the net filling rate in the main stem about 37 percent. A comparison of the maximum water surface elevations along the main stem between the original Plan E and the best modified Plan E is given in Figure 69. Comparison indicates the best alternative would also provide a better level of flood protection compared to the original Plan E. However, the maintenance dredging is still required to provide adequate flood protection. The requirement for maintenance dredging is strongly indicated in Figures 70 and 71. These two figures show the change in the stage-discharge relationship

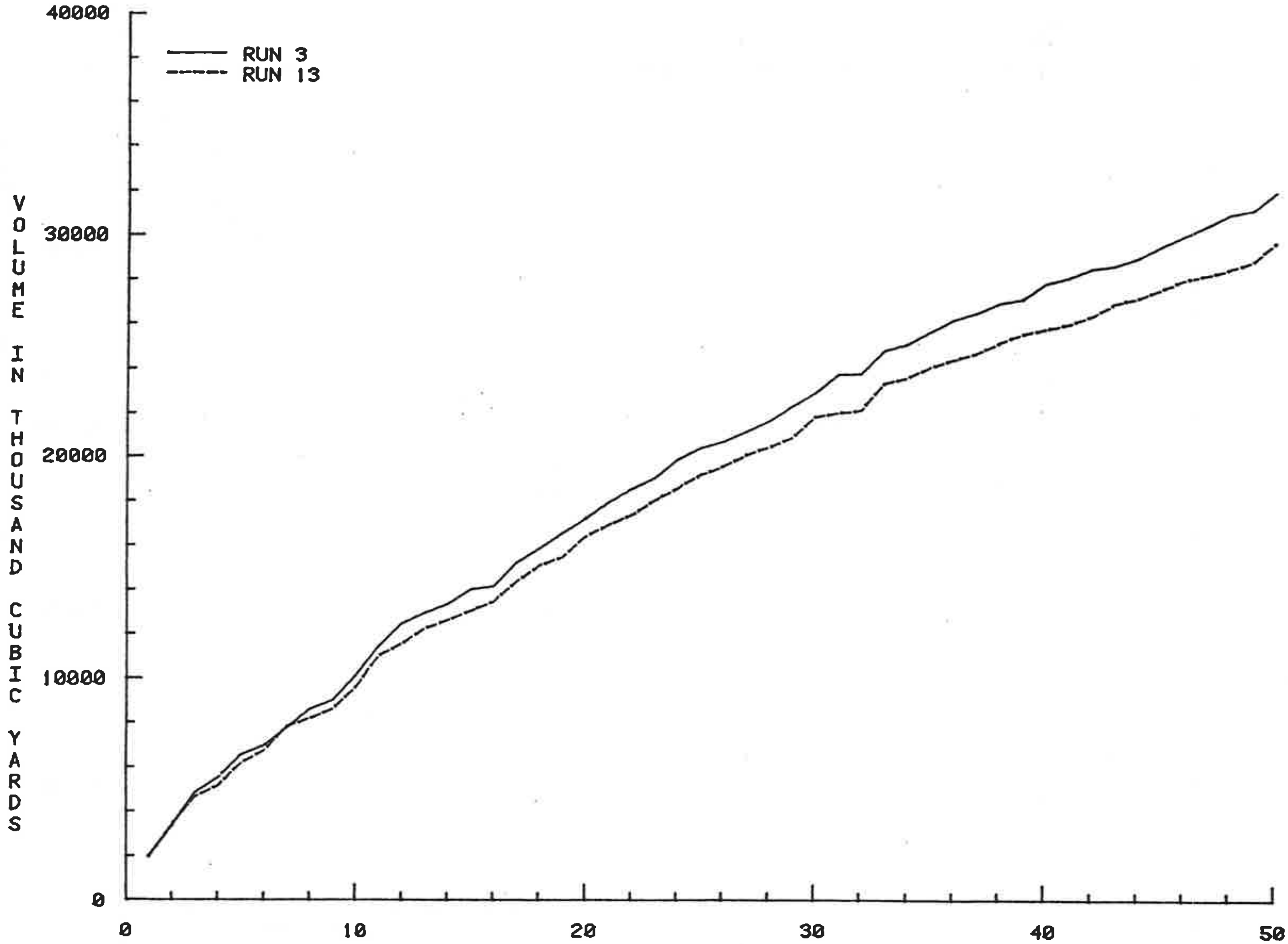


Figure 67. Cumulative aggradation (positive) volumes in the main stem for Runs No. 3 and 13 (effectiveness of channel modification).

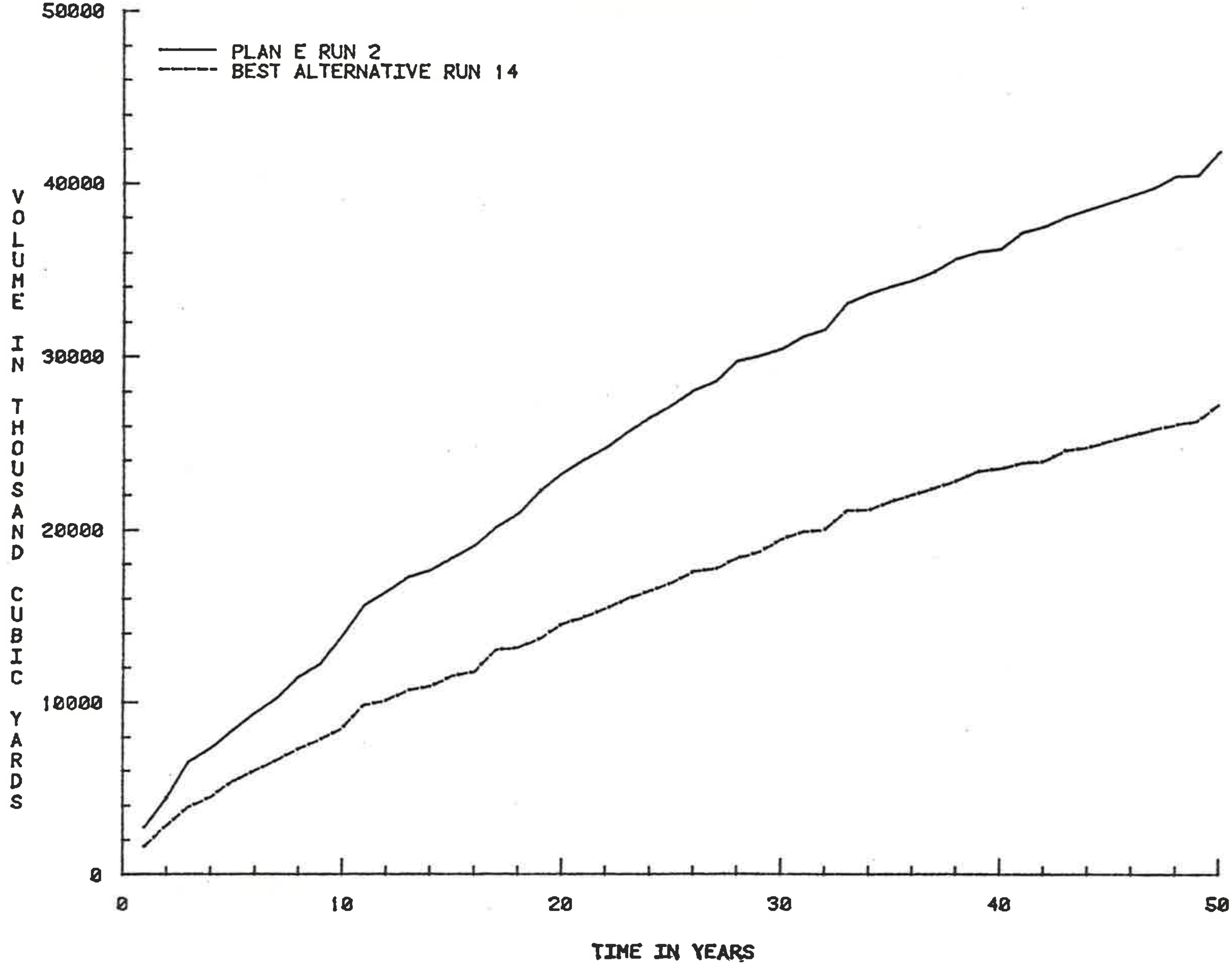


Figure 68. Cumulative aggradation (positive) volumes in the main stem for Plan E and the best alternative (Runs 2 and 14).

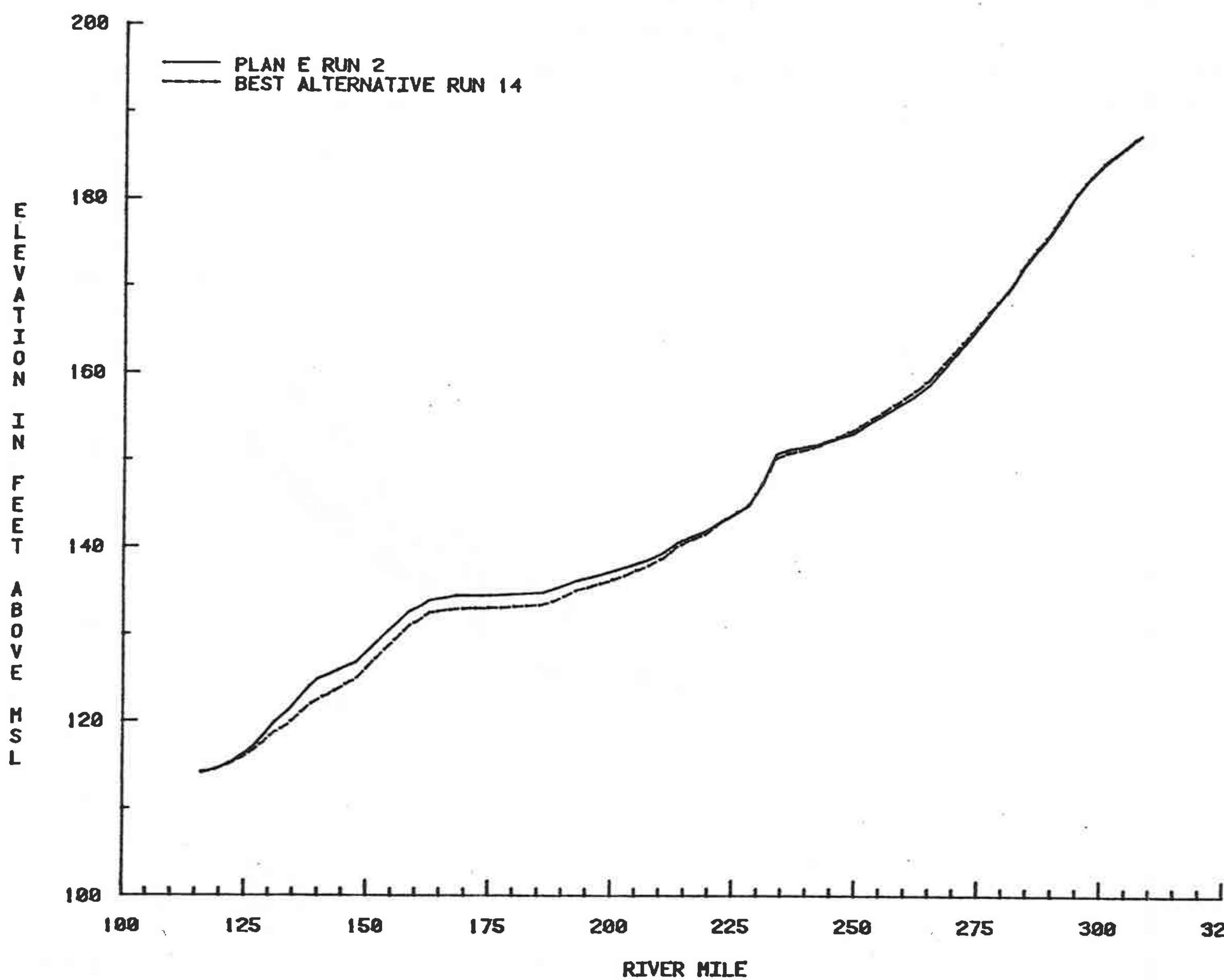


Figure 69. Maximum water surface elevations for Plan E and the best alternative (Runs No. 2 and 14).

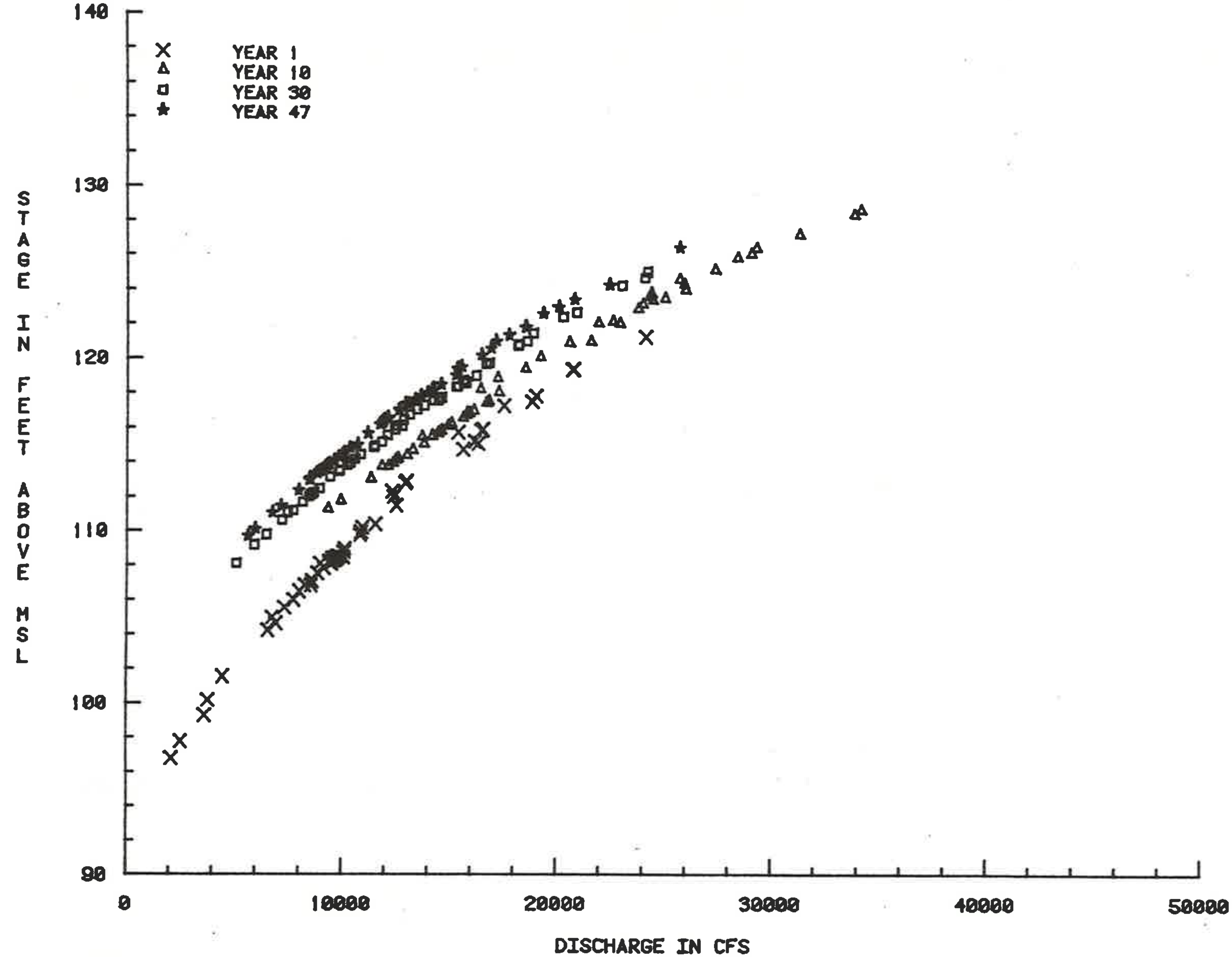


Figure 70. Stage discharge relationship at Greenwood Bendway (river mile 162.5) for the best alternative.

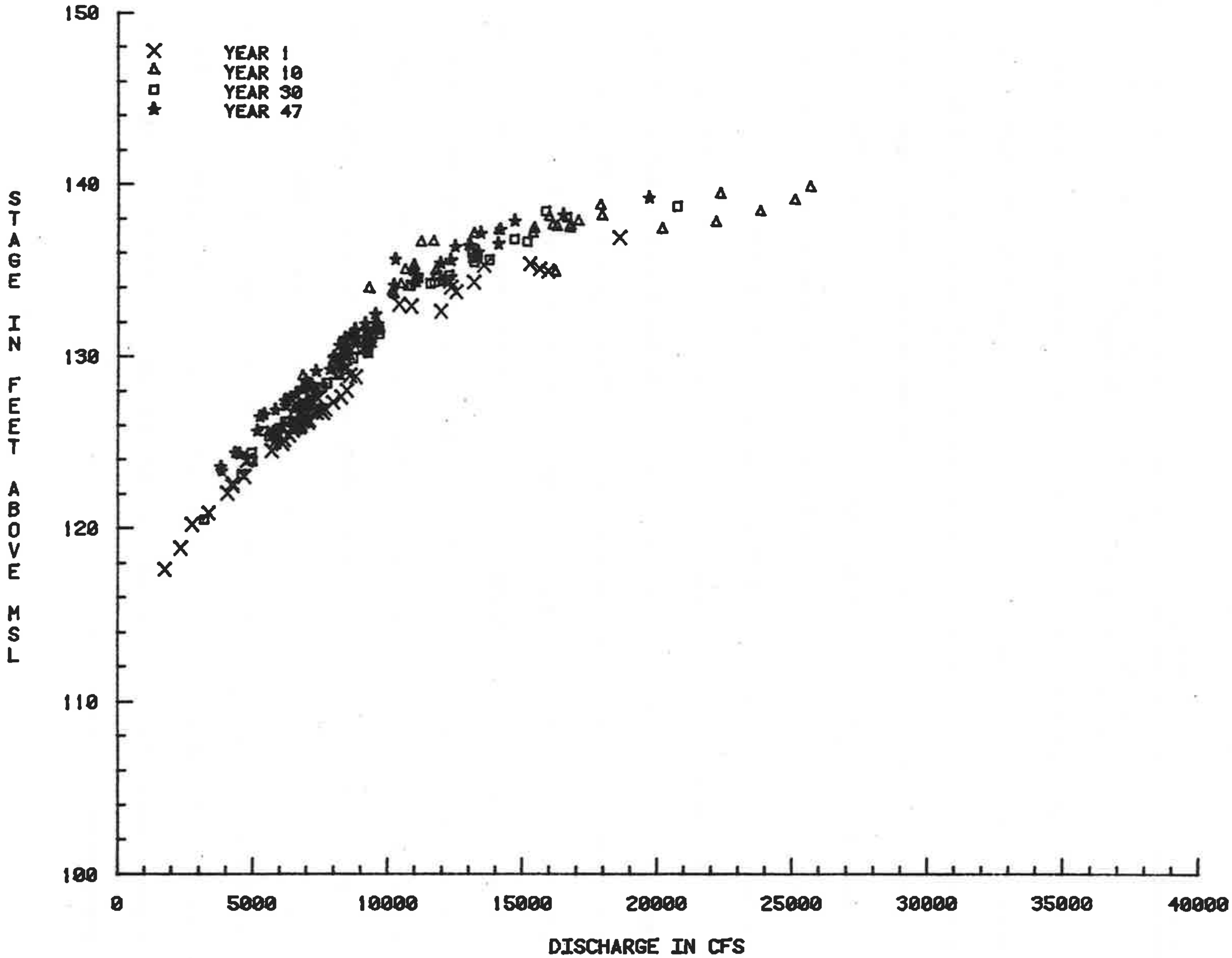


Figure 71. Stage discharge relationship at Swan Lake (river mile 219.08) for the best alternative.

at Greenwood and Swan Lake over 50 years of simulation for Run No. 14. As can be seen, both locations experienced an increase in the stage for a given discharge over a period of time. This is particularly true for reaches downstream of Money.

6.2.9 Runs No. 11 and 15 (Impact on the Lower Reach and the Proposed 9-Foot Navigation Channel)

The majority of study runs were conducted for the river system above Belzoni. The sediment deposition rate caused by the Upper Yazoo Project on the lower reach, from Belzoni to Vicksburg, was evaluated in Run No. 11. As presented in Table 7, the net deposition and aggradation volumes above Belzoni are very close for Runs No. 2 and 11. This indicates that the assumption of Belzoni being a downstream control for evaluating the response of the Plan E channel is reasonable.

The feasibility analysis of the proposed nine-foot navigation channel was studied in Run No. 15. This study run assumed that a minimum pool with a water surface elevation of 65 feet exists at the proposed lock and dam near Vicksburg (for simplicity it was assumed to exist at the mouth of the Big Sunflower), and the channel has a 200-foot bottom width from Vicksburg to Greenwood. The sediment filling rate in the lower reach is 78 percent larger than that without development of a navigable channel. In addition, this development would increase the quantity of deposition in the upper main stem reach. The cumulative aggradation volumes in the lower reach (River Segment No. 1) for both Plan E and navigation conditions are presented in Figure 72. This figure shows that the impact of the navigation channel on the lower reach is not too significant.

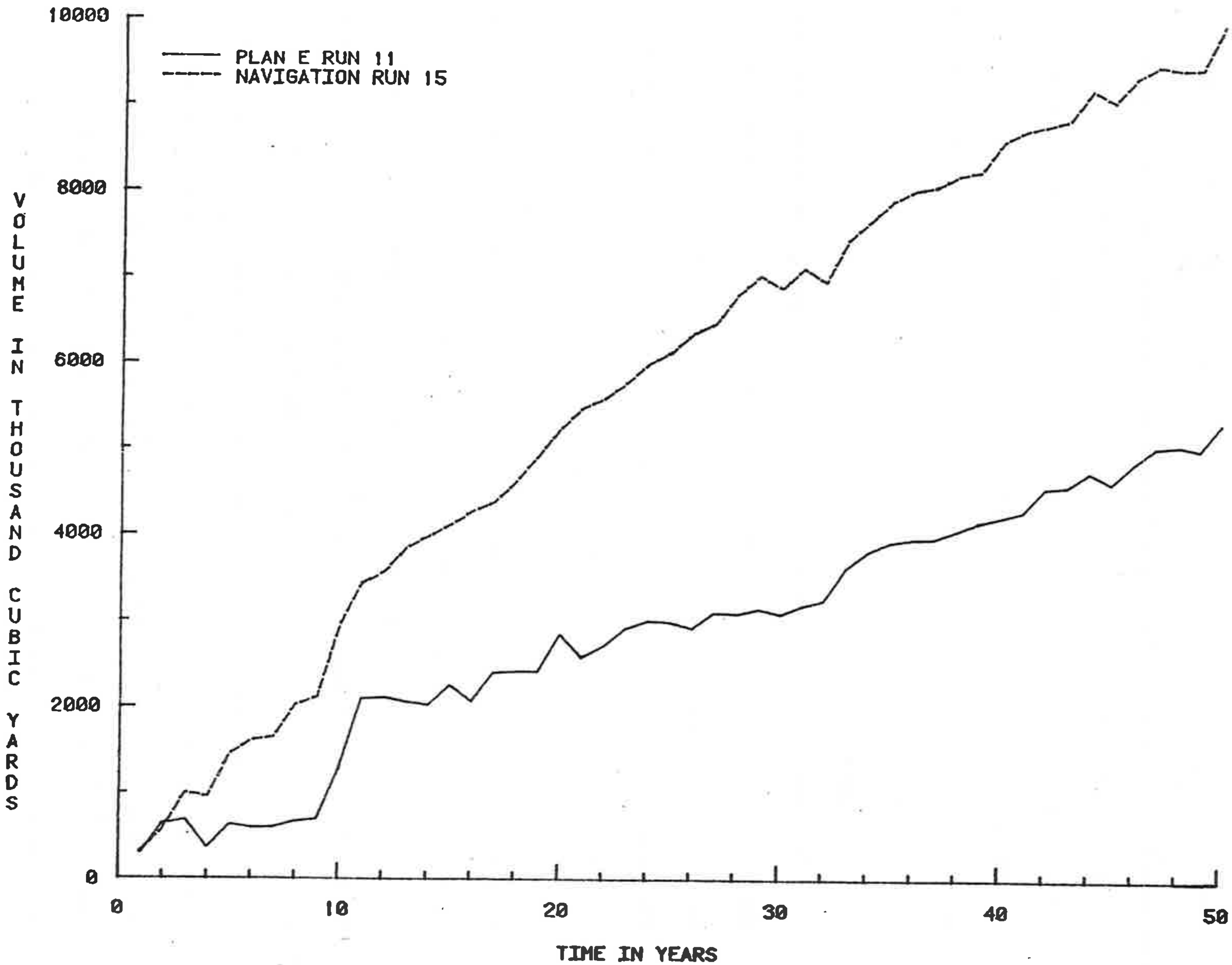


Figure 72. Cumulative aggradation (positive) volumes in the lower reach for both Plan E and navigation channels (Runs No. 11 and 15).

Figure 73 shows that minimum thalweg depths along the main stem from the mouth of the Big Sunflower to Greenwood for the 50-year simulation are generally larger than nine feet except in the Greenwood Bendway and downstream of Abiaca Creek. Therefore, the proposed nine-foot navigation channel with a lock and dam near Vicksburg is a feasible development alternative if the Plan E channel is maintained, particularly in the reach between Belzoni and Greenwood. However, a more detailed analysis is needed for designing, implementing, and maintaining the nine-foot navigation channel.

6.3 Summary

In summary, the proposed Plan E would slightly increase the peak discharge, generally reduce the flood stages if the channel is maintained, and significantly increase the amount of deposition in the main stem. The proposed Plan E can provide adequate flood protection for the upper headwater areas if the Plan E channel is maintained by the reduction of sediment inflows from major contributing tributaries and/or providing maintenance dredging about every five years. Furthermore, the necessity of maintenance dredging is more critical below Money.

Modification of the design channel is recommended to allow the higher base level near the mouth of the P-Q Floodway and an increase in the sediment transporting capacity from Belzoni to Greenwood by narrowing the channel bottom width. This modification to the Plan E channel plus reducing 50 percent of the sediment inflows from Abiaca, Pelucia, Big Sand, and Tillatoba Creeks, will result in a 37 percent reduction of the net filling rate in the main stem as compared to the original Plan E. The impact of construction scheduling does not significantly change the net filling rate in the main stem. In addition, the operation of the

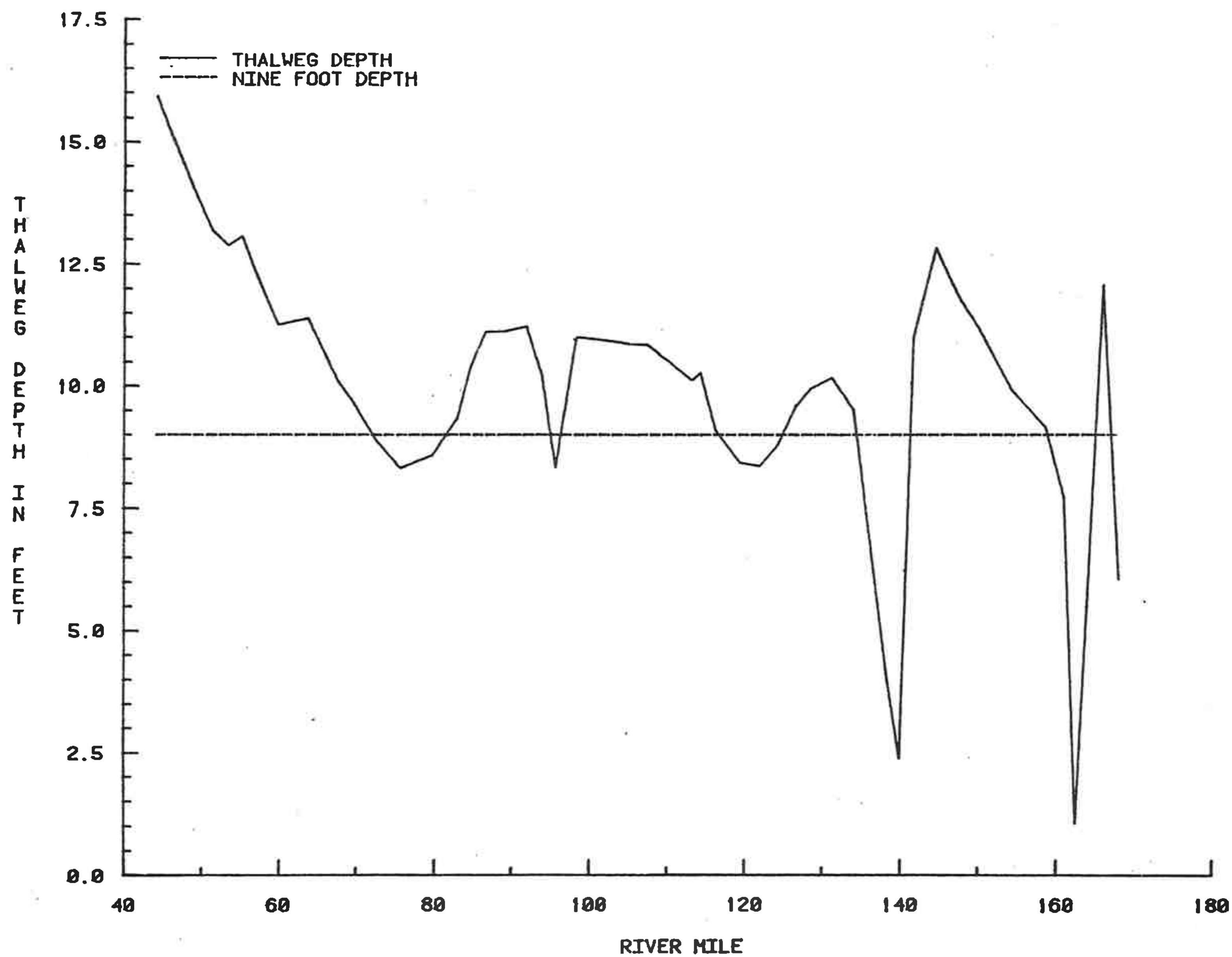


Figure 73. Minimum thalweg depth along the main stem.

proposed nine-foot navigation channel will increase sediment deposition in the lower Yazoo by 78 percent. A more detailed analysis of the tributaries and the maintenance dredging schedule is needed for adequate design, implementation, and maintenance of the Upper Yazoo Project.

VII. RECOMMENDATION FOR FUTURE STUDIES

7.1 General Recommendations

The system analysis presented in this report is a very useful methodology for predicting the response of a complicated system like the Yazoo River Basin. Due to time and funding constraints, the Phase I study was limited to evaluation of the response of the main stem and major tributaries only. Future studies that would provide additional information important for the design and implementation of the Upper Yazoo Project are recommended as follows:

1. A detailed study of tributaries and watersheds, and their effects on the main stem.

2. A refined analysis of construction and dredging scheduling of the Upper Yazoo Project. This refined analysis would provide necessary information for scheduling construction and/or dredging activities in order to minimize environmental impacts and construction costs.

3. A refined design of channel dimensions considering the trade-offs between the minimization of sedimentation problems and of flooding potentials.

4. Development of a management model for automatically selecting the design alternatives for minimizing the main stem maintenance problems considering tributary sediment control, channel dimensions, flow regulation, flood control, environmental impact, and navigation.

5. Development of a comprehensive flood forecasting system that considers rainfall as the primary input which routes water from watersheds to tributaries and on to the main stem.

6. Development of a comprehensive data storage and retrieval system for managing the large amount of data necessary for such concise river planning and analysis models.

7.2 Specific Recommendations for the Phase II Study

In the Phase I study, the emphasis was to evaluate the response of the main stem and major tributaries to the various design alternatives. Initial results indicate that hill tributaries and watersheds are important in maintaining the main stem. The Phase II analysis will produce a detailed study of tributaries and watersheds, and their effects on the main stem. This analysis is essential to provide sufficient information for detailed basin development designs and economic considerations. This expanded Phase II study will evaluate the effects of river response to long-range development of the river and watersheds with particular emphasis on flood control, navigation needs, drainage refinements, environmental impacts, and water quality. The following is a list of general recommendations regarding work to be undertaken in the proposed tributary study.

1. Refinement and expansion of the water and sediment routing model developed in the Phase I study should be continued to include detailed analysis of the major sediment contributing tributaries in the Basin. This expanded model will include additional hydrologic, geologic, water, sediment, and water quality data for each major tributary outlined for investigation in the initial (Phase I) study. It will also include general, available data for other tributaries not specifically analyzed in the initial sedimentation study. Major modifications to the model will include a more detailed analysis system consisting of watersheds, tributaries, and the main channel. The watershed model will utilize a

two plane-one channel geometry representation, and the tributary model will consider the response of grade control structures and a series of detention dams. Other modifications should consider the future inclusion of additional water quality parameters as identified in Item 8.

2. The existing model of the main stem Yazoo River Basin and the expanded model to be developed for the major sediment contributing hill tributaries should be utilized to determine the extent of existing sediment problems in each of the hill tributaries outlined for study. Identification of solutions should be made for the existing instabilities on these tributaries. The extent of effort on data collection and modeling for each tributary is dependent on the relative importance of the tributary considering main stem sedimentation problems, availability of data, type of control structure to be implemented, and other factors.

Twenty-four tributaries were identified in the Phase I study as primary sources of input to the Upper Yazoo Project. These tributaries are:

Short Creek
Piney Creek
Techeva Creek
Fannegusha Creek
Black Creek
Tchula Lake
Abiaca Creek
Pelucia Creek
Yalobusha River
Tillatoba Creek
P-Q Floodway
Little Tallahatchie River
Strayhorn Creek
Arkabutla Creek
Lake Cormorant Bayou
Yocona River
Peters Creek
McIvor Ditch
Big Sand Creek
Teoc Creek
Potococowa Creek
Ascalmore Creek
Cane Creek
Batupan Bogue

For convenience in identifying the levels of study needed for each tributary, the following four groups of tributaries are defined: Group A consists of important tributaries below the major dams and includes the Yalobusha River, P-Q Floodway, Little Tallahatchie River, and Yocona

River. These tributaries were included in the Phase I study and most of the required data are available except new cross section and sediment data. Detailed analysis considering possible grade control structures and other measures to minimize sedimentation problems, will be conducted in these tributaries.

Group B tributaries include important sediment contributing tributaries not included in the Phase I study. These tributaries require intensive data collection and include Abiaca Creek, Pelucia Creek, Big Sand Creek, Tillatoba Creek, Potococowa Creek, and Arkabutla Creek. Analysis will assume certain control measures are in place for minimizing sedimentation problems. In addition, a watershed study is also needed to estimate discharge hdyrographs.

Group C consists of relatively unimportant tributaries but ones required for estimating discharge hydrographs. Limited efforts on data collection are needed for this group, particularly with regard to cross section and sediment data. The modeling effort involving this group will provide a correct estimation of water and sediment inflow to the main river system. Group C includes: Techeva Creek, Fannegusha Creek, Black Creek, Teoc Creek, Strayhorn Creek, and Ascalmore Creek.

Group D requires no major effort for modeling these watersheds or tributaries; however, limited data collection on the cross sections and sediment transport is needed. This group includes Short Creek, Piney Creek, Tchula Lake, Lake Cormorant Bayou, McIvor Ditch, Peters Creek, Cane Creek, and Batupan Bogue.

The classification of tributaries is tentative and will be finalized in consultation with the U.S. Army Corps of Engineers. As a minimum, the Phase II study will include: Four Group A tributaries (Yalobusha

River, P-Q Floodway, Little Tallahatchie River, and Yocona River), three Group B tributaries (Abiaca Creek, Pelucia Creek, and Tillatoba Creek), three Group C tributaries (Techeva Creek, Teoc Creek, and Strayhorn Creek), and four Group D tributaries (Tchula Lake, Peters Creek, McIvor Ditch, and Cane Creek).

3. The existing main stem model and the expanded model of the major tributaries (Group A and B) will be utilized to determine the response of major hill tributaries to the proposed Upper Yazoo Project. Through detailed studies of each tributary, the existing channel instabilities, and those occurring as a result of construction of the Upper Yazoo Project will be delineated. Proposed work may include grade control structures, additional bank stabilization, detention dams or other structural measures, and evaluation of practices such as land use changes, and effects of varying watershed cover. These evaluations will also include and assess the overall effects of proposed U.S. Army Corps of Engineers, U.S. Soil Conservation Service, and other State, Federal, or private interest projects such as dams, channel improvement, levee systems, and bank stabilization schemes. In addition, consideration to projected land use changes in the watersheds will be made. Study runs for each tributary will consist of a 50-year hydrograph for the entire system and design storms of return periods of 5, 10, 25, 50, and 100 years on the tributary studied.

4. The watershed model will be applied to estimate the discharge hydrograph for Group B and C tributaries. Effects of land use changes and/or varying watershed vegetative cover will also be evaluated using the watershed model.

5. Evaluations of the existing and projected sedimentation problems in the major hill tributaries will be made by routing water and sediment through the tributaries and the Yazoo River main stem assuming various alternative plans are finalized. These evaluations will indicate not only changes or improvements in the stability of the tributary under study, but will provide important information regarding the reduction of sediment deposition in the main stem resulting from the alternative plan. This evaluation should be made so economic considerations involving use of grade control, stabilization, or other works on the hill tributaries versus maintenance dredging on the main stem can be defined. After the completion of Items 2, 3, and 4, a set of feasible alternative plans can be selected. Evaluation of existing and projected sediment problems in the major hill tributaries and the main channel can be made assuming these identified alternative plans are completed and utilize a 50-year hydrograph.

6. The existing model and the proposed tributary model will be updated using all available data in the Yazoo Basin, some of which will be collected specifically for this study and periodically updated throughout the study. Routings will be made with this updated data to determine major sources and quantities of sediment that will be deposited in the main stem, and the magnitude of the maintenance program that would be required to keep the Upper Yazoo Project operational. These estimates of maintenance dredging quantities will facilitate comparison of alternative structural solutions for hill tributary problems with respect to economic and environmental viewpoints.

The current data base for each tributary is not adequate and it is estimated that as a minimum, additional data from field surveys at approximately 100 cross sections on the tributaries identified in Item 2 are needed. The estimated numbers of cross sections required for each tributary are: 10 cross sections for each Group A tributary, 12 cross sections for each Group B tributary, four cross sections for each Group C tributary, and three cross sections for each Group D tributary. Data to be collected must include: cross sections, suspended sediment samples, bed and bank material samples, velocity, and water discharge.

7. The main stem and tributary model and results obtained from routing that consider various structural or other alternatives will be used in the preliminary design of structural alternatives on Group A and B tributaries. These include:

a. Grade control works--size, location, elevations, number, type of structure, and other pertinent data.

b. Detention dams--location, size, and operating criteria.

c. Bank stabilization works--location of areas requiring work, estimate of type of stabilization, e.g., riprap protection, dikes, fences, and vegetation on waterways.

d. Other structural measures--pertinent engineering data for structural and economic analysis and design.

8. The main stem and tributary model will be developed to serve as an even more sophisticated model for future analysis of environmental problems in the Yazoo Basin. Transport of nutrients, pesticides and other contaminants through the system that move in suspension, as well as those attached to the sediments, can be studied. The model should be designed and constructed so modifications can be made to route various chemicals through the system. Such a model will provide a means of studying all aspects of proposed developments in the Yazoo Basin.

APPENDIX A

This appendix contains the detailed spatial design of the Yazoo River Basin used in the known discharge sediment routing model. These figures show the locations of all cross sections used in the model as well as locations of tributaries and important towns. Nonpoint sources of water, which can be positive or negative, are represented by the initials NPS. Brackets or arrows indicate the areas or points at which the nonpoint sources were added to the river flow.

This appendix also contains bed profile plots of the four different channel designs that were analyzed: natural, Plan E, Modified Plan E, and navigation. The navigation channel's profile is identical to Plan E above river mile 75.5, while the modified channel profile is identical to Plan E below Greenwood Bendway, river mile 162.5. Additional data on the spatial and temporal designs used in the known discharge model are contained in the following reports:

"Cross Sectional Data," by D. B. Simons, R. M. Li, and G. O. Brown.

"Temporal Design," by D. B. Simons, R. M. Li, T. J. Ward, and N. Duong.

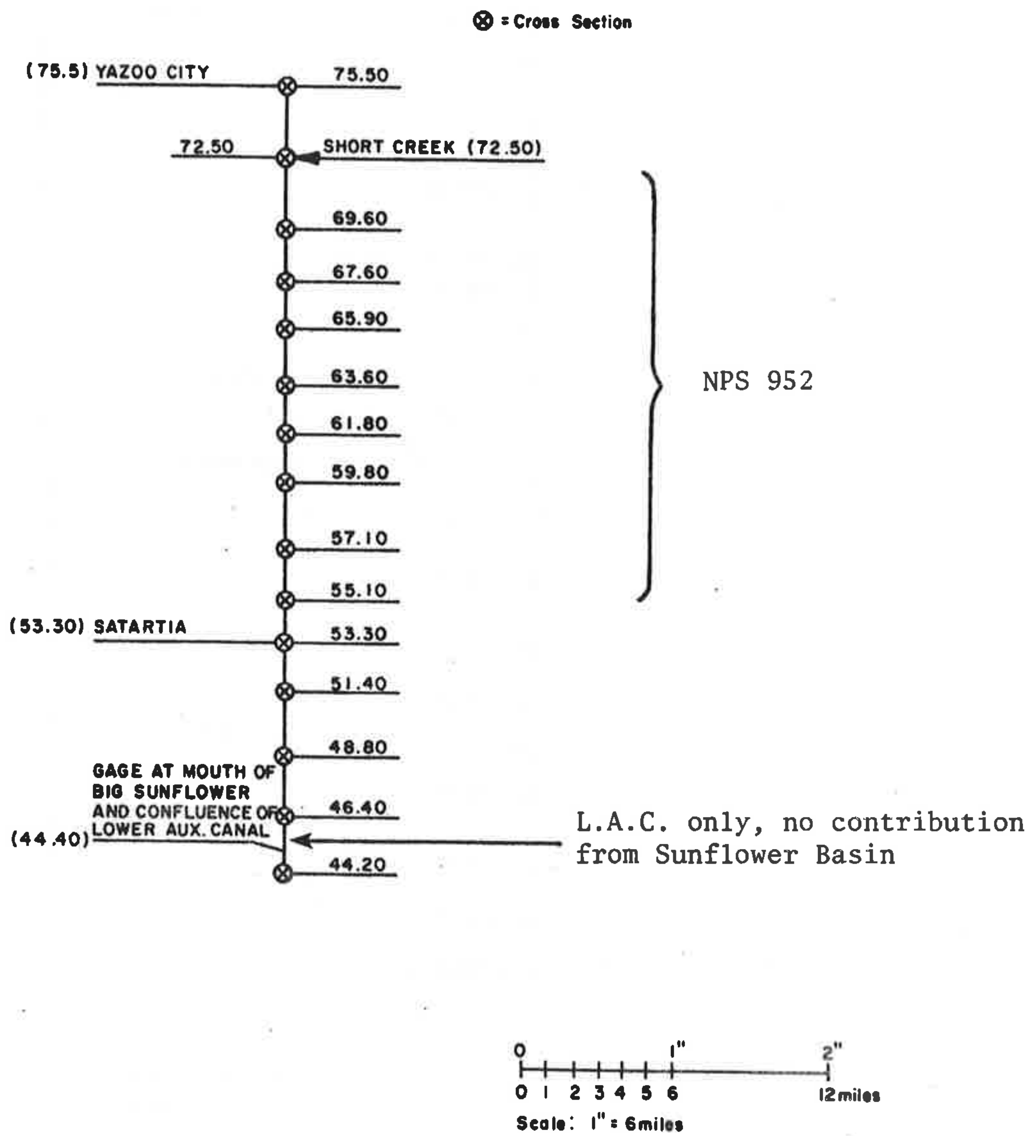


Figure A-1. Main stem--Lower Auxiliary Canal to Yazoo City.

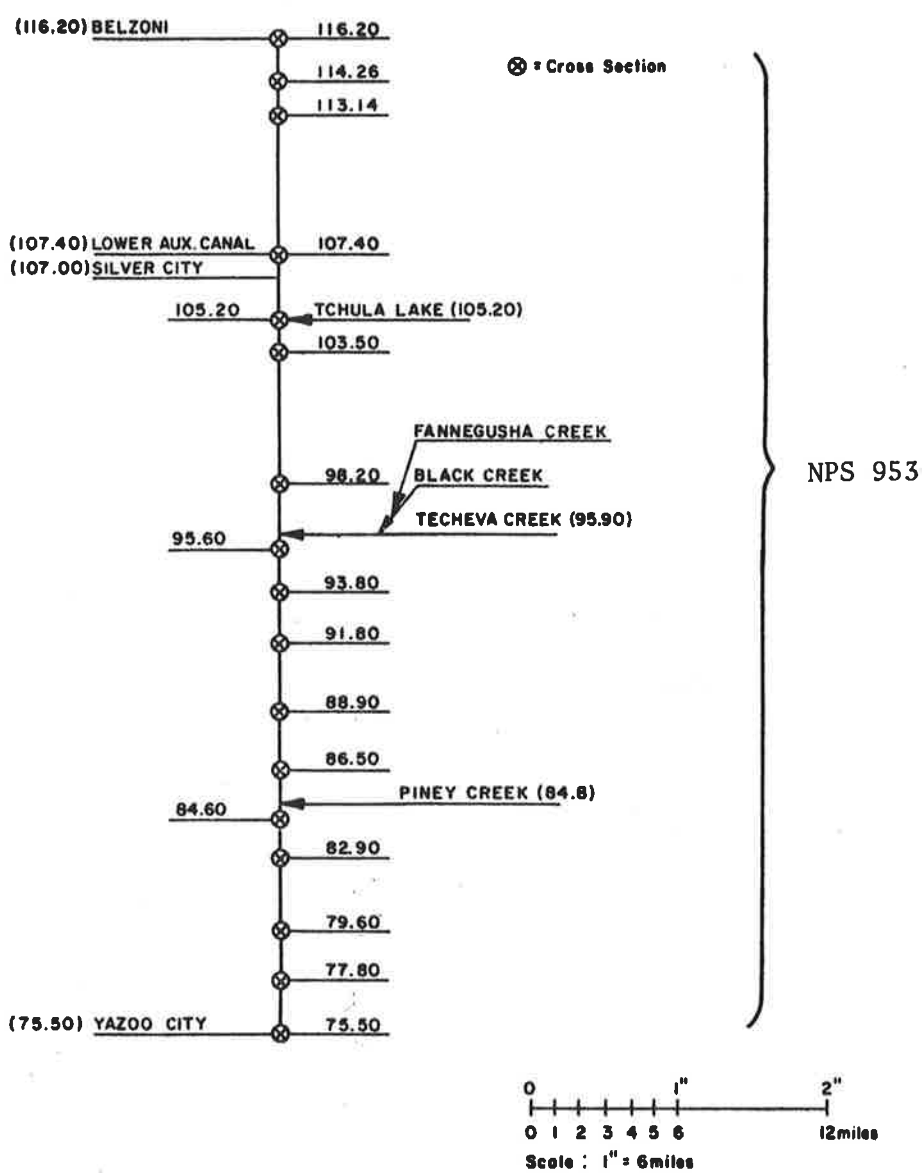


Figure A-2. Main stem--Yazoo City to Belzoni.

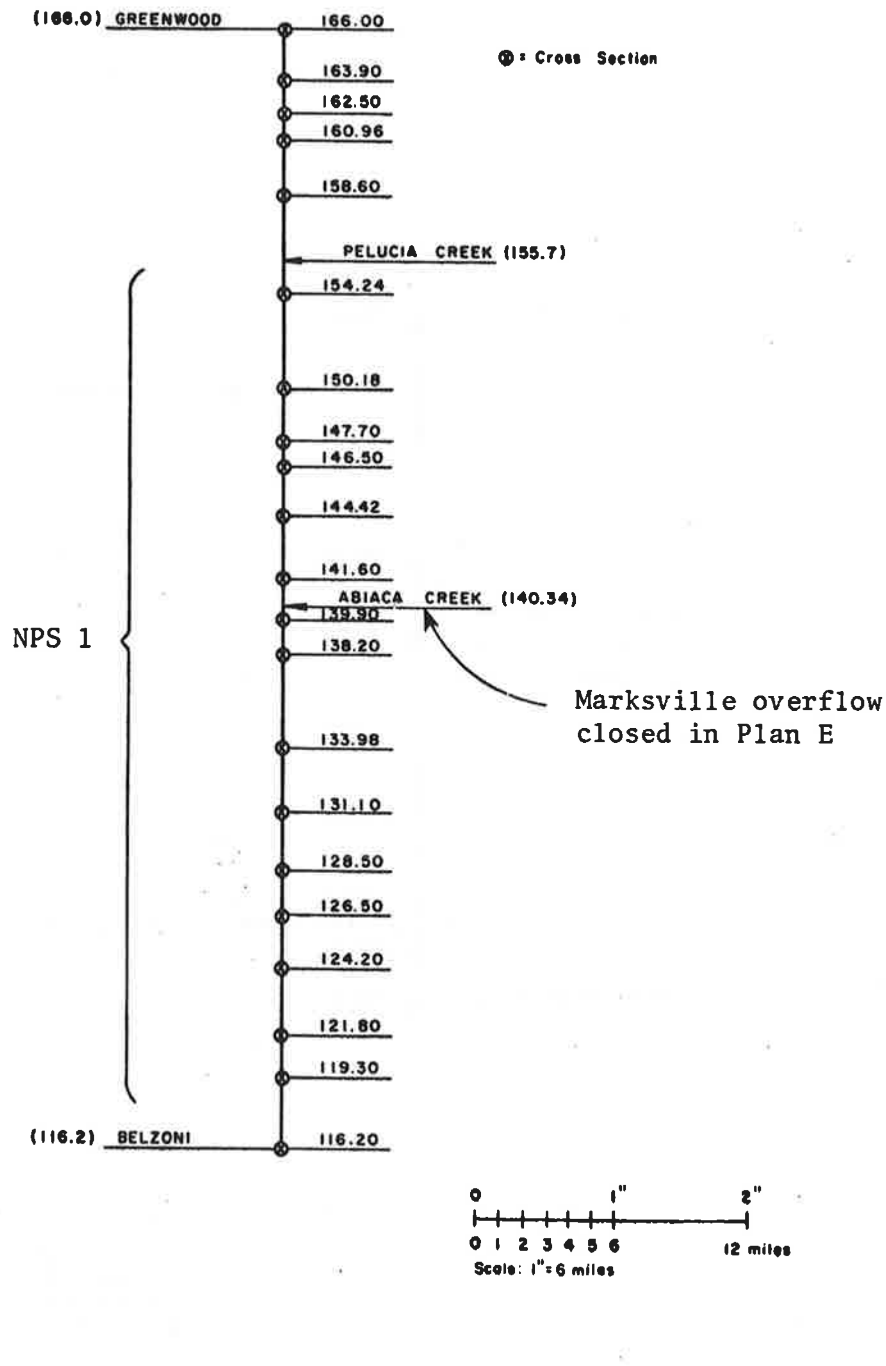


Figure A-3. Main stem--Belzoni to Greenwood.

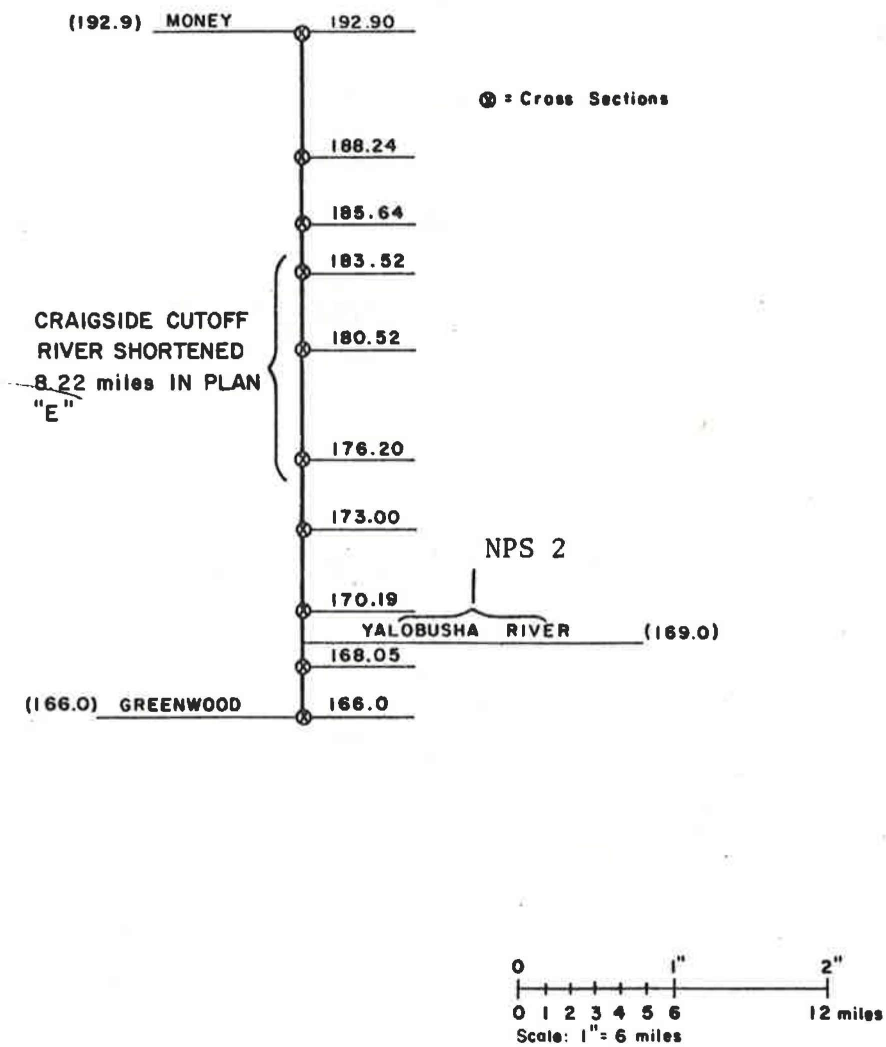


Figure A-4. Main stem--Greenwood to Money.

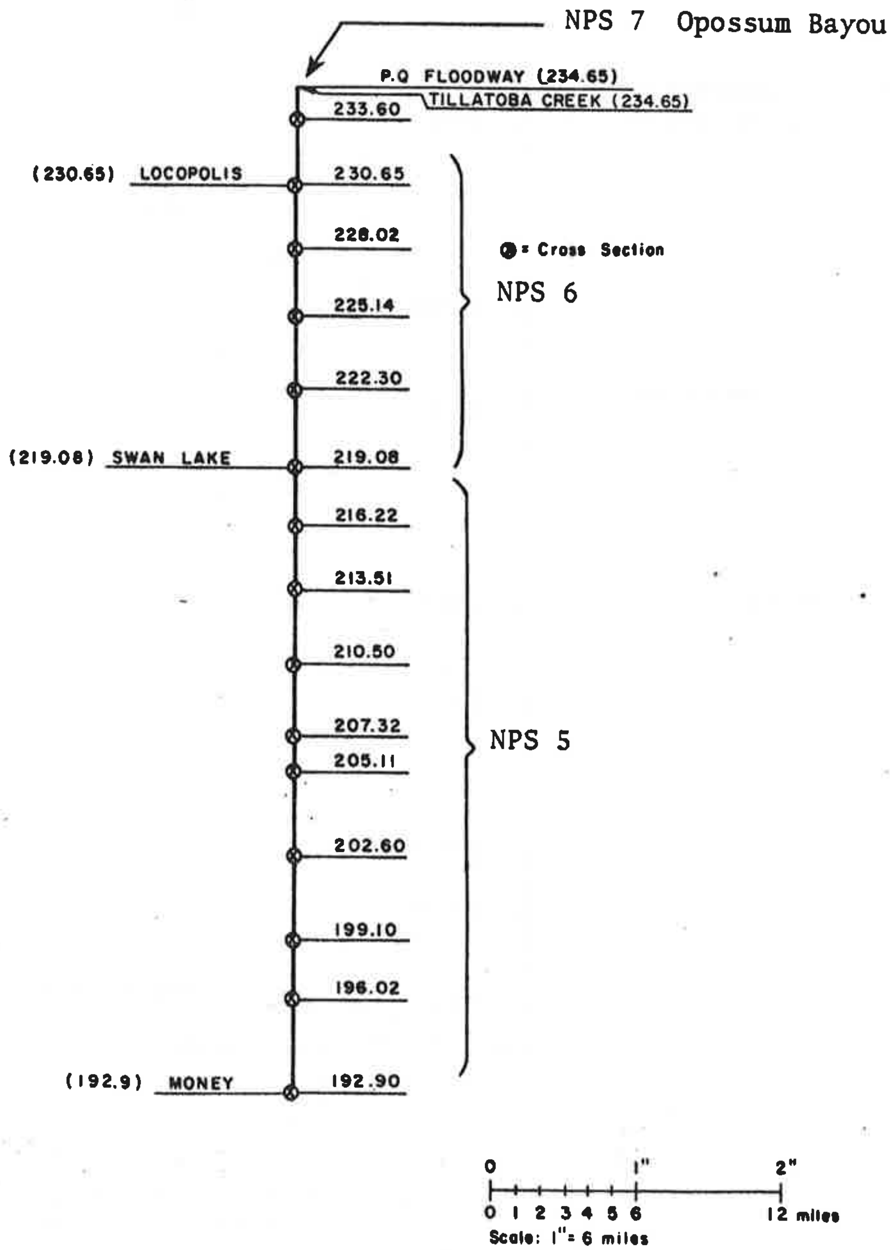


Figure A-5. Main stem--Money to P-Q Floodway.

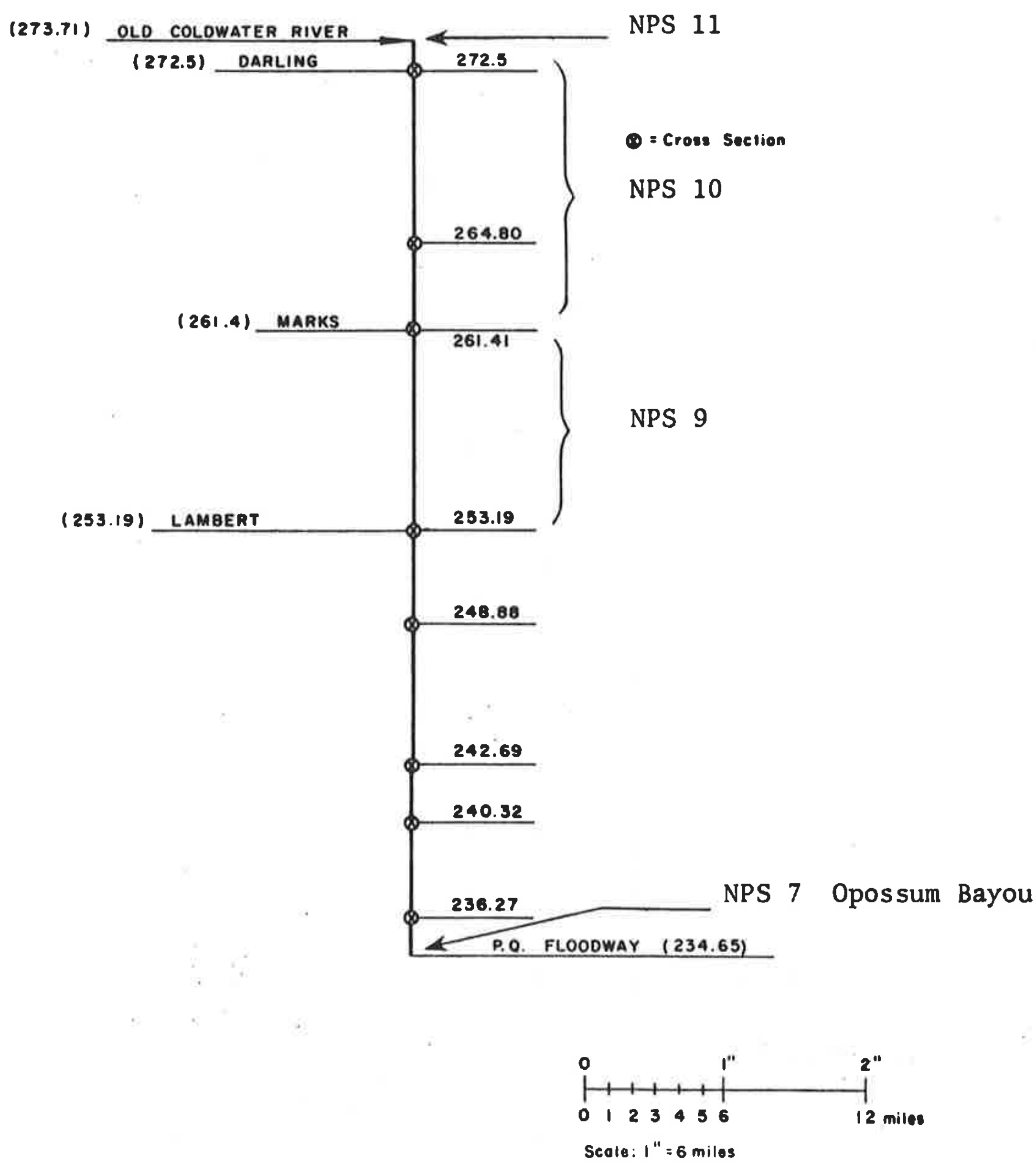


Figure A-6. Main stem--P-Q Floodway to Old Coldwater River.

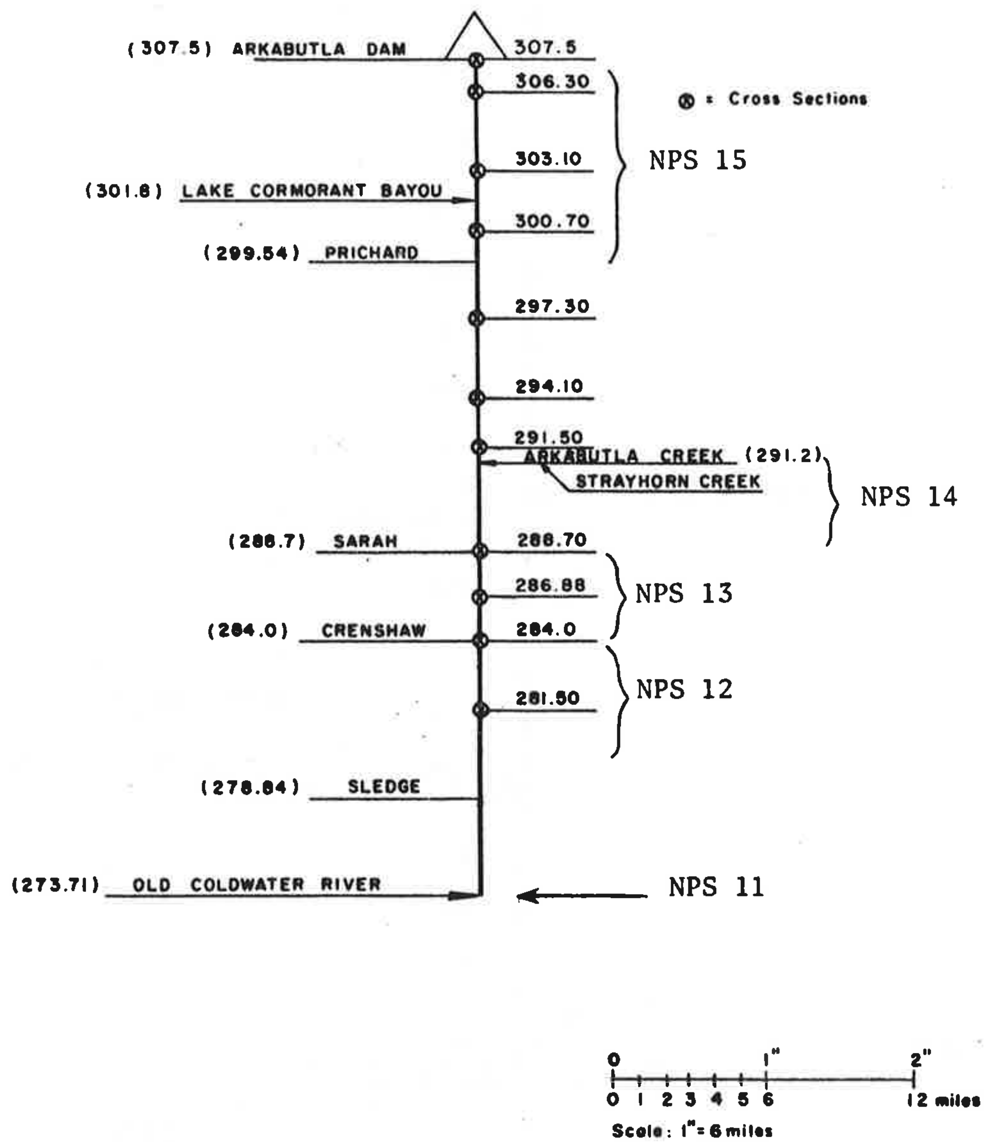


Figure A-7. Main stem--Old Coldwater River to Arkabutla Dam.

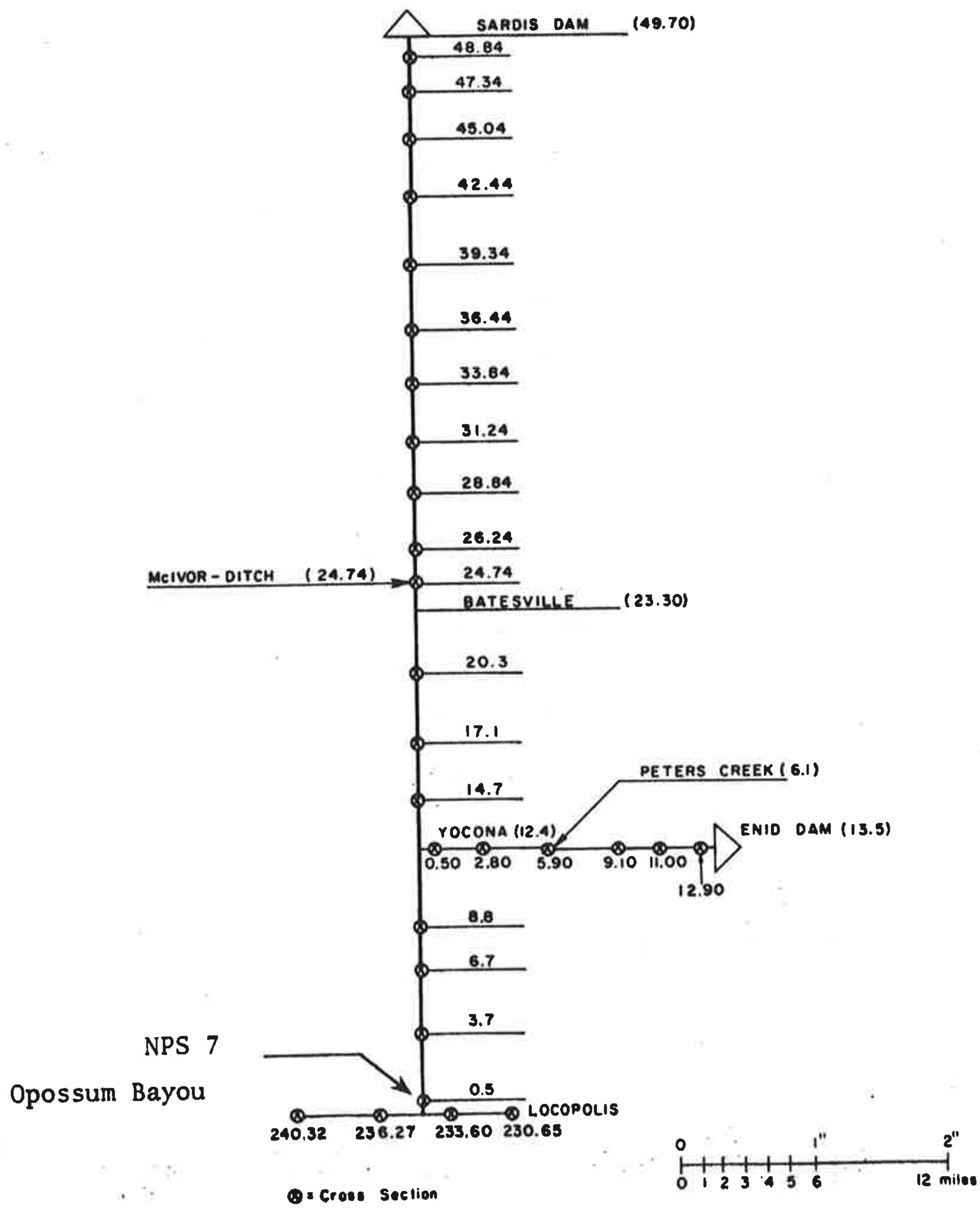


Figure A-8. P-Q-Little Tallahatchie River Confluence to Sardis Dam--
Yocona River Confluence to Enid Dam.

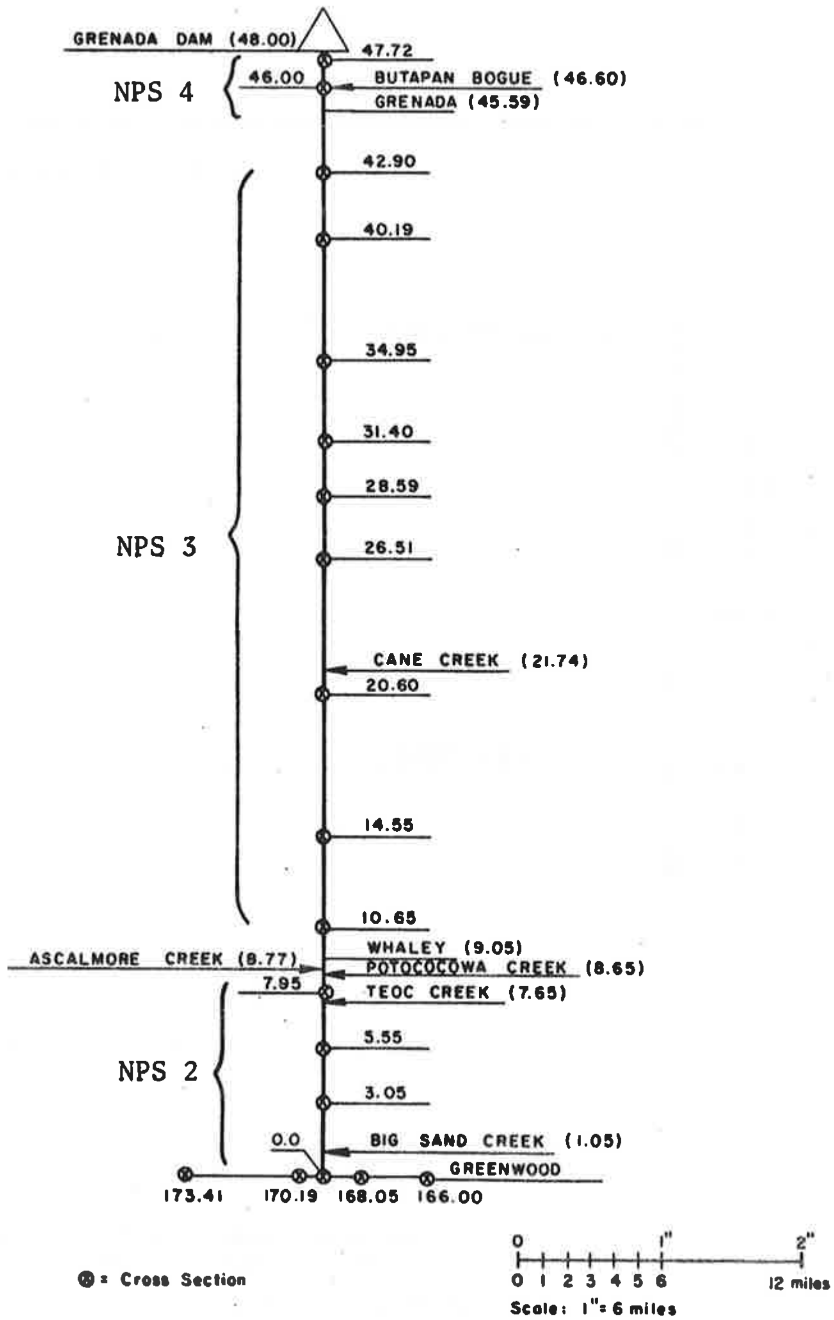


Figure A-9. Yalobusha River Confluence to Grenada Dam.

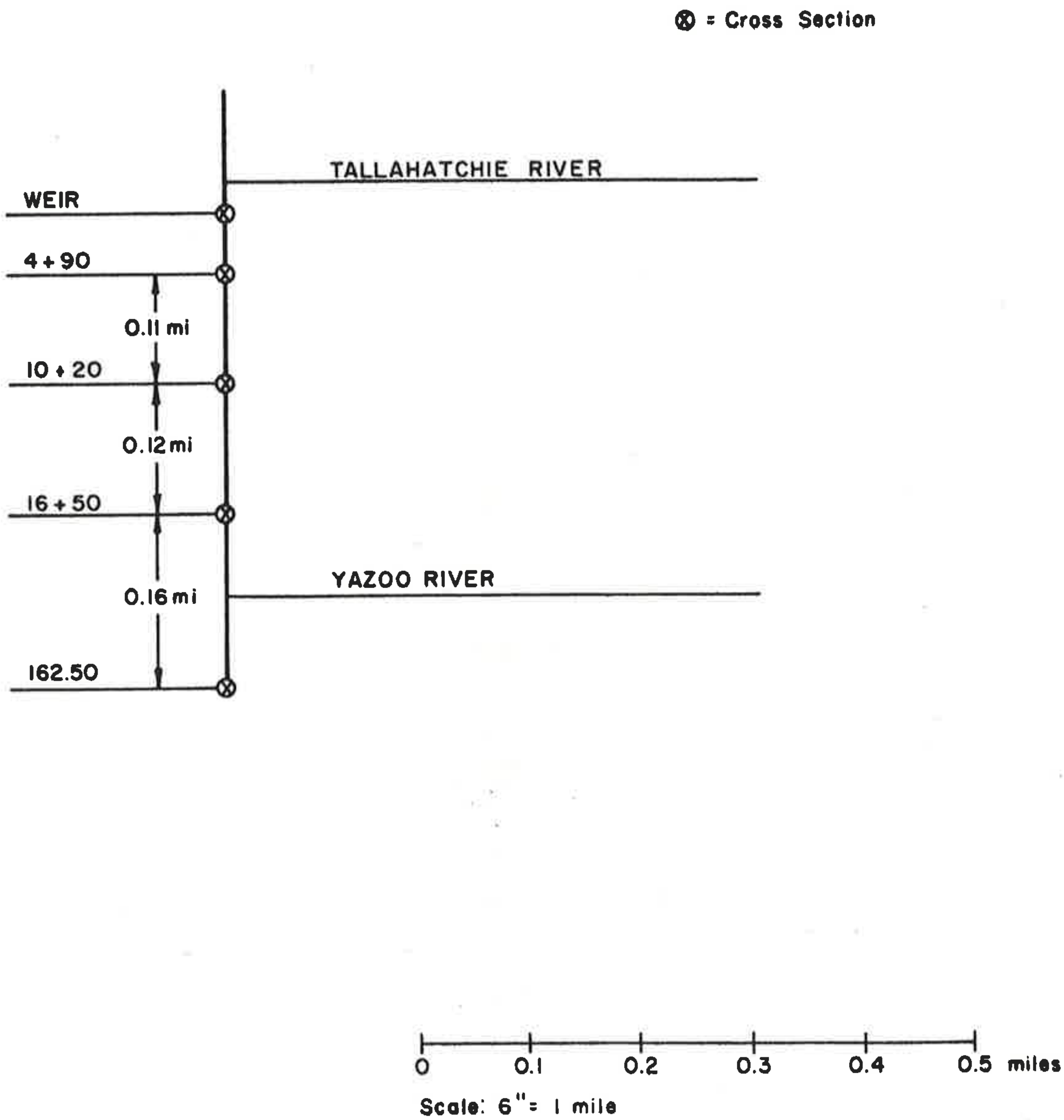


Figure A-10. Greenwood Cutoff.

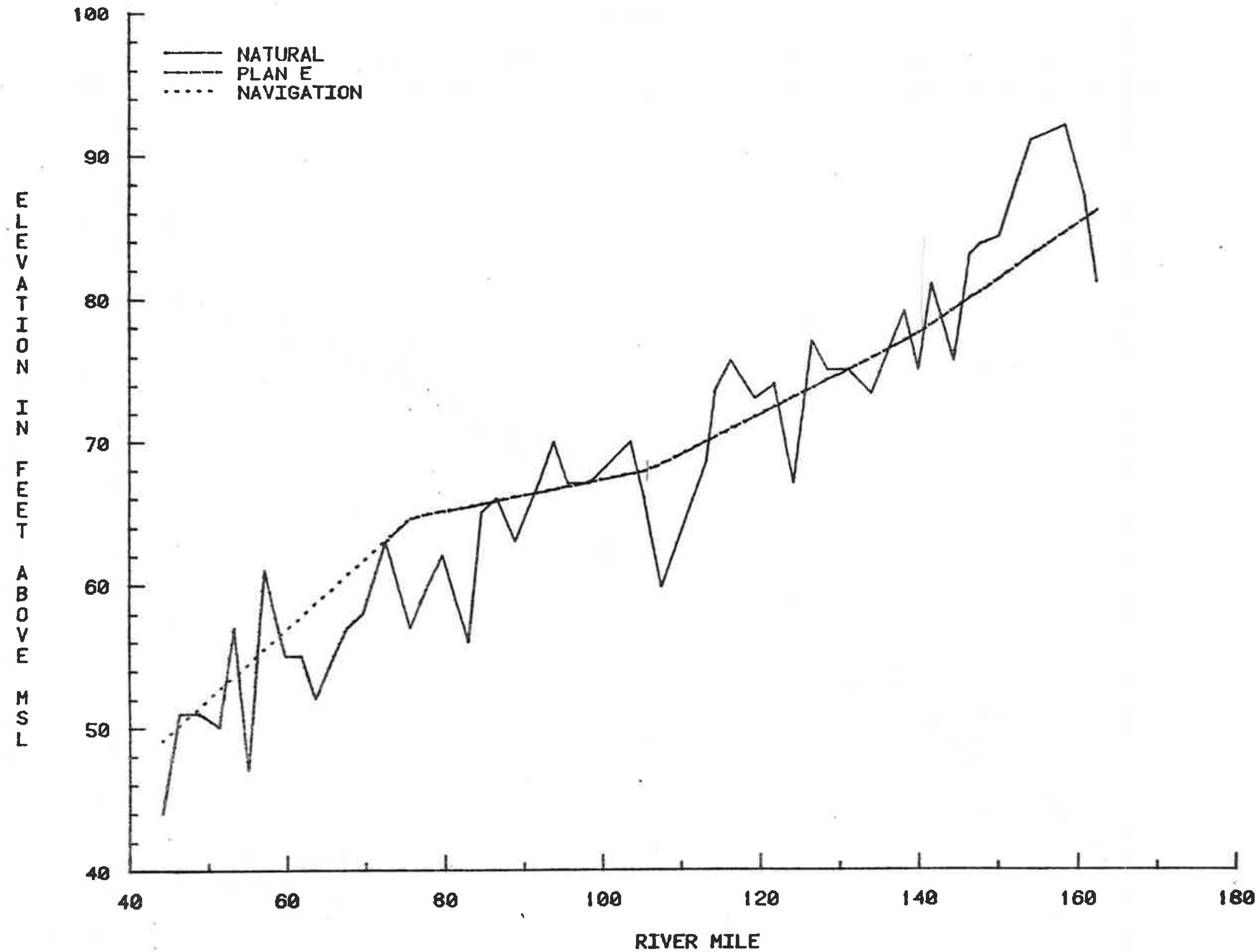


Figure A-11. Bed profiles for river miles 44.0 to 166.0.

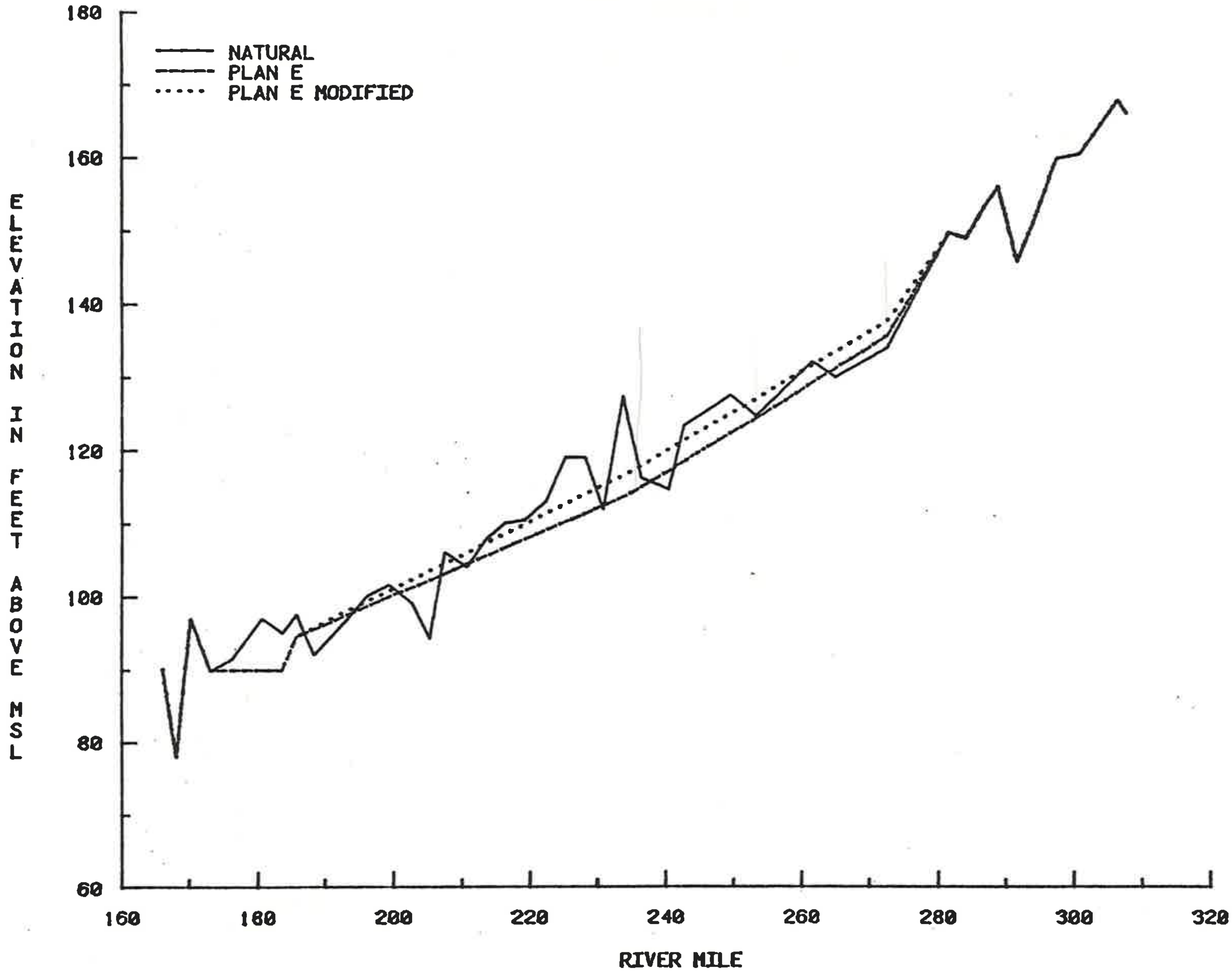


Figure A-12. Bed profiles for River miles 166.0 to 307.0.

APPENDIX B

This appendix contains plots of the 50-year weekly hydrographs at Greenwood, Swan Lake, and Lambert. These hydrographs were used in the known discharge model and represent 11 years of measured data and 39 years of generated data.

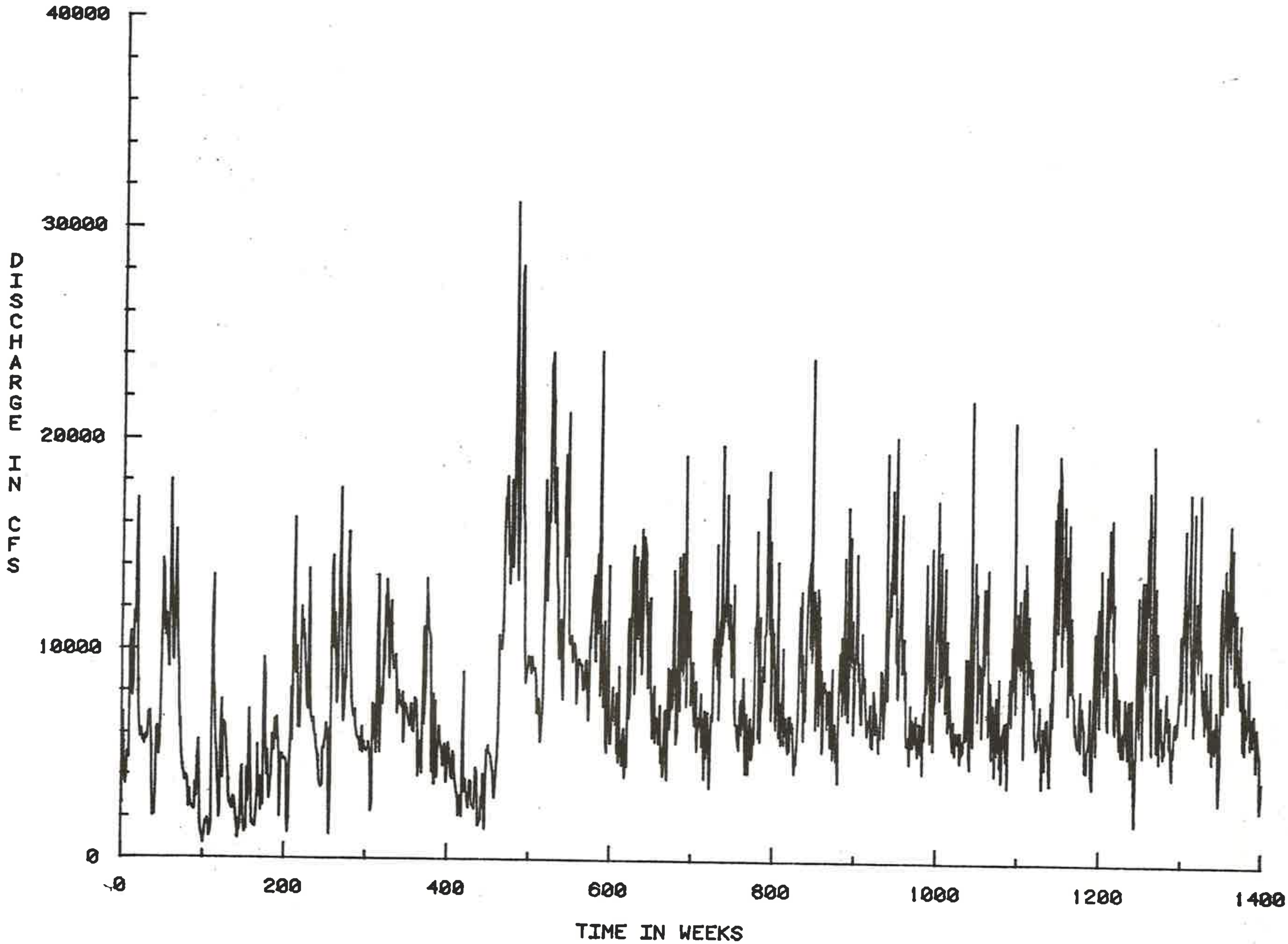


Figure B-1. 50-year weekly hydrograph at Greenwood.

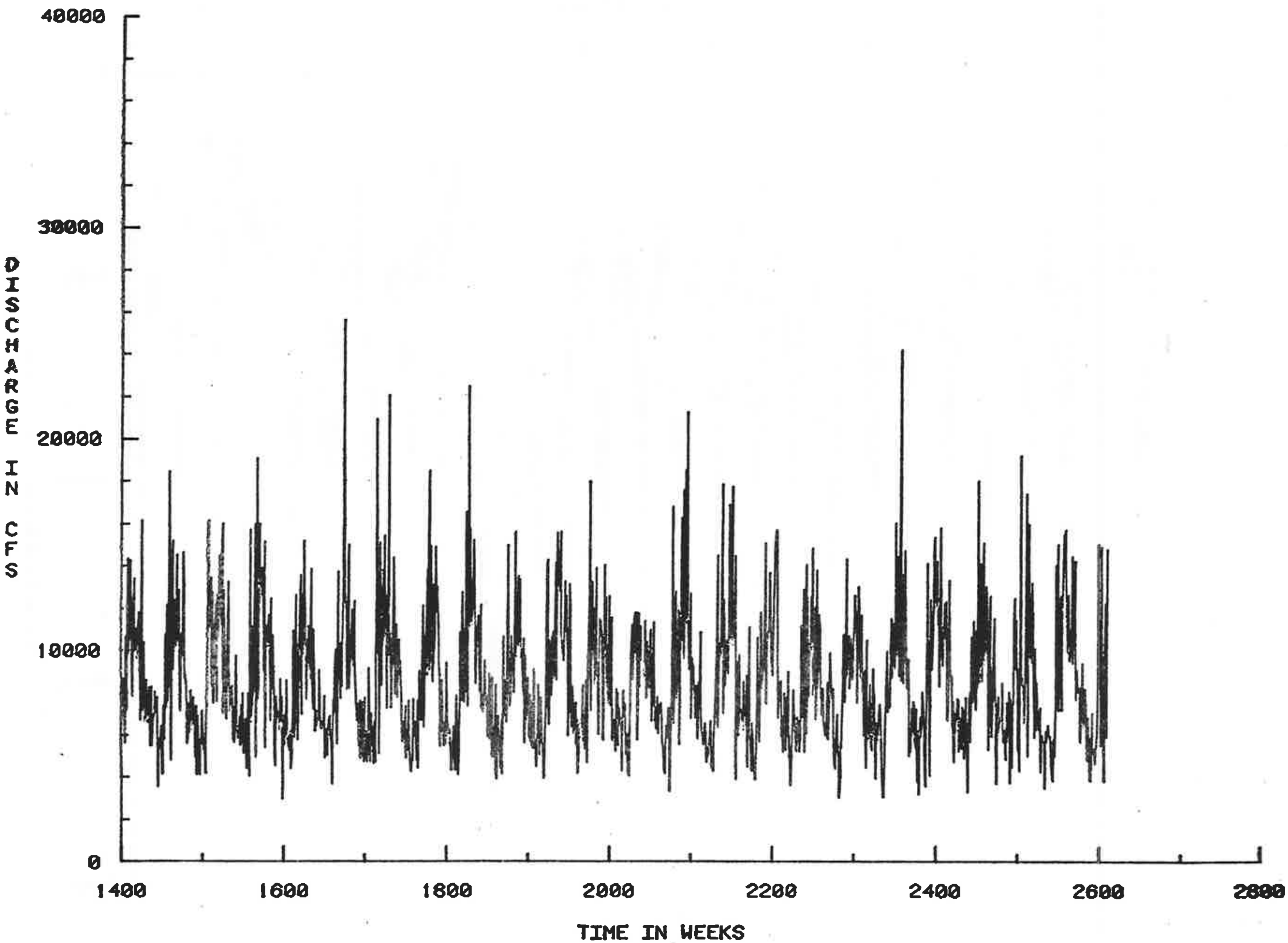


Figure B-1. (continued)

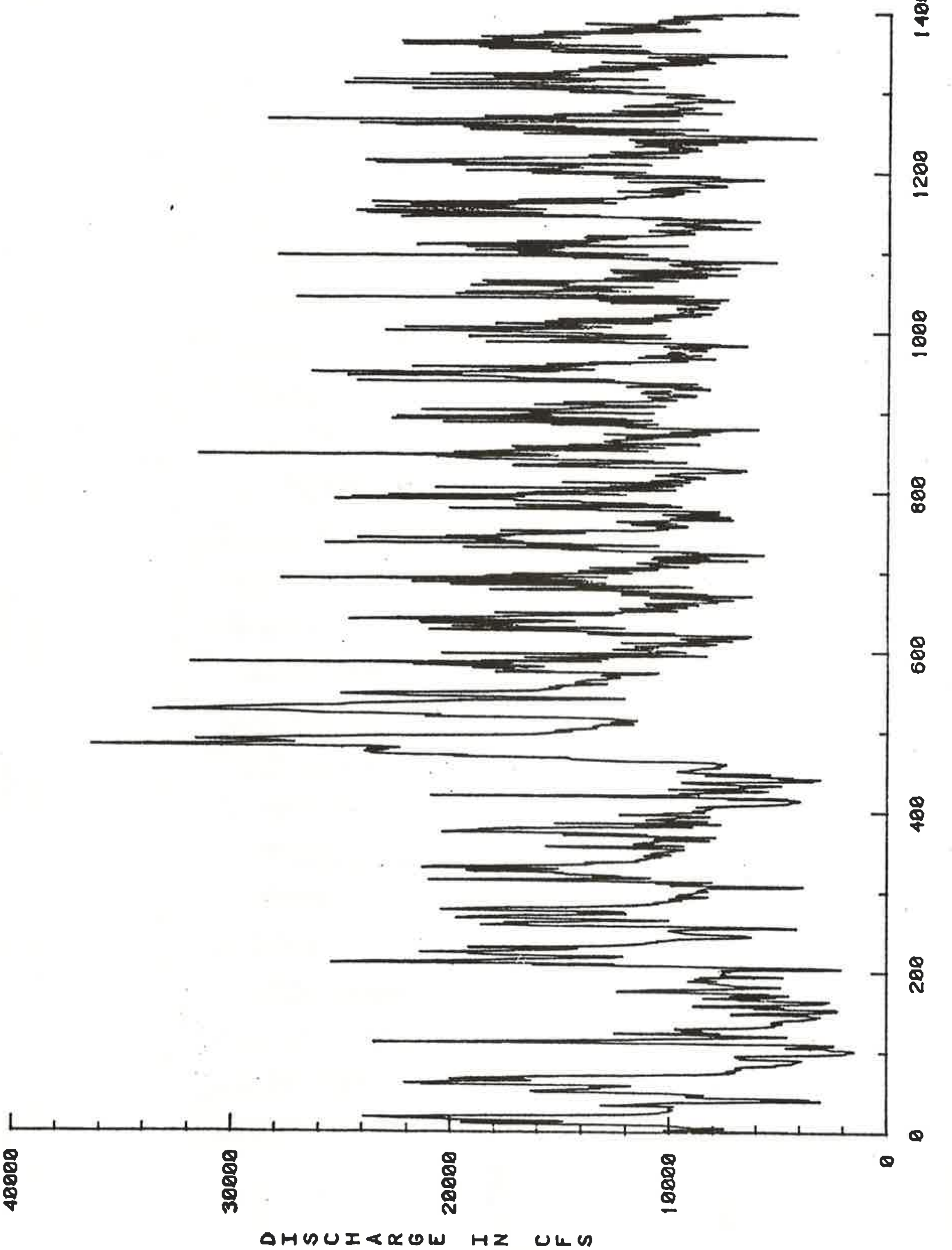


Figure B-2. 50-year weekly hydrograph at Swan Lake.

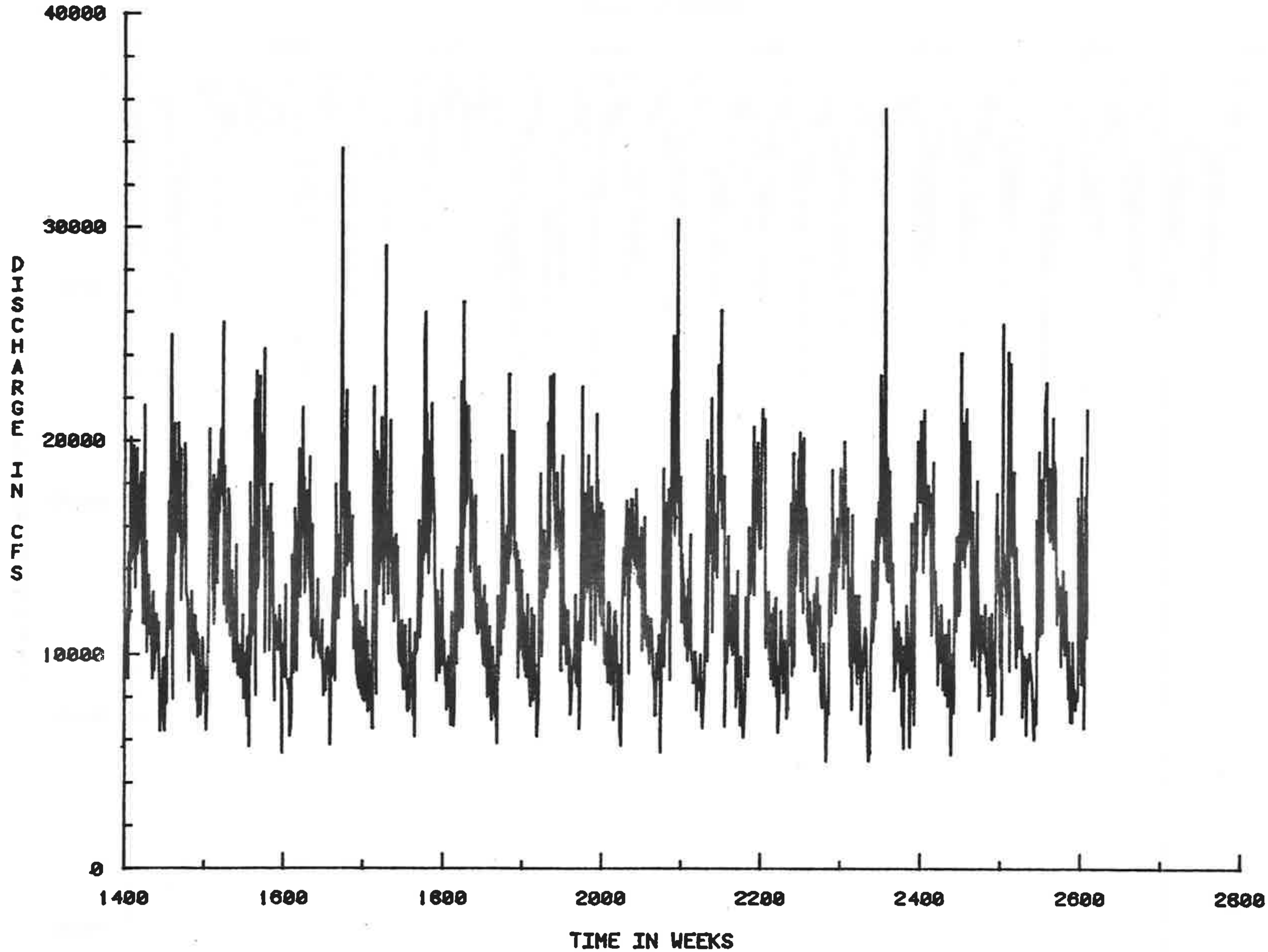


Figure B-2. (continued)

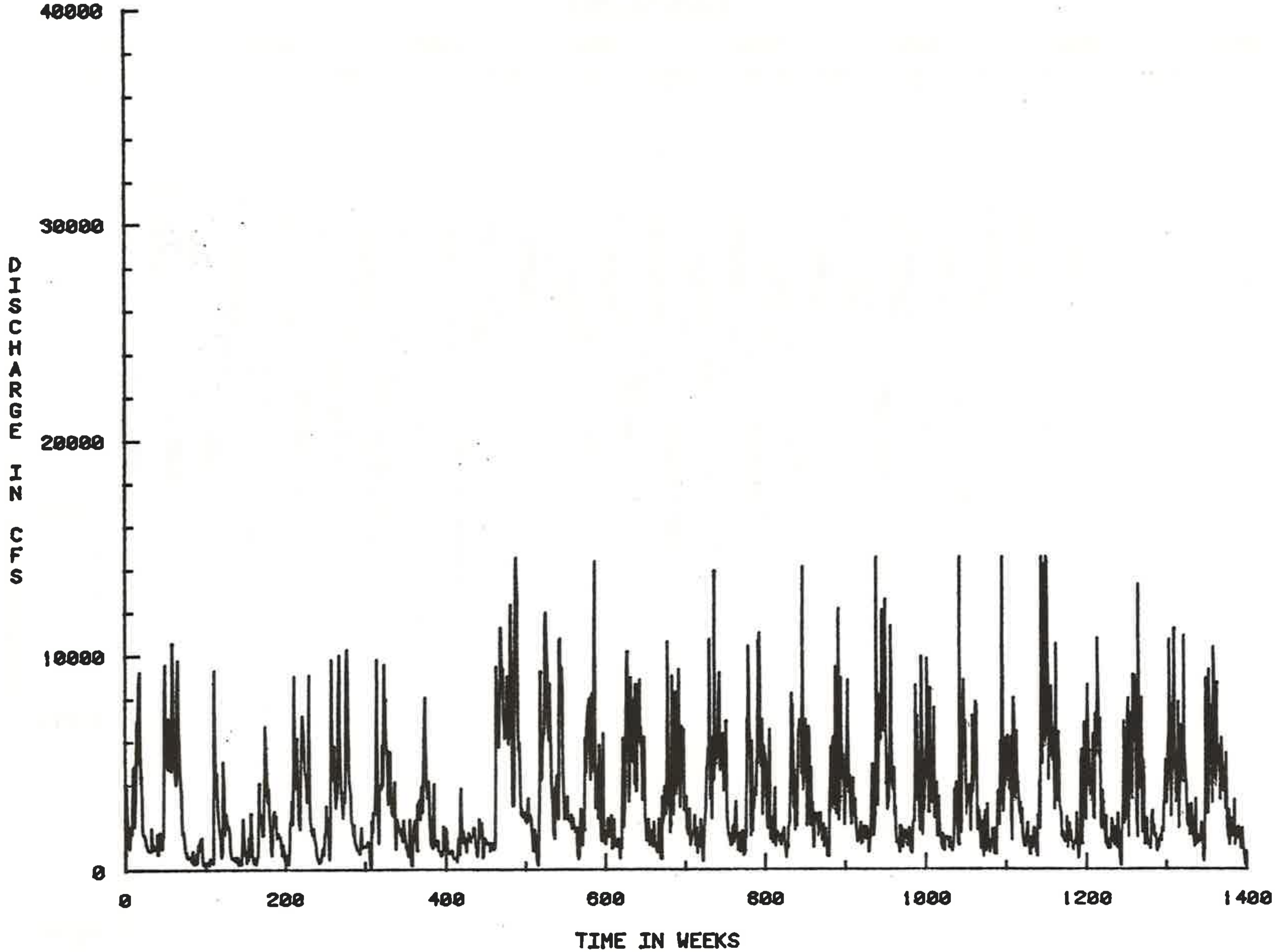


Figure B-3. 50-year weekly hydrograph at Lambert.

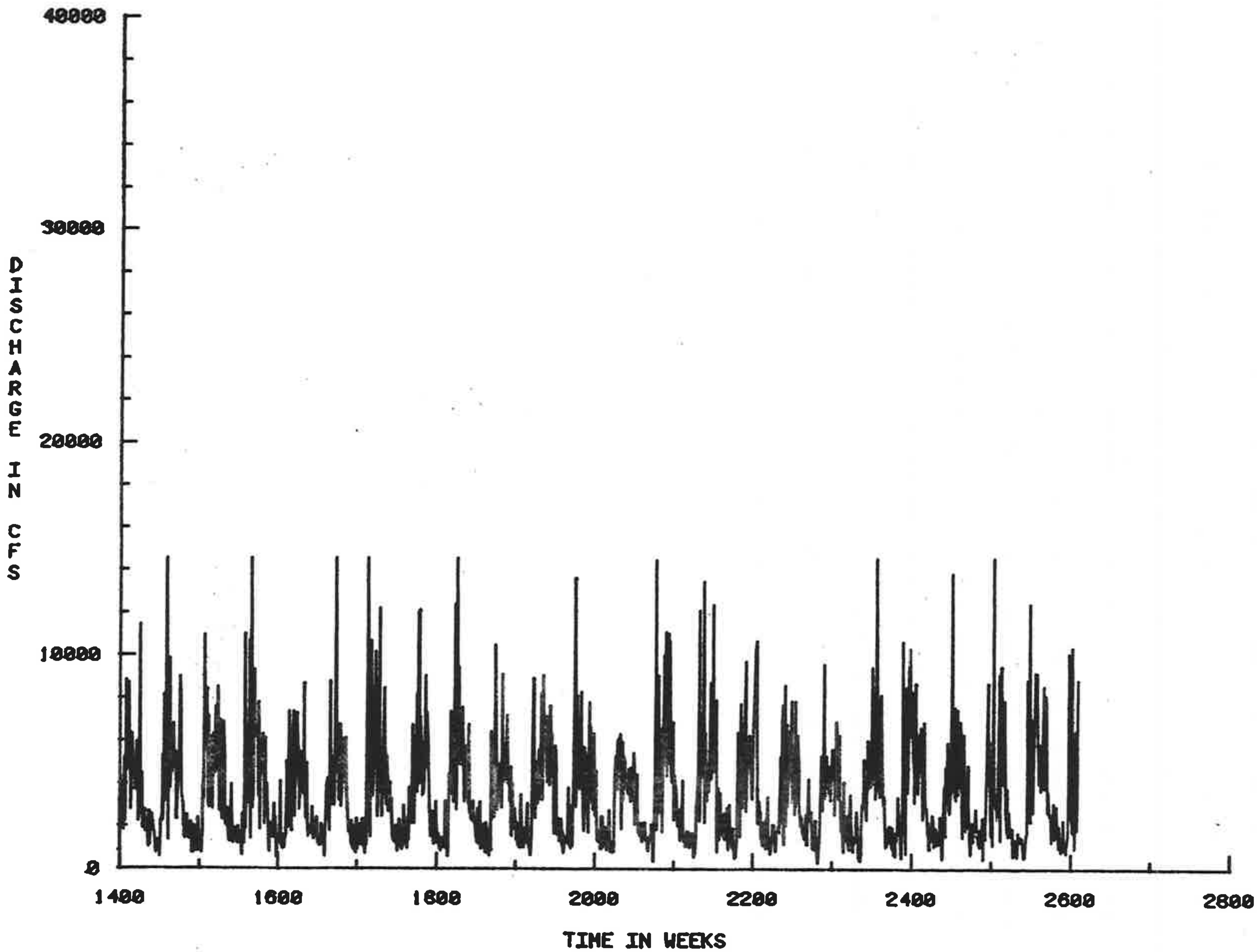


Figure B-3. (continued)

APPENDIX C

This appendix contains stage discharge relationships that were computed by the known discharge sediment routing model. Each curve, composed of 52 points, represents the weekly discharge and stage for the indicated year as computed by the model. Some plots are given which show the change in the stage-discharge relationship with time for a given station and alternative run, while other plots are given which show the difference between alternatives for a given station and year.

The stations selected for the plots were Greenwood, Swan Lake, and Lambert, while the alternative runs selected were Run 1--Natural, Run 2--Plan E, Run 12--Five-year dredging, and Run 14--Best alternative. These stations and runs were selected because of their general importance.

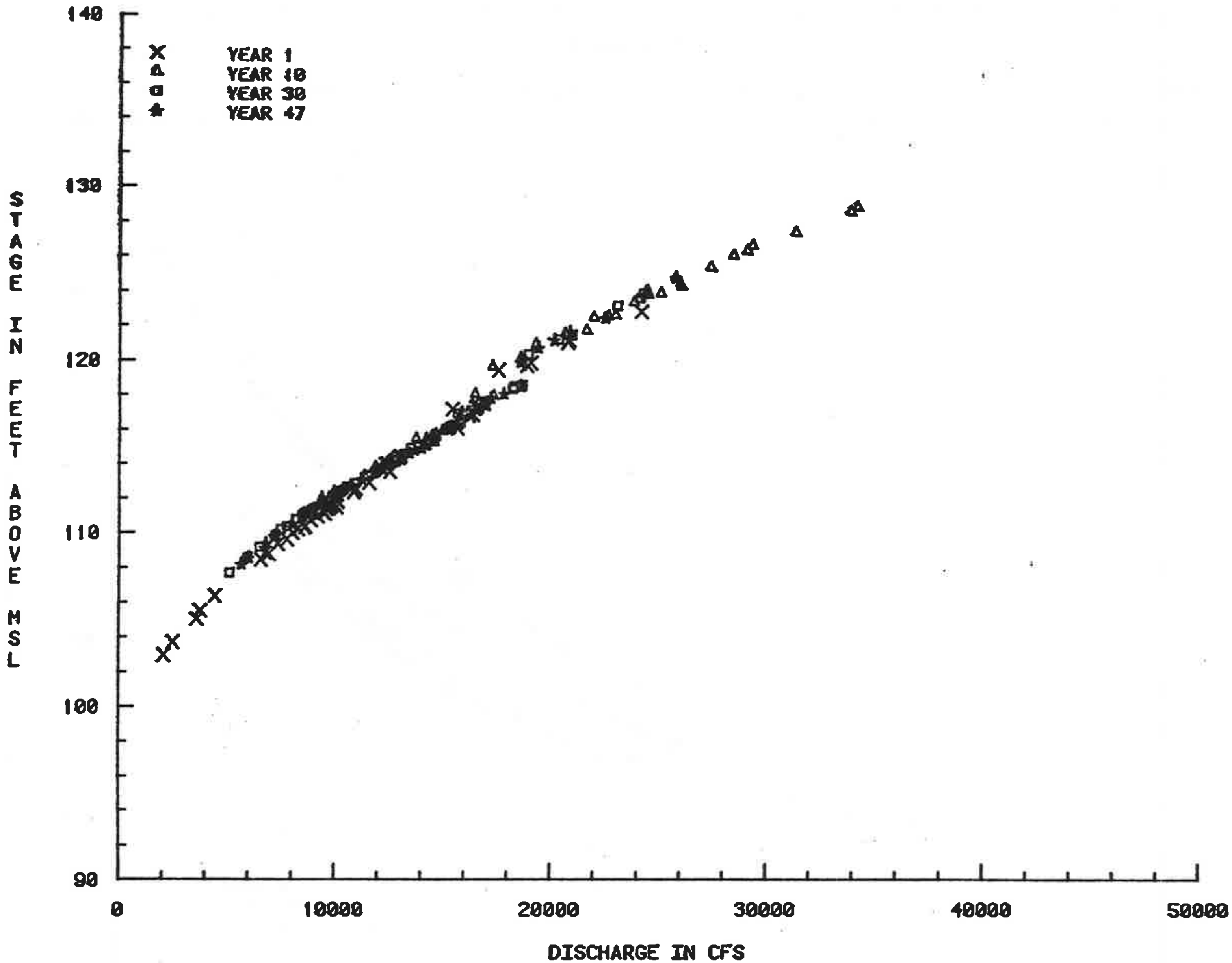


Figure C-1. Stage discharge relationship downstream of Greenwood Bendway (river mile 162.5) for natural conditions.

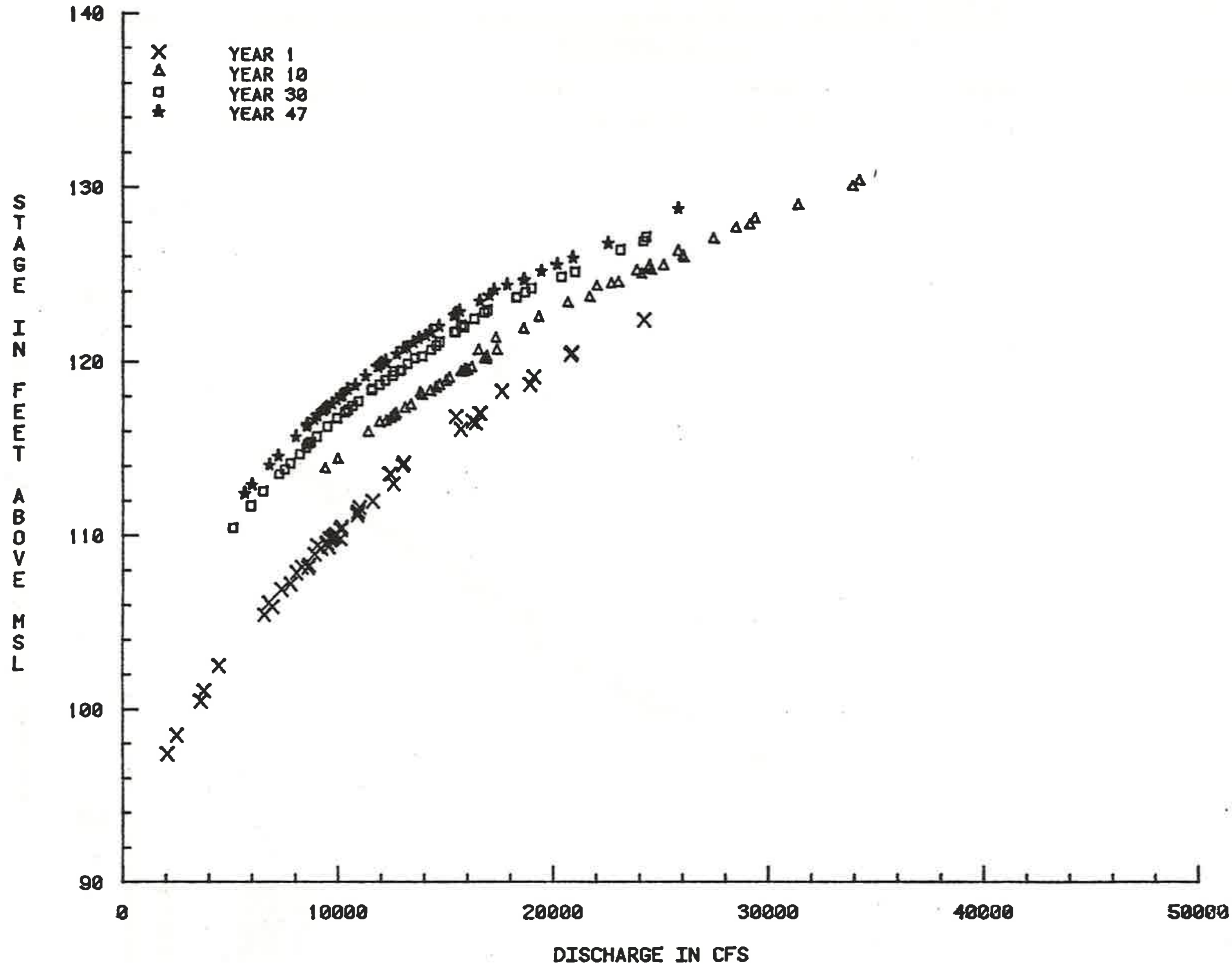


Figure C-2. Stage discharge relationship downstream of Greenwood Bendway (river mile 162.5) for Plan E.

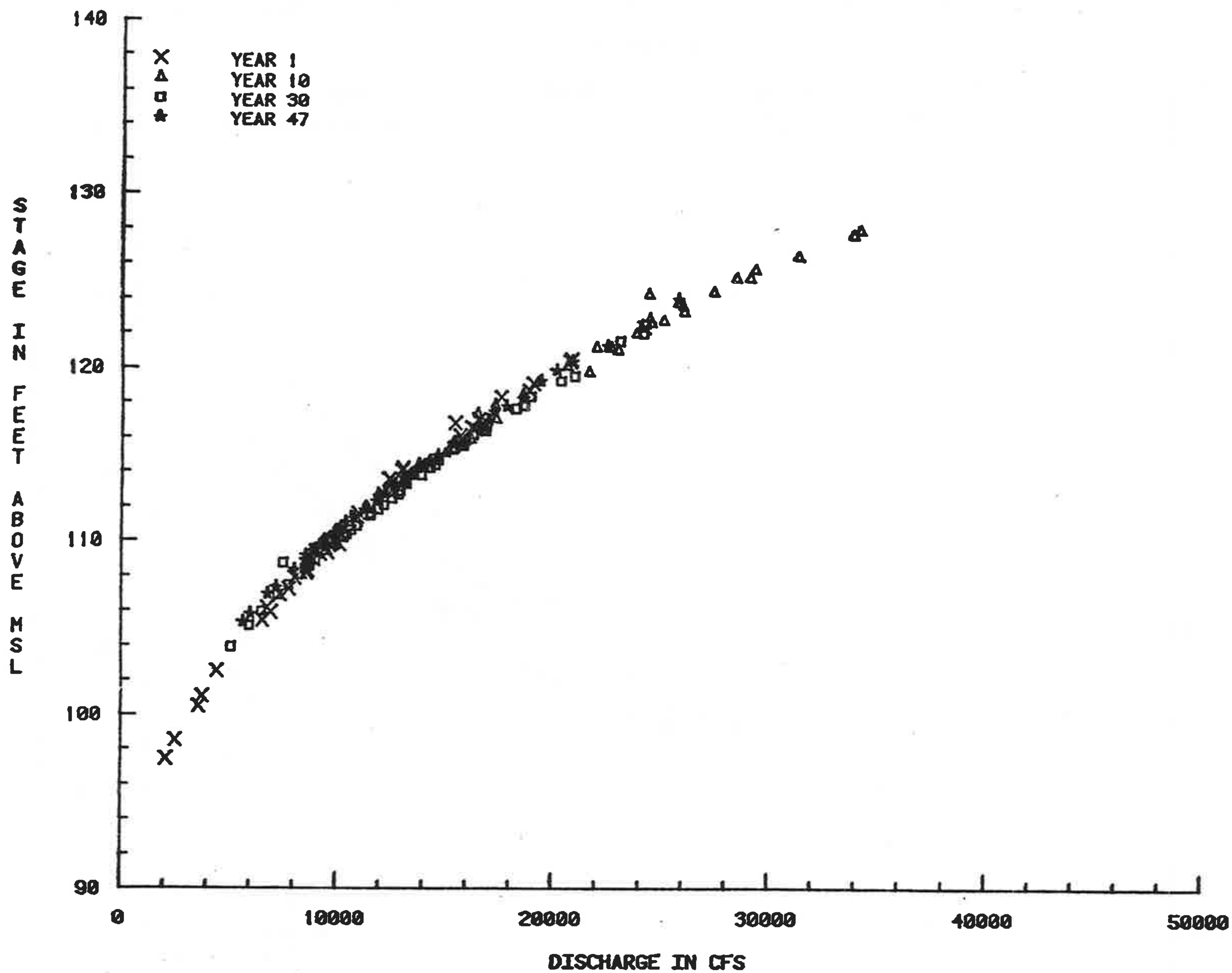


Figure C-3. Stage discharge relationship downstream of Greenwood Bendway (river mile 162.5) for every 5-year dredging.

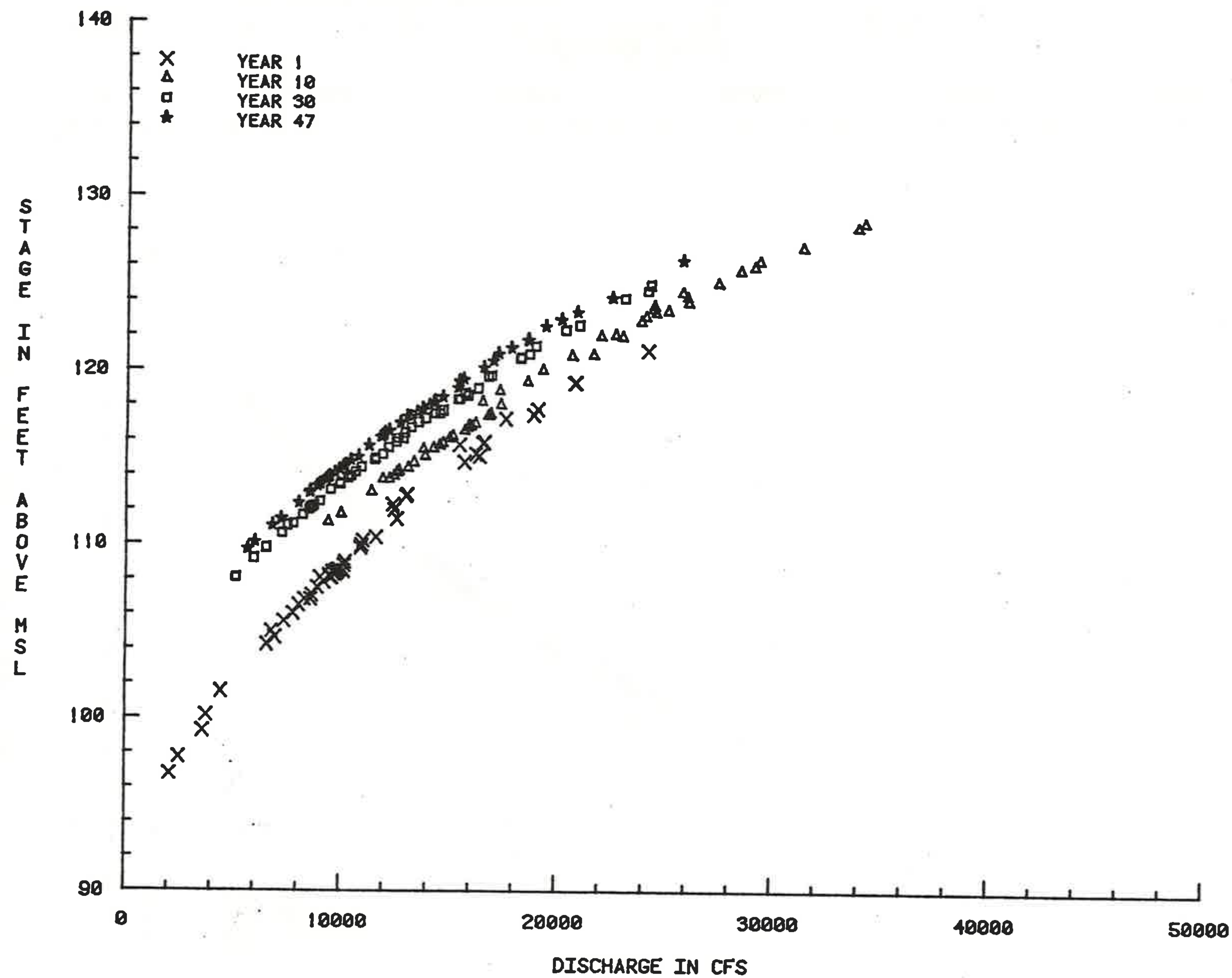


Figure C-4. Stage discharge relationship downstream of Greenwood Bendway (river mile 162.5) for the best alternative.

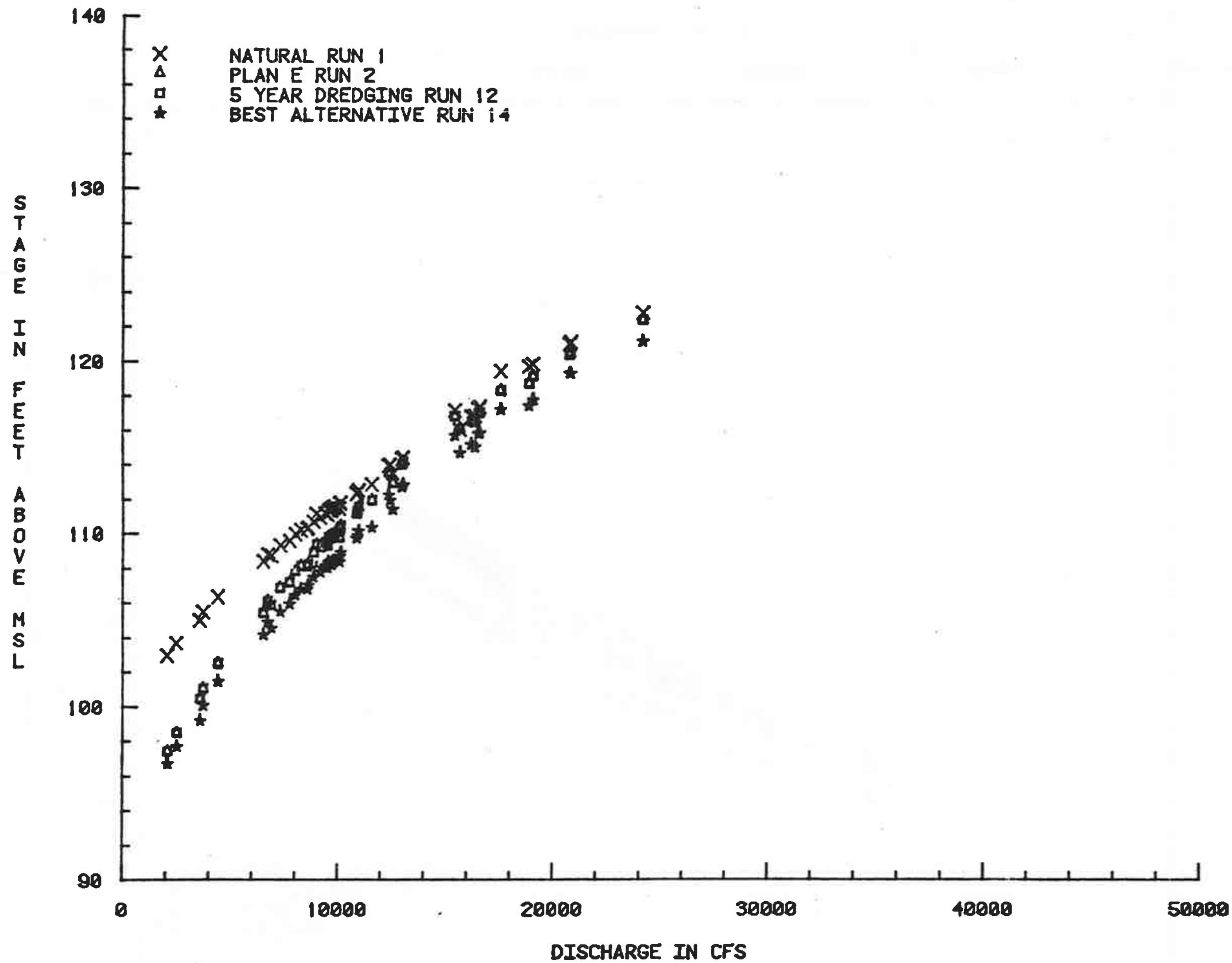


Figure C-5. Stage discharge relationship downstream of Greenwood Bendway (river mile 162.5) for Year 1.

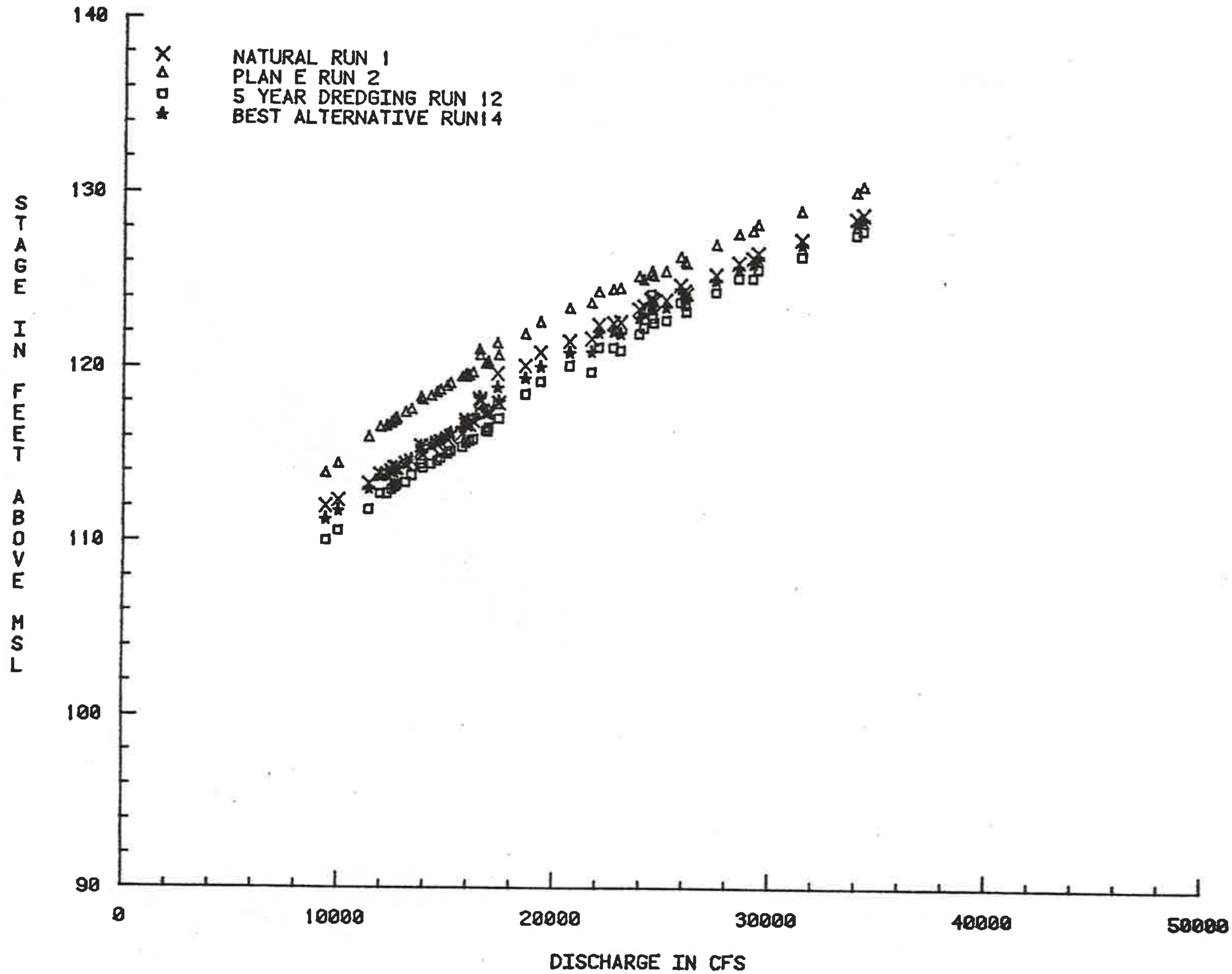


Figure C-6. Stage discharge relationship downstream of Greenwood Bendway (river mile 162.5) for Year 10.

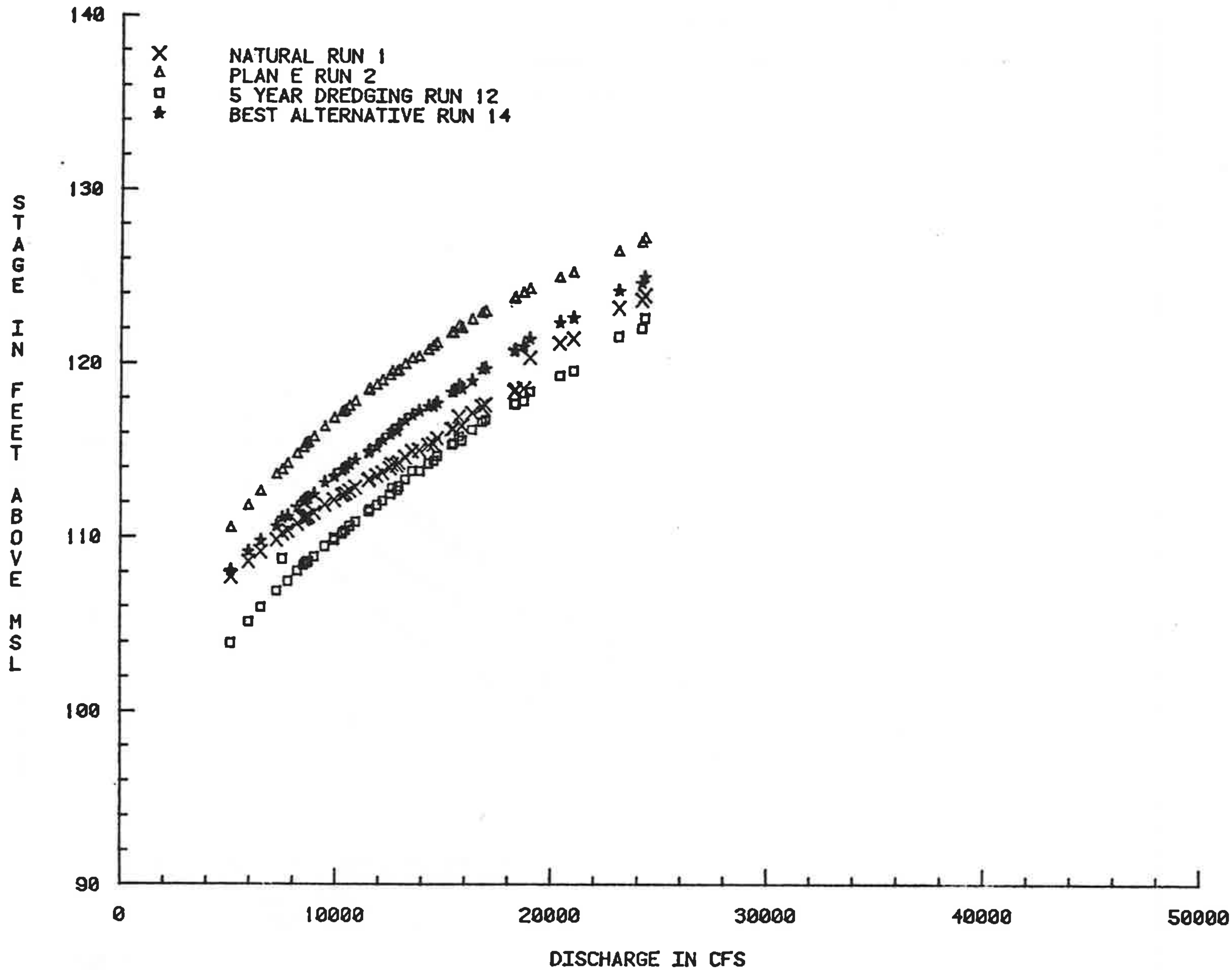


Figure C-7. Stage discharge relationship downstream of Greenwood Bendway (river mile 162.5) for Year 30.

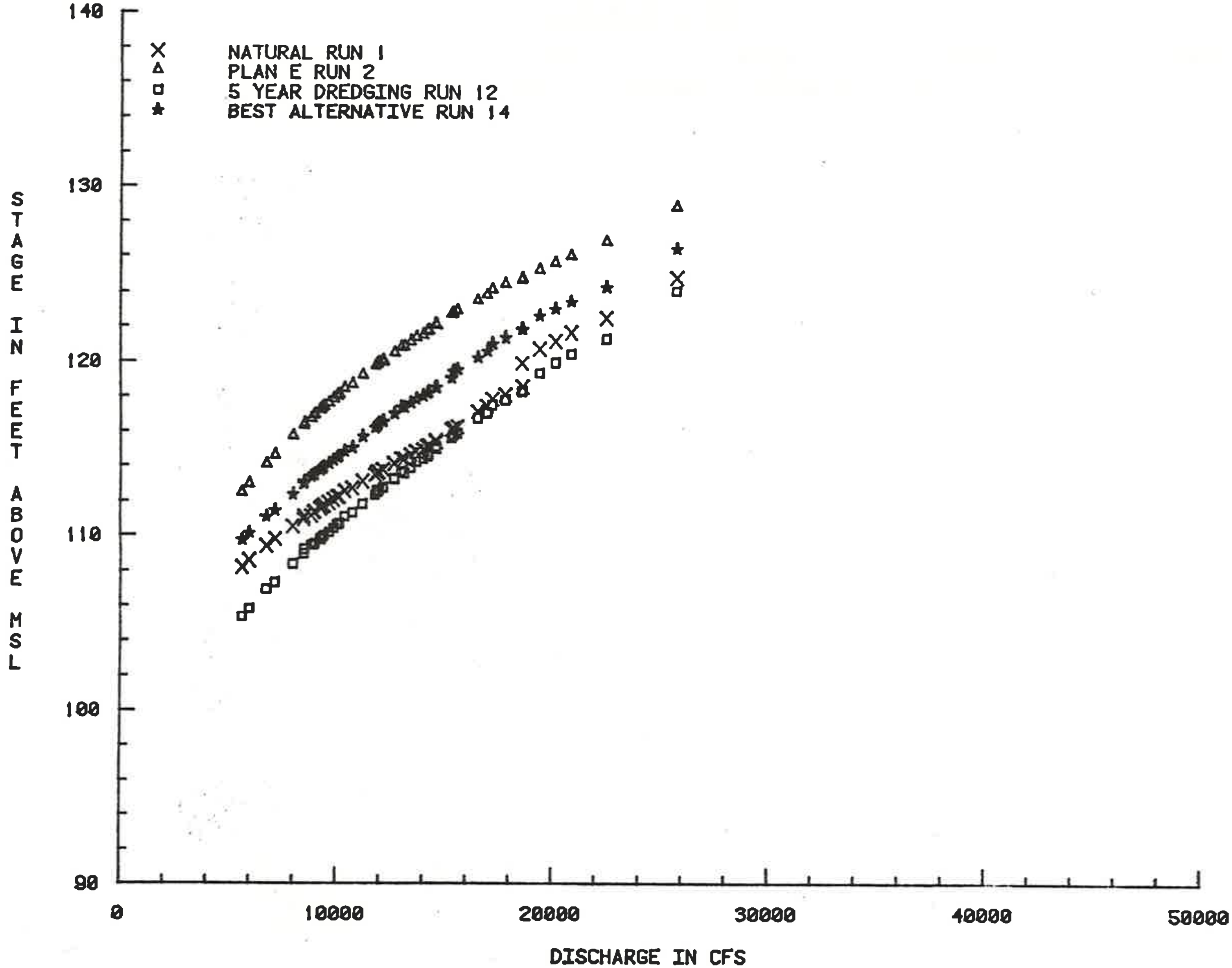


Figure C-8. Stage discharge relationship downstream of Greenwood Bendway (river mile 162.5) for Year 47.

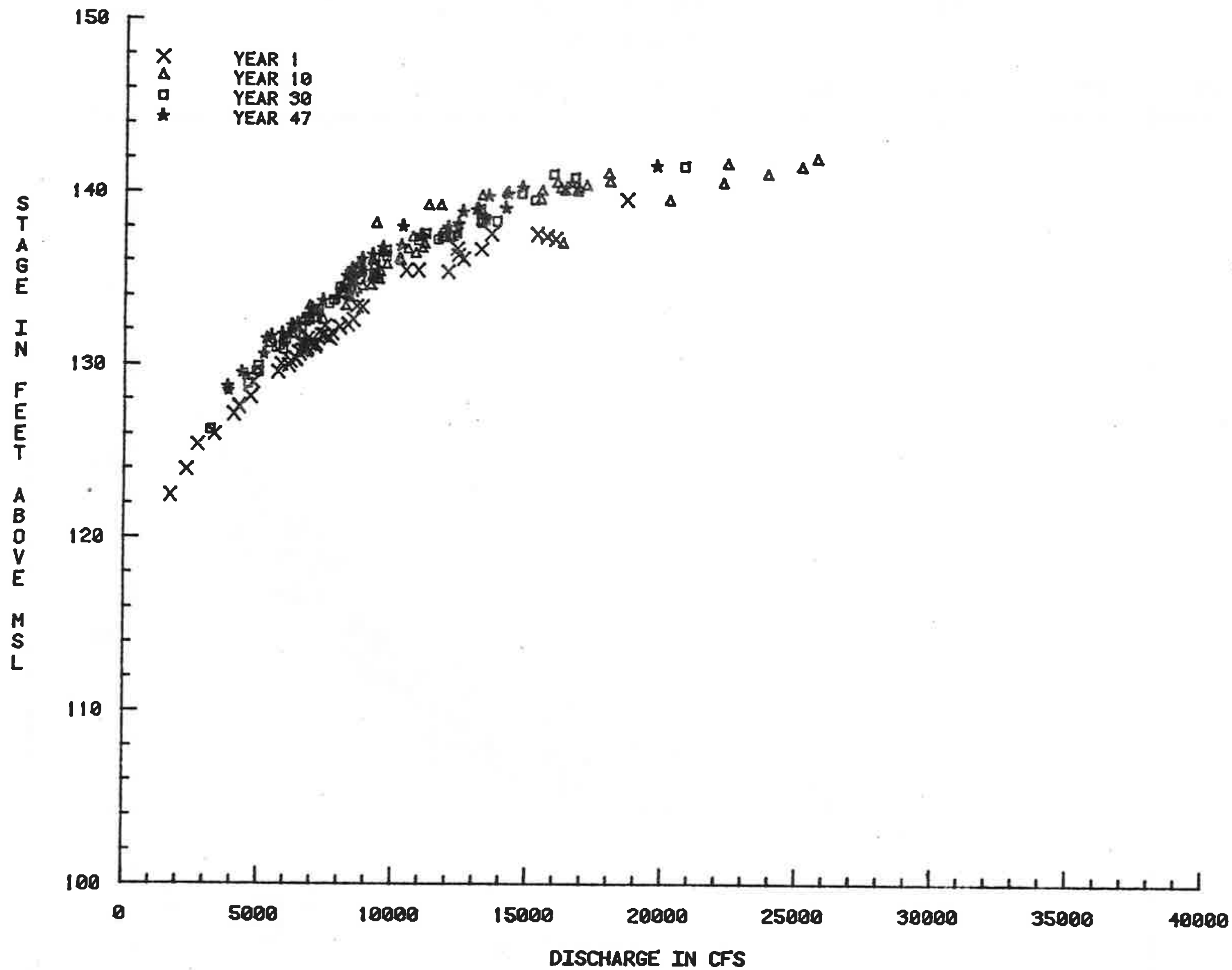


Figure C-9. Stage discharge relationship at Swan Lake (river mile 219.08) for natural conditions.

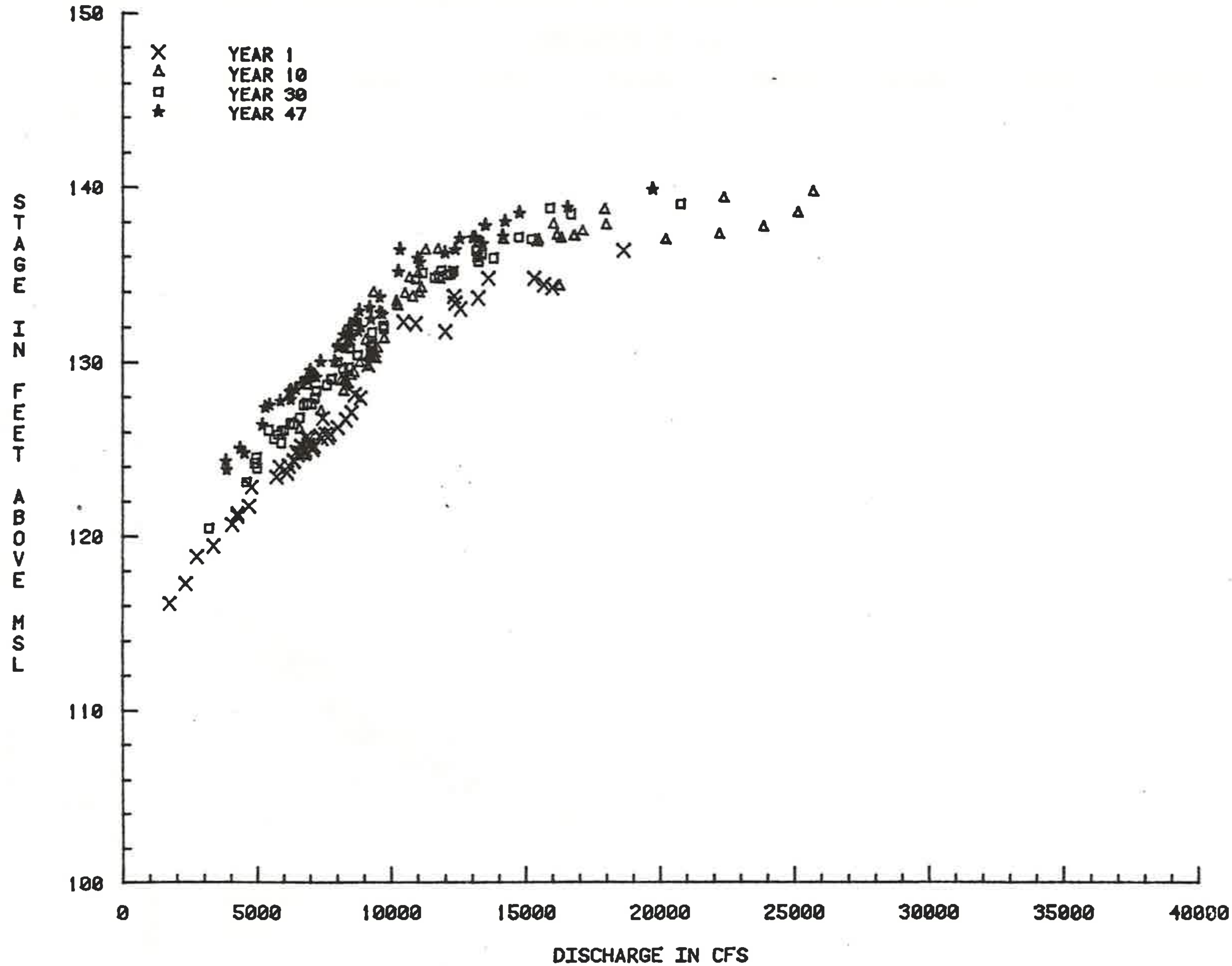


Figure C-10. Stage discharge relationship at Swan Lake (river mile 219.08) for Plan E.

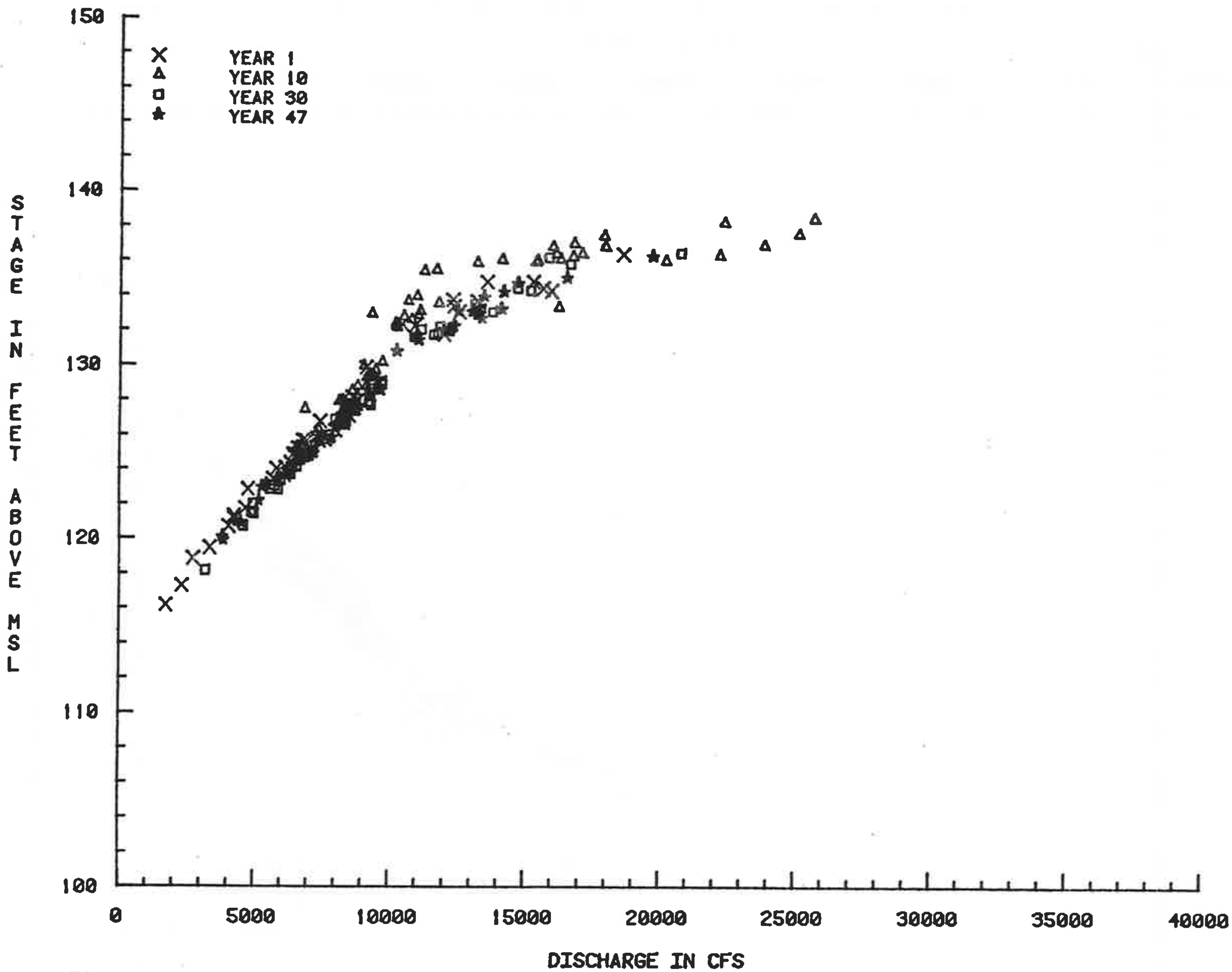


Figure C-11. Stage discharge relationship at Swan Lake (river mile 219.08) for every 5-year dredging.

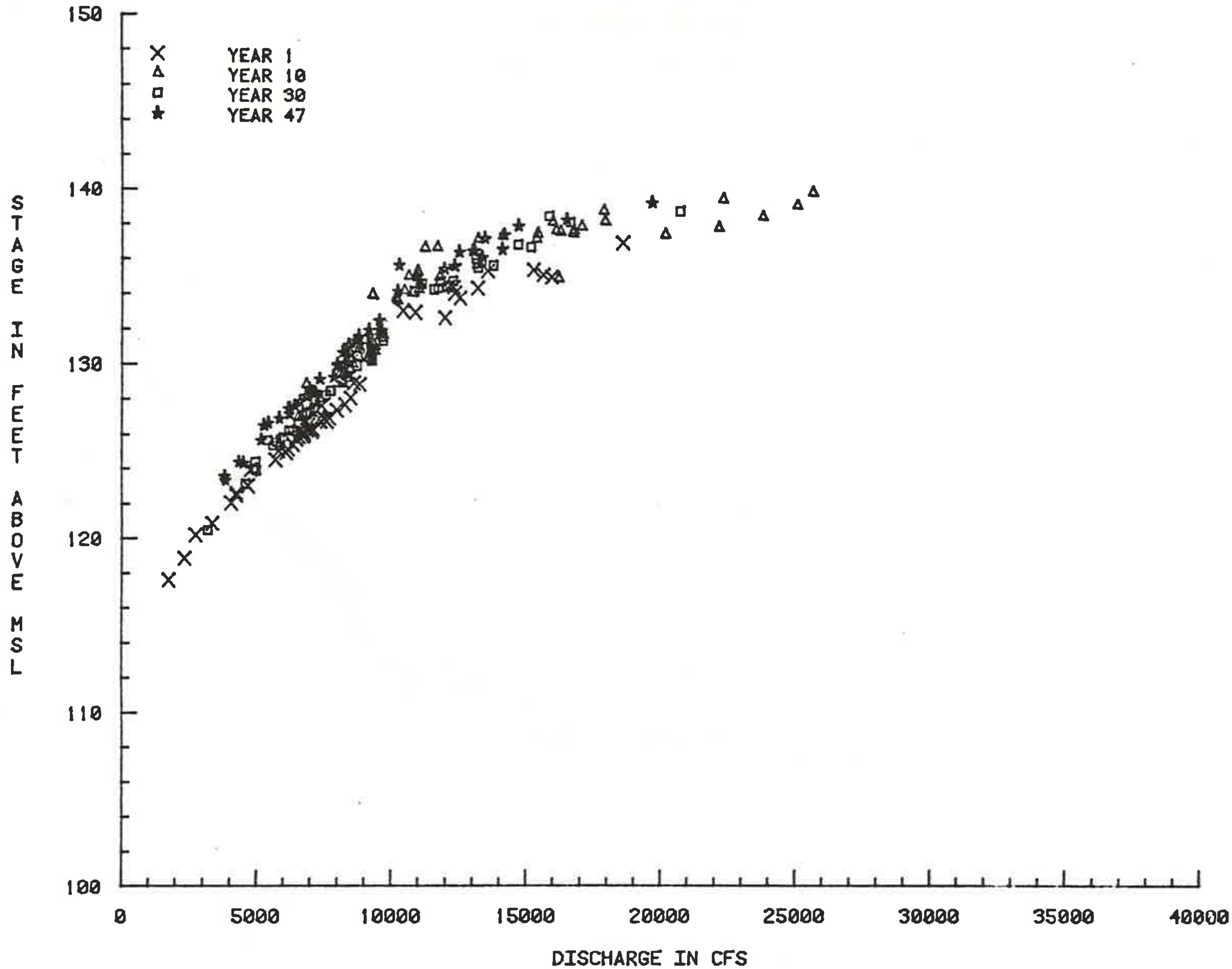


Figure C-12. Stage discharge relationship at Swan Lake (river mile 219.08) for the best alternative.

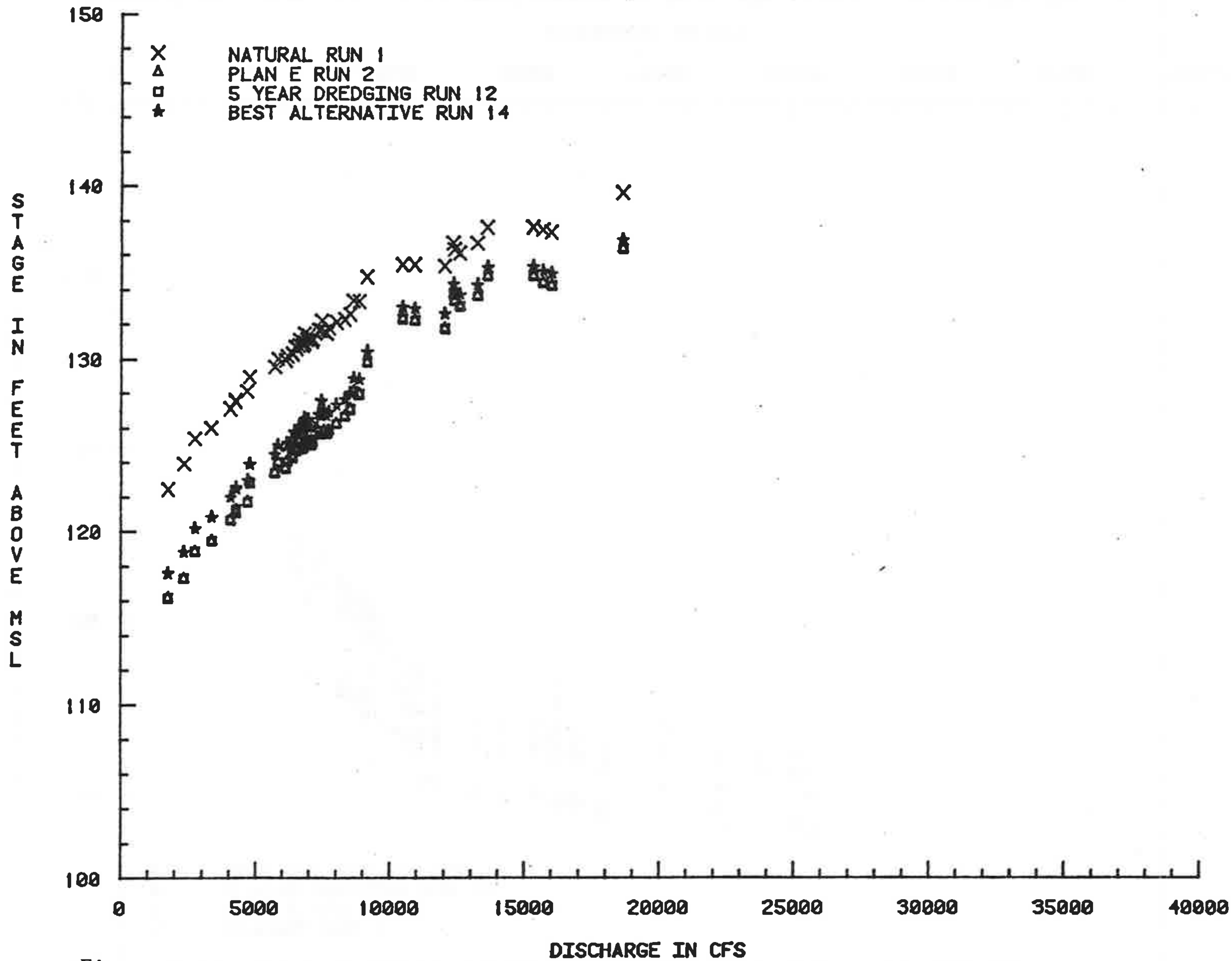


Figure C-13. Stage discharge relationship at Swan Lake (river mile 219.08) for Year 1.

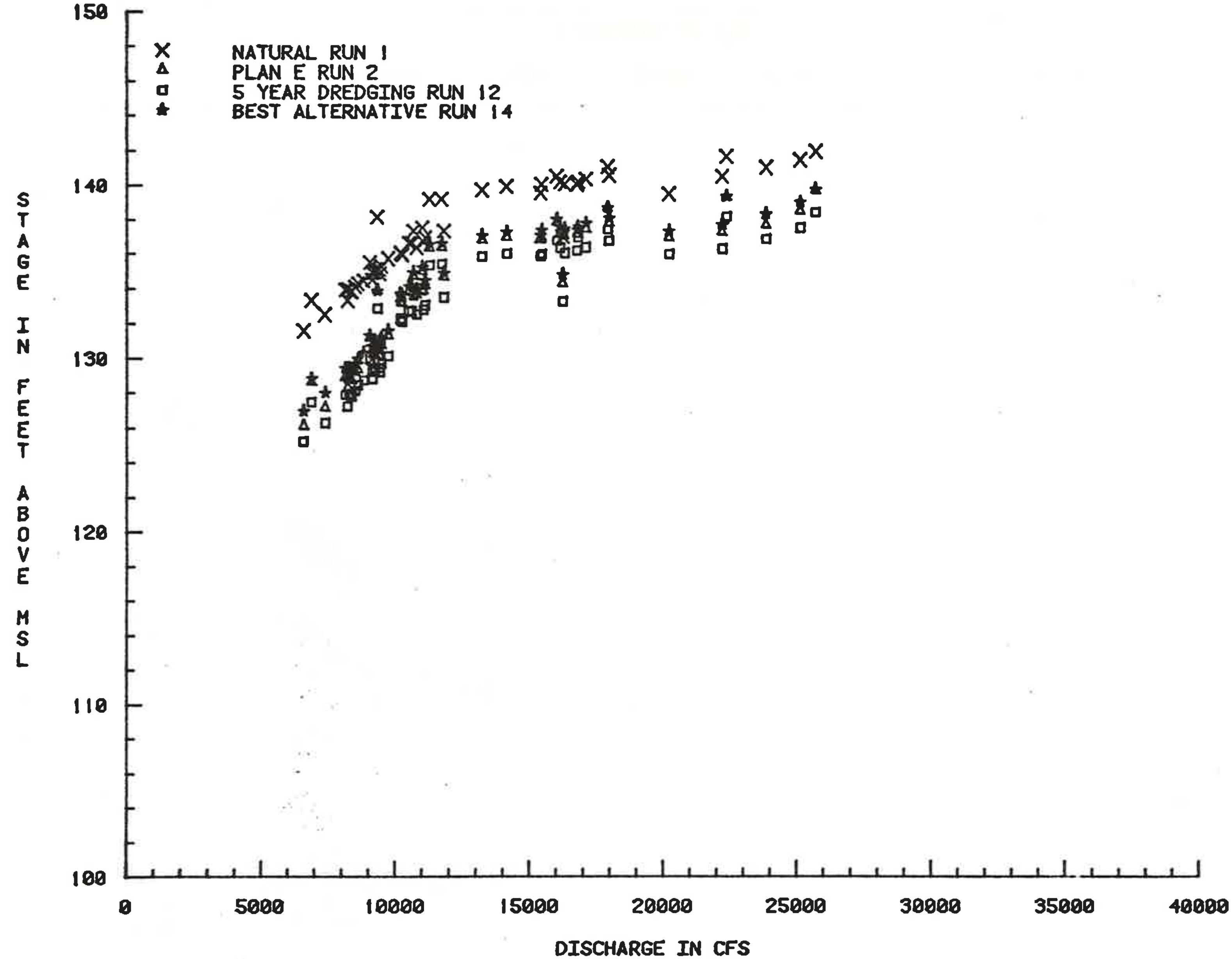


Figure C-14. Stage discharge relationship at Swan Lake (river mile 219.08) for Year 10.

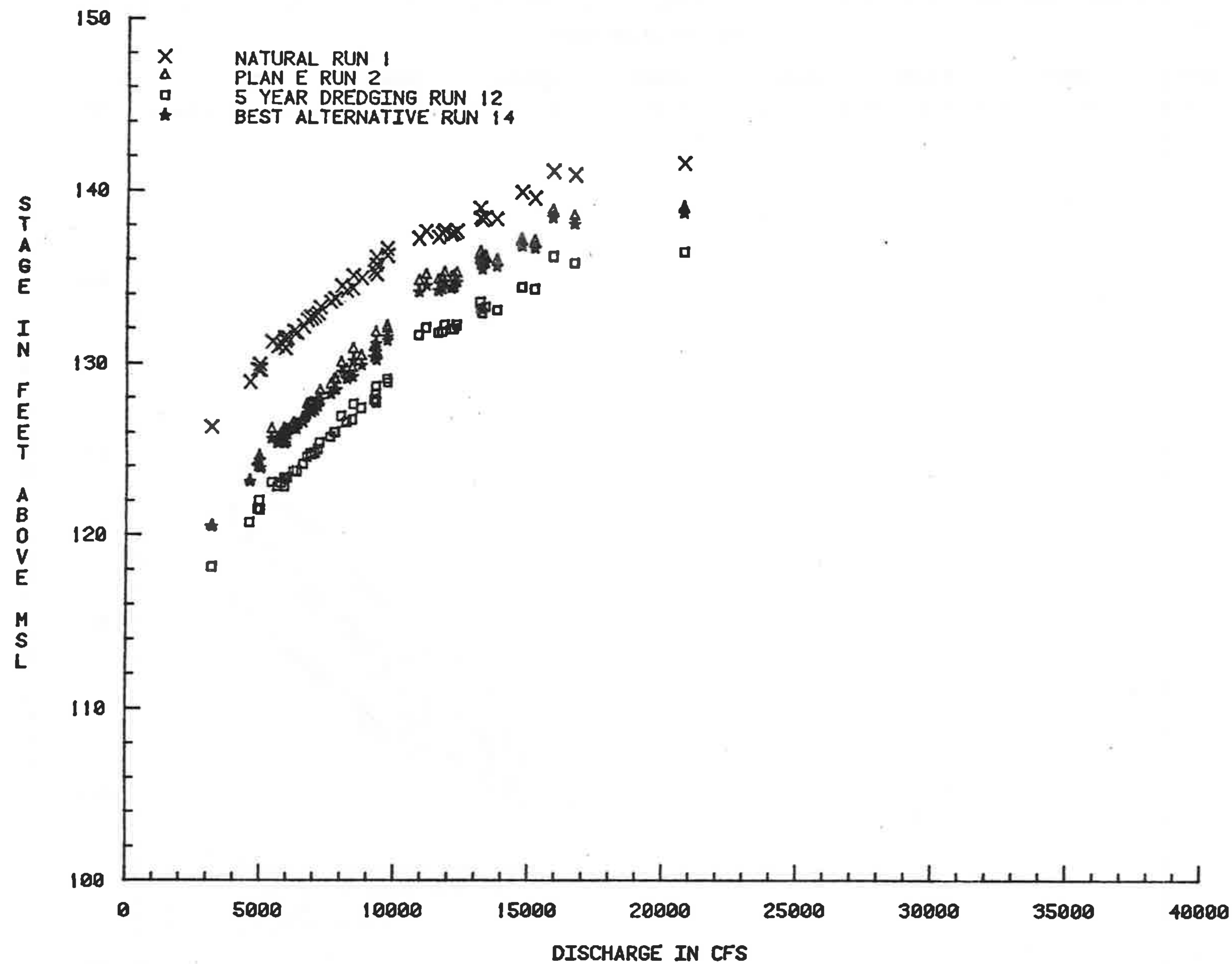


Figure C-15. Stage discharge relationship at Swan Lake (river mile 219.08) for Year 30.

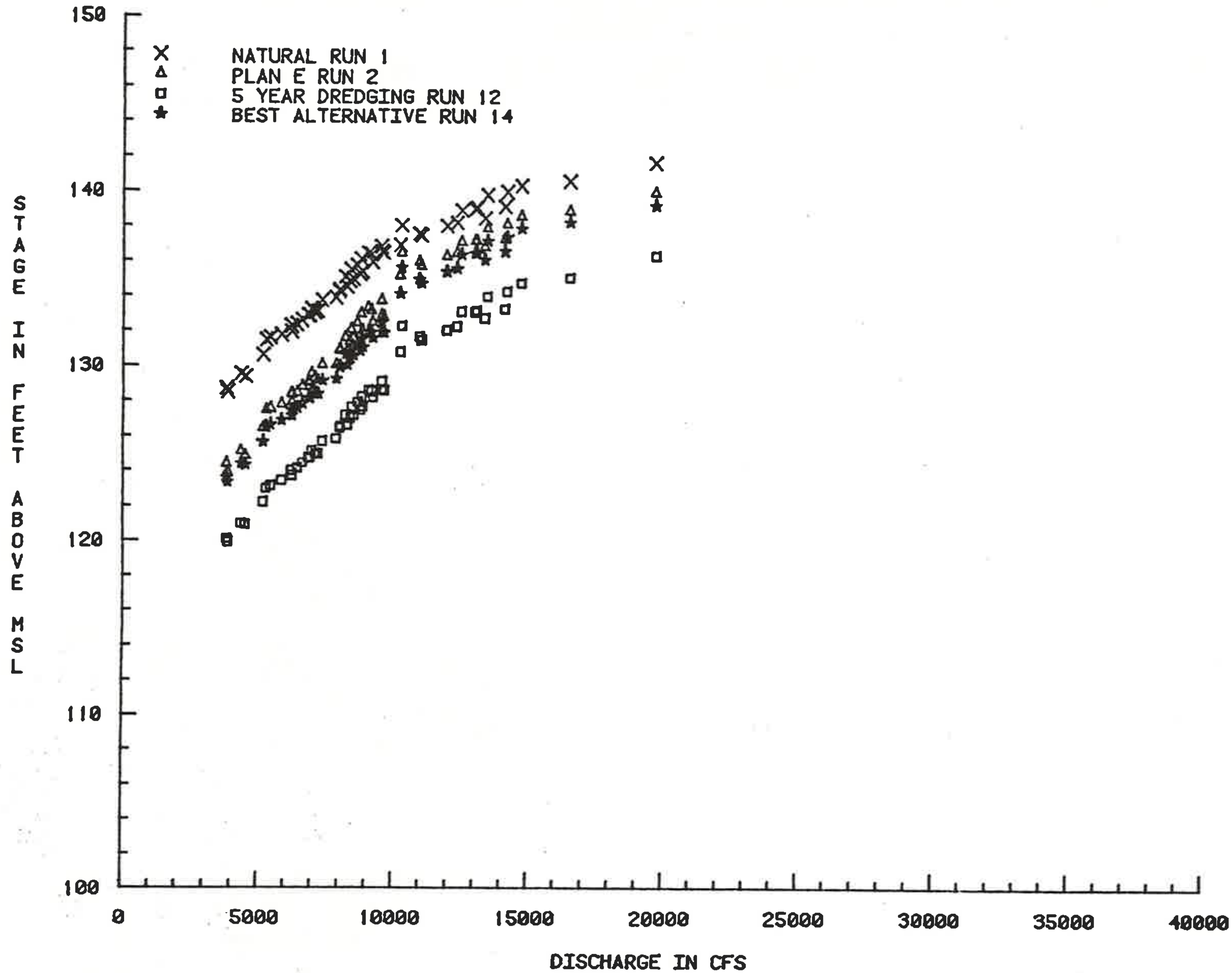


Figure C-16. Stage discharge relationship at Swan Lake (river mile 219.08) for Year 47.

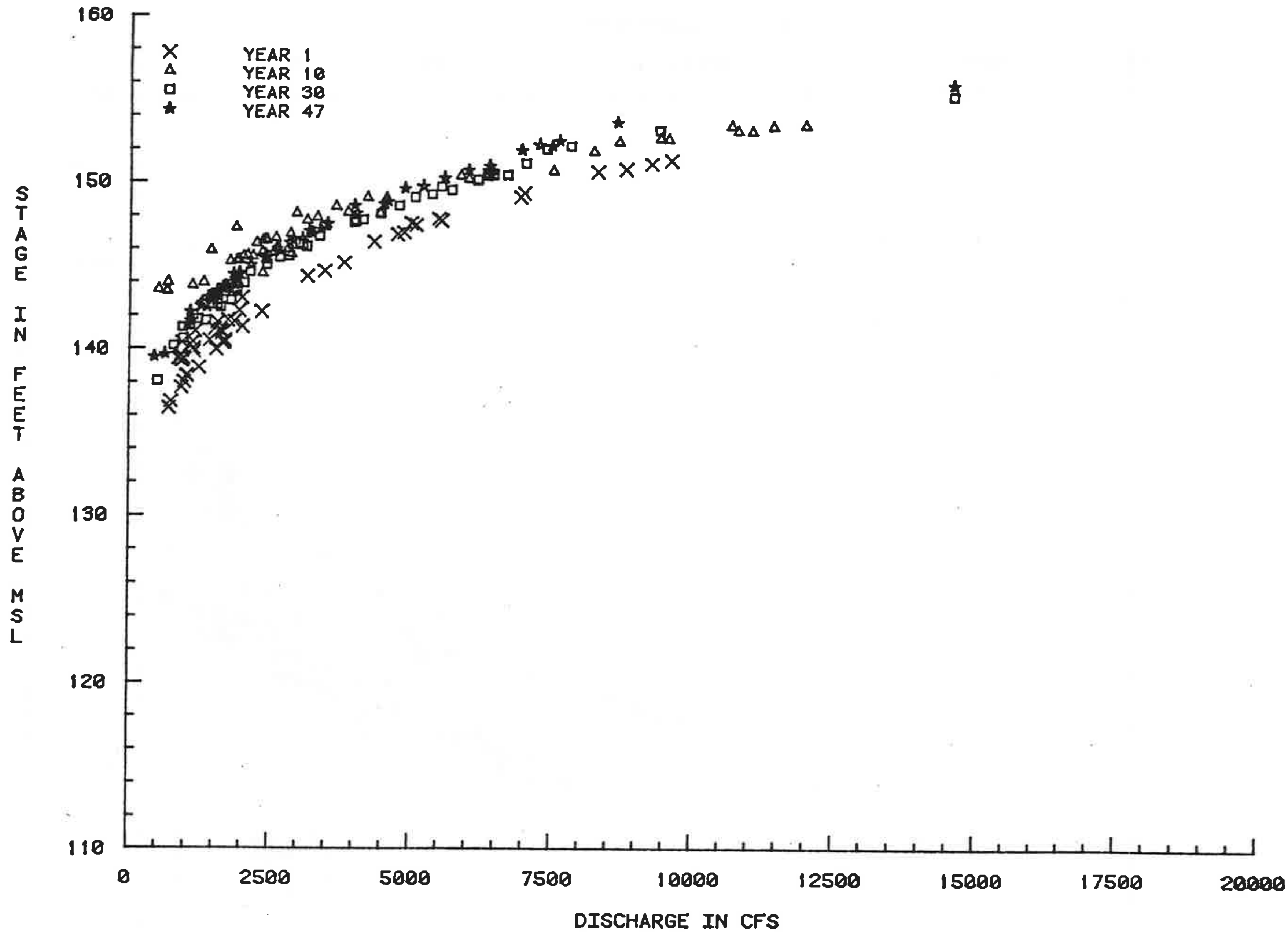


Figure C-17. Stage discharge relationship at Lambert (river mile 253.19) for natural conditions.

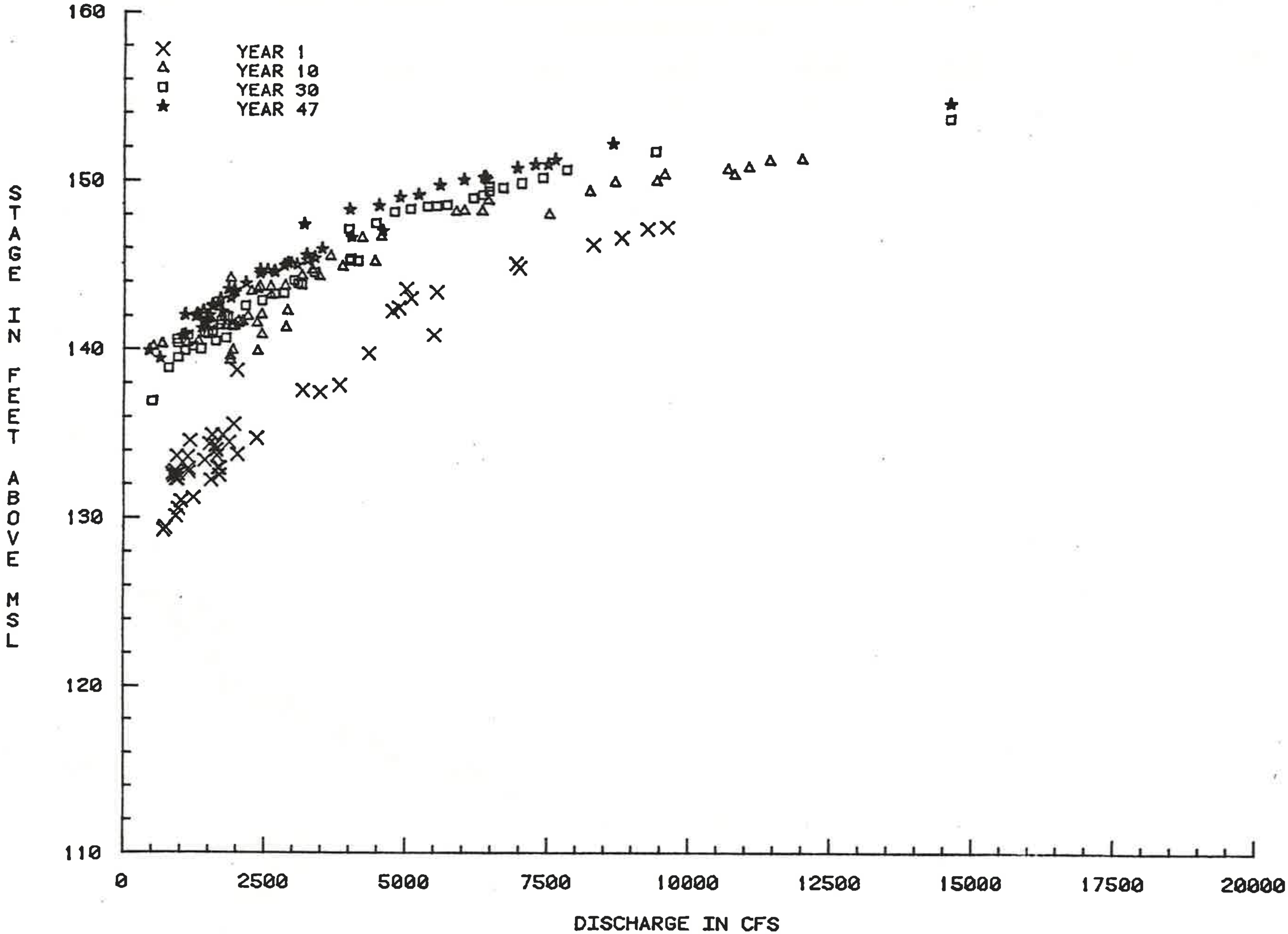


Figure C-18. Stage discharge relationship at Lambert (river mile 253.19) for Plan E.

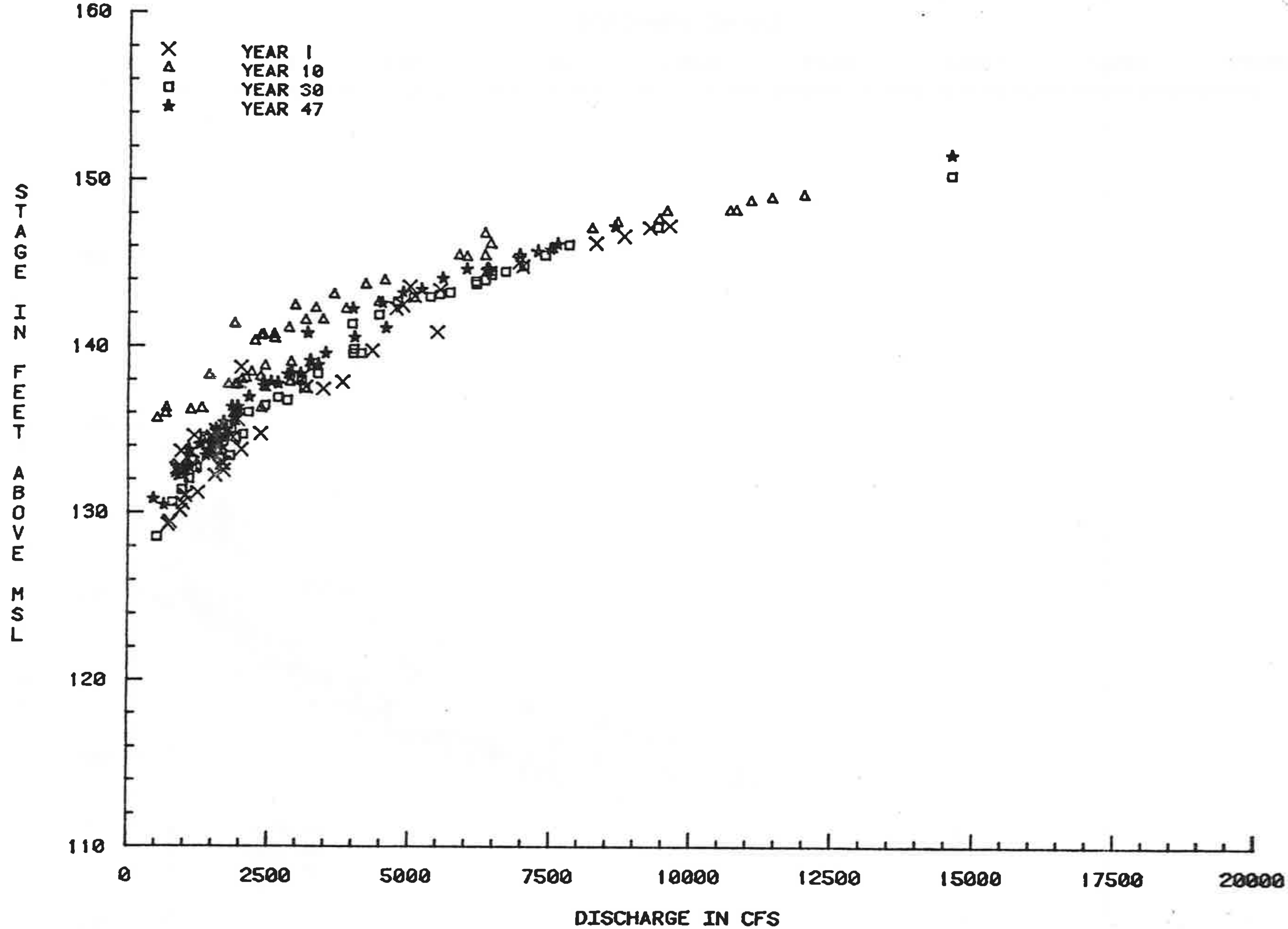


Figure C-19. Stage discharge relationship at Lambert (river mile 253.19) for every 5-year dredging.

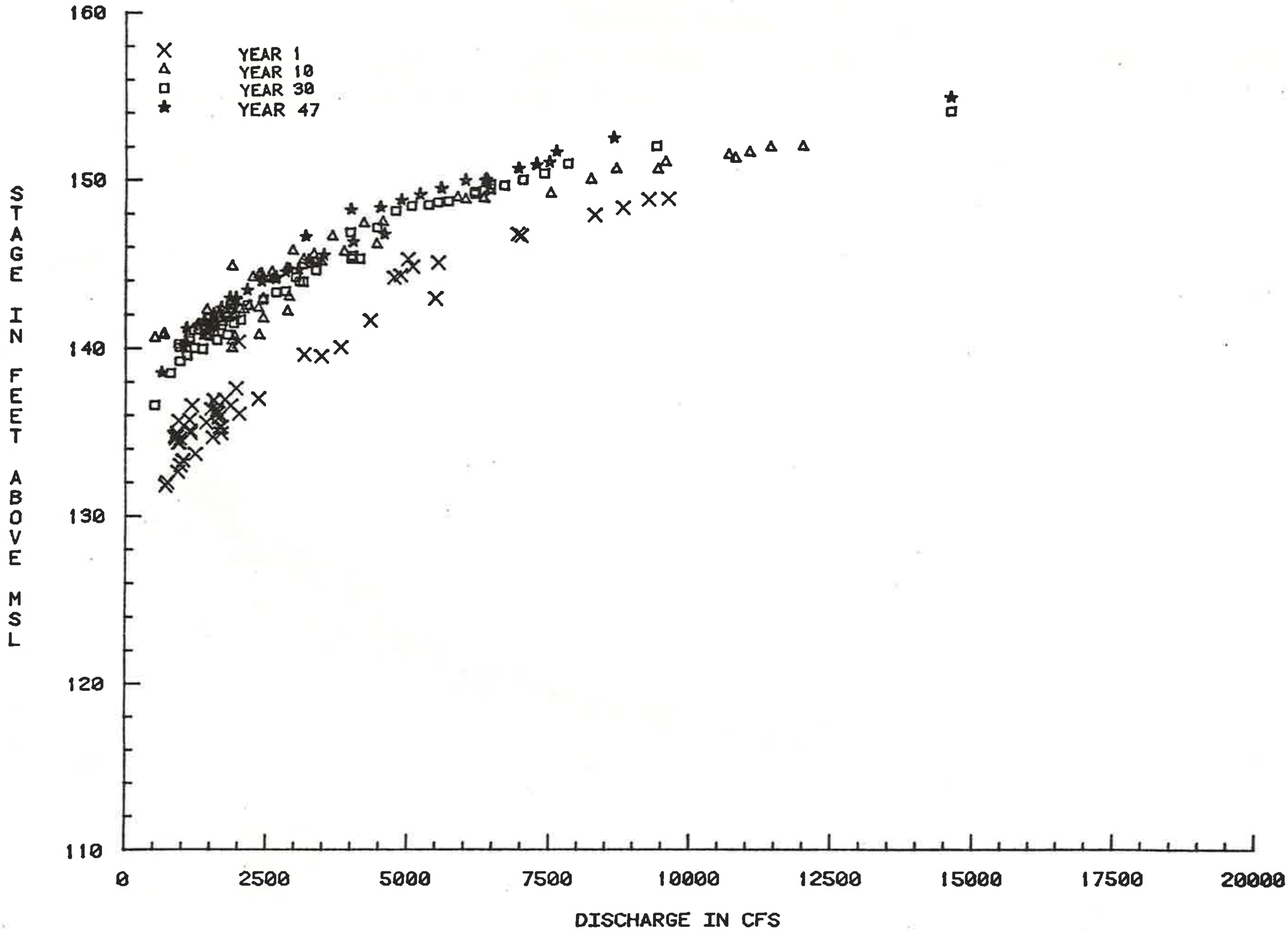


Figure C-20. Stage discharge relationship at Lambert (river mile 253.19) for the best alternative.

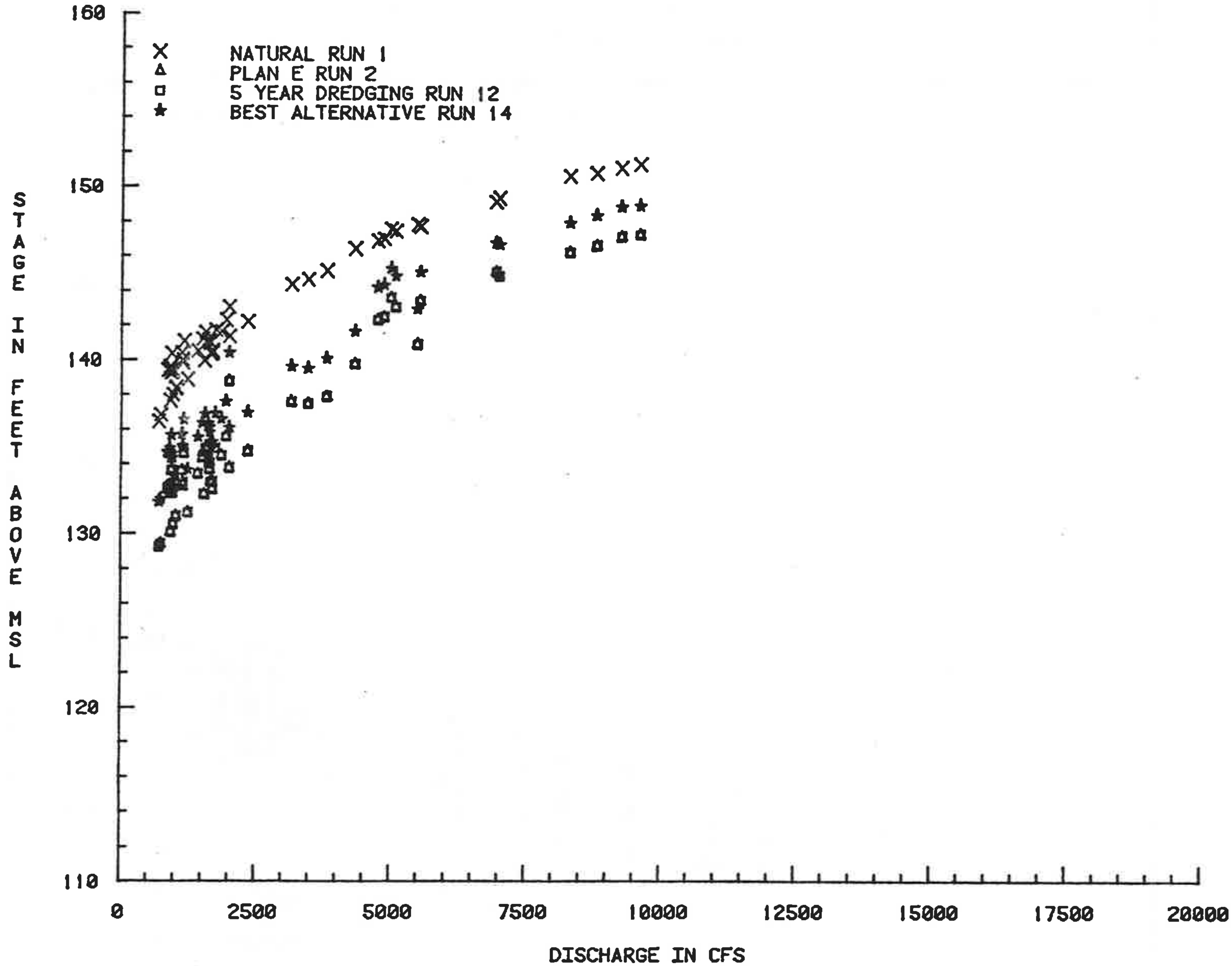


Figure C-21. Stage discharge relationship at Lambert (river mile 253.19) for Year 1.

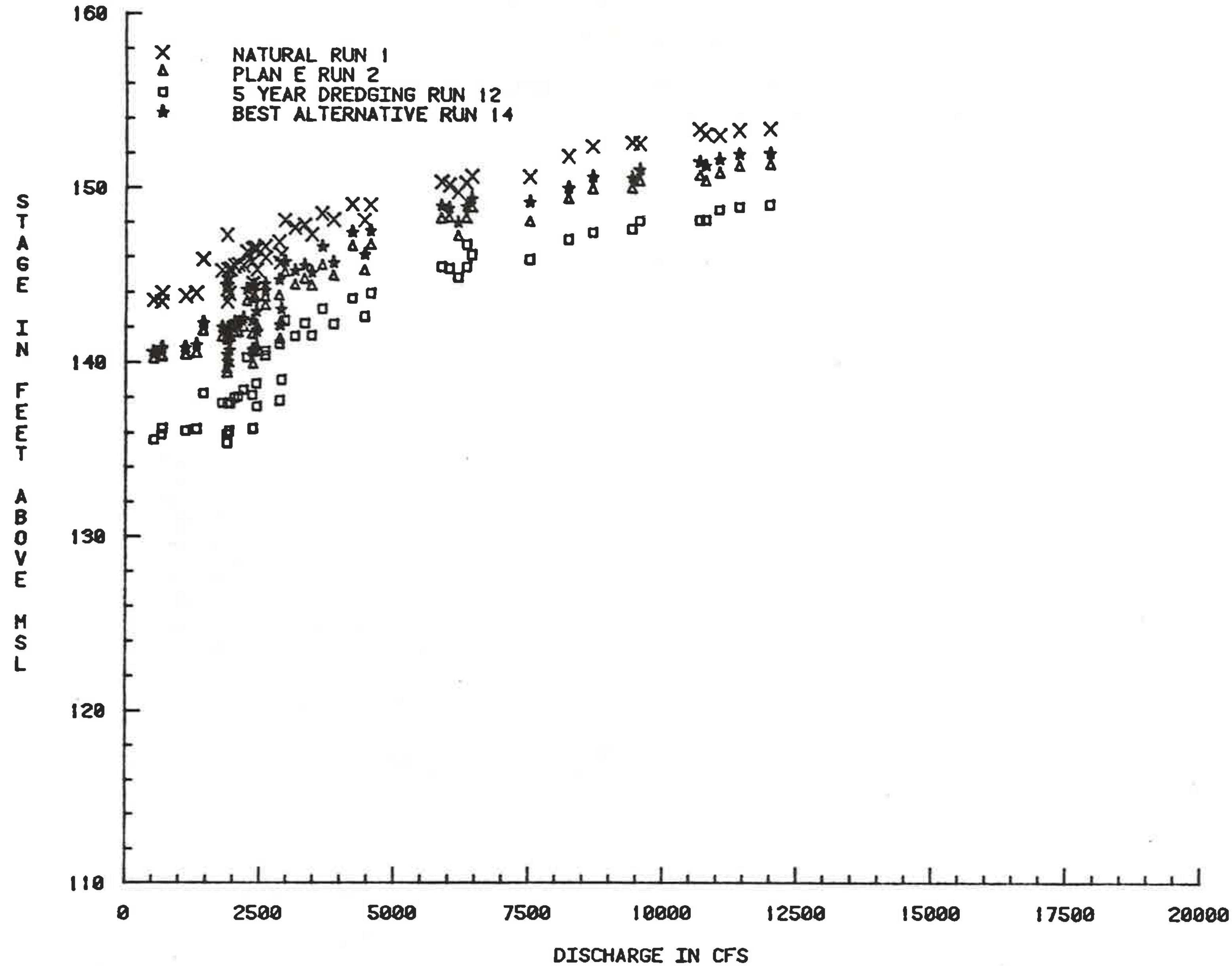


Figure C-22. Stage discharge relationship at Lambert (river mile 253.19) for Year 10.

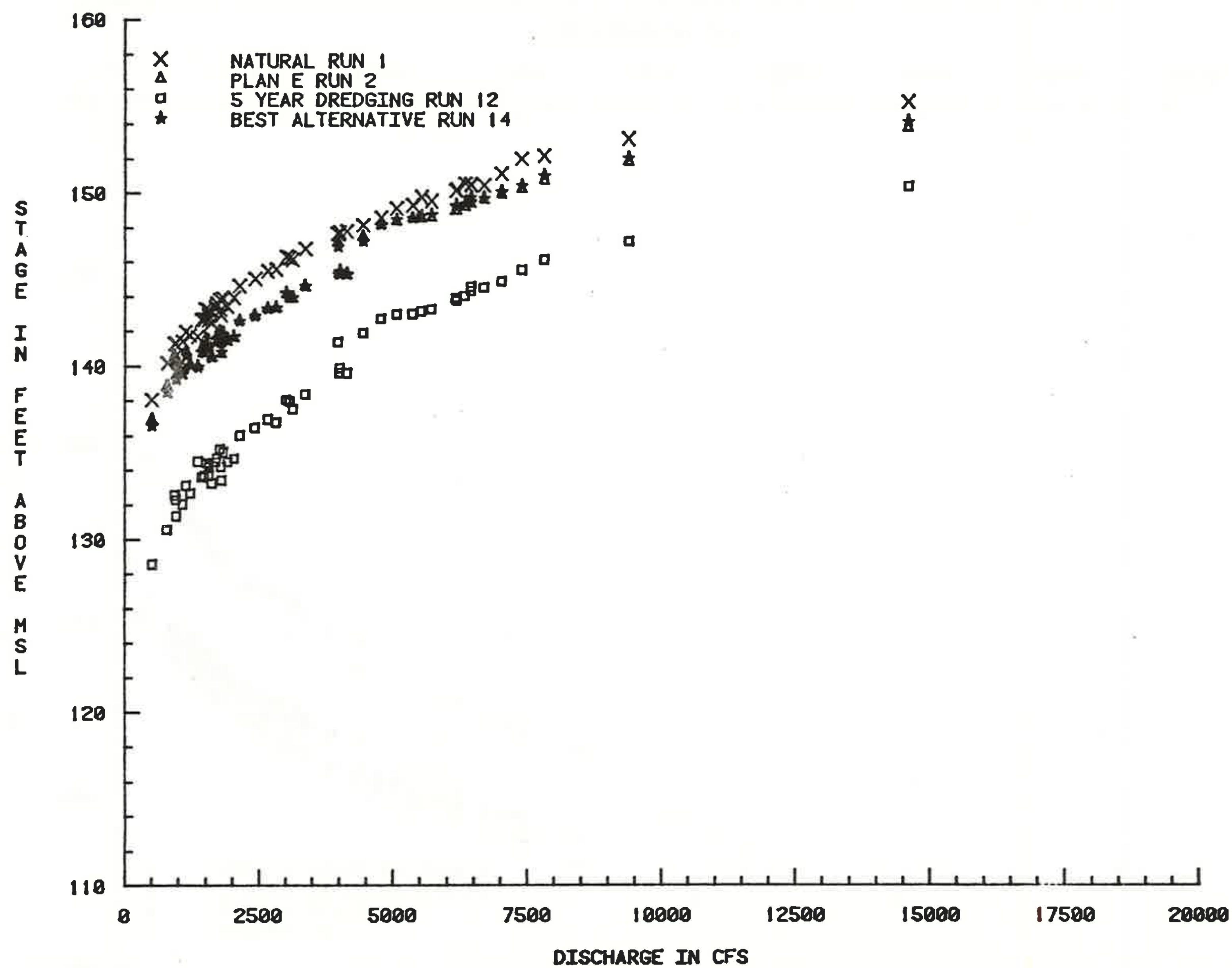


Figure C-23. Stage discharge relationship at Lambert (river mile 253.19) for Year 30.

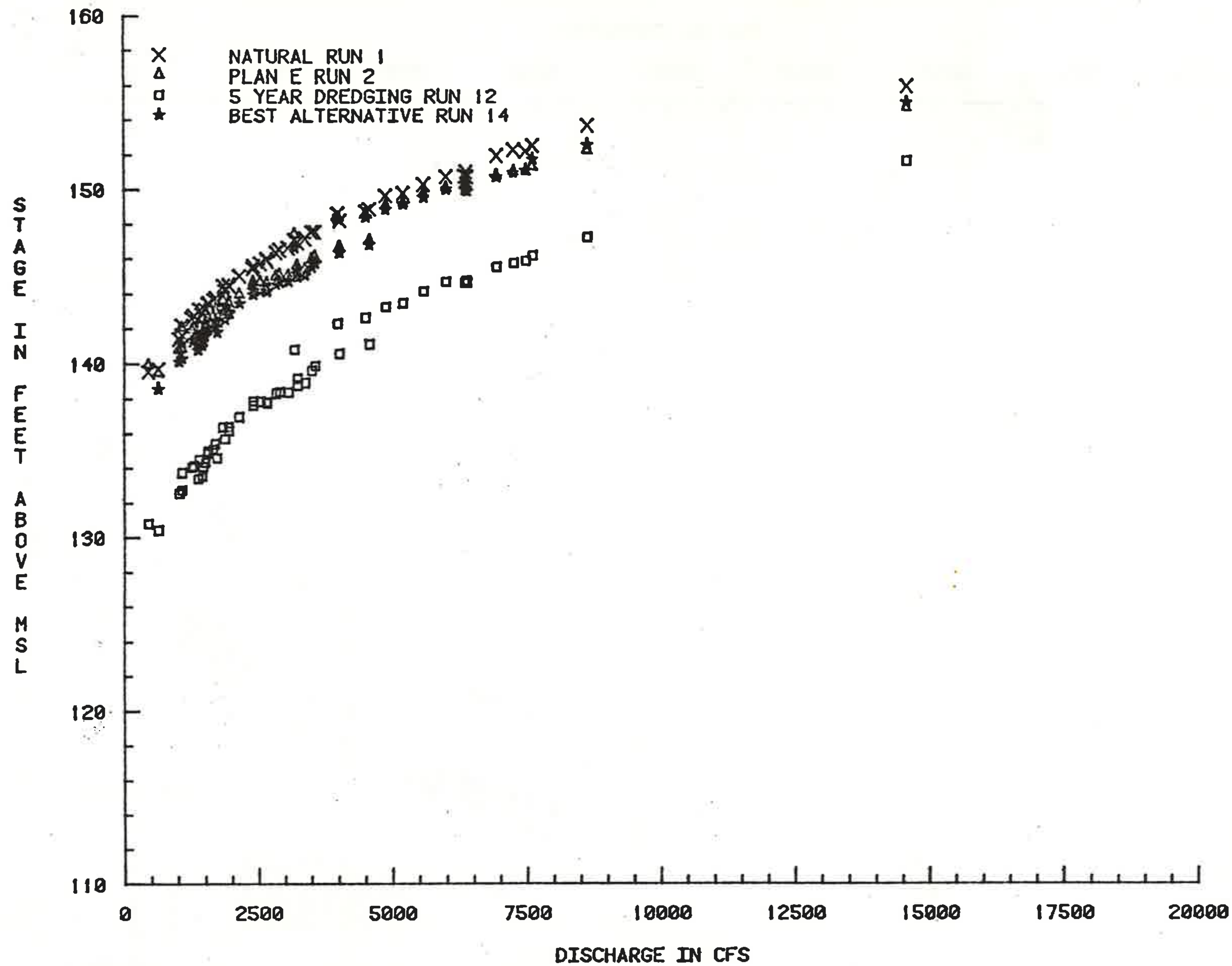


Figure C-24. Stage discharge relationship at Lambert (river mile 253.19) for Year 47.