State Water Survey Division

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WAVES AND DRAWDOWN GENERATED BY RIVER TRAFFIC ON THE ILLINOIS AND MISSISSIPPI RIVERS

Submitted to:

The Environmental Work Team
Master Plan Task Force
Upper Mississippi River Basin Commission

Submitted by:

Nani G. Bhowmik, Principal investigator

Prepared by:

Misganaw Demissie and Sid Osakada

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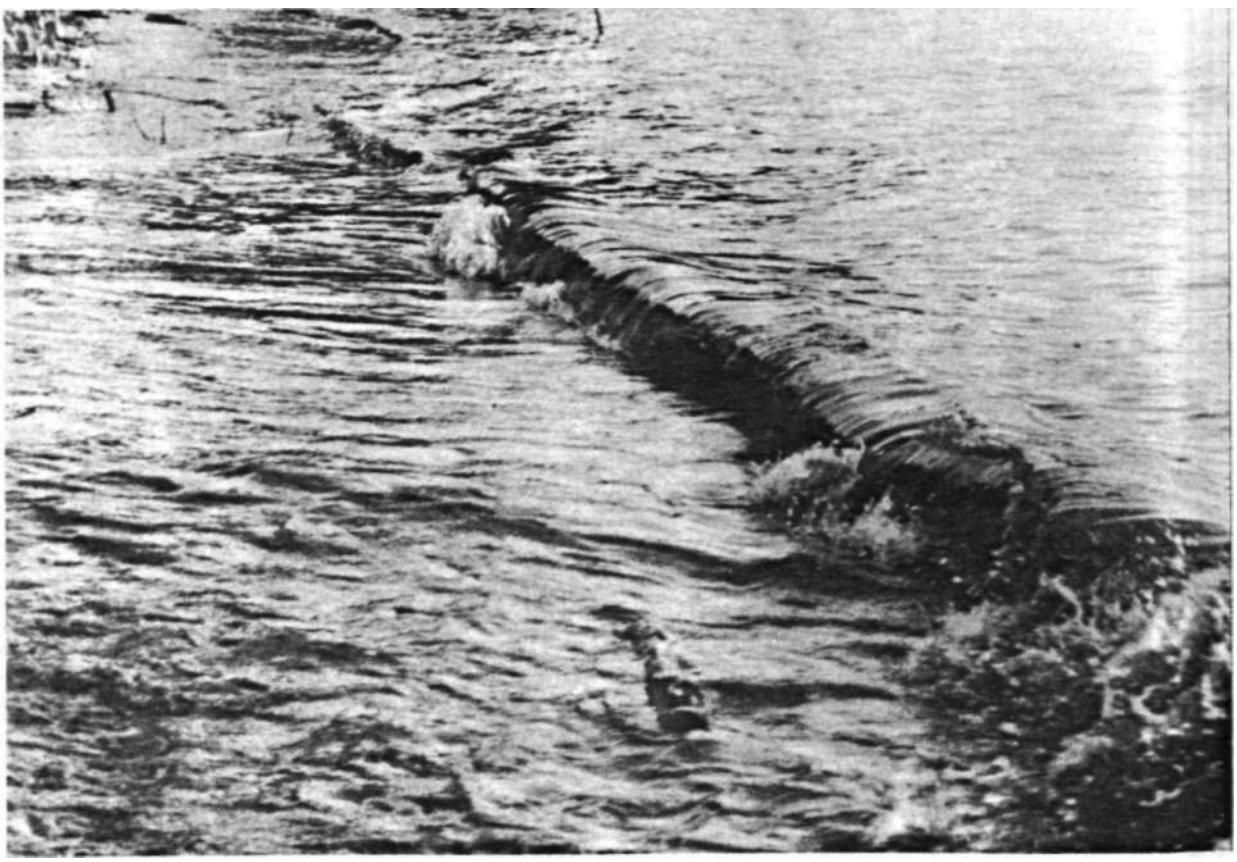

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ABSTRACT

An excessive amount of bank erosion along a number of waterways in Illinois and the surrounding states exists at the present time. Erosion of stream banks attracts public attention, reduces property value, results in permanent loss of real estate, increases turbidity of streams, and accelerates the silting of reservoirs and backwater lakes along the stream course. One of the main causes of bank erosion along navigable rivers is waves generated by river traffic and wind. In order to prevent the erosion of stream banks by waves, an understanding of the characteristics and energy content of the waves generated by river traffic and wind is necessary. Also associated with river traffic is the drawdown of the water level in the channel. The understanding of the drawdown caused by loaded tows is significant since the lowering of the water level exposes shore area and also changes flow characteristics of tributary streams close to their outlets.

To investigate and collect data on waves and drawdown associated with river traffic, five field trips were taken to the Illinois and the Mississippi Rivers. Wave and drawdown data were collected for a total of 41 and 27 tow passages, respectively. Additional wave data was collected during the passage of a towboat without barges and a cabin cruiser.

The maximum wave heights measured in the field ranged from a low of 0.1 foot to a high of 1.05 foot, while the maximum drawdown ranged from 0.05 foot to 0.69 foot. The measured maximum wave heights and drawdowns were compared to existing predictive equations. The correlations between the measured and calculated wave heights and drawdowns were low and for most parts the equations underestimated the measured wave heights and

drawdowns, except for one equation which overestimated the maximum wave heights. From the analysis of previous investigations and running a multi-variate regression analysis, it can be concluded that the maximum wave height depends on the velocity of the vessel, the blockage factor, and the length of the vessel. Similarly, the maximum drawdown depends on the velocity of the vessel, the blockage factor, the length of the vessel, and the distance from the sailing line.

INTRODUCTION

General Background

In the 1960s, the U.S. Army Corps of Engineers began planning to repair, replace, or enlarge the locks at Locks and Dam 26, Alton, Illinois. This locks structure controls access to the Illinois Waterway and the Upper Mississippi River. The following description of the steps from a 1968 recommendation to replace Locks and Dam 26 with a new structure with at least 4 times the lock capacity to the present status of the Master Plan is based directly on the introduction to the "Preliminary Comprehensive Master Plan for the Management of the Upper Mississippi River System" (Upper Mississippi River Basin Commission, 1981).

Master Plan Studies

In 1968, the District Engineer of the U.S. Army Corps of Engineers, St. Louis District, recommended a replacement project for Locks and Dam 26 providing for the construction of a new dam and two 1200-foot locks two miles downstream of the existing dam at Alton, Illinois. The Secretary of the Army approved the project, and in 1970 Congress appropriated funds for its design. Congress continued to appropriate funds for the project through fiscal year 1975.

On August 6, 1974, the Isaak Walton League, the Sierra Club, and 21 midwestern railroads filed lawsuits in the U.S. District Court to enjoin the U.S. Army Corps of Engineers from beginning construction on the Locks and Dam 26 replacement project at Alton, Illinois. The court ruling of September 5, 1974, stopped further actions toward construction until the U.S. Army Corps of Engineers obtained the consent of Congress and remedied the defects in the Environmental Impact Statement. In response to this ruling, the Corps of Engineers conducted additional studies which were submitted to Congress and ordered to be printed August 26, 1976—House Document No. 94-584 (Board of Engineers for Rivers and Harbors, 1976).

Among other conclusions, the Chief of Engineers recommended that:

> "Congress authorize the replacement of Locks and Dam 26 with a new dam and 110-foot by 1,200-foot main lock at a location two miles downstream from the existing dam, the design and construction of the new dam to provide for the addition of an auxiliary lock at such time as it may be authorized.
>
> and
>
> Congress authorize the Secretary of the Army, acting through the Chief of Engineers, in cooperation with the Departments of Transportation and Interior, the Environmental Protection Agency, and other interested Federal and State agencies to make an economic evaluation and a comprehensive study of the river environment to determine the impacts of increased navigation which would result from provision of a second lock and submit a report to the Congress on the feasibility and desirability of constructing a second lock."

During 1976 and 1977, Congress debated several bills to authorize the Corps of Engineers to begin construction of a replacement for Locks and Dam 26. Alternative navigation improvements were also being proposed. Because construction of a second lock would increase the river's capacity for waterway traffic, Congress would have to know the impact that increased traffic would have on the total river system and other modes of transportation. At issue were conflicts that had been brewing for decades: the demand for increased waterway commerce; environmental demands to preserve natural wildlife habitats and prevent damage to the rivers' ecology; and the impact that increased waterway commerce would have on other modes of commercial transportation...most notably the railroads and trucking.

On October 21, 1978, President Carter signed into law P.L. 95-502 directing the Upper Mississippi River Basin Commission (UMRBC) to prepare a Comprehensive Master Plan for the Management of the Upper Mississippi River System in cooperation with appropriate federal, state, and local officials. under this law, the Master Plan is to be submitted to Congress by January 1, 1982, with a preliminary plan to be completed by January 1, 1981.

The UMRBC established a task force to prepare the first draft of the Master Plan-Plan of Study. They soon realized the time constraints of P.L. 95-502 did not allow sufficient time to properly answer the questions and concerns raised by Congress. On August 15, 1979, the Commission adopted a Plan of Study for the development of a Comprehensive Master Plan for the Management of the Upper Mississippi River System. The Plan of Study outlined a four year study effort, 19 months longer than P.L. 95-502 specified. In September 1980, Congress denied the UMRBC's request for a time extension, so the Commission rescoped all study efforts as necessary to meet the dates specified in the law.

During the development of the Plan of Study and the initial work efforts, appropriations for the Master Plan were made by Congress in three separate appropriation actions. A supplemental Fiscal Year 1979 appropriation of $2,000,000 was approved in July 1979. Fiscal Year 1980 appropriation of $4,000,000 was approved in October 1979, and a $2,400,000 appropriation for Fiscal Year 1981 was approved in September 1980. A budget proposed for Fiscal Year 1982 is currently being considered through the Water Resources Council for $1,000,000.

The Master Plan is intended to identify the economic, environmental, and recreation objectives of the river system. Specifically, the plan will guide and direct any future expansions of navigation capacity, including but not limited to construction of a second lock at Locks and

Dam 26. The plan will also address projected effects of natural and man-made activities on the system. Within this overall goal of balancing economic, environmental, and recreational objectives of the river system are two subgoals: 1) to develop technical recommendations, and 2) to develop a management framework for resolving differences between competing interests. Most of the technical questions relate to transportation and environmental issues specifically requested by Congress. These include:

- the navigation carrying capacity of the Mississippi River System;
- the effect of expanded navigation on national transportation policy and the railroads;
- the cost and benefit to the nation of expanded navigation capacity;
- an evaluation of the need for a second lock at Locks and Dam 26;
- the effects of disposing of dredged material in areas outside of the floodplain;
- the development of a study to determine the feasibility of a computerized analytical inventory and analysis system; and
- the effect that increased navigational capacity would have on fish and wildlife, water quality, wilderness, and public recreational opportunities (UMRBC, 1981: 1-4).

Lead members of the Commission were given responsibility for carrying out specific technical studies in these areas. Responsibility for the study of navigation effects was given to the Department of Interior.

<u>Water Survey Involvement</u>

In response to the Plan of Study, a consortium of researchers submitted a combined proposal to accomplish navigation effects studies on Pools 9 and 26 from May 1980 through September 1982. When Congress required the UMRBC to meet the original completion date of January 1, 1982, for the Master Plan, these studies were rescoped in November 1980. The rescoped studies did not include a comprehensive consolidated final report, and field studies were curtailed so that the individual principal investigators could submit final reports by September 1, 1981. This reduction in project duration from 29 to 16 months severely limits the amount of analysis and model development by the principal investigators.

The Illinois State Water Survey has conducted studies on several tasks in each phase of the navigation effects program.

Phase I. Literature review and impact analysis based on existing information.

This was a joint effort by the Water Survey and the Illinois Natural History Survey. A literature review was completed in September 1980 and a revised report was submitted in May 1981 (Lubinski et al., 1981). A report which proposed studies to provide data to address information gaps about the effects of navigation on the physical, chemical, and biological regime was submitted in December 1980 (Lubinski et al., 1980).

Phase II. Reconnaissance and selection of sites for site specific studies.

A report on the bed material characteristics and channel geometry of the 80 miles of the Illinois River in Pool 26 was submitted in July 1981 (Schnepper et al., 1981). This task was done by the Water Quality Section of the Water Survey.

Phase III. Impact studies at selected sites.

Three separate but related studies on the physical effects of navigation were performed by the Surface Water Section of the Water Survey.

1. Determination of the Magnitude, Nature, Energy Content, and Patterns of Waves Caused by the Movements of Various Types of Commercial and Recreational Vessels on Pool 26.
2. Determination of the Water and Sediment Inputs to Selected Side Channels and/or Backwater Lakes Associated with the Passage of Large Commercial and Recreational Vessels in Pool 26.
3. Determination of the Extent and Magnitude of Lateral Movements of Sediments Associated with the Movements of Recreational and Commercial Vessels in Pool 26.

The results of the first task are detailed in this report.

Waves Generated by River Traffic

An excessive amount of bank erosion along a number of waterways in Illinois and the surrounding states exists at the present time. Along some reaches of the Illinois River, it is estimated that 75 percent of the banks are being eroded away by waves generated by river traffic and wind (Bhowmik and Schicht, 1980). Similar types of bank erosion problems also exist along the Mississippi and Ohio Rivers. Erosion of stream banks attracts public attention, reduces property value, results in permanent loss of real estate, increases turbidity of streams, and accelerates the silting of reservoirs or backwater lakes along the stream course.

One of the main causes of bank erosion along navigable rivers is waves generated by river traffic and wind. In order to prevent the erosion of stream banks by waves, an understanding of the characteristics and energy content of the waves generated by river traffic and wind is necessary.

The characteristics of the waves thus determined can be used to evaluate the relative magnitude of the effects of river traffic generated waves and wind generated waves on the shoreline.

When the project was initially funded by the Upper Mississippi River Basin Commission, it was to last for two and one-half years. This time period was to be used to collect field data on waves generated by river traffic and wind, analyze the data, and prepare a report. However, the duration of the project was suddenly cut to 16 months, and during that period, funding for the study was uncertain. These two factors, shortening the project duration and constant uncertainty about funding, have greatly reduced the data base and limited the analysis of the data. This report will include all the data collected during the project period and some analysis of the data collected.

Objectives

The objective of this project can be divided into two main parts as follows:

A. To collect a set of data on waves generated by river traffic and winds on the Illinois or Mississippi River (a representative waterway of the U.S.) in a systematic manner to answer questions such as: What are the characteristics of tow, barge, or boat-generated waves in an inland waterway? What are the similarities and dissimilarities between these waves and those produced by natural effects, such as wind? How does the intensity of the waves change with increasing river traffic?

B. To determine the bank erosion potential of these waves.

Acknowledgements

This project was conducted under the administrative guidance of Stanley A. Changnon, Jr., Chief, Illinois State Water Survey, and Michael L. Terstriep, Head, Surface Water Section.

Several Surface Water Section personnel assisted greatly in the field data collection program. Dr. J. Rodger Adams, Richard Allgire, G. Michael Bender, William C. Bogner, Allen P. Bonini, John Buhnerkempe, D. Kevin Davie, William Fitzpatrick, and Vernon Knapp all assisted in the field work. David Kisser, David Jennings, and William C. Bogner did all the surveying needed for this project.

Bruce Komadina of the Data Management Unit of the Water Survey designed and built the electronic wave gages used for this project. He also wrote the description of the wave gage in this report. Randall K. Stahlhut wrote the sampling and data storage programs for the CBM computer. Sid Osakada and Shohei Nagao, graduate students in the Civil Engineering Department at the University of Illinois, and Sunil Mishra, a graduate student in the Mechanical Engineering Department at the University of Illinois, assisted in data reduction and analysis.

Illustrations for the report were prepared under the direction of John Brother, Jr, and Pamela Lovett typed the camera-ready text.

LITERATURE REVIEW

River Traffic Generated Waves

Most of the early and contemporary research concerning vessel motion in water is concentrated on the reduction of resistance forces generated by a vessel to improve the speed and maneuverability of the vessel. As a vessel moves through water, it experiences resistance to its motion. The resistance to motion is composed of three types of forces generated as a reaction to its motion and because of the disturbance created by the vessel. The first form of resistance is the friction drag acting tangent to the wetted surface of the vessel. The frictional drag is generated by viscous resistance of the water. The second form of resistance is the eddy drag generated by the turbulent wake created by the vessel. The third form of resistance is the wave drag due to the waves generated by the vessel's motion.

The literature on the characterization, quantification, and reduction of the different types of forces generated by a vessel motion is enormous (Comstock, 1967; Sorensen, 1973). Since the primary interest of this research is in the effects of waves generated by river traffic on the stream banks, only literature as it relates to the present study will be reviewed.

As a vessel moves on or near the free surface of a water body, it generates a disturbance in the flow field. The flow around the hull of the vessel is accelerated due to changes both in magnitude and direction. The flow in front of the bow is decelerated until it reaches the stagnation point (where the velocity is zero) at the bow because of the blockage of the flow area by the vessel. These accelerations and decelerations result in corresponding changes in pressure and thus water level elevation. In areas where the flow is accelerated, the pressure and thus the water level elevation drops, and vice versa. Waves are generated at the bow, stern, and any points where there are abrupt changes in the vessel's hull geometry to cause disturbance in the flow field. As the vessel moves forward with respect to the water, the energy transferred to the water from the vessel generating the disturbance is carried away laterally by a system of waves similar to that shown in figure 1 (Sorensen, 1973; Comstock, 1967). Figure 1 is for deep water conditions where the depth has no affect on the flow field. In general the system of waves will consist of two sets of diverging waves and one set of transverse waves. The diverging waves move forward and out from the vessel, while the transverse waves move in the direction of the vessel. The transverse waves meet the diverging waves on both sides of the vessel along two sets of lines called the cusp lines which form a 19°21' angle with the sailing line for a point disturbance moving at a constant velocity in an initially still, deep and frictionless fluid (Sorensen, 1973). The theory to describe the above wave pattern was first developed by Lord Kelvin. Sorensen has shown that the general wave pattern generated by a model hull in deep water agrees well with the wave pattern described by Lord Kelvin (1887) except for a small change in the cusp angle.

A descriptive sketch of a wave system is shown in figure 2. C is the wave celerity (the speed the wave propagates forward), H is the wave height, L is the wave length (distance between adjacent wave crests or troughs), and d is the water depth. The wave period, T, which is the time elapsed between two adjacent waves crests or troughs past a point, is

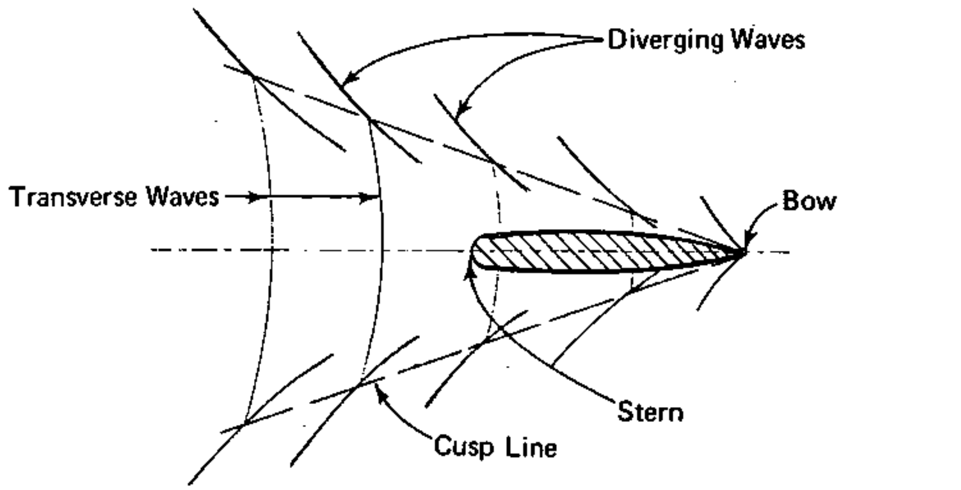


Figure 1. Wave pattern generated by a model ship in deep water

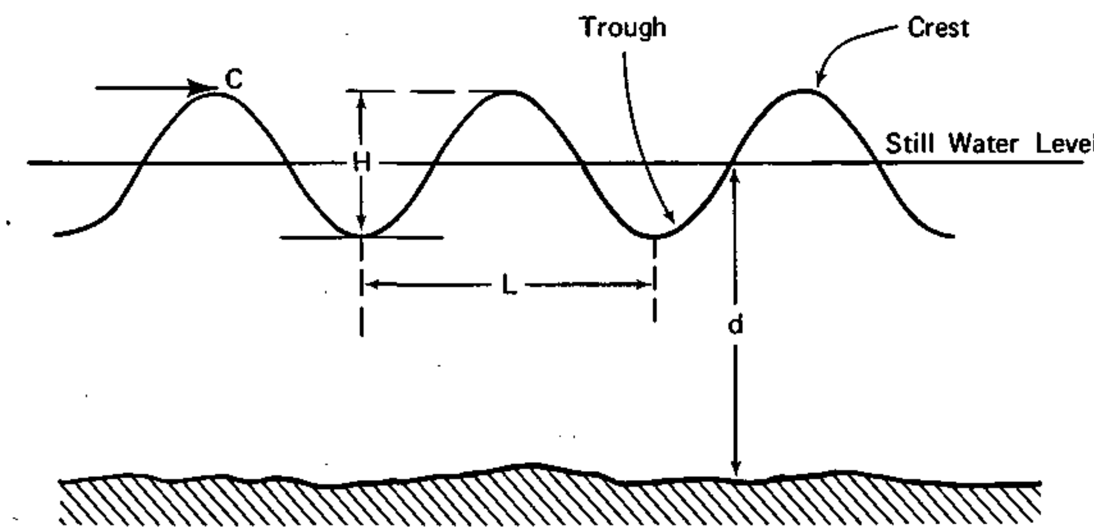


Figure 2. Descriptive sketch of wave profile

given by T=L/C. The ratio d/L determines whether the wave system is in deep or shallow water. For deep water waves, $d/L \geq 0.5$. In deep waters the wave celerity and wave length depend only on wave period, while in shallow water ($d/L < 0.5$) the wave celerity and wave length depend on depth as well as wave period (Ippen, 1966; Sorensen, 1973).

Since waves are generated both at the bow and stern of a vessel, they interact with each other at some distance from the vessel. If the waves generated at the bow and stern are in phase, i.e. if the crest and trough of one set coincide with the other, they tend to reinforce each other resulting in higher waves. If the waves are out of phase, they tend to cancel each other resulting in relatively smaller waves. Whether the waves will reinforce or cancel each other depends on the length Froude number, $F_l = V/l^{1/2}$ (Comstock, 1967; Sorensen, 1973). V and l are the vessel velocity and length, respectively.

In deep water the wave heights generally increase with increasing velocity, except at certain velocities where the bow and stern waves tend to cancel each other. The wave heights then decay with distance from the vessel as the total energy per wave is distributed over a larger area (Sorensen, 1973; Das, 1969; Bhowmik, 1976; Johnson, 1968; Das and Johnson, 1970).

In shallow water, the water particle motion generated by the waves will reach the bottom and the wave pattern will change significantly. The important parameter in shallow water waves is the depth Froude number, $F_d = V/(gd)^{1/2}$. V is the vessel velocity, d is the water depth, and g is the gravitational acceleration. For F_d above approximately 0.4, the waves will reach the bottom. As F_d increases, with an increase in vessel velocity or a decrease in depth, the diverging waves rotate forward and finally make a right angle with the sailing line for F_d=1. Therefore at F_d=1, both the diverging and transverse waves form a single wave, which travels with the same speed as the vessel. The limiting vessel velocity, determined at critical F_d, is given by $(gd)^{1/2}$ (Sorensen, 1973).

Channel Constriction Effects

In shallow water, the depth restriction has been shown to play a significant role in modifying the wave pattern. If a water body is narrow in the lateral dimension, a complex flow condition and wave pattern will result. When the channel is narrow so as to affect the flow pattern around a vessel, the waves generated will be relatively higher than those generated in unrestricted waters by the same vessel moving at the same speed. This is because of a significant reduction in the flow area and the associated higher accelerations of flow around the vessel. Higher acceleration results in lower pressures generating higher waves. If in addition to being narrow the channel is shallow, the combined effect will result in more complex flow conditions and much higher wave heights (Sorensen, 1973).

Drawdown

As a vessel moves forward, it pushes the water in front of it sideways and down underneath it. At the same time, it leaves an open space behind it momentarily causing water to flow from all directions to fill the void. The propellers of the vessel also suck a large amount of water from beneath the vessel. All these flow conditions cause acceleration of the water in the vicinity of the vessel. As the water is accelerated, increasing in velocity, a drop in pressure results. In energy terms, the kinetic energy of the water increases while its potential energy decreases. Decrease in potential energy and pressure manifest themselves in the lowering of the water elevation. As the water level drops, the vessel also drops down. The drop or lowering of the vessel is known as "squat". The drop of the water level in the whole flow field is known as the drawdown.

In canal and harbor entrance design, the squat is of primary importance because of grounding and loss of control of the vessel at high squats. In stream bank erosion studies, however, the water elevation fluctuation at the stream banks is of greater significance. Generally the drop in water elevation is the greatest around the vessel and decreases with increasing distance from the vessel. It is, therefore, reasonable to assume that the drawdown at the stream banks is less than the squat; however, it is generally assumed that both the squat and drawdown are equal to simplify the physical process into one-dimensional flow for analytical analysis (Schijf and Jansen, 1953; Kaa, 1978).

Channel constrictions both in depth and width greatly increase the drawdown since the flow in restricted channels is accelerated more than the flow in unrestricted waterways. If a vessel travels close to one of the banks, the drawdown will be higher in the region between the vessel and the stream bank than it would have been if the vessel was traveling along the middle of the channel (Bouwmeester et al, 1978; Kaa, 1978).

DATA COLLECTION PROGRAM

Site Selection and Description

In the initial stages of all the Upper Mississippi River Basin Commission funded projects both in Pools 26 and 9, the study sites for site specific impact investigations were to be chosen in a coordinated effort between the principal investigators for the different projects. It was hoped that physical, chemical, and biological data will be collected at the same sites if not at the same time. After several coordination meetings, separate site selection criteria were developed for physical and biological studies. Reconciling the two criteria for physical and biological studies into one criteria never materialized because of the termination of the coordination phase of the project.

The criteria for selecting study sites for physical effects both in Pools 26 and 9 are shown in table 1. The favorable site characteristics were divided into two catagories: primary and secondary site characteristics. The primary criteria were related mainly to the channel geometry and alignment arid to the avoidance of obstructions which might require river traffic to stop, coast, or maneuver.

The secondary criteria were related mainly to logistic requirements. Land and river access to the test site and shore area suitable for installing wave gages were considered important.

After reviewing hydrographic maps (U.S. Corps of Engineers, 1971), topographic maps, and navigation charts of the Illinois and Mississippi Rivers (U.S. Corps of Engineers, 1974 and 1978), several sites were identified as possible locations for site specific studies according to the criteria established in table 1. Aerial reconnaissance of these sites

Table 1. Site Selection Criteria

Primary Site Characteristics

- Typical 9 foot navigation channel
  - Depth about 15'
  - Representative width
  - Natural or dredged channel
  - Representative channel geometry and configuration
  - Alignment
    - Straight
    - Representative radius bend
  - No obstructions
    - Wing dams
    - Bridges
    - Loading docks
    - Fleeting areas
    - Marinas/boat launches
    - Ferry crossing
  - Coincide with side channel site, if possible
  - Possible site for intensive biological study
  - Barges not coasting or maneuvering

Secondary Site Characteristics

- Land access
  - Vehicles
  - Survey stations and related lines-of-sight
- Shore area
  - For installation of wave instrumentation (Pool 26 only)
  - Boat landing
- River access
  - Boat launching site nearby
  - Secure boat harbor for boats

was taken to further narrow down the number of possible sites. The aerial reconnaissances provided information on whether river traffic maneuvered or coasted around the sites and also provided additional and up-to-date information on access to the sites. After the aerial reconnaissance, field trips were taken to each site to further reduce the number of possible study sites.

Four sites, two on the Illinois River and two on the Mississippi River, were then finally selected as the best sites to conduct site specific studies. The names of the sites and their locations are shown in table 2. Their relative positions on the Illinois and Mississippi Rivers are shown in figure 3.

Table 2. Name and Location of Study Sites

Site No.	Site Name	River	River Mile
1	Hadley's Landing	Illinois	13.2
2	Rip Rap Landing	Mississippi	265.1
3	McEver's Island	Illinois	50.
4	Mosier Landing	Mississippi	260.2

The Hadley's Landing site, shown in figure 4, is located at river mile 13.2 on the Illinois River in Pool 26. It is located about 8 miles south of Hardin on the west bank of the river. The test site is situated approximately at the middle of a gradual bend around Twelve Mile Island. Since the bend is very gradual, river traffic does not slow down to maneuver around the bend. The cross-sectional profile at Hadley's Landing test site is shown in figure 5. Also shown on the figure are the values for the discharge, Q, the cross-sectional area, A, and the mean velocity, 7, during the field trips to the site.

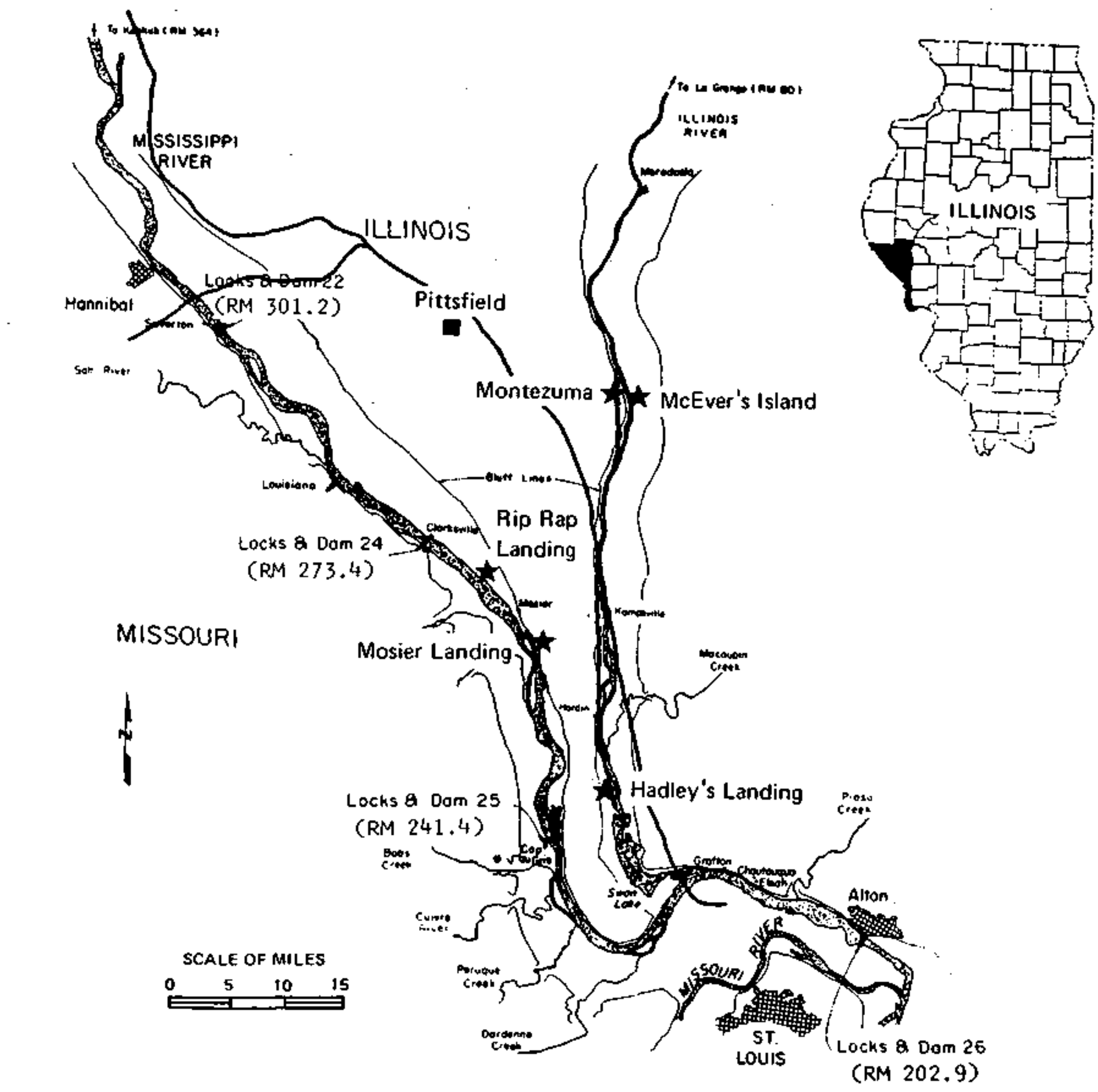


Figure 3. Location of study sites on the Mississippi and Illinois Rivers

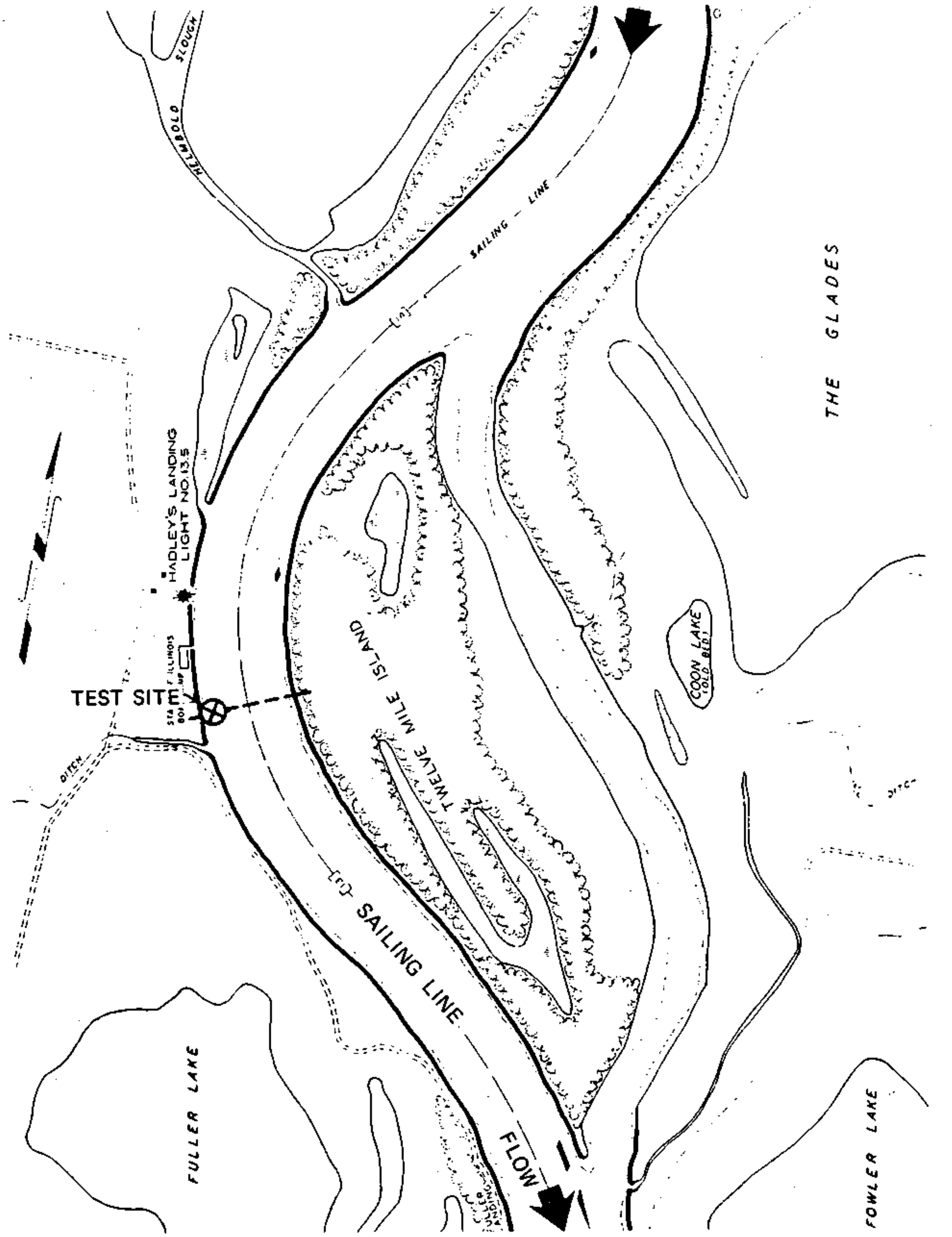


Figure 4. Location of Hadley's Landing test site on the Illinois River

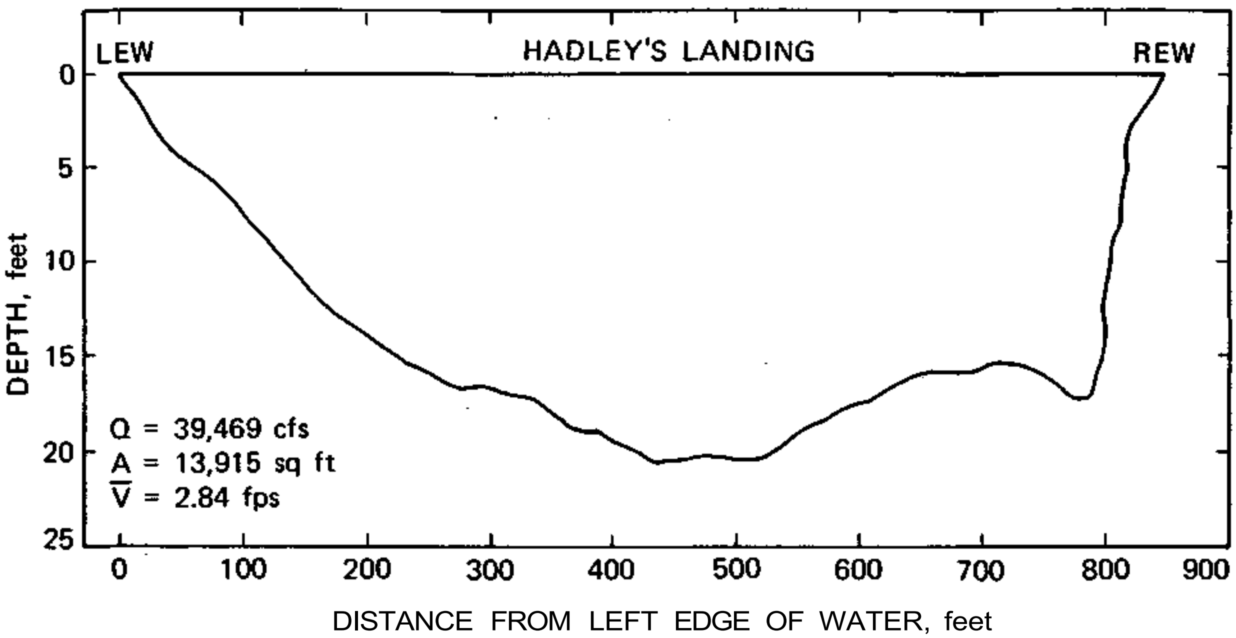


Figure 5. Cross-sectional profile of the Hadley's Landing test site

Even though the test site is located on a bend, the cross-sectional profile is similar to that found in straight segments of rivers, with the main channel approximately at the middle of the river and not very close to either of the banks.

Access to the site was very easy and there was a State of Illinois boat ramp at the site.

The Rip Rap Landing site, shown in figure 6, is located at river mile 265.1 on the Mississippi River in Pool 25. It is located about three miles south of Belleview in Calhoun County conservation area. The test site is on the east side of the river. On the west side of the river is a narrow side channel, called Slim Chute. Between the side channel and the main river there are several islands of which Slim Island is the largest.

The test site is located on the outside of a gradual bend. The cross-sectional profile at the test site is shown in figure 7. As shown in figure 7, the main channel is on the outside of the bend close to the west bank of the river. River traffic does not, however, slow or maneuver close to the site. Access to the site was easy on land and only a two mile trip by boat from a private boat ramp.

The McEver's Island site, shown in figure 8, is located at river mile 50 on the Illinois River in Pool 26. It is located on the east bank of the river opposite Montezuma. The test site is located on a very gradual bend about 0.4 mile north of McEver's Island. The cross-sectional profile at the test site is shown in figure 9.

Even though the test site is located on a gradual bend, the cross-sectional profile is similar to that found in straight segments of rivers with the main channel at the middle of the river and not close to either of the banks.

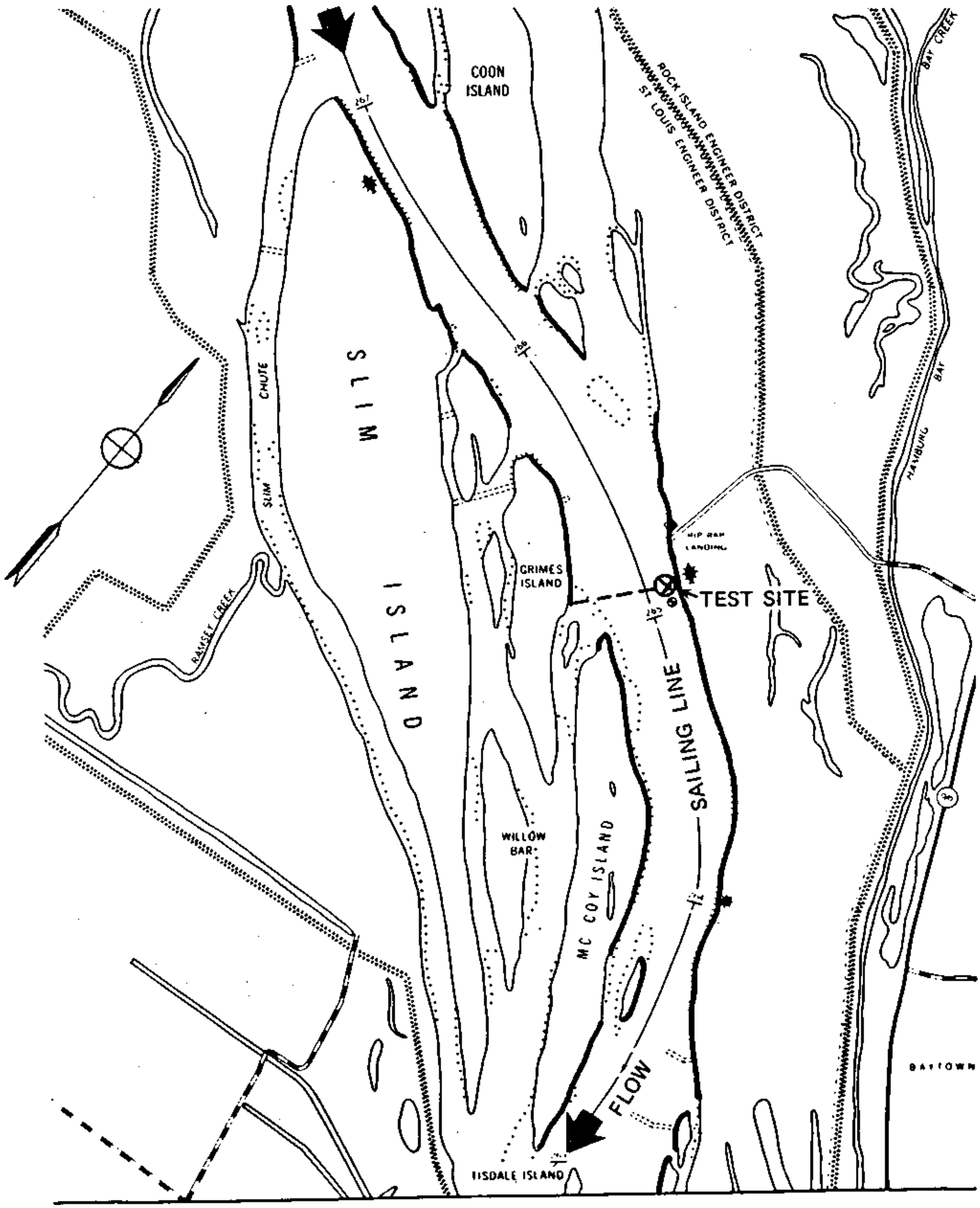


Figure 6. Location of Rip Rap Landing test site on the Mississippi River

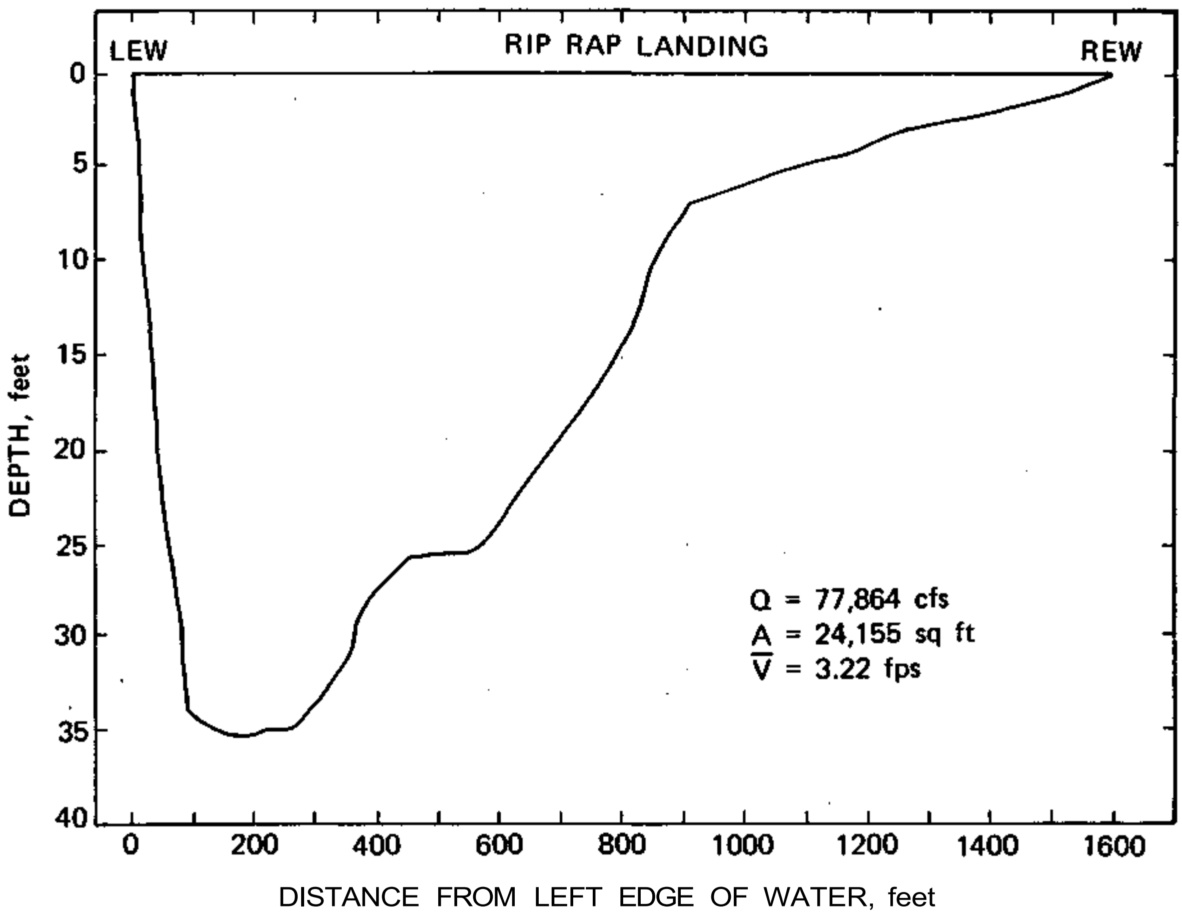


Figure 7. Cross-sectional profile of the Rip Rap Landing test site

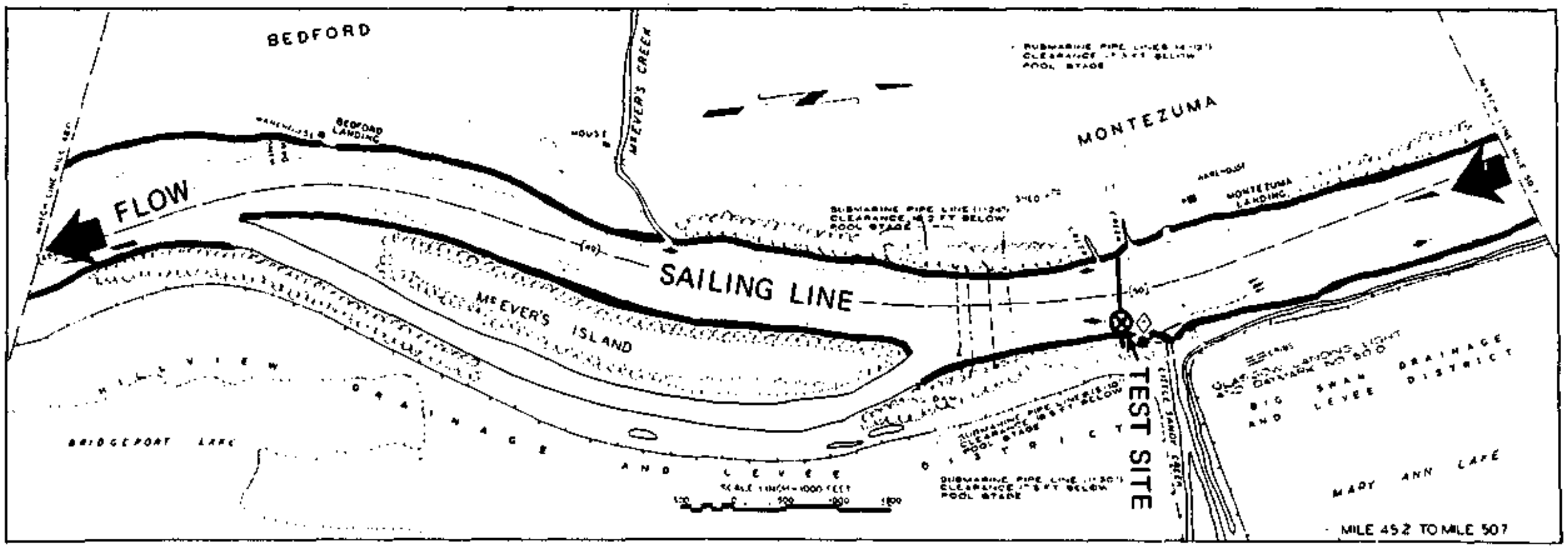


Figure 8. Location of the McEver's Island test site on the Illinois River

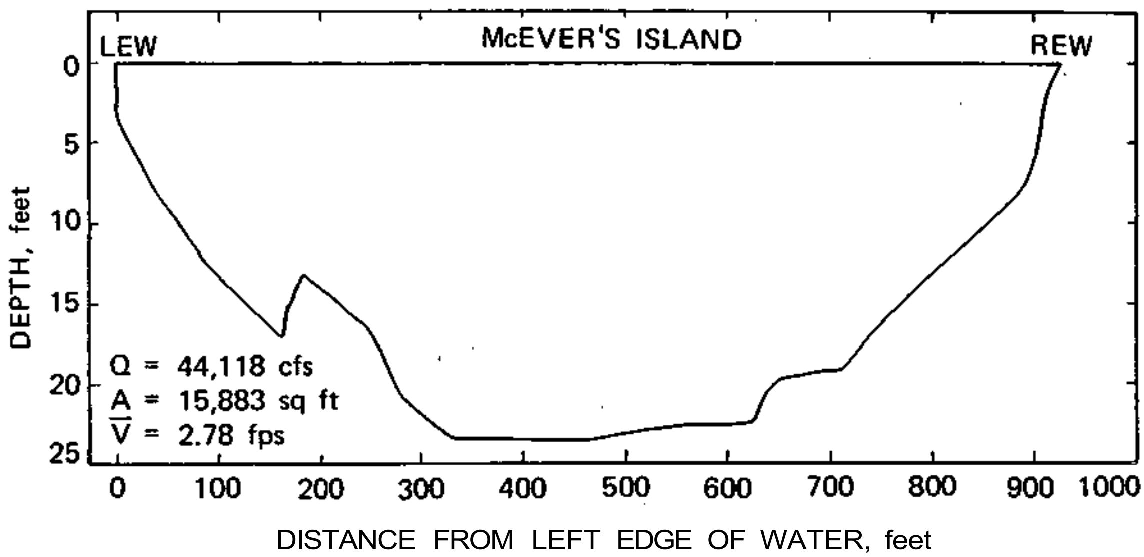


Figure 9. Cross-sectional profile of the McEver's Island test site

River traffic does not slow down or maneuver near the site. Access to the site was good both on land and water.

The Hosier Landing site, shown in figure 10, is located at river mile 260.2 on the Mississippi River in Pool 25. It is situated on the east bank of the river about 1.7 miles north of Hamburg. On the western side of the river there is a side channel called Thomas Chute. Between the side channel and the main channel there are three islands, of which Mosier Island is the largest. The test site is located on the outside and the tail end of a bend around the northern portion of Mosier Island.

The cross-sectional profile at the test site during normal pool level is shown in figure 11. As shown in the figure, the main channel is found on the outside of the bend very close to the east shore of the river.

Access to the site was very good, both on land and water. The site is located just behind a boat dealership with a boat ramp at the site. River traffic does not slow down or maneuver at the site, except when two tows need to pass each other just north of the site.

Instrumentation

The instruments used to collect field data can be grouped into three categories. The first group includes those instruments used to measure wave height and drawdown. Two different systems were utilized to measure wave height and drawdown. The first system was a staff gage and a movie camera, and the second system was an electronic wave gage connected to a mini-computer. A typical wave instrument set-up is shown in figure 12. Shown is he electronic wave gage with the electrical cable, the staff gages to the left, the movie camera in the middle at the water edge, and the van with the computer system.

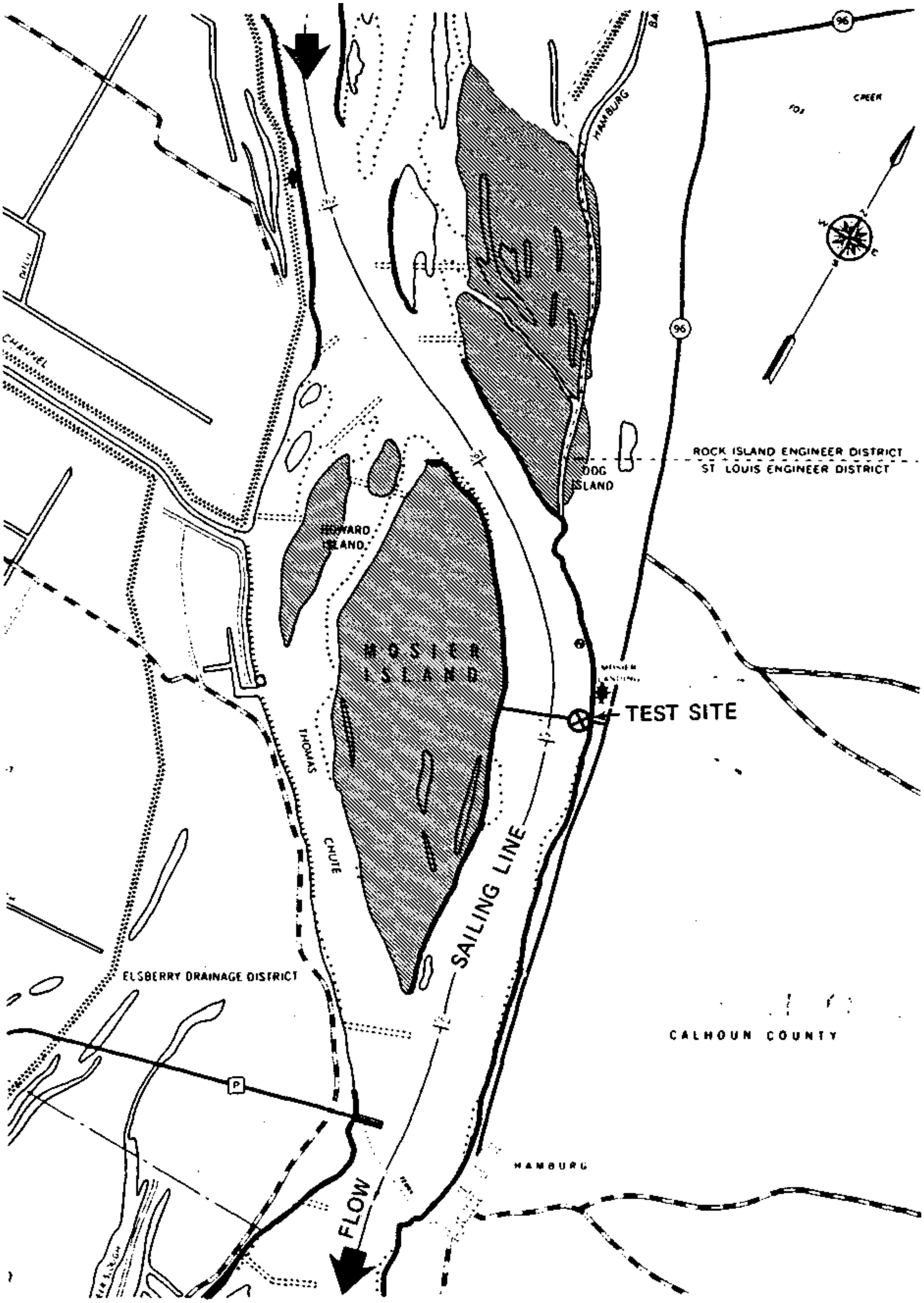


Figure 10. Location of the Mosier Landing test site on the Mississippi River

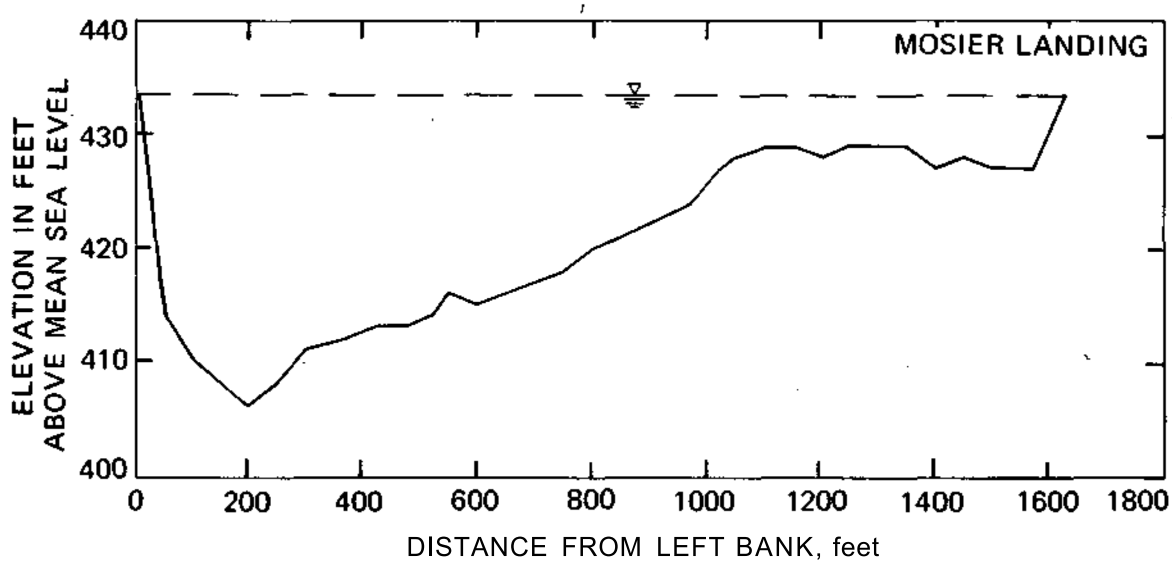


Figure 11. Cross-sectional profile of the Mosier Landing test site

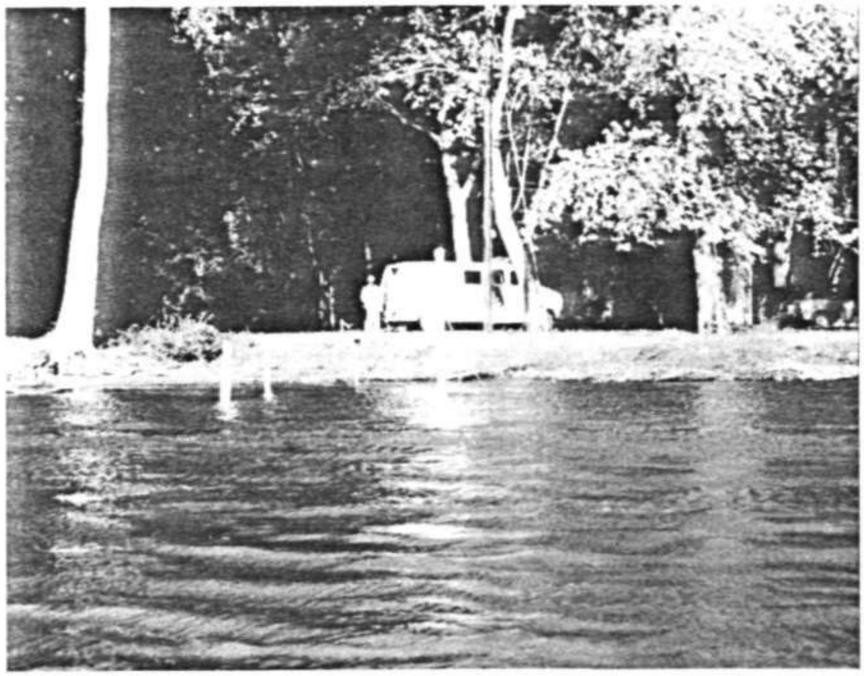


Figure 12. Typical wave instrument set-up during field data collection periods

Figure 13. Installation of staff gage

The second group of instruments were the surveying instruments for measuring vessel speed, track of tow, and distance of tow from the shore. The third group was the wind set to measure wind speed and direction. Each system will be discussed in more detail in the following sections.

Staff Gage and Movie Camera

This system includes a staff gage and a super 8 mm movie camera with a tripod. To install the system, a fence post 10 feet long is first driven into the river bottom about 30 to 50 feet from the edge of the water. Then the staff gage, which is 5 feet long, it bolted on to the fence post as shown in figure 13. It is always attempted to have the still water level at the midpoint of the staff gage. The depth of water where the staff gage was installed varied from 5 to 8 feet. After the staff gage is installed it appears as shown in figure 14.

The movie camera is then positioned on a tripod at the closest possible location with respect to the staff gage. It is very important to position the movie camera so as to reduce reflection from the water surface and also avoid being on the dark side of the staff gage. The camera could be positioned on the river bank or in shallow water, depending on the site characteristics.

The movie camera is then focused on the staff gage and its filter adjusted to minimize reflection from the water surface. The camera is fitted with a remote control to start and stop taking pictures at a convenient location. The camera and the staff gage are shown in figure 15 during an event at Hadley's Landing on the Illinois River.

The speed of the movie camera is 18 frames per second. Therefore, it was possible to obtain 18 readings per second during an event. This

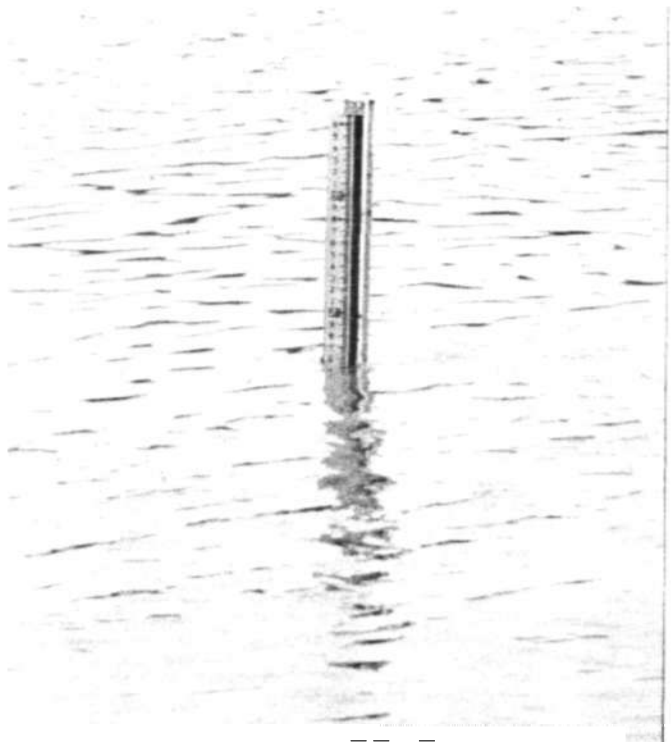


Figure 14. Staff gage in calm water

Figure 15. Movie camera and staff gage during data collection period

provided more than enough data points to construct the wave profiles. One film cartridge lasts for about 3 minutes. For some events, one cartridge was sufficient, for others more than one was required.

Electronic Wave Gage and Mini-Computer

During the progress of the project, it was soon realized that the time required to analyze the wave data from the films was an extremely lengthy and cumbersome process. It was then decided to investigate a more efficient technique of collecting the field data.

A new electronic system was built at the State Water Survey and tested in the field. The system includes electronic wave gages with exposed contact points 0.05 feet apart and a mini-computer to control data collection and store the data on cassette tapes.

The wave gage consists of a PVC pipe case, 3 feet sensor grid, and electronics package (figure 16). The case is divided into two main areas, the case protecting the sensor grid and the one protecting the electronics. The case protecting the sensor grid is a 60 inch length of 2-inch PVC pipe with a 42 inch long by 1/2 inch wide slot cut starting 12 inches down from the top of the pipe to 6 inches from the bottom. The sensor grid protrudes through this slot to monitor the waves. A 2-inch PVC cap is connected to the bottom of the section of pipe. Cemented on top is the electronics case, which consists of PVC fitting to expand to a diameter of 4 inches. This section is one foot tall and is split approximately in half by a threaded section to gain access to the electronics. In the lower half section is a 35 pin connector that is used to interconnect the gage with the interface on shore. The wires from the sensor grid case to the electronics package compartment and from that

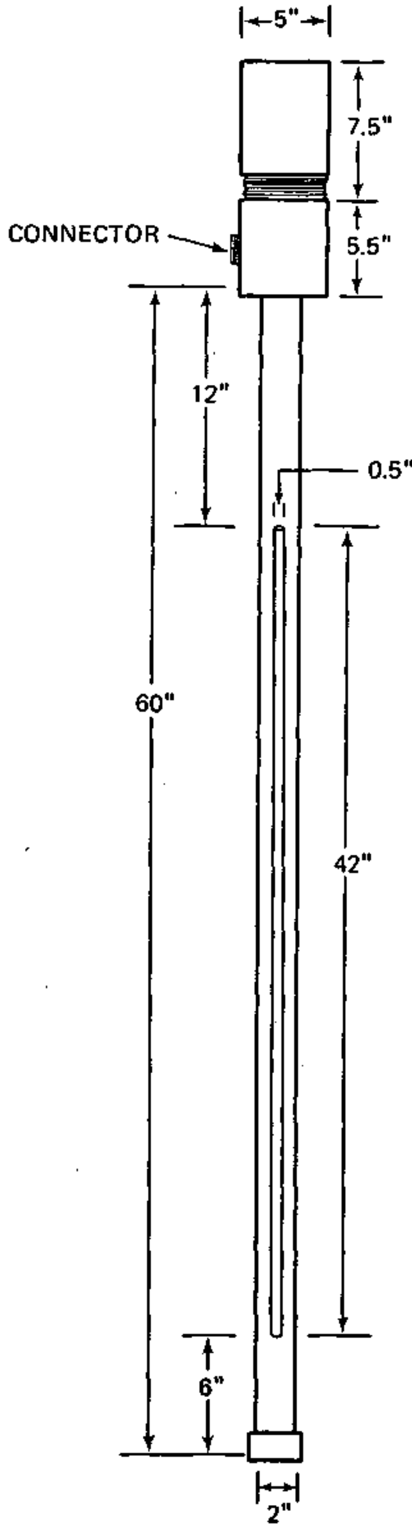


Figure 16. Electronic wave gage

compartment to the connector are sealed against moisture entering with silicon caulking compound.

The sensor grid consists of 3 one-foot long by 2-1/4 inches wide single-sided 1/16 inch thick copper-clad board. These boards have been etched to form a pattern of fingers spaced 1/20 of a foot apart (figure 17). The fingers are gold plated to maintain a good electrical contact with water. Twenty conductor ribbon cables are soldered to pads at the end of the fingers and the contact side of the boards are painted with spar varnish except for approximately 1/16 inch of contact end. This protects the solder joint and makes the contact area small to help accuracy. The 3 boards are then lined up to make a three-foot sensor grid. This grid is then sandwiched between a 1/8-inch x 2-inch x 3-foot and 1/16-inch x 1/2-inch x 3-foot aluminum strips and bolted into a 3-foot piece of 25/64-inch x 1/2-inch x 1/16-inch aluminum channel. This channel is then bolted to the inside of the 2-inch PVC pipe so that the sensor extends through the 1/2-inch slot in the pipe. The aluminum strips and channel form a rigid support for the sensor grid and a secure method of mounting to the PVC pipe. The 1/8-inch and 1/16-inch aluminum strips, as well as the copper-clad board are notched every 6 inches to provide space for the 8-32 pop rivet thread inserts which are used to mount the sensor grid strip to the inside of a PVC pipe.

Atop the sensor grid section of PVC housing is the section that houses the electronics and cable connector. It is constructed out of 4-inch x 3-inch, 3-inch x 2-inch reducers, and 4-inch threaded coupling. This allows the pipe diameter to increase to 4 inches to house the electronics easier. There are two circuit boards, as shown in figure 18, and a 4-inch aluminum disk separating the boards by 2 inches. The disk

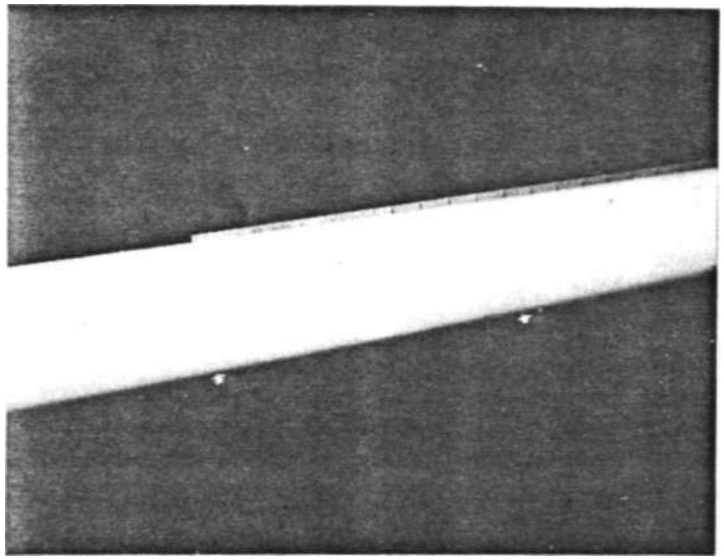


Figure 17. Wave gage sensoring grid

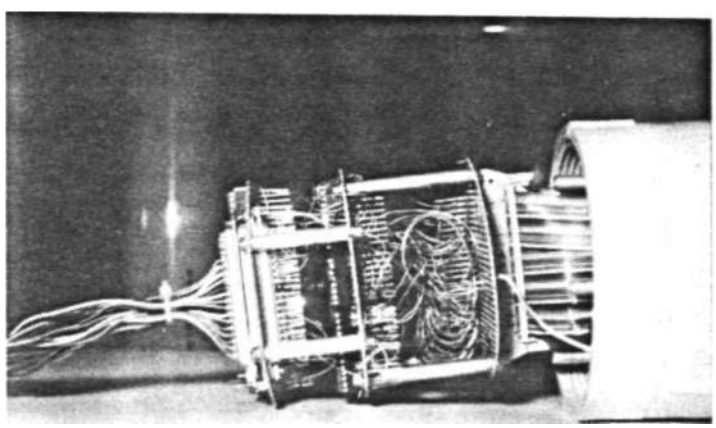


Figure 18. Electronic circuit boards

has a 2-1/4 inch hole in its center to pass cables from the sensor grid and 35 pin connectors to the electronics package. The lower circuit board has all the connectors for these cables and the upper board has the electronic circuit itself. This allows for easy access to and removal of the electronics package if needed. The housing is sealed with a cap of a threaded PVC coupling, a short length of 4-inch PVC and 4-inch PVC cap. Vacuum grease is used to seal the threads against water leakage. The electronics housing is also sealed from the sensor grid housing and connector with silicon rubber compound.

Therefore, using PVC pipe makes for a easily constructed instrument that is lightweight, waterproof, corrosion proof, and strong.

The wave gage receives power and 1 KHZ clocking signal from the wave gage interface via a 100 foot, 15 twisted pair cable. The wave gage using these inputs sequences up the contacts one by one starting at the bottom of the gage. When the gage gets to a contact which is out of the water it stops the sequence and loads that number onto the 8 data lines to the interface every 1/10 of a second. During that loading time it inhibits the computer from getting information until the data lines are stable.

The wave gage interface generated 1 KHZ timing and power to run wave gages and pass data from the wave gage to the computer. The computer sequentially scans the output of the wave gage and loads the wave height information into memory.

Figure 19 shows the electronic gage installed in the river at the McEver's Island test site. The installation of the wave gage requires driving a 10-foot fence post into the river bottom and bolting another piece of 5-foot fence post to extend the height. Two supporting brackets are then bolted to the fence post, one about 2 feet below the water

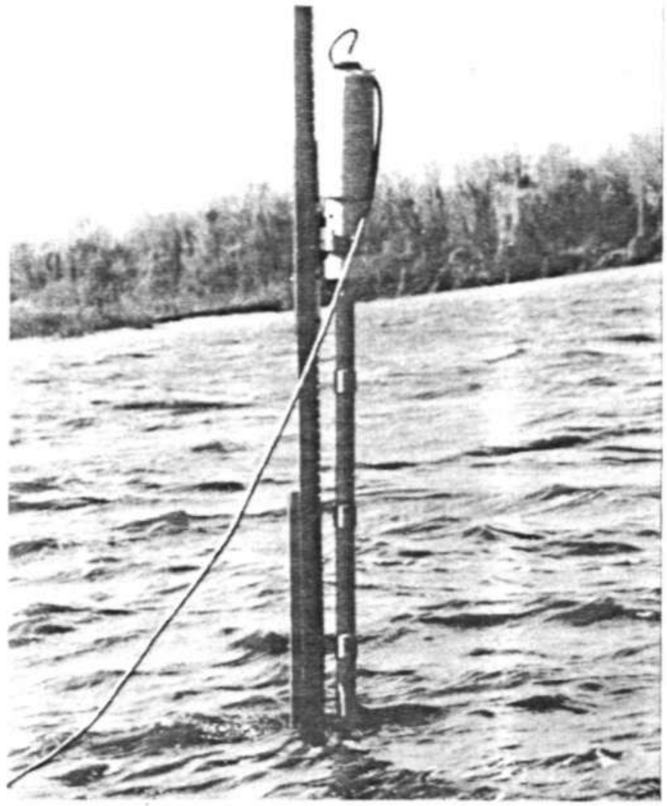


Figure 19. Installed wave gage at a test site

Figure 20. Mini-computer and its peripherals during data collection period

surface and the other one at a variable distance above the water surface depending on the length of the wave gage. The wave gage is then set on the lower supporting bracket and fastened to the upper bracket with a hose clamp. The electrical cable from the wave gage is then carefully laid on the river bottom in the direction of the computer where it is plugged to the IEEE interface.

The mini-computer is a CBM computer Model 80032 with 32K memory. Peripherals include two cassette drives, a printer, a modem, and an IEEE interface between the computer and the wave gages. The computer, the cassette drive, and the IEEE interface are shown in figure 20 as they were being used in the field. The whole system sits in the back of a station wagon or a van. The printer is just behind the computer. The IEEE interface transforms the electrical output of the wave gages into binary signals which can be read by the computer.

The power supply to the computer and the wave gages comes either from a small portable generator or from some private power lines as shown in figure 21.

The modem is used to interface the mini-computer with a CYBER computer for data transfer and analysis.

A schematic diagram of the wave and drawdown data collection system is shown in figure 22. Water level readings from the wave gages go to the interface, which transforms the data into readable form for the computer. The computer then reads the data and stores it in memory. At the end of an event, the data is stored onto cassette tapes. The data can also be printed on paper for inspection. Later on the mini-computer sends the data to the CYBER computer through the modem and phone lines for further analysis.

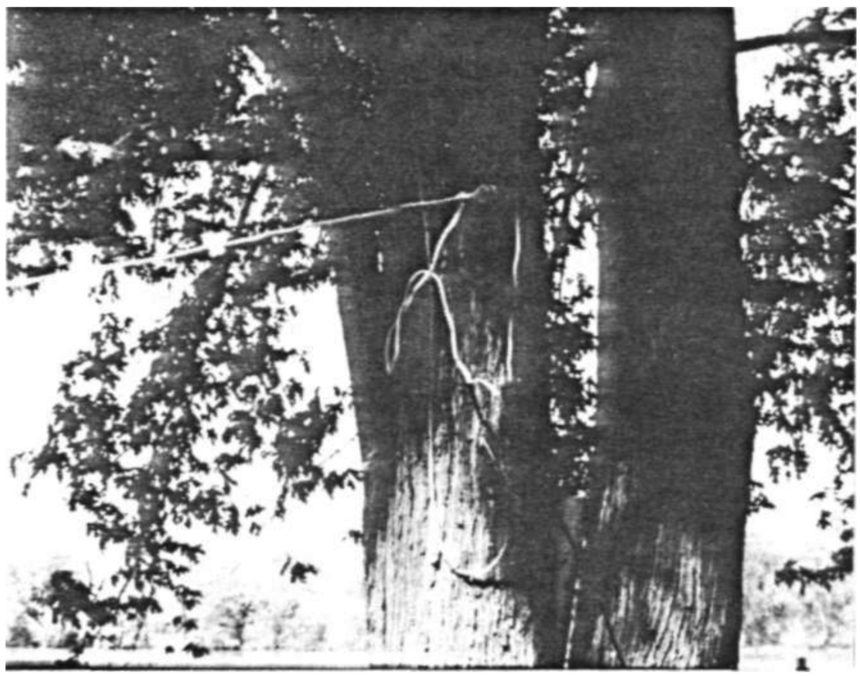


Figure 21. Electrical power supply in the field

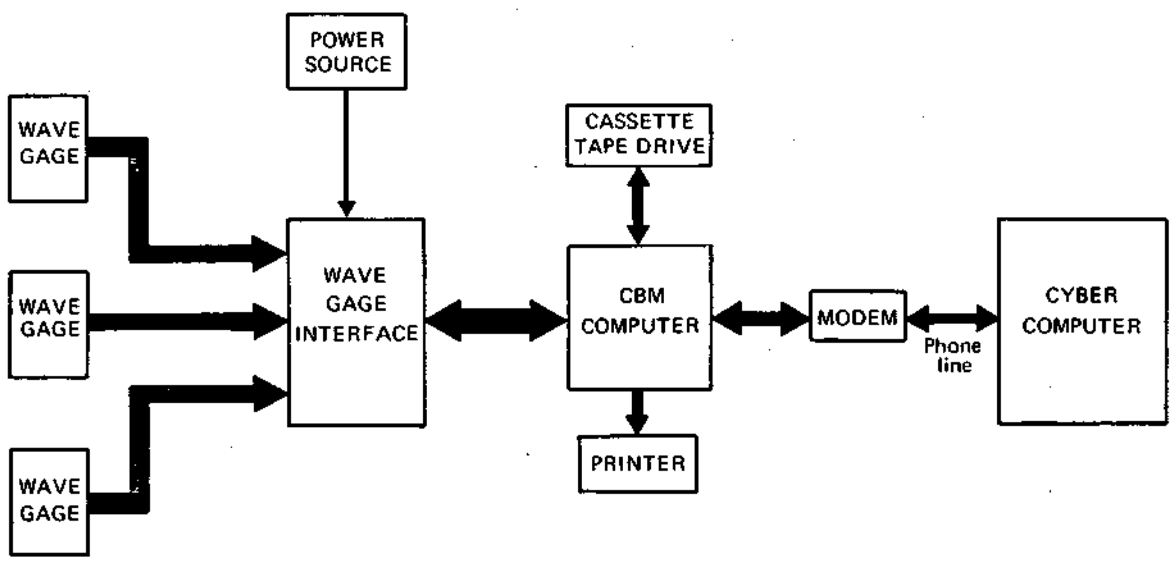


Figure 22. Skematics of the wave and drawdown measuring instruments

Wind Set

During each field trip, a wind set, shown in figure 23, was installed at the test site in an area clear of trees and other obstructions. The wind set consists of the wind speed measuring bucket-type propeller, and the wind direction measuring arrow on top of a 25-foot high pole. The pole is supported on a stable heavy base and four guy wires. The electrical wires from the propeller and arrows run down the pole to a twelve volt battery, which supplies the power, and a chart recorder, which records the speed and direction of the wind.

The wind set is turned on each morning and runs continuously during the day until the last event of the day.

Surveying Instruments

The surveying instruments used during the field work include two Lietz TM-10C precision theodolites, range finders, timing watches, and measuring tapes. The two theodolites were used primarily for the tracking of the sailing line of the tows within the test site, as will be discussed in the following section. The theodolites were also used for the site survey to define the shore line position and the location of all data gathering instruments.

The range finders were used in place of the theodolites for determining the distance of vessels from the shore line. The timing watches were used to determine the time taken by a vessel to travel a known distance. The tape measures were used to establish baselines on shore.

Data Collection Procedure

A general picture of the field conditions and the data that need to be collected are illustrated in figure 24. As a tow passes a test

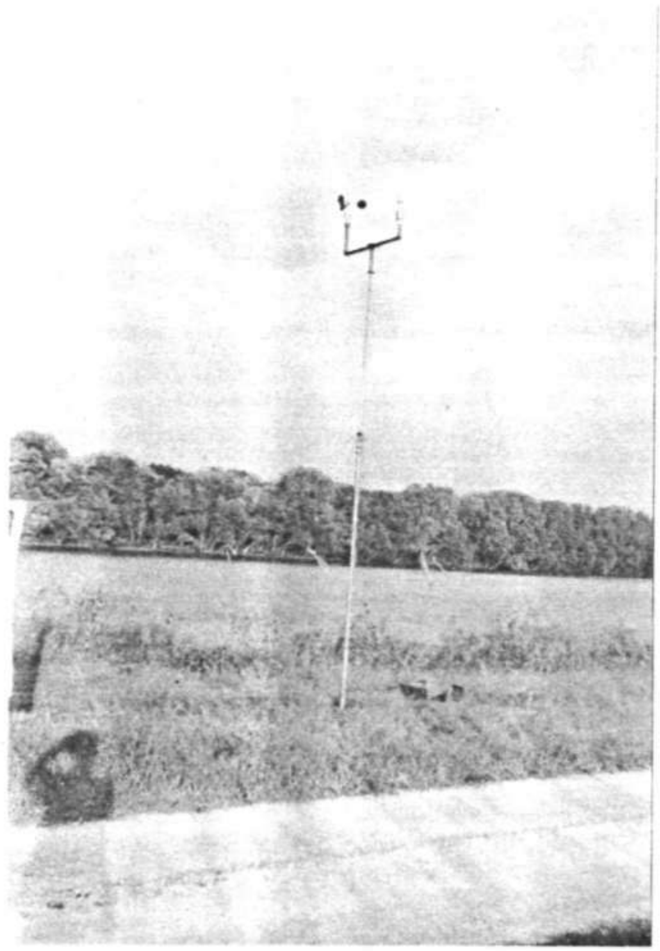
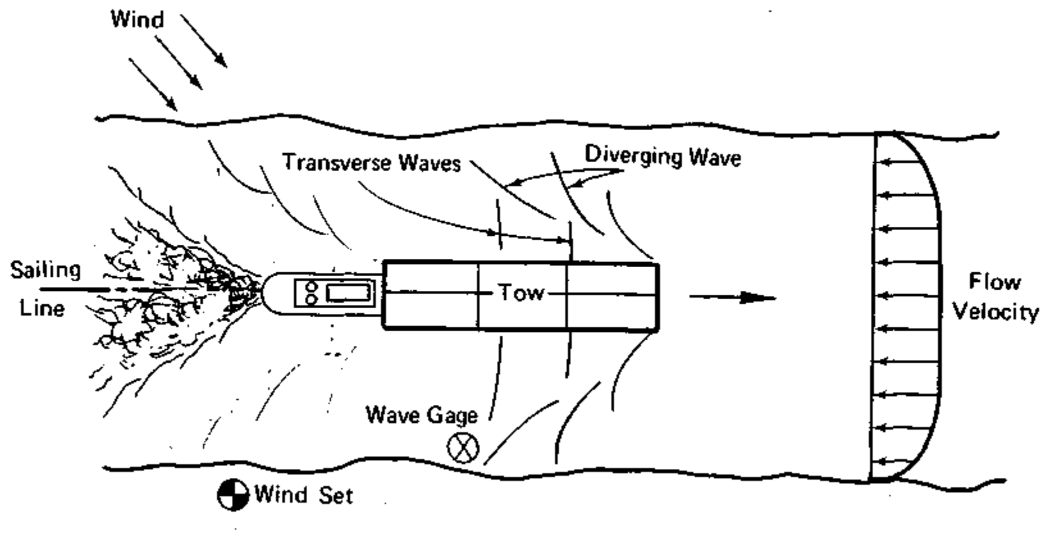
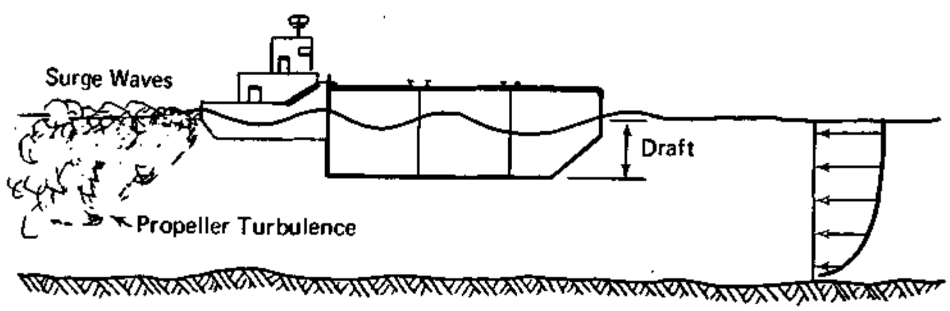


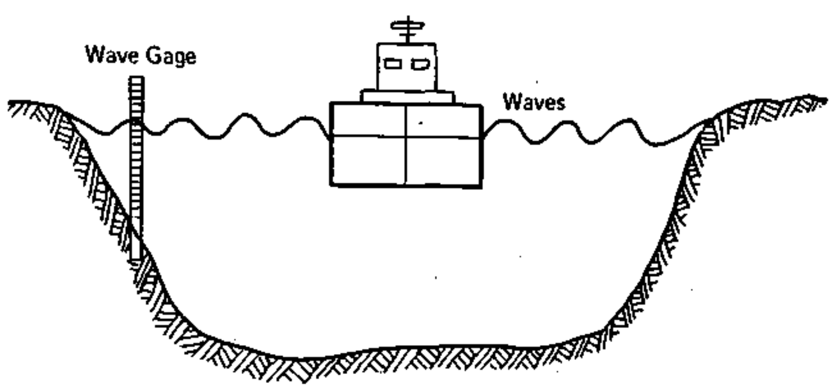
Figure 23. Wind speed and direction measuring instrument at a test site



PLAN



SECTION



CROSS SECTION

Figure 24. Illustration of field conditions during a tow passage

section, it generates waves and depresses the water level by changing the natural flow field in the river. The propeller of the towboat also generates great turbulence behind it. At the same time or some other time, strong winds could blow over the water surface and generate a different kind of wave.

For this project it was necessary to collect data about the channel and flow conditions without the presence of a tow, to establish the undisturbed conditions, and to determine some important parameters needed for analysis of wave and drawdown data. The wave and drawdown data are recorded as a tow passes the test site by utilizing the appropriate instruments; and at the same time, information about the tow such as speed, draft, number of barges, towboat name, and distance from shore are collected.

A detailed discussion of the data collection procedure is presented in the following sections.

Channel and Flow Parameters

Channel geometry and flow measurements were taken at Hadley's Landing, Rip Rap Landing, and McEver's Island during or shortly after the field trips to the sites. The measurements were not performed at Mosier Landing because of high flow conditions and rapidly changing water elevation. However, Mosier Landing was only 5 miles downstream of Rip Rap Landing, and Mosier Landing's cross-sectional profile is similar to that of Rip Rap Landing as shown in figures 7 and 11. Therefore, it can be assumed that the flow conditions at both sites are also similar without much error involved. The channel profile for Mosier Landing was taken from the U.S. Corps of Engineers hydrographic survey maps of the Mississippi River, 1971.

The cross section and velocity data were taken according to the procedure described by Buchanan and Somers (1969) for stream gaging. The instrument used was a standard Price-type current meter with a 30 lb. fish (figure 25) suspended on cable from a crane with a winch. The crane was then mounted on a work boat, which was used for measuring velocity and discharge data.

The boat was positioned at different distances from the shore along the cross section and anchored to the river bottom to hold position while measuring depth and velocity. The distance from the shore to the boat was determined by a transit on shore.

Velocities were measured at 0.2 and 0.8 depths at each vertical. The average velocity at the vertical is then determined by dividing the sum of the two readings. The cross-sectional profiles for the test sites at Hadley's Landing, Rip Rap Landing, and McEver's Island are shown in figures 5, 7, and 9, respectively. Velocity and depth measurements were taken at 17, 18, and 16 verticals for Hadley's Landing, Rip Rap Landing, and McEver's Island, respectively. The velocity and depth data along with the discharge computations are given in tables 3, 4, and 5 for Hadley's Landing, Rip Rap Landing, and McEver's Island, respectively.

As shown in table 3 for Hadley's Landing test site, the discharge, Q, the cross-sectional area, A, and the average velocity, V, were 39,469 cfs, 13,915 square feet, and 2.84 ft/sec, respectively. Also from figure 5, it can be determined that the top width of the channel was 845 feet. By dividing the cross-sectional area by the top width, one can determine the average depth, which is 16.5 feet.

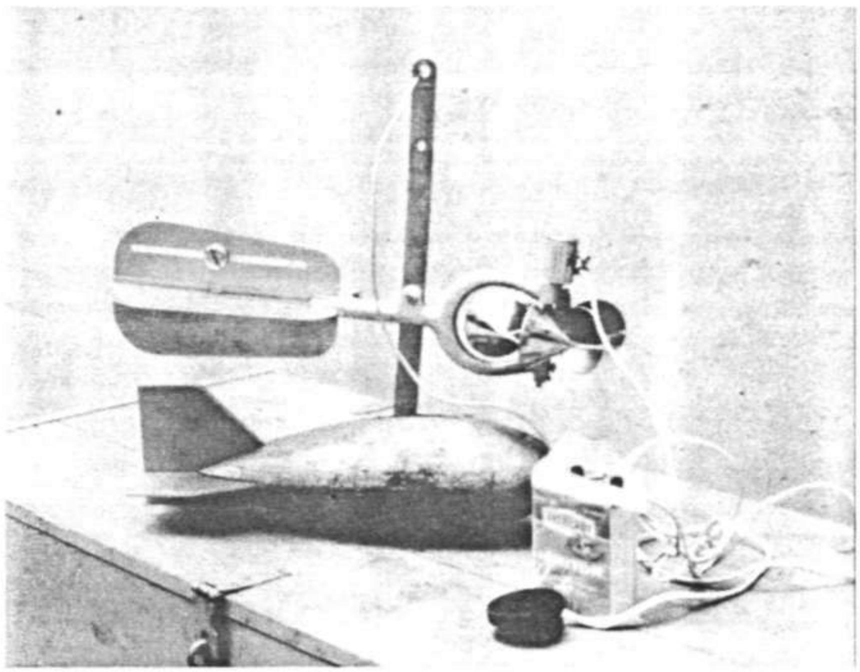


Figure 25. Velocity measuring instrument

Table 3. Velocity and Discharge Measurement at Hadley's Landing, Illinois River, River Mile 13.2

Date of data collection: 5/7/81

Measured discharge: 39469 cfs

Cross-sectional area: 13915 sq ft

Average velocity: 2.84 ft/sec

Distance from left edge looking downstream (ft)	Depth of water (ft)	V(.2) (ft/sec)	V(.8) ft/sec)	Ave. velocity in the vertical (ft/sec)	q (cfs)
0	0.0		-	-	-
45	4.96	1.21	1.46	1.34	244
85	7.27	2.08	1.57	1.88	820
155	13.25	2.72	2.28	2.49	2496
240	17.79	2.93	2.38	2.65	3270
300	19.15	3.26	2.48	2.87	2756
340	20.74	3.11	2.59	2.85	1902
365	21.46	3.32	2.54	2.93	1567
390	22.03	3.39	2.59	2.99	2343
435	23.96	3.46	2.91	3.19	3163
475	23.51	3.54	2.91	3.23	3613
530	23.39	3.45	2.97	3.22	3849
580	20.85	3.43	2.59	3.01	2876
620	19.83	3.26	2.79	3.02	2533
665	18.32	3.43	2.72	3.07	3132
730	17.64	2.97	2.79	2.89	3181
785	20.06	2.72	1.95	2.33	1542
815	4.47	.60	1.06	.83	182
845	0.0	-	-	-	-
					Q=39469

V(.2) = velocity at 0.2 of total depth from the surface
V(.8) = velocity at 0.8 of total depth from the surface
q = partial discharge

Table 4. Velocity and Discharge Measurement at Rip Rap Landing, Mississippi River, River Mile 265.1

Date of data collection: 4/10/81

Measured discharge: 77864 cfs

Cross-sectional area: 24155 sq ft

Average velocity: 3.22 ft/sec

Distance from left edge looking downstream (ft)	Depth of water (ft)	V(.2) (ft/sec)	V(.8) ft/sec)	Ave. velocity in the vertical (ft/sec)	q (cfs)
0	0.0	-	-	-	-
62	25.95	3.89	3.34	3.62	4089
105	34.16	3.89	2.97	3.43	6271
175	35.15	3.70	2.53	3.12	6138
218	34.82	3.79	2.47	3.13	4752
262	34.82	3.70	2.65	3.18	5056
312	32.85	3.89	3.34	3.62	6340
350	31.21	3.79	3.18	3.49	3034
368	28.91	3.89	3.03	3.46	4958
450	25.62	3.89	3.18	3.54	9192
565	25.29	3.89	3.34	3.62	9459
665	21.02	3.10	3.18	3.14	8055
820	13.14	2.78	2.23	2.51	4084
905	6.93	2.97	2.65	2.81	1967
990	5.95	2.90	2.27	2.59	1313
1075	5.03	2.37	2.17	2.27	990
1165	3.94	2.59	2.12	2.36	862
1260	2.89	2.32	1.82	2.07	813
1440	1.71	1.98	1.72	1.85	487
1595	0.0	-	-	-	-
					Q=77864

V(.2) = velocity at 0.2 of total depth from the surface
V(.8) = velocity at 0.8 of total depth from the surface
q = partial discharge

Table 5. Velocity and Discharge Measurement at McEver's Island, Illinois River, River Mile 50

Date of data collection: 5/6/81

Measured discharge: 44118 cfs

Cross-sectional area: 15883 sq ft

Average velocity: 2.78 ft/sec

Distance from left edge looking downstream (ft)	Depth of water (ft)	V(.2) (ft/sec)	V(.8) ft/sec)	Ave. velocity in the vertical (ft/sec)	q (cfs)
0	0.0	-	-	-	-
40	8.18	1.35	1.35	1.35	606
97	13.30	2.62	1.96	2.29	1784
160	17.18	3.06	2.23	2.65	1681
180	13.50	3.06	2.62	2.84	1726
240	16.52	3.23	2.73	2.98	2663
285	21.52	3.23	2.62	2.93	2738
330	23.65	3.29	2.46	2.87	2987
375	23.65	3.25	2.68	2.97	4386
455	23.72	3.34	2.51	2.93	4489
505	23.32	3.08	2.80	2.94	3761
575	22.73	3.06	2.95	3.01	3773
615	22.44	3.06	2.85	2.96	2265
645	19.87	3.14	2.29	2.72	3609
705	19.31	2.97	2.46	2.72	2761
755	15.44	3.08	2.73	2.90	3610
875	8.31	2.13	1.87	1.99	1279
915	0.0	-	-	-	-
					Q=44118

V(.2) = velocity at 0.2 of total depth from the surface
V(.8) = velocity at 0.8 of total depth from the surface
q = partial discharge

Similarly from table 4, the discharge, cross-sectional area, and average velocity for the Rip Rap Landing test site were 77,864 cfs, 24,155 square feet, and 3.22 ft/sec, respectively. The top width of the channel from figure 7 is 1595 feet. The average depth is therefore 15.1 feet.

From table 5, the discharge, cross-sectional area, and average velocity for the McEver's Island test site were 44,118 cfs, 15,883 square feet, and 2.78 ft/sec, respectively. From figure 9, the top width of the channel is 915 feet. The average depth is then 17.4 feet.

Vessel Parameters

During each event all the pertinent information about the river traffic was collected on the data sheet shown as figure 26. Information such as the vessel type, size, draft, distance from shore, and direction of movement were recorded during the event. The names of the vessels, especially those of the tows, were also recorded. The tow name can be used to check the tow characteristics such as size, engine power, and propeller type from Inland River Record (1981).

The vessel types encountered during the field trips were a tow pushing barges, a tow without barges, and pleasure crafts such as cabin cruisers and small speed boats. Most of the barges were jumbo barges, but there were also some standard size and petroleum barges. The sizes of the jumbo, standard, and petroleum barges were 195 feet x 35 feet, 175 feet x 26 feet, and from 150-300 feet x 50 feet, respectively. The tow sizes ranged from 85 feet by 26 feet for the smallest tow to 180 feet by 52 feet for the largest. The size, number of screw, power, and nozzle type for most of the tows sited during all the field trips is summarized in table 6.

WAVE & BARGE
DATA

With
Lateral Movement
Side Channel

Site # McEver's Island
Trip # 4

Date 5/1/81
Page # 10 of 10
Recorder MD

General Information	Barge Information
Event # 10 up/down stream	Tow name & ID Normania
Time - Start 9:48 A.M.	
Finish Film roll #11	
Stage:	Draft 9.0'
Depth at Wave Gage:	Width 105'
Distance of gage from shore: 15 ft	Length 585'
	Displacement
Air temperature:	Velocity 12.8 fps
Remarks:	Distance from shore 175 ft
	Track (attachment)
	RPM
	# of barges 9 (3x3)
Wind data (Mark recorder chart)	Remarks:

Time	Speed	Direction

Figure 26. Sample data sheet

Table 6. Tow Characteristics

Name of tow boat	No. of screw	Size (ft)	Power (HP)	Type of nozzle
Andrew Benedict	2	114x35	4100	Kort
Arthur E. Snider	2	152x34	3200	
Atlas	3	70x26	1275	
Barbara Jeanne Meyer	2	145x27	2200	
Betty Brent	2	135x32	3000	
Chicago Trader	2	90x32.1	1530	
Clark Frame	2	111.8x35	3200	
Colonel George Lambert	2	140x42	4200	Kort
Conti Afton	2	140x44	4200	
Craig M.	2	148x34.7	2400	Kort
Creole Belle	2	130x37	3900	
Fort Pierre	2	135x32	2800	
Frederick B. Wells	2	140x38	3800	
Gordon Jones	2	147x38.5	4200	
Hawkeye	2	150x35	4300	
Herb Schreiner	2	85x26	1700	
Irene Chotin	2	148x34.5	3200	Kort
Joanne				
John M. Warner	2	103x30.8	1800	
Kathy Ellen	2	150.8x34.8	3800	
Leviticus	2	147x37.9	4200	
Lillian Clark	2	180x52	6450	Kort
Luke Gladders	2	150x35	3200	
Lynn B.	2	148x34.7	2400	Kort
Magnolia	2	116x45	3800	
Marvin E. Norman	2	102x34	1800	
Mr. Joey	2	145x48	5600	Kort
New Dawn	2	140x42	5600	Kort
Nohab Express	2	91x30	3000	
Patsy Swank	2	141x38.5	3500	
Prairie Dawn	2	160x40	5000	Kort
Robin Mott	3	148x45	4800	
Rose Marie Walden	2	90x32	2400	
Sally Barton	2	116x27.5	2400	
Sierra Dawn	2	164x40	5000	Kort
Virginia E. Towey	3	140x45	5850	
White Dawn	2	156x35	3200	
White Knight	2	150x33.6	3200	
Yankton	2	125x28	2200	

The draft of barges was mostly standard in that loaded barges had 9 feet of draft and unloaded barges had 2 feet of draft. The draft was read on the draft indicators on the sides of the barges. There were some cases where tows were pushing a combination of unloaded and loaded barges. These instances were recorded on the data sheet in the field and the draft noted at different locations.

The distance of vessels from the shore and their speed were determined by two methods. The first method required two surveyors in addition to the person who operates the computer, while the second method required only one. The surveying method was more elaborate. In addition to the speed and distance from shore, it provided the track of the vessel within the test site. The surveying procedure to determine the track, distance from shore, and speed of the vessel was as follows.

A standard bearing intersection system of survey was used to determine the track and distance from shore of the vessel. A baseline of sufficient length, usually eight hundred to fifteen hundred feet, was established on one shore adjacent to the test site. A semi-permanent marker was set at each end of this line. One of these was then referred to as the primary survey station, and the other was termed the secondary survey station as shown in figure 27a.

It was desirable but not critical to have the baseline situated to allow a clear line-of-sight from end to end. Locations were selected which provided the greatest unobstructed view of the test site and channel approaches. A one-half to one-mile section of the river could be viewed with little difficulty in most cases. This enabled the surveying crews to measure tracks which extended at least one thousand feet above or below the test sites.

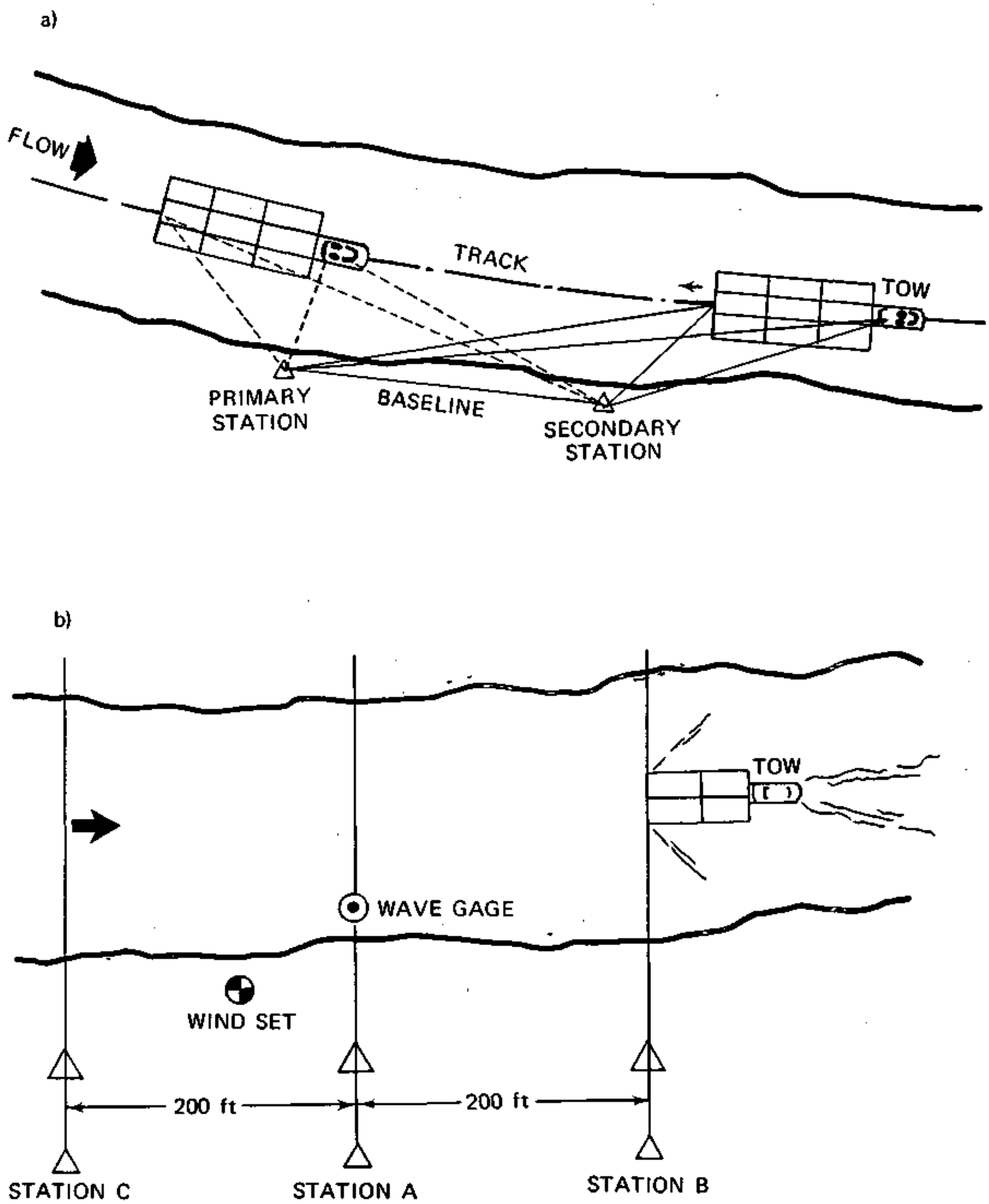


Figure 27. Surveying procedures

After establishing the baseline location, the next task was to perform a site survey. The site survey was required to define the shape and position of the shore lines adjacent to the test site and the locations of all data-gathering instruments. The precise distance between the two survey stations (the length of the baseline) was measured electronically, and routine land-survey procedures were employed to produce a site configuration base map. This map was reproduced in quantity and subsequently used to plot the track data for each tow, providing a visual representation of events as they actually occurred in the field.

Given this basic set-up, the determination of track was accomplished relatively easily. For example, a tow was observed entering the approach to one end of the test site as shown in figure 27a. Tracking operations began as soon as the entire length of the vessel was observable from both survey stations.

Each theodolite was zeroed on the opposite station, and this provided the index for all angular measurements. Horizontal angles were measured simultaneously from each station to a previously agreed-upon point on the tow, usually the centerline foresight mast which was present on the bow of the leading barge of most tows. These angles were measured to the nearest one minute of arc and recorded. The procedure was repeated for the stern of the tug, where the sighting point was usually the radar mast or the flag-staff.

These angular measurements followed each other as rapidly as possible and were always taken in pairs: In other words, an angle to the bow from each station was measured at the same instant, and an angle to the stern from each station was measured at the same instant, forming a set of angles consisting of two pairs.

To make recording the angles easier and faster, a pocket-size, battery-powered tape recorder was utilized at each survey station. A running account of the tracking operation and the angular data was recorded and later transcribed.

This process continued at regular intervals until the tow passed from the observed area. At first glance this procedure seems cumbersome, but in actuality each pair of angles could be measured and recorded in about thirty seconds. Usually an interval of about one minute was left between sets of angles. The coordination between survey stations was maintained by continuous radio communication, and all actions were initiated and directed from the primary station.

A graphic depiction of each track was developed by plotting the point of intersection of each pair of angles on the base maps and connecting the resulting points. Having measured angles to both bow and stern, it was possible to show differences in the tracks of each end of the tows as shown in figure 28. It was also possible to determine the distance of the tow from the shore line.

The speed of the tows was determined by timing the tracks. At the precise instant of measuring the first angle, bow or stern as convenient, a stopwatch was started. The watch was stopped at the instant of the appropriate last angle measurement. The resulting elapsed time was compared with the track length to obtain the average velocity. This calculation was usually performed at the same time that the tracks were plotted. Velocities thus determined were relative to the shore line. It may be desirable to adjust them for the effects of current. The speed of

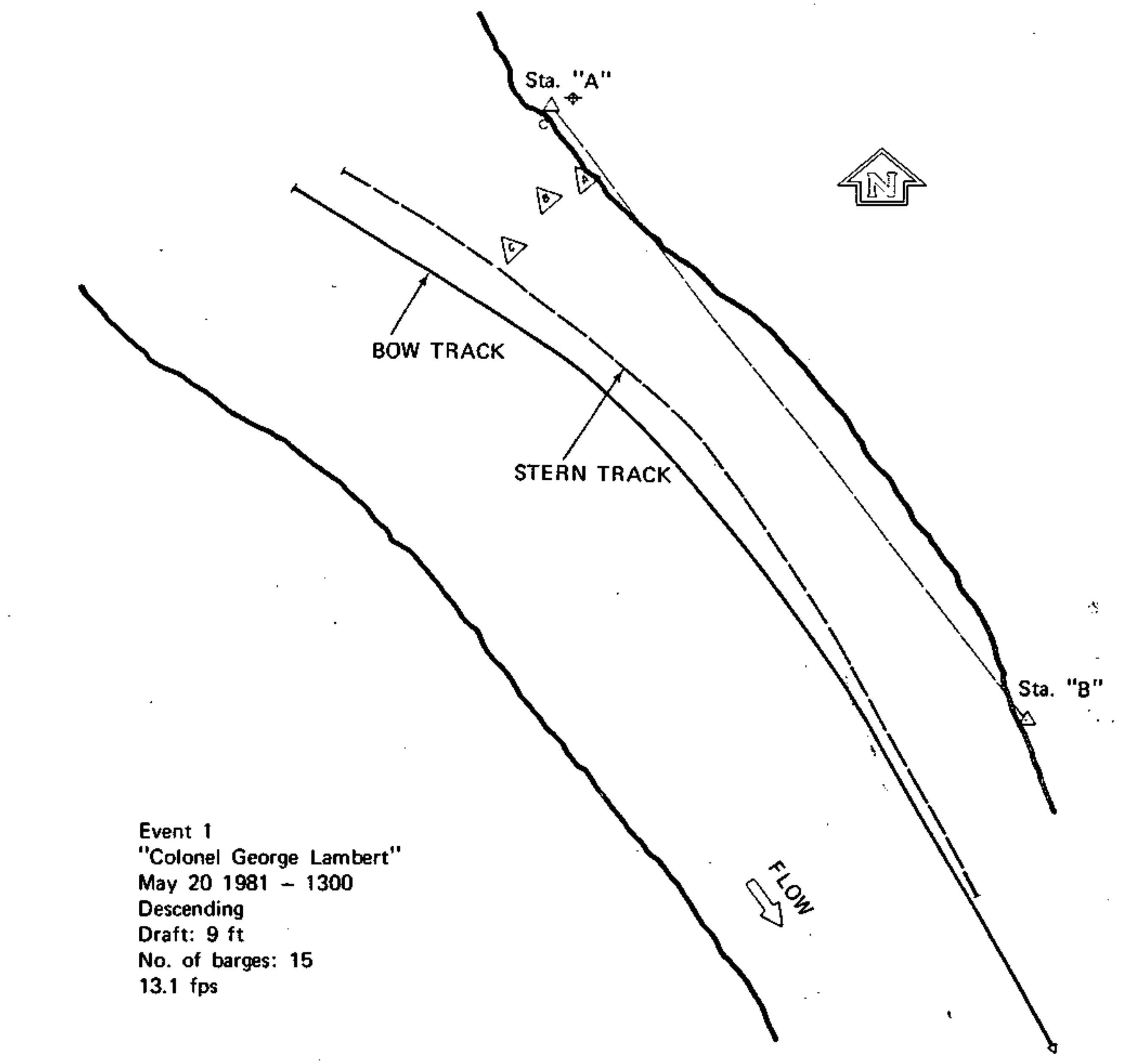


Figure 28. Tow tracking data at Mosier Landing test site

the tows was actually a by-product of the bearing intersection procedure—another good reason why that system was adopted.

The second technique of measuring the speed and distance from shore to the vessel was relatively simple. First a baseline, with three stations, as shown in figure 27b, is established close to the shore line. The stations are located from 150 to 300 feet apart. About 10 to 20 feet behind the baseline, a second line is established. Three other stations are then established on the second line, such that they form two rectangles. Six posts with flags are driven into the ground to mark the stations. By standing behind the second post, one can see the passage of the bow or stern of a vessel across the line of sight.

Station A is located closest to the wave gage. The person who operates the wave instruments, such as the computer and movie camera, is positioned at station A. Another person is positioned either at station B or C, depending on the direction of the vessel. If the vessel is moving upstream as shown in figure 27b, then the second person will be positioned at station C. If the vessel is moving downstream, he will be positioned at station B. The person at station A has a whistle which he blows when a specified part of the vessel (the bow or stern) crosses station A. The person at station B (or station C) starts his stopwatch at the sound of the whistle and stops it as the same part (bow or stern) of the vessel crosses his section. Since the distance between stations is known, the speed is calculated by dividing the distance between the stations by the time it took to travel that distance.

The distance from shore is determined by using a range finder (model 600) manufactured' by Ranging Measuring Systems, Inc. The range finder can read distance from 50 to 600 feet with an accuracy of 96.7 to 99.5 percent.

Wave and Drawdown

The wave and drawdown data were collected using the movie camera and the staff gage, and the electronic wave gage with the computer. Movies of the water surface elevation at the staff gage were taken while tows were passing the section for drawdown, and shortly afterwards for the waves.

The movie camera was run for varying lengths of time, depending on the nature of the event. Some events required more than one cartridge of film, while other did not. The movie films were then developed, and with the aid of a movie editor and movie projector, the water elevations were read frame by frame. One movie cartridge runs for about three minutes at a speed of 18 frames per second, providing approximately 6,480 frames of water level readings. After the water elevations are read from the films, they are entered to the CYBER computer at the University of Illinois and stored in data files for further analysis.

The wave and drawdown collection procedure using the electronic wave gage and the computer is relatively easier than the movie method. About 5 minutes before an event is to commence, the computer is turned on and a master computer program read into the computer from a cassette tape. The program is then stored in the computer's memory space. When the program is run, it starts asking for two lines of title description such as trip and event number, date, time, and site description. After reading in the title lines, it is ready to sample. By pressing "G" on the keyboard, the system starts to collect water elevation data every second. This sampling frequency is kept until the front edge of the waves reaches the wave gages. Once the waves reach the wave gages, the sampling frequency is changed to every one-tenth of a second by pushing the letter "W" on the keyboard. The computer program also has a means of keeping the time the bow and stern of the vessel passed the test cross section. By pushing the letters "B" and "S" for the bow and stern, respectively, the time the bow and stern passed the cross section is kept in the data file. By knowing the length of the vessel, an approximate check on the speed of the vessel is possible. It also helps to keep track of the sequence of water level fluctuations with respect to the vessel's position along the channel. The data collection is terminated by pushing the letter "E" on the keyboard. After the termination of the data collection, the computer processes the data and arranges it in a desirable tabular form and stores it in memory. The computer then asks if the data should be stored onto a cassette tape. By rewinding the cassette tape and pushing the "RETURN" key on the computer keyboard, the data is transferred to a cassette tape. If another tape is required, the computer will ask for another tape until all the data is transferred to cassette tapes.

The tapes are then brought back to the office and played back on the tape into the mini-computer, which sends the data files to the University of Illinois CYBER computer for further processing and analysis.

PRESENTATION OF DATA

There were a total of five field trips to collect wave and drawdown data. Three of the field trips were to the Illinois River, while two were to the Mississippi River. At the beginning of the project in May 1980, there was a field trip to Havana on the Illinois River to test the field data gathering procedures. The movie films taken during that trial field trip were not easily readable because of improper positioning of the movie camera with respect to the staff gage. However, the experience was very useful in making the proper adjustments in the positioning of the movie camera and the staff gage in the subsequent field trips.

Summary of Field Trips

Summaries of the five field trips and the events where wave and drawdown data were collected are given in tables 7 through 11. In each table, the date and local time of the event, the number of barges, direction, draft, and speed of the tow are indicated.

Trip 1 was to Hadley's Landing on the Illinois River, river mile 13.2, during the period July 22 to 24, 1980. There were a total of five events where four of the events were downstream traffic and one upstream. The number of barges pushed by the tows varied from 6 to 15. The draft and speed ranged from 2 to 9 feet and from 5.7 to 7.9 ft/sec, respectively.

Trip 2 was again to Hadley's Landing during the period September 22 to 26, 1980. There were a total of 9 events of which 5 were upstream traffic and 4 downstream. The number of barges pushed by the tows varied from 6 to 18. Eight of the tows were fully loaded with 9 feet of draft

Table 7. Summary of Trip 1 at Hadley's Landing, Illinois River, River Mile 13.2

Date	Local time	Event	Traffic direction	Tow name	No. of barges	Draft (ft)	Speed (ft/sec)
7/23/80	1520	1	Down	Chicago Trader	6	8.0	7.9
7/23/80	1541	2	Down	Herb Schreiner	12	9.0	5.7
7/23/80	1614	3	Up	Marvin E. Norman	15	2.0	7.8
7/24/80	0959	4	Down	Yankton	12	9.0	6.4
7/24/80	1145	5	Down	Fort Pierre	15	9.0	6.7

Table 8. Summary of Trip 2 at Hadley's Landing, Illinois River, River Mile 13.2

Date	Local time	Event	Traffic direction	Tow name	No. of barges	Draft (ft)	Speed (ft/sec)
9/23/80	1236	2	Down	Leviticus	6	9.0	8.2
9/23/80	1454	3	Up	Craig M.	14	9.0	5.9
9/24/80	1315	4	Up	Irene Chotin	18	9.0	7.1
9/25/80	1015	5	Up	Barbara Jeanne Meyer	8	2.0	8.0
9/25/80	1310	6	Down	Luke Gladders	15	9.0	8.5
9/25/80	1450	7	Down	Fort Pierre	12	9.0	6.0
9/25/80	1747	9	Down	Virginia E. Towey	11	9.0	3.2
9/26/80	0950	10	Up	Clark Frame	12	9.0	5.9
9/26/80	1115	11	Up	Betty Brent	12	9.0	8.7

Table 9. Summary of Trip 3 at Rip Rap Landing, Mississippi River, River Mile 265.1

Date	Local time	Event	Traffic direction	Tow name	No. of barges	Draft (ft)	Speed (ft/sec)
4/08/81	0947	3	Up	Atlas	9	5.0	5.7 '
4/08/81	1044	4	Up	Prairie Dawn	15	2.0	10.2
4/09/81	0823	6	Up	Lillian Clark	16	8.0	11.6
4/09/81	1005	7	Down	Arthur E. Snider	12	9.0	11.9
4/09/81	1300	9	Down	White Dawn	15	9.0	12.8
4/10/81	0825	12	Up	Conti Afton	15	2.0	8.6
			Up	Gordon Jones	13	9.0	7.4

Table 10. Summary of Trip 4 at McEver's Island, Illinois River, River Mile 50

Date	Local time	Event	Traffic direction	Tow name	No. of barges	Draft (ft)	Speed (ft/sec)
4/28/81	1323	1	Up		4	2.0	9.3
4/28/81	1406	2	Down	Kay D.	2	2.0	12.5
4/29/81	0849	3	Down	H. F. Leonard	12	9.0	9.4
4/29/81	1125	4	Up	Cooperative Ambassador	15	2.0	12.1
4/30/81	1203	6	Up	Captain Caplener	10	2.0	8.1
4/30/81	1508	7	Down	Betty Brent	4	2.0	11.5
4/30/81	1534	8	Up	Irene Chotin	15	9.0	8.8
5/01/81	0920	9	Up	Sarah Elizabeth	8	5.5	11.5
5/01/81	0948	10	Down	Normania	9	9.0	12.8
5/01/81	1030	11	Down	Sally Barton	12	9.0	12.7
5/01/81	1056	12	Down	National Enterprise	9	6.6	14.4
5/01/81	1116	13	Down	Cooperative Ambassador	15	9.0	15.6
5/01/81	1206	14	Down	Katherine L.	1	2.0	20.3
5/01/81	1326	15	Up	Luke Gladders	15	2.0	5.9
5/01/81		16	Down	Evelyn Rushing	12	9.0	12.8

Table 11. Summary of Trip 5 at Mosier Landing,
Mississippi River, River Mile 260.2

Date	Local time	Event	Traffic direction	Tow name	No. of barges	Draft (ft)	Speed (ft/sec)
5/21/81	0900	4	Down	Kathy Ellen	12	9.0	14.0
5/21/81	1050	6	Down	Frederick B. Wells	12	9.0	13.1
5/22/81	1205	9	Up	Nohab Express	3	9.0	9.8
5/22/81	1720	10	Down	White Knight	15	9.0	13.3
5/22/81	1730	11	Up	Colonel George Lambert	15	2.0	9.3

and one unloaded tow with 2 feet of draft. The speed of the tows varied from 3.2 to 8.7 ft/sec.

Trip 3 was to Rip Rap Landing on the Mississippi River, river mile 265.1, during the period April 7 to 10, 1981. There were a total of six events with five tows moving upstream and two downstream. Event 12 involved two tows which passed the test site at the same time while one was attempting to pass the other. The number of barges pushed by the tows varied from 9 to 16. the draft and speed of the tows ranged from 2 to 9 feet and from 5.7 to 12.8 ft/sec, respectively.

Trip 4 was to McEver's Island on the Illinois River, river mile 50, during the period April 27 to May 1, 1981. There were a total of 15 events with six tows moving upstream and 9 moving downstream. The number of barges pushed by the tows varied from 1 to 15. The draft and speed ranged from 2 to 9 feet and 5.9 to 20.3 ft/sec, respectively.

Trip 5 was to Mosier Landing on the Mississippi River, river mile 260.2, during the period May 20 to 22, 1981. There were a total of five events with two tows moving upstream and three moving downstream. The number of barges pushed by the tows varied from 3 to 15. The draft was 9 feet for all of them except one. The speed of the tows ranged from 9.3 to 14 ft/sec.

Waves and Drawdown Generated by River Traffic

Wave Patterns

Most of the wave data collected during the field trips were generated by tows, with few recreational and other river traffic. This was because of the reduction of the data collection period and the low frequency of recreational vessels during the field trips.

The wave pattern generated by tows in restricted channels is much more complex than those generated by streamlined vessels traveling in open and deep waters. Even though the diverging and transverse waves are generated both at the bow and stern, there are also surge waves behind the tows generated because of the displacement of a large portion of the water in the river by the loaded barges. In some instances the surge waves totally predominate the other types of waves. There is also a narrow band of disturbed water surface behind the towboat due to the propeller discharging near the water surface. The water surface fluctuation due to the propeller jet seems to be higher than the waves which reach the shore when observed behind the tow. This water surface fluctuations is, however, dissipated in the middle of the channel before it reaches the shore.

An example of a tow generated wave is shown in figure 29. The wave data was collected at the Hadley's Landing test" site on the Illinois River during a passage of a downstream-bound tow with 15 loaded barges traveling at a speed of 8.54 ft/sec. In this wave pattern it is possible to identify the bow, stern, and the towboat stern waves as shown in the figure. During this event the maximum wave height, which is 0.39 feet, was generated by the bow of the tow. Another example of tow generated waves is shown in figure 30. The wave data was collected at the Rip Rap Landing test site on the Mississippi River during a passage of two tows. One of the tows was attempting to pass the other. The data also includes the drawdown during the event. The maximum drawdown reached 0.34 feet, while the maximum wave height was 0.81 feet. A comparison of the two wave patterns in figures 29 and 30 shows how the waves generated at Rip Rap

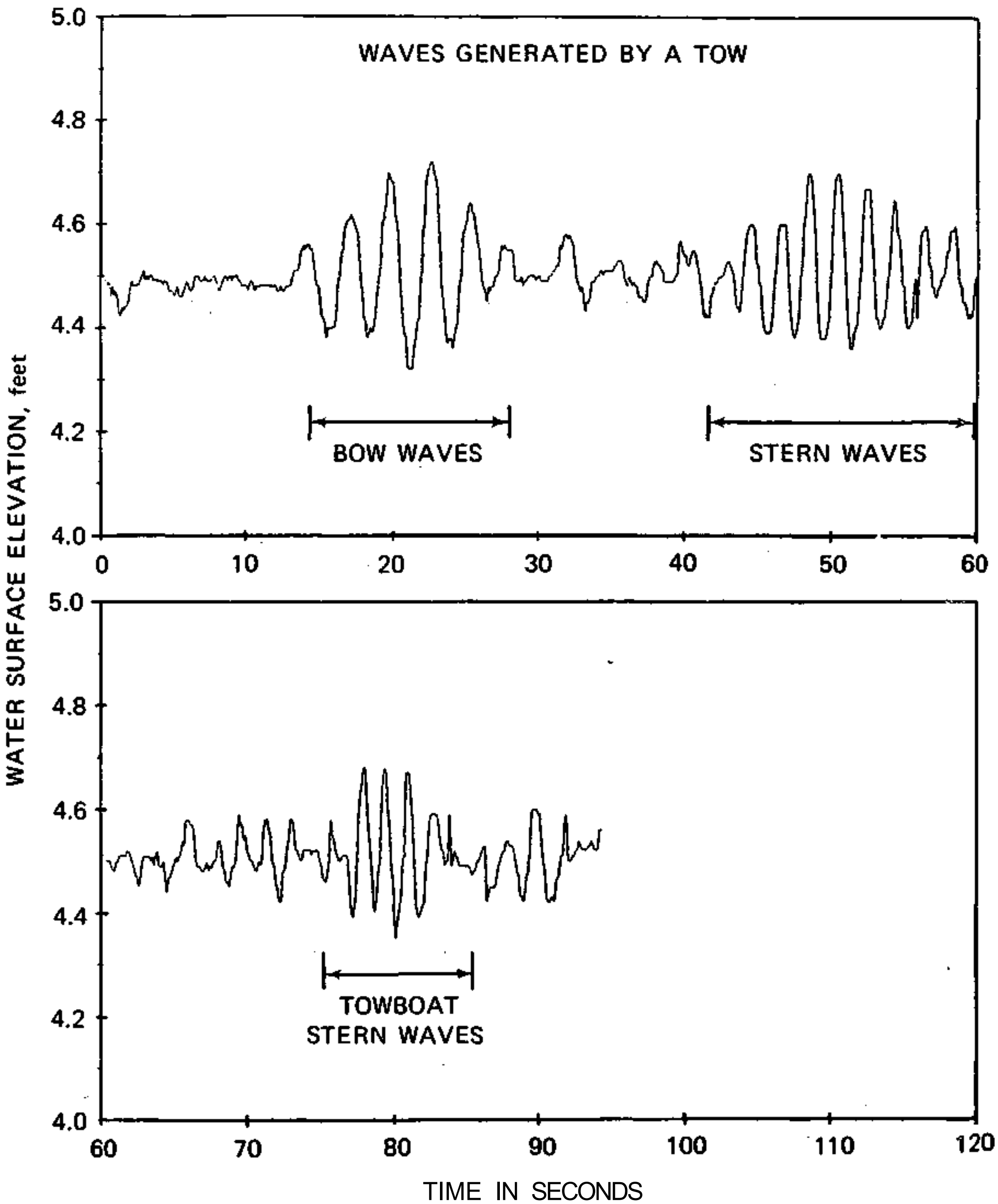


Figure 29. Wave pattern generated by a tow

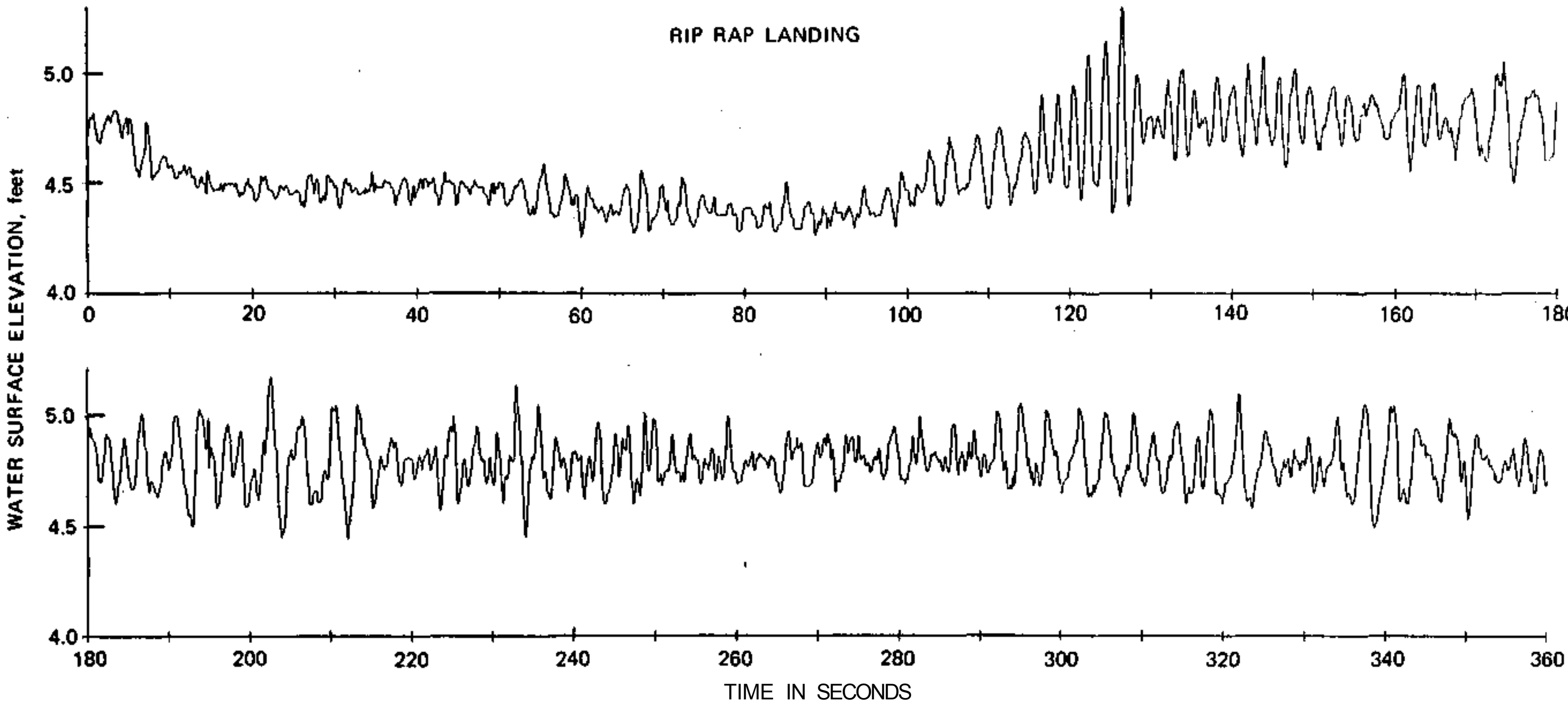


Figure 30. Wave pattern generated by two tows

Landing are more complex and last for a much longer duration and are generally higher.

As mentioned earlier there were other types of river traffic other than tows during the field trips. The waves generated by such vessels were also measured even though there were not enough events to compile enough wave data for different types of vessels. The data collected gives some basis for comparison. The kind of waves generated by a single towboat shown in figure 31 is given in figure 32. The wave pattern is significantly different than that generated by tows in that it only consists of a couple of sharp, well-defined waves and dies out quickly. However, the maximum wave height is 0.89 feet, which is higher than most of the waves generated by tows.

The wave pattern generated by a cabin cruiser shown in figure 33 is given in figure 34. Here again the wave pattern is somewhat different than those generated by tows. The wave peaks and troughs are relatively well-defined and the duration of the wave is relatively shorter than those waves generated by tows.

Maximum Wave Heights

The maximum wave heights for all the events during the five field trips were determined from plots similar to those in figures 29 and 31. All the maximum wave heights are summarized in table 12. Table 12 also includes the data for the maximum drawdown, tow description, direction, distance from wave gage, and channel cross-sectional area.

There were a total of 41 events where the maximum wave heights were determined. The maximum wave heights ranged from a low of 0.1 feet to a high of 1.05 feet. The maximum wave height of 1.05 feet occurred at the

Figure 31. Towboat moving past a test site

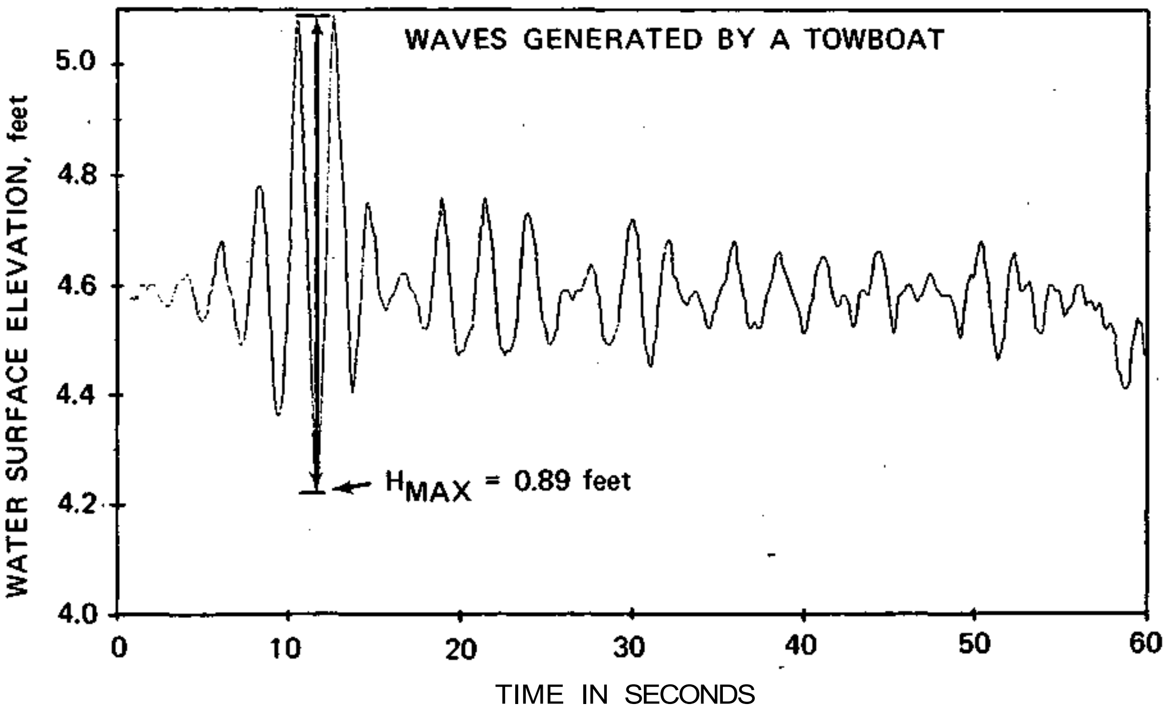


Figure 32. Wave pattern generated by a towboat

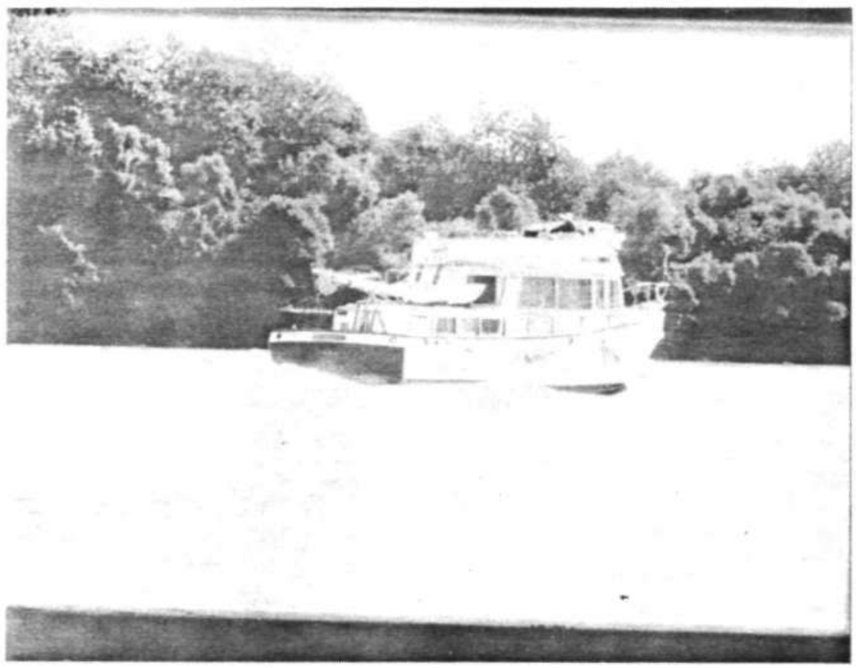


Figure 33. Cabin cruiser moving past a test site

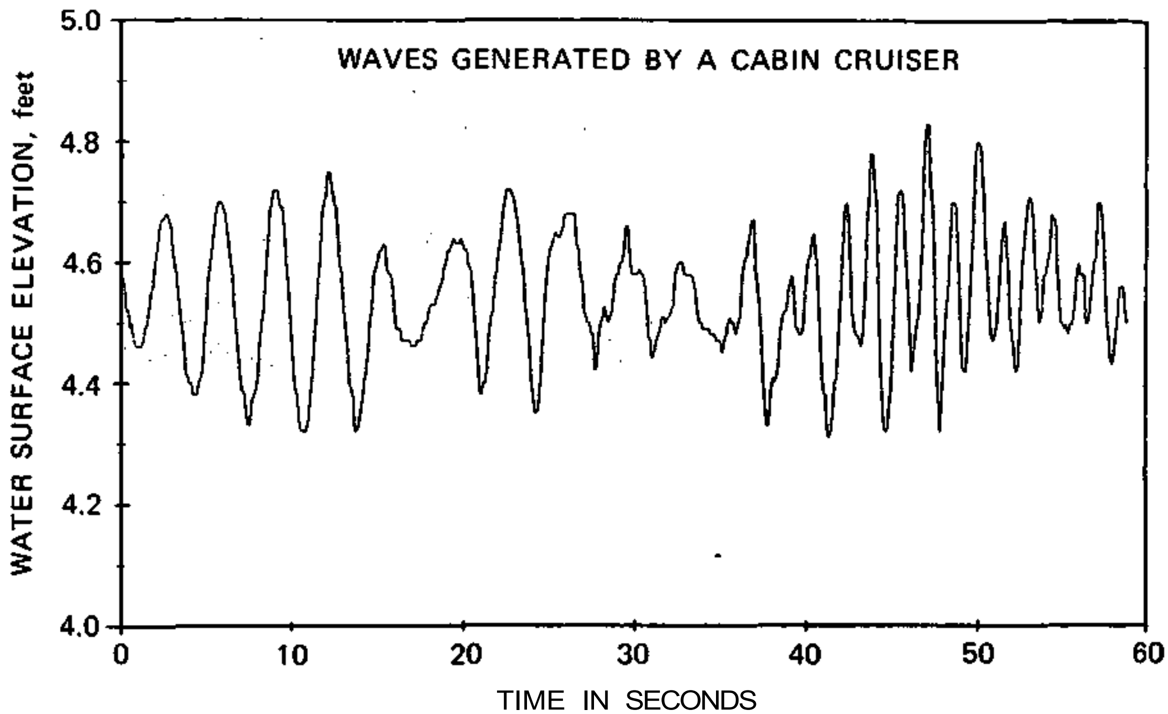


Figure 34. Wave pattern generated by a cabin cruiser

Table 12. Summary of Field Data

TRIP NO.	EVENT NO.	DIRECTION U/D STREAM	WAVE GAGE DISTANCE (FT)	TOW DESCRIPTION					CHANNEL X-AREA (SQ.FT)	MAX. WAVE HEIGHT (FT)	MAX. DRAWDOWN (FT)
				NO. OF BARGES	TOTAL. LENGTH (FT)	TOTAL WIDTH (FT)	DRAFT (FT)	SPEED (FT/S)			
1	1	DOWN	480.	6	585.	70.0	8.0	7.87	13915.	.15	--
1	2	DOWN	405.	12	780.	105.0	9.0	5.70	13915.	.20	--
1	3	UP	450.	15	975.	105.0	2.0	7.81	13915.	.30	--
1	4	DOWN	450.	12	780.	105.0	9.0	6.42	13915.	.25	--
1	5	DOWN	510.	15	975.	105.0	9.0	6.68	13915.	.40	--
2	2	DOWN	450.	6	585.	70.0	9.0	8.16	13915.	.86	--
2	3	UP	390.	14	975.	105.0	9.0	5.93	13915.	.20	.20
2	4	UP	420.	18	1170.	105.0	9.0	7.05	13915.	.46	.43
2	5	UP	450.	8	585.	105.0	2.0	7.98	13915.	.23	.14
2	6	DOWN	540.	15	975.	105.0	9.0	8.54	13915.	.39	.13
2	7	DOWN	525.	12	780.	105.0	9.0	6.01	13915.	.10	.20
2	9	UP	435.	11	780.	70.0	9.0	3.19	13915.	.31	
2	10	UP	390.	12	780.	105.0	9.0	5.90	13915.	.80	.30
2	11	DOWN	420.	12	780.	105.0	9.0	8.70	13915.	.18	.27
3	3	UP	330.	9	585.	105.0	5.0	5.70	24155.	.40	.10
3	4	UP	305.	15	975.	105.0	2.0	10.20	24155.	.45	.22
3	6	UP	255.	16	1170.	123.0	8.0	11.60	24155.	1.05	.30
3	7	DOWN	280.	12	780.	105.0	9.0	11.90	24155.	.38	.11
3	9	DOWN	330.	15	975.	105.0	9.0	12.80	24155.	.56	--
3	12	UP	655.	15	975.	105.0	2.0	8.60	24155.	.54	.30
3	12	UP	30.	13	975.	105.0	9.0	7.40	24155.	.81	.34
4	1	UP	235.	4	390.	70.0	2.0	9.30	15883.	.45	--
4	2	DOWN	225.	2	390.	35.0	2.0	12.50	15883.	.30	--
4	3	DOWN	175.	12	780.	105.0	9.0	9.40	15883.	.34	.24
4	4	UP	245.	15	975.	105.0	2.0	12.10	15883.	.41	.10
4	6	UP	35.	10	780.	105.0	2.0	8.10	15883.	.35	.30
4	7	DOWN	205.	4	390.	70.0	2.0	11.50	15883.	.62	.21
4	8	UP	195.	15	2100.	100.0	9.0	8.80	15883.	.35.	.40
4	9	UP	165.	8	585.	105.0	5.5	11.50	15883.	.89	.27
4	10	DOWN	160.	9	585.	105.0	9.0	12.80	15883.	.52	.21
4	11	DOWN	150.	12	780.	105.0	9.0	12.70	15883.	.55	--
4	12	DOWN	155.	9	585.	105.0	6.6	14.40	15883.	.72	.30
4	13	DOWN	145.	15	975.	105.0	9.0	15.60	15883.	.96	.69
4	14	DOWN	150.	1	195.	35.0	2.0	20.30	15883.	.93	.15
4	15	UP	155.	15	975.	105.0	2.0	5.90	15883.	.30	.10
4	16	DOWN	150.	12	780.	105.0	9.0	12.80	15883.	.44	.25
5	4	DOWN	604.	12	780.	105.0	9.0	14.00	24000.	.20	.13
5	6	DOWN	600.	12	780.	105.0	9.0	13.10	24000.	.50	--
5	9	UP	688.	3	900.	50.0	9.0	9.80	24000.	.35	.05
5	10	DOWN	720.	15	975.	105.0	9.0	13.30	24000.	.90	--
5	11	UP	724.	15	975.	105.0	2.0	9.30	24000.	.50	--

Rip Rap Landing test site on the Mississippi River for a loaded tow with 16 barges at 8 feet of draft moving upstream with a speed of 11.6 ft/sec.

Drawdown

At the beginning of the wave data collection program, there was no plan to collect drawdown data; however, as the data collection program progressed, significant drawdown was being observed at the test sites. Therefore, it was decided to gather drawdown data along with the wave data. Because of the late start in collecting drawdown data and 8 events where drawdown measurements were missed, the total number of events where drawdown was measured is 27.

The maximum drawdown for all the 27 events is summarized in table 12 along with the maximum wave height. The maximum drawdown ranged from 0.05 to 0.69 feet. The maximum drawdown of 0.69 feet was measured at the McEver's Island test site on the Illinois River during the passage of a loaded tow with 15 barges at 9 feet of draft moving downstream with a speed of 15.6 ft/sec.

The maximum drawdown is usually treated as the most important parameter, partially because of the traditional interest in squat and the associated grounding and maneuverability problems of vessels in restricted waterways. However, the total drawdown period lasts for several minutes depending on the length of the vessel. The extent of shoreline exposure and its duration might be of higher importance in bank erosion and biological studies than just the maximum drawdown value.

The plot of water elevation in figure 30 shows the water level drop for almost 2 minutes before the bow waves arrive at the wave gage. The maximum drawdown was 0.35 feet. The water level fluctuation during the drawdown period is due to wind waves at the time of the event. Another example of drawdown is shown in figure 35. The maximum drawdown was 0.30 feet and the total drawdown period was almost 3 minutes. The data was collected at the Hadley's Landing test site during the passage of a loaded tow with 12 barges at 9 feet of draft moving downstream with a speed of 5.9 ft/sec.

Mathematical Modeling

River Traffic Generated Waves

There has been very limited research in the area of river traffic generated waves on restricted waterways. As discussed in the literature review, most of the investigations have been about waves generated by ships traveling in deep and unrestricted waters. The few investigations dealing with waves in restricted waterways were mostly done in Europe in relation to ship canal design.

Based on laboratory and field observations, some investigators have developed empirical equations for predicting wave heights based on channel and vessel parameters. Balanin and Bykov (1965) used the vessel velocity and a modified blockage factor as the primary variables to develop the following equation for estimating the wave height in the vicinity of a ship.

$$H_s = 2.5\,\frac{V^2}{2g}\left[1 - \left(1 - 1\Big/\left(4.2 + \frac{A_c}{A_m}\right)^{0.5}\right)\left(\frac{A_c/A_m - 1}{A_c/A_m}\right)^2\right] \qquad (1)$$

where: H_s = wave height in feet

V = vessel velocity in ft/sec

g = gravitational acceleration in ft/sec^2

A_c = the cross-sectional area of the channel

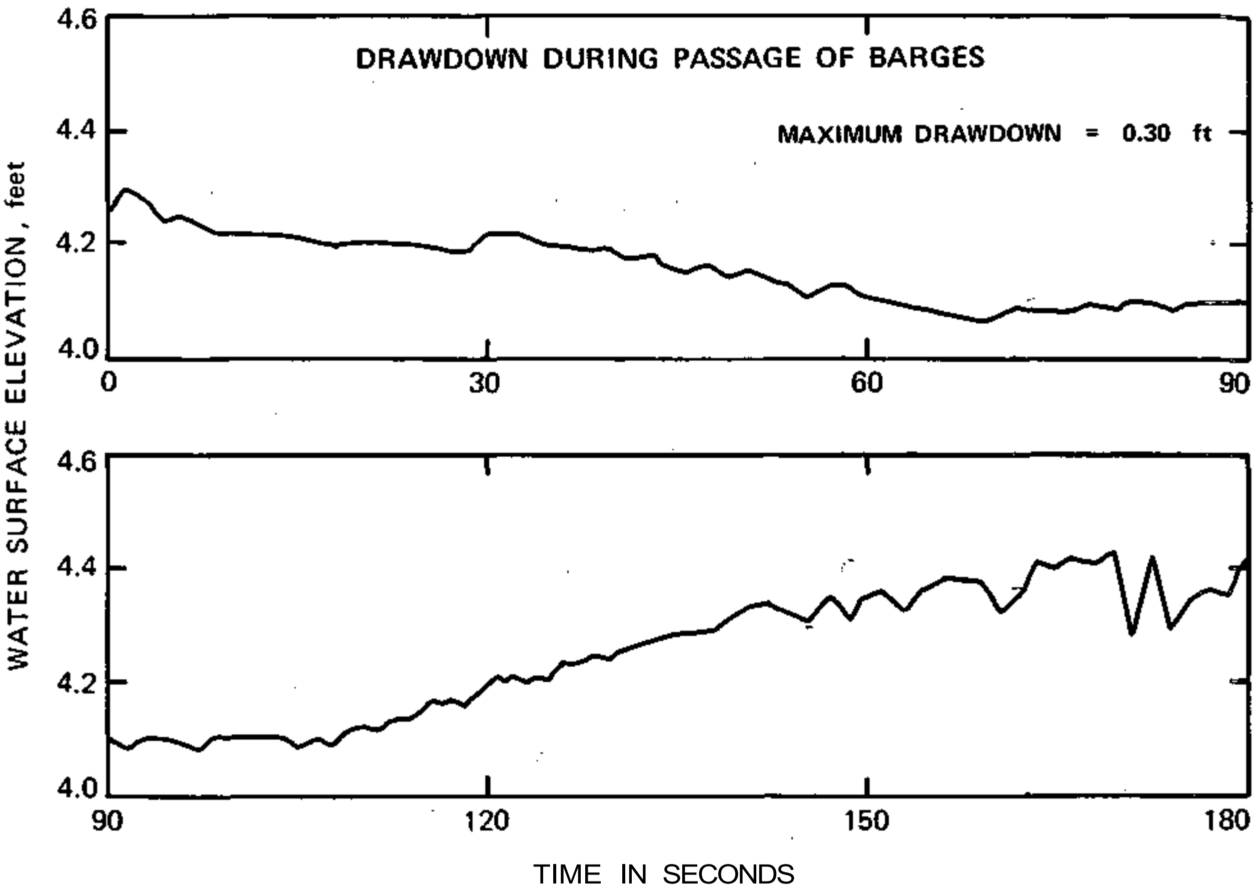


Figure 35. Drawdown during passage of barges at Hadley's Landing

A_m = bxd = the submerged cross-sectional area of the vessel in square feet

b = the width of the vessel in feet

d = the draft of the vessel in feet

Another equation for estimating maximum wave height is given by Hochstein (1980) as follows:

$$H_{max} = 0.0448\ V^2 \left(\frac{d}{L}\right)^{0.5} \left(1 - \frac{bd}{A_c}\right)^{-2.5} \quad (2)$$

where H_{max} = maximum wave height in feet

L = length of the vessel in feet

All other variables are as defined before.

The main difference between the two equations other than their form is the inclusion of the vessel's length in Hochstein's equation. The results of both equations are compared to the measured wave heights in table 13. The correlation coefficients between the measured and calculated maximum wave heights are 0.59 and 0.65 for Balanin and Bykov and for Hochstein, respectively. The calculated maximum wave heights are also compared in figures 36 and 37 for Hochstein, and Balanin and Bykov, respectively. As shown in figure 36, Hochstein's equation underestimates the measured maximum wave height, while the Balanin and Bykov equation overestimates the measured maximum wave heights for most parts as shown in figure 37. In any case, the correlation coefficients for both equations are low, and therefore their use for the conditions investigated in this project is not highly reliable.

Multi-variate regression analysis between the measured maximum wave heights and the important parameters which were felt to influence the maximum wave heights yielded the following equation.

Table 13. Comparison of Calculated and Measured Maximum Wave Heights and Drawdowns

		WAVE HEIGHT			DRAWDOWN				
TRIP NO.	EVENT NO.	MEASURED HMAX (FT)	CALCULATED HMAX (FT) HOCHSTEIN	BALANIN	MEASURED DMAX (FT)	CALCULATED DMAX (FT) DAND	SCHIF	GELENCSER	HERBICH
1	1	.15	.15	.25	—	.01	.03	.00	.03
1	2	.20	.05	.11	—	.03	.02	.00	.02
1	3	.30	.21	.61	—	.01	.07	.01	.05
1	1	.25	.07	.16	—	.01	.03	.00	.03
1	5	.10	.08	.19	—	.05	.01	.01	.03
2	2	.86	.18	.29	—	.05	.01	.00	.01
2	3	.20	.39	.99	.20	.24	.22	.05	.18
2	1	.16	.16	1.26	.13	.31	.29	.07	.23
2	5	.23	.32	.66	.11	.05	.07	,01	.06
2	6	.39	.17	.12	.13	.10	.08	.01	.08
2	7	.10	.06	.13	.20	.03	.02	.00	.02
2	9	.31	.20	.38	—	.07	.06	.01	.05
2	10	.80	.11	.98	.30	.21	.21	.03	.18
2	11	.18	.20	.11	.27	.11	.09	.01	.08
3	3	.10	.35	.55	.10	.05	.06	.01	.06
3	1	.15	.37	.75	.22	.03	.07	.02	.05
3	6	1.05	.90	2.11	.30	.31	.63	.20	.30
3	7	.38	.10	.72	.11	.11	.11	.03	.10
3	9	.56	.11	.88	—	.13	.11	.05	.12
3	12	.51	.29	.58	.30	.02	.05	.01	.01
3	12	.81	.51	1.08	.34	.16	.19	.55	.15
1	1	.15	.18	.61	—	.03	.05	.00	.01
1	2	.30	.31	.27	—	.01	.01	.00	.01
1	3	.31	.25	.52	.21	.11	.10	.03	.09
1	1	.11	.17	1.16	.10	.07	.15	.01	.09
1	6	.35	.28	.62	.30	.01	.06.	.10	.05
1	7	.62	.25	.32	.21	.01	.02	.00	.02
1	8	.35	.16	1.57	.10	.33	.36	.55	.26
1	9	.89	.97	1.87	.27	.27	.11	.06	.21
1	10	.52	.65	1.20	.21	.26	.26	.05	.20
1	11	.55	.55	1.18	--	.26	.25	.09	.20
1	12	.72	.72	1.37	.30	.23	.27	.05	.20
1	13	.96	.82	1.96	.69	.13	.52	.22	.33
1	11	.93	1.11	.88	.15	.02	.08	.00	.01
1	15	.30	.16	.10	.10	.02	.03	.02	.03
1	16	.11	.56	1.20	.25	.26	.26	.09	.20
5	1	.20	.59	1.06	.13	.16	.20	.02	.11
5	6	.50	.19	.88	—	.11	.15	.02	.12
5	9	.35	.83	1.13	.05	.09	.18	.02	.11
5	10	.90	.16	.92	--	.14	.16	.02	.12
5	11	.50	.31	.68	—	.03	.07	.01	.05

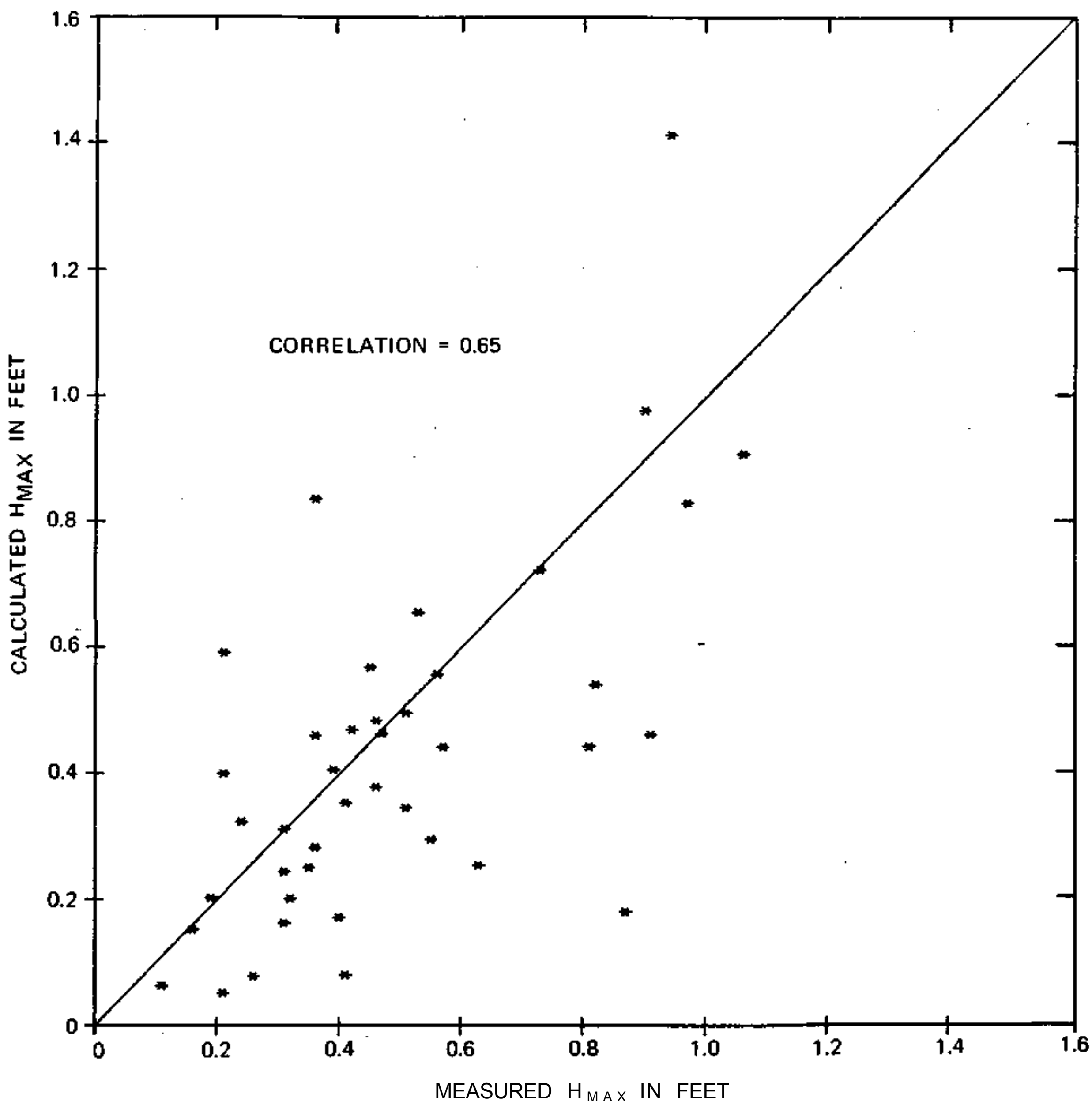


Figure 36. Comparison of measured and calculated maximum wave heights (Hochstein)

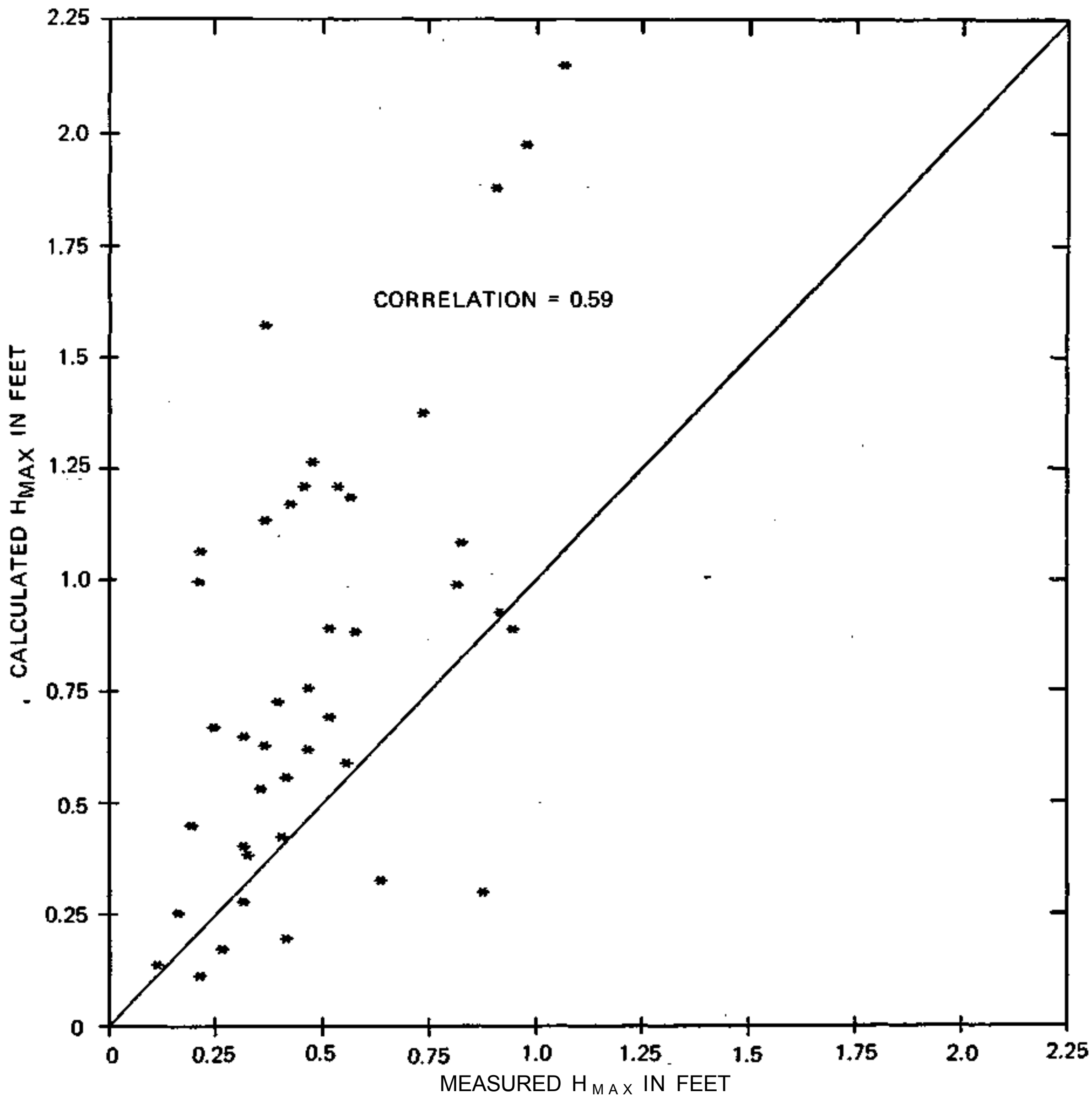


Figure 37. Comparison of measured and calculated maximum wave heights (Balanin and Bykov)

$$H_{max} = 0.867 \frac{V^2}{2g} \left(\frac{d}{L}\right)^{0.26} \left(1 - \frac{bd}{A_c}\right)^{-3.0} \quad (3)$$

All the variables are as defined before. The spread of the data points around the regression line is shown in figure 38. The correlation coefficient is 0.64. The regression analysis and Hochstein's equation give practically the same result.

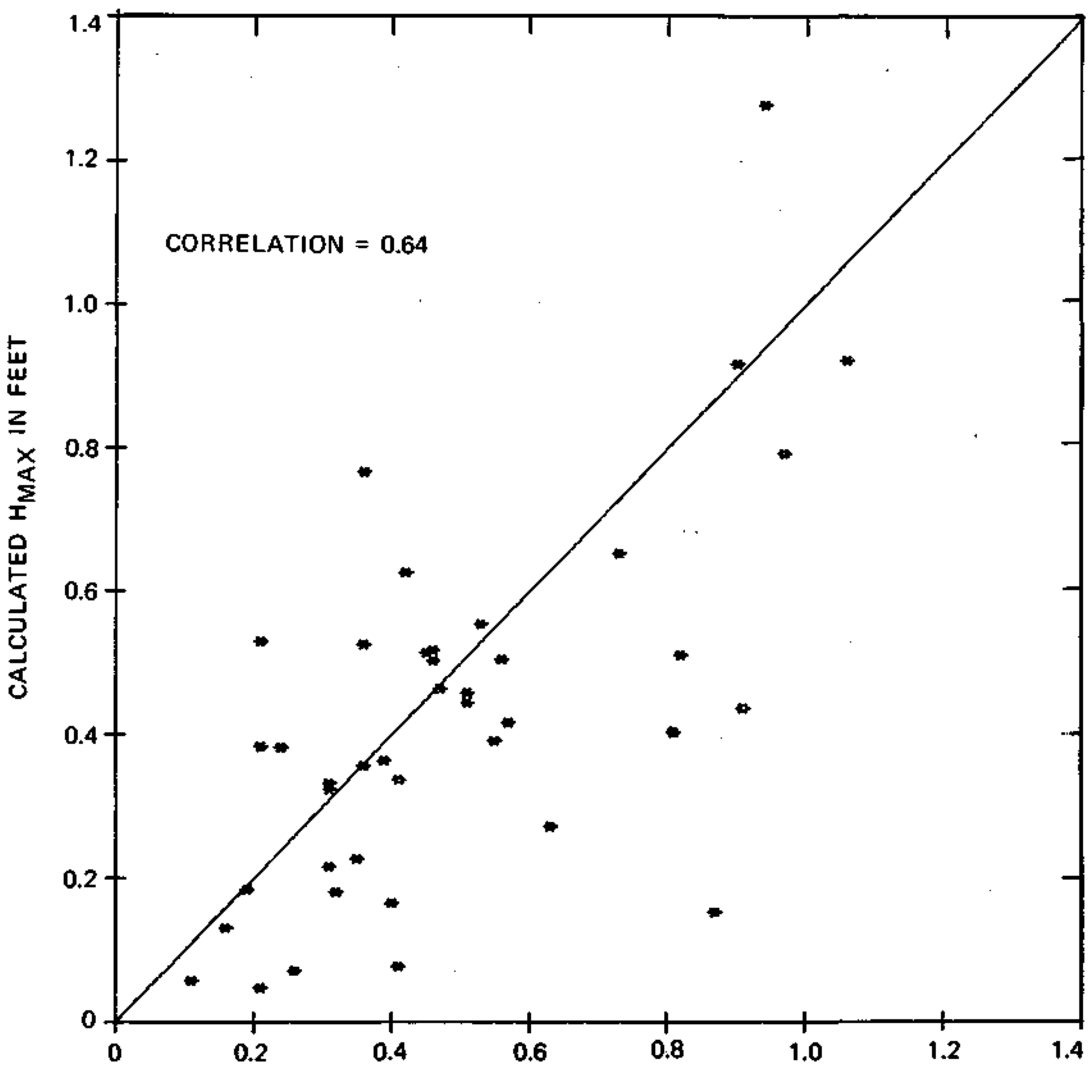
Drawdown

There have been several attempts to determine the squat of vessels in canals and harbor entrances because of the problem of grounding and loss of control of vessels in shallow and restricted waterways at high values of squat. The problem of squat has also become more serious in recent years as larger sizes of modern vessels transporting larger cargo need to use channels and harbor entrances designed for smaller vessels.

As discussed in the literature review, squat and drawdown are generally treated as equal to simplify the physical phenomena as one dimensional flow. Further assumptions made in drawdown or squat analysis include constant vessel velocity in a straight channel, uniform vessel cross section and backflow throughout the flow section, uniform squat over the length of the vessel, and no frictional losses.

Under the above assumptions, Schijf and Jansen (1953) developed a method to estimate the drawdown from one dimensional energy and continuity equations as follows. The drawdown or squat, Ah, is given by the equation:

$$\Delta h = \frac{(V_W + \Delta V)^2 - V_W^2}{2g} \quad (4)$$



MEASURED H_{MAX} IN FEET

Figure 38. Comparison of the regression equation with the measured maximum wave heights

where AV is the backflow velocity beneath the vessel, V_w is water velocity, and the other variables are as defined earlier. Equation 3 is the Bernoulli equation, which states the increase in kinetic equation is equal to the decrease in potential energy if frictional losses are neglected. The term on the right-hand side of the equal sign represents the increase in kinetic energy while the left-hand side represents the decrease in potential energy. The backflow velocity, AV, is computed from the one dimensional continuity equation given as follows:

$$A_c \times V_w = (A_c - A_m - (\Delta h \times B_c))(V_w + \Delta V) \tag{5}$$

where A_m is the submerged vessel cross-sectional area and B_c is the channel width. All the other terms are as defined earlier. Ah is calculated by solving the two simultaneous equations.

Another equation for the drawdown was developed by Gelencser (1969) from prototype and model test results. His equation relates the drawdown to the vessel's length, beam, draft, velocity, and the channel's cross-sectional area and distance from the sailing line as follows.

$$\Delta h = 2 \times 10^{-6} \left[\left(\frac{V \times A_m \times L^2}{x \sqrt{A_c}} \right)^{1/3} \right]^{2.8} \tag{6}$$

where x is distance from the sailing line in meters, L is the length of the vessel in meters, and all other variables are as defined before, except all the length units are in meters. The equation was developed by finding the equation of the best fit line between the variable in the bracket and the observed drawdown data.

Two other drawdown equations which are slightly different from each other were presented by Dand and White (1978) and Gates and Herbich

(1978). Dand's equation was based on scale ship model experiments and is given as follows:

$$\Delta h = 8.8 \left(\frac{A_c}{A_m}\right)^{-1.4} \times \left(\frac{V^2}{2g}\right) \tag{7}$$

All the variables are as defined before.

The drawdown equation presented by Gates and Herbich was derived at both the National Research Council of Canada (1966) and the David Taylor Model Basin (1948). The equation is given as:

$$\Delta h = \frac{V^2}{22.6} \left[\left(\frac{A_c}{A'}\right)^2 - 1\right] \tag{8}$$

where V is the velocity of the vessel in knots, and A' is the cross-sectional area of the channel after the drawdown minus the cross-sectional area of the vessel in square feet. A_c is as defined before.

The drawdowns calculated using the four equations for all the 41 events are summarized in table 13. The results of the four equations are" compared to the measured drawdowns in figures 39, 40, 41, and 42 for Schijf and Jansen, Gelenscer, Dand and White, and Gates and Herbich, respectively. The corresponding correlation coefficients between the measured and calculated drawdowns are 0.59, 0.52, 0.69, and 0.66. In addition to the low correlation coefficients, all the drawdown equations underestimate the measured drawdowns for most cases.

Multi-variate regression analysis for the drawdown data resulted in the following equation,

$$\Delta h = 1.03 \frac{V^2}{2g} \left(\frac{bd}{A_c}\right)^{0.81} \left(\frac{L}{Z}\right)^{0.31} \tag{9}$$

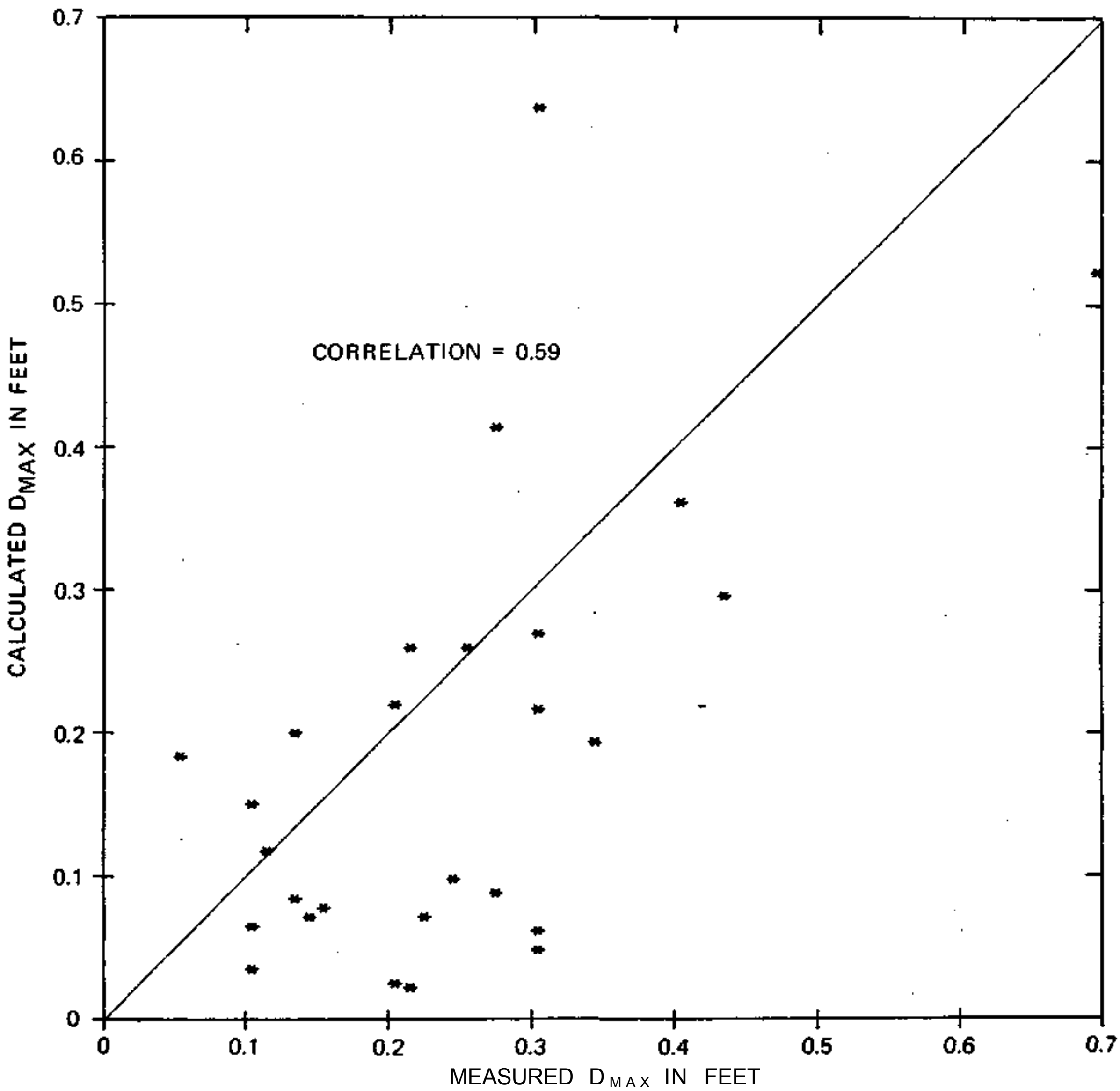


Figure 39. Comparison of measured and calculated drawdown (Schijf)

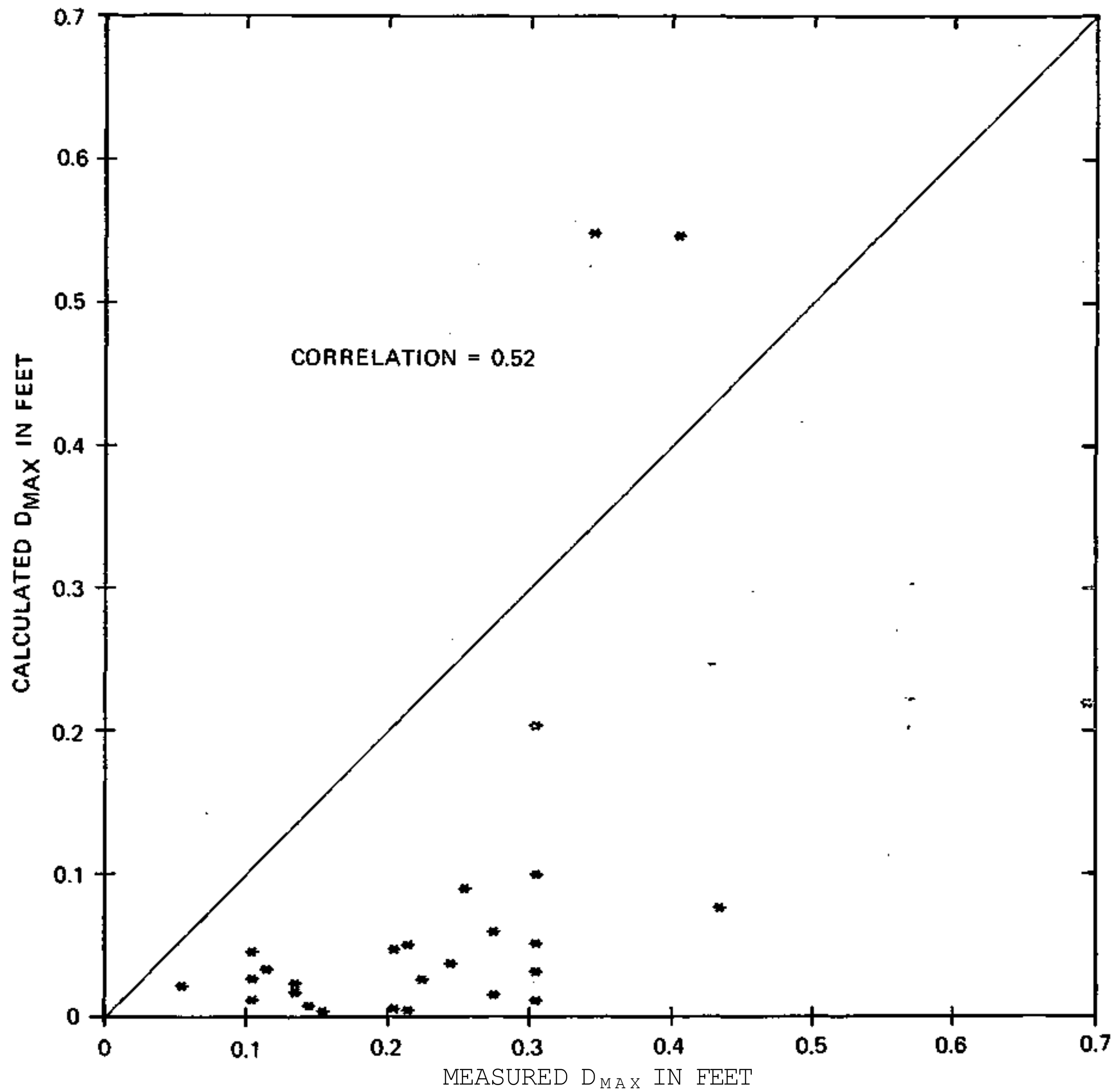


Figure 40 Comparison of measured and calculated drawdown (Gelenscer)

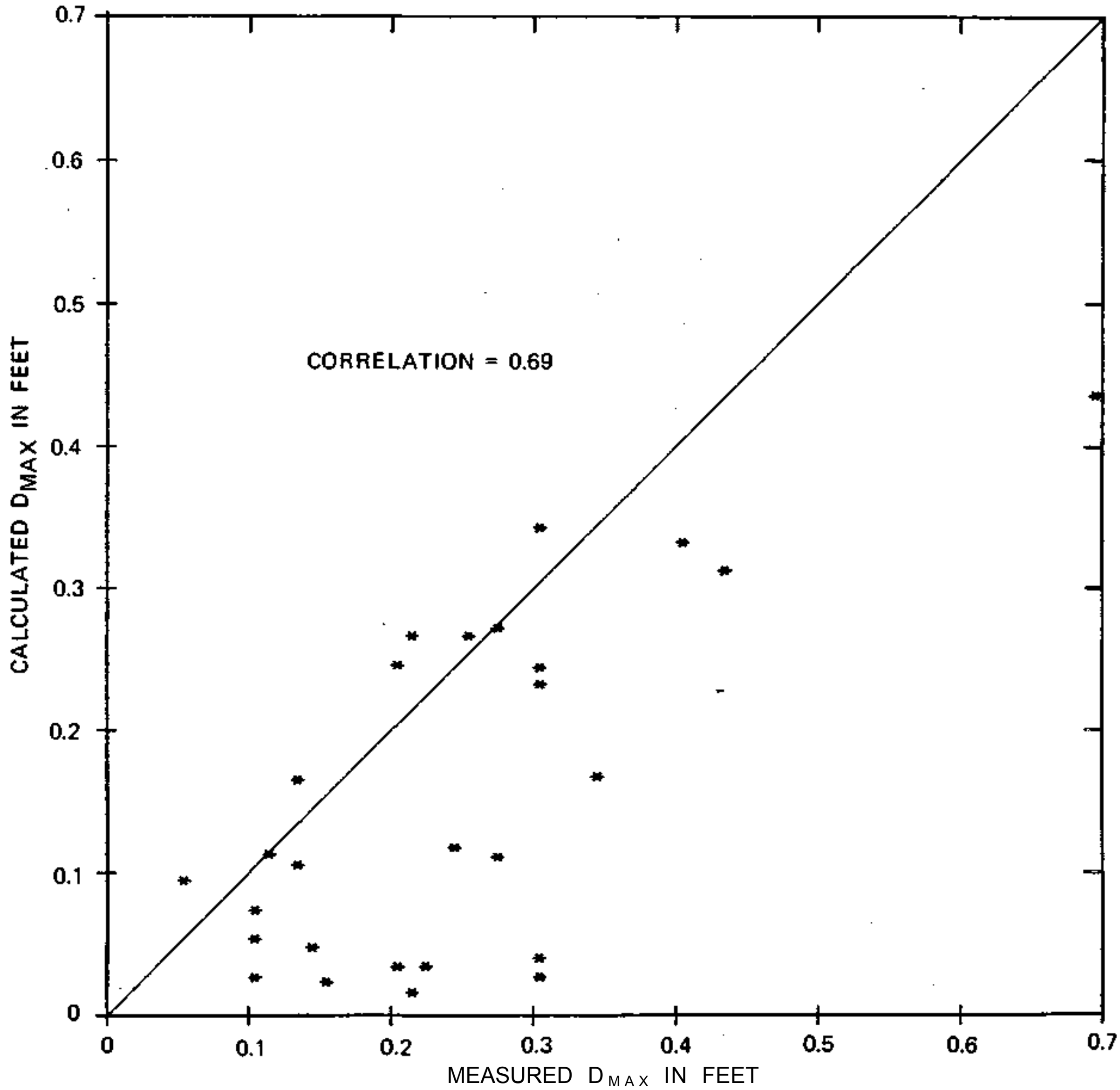


Figure 41. Comparison of measured and calculated drawdown (Dand and White)

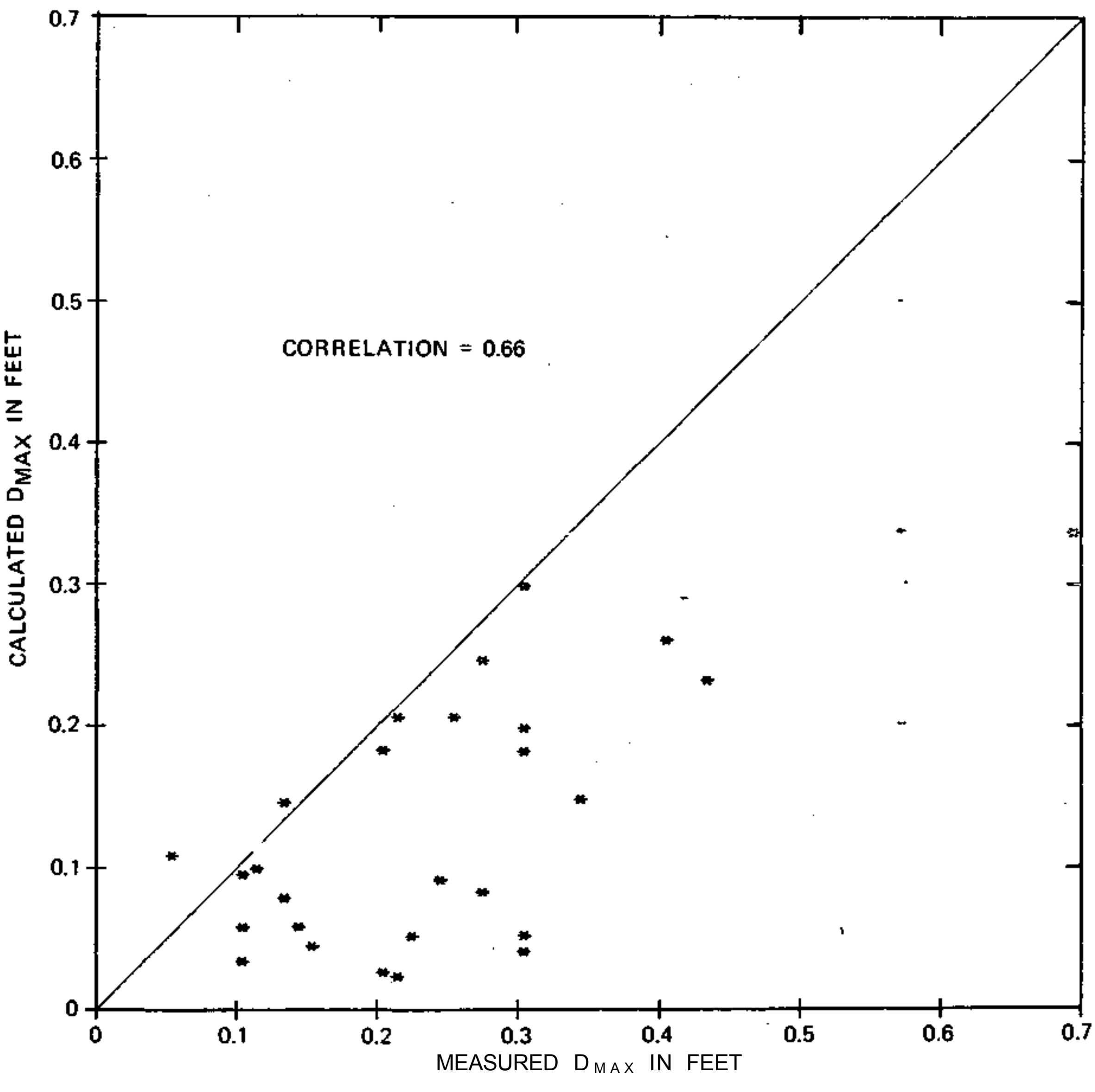


Figure 42. Comparison of measured and calculated drawdown (Gates and Herbich)

where Z is the distance between the wave gage and the sailing line of the vessel in feet. All other variables are as defined before. The spread of the data points around the regression line is shown in figure 43. The correlation coefficient is 0.70, which is better than all the other correlation coefficients.

More data points are needed to evaluate the equations mentioned above and to recommend the best predictive equation for both maximum wave heights and drawdown.

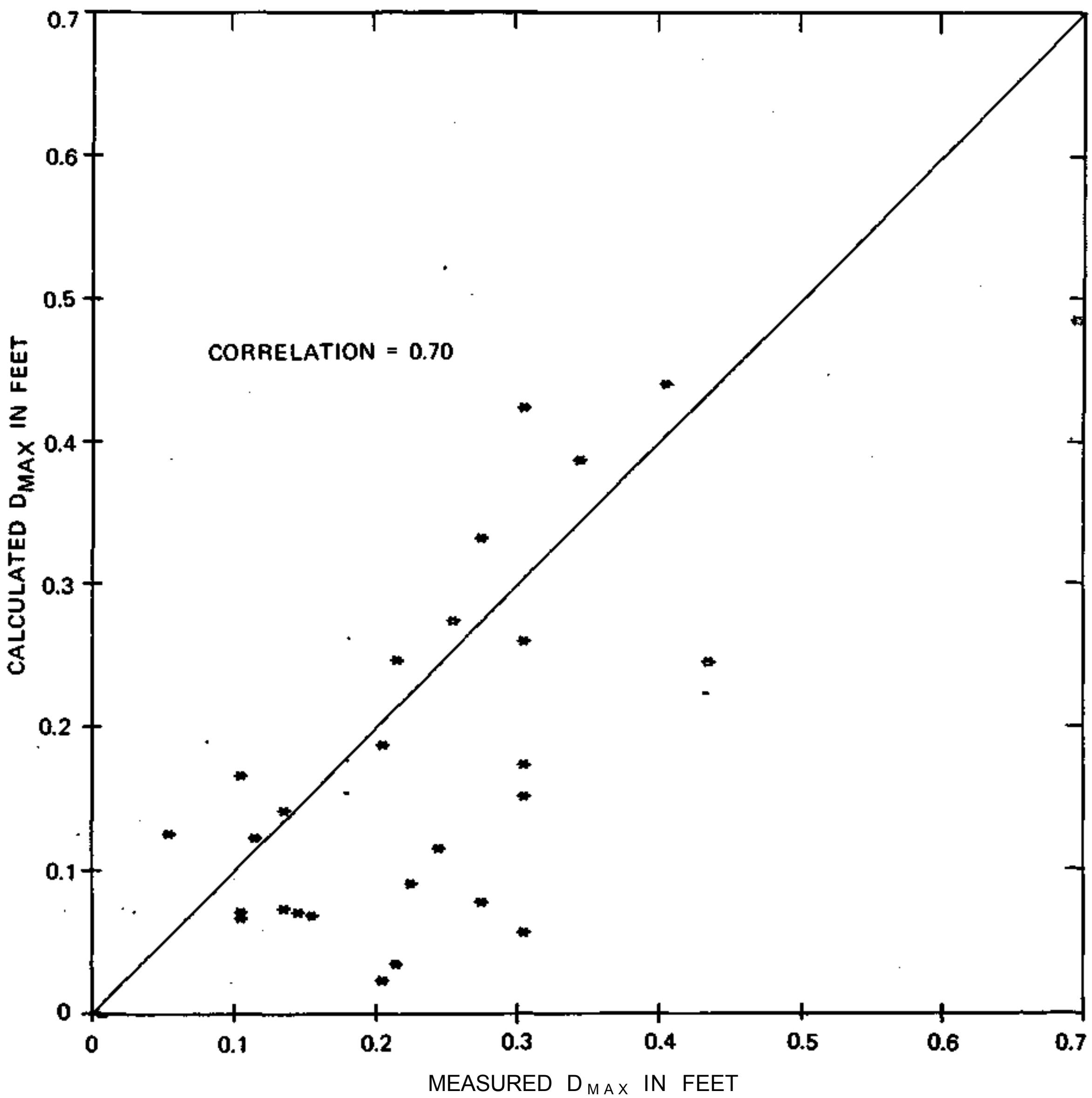


Figure 43. Comparison of the regression equation with the measured maximum drawdown

SUMMARY AND CONCLUSIONS

There were five field trips to collect wave and drawdown data. Three of the field trips were to the Illinois River, while two were to the Mississippi River. Wave data was collected for a total of 41 events, and drawdown data was collected for 27 events. Additional wave data was collected during the passage of a towboat without barges and a cabin cruiser.

The maximum wave heights measured in the field ranged from a low of 0.1 foot to a high of 1.05 foot, while the maximum drawdown ranged from 0.05 foot to 0.69 foot.

Comparison of the measured maximum wave heights and drawdowns with existing predictive equations was not satisfactory. The correlations between the measured and calculated wave heights and drawdowns were low and for most parts the equations underestimated the measured wave heights and drawdowns, except for one equation which overestimated the maximum wave heights. Therefore, their use for the conditions studied in this project without further investigations both analytically and in the field is unreliable.

From the analysis of previous investigations and running a multi-variate regression analysis, it can be concluded that the maximum wave height depends on the velocity of the vessel, the blockage factor, and the length of the vessel. Similarly, the maximum drawdown depends on the velocity of the vessel, the blockage factor, the length of the vessel, and the distance from the sailing line.

In general, the observed wave heights and wave actions are significant enough to cause bank erosion. However, the extent of bank erosion due to river traffic generated waves could not be quantitatively identified and compared to other causes of bank erosion because of the time limitation imposed on the research project.

The drawdown caused by loaded tows is also significant and can expose shore area for a couple of minutes on the average. It can also significantly change the flow characteristics of small tributary streams by changing the hydraulic gradient at their outlet.

It should, however, be emphasized that further study is required to quantify the drawdown and wave erosion phenomena in the Upper Mississippi River Basin with greater accuracy than is possible at the present time.

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