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Waves Generated by River Traffic and Wind on the Illinois and Mississippi Rivers

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Illinois State Water Survey

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ABSTRACT

An excessive amount of bank erosion along a number of waterways in Illinois and surrounding states exists at the present time. Erosion of stream banks attracts public attention, reduces property values, results in the permanent loss of real estate, increases the turbidity of streams, and accelerates the silting of reservoirs and backwater lakes along stream courses. Among the main causes of bank erosion along navigable rivers are waves generated by river traffic and wind. Also associated with river traffic is the drawdown of the water level in the channel, which exposes shore area and changes the flow characteristics of tributary streams close to their outlets.

To investigate and collect data on waves and drawdown associated with river traffic on the Illinois and Mississippi Rivers, six field trips were taken to four test sites.

The maximum wave heights measured in the field ranged from a low of 0.1 . foot to a high of 1.08 foot, while the maximum drawdown ranged from 0.05 foot to 0.69 foot.

The measured maximum wave heights and drawdowns were compared to those expected on the basis of existing predictive equations and the correlations were found to be low. Multivariate regression analyses between the measured values and the important hydraulic and geometric parameters which were felt to influence the generation of waves and drawdowns resulted In two equations which predict maximum wave heights and drawdowns fairly well.

Significant wave heights for wind-generated waves were also calculated. On the Illinois River significant wave heights were in the range of 0.9 and 1.6 ft for 2-yr and 50-yr winds of 6-hr duration, respectively, while on the Mississippi River the corresponding values were 1.3 and 2.4 ft.

In general, the observed and calculated waves generated by both river traffic and wind are significant enough to cause stream bank erosion along the Mississippi and Illinois Rivers. The relative significance of waves generated by river traffic in comparison with those generated by wind cannot be determined qualitatively at the present time because of the differences in frequency, duration, and magnitude. Further research is needed.

The drawdown caused by loaded tows on the two rivers is also significant, exposing shore areas and changing the flow characteristics of small tributary streams during each tow passage.

REFERENCE: Bhowmik, N.G., M. Demissie, and C.Y. Guo, WAVES GENERATED BY RIVER TRAFFIC AND WIND ON THE ILLINOIS AND MISSISSIPPI RIVERS, University of Illinois Water Resources Research Center Report No. 167, March 1982.

KEY WORDS: Barges, boats, erosion, rivers, streams, waterways, waves, wind.

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INTRODUCTION

An excessive amount of bank erosion along a number of waterways in Illinois and 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 values, results in the permanent loss of real estate, increases the turbidity of streams, and accelerates the silting of reservoirs or backwater,lakes along the stream course.

Among the main causes of bank erosion along navigable rivers are 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 can be used to evaluate the relative magnitude of the effects on the shoreline of waves generated by river traffic and waves generated by wind.

Objective

The objective of this project was to collect a set of data on waves generated by river traffic and winds on the Illinois and Mississippi River (representative waterways of the U.S.) in a systematic manner to answer questions such as: What are the characteristics of waves generated by tows, barges, or boats 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?

Acknowledgments

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.

The work upon which this research is based was supported, in part, by funds provided by the Upper Mississippi River Basin Commission and the U.S. Department of the Interior as authorized under the Water Resources Act of 1964, Public Law 88-379, Project No. A-104-ILL. An earlier contract report (Bhowmik et al., 1981b) which is similar to this report, was prepared for the UMRBC.

Several Surface Water Section staff members assisted greatly in the field data collection program. 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 the project.

Bruce Komadina of the Data Management Unit of the Water Survey designed and built the electronic wave gages used for the 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

Vessel-Generated Waves

Most of the early and contemporary research concerning vessel motion in water has 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. This resistance 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 frictional drag acting tangent to the wetted surface of the vessel, which 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 resulting from the waves generated by the vessel's motion.

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

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 that cause disturbances 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 represents deep water conditions where the depth has no effect 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 (1887). 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 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 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

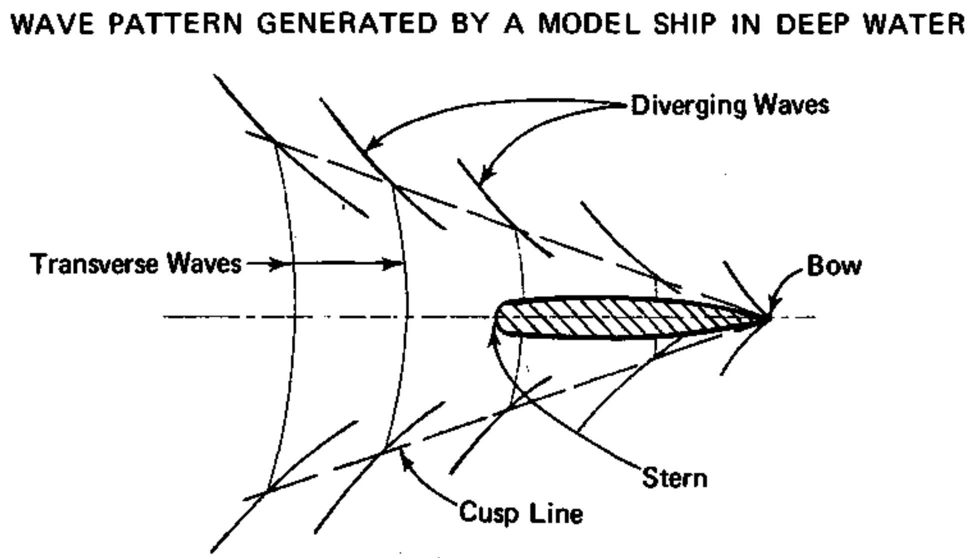

Figure 1. Wave pattern generated by a model ship in deep water

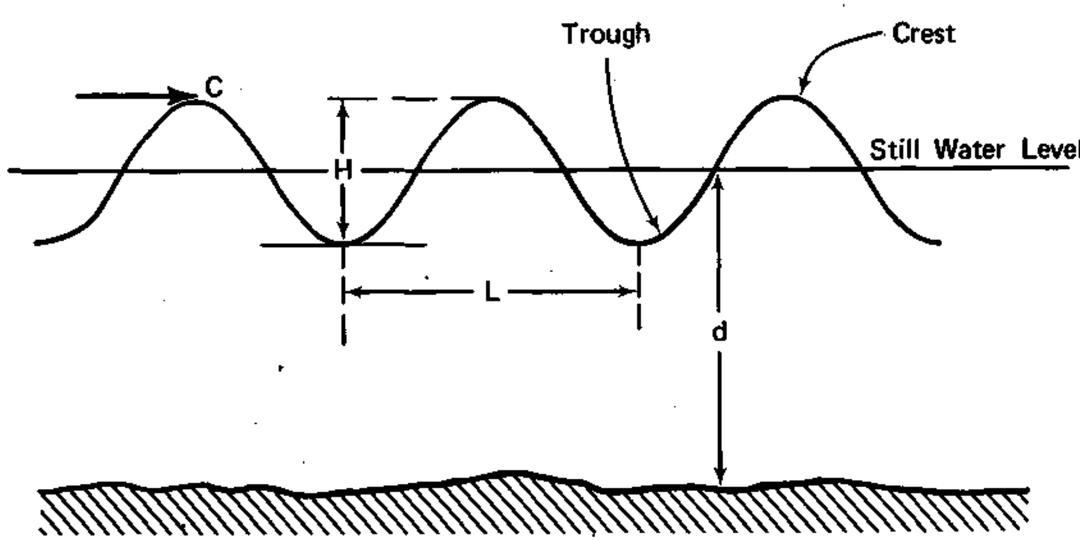

Figure 2. Descriptive sketch of a wave profile

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=l, 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 so narrow 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).

Waves Generated by River Traffic

There has been very limited research in the area of waves generated by river traffic on restricted waterways. Most of the investigations have concerned waves generated by ships traveling in deep and unrestricted waters. The few investigations dealing with waves in restricted waterways were done mostly 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.

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

where

H = 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

A_m = bxD = the submerged cross-sectional area of the vessel in ft, where b = the width of the vessel in ft and

D = the draft of the vessel in ft

Another equation for estimating maximum wave height is given by Hochstein (U.S. Army Corps of Engineers, 1980) as follows:

$$H_{max} = 0.0448\ V^2 \left(\frac{D}{L}\right)^{0.5} \left(1 - \frac{bD}{A_c}\right)^{-2.5} \qquad (2)$$

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.

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. The decreases 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, both the squat and drawdown are generally assumed to be 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 had been traveling along the middle of the channel (Bouwmeester et al., 1977; Kaa, 1978).

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 modern vessels transporting larger cargo need to use channels and harbor entrances designed for smaller vessels.

As discussed earlier, 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. A schematic representation of the drawdown phenomenon is shown in figure 3. Δh is the drawdown; D is the draft; Y is the hydraulic depth; b, L, and A_m are the width, length, and submerged cross-sectional area of the vessel; Z is the distance from the vessel to the water level monitoring device.

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

$$\Delta h = \frac{(V + \Delta V)^2 - V^2}{2g} \tag{3}$$

where ΔV is the backflow velocity beneath the vessel, V is the vessel velocity relative to the water, and the other variables are as defined earlier. Equation 3 is the Bernoulli equation, which states that increase in kinetic energy 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

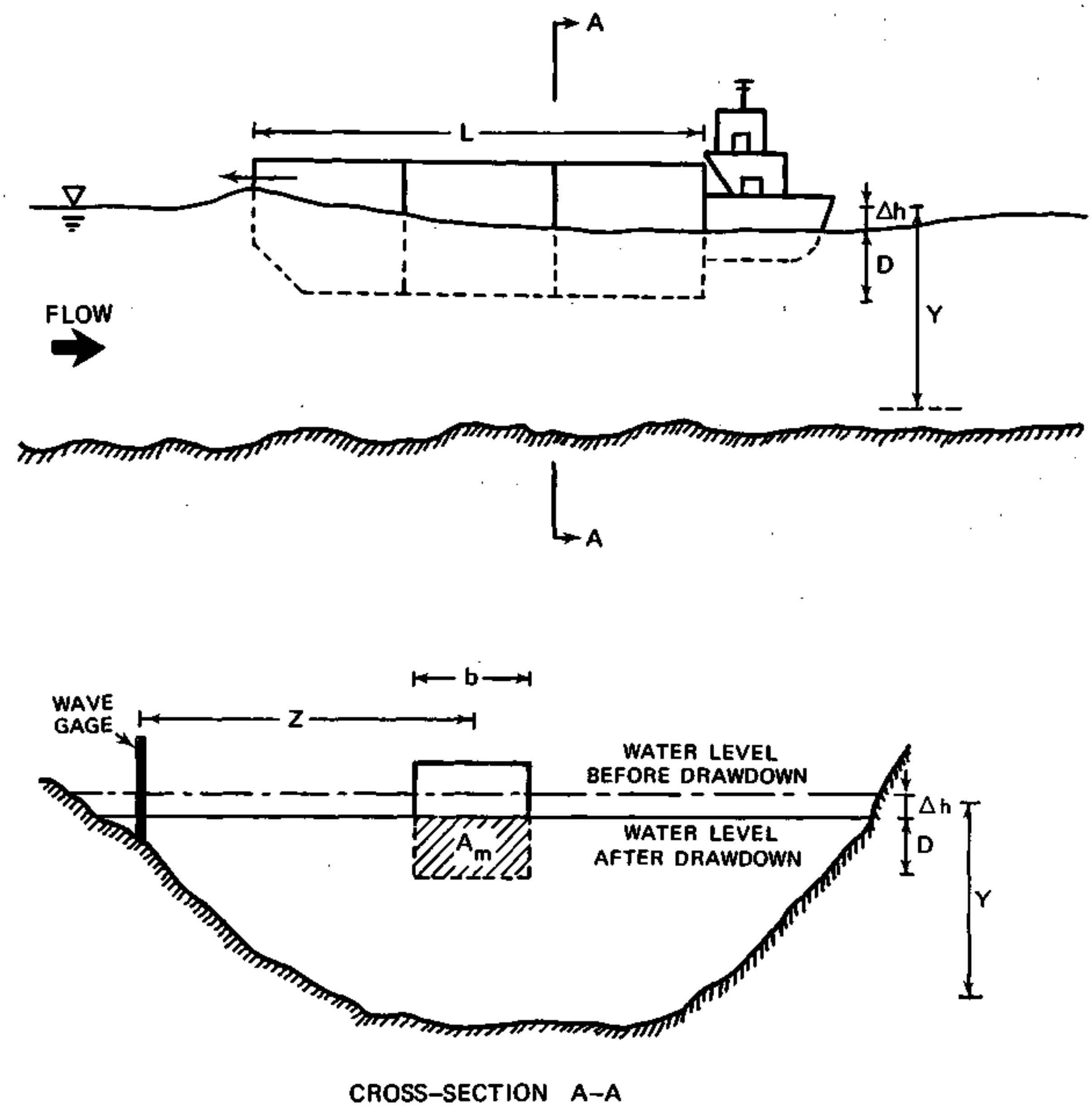

Figure 3. Schematic representation of the drawdown phenomenon

side represents the decrease in potential energy. The backflow velocity, V, is computed from the one-dimensional continuity equation given as follows:

$$A_c \times V = [A_c - A_m - (\Delta h \times B_c)](V + \Delta V) \tag{4}$$

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 (1977) from prototype and model test results. His equation relates the drawdown to the vessel's length, velocity, and the channel's cross- sectional area and distance from the sailing line as follows:

$$\Delta h = 2 \times 10^{-6} \left[\left(\frac{V^2 \times A_m \times L^2}{Z \sqrt{A_c}} \right)^{1/3} \right]^{2.8} \tag{5}$$

where Z is the 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 that 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 (1977). Dand's equation was based on scale ship model experiments and is given as follows:

$$\Delta h = 8.8 \left(\frac{A_c}{A_m} \right)^{-1.4} \times \left(\frac{V^2}{2g} \right) \tag{6}$$

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 (Tothill, 1966) and the David Taylor Model Basin (Garthune et al., 1948). The equation is given as:

$$\Delta h = \left(\frac{V^2}{22.6}\right)\left(\left(\frac{A_c}{A'}\right)^2 - 1\right) \tag{7}$$

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.

Wind-Generated Waves

As mentioned in the introduction, waves generated by wind are among the major causes of bank erosion in inland waterways. As opposed to the lack of research on waves generated by river traffic, there has been extensive research in wind-generated waves and as a result there are several methodologies to calculate wind waves. For engineering applications, however, the most widely used method of computing wind wave heights is the Significant Wave Height method, referred to as the Sverdrup-Munk-Bretschneider (S-M-B) method after the researchers who made a significant contribution to its development. The significant wave height is defined statistically as the average wave height of the highest one-third waves observed during a given period. The Significant Wave Height method was first developed by Sverdrup and Munk (1947) for predicting ocean waves. The significant wave height, H_s, wave celerity, C, and wave period, T, were expressed as functions of fetch, F, wind velocity, U, and wind duration, t. The fetch is the length of the reach of water over which the wind blows at a velocity of U, as shown in figure

4. The wind duration is the period during which the wind velocity, U, remains constant.

Bretschneider (1952, 1954, 1958) later modified the Sverdrup-Munk analysis to account for limiting fetch, length, wind duration, and some cases of shallow water. The S-M-B relation which is given in graphical form is approximated by the equation (Bhowmik, 1976):

$$gH_s/U^2 = 3.23 \times 10^{-3} (gF/U^2)^{0.435} \tag{8}$$

for $gF/U^2 < 3 \times 10^4$.

The S-M-B relation is good when the width of the fetch is of the same magnitude as the length. Saville (1954) developed a method to correct the fetch length for limiting fetch width. For a width to fetch, W/F, ratio of 0.05 to 0.6 as shown in figure 4, the effective fetch, F_e, can be approximated by the equation:

$$F_e = 1.054\, W^{0.6}\, F^{0.4} \tag{9}$$

When the wind direction is not coincident with the stream alignment, the effective wind velocity, U_e, is approximated by the following relation (Carlson and Sayre, 1961):

$$U_e = U \cos \phi \tag{10}$$

where ø is the angle between the wind direction and the stream bearing. The above relation is recommended for ø less than 45°. The effective wind velocity should also be corrected for stream velocity, since wave generation should depend on the wind velocity relative to the water surface. The effective wind velocity for streams can be modified similar to that of canals as suggested by Carlson and Sayre (1961):

$$U_e = U \cos \phi + \overline{V} \tag{11}$$

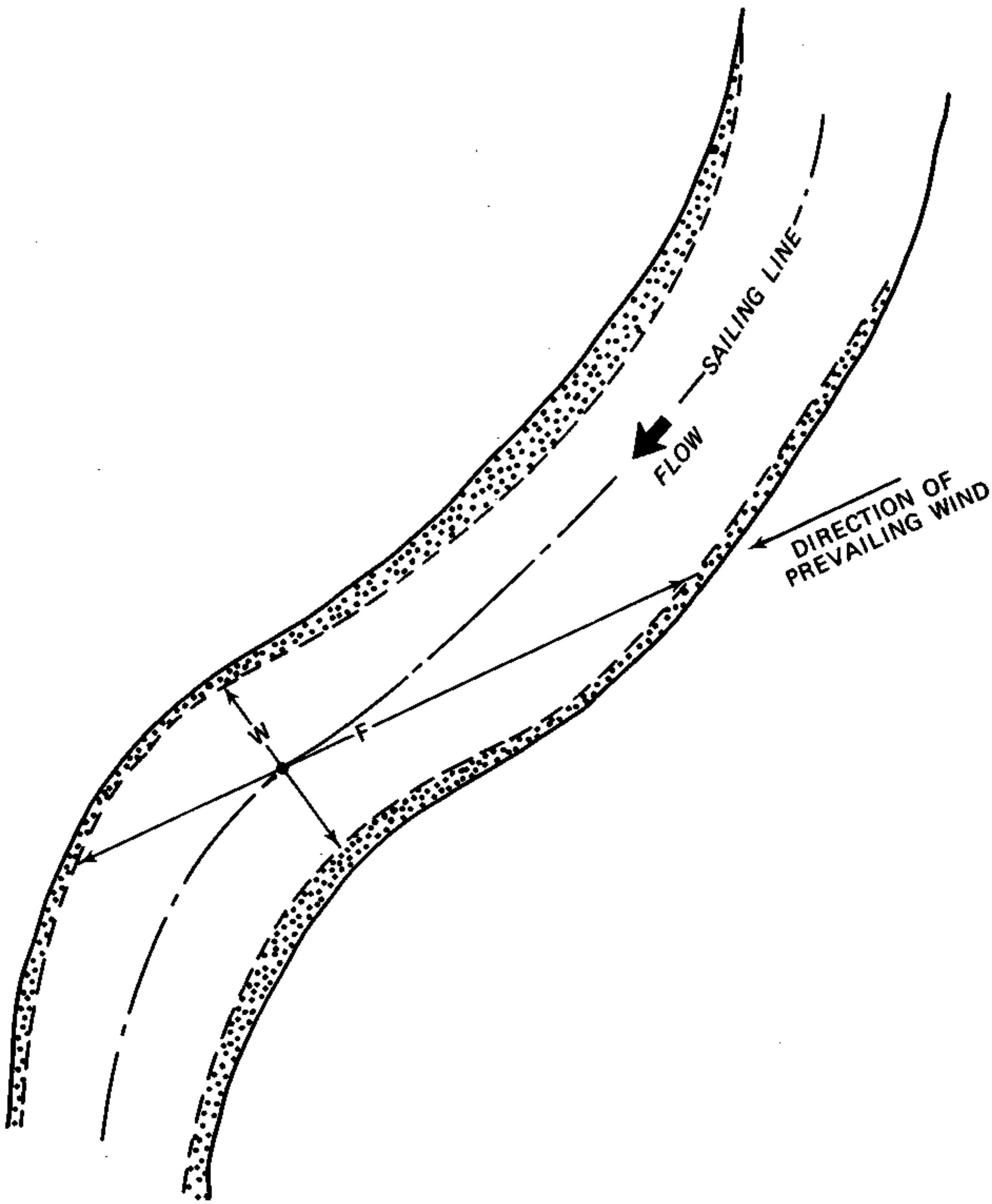

Figure 4. Sketch showing fetch for a river

where V is the stream velocity. V is positive when the wind is blowing in the general upstream direction, and is negative when the wind is blowing in the downstream direction. This correction should be applied when ø is less than 45°. Equation 8 can now be rewritten in a final form by replacing the wind velocity, U, and the fetch length, F, by their corresponding effective values as follows:

$$gH_s/U_e^2 = 3.23 \times 10^{-3}(gF_e/U_e^2)^{0.435} \qquad (12)$$

The significant wave height, H_s, can therefore be calculated from equation 12 when the effective fetch, F_e, and the effective wind velocity, U_e, are known.

DATA COLLECTION PROGRAM

Site Selection and Description

The criteria for selecting study sites are listed in table 1. The favorable site characteristics were divided into two categories: primary and secondary site characteristics. The primary criteria were related mainly to the channel geometry and alignment and 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 a shore area suitable for installing wave gages were considered important.

After a review of hydrographic maps (U.S. Corps of Engineers, 1971, 1977), topographic maps, and navigation charts of the Illinois and Mississippi Rivers (U.S. Army Corps of Engineers, 1974, 1978), several sites were identified as possible locations for site-specific studies according to the criteria established in table 1. Aerial reconnaissances of these sites were 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 reconnaissances, 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 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 5.

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

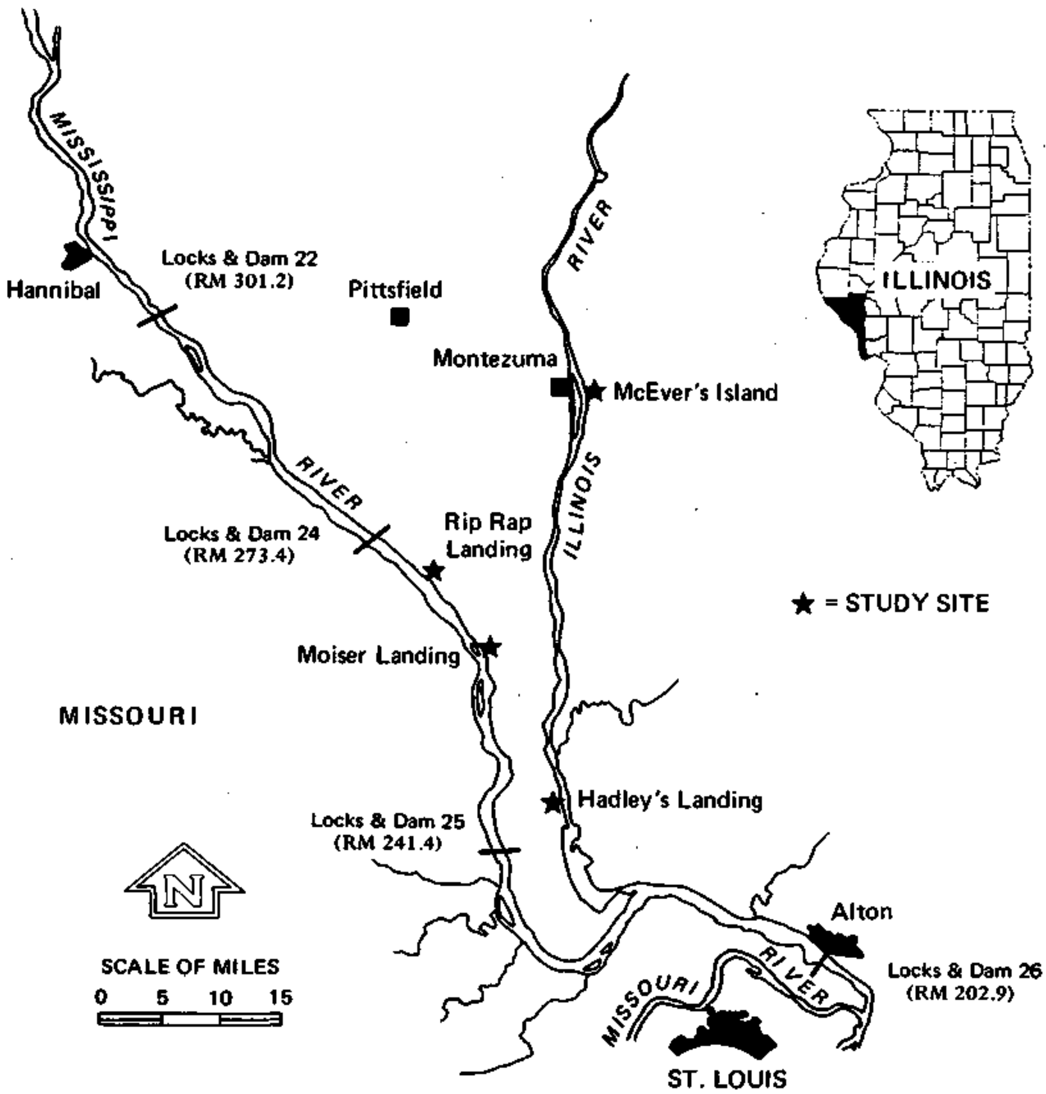

Figure 5. Locations of study sites on the Mississippi and Illinois Rivers

Table 2. Names and Locations of Study Sites

Site no.	Site name	River	River mile
1	Hadley's Landing	Illinois	13.2
2	McEver's Island	Illinois	50.0
3	Rip Rap Landing	Mississippi	265.1
4	Mosier Landing	Mississippi	260.2

The Hadley's Landing site, shown in detail in figure 6, 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 the Hadley's Landing test site is shown in the upper portion of figure 7. Also shown in the figure are the values for the discharge, Q, the cross-sectional area, A, and the mean velocity, V, during the field trips to the 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 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 the lower portion of figure 7.

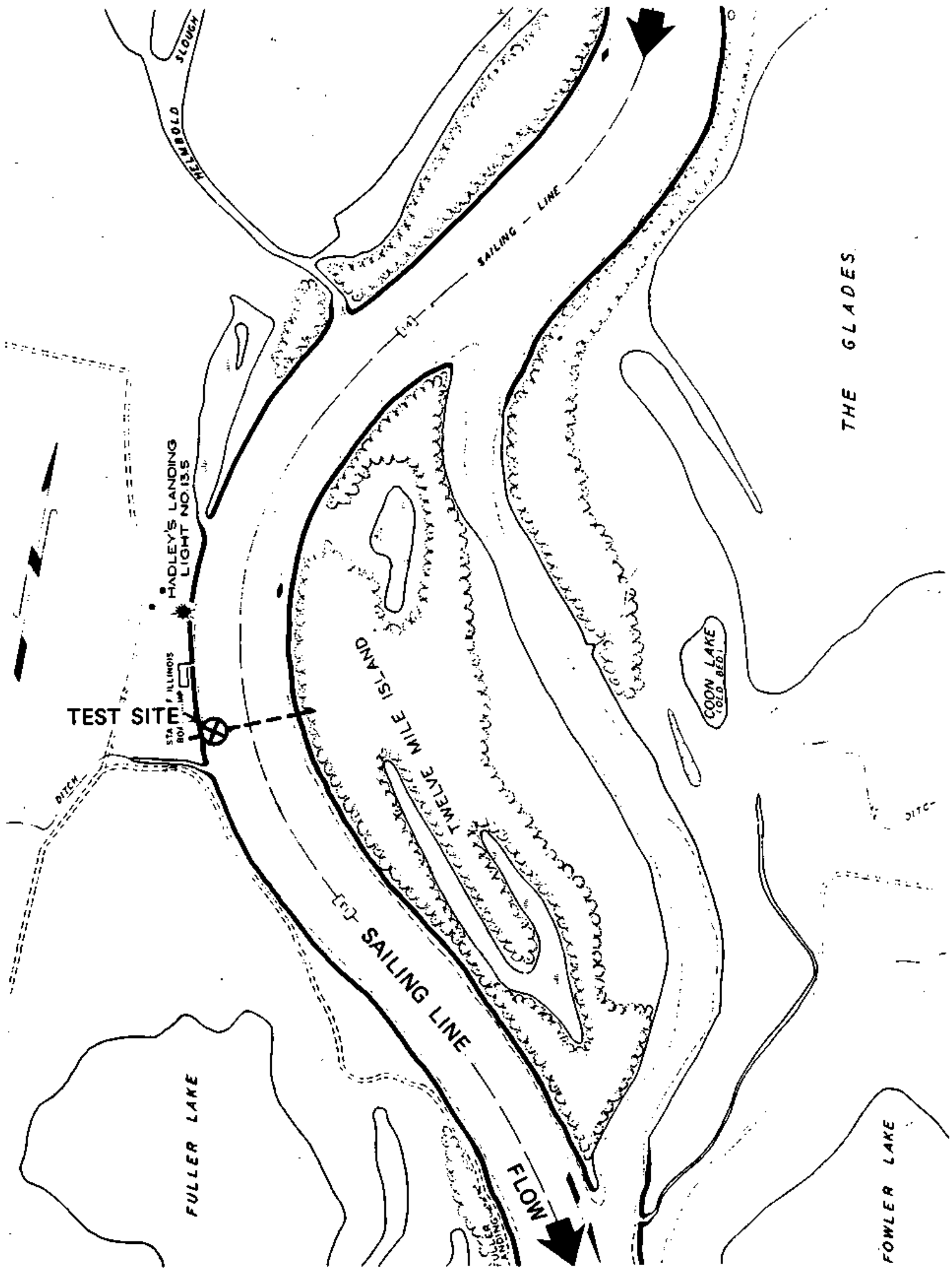

Figure 6. Location of Hadley's Landing test site on the Illinois River

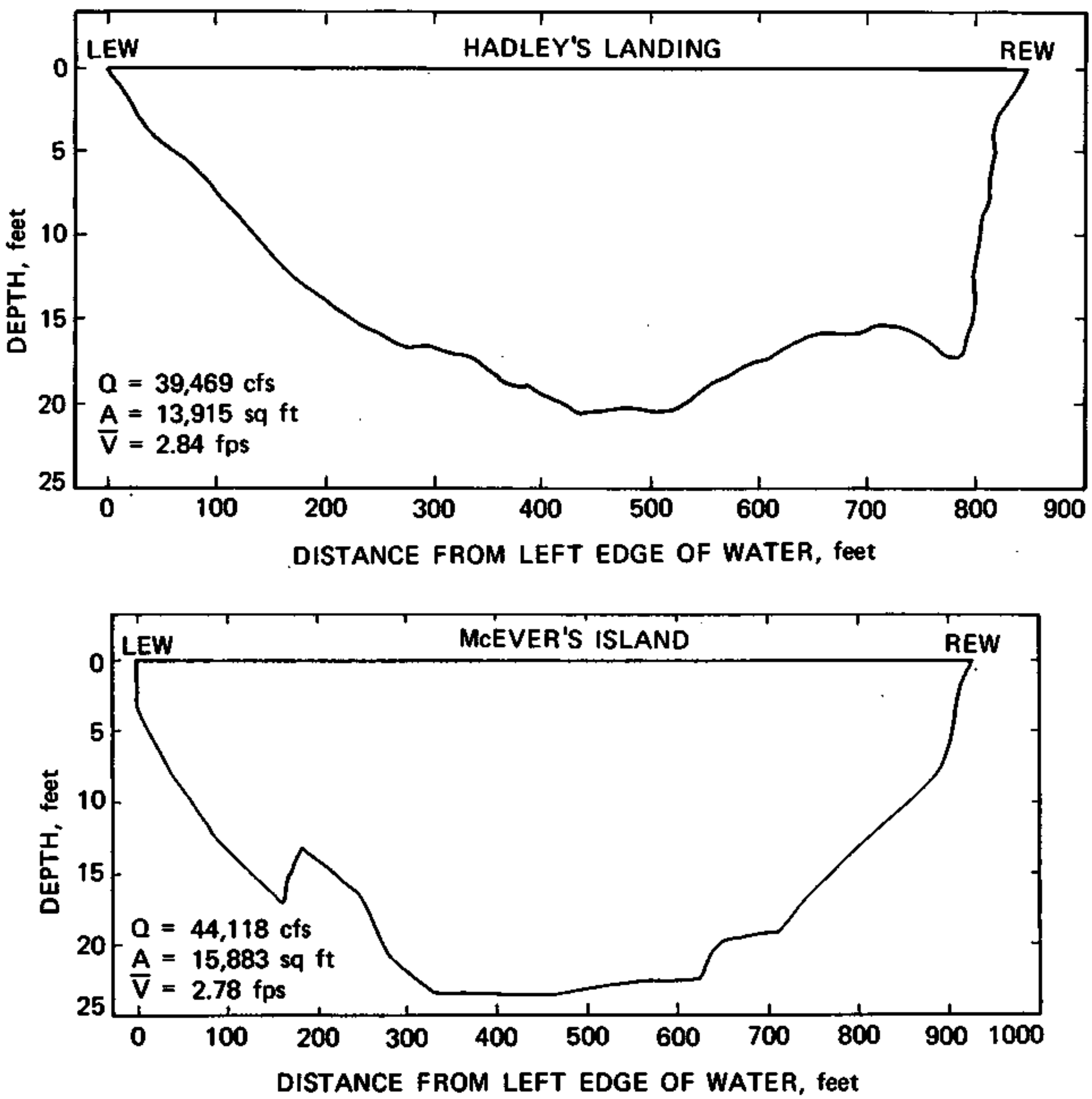

Figure 7. Cross-sectional profiles of the Hadley's Landing and McEver's Island test sites

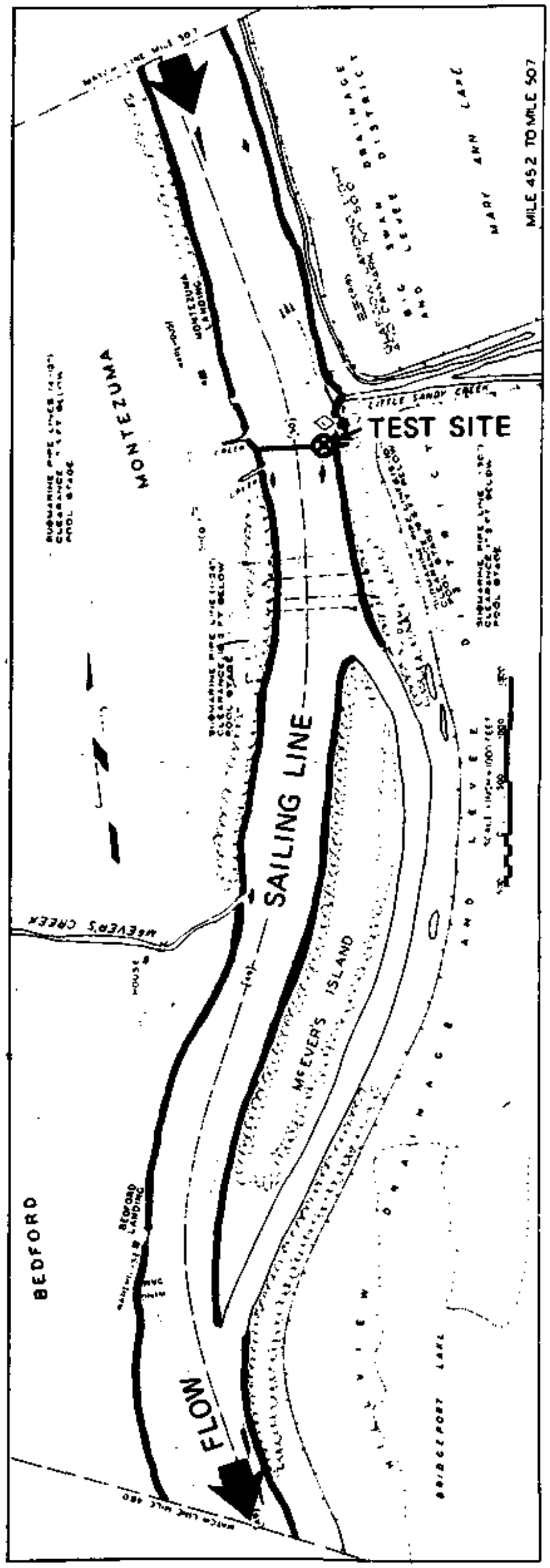

Figure 8. Location of the McEver's Island test site on the Illinois River

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.

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

The Rip Rap Landing site, shown in figure 9, 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 the upper portion of figure 10. As shown in figure 10, the main channel is on the outside of the bend close to the left 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 a 7.5 mile trip by boat from a private marina.

The Mosier Landing site, shown in figure 11, 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.

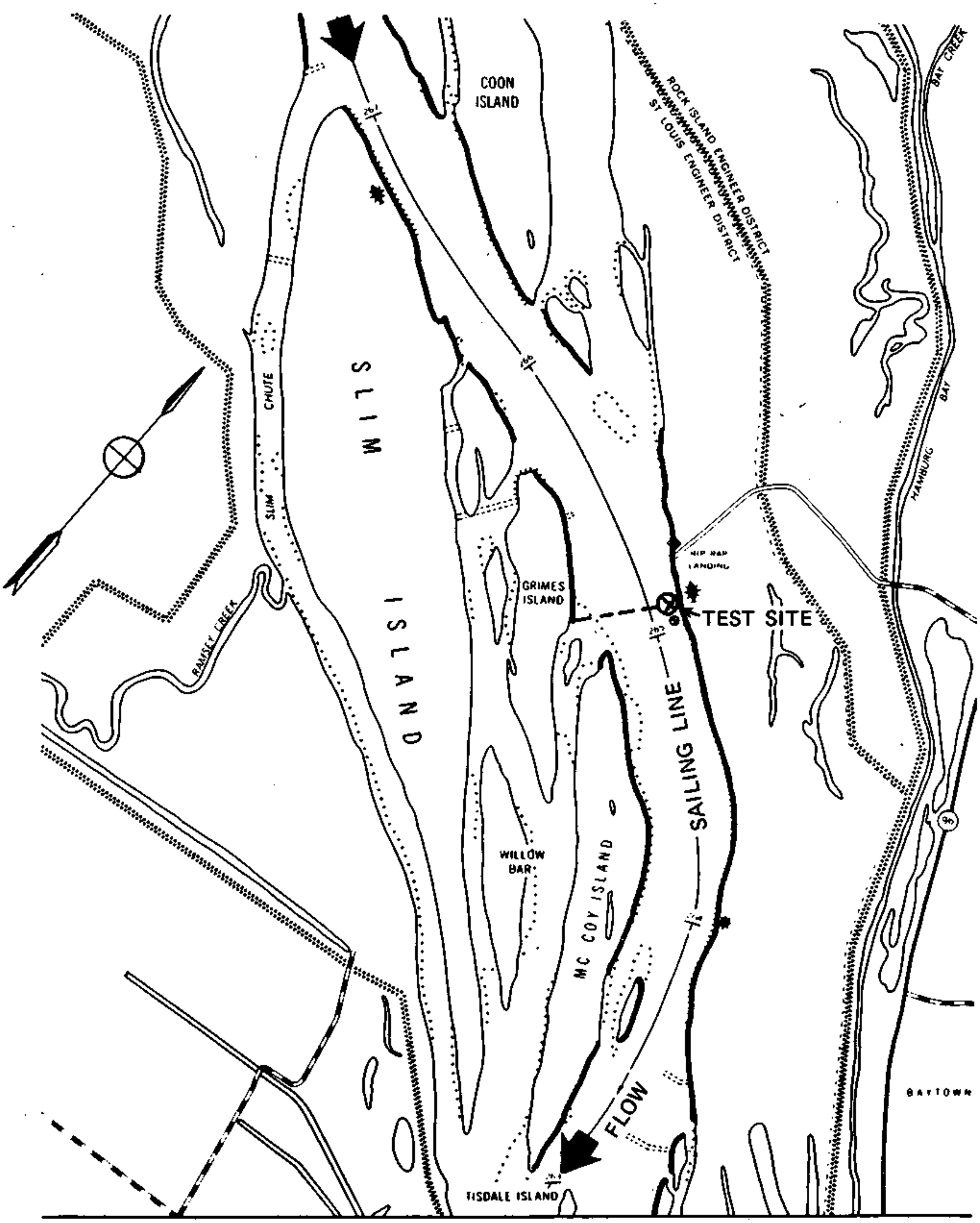

Figure 9. Location of Rip Rap Landing test site on the Mississippi River

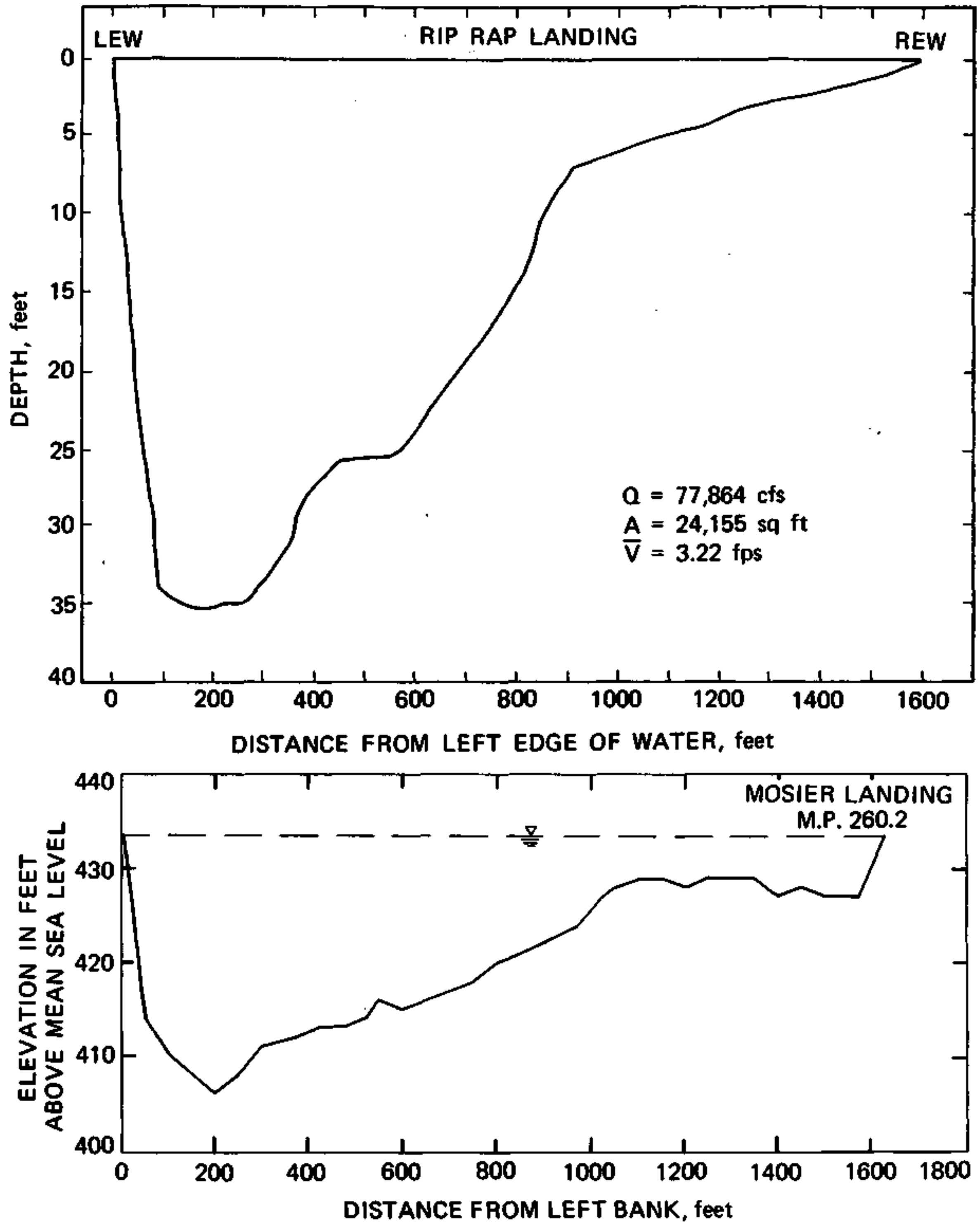

Figure 10. Cross-sectional profiles of the Rip Rap Landing and Mosier Landing test sites

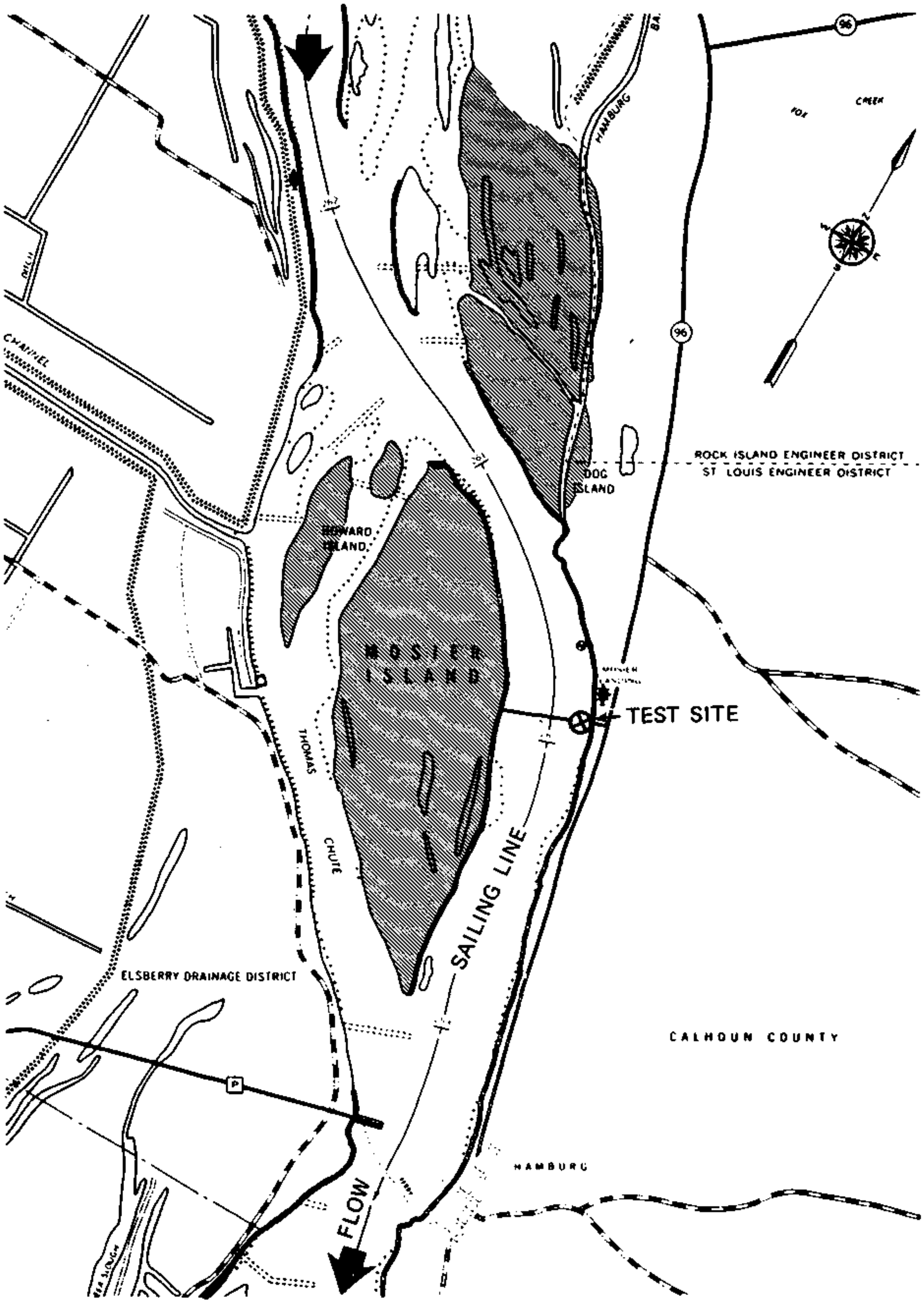

Figure 11. Location of the Mosier Landing test site on the Mississippi River

The cross-sectional profile at the test site during normal pool level is shown in the lower portion of figure 10. 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 two 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 the 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.

The second group of instruments were the surveying instruments for measuring vessel speed, track of tow, and distance of tow from the shore. 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

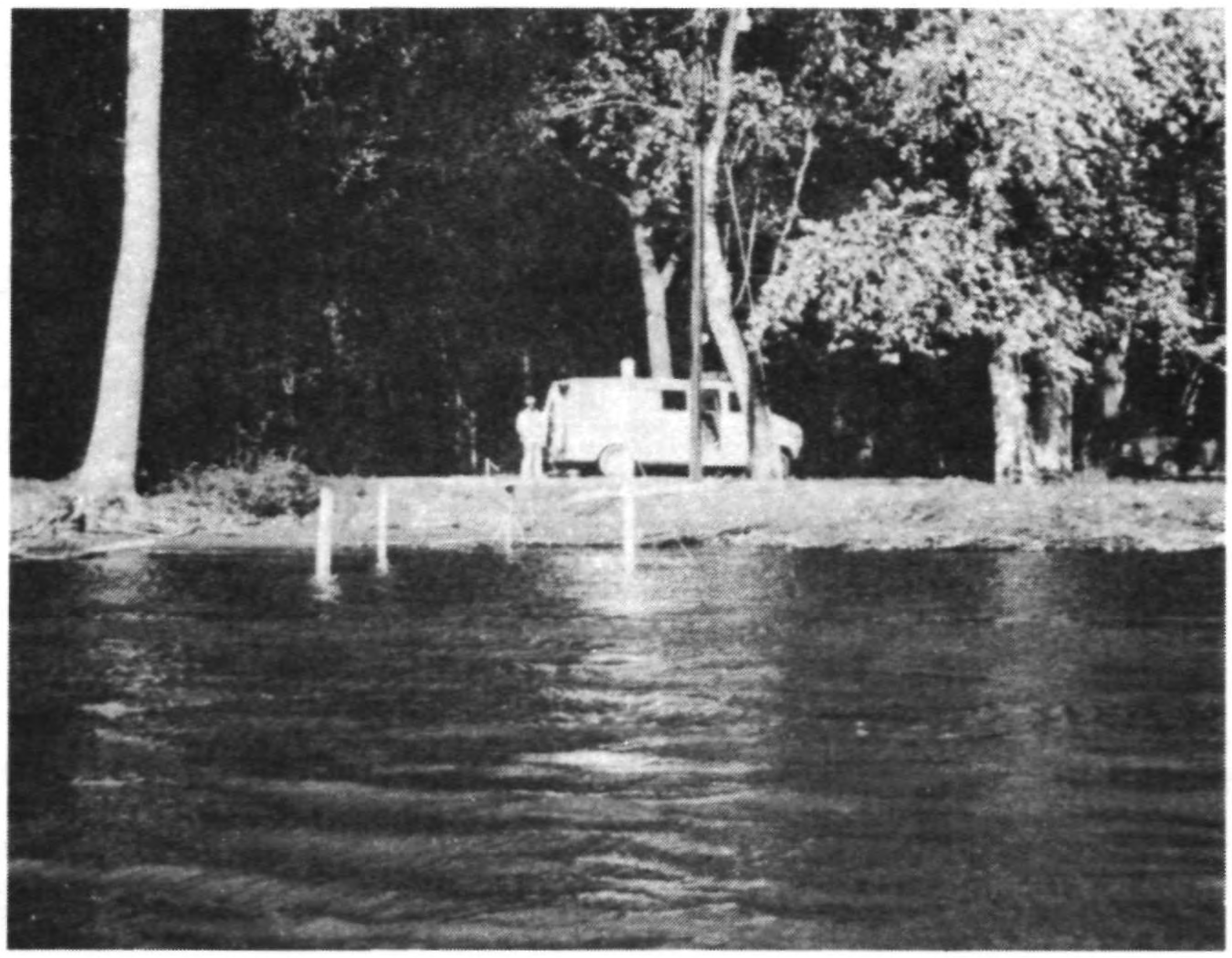

Figure 12. Typical wave instrument set-up during field data collection periods

fence post. An attempt is always made 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 13.

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 to avoid its being on the dark side of the staff gage. The camera can 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 14 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 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 analysis of 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.

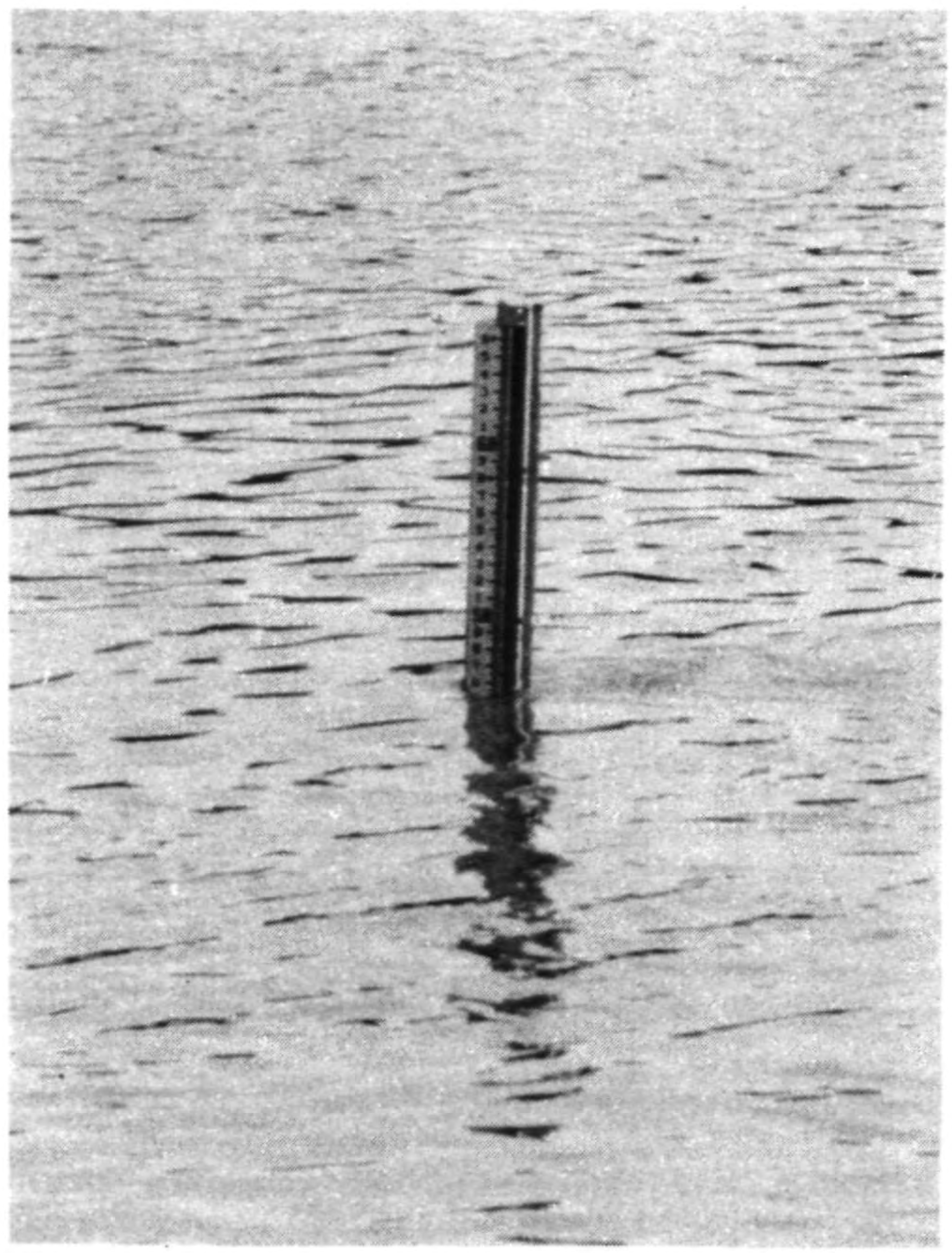

Figure 13. Staff gage in calm water

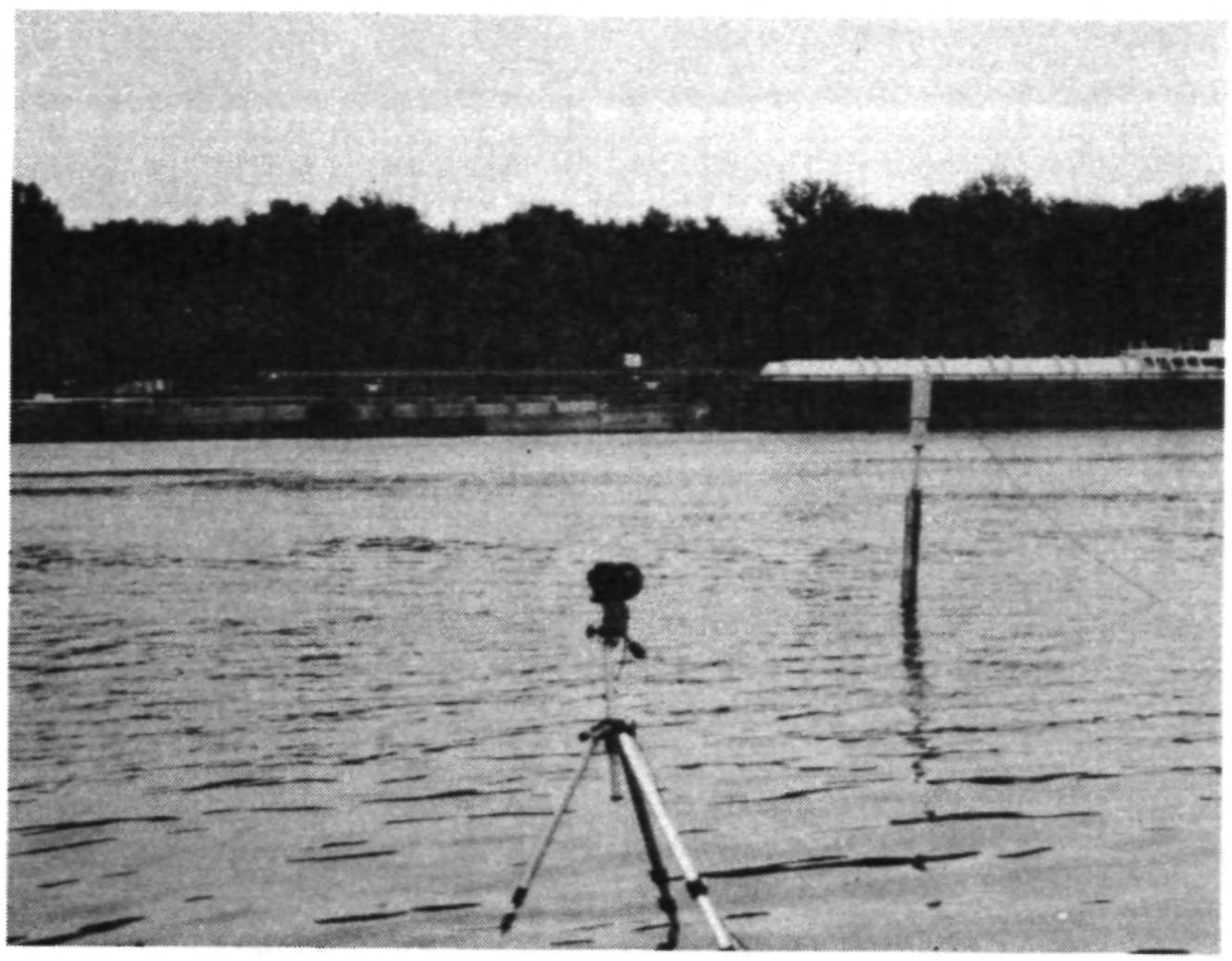

Figure 14. Movie camera and staff gage during data collection period

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, a 3 ft sensor grid, and an electronics package (figure 15). 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 1 foot tall and is split approximately in half by a threaded section to permit access to the electronics. In the lower half section is a 35 pin-connector that is used to interconnect the gage and the interface on shore. The wires from the sensor grid case to the electronics package compartment and from that compartment to the connector are sealed with silicon caulking compound to prevent moisture from entering.

The sensor grid consists of 3 1-foot-long by 2-1/4-inch-wide single-sided 1/16-inch-thick copper-clad boards. These boards have been etched to form a pattern of fingers spaced 1/20 of a foot apart (figure 15). 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 sides of the boards are painted with spar varnish except for approximately 1/16 inch of contact end. This

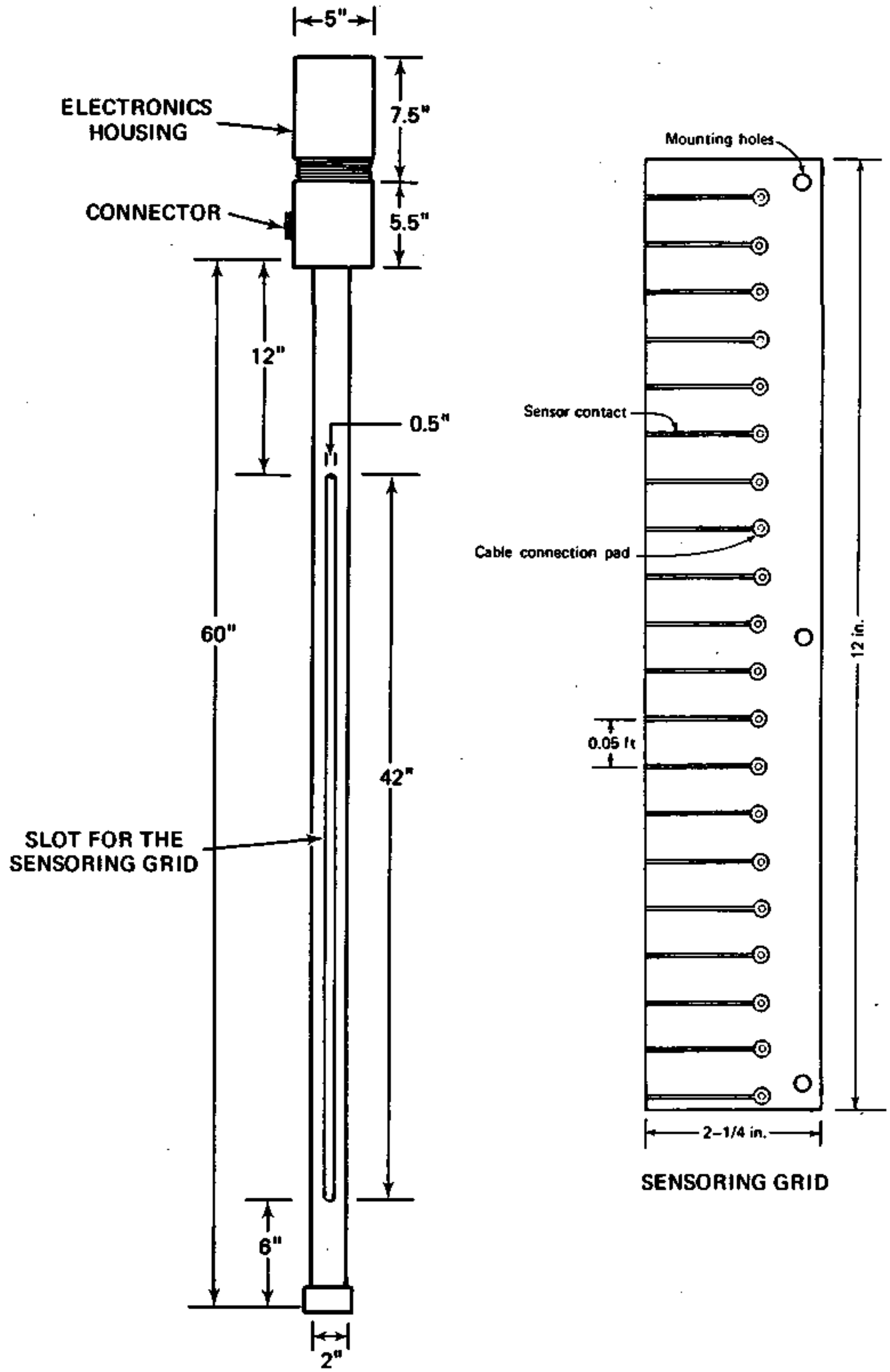

Figure 15. Electronic wave gage

protects the solder joint and makes the contact area small to help accuracy. The 3 boards are then lined up to make a 3-foot sensor grid. This grid is then sandwiched between 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 it 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 to 3-inch, 3-inch to 2-inch reducers, and a 4-inch threaded coupling. This allows the pipe diameter to increase to 4 inches to house the electronics more easily. There are two circuit boards, as shown in figure 16, and a 4-inch aluminum disk separating the boards by 2 inches. The disk has a 2-1/4-inch hole in its center to pass cables from the sensor grid and the 35-pin connector 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 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.

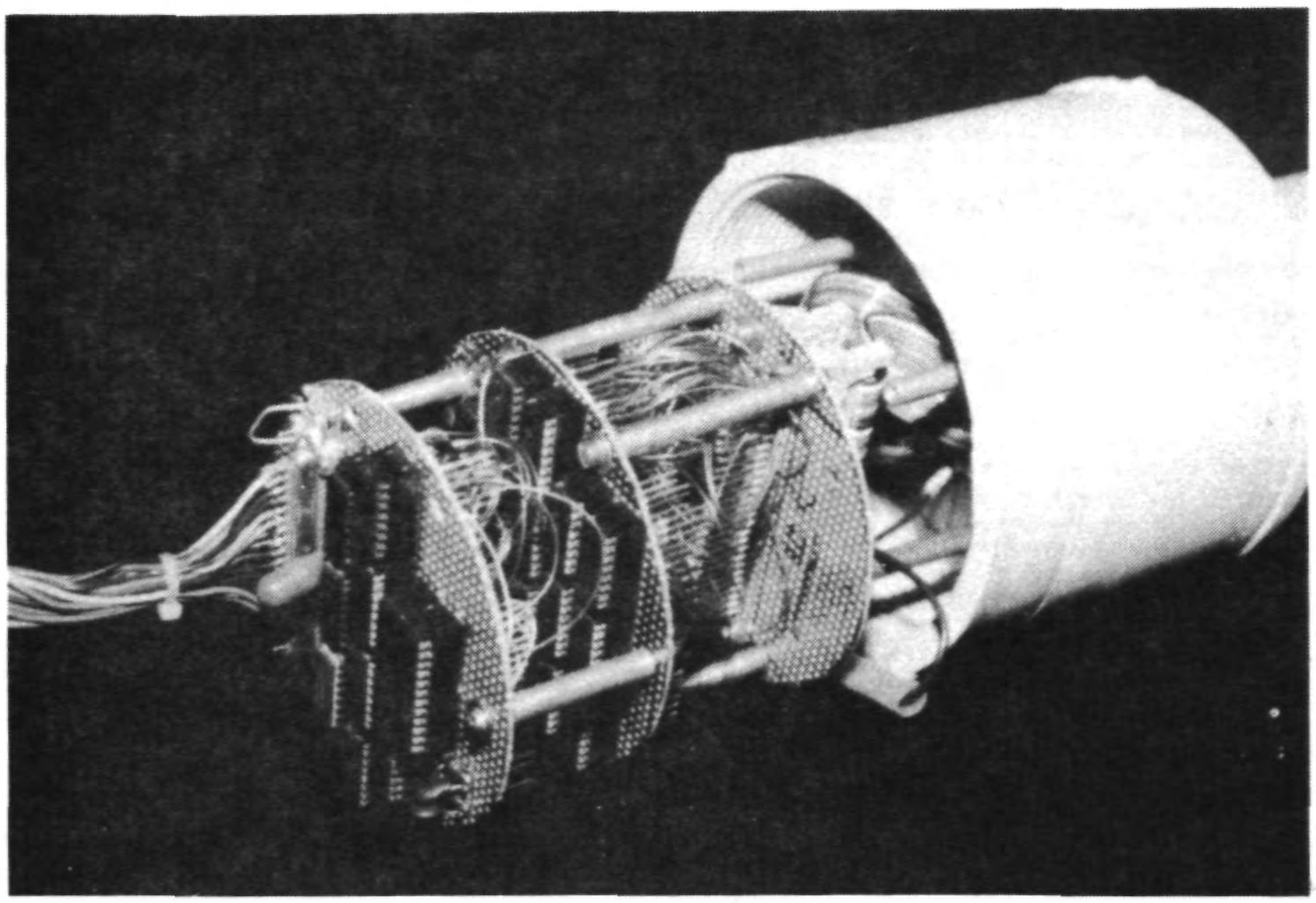

Figure 16. Electronic circuit boards

The use of PVC pipe makes for an easily constructed instrument that is lightweight, waterproof, corrosion proof, and strong.

The wave gage receives power and a 1 KHz clocking signal from the wave gage interface via a 100-foot, 15 pair twisted 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 generates 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 17 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 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

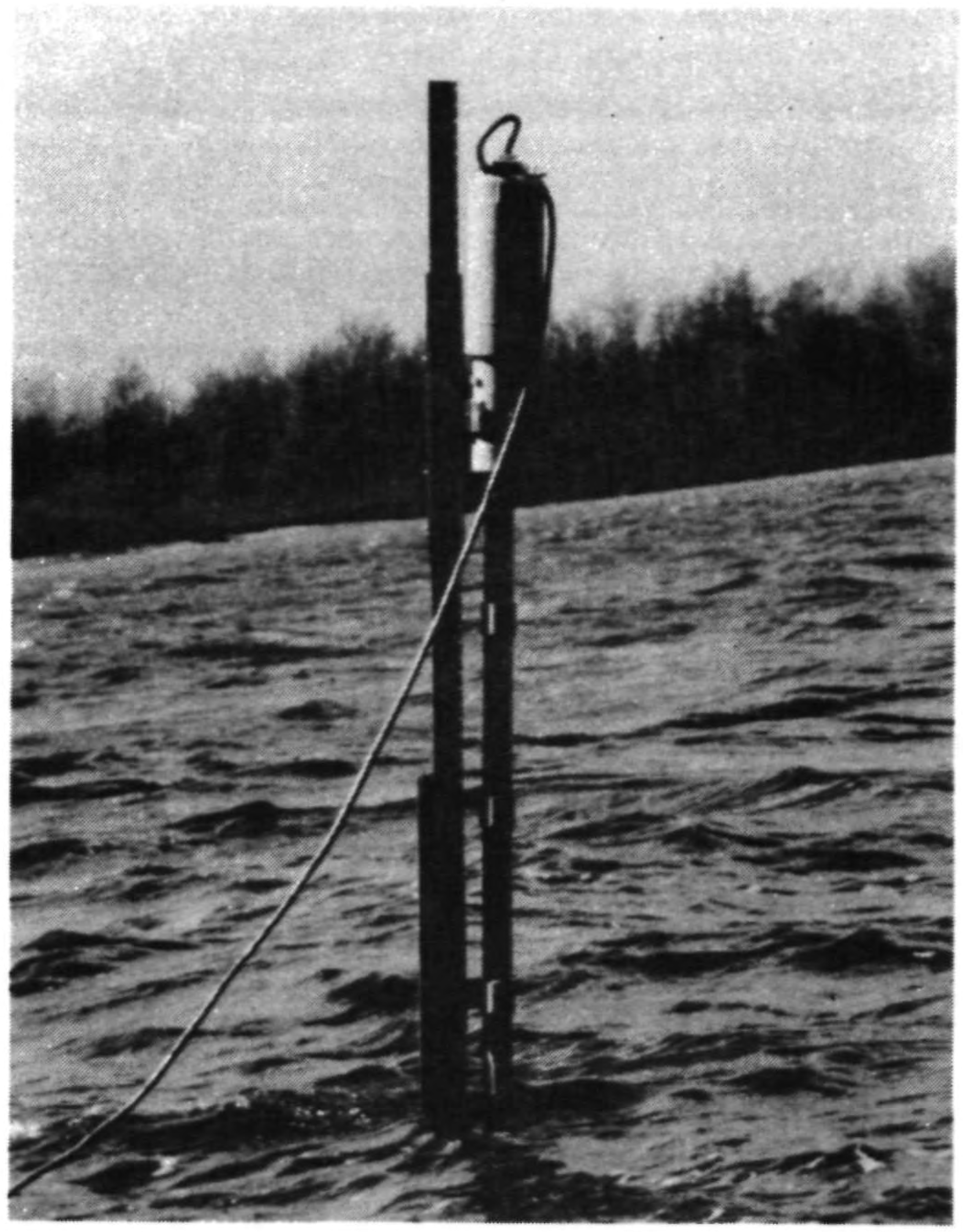

Figure 17. Installed wave gage at a test site

interface between the computer and the wave gages. The computer, the cassette drive, and the IEEE interface are shown in figure 18 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 if available at the sites.

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 19. 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 them in memory. At the end of an event, the data are stored on cassette tapes. The data can also be printed on paper for inspection. Later the mini-computer sends the data to the CYBER computer through the modem and phone lines for further analysis.

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

Figure 18. Mini-computer and its peripherals during data collection period

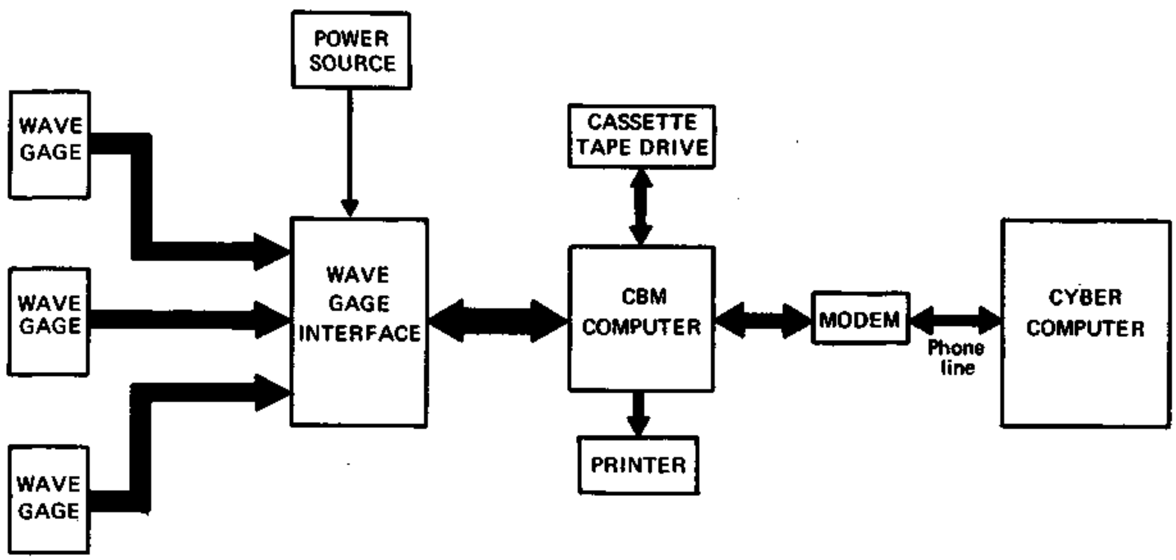

Figure 19. Schematic of the wave and drawdown measuring instruments

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 diagram of the field conditions and the data that need to be collected is presented in figure 20. As a tow passes a test 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 on 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 is collected.

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

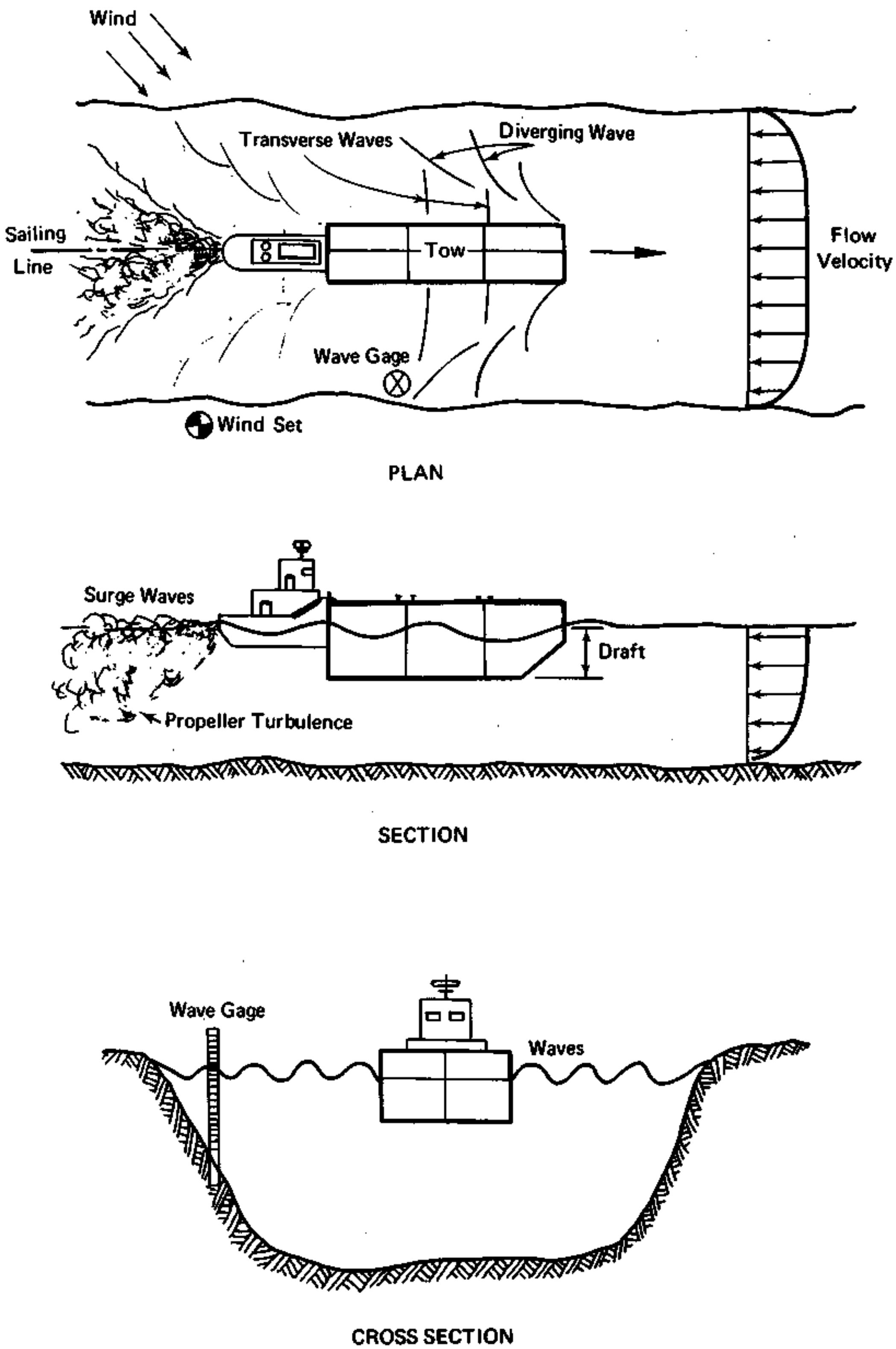

Figure 20. Illustration of field conditions during a tow passage

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 elevations. 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 figure 11. Therefore, it can be assumed that the flow conditions at both sites are also similar. The channel profile for Mosier Landing was taken from the U.S. Army 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 21) 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 of the total depth 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 and McEver's Island are shown in figure 7, and that of Rip Rap Landing is shown in figure 10. 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

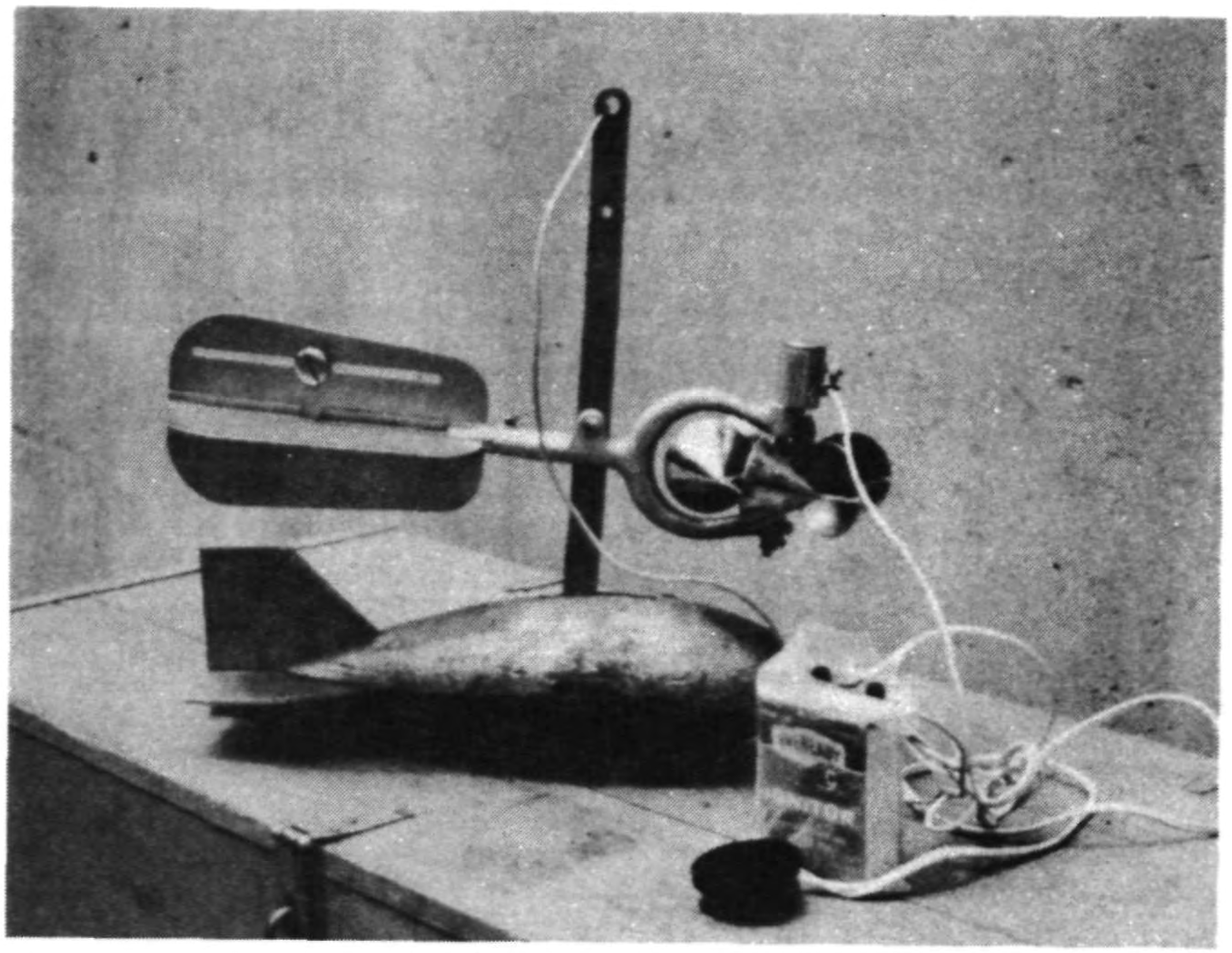

Figure 21. Velocity measuring instrument

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 the Hadley's Landing test site, the discharge, Q, the cross-sectional area, A, and the average velocity, V, were 39,469 cfs, 13,915 sq ft, and 2.84 ft/sec, respectively. Also, from figure 7 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.

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 sq ft, and 3.22 ft/sec, respectively. The top width of the channel from figure 10 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 sq ft, and 2.78 ft/sec, respectively. From figure 7, 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 22. Information on such variables as the vessel type, size, draft, distance from shore, and direction of movement was 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 in the Inland River Record (Owen, 1981).

Table 3. Velocity and Discharge Measurements 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 Measurements 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 Measurements 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

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'
Air temperature:	Displacement
Remarks:	Velocity 12.8 fps
	Distance from shore 175 ft
	Track (attachment)
	RPM
	# of barges 9 (3x3)
Wind data (Mark recorder chart)	Remarks:

Time	Speed	Direction

Figure 22. Sample data sheet

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 x 26 feet for the smallest tow to 180 feet x 52 feet for the largest. The size, number of propellers, power, and nozzle type for most of the tows sited during all the field trips is summarized in table 6.

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 survey system was used to determine the track and distance from shore of the vessel. A baseline of sufficient length, usually 800 to 1500 feet, was established on one shore adjacent to the test site. A semi-permanent marker was set at each end of this line.

Table 6. Tow Characteristics

Name of tow boat	No. of propellers	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	

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 23a.

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 1/2-to 1-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 1000 feet above or below the test sites.

After the baseline location was established, 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 23a. 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

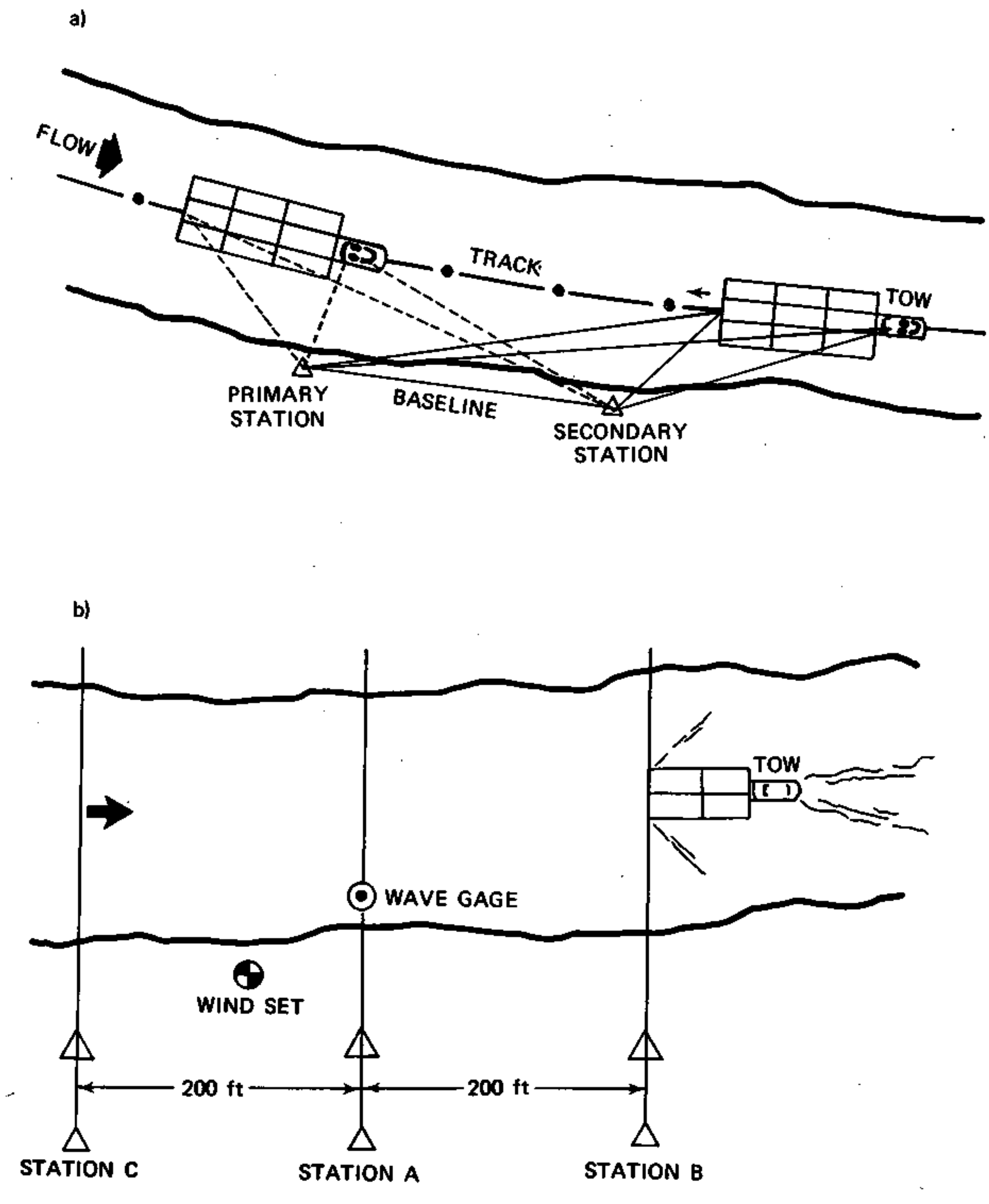

Figure 23. Surveying procedures

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 1 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 flagstaff.

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 impression 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 1 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. After measuring 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 24. It was also possible to determine the distance of the tow from the shore line.

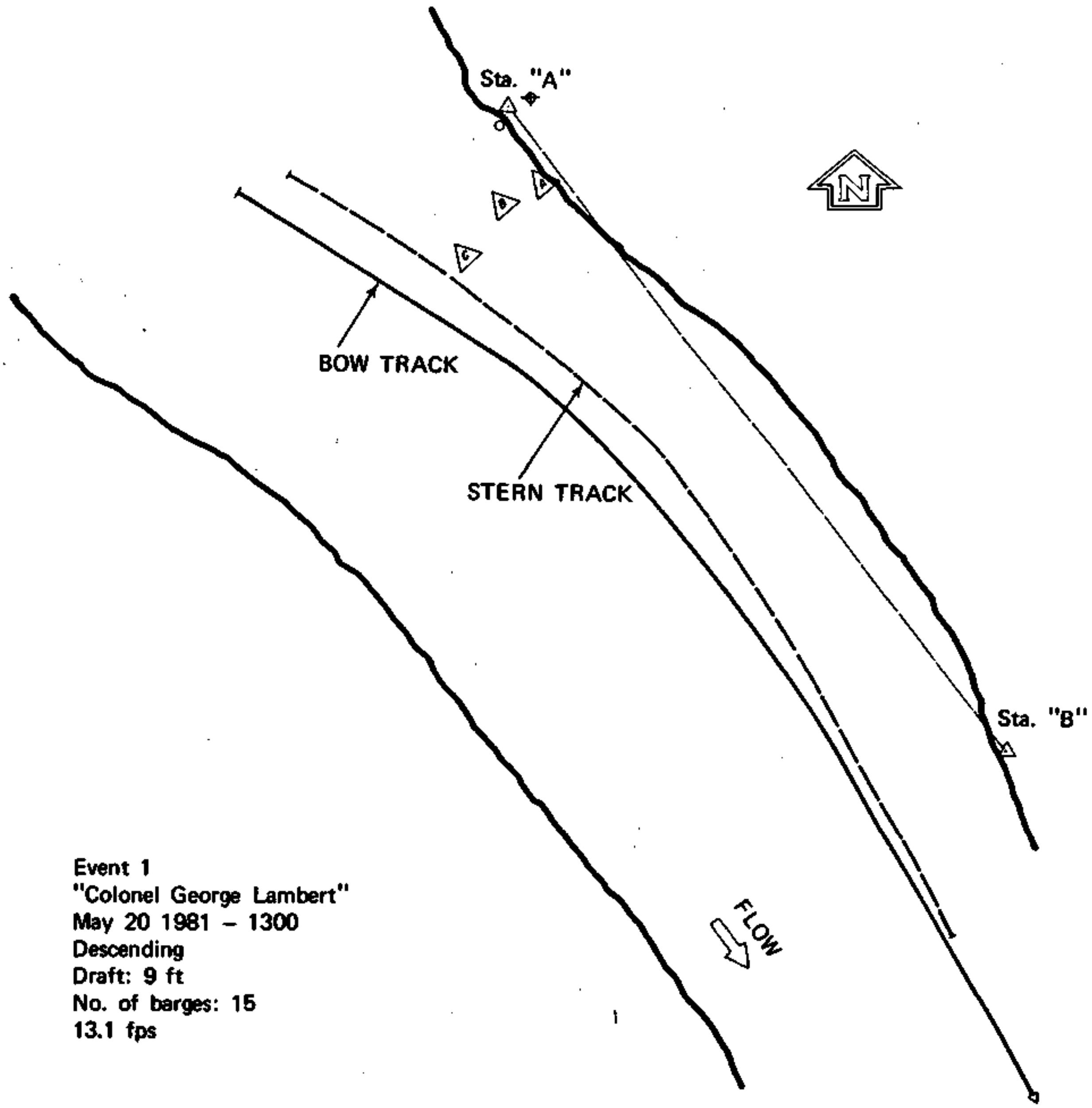

Figure 24. Tow tracking data at Mosier Landing test site

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 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 23b, 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 23b, 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 di stance 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 to determine drawdown while tows were passing the section, and shortly afterwards movies were taken to obtain wave data.

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 others did not. Each 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 6480 frames of water level readings. After the water elevations are read from the films, they are entered in 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 easier than the movie method. About 5 minutes before an event is to commence, the computer is turned on and a master computer program is 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 by asking for two lines of title description such as trip and event number, date, time, and site description. It is then ready to sample. When "G" on the keyboard is pressed, 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 when the letter "W" on the keyboard is pressed. The computer program also has a means of keeping the time the bow and stern of the vessel passed the test cross section. When the letters "B" and "S" are pushed for the bow and stern, respectively, the time the bow and stern passed the cross section is kept in the data file. When the length of the vessel is known, 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 when the letter "E" on the keyboard is pushed. After the termination of the data collection, the computer processes the data, arranges it in a desirable tabular form, and stores them in memory. The computer then asks if the data should be stored onto a cassette tape. When the cassette tape is rewound and the "RETURN" key on the computer keyboard is pressed, the data is transferred to a cassette tape. If another tape is required, the computer will ask for another tape until all the data are 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 six field trips to collect wave and drawdown data. Four of the field trips were to the Illinois River, and two were to the Mississippi River. 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 terms of making the proper adjustments in the positioning of the movie camera and the staff gage on the subsequent field trips.

Summary of Field Trips

Summaries of the six field trips and the events where wave and drawdown data were collected are given in table 7. In the table, the trip and event numbers, the number of barges, direction, wave gage distance, draft, and speed of the tow are indicated along with the measured maximum wave height and drawdown.

Trip 1 was to Hadley's Landing on the Illinois River, river mile 13.2, on July 22-24, 1980. There were a total of 5 events of which 4 of the events were downstream traffic and one upstream. The number of barges pushed by the tows varied from 6 to 15. The draft ranged from 2 to 9 feet and the speed from 5.7 to 7.9 ft/sec.

Trip 2 was again to Hadley's Landing on 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 and one tow was

Table 7. Summary of field data

TOW DESCRIPTIONS

TRIP NO	EVENT NO	DIRECTION U/D STREAM	WAVE GAGE DISTANCE (FT)	NO. OF BARGES	TOTAL LENGTH (FT)	TOTAL WIDTH (FT)	DRAFT (FT)	SPEED (FT/S)	CHANNEL X-AREA (SQ.FT)	BLOCAKAGE RATIO	MAX WAVE HT. (FT)	MAX DRAWDOWN (FT)
1	1	D	480.	6	585.	70.	8.	7.87	13915.	21.85	.15	--
1	2	D	405.	12	780.	105.	9.	5.70	13915.	14.72	.20	--
1	3	U	150.	15	975.	105.	2.	7.81	13915.	66.26	.30	--
1	1	D	150.	12	780.	105.	9.	6.12	13915.	11.72	.25	--
1	5	D	510.	15	975.	105.	9.	6.68	13915.	11.72	.10	--
2	2	D	150.	6	585.	70.	9.	8.16	13915.	22.09	.86	--
2	3	U	390.	11	975.	105.	9.	5.93	13915.	11.72	.20	.20
2	1	U	120.	18	1170.	105.	9.	7.05	13915.	11.72	.16	.13
2	5	U	150.	8	585.	105.	2.	7.98	13915.	66.26	.23	.11
2	6	D	510.	15	975.	105.	9.	8.51	13915.	11.72	.39	.13
2	7	D	525.	12	780.	105.	9.	6.01	13915.	11.72	.10	.20
2	9	U	135.	11	780.	70.	9.	3.19	13915.	22.09	.31	--
2	10	U	390.	12	780.	105.	9.	5.90	13915.	11.72	.80	.30
2	11	D	120.	12	780.	105.	9.	8.70	13915.	11.72	.18	.27
3	3	U	330.	9	585.	105.	2.	5.70	21155.	115.02	.10	.10
3	1	U	305.	15	975.	105.	2.	10.20	21155.	115.02	.15	.22
3	6	U	255.	16	975.	105.	2.	11.60	21155.	115.02	1.05	.30
3	7	D	280.	12	780.	105.	9.	11.90	21155.	25.56	.38	.11
3	9	D	330.	15	975.	105.	9.	12.80	21155.	25.56	.56	--
3	12	U	655.	15	975.	105.	2.	8.60	21155.	115.02	.51	.30
3	12	U	30.	13	780.	105.	9.	7.10	21155.	25.56	.81	.31
4	1	U	235.	1	390.	70.	2.	9.30	15883.	113.15	.15	--
4	2	D	225.	2	390.	35.	2.	12.50	15883.	226.90	.30	--
4	3	D	175.	12	780.	105.	9.	9.10	15883.	16.81	.31	.21
4	4	U	215.	15	975.	105.	2.	12.10	15883.	75.63	.11	.10
4	6	U	35.	10	585.	105.	2.	8.10	15883.	75.63	.35	.30
4	7	D	205.	1	390.	70.	2.	11.50	15883.	113.15	.62	.21
4	8	U	195.	15	1110.	70.	9.	8.80	15883.	25.21	.35	.10
4	9	U	165.	8	585.	70.	6.	11.50	15883.	11.25	.89	.27
4	10	D	160.	9	585.	105.	9.	12.80	15883.	16.81	.52	.21
1	11	D	150.	12	780.	105.	9.	12.70	15883.	16.81	.55	--
1	12	D	155.	9	585.	105.	7.	11.10	15883.	22.92	.72	.30
1	13	D	115.	15	975.	105.	9.	15.60	15883.	16.81	.96	.69
1	11	D	150.	1	195.	35.	1.	20.30	15883.	113.15	.93	.15
1	15	U	155.	15	975.	105.	2.	5.90	15883.	75.63	.30	.10
1	16	D	150.	12	780.	105.	9.	12.80	15883.	16.81	.11	.25
5	1	D	601.	12	780.	105.	9.	11.00	21000.	25.10	.20	.13
5	6	D	600.	12	780.	105.	9.	13.10	21000.	25.10	.50	--
5	9	U	688.	3	585.	50.	9.	9.80	21000.	53.33	.35	.05
5	10	D	720.	15	975.	105.	9.	13.30	21000.	25.10	.90	--
5	11	U	721.	15	975.	105.	2.	9.30	21000.	111.29	.50	--
6	1	U	150.	15	975.	105.	9.	6.00	13915.	11.72	.59	--
6	1	U	190.	15	975.	105.	2.	1.70	13915.	66.26	.11	--
6	6	D	280.	8	780.	70.	9.	5.20	13915.	22.09	.30	--
6	8	U	110.	12	780.	105.	2.	1.80	13915.	66.26	.60	--
6	12	U	350.	8	780.	70.	9.	5.60	13915.	22.09	.30	--
6	15	D	150.	9	585.	105.	9.	7.70	13915.	11.72	.27	--
6	16	D	100.	12	780.	105.	9.	6.80	13915.	11.72	.57	--
6	18	U	350.	11	975.	105.	9.	5.30	13915.	11.72	1.00	--
6	19	U	180.	1	780.	35.	9.	9.20	13915.	11.17	.88	--
6	21	U	600.	1	195.	35.	5.	11.30	13915.	79.51	.16	--
6	22	U	315.	1	780.	35.	9.	7.10	13915.	11.17	.82	--
6	23	U	500.	8	780.	70.	9.	6.80	13915.	22.09	.50	--
6	21	U	560.	15	975.	105.	2.	7.90	13915.	66.26	.51	--
6	25	U	700.	1	195.	35.	3.	7.90	13915.	132.52	.16	--
6	26	U	510.	6	585.	70.	8.	10.20	13915.	26.50	1.08	--
6	27	D	150.	9	585.	105.	9.	8.20	13915.	11.72	.18	--
6	29	D	375.	12	1170.	70.	2.	7.10	13915.	99.39	.30	--
6	32	D	375.	1	780.	30.	6.	9.60	13915.	77.31	.12	--

unloaded, 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, on April 7-10, 1981. There were a total of 6 events with 5 tows moving upstream and 2 downstream. (One of the events 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, on April 27-May 1, 1981. There were a total of 15 events, with 6 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, on May 20-22, 1981. There were a total of 5 events, with 2 tows moving upstream and 3 moving downstream. The number of barges pushed by the tows varied from 3 to 15. The draft was 9 feet for 4 of them and 2 feet for the other. The speed of the tows ranged from 9.3 to 14 ft/sec.

Trip 6 was again to Hadley's Landling on June 4-19, 1981. There were a total of 18 events, with 12 tows moving upstream and 6 moving downstream. The number of barges pushed by the tows ranged from 1 to 15. The draft and speed of the tows ranged from 2 to 9 feet and from 4.7 to 11.3 ft/sec, respectively.

Waves and Drawdown Generated by River Traffic

Wave Patterns

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

The wave patterns generated by tows in restricted channels are 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 over the other types of waves. There is also a narrow band of disturbed water surface behind the towboat resulting from the discharging of the propeller near the water surface. The water surface fluctuation caused by the propeller jet seems to be higher than the waves which reach the shore when observed behind the tow. This water surface fluctuation is, however, dissipated in the middle of the channel before it reaches the shore.

An example of a tow generated wave is represented in figure 25. The wave data were 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 represented in figure 26. The wave data were collected at the Rip Rap Landing test site on the Mississippi River during a passage of two

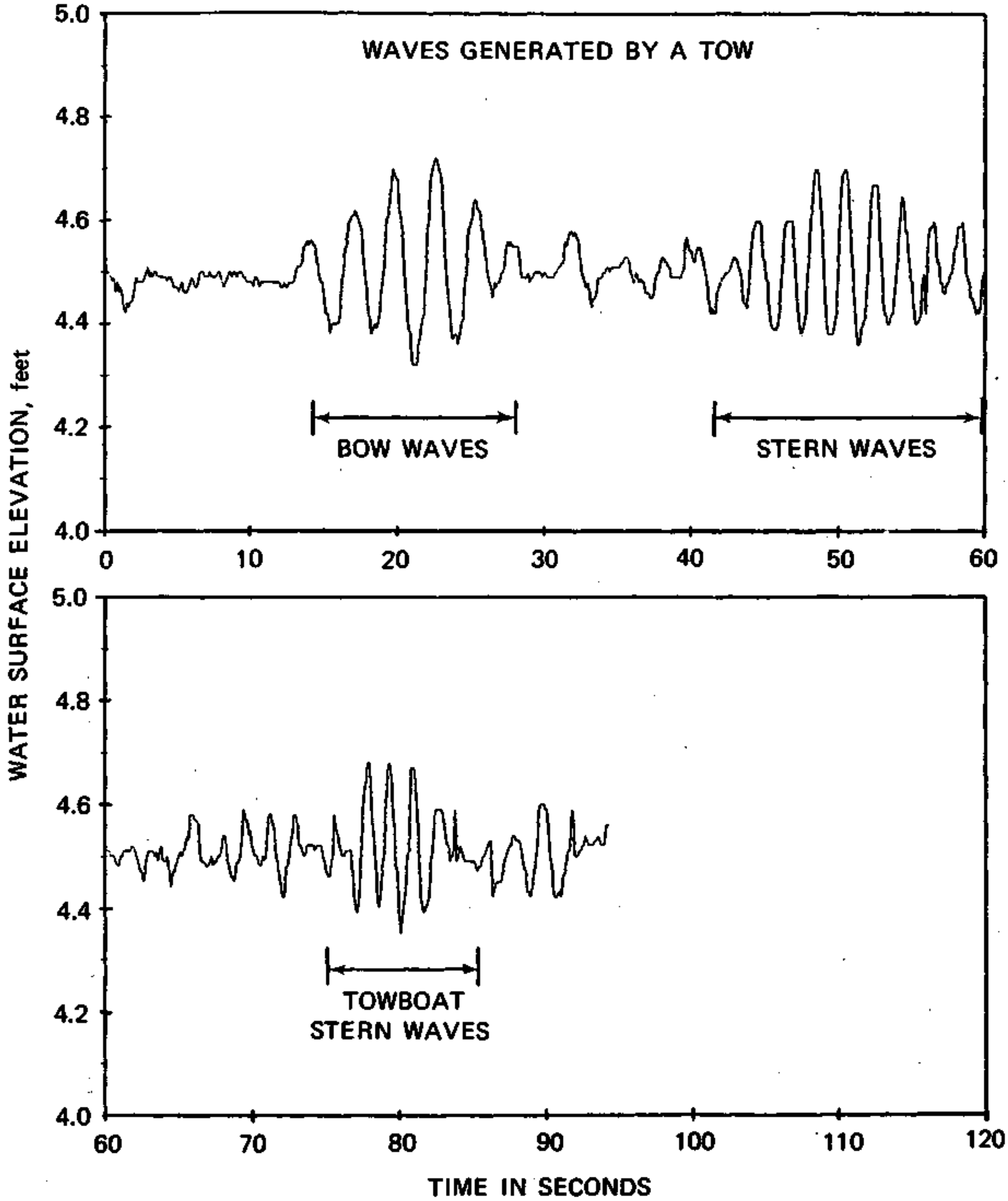

Figure 25. Wave pattern generated by a tow

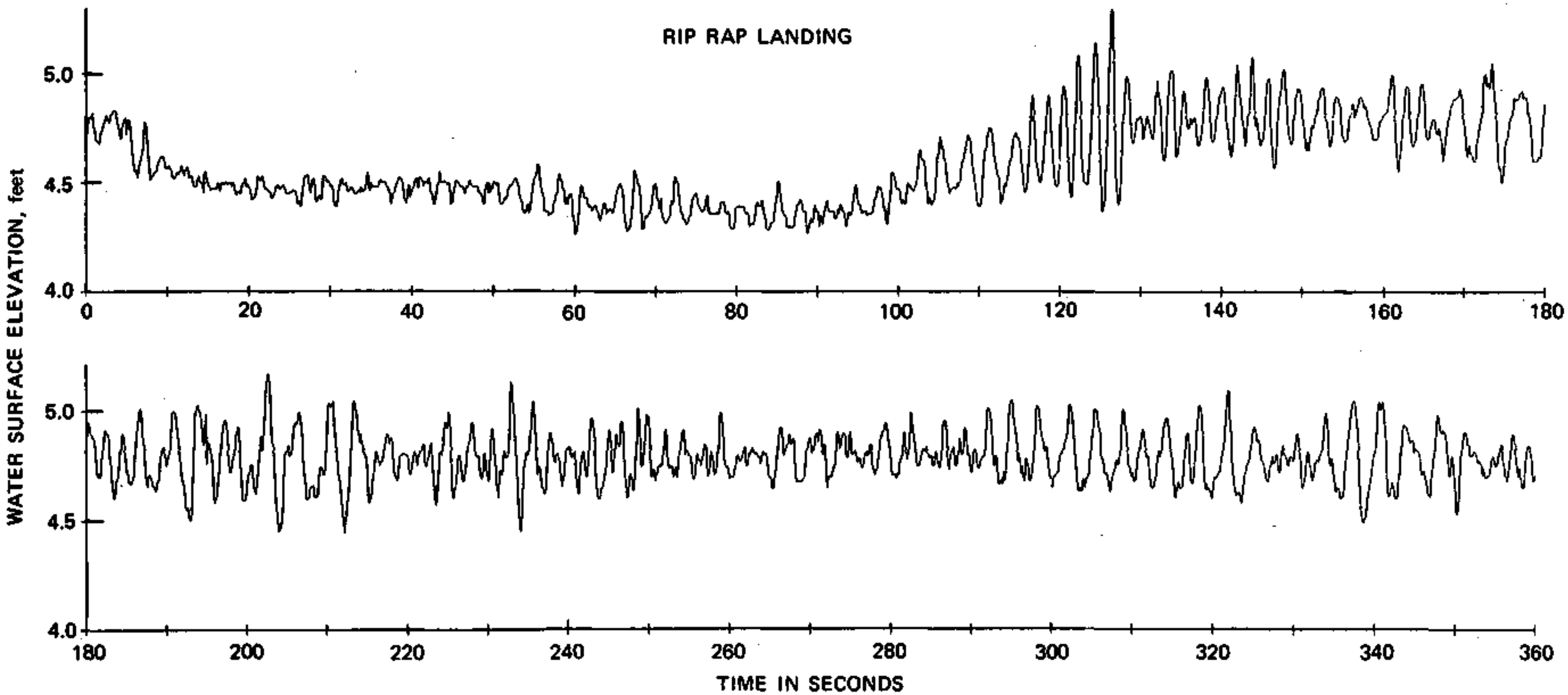

Figure 26. Wave pattern generated by two tows

tows, as one of the tows was attempting to pass the other. The data also include 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 25 and 26 shows that the waves generated at Rip Rap Landing are more complex, last for a much longer duration, and are generally higher.

As mentioned earlier there were other types of river traffic 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 give some basis for comparison. The kind of waves generated by a single towboat (shown in figure 27) is indicated in figure 28. The wave pattern is significantly different than that generated by tows in that it consists of only 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 29) is given in figure 30. 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 59 events during the six field trips were determined from plots similar to those in figures 25 and 27. All the maximum wave heights are summarized in table 7. The maximum wave heights ranged from a low of 0.1 feet to a high of 1.08 feet. The maximum wave

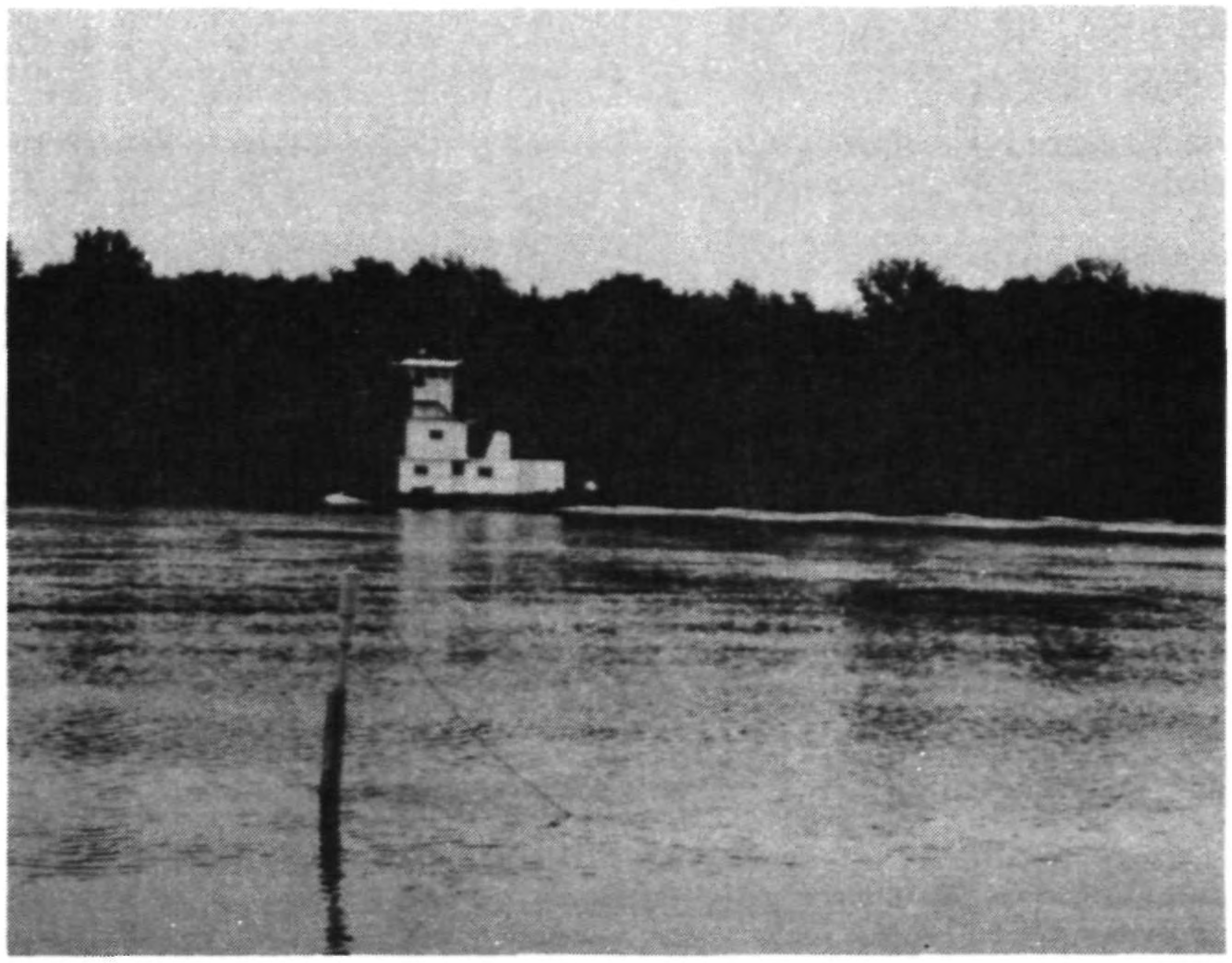

Figure 27. Towboat moving past a test site

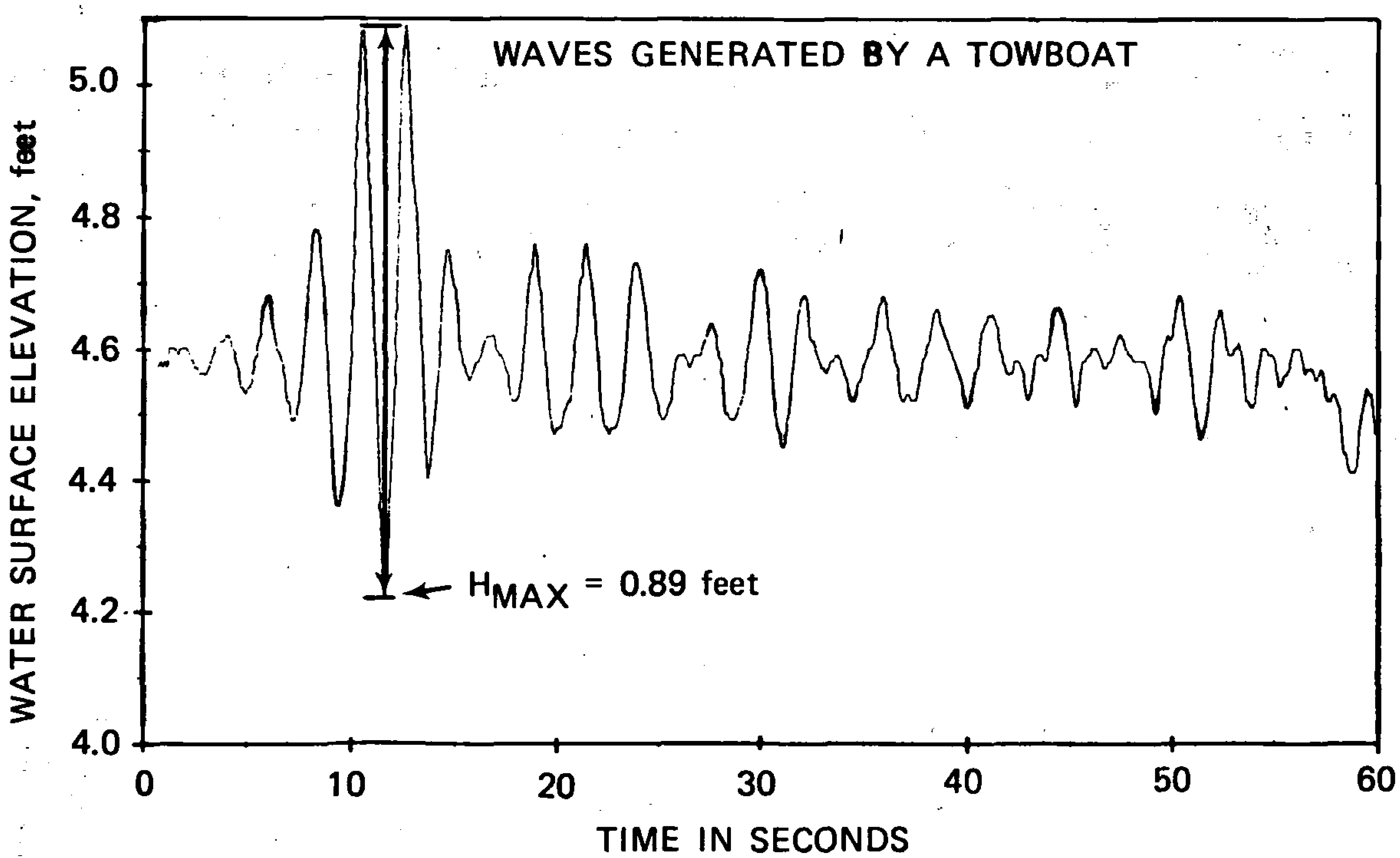

Figure 28. Wave pattern generated by a towboat

Figure 29. Cabin cruiser moving past a test site

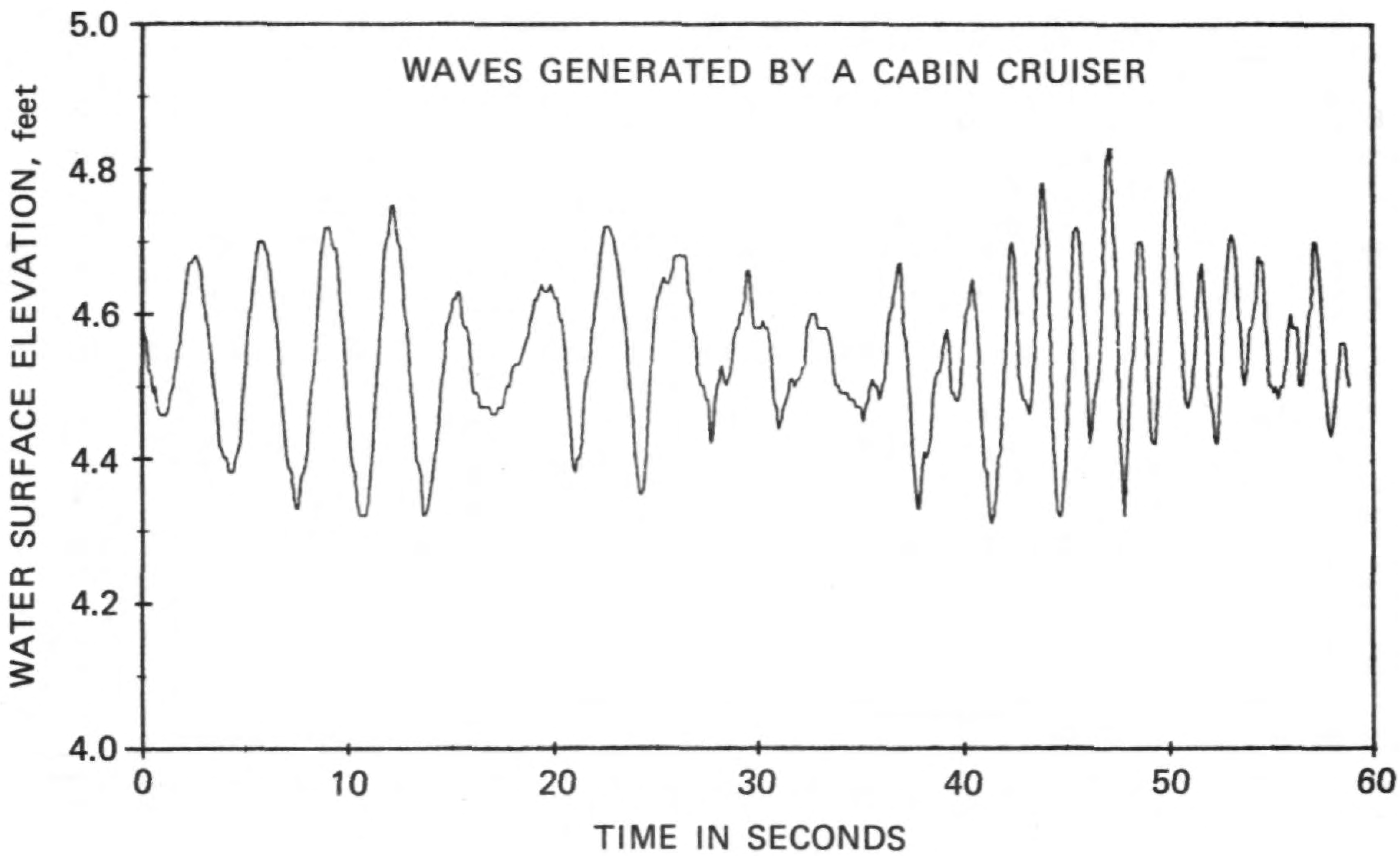

Figure 30. Wave pattern generated by a cabin cruiser

height of 1.08 feet occurred at the Hadley's Landing test site on the Illinois River for a loaded tow with 6 barges at 8 feet of draft moving upstream with a speed of 10.2 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 some trips where drawdown measurements were not taken, the total number of events where drawdown was measured is 27.

The maximum drawdown for all the 27 events is summarized in table 7 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 greater importance in bank erosion and biological studies than just the maximum drawdown value.

The plot of water elevation in figure 26 shows that the water level drops 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 was due to wind waves at the time of the event. Another example of drawdown is shown in figure 31. The maximum drawdown was 0.30 feet and the total drawdown period was almost 3 minutes. The data were 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.

Wind-Generated Waves

Wave heights generated by wind were calculated at four study sites for the purpose of comparing the relative significance of waves generated by wind and by river traffic. Wind records from Springfield (Illinois) and St. Louis (Missouri) were first used to calculate the wave heights, since the two stations are located in the general vicinity of the study sites. Bhowmik (1976) has analyzed the historical wind records at both the Springfield and St. Louis stations for different frequencies. Wind velocity and direction for 2-year and 50-year return periods, and for 6-hour durations, were taken from Bhowmik's data for analysis in this report.

Wind generated waves at the four study sites were calculated by using equation 12. In almost all cases the wind records from Springfield generated higher waves than those from St. Louis, corresponding to the higher wind velocities recorded at Springfield than at St. Louis. Therefore, only the results of the computations based on the Springfield records are summarized in tables 8 and 9. Table 8 is for the 2-year wind, while table 9 is for the 50-year wind.

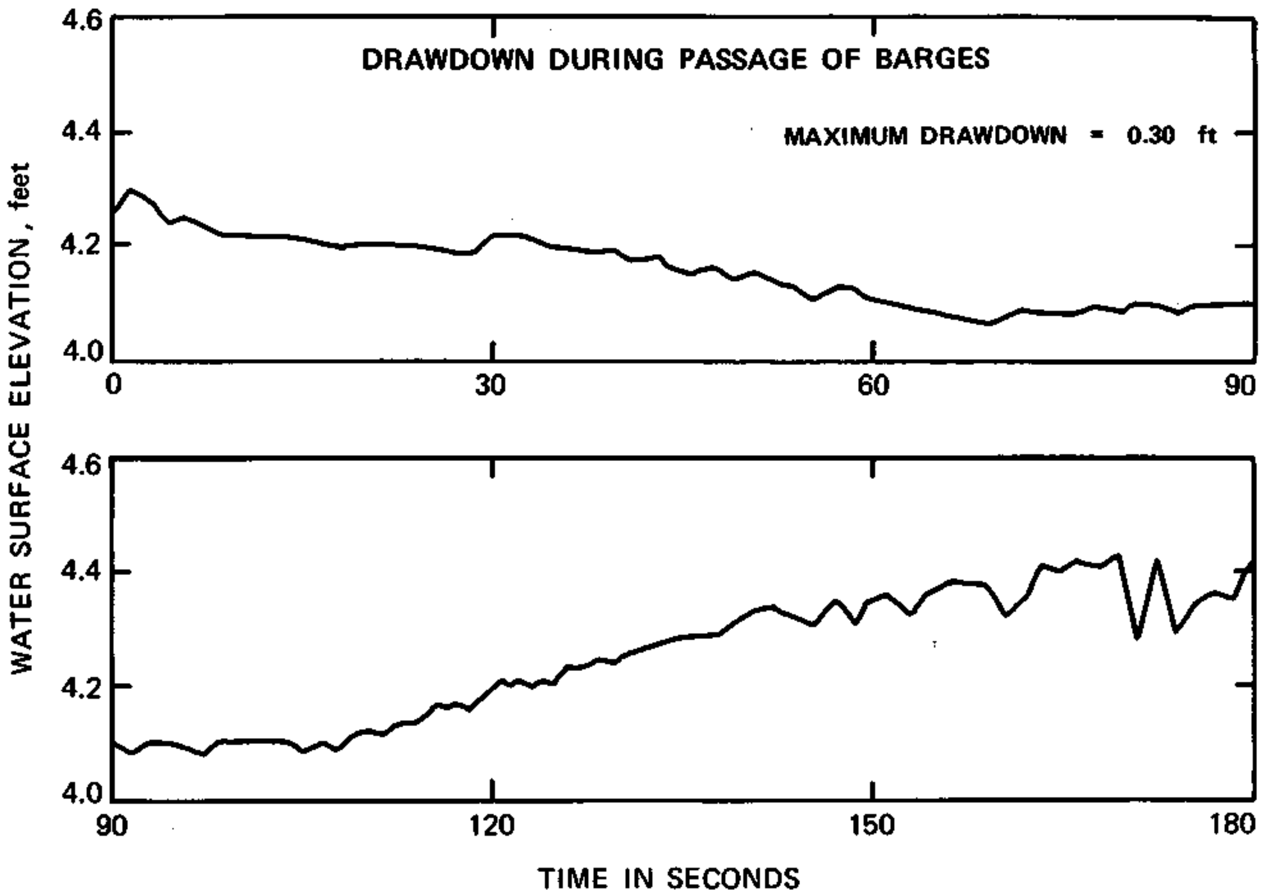

Figure 31. Drawdown during passage of barges at Hadley's Landing

Table 8. Wind Generated Waves Based on Wind Records from Springfield, Illinois, for 2-Year Return Period and 6-Hour Duration

Site	Width (ft)	Fetch (ft)	Eff. fetch (ft)	Mo.	Wind velocity (mph)	Angle ø°	Stream velocity (fps)	Eff. wind velocity (fps)	Wave height (ft)
Hadley's Landing	850	4500	1745	Jan.	27.9	10	2.84	37.55	0.70
	850	4500	1745	Feb.	29.0	0	2.84	39.79	0.75
	850	4500	1745	Mar.	32.3	10	2.84	44.64	0.85
	850	4500	1745	Apr.	29.5	10	2.84	39.87	0.75
	850	6000	1958	May	27.3	45	2.84	31.22	0.60
	850	6000	1958	June	21.9	45	2.84	25.60	0.48
McEver's Island	920	4500	1830	Jan.	27.9	5	2.78	43.64	0.85
	920	4500	1830	Feb.	29.0	10	2.78	44.76	0.87
	920	4500	1830	Mar.	32.3	10	2.78	43.98	0.86
	920	4500	1830	Apr.	29.5	5	2.78	40.42	0.78
	920	4500	1830	May	27.3	5	2.78	42.76	0.83
	920	4500	1830	June	21.9	5	2.78	34.85	0.66
Rip Rap Landing	1600	18450	4485	Jan.	27.9	0	3.22	37.79	1.07
	1600	18450	4485	Feb.	29.0	0	3.22	39.41	1.12
	1600	18450	4485	Mar.	32.3	0	3.22	44.26	1.28
	1600	18450	4485	Apr.	29.5	0	3.22	40.15	1.14
	1600	18450	4485	May	27.3	15	3.22	35.54	1.00
	1600	18450	4485	June	21.9	15	3.22	27.88	0.76
Mosier Island	1600	14520	4075	Jan.	27.9	0	3.50	37.51	1.02
	1600	14520	4075	Feb.	29.0	0	3.50	39.13	1.06
	1600	14520	4075	Mar.	32.3	0	3.50	43.98	1.21
	1600	14520	4075	Apr.	29.5	0	3.50	39.86	1.09
	1600	14520	4075	May	27.3	15	3.50	42.26	1.16
	1600	14520	4075	June	21.9	15	3.50	27.60	0.72

Table 9. Wind Generated Waves Based on Wind Records from Springfield, Illinois, for 50-Year Return Period and 6-Hour Duration

Site	Width (ft)	Fetch (ft)	Eff. fetch (ft)	Mo.	Wind velocity (mph)	Angle ø°	Stream velocity (fps)	Eff. wind velocity (fps)	Wave height (ft)
Hadley's Landing	850	4500	1745	Jan.	38.6	10	2.84	53.04	1.04
	850	4500	1745	Feb.	45.3	0	2.84	63.75	1.28
	850	4500	1745	Mar.	55.3	0	2.84	78.45	1.62
	850	4500	1745	Apr.	48.4	10	2.84	67.23	1.36
	850	6000	1958	May	42.9	45	2.84	47.43	0.96
	850	6000	1958	June	31.1	45	2.84	35.17	0.69
McEver's Island	920	4500	1830	Jan.	38.6	5	2.78	59.31	1.20
	920	4500	1830	Feb.	45.3	10	2.78	68.36	0.71
	920	4500	1830	Mar.	55.3	10	2.78	77.28	1.62
	920	4500	1830	Apr.	48.4	5	2.78	68.10	1.41
	920	4500	1830	May	42.9	5	2.78	65.60	1.35
	920	4500	1830	June	31.1	5	2.78	48.32	0.95
Rip Rap Landing	1600	18450	4485	Jan.	38.6	0	3.22	53.52	1.58
	1600	18450	4485	Feb.	45.3	0	3.22	63.37	1.91
	1600	18450	4485	Mar.	55.3	0	3.22	78.07	2.42
	1600	18450	4485	Apr.	48.4	0	3.22	67.93	2.07
	1600	18450	4485	May	42.9	15	3.22	57.69	1.72
	1600	18450	4485	June	31.3	15	3.22	41.22	1.18
Mosier Island	1600	14520	4075	Jan.	38.6	0	3.50	53.24	1.51
	1600	14520	4075	Feb.	45.3	0	3.50	63.09	1.83
	1600	14520	4075	Mar.	55.3	0	3.50	77.79	2.31
	1600	14520	4075	Apr.	48.4	0	3.50	67.65	1.98
	1600	14520	4075	May	42.9	15	3.50	64.41	1.87
	1600	14520	4075	June	31.3	15	3.50	47.94	1.34

The wave heights in tables 8 and 9 show that the highest waves are generally generated in March on both the Illinois and Mississippi Rivers. On the Illinois River, the significant wave heights are in the range of 0.9 ft and 1.6 ft for the 2-year and 50-year winds of 6 hour duration, respectively, while on the Mississippi River, the corresponding values are 1.3 ft and 2.4 ft. The cause of the higher waves on the Mississippi is the longer fetch on that river.

Comparison of Data with Theory

Waves Generated by River Traffic

The maximum wave heights measured in the field were compared with wave heights calculated with equations 1 and 2, which were discussed in the literature review section. The equations were made dimensionless by dividing both sides of the equations by the draft, D. The agreement between both equations and the measured data were not very good, as shown in table 10. The correlation coefficients between the measured and calculated maximum wave heights are .69 and .80 for equations 2 and 1, respectively.

In an effort to develop an empirical equation which could better predict the measured wave heights, a multivariate regression analysis was carried out between the measured maximum wave heights and the important parameters which were felt to influence the generation of waves. After many combinations of variables were tried, the following equation, which predicts the non-dimensional maximum wave height based only on the draft Froude number, F_D , was found to give the best result:

Table 10. Comparison of measured and calculated maximum wave heights

TRIP NO	EVENT NO	MEASURED HMAX/DRAFT	CALCULATED HMAX/DRAFT HOCHSTEIN (EQ. 2)	BALANIN (EQ. 1)	EQ. 13
1	1	.019	.020	.031	.042
1	2	.022	.006	.012	.022
1	3	.150	.130	.320	.177
1	4	.028	.009	.018	.028
1	5	.044	.009	.021	.030
2	2	.096	.021	.033	-042
2	3	.022	.048	.110	.069
2	4	.051	.055	.140	.077
2	5	.115	.173	.331	.179
2	6	.043	.020	.046	.045
2	7	.011	.007	.014	.025
2	9	.034	.024	.042	.047
2	10	.089	.053	.109	.068
2	11	.020	.024	.049	.046
3	3	.200	.116	.166	.148
3	4	.225	.203	.375	.222
3	7	.042	.048	.080	.068
3	9	.062	.053	.097	.075
3	12	.270	.158	.291	.196
3	12	.090	.073	.120	.083
4	1	.225	.261	.306	.200
4	2	.150	.167	.136	.161
4	3	.038	.030	.058	.052
4	4	.205	.253	.581	.247
4	6	.175	.175	.311	.180
4	8	.039	.071	.143	.090
4	9	.162	.187	.272	.143
4	10	.058	.079	.134	.078
4	11	.061	.067	.131	.078
4	12	.109	.119	.207	.106
4	13	.107	.100	.219	.100
4	14	.233	.548	.322	.205
4	15	.150	.086	.198	.144
4	16	.049	.068	.134	.078
5	4	.022	.071	.117	.082
5	6	.056	.059	.098	.075
5	9	.039	.125	.125	.104
5	10	.100	.055	.102	.077
5	11	.250	.185	.342	.212
6	1	.066	.049	.112	.069
6	4	.070	.065	.161	.125
6	6	.033	.004	.006	.018
6	12	.033	.047	.082	.066
6	15	.030	.019	.034	.038
6	16	.063	.011	.022	.031
6	18	.111	.041	.095	.064
6	19	.098	.089	.114	.094
6	21	.092	.323	.204	.148
6	22	.091	.065	.082	.080
6	23	.056	.061	.107	.075
6	24	.270	.132	.326	.178
6	26	.144	.138	.212	.112
6	27	.053	.023	.041	.042
6	29	.150	.022	.047	.076
6	32	.070	.034	.040	.065
CORRELATION C0EF.			.69	.80	.87

$$\frac{H_{max}}{D} = 0.133\ (F_D) \qquad (13)$$

The draft Froude number, F_D is equal to $V/\sqrt{gD}$. All other variables are as defined earlier. The correlation coefficient between the measured maximum wave heights and equation 13 is 0.87, which is much better than those for equations 1 and 2. The results of equation 13 are compared with the measured values and those computed by equations 1 and 2 in table 10. The spread of the data points around the regression line (equation 13) is shown in figure 32.

Drawdown

The theoretical and empirical equations discussed in the literature review (equations 3 through 7) were compared with the measured drawdowns. The results of the comparison, and the correlation coefficients between the different equations and the measured values are presented in table 11. Also shown in table 11, are the results of the following equation, which was developed through a multivariate regression analysis of the measured values and the important parameters which were felt to influence drawdown.

$$\frac{\Delta h}{Y'} = 0.478\ (F_{Y'})^{0.5} \left(\frac{A_m}{A_c}\right)^{0.81} \left(\frac{L}{Z}\right)^{0.26} \qquad (14)$$

where $F_{Y'} = V/\sqrt{gY'}$ is the Froude number based on Y', Y' = Y-D, and Y = A_c/B_c = hydraulic depth of the channel. All other variables are as defined earlier.

As shown in table 11, equation 14 gives the higher correlation coefficient 0.84 than to the other three equations discussed in the literature review. The results of equation 14 were further compared with the measured values in figure 33. The spread of the data points around

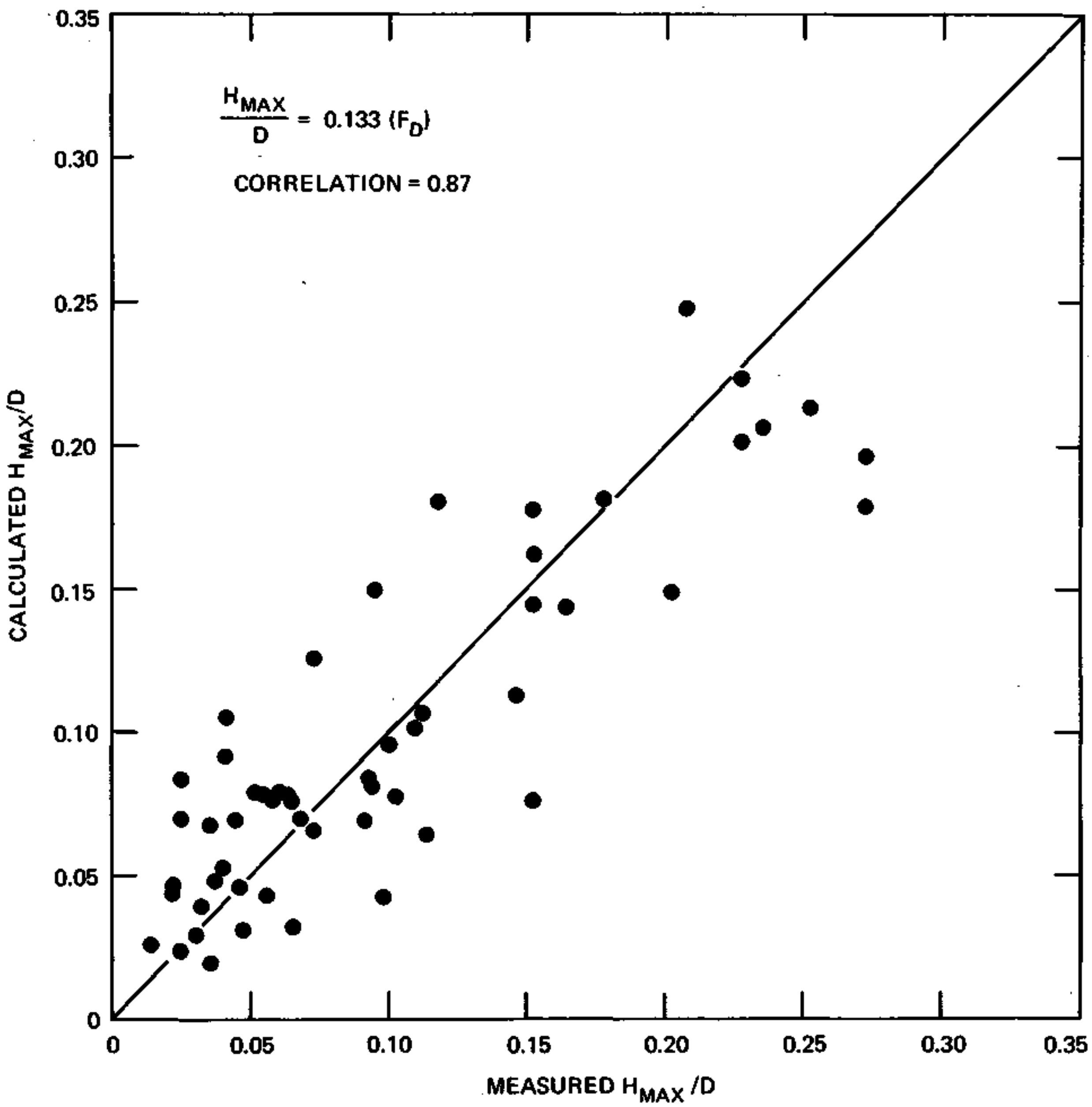

Figure 32. Comparison of the regression equation with the measured maximum wave heights

Table 11. Comparison of measured and calculated maximum drawdowns

TRIP NO	EVENT NO	MEASURED $\Delta h/Y'$	CALCULATED $\Delta h/Y'$				
			DAND (EQ. 6)	SCHIF (EQ. 3 & 4)	HERBICH (EQ. 7)	GELENCSER (EQ. 5)	EQ. 14
2	3	.027	.033	.029	.055	.006	.039
2	4	.058	.041	.039	.069	.010	.045
2	5	.010	.003	.005	.009	.000	.009
2	6	.017	.014	.011	.023	.002	.023
2	7	.027	.004	.003	.007	.000	.012
2	10	.040	.032	.029	.054	.004	.037
2	11	.036	.015	.011	.024	.002	.024
3	3	.008	.001	.001	.004	.000	.005
3	4	.017	.002	.005	.008	.002	.009
3	6	.023	.003	.007	.010	.003	.010
3	7	.018	.018	.018	.036	.005	.028
3	12	.023	.002	.003	.007	.001	.007
3	12	.055	.027	.030	.054	.059	.061
4	3	.029	.014	.011	.024	.004	.029
4	4	.007	.005	.009	.014	.003	.014
4	6	.020	.002	.004	.007	.004	.015
4	7	.014	.001	.001	.003	.000	.005
4	8	.048	.024	.028	.047	.015	.039
4	9	.023	.013	.021	.030	.003	.024
4	10	.025	.032	.030	.055	.006	.042
4	12	.028	.021	.024	.041	.005	.033
4	13	.083	.052	.062	.090	.026	.062
4	14	.011	.004	.013	.014	.000	.009
4	15	.007	.002	.002	.005	.002	.009
4	16	.030	.032	.030	.055	.010	.046
5	4	.031	.039	.046	.077	.005	.033
5	9	.012	.022	.042	.057	.002	.021
CORRELATION COEF.			.78	.71	.73	.67	.84

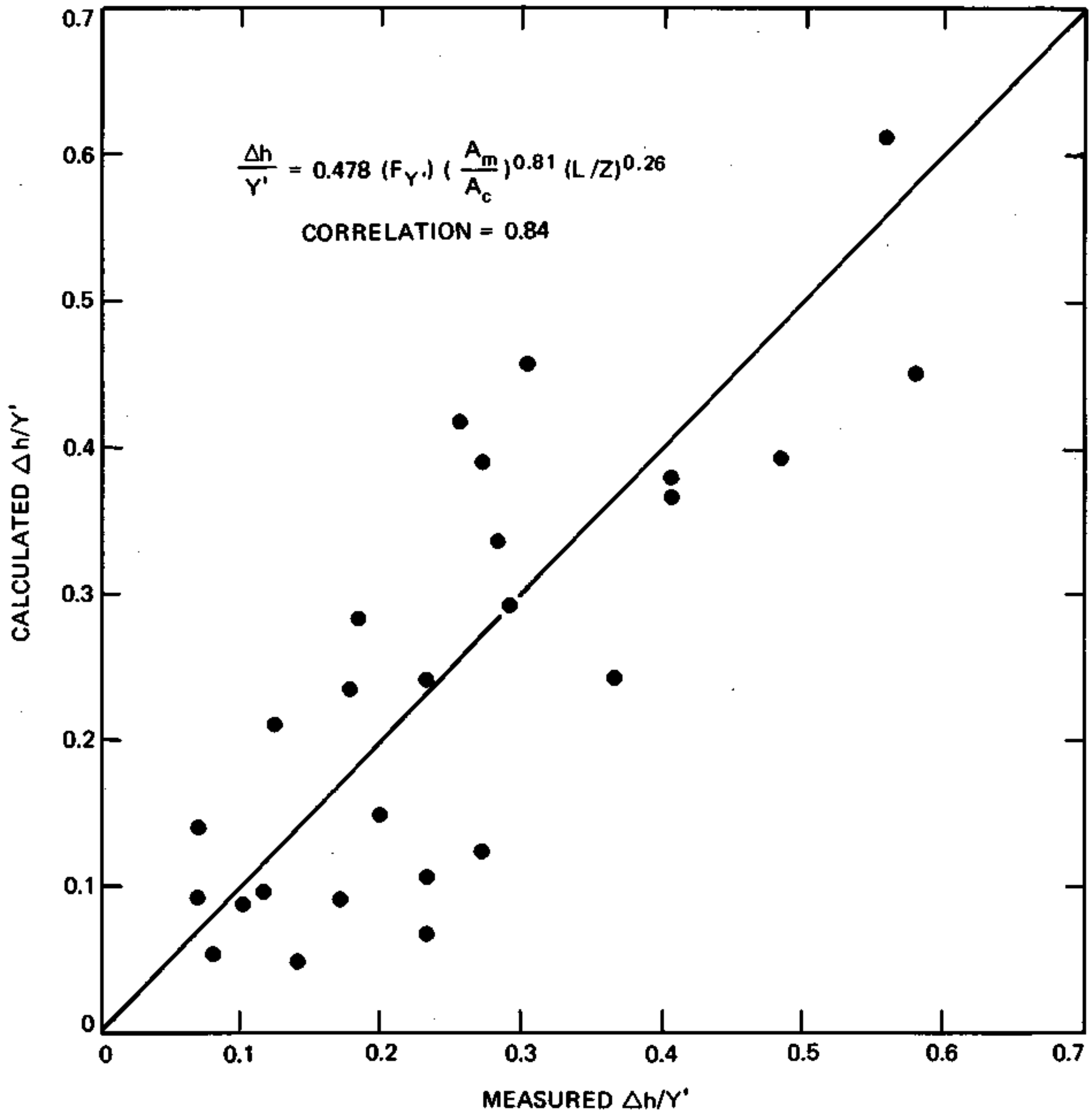

Figure 33. Comparison of the regression equation with the measured drawdowns

the regression line indicates a reasonably good agreement between measured and calculated values.

Comparison Between Waves Generated by Wind and by River Traffic

The comparison of waves generated by wind with those generated by river traffic is a very difficult task due to the different nature of the waves. Wind-generated waves generally last for longer durations than those generated by river traffic; however, river traffic generated waves occur more frequently than wind-generated waves. While the effects of river traffic generated waves on steam banks last for only a few minutes after the passage of a tow or a boat, wind-generated waves last for hours during periods of high wind velocities. On the other hand, river traffic generated waves occur several times a day, while significant wind-generated waves might occur only a very few times during the whole year.

As shown in table 7, waves generated by river traffic are for the most part less than 1 ft high. The duration of the wave action generated by a single vessel is generally 2 to 5 minutes. The tow traffic alone on the Illinois and Mississippi Rivers is estimated to be in the range of 15 tows a day (Bhowmik et al, 1981a). Therefore the duration of wave action on the stream banks due to tow traffic alone ranges from 30 to 75 minutes daily. When smaller boats are included the duration will be even longer.

On the other hand, significant wind-generated waves do not occur every day, but when they occur their magnitudes are larger and their durations longer than those of river traffic generated waves. For the-2-year, 6-hour duration wind, which is expected to occur once every two years, on the average the significant wave heights can reach 0.9 ft on the Illinois River, and 1.3 ft on the Mississippi River, as shown in table 8.

When more severe wind conditions are considered, the significant wave height gets higher. For the 50-year, 6-hour duration wind the significant wave heights reach 1.6 ft on the Illinois River and 2.4 ft on the Mississippi, as shown in table 9.

However, such wave heights are expected only once in every 50 years on the average. Other wind conditions with different return periods and durations also will have to be included in the total analysis.

It is therefore very difficult at this time to say whether the total impact of daily wave action due to river traffic is more or less significant than the impact of higher waves generated by winds of longer duration but of lesser frequency.

SUMMARY AND CONCLUSIONS

Six field trips were taken to collect wave and drawdown data. Four of the field trips were to the Illinois River, and two were to the Mississippi River. Wave data were collected for a total of 59 events, and drawdown data were collected for 27 of the events. Additional wave data were 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 1.08 foot, while the maximum drawdown ranged from 0.05 foot to 0.69 foot.

Comparison of the measured maximum wave heights and drawdowns with those predicted through existing predictive equations was not satisfactory. The correlations between the measured and calculated wave heights and drawdowns were found to be low. By performing a multivariate regression analysis, it was possible to obtain equations which predict wave heights and drawdowns better than the previously existing equations. In the new equation for maximum wave height, the non-dimensional wave height is a function of the draft Froude number only. In the equation for maximum drawdown, the non-dimensional drawdown is a function of the Froude number based on the hydraulic depth minus draft, blockage factor, and a dimensionless distance from the sailing line to the wave gage.

Significant wave heights for wind-generated waves were also calculated at the four study sites for 2-and 50-year return periods and 6-hr duration winds. On the Illinois River the significant wave heights were found to be in the range of 0.9 ft and 1.6 ft for the 2-year and

50-year winds of 6-hr duration, respectively, while on the Mississippi River the corresponding values were 1.3 ft and 2.4 ft.

In general, the observed and calculated waves generated by both river traffic and wind are significant enough to cause stream bank erosion on both the Illinois and Mississippi Rivers. The relative significance of waves generated by river traffic in comparison with those generated by wind can not be determined qualitatively at the present time because of the differences in frequency, duration, and magnitude between the two types of waves. Further research in this aspect of the analysis is needed before a definite conclusion can be made.

The drawdown caused by loaded tows is also significant and can expose shore areas for several minutes on the average during each tow passage. It can also significantly change the flow characteristics of small tributary streams in the vicinity of their junction with the navigable river by changing the hydraulic gradient at their outlets.

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NOTATIONS

The following symbols are used in this report:

A_c = Channel cross-sectional area

A_m = Submerged cross-sectional area of a vessel

A' = Channel cross-sectional area after drawdown minus A_m

b = Width of a vessel

B_c = Channel width

C = Wave celerity

d = Channel depth

D = Draft of a vessel

F = Fetch

F_d = Draft Froude number

F_e = Effective fetch

F_ℓ = Vessel length Froude number

F_y' = Froude number based on Y'

g = Gravitational acceleration

H = Wave height

H_s = Significant wave height

H_{max} = Maximum wave height

Ah = Drawdown or squat

L = Wave length

ℓ = Length of a vessel

q = Partial discharge through a portion of a channel cross section

Q = Total discharge in a river

T = Wave period

t = Wind duration

U = Wind velocity

U_e = Effective wind velocity

V = Vessel velocity

$\overline{V}$ = Mean flow velocity

V(.2) = Velocity of water at 0.2 of total depth from the surface

V(.8) = Velocity of water at 0.8 of total depth from the surface

V = Backflow velocity beneath the vessel

W = Channel width

Y = Hydraulic depth of channel

Y' = Hydraulic depth minus draft

Z = Distance of wave gage from sailing line

Φ = Angle between wind direction and stream bearing