arch Report 117

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ILLINOIS STATE WATER SURVEY
DEPARTMENT OF ENERGY AND NATURAL RESOURCES

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RESEARCH REPORT 117

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Abstract: To determine the characteristics of waves generated by recreational craft within the Upper Mississippi River System (UMRS), 246 controlled runs were made at an Illinois River site and a Mississippi River site. Data showed thatrecreational boats can generate from 4 to 40 waves per event, with a mean of about 10 to 20 waves. Average wave heights varied from 0.01 to 0.25 meter, with a median of about 0.06 to 0.12 meter. The maximum wave height was as much as 0.6 meter. The data were used to develop a regression equation for estimating maximum wave heights on the basis of the speed, draft, and length of the boats, and their distance from the measuring point. Data from uncontrolled boating events on the Mississippi River indicated that as many as 704 boats passed a highly used area of the UMRS in a single day on a busy weekend. Sustained movement of recreational boats can generate essentially continuous waves. During the day of heaviest boating activity, the maximum wave height measured was 0.52 meter, and the average for the day was 0.065 meter. Calculations showed that for waves of 0.4 meter in height to develop at the Mississippi River site from wind alone, the wind would have to be blowing at a speed of about 26 meters per second (58 mph) across the measuring point Additional analyses showed that the shorelines are subjected to wave activity of fairly high intensity.

Reference: Bhowmik, Nani G., Ta Wei Soong, Walter F. Reichelt, and Noha M.L. Seddik. Waves Generated by Recreational Traffic on the Upper Mississippi River System. Illinois State Water Survey, Champaign, Research Report 117,1991.

Indexing Terms: Boating, Illinois River, Mississippi River, recreational boats, rivers, wave energy, wave height, waves.

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CONTENTS

NOMENCLATURE

a = wave amplitude, meters

D = depth of water, meters

d = draft of the boat, meters

E = total wave energy per unit surface area

e = exponential function

F_d = depth Froude number

Fe = effective wind fetch, meters

g = acceleration due to gravity, meters/second^2

H_m = maximum wave height, meters

Hs = significant wave height, meters

Hs_1 = significant wave height at wave gage 1, meters

Hs_2 = significant wave height at wave gage 2, meters

h_p = horsepower, 1 h_p = 746 newton meters/second

KE = kinetic energy per unit surface area

L = length of the boat, meters

N = number of boats passing per hour at study site

PE = potential energy per unit surface area

S_8 = specific gravity of the sediment

U = uniformity coefficient for bed material

Ve = effective wind velocity, meters/second

v = relative speed of the boat, meters per second

v_b = absolute speed of the boat, m/s

v_f = water velocity, m/s

W_8 = weight of the boat, newtons

W_{50} = weight of the median diameter of bank materials, newtons

x = distance between the boat and wave gage, meters

a = slope of the bank, degrees

γ = unit weight of water, kilo-newtons/meter^3

v = kinematic viscosity, meterVsecond

a = standard deviation for bed materials

Waves Generated by Recreational Traffic on the Upper Mississippi River System

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ABSTRACT

Movement of recreational boats in a waterway such as the Upper Mississippi River System (UMRS) generates waves that can impact the river biota and the stability of the shorelines. This report presents the results of a research project undertaken to determine the characteristics of waves generated by recreational craft within the UMRS. To meet the goals of the project, 246 controlled runs were made with 12 different boats at two sites, one on the Illinois River and the other on the Mississippi River. Data from the controlled runs indicated that recreational boats can generate from 4 to 40 waves per event, with a mean of about 10 to 20 waves. These waves can last from 6 to 40 seconds or more. Average wave heights for these controlled events varied from 0.01 to 0.25 meter, with a median of about 0.06 to 0.12 meter. The maximum wave height was as much as 0.6 meter.

The wave data from the controlled runs were used to develop a regression equation for estimating maximum wave heights on the basis of the speed, draft, and length of the boats, and their distance from the measuring point. This relationship is now recommended for use in determining wave heights generated by recreational boats.

Data from uncontrolled boating events on the Mississippi River indicated that as many as 704 boats passed a highly used area of the UMRS in a single day on a busy weekend. Up to 120 boats passed the site in a single hour. Sustained movement of recreational boats can generate essentially continuous waves, giving the appearance of random waves at or near the shoreline. During the day of heaviest boating activity at the Mississippi River site, the maximum wave height measured was 0.52 meter, and the average for the whole day was 0.065 meter.

Analyses were also performed by partitioning the wave heights on an hourly basis. These analyses indicated that significant wave height can reach a magnitude of 0.4 meter or higher, and maximum wave height can reach 0.5 meter or higher. Calculations were also performed to show that for waves of 0.4 meter in height to develop at the Mississippi River site from wind alone, the wind would have to be blowing at a speed of about 26 meters per second (58 mph) across the measuring point. Wave energies were computed by partitioning the waves into five-minute intervals. These analyses showed that the shorelines are subjected to wave activity of fairly high intensity.

No analyses were performed to determine the bank erosion potential or sediment resuspension characteristics of the waves generated by recreational boats. However, existing mathematical formulations can be used to analyze the stability of banks composed of noncohesive bank materials. Additional research should be initiated to determine the effects of recreational boats on the stability of cohesive and noncohesive banks, and the way in which wave activity resuspends bed materials.

INTRODUCTION

Recreational boats of many sizes and shapes travel on the Upper Mississippi River System (UMRS). While traveling up and down the river, they generate waves of varying characteristics depending upon their size, shape, speed, draft, and distance from the measuring point. Frequent movement of recreational boats can cause waves to be superimposed on each other and can result in continuous wave activity as long as the boating activity lasts. Waves generated by recreational traffic are somewhat different from those generated by wind or barge-tow traffic. Normally waves generated by a single recreational boat have few peaks and dissipate fairly soon, whereas waves generated by barge tows have more peaks and usually lead by a negative surge (drawdown) (Bhowmik, 1975,1976; Bhowmik et al., 1981, 1982, 1989). Waves generated by sustained wind activity can last for a long time, and the wave heights are generally randomly distributed (Bhowmik, 1976).

Until now no systematic research has been undertaken on the UMRS to determine the characteristics of waves generated by recreational boats. The present research was undertaken to determine the physical characteristics of waves generated by recreational traffic on the Mississippi and Illinois Rivers. It is designed to fill an important gap in existing knowledge of the effects of recreational traffic within a waterway such as the UMRS.

Objectives

The main objectives of this investigation were:

1) Measure wave heights and periods for waves generated by a range of recreational boats from 4.6 to 15 meters in length.
2) Use data from controlled experiments to develop models for waves generated by recreational boats.
3) Quantify the range of wave heights and periods of waves resulting from random, uncontrolled recreational traffic.
4) Estimate the energy content of boat-generated waves.
5) Show the application of the developed model.

Background

This section of the report briefly describes the available literature on waves generated by recreational boats. Recreational craft are smaller and have different hull forms than commercial barges. In many reaches of the UMRS, they may dominate the waterway traffic, thus requiring attention from river managers.

Waves can be a contributing factor to the instability of banks and shorelines. Hagerty et al. (1981) found that no significant correlation existed between boat traffic level and bank erosion on the Ohio River. Studies conducted by Hurst and Brebner (1969), Limerinos and Smith (1975), Karaki and Van Hoften (1975), and Maynord and Oswalt (1986) indicated that bank erosion may be caused partly by waves generated by vessels moving on the waterway. Besides causing bank erosion, waves can cause resuspension of fine sediments and increased turbidity, which can then be carried to the side channels and 'backwater areas. Increased turbidity and suspended sediment concentrations may impair riverine ecosystems. However, these impacts have not yet been quantified.

When a vessel moves on the free surface of a water body, the flow around its hull changes in both magnitude and direction. Simultaneously dynamic pressures develop and distribute over the submerged hull surface. This distribution of dynamic pressures governs the wave patterns that propagate outward from the vessel. The pressure rises in front of the bow, reaches a maximum equal to the stagnation pressure (zero velocity) at the bow because of the blockage of the flow area by the vessel, and then starts to drop below the free stream pressure toward midsection according to the acceleration of fluid (Comstock, 1967). It is apparent that such distribution of dynamic pressure is three-dimensional and dependent on the hull form.

Two-dimensional representations that can lead to insights into such distributions have been developed in a number of theoretical and experimental studies. For recreational-type sharp-bowed vessels, Lighthill (1978) theoretically derived the distribution of pressure on a horizontal plane along the hull for an elliptically shaped, fully immersed ship. The dynamic pressure has a positive peak value near the bow and drops to a negative value over most of the midsection. Lighthill indicated that positive peak dynamic pressure near the bow is characteristic of sharp-bowed ships. Although the excess pressure near the stern was not included in the analysis, he suggested that the case of symmetry may apply if the ship is symmetric about its center line.

orensen (1973a) noted that any irregularities in the hull form will superimpose additional pressure fluctuations. He also indicated that the pressure distribution near the stern is different from the distribution at the bow for real fluid because the boundary layer separation occurs near the stern. Therefore the amplitude of waves is generally lower near the stern.

The wave patterns behind a vessel are made up of two different trains of waves (Kelvin, 1887; Stoker, 1957). One set is the oblique waves that move forward and out from the vessel, and the other set is arranged roughly at right angles to the ship's course. These two systems of waves are called the diverging and transverse waves, respectively (figure 1).

According to the distribution of dynamic pressure, diverging and transverse waves are generated at both the bow and stern. The transverse waves meet the diverging waves on both sides of the vessel along two sets of lines called the cusp lines. Waves following the ship are bounded by the cusp lines, which intersect the sailing line at a semi-angle of 19° 28' (Kelvin, 1887; Stoker, 1957). Sorensen (1973b) showed that the general wave pattern generated by a model hull towed in deep water agrees well with this wave pattern except for a small change in the cusp angle.

The diverging waves that travel outward generally are independent of each other if the bow and stern are sufficiently separated, but the bow and stern transverse waves will be superimposed. At cusp points, the superimposition of diverging and transverse waves generates higher amplitudes than at other locations. After waves pass the cusp line, their amplitudes do not become zero, but they become smaller. Stoker (1957) demonstrated that they are of a different order from the waves inside the region and have different attenuation rates.

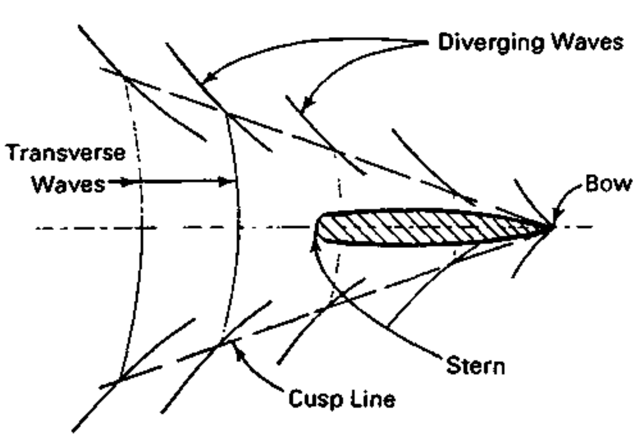

Figure 1. Wave pattern generated by a model ship in deep water

A majority of the research on wave patterns generated by the boat hull and traveling speed was conducted on ships in the ocean environment or model boats in the laboratory. During field surveys, the current investigators observed that wave trains were generated from hull displacement as well as from the propulsion systems. Waves generated by propulsion systems (called surge waves) are important for recreational boats. A surge wave is a moving wave front that can suddenly change the water depths (Henderson, 1966). Few studies have addressed this topic, but its effects were included in the modeling done for this study.

Waves interact with the flow field within the water body. In inland waterways, the water depth and the channel width vary from section to section and also from main channel to channel border areas. All these factors affect the propagation of waves and wave characteristics. Waves normally break on the shoreline, and the breaking zone is a function of bank slope. Normally waves are classified on the basis of whether the water particle underneath a wave will interact with the bottom (Ippen, 1966; Henderson, 1966).

In deep water, the water particle orbit is circular and extends to about one-half of the wave length from the mean water surface. In shallow water, the particle orbit becomes elliptical and can touch the bottom. As a wave moves from deep to shallow water, the wave profile changes from sinusoidal to approximately trochoidal, and its length and phase velocity decrease. The wave group velocity also approaches phase velocity. This behavior has a pronounced effect on the boat wave crest pattern. Sorensen (1973a,b) used the depth Froude number (F_d) to classify wave patterns generated by boats in shallow water:

$$\mathbf{F_d = v/\sqrt{gD}} \qquad \textbf{(1)}$$

in which v is the speed of the boat, g is the acceleration due to gravity, and D is the water depth at the wave gage. The Froude number is used for open-channel flow problems to indicate the relative importance of inertia force to gravity forces. In such cases, the Froude number is defined as the ratio of $F_d = v_f/\sqrt{gD}$, where v_f is the velocity of water. For such water movement, when $F_d = 1$ the flow is said to be critical, when $F_d < 1$ the flow is subcritical, and when $F_d > 1$ the flow is supercritical. However, for waves in open-channel flow problems, the depth Froude number defined by equation 1 is normally used.

As the depth Froude number increases in shallow water, the diverging wave crests rotate forward to a position at right angles to the sailing line, and the cusp locus angle increases. The leading transverse and diverging waves are accentuated. When the depth Froude number reaches unity, the transverse and diverging waves form a single large wave with its crest normal to the sailing line, which travels at the same speed as the disturbances. For depth Froude numbers larger than 1, transverse waves disappear and diverging waves radiate from the disturbance. Inui (1936) demonstrated that the inner wave crest lines for a supercritical disturbance are concave outward.

On the other hand, channel width constraint is responsible for acceleration of the flow around the ship. This then induces an increase in dynamic pressure near the bow and stern and further decreases the pressure through the midsection. Such influences increase the magnitude of the boat waves.

A factor, blocking ratio, is generally used to indicate whether the channel cross-sectional area has any influence on the movement of a boat. The blocking ratio is the value obtained by dividing the channel cross-sectional area by the maximum submerged projection area of the boat. When the blocking ratio is larger than 20, the channel width does not have any significant influence on the movement of the boats.

When waves propagate near the shore, the geometry of the shore zone can cause waves to reflect, refract, and diffract. If the side slope is large and the bottom material is impermeable, the reflecting waves will be large, will interact with incoming waves, and can produce complex wave patterns.

From the above discussion, it is easy to understand that waves measured near the shoreline can be incoming waves, refracted waves, reflected waves, or some combination of the above. However, when bank stability against wave action is considered, then the impacts of the breaking waves and runup along the shoreline must be taken into account.

Even though much research has been conducted on waves generated by wind in large bodies of water such as bays and oceans, very little research has been reported in the literature on waves generated by recreational boats within restricted waterways or lakes. Das (1969) and Das and Johnson (1970) conducted laboratory investigations to determine the wave characteristics and the peak wave energy produced by model ships and pleasure craft. Garrad and Hey (1987) have indicated that an increase in suspended sediment concentration and hence in turbidity is related to the diurnal movement of recreational traffic within the Broadland waterways of East Anglia, United Kingdom.

Waves generated by recreational boats and winds within inland lakes were investigated by Bhowmik (1975, 1976, 1978). His research was concerned primarily with the stabilization of lakeshores against wind-generated waves. However, during the data collection on wind-generated waves, Bhowmik (1975, 1976) also collected data on waves generated by recreational boats at Carlyle Lake in Illinois. Controlled runs were conducted to gather wave data by using a Super 8 millimeter movie camera. These data were then reduced for estimating the maximum wave heights generated by recreational boats. On the basis of these data and Das's (1969) data, Bhowmik (1975, 1976) developed the following generalized equation:

$$(H_m/d)^2 = 0.139\, v^{1.174}\, (x/L)^{-0.915} \qquad (2)$$

where H_m is the maximum wave height in meters, d is the draft of the boat in meters, v is the speed of the boat in meters per second, x is the distance of the boat from the measuring point in meters, and L is the length of the boat in meters.

Subsequently Bhowmik et al. (1981, 1982, 1989) collected data on waves and drawdown resulting from the movement of barge traffic on the Mississippi, Illinois, and Ohio Rivers. However, data on waves generated by recreational traffic were not collected or analyzed.

This brief review of the literature indicates that very little information is available on the characteristics of waves generated by recreational traffic in inland waterways. Moreover, except for the regression equation proposed by Bhowmik (1975, 1976), no relationships are available for estimating the wave heights generated by recreational traffic.

At the same time, very little information is available on the effects of waves generated by recreational boats on the resuspension characteristics of sediments, or on the way these waves can destabilize streambanks and lakeshores. However, the methodology that was proposed by Bhowmik (1976, 1978) for stabilizing lakeshores against wind-generated waves can be used once the characteristics of the waves generated by recreational boats are either known or estimated.

Acknowledgments

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The Project Coordinators were Ken Lubinski (USFWS) and Bill Donels (IDOC). Ken Lubinski, Jerry Rasmussen, Bob Delaney, and Gail Carmody from the USFWS were extremely helpful in the initiation and completion of this project.

Mike Davis and Ceil Strauss from the Minnesota Department of Natural Resources (DNR) were very supportive and helpful in the initiation of this project and provided assistance in the field data collection at the Red Wing site. Bill Nafe of Bill's Bay Marina helped with the use of two boats (Propinquity and Scorpion) in the controlled runs at Red Wing. He also assisted us in obtaining help from the owners of two additional boats (Trojan and Sea Ray) in our controlled field experiments.

The Park District of the city of Red Wing permitted temporary use of a measuring site inside Colvill Park. Staff from the Illinois Natural History Survey assisted in the field data collection effort at the Havana site. Agriculture Business at Havana made arrangements so we could access the site.

Among the Water Survey staff members who worked very hard on the formulation and collection of field data were Rodger Adams, Bill Bogner, and Ed Delisio. John Brother and Linda Hascall prepared the graphics, Kathleen J. Brown typed the camera-ready copy, Gail Taylor and Eva Kingston edited the report, and Linda Hascall helped format the final report. To all of them and other supporting staff members, a hearty "thank you" for their fine efforts.

Last but not least, the authors would like to extend their sincere thanks to the 17 reviewers of the draft copy of this report. Their comments were helpful and constructive, and the major suggestions have been incorporated in this report. A special thanks to Scott Johnson of the Minnesota Department of Natural Resources for pointing out some of the limitations of the predictive relationship.

DATA COLLECTION

Before this project was initiated, a thorough discussion was held among the sponsors, the investigators, personnel from the Wisconsin and Minnesota Departments of Natural Resources, and other midwestern state representatives to the Ecological Advisory Committee (EAC) of the Environmental Management Program (EMP) administered by the U.S. Fish and Wildlife Service. During this preplanning stage, the decision was made that data from two sets of controlled runs should be collected for this project. The first set of data would be collected from a site on the Illinois River, and the second set from a site along the Mississippi River within the boundaries of Minnesota or Wisconsin. Minnesota has the third-highest, Wisconsin has the sixth-highest, and Illinois has the eleventh-highest number of registered recreational boats in the nation.

Also it was agreed that a set of data on waves generated by recreational craft would be collected from the upper portion of the Mississippi River during a busy holiday weekend to determine the variability and magnitudes of these waves. On the basis of these initial discussions and some site visits, an Illinois River site and a Mississippi River site were selected. Descriptions of the instrumentation used and of the data collection procedures at the two sites are given in the following subsections.

Instrumentation

Wave Measuring System

The wave measuring system consists of two electronic wave gages, an interface, two 30.5-meter (100-foot) cables connecting the wave gages and the interface, and a portable personal computer (PC). The two wave gages and the interface were built at the Illinois State Water Survey in 1982 (Bhowmik et al., 1982) and were modified for this project to connect to a portable PC. Thus the data input-output became faster than with the original Commodore-32 minicomputer.

These wave gages operate on the basis of counting the number of contacts on their sensor boards. One wave gage is equipped with a 0.91-meter (36-inch) sensor board with 60 equally spaced sensor grids, and the other has a 1.52-meter sensor board with 100 equally spaced sensor grids. The distance between sensor grids is 0.015 meter. Each sensor grid connects to an electronic package on the top of the wave gage. Both the sensor board and electronic package are protected by PVC pipe cases. For a detailed description of the wave gage structure, readers are referred to Bhowmik et al. (1982).

The wave gage receives power and a 1 kHz clocking signal from the wave gage interface via a 33.3-meter, 15-twisted-pair cable. Using these inputs, the wave gage sequences up the contacts one by one, starting at the bottom of the gage. When the gage gets to a contact that is out of the water, it stops the sequence and loads that number onto the eight data lines to the interface every 1/10 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 the wave gages. Contained within the wave gage interface is a miniature data-logging computer. The data logging is controlled by a software program written in BASIC and residing in EPROM (Erasable Programmable Read Only Memory). The maximum sampling rate is 10 samples per second and the total storage space is 512 K.

This data-logging computer sequentially scans the output of the wave gage and loads the contact number information into its memory. Communication with outside media (main storage, e.g., a PC) is through a standard 9-pin RS232 serial port, which is mounted on the outside of the interface.

Wind Monitoring System

A recording wind set was used in this project. The wind set can measure both wind speed and direction, and these data were collected by an automated data-logging system. During each field trip the wind set was installed at the site in an area clear of trees and other obstructions. The wind set was turned on each morning and was run continuously until the monitoring stopped in the evening.

Surveying Instruments

The surveying instruments used during the field work included two Lietz TM-10C precision theodolites. The theodolites were used primarily for tracking the sailing line of the boats within the test site. These instruments were also used for the site survey to define the shoreline position and the location of all data gathering instruments.

Timing watches were used to determine the time taken by a boat to travel a known distance. Tape measures were used to establish baselines on shore.

Procedures Used at Illinois River Site near Havana, Illinois

The Illinois River site is located near Havana, Illinois, between River Miles 121 and 122. Data were collected on July 17—20, 1989. The general plan view of this site is shown in figure 2. Access to the site was obtained through the levee operated by the Thompson Lake Drainage and Levee District. The site is fairly uniform and has a steep levee bank above a beach sloping gently to the river.

Figure 3 is a photograph of the site. Figure 4 shows the site with the locations of the boat tracks and approximate locations where bed materials were sampled, and figure 5 shows where all the measuring equipment was installed. Wave gage 1 was installed 11 meters from the shore where the depth of water was 0.7 meter, and wave gage 2 was installed 19.2 meters from the shore where the depth of water was 1.4 meters.

Baseline X-X was established near the shoreline as shown in figure 5. Two transponders for the microfix system were installed at stations Z_1 and Z_2 at a distance of 61 meters on either side of the index line (figures 5 and 6). These two transponders and a third transponder on the Water Survey's research boat "Monitor" were used for detailed surveying of the site, including determination of the locations of the track lines.

Two wind sets were installed as shown in figures 7 and 8. Wind set 1 was installed at a height of 3.3 meters above the ground surface, and wind set 2 was installed at a height of 1 meter above the water surface and at the bankline.

The wave gages (identified by highway cones on the top) may be seen in figures 8 and 9 along with staff gages and other instruments. Wave gage 1 was installed at a depth of 1.2 meter, and wave gage 2 was installed at a depth of 1.5 meters. A close-up view of the wave gages is shown in figure 10, and figure 11 shows a tow passing the wave measuring setup.

After the measuring equipment was installed, tracks for the controlled runs were established at 26.3,41.5, 72, and 102.5 meters from the shore (figures 4 and 5). Since the wave gages were 11 and 19.2 meters from the shore, the distances of these tracks from the wave gages were as shown in table 1.

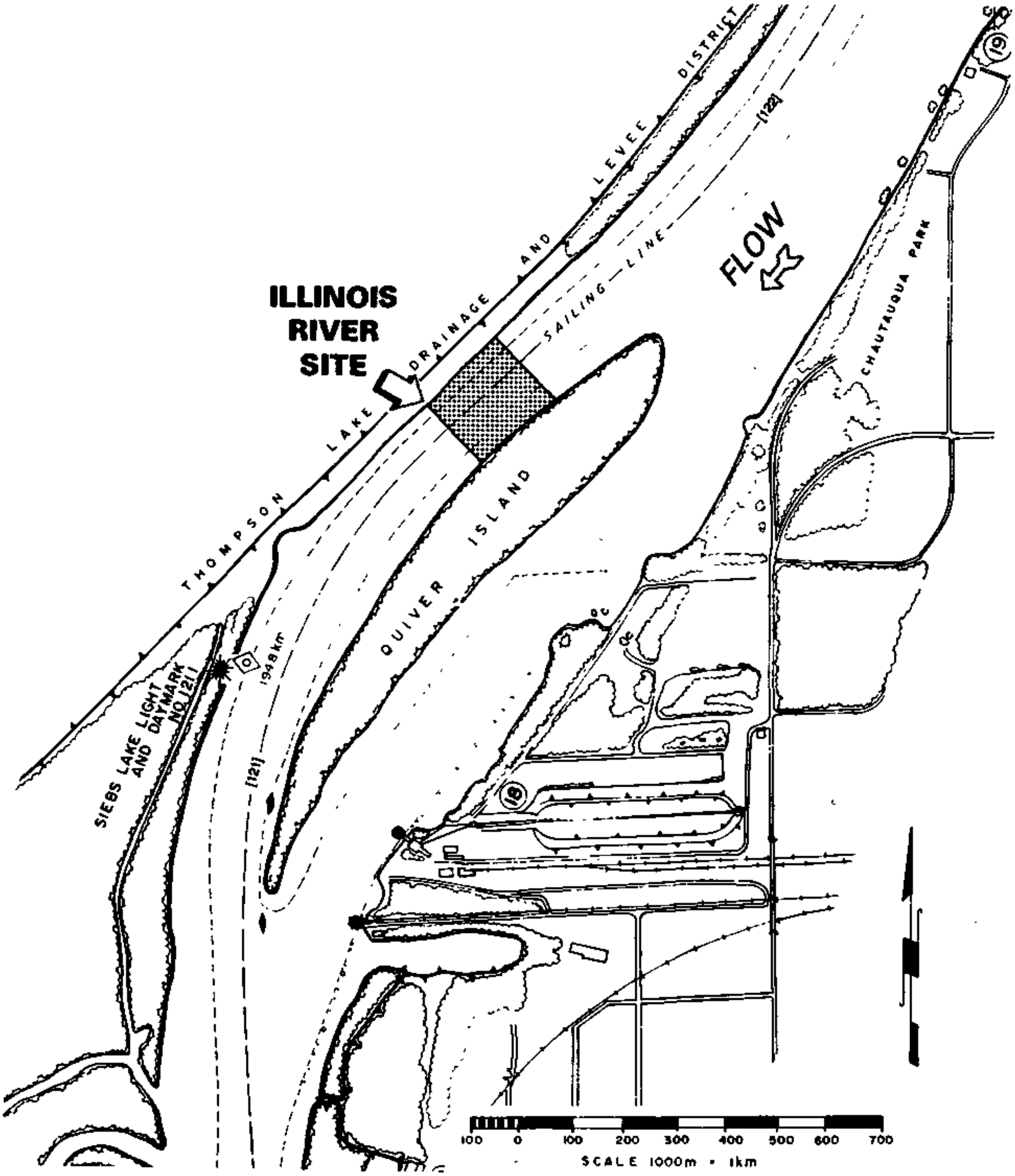

Figure 2. General plan view of the Illinois River site near Havana

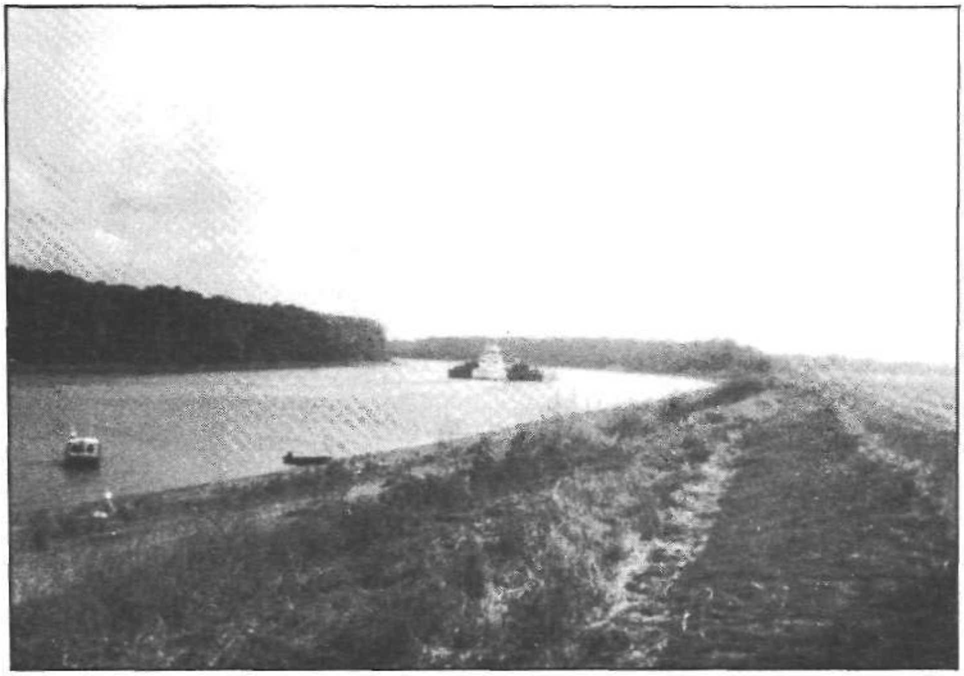

Figure 3. Illinois River site near Havana, showing the levee and the Illinois River

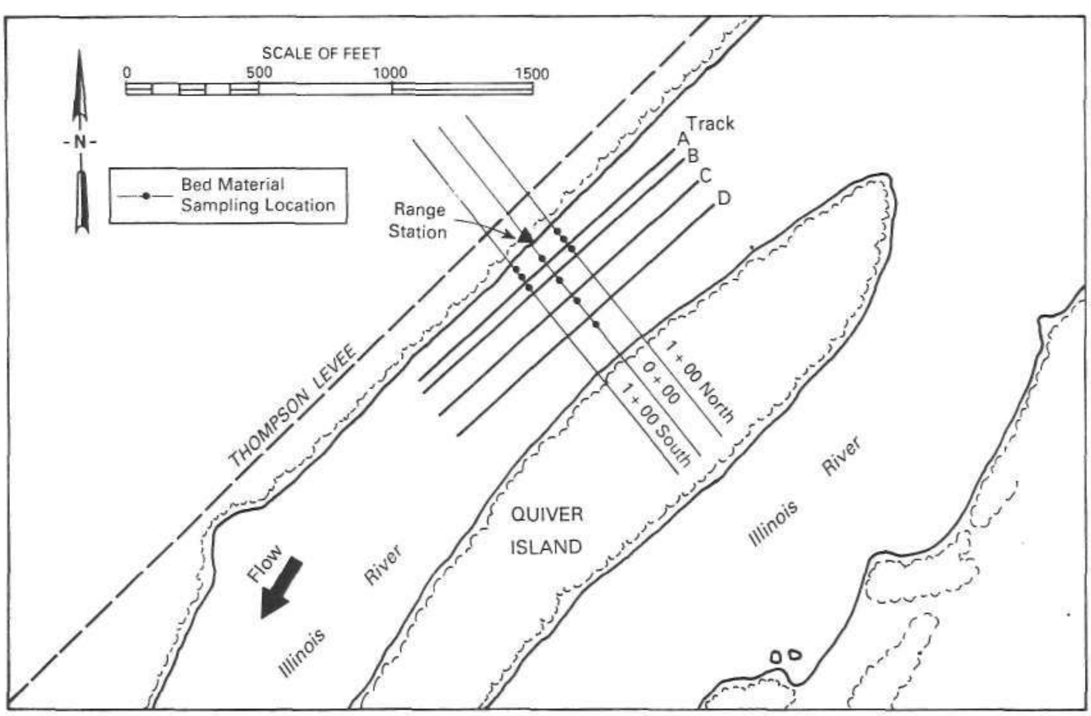

Figure 4. Havana site, showing locations of the boat tracks
and bed material sampling sites

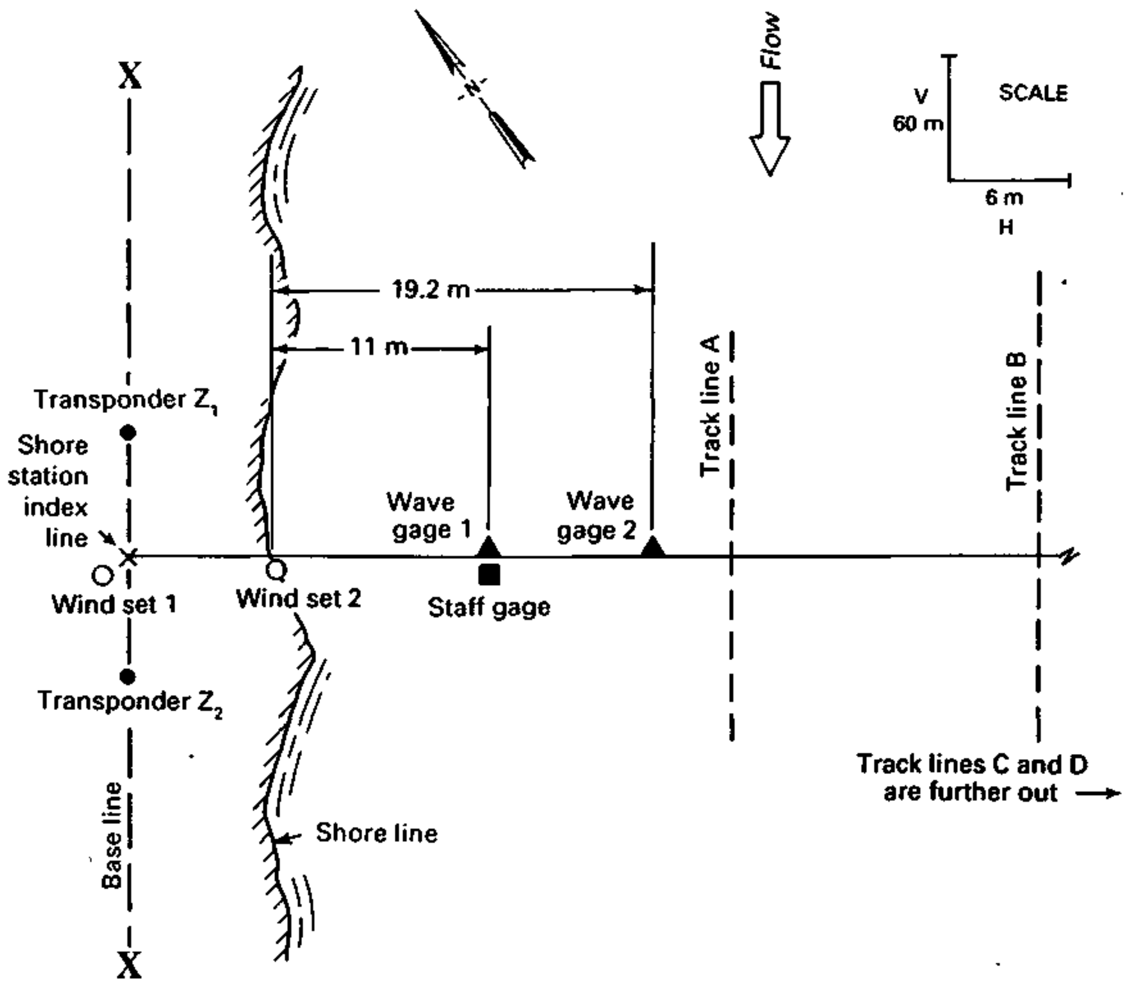

Figure 5. Instrumentation setup at the Havana site

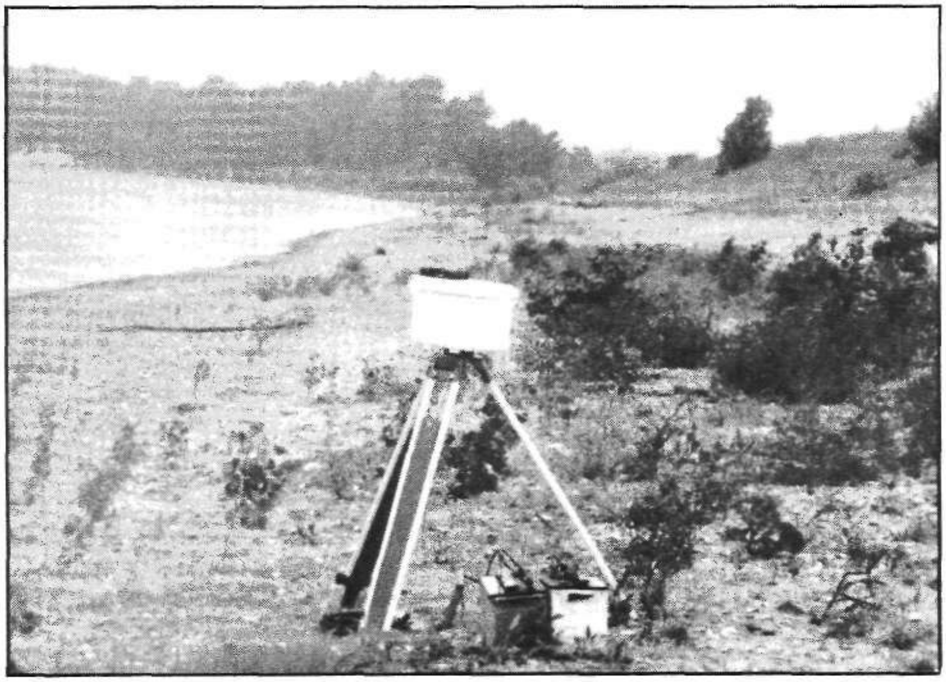

Figure 6. Transponder setup at the Havana site

Figure 7. Recording wind set 1 installed at a height of 3.3 meters
above the water surface

Figure 8. Instrumentation setup at the Havana site
(Note wind set 2, installed at a height of 1 meter
at the water and bankline interface)

Figure 9. Wave gages and staff gage setup at the Havana site

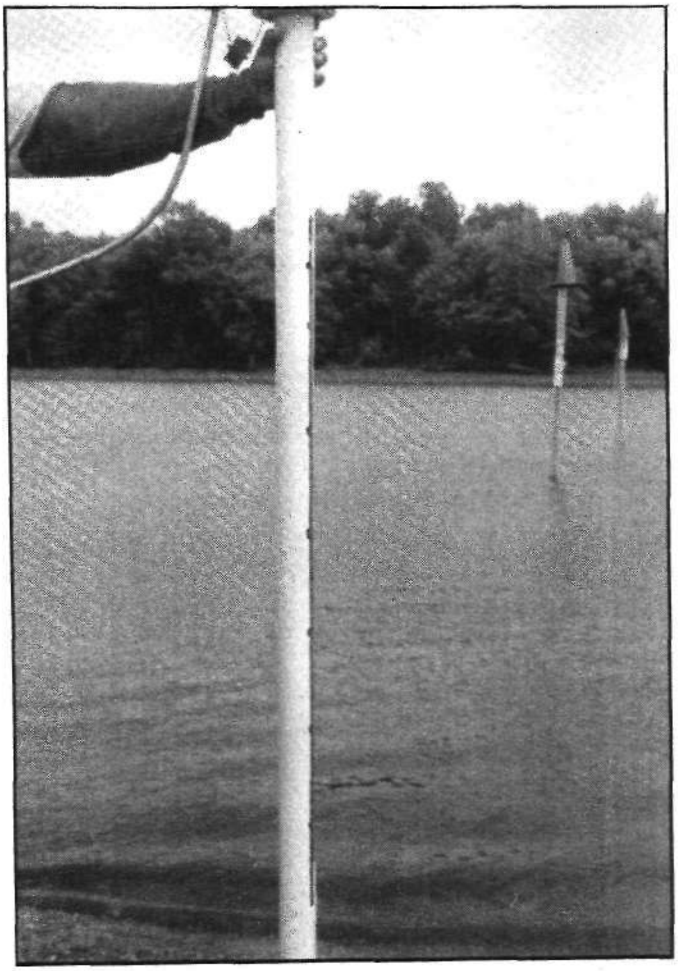

Figure 10. Close-up view of the wave gages

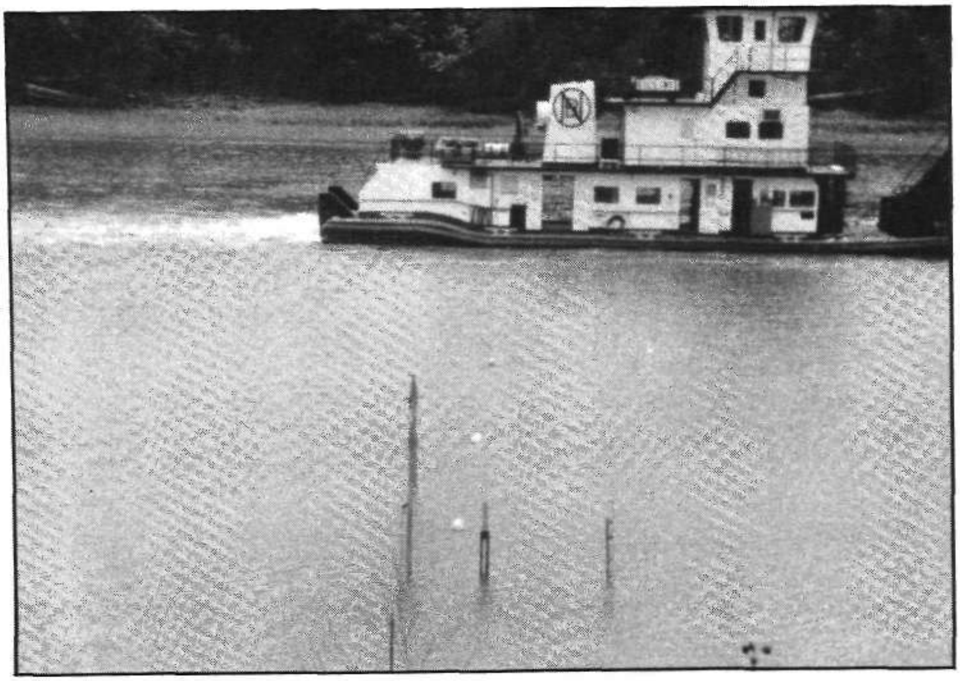

Figure 11. Wave measuring setup with a tow passing in the background

Table 1. Distances of the Wave Gages from the Boat Tracks, Havana Site

Track	Distances, meters	
	Wave gage 1	*Wave gage 2*
A	15.3	7.0
B	30.5	22.3
C	61.0	52.8
D	91.5	83.3

The track locations were selected such that the waves generated by boats travelling along those tracks would give a variety of waves that would be representative of the waves generated by uncontrolled boat movement. Also the researchers have observed that a distance of about 100 meters is the outer limit where the waves caused by an average recreational boat would have effects on the shoreline. The track locations were determined in the field on the basis of a general evaluation of the site.

Each track was marked with three colored floats to facilitate the running of the boats along the track lines. Figure 12 shows three of the floats for tracks A, B, and C. The float closest to the staff gage in figure 12 indicates the location of another auxiliary measuring station. The cruiser shown in the background of this figure is one of the recreational craft that happened to pass this site during the field data collection. Figure 13 shows a panoramic view of the field setup. The procedure for collecting the data was as follows:

1) All equipment was checked for calibration and operational problems.
2) Preliminary runs were made by several boats along all the tracks to determine the feasibility of running these boats along the track lines at fairly constant speeds.
3) Once the readiness of all the equipment was checked, a two-person crew ran one of the boats along track A in one direction at a constant speed.
4) The speed of the boat was determined by the time it took to move a distance of 122 meters between two markers on the shore.
5) Each boat was then allowed to run along each track at three speeds (low, medium, and high) in both the upstream and downstream directions. Low speed indicates that the boat was essentially sitting flat on the water and moving with an approximately equal draft from bow to stern. Medium speed indicates that the boat was plowing through the water. At high speed, the boat was essentially skimming the surface (on its "plane"). Thus 24 runs were normally made for each boat along all the four tracks.
6) Sufficient time was allocated between runs to allow the water surface to return to the ambient condition.
7) Wave data were collected continuously by both wave gages for all the runs.
8) In addition to the wave data, additional data on velocity changes, turbidity, and pressure fluctuations during the passage of normal river traffic were collected. These data will be presented in a subsequent report.
9) Other data that were collected included:
    - Bed material samples at different locations across the width of the river.
    - Average velocities at 15 to 20 verticals to determine the discharge at the time of the field experiment.
    - Depth-integrated suspended sediment samples at 15 to 20 verticals to determine the suspended sediment load of the river.
    - Wind data, including data on direction and magnitude.

Thus for each trip the data collection included variables such as wave height, duration, wave height distribution, distances and draft of the stationary boats, boat speed and directions, water depths, bed materials, ambient velocities, suspended sediment concentrations, water temperatures, and other related variables.

A chronological record of all the event3 during the data collection period was kept for future use and reference. Except for uncontrolled boating events such as the passage of barges or recreational boats, no attempts were made to gather other data related to navigation activity at the site. Figure 14 shows a test run at the site with a runabout, and figure 15 shows typical waves generated from a passing recreational boat at the test section.

Procedures Used at Mississippi River Site near Red Wing, Minnesota

Controlled Runs

The second set of data on controlled field experiments was collected from the Mississippi River near Red Wing, Minnesota, between River Miles 788 and 789. This site is located within the Colville Park area of the city of Red Wing. Figure 16 shows a plan view of this site, including the locations of the tracks used

Figure 12. Field setup, with floats for three of the tracks shown

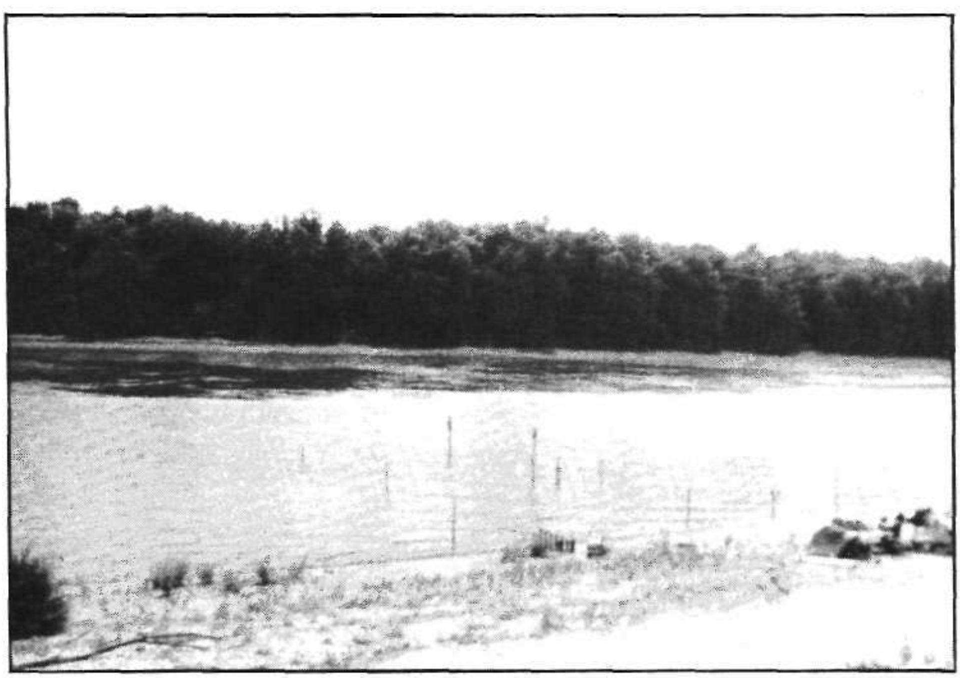

Figure 13. Panoramic view of the field measuring arrangements

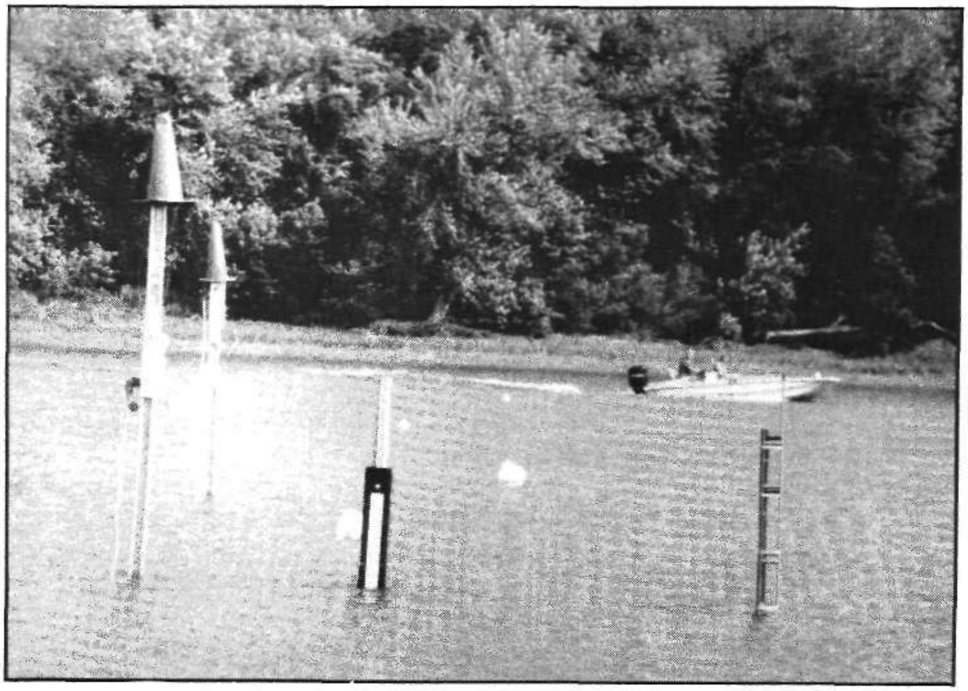

Figure 14. Typical control run at the Havana site

Figure 15. Waves generated by a passing towboat at the Havana site

in the controlled experiment and the approximate locations where bed material samples were taken.

Data collection for all the controlled runs was identical to that at the Havana site, described previously. However, only three tracks were used at this site instead of the four tracks used at the Havana site (figures 4 and 5). These tracks were located 30.5, 45.7, and 76.2 meters from the shore. Thus, 18 runs were normally made for each boat, consisting of three speeds and two directions along each track.

Figure 17 shows the detailed plan view of the wave gage, staff gage, and wind set installations at this site. Wave gage 1 was installed 13 meters from shore at a depth of 1.0 meter, and wave gage 2 was installed 19 meters from shore at a depth of 1.9 meters. Figures 18 and 19 show the typical controlled wave data collection arrangements. All the necessary background data were collected by using similar procedures to the procedures used at the Havana site.

Uncontrolled Boating Events

The main objective in selecting the Minnesota site was to attempt to collect wave data generated by recreational boats during a holiday weekend when heavy recreational use of the river was expected. Historically, intensive use of the river by recreational boaters occurs during holiday weekends such as the Labor Day weekend. Thus the two-week period from August 28 through September 8, 1989, was selected for these field experiments.

During the initial two to three days of the field experiments at Red Wing, data were collected on waves generated by controlled movement of boats. However, from Friday, September 1, through Monday, September 4, all efforts were concentrated on collecting wave data generated by the recreational boats passing the test site. Because of the excellent weather conditions during this period, the Mississippi River near Red Wing was extremely crowded

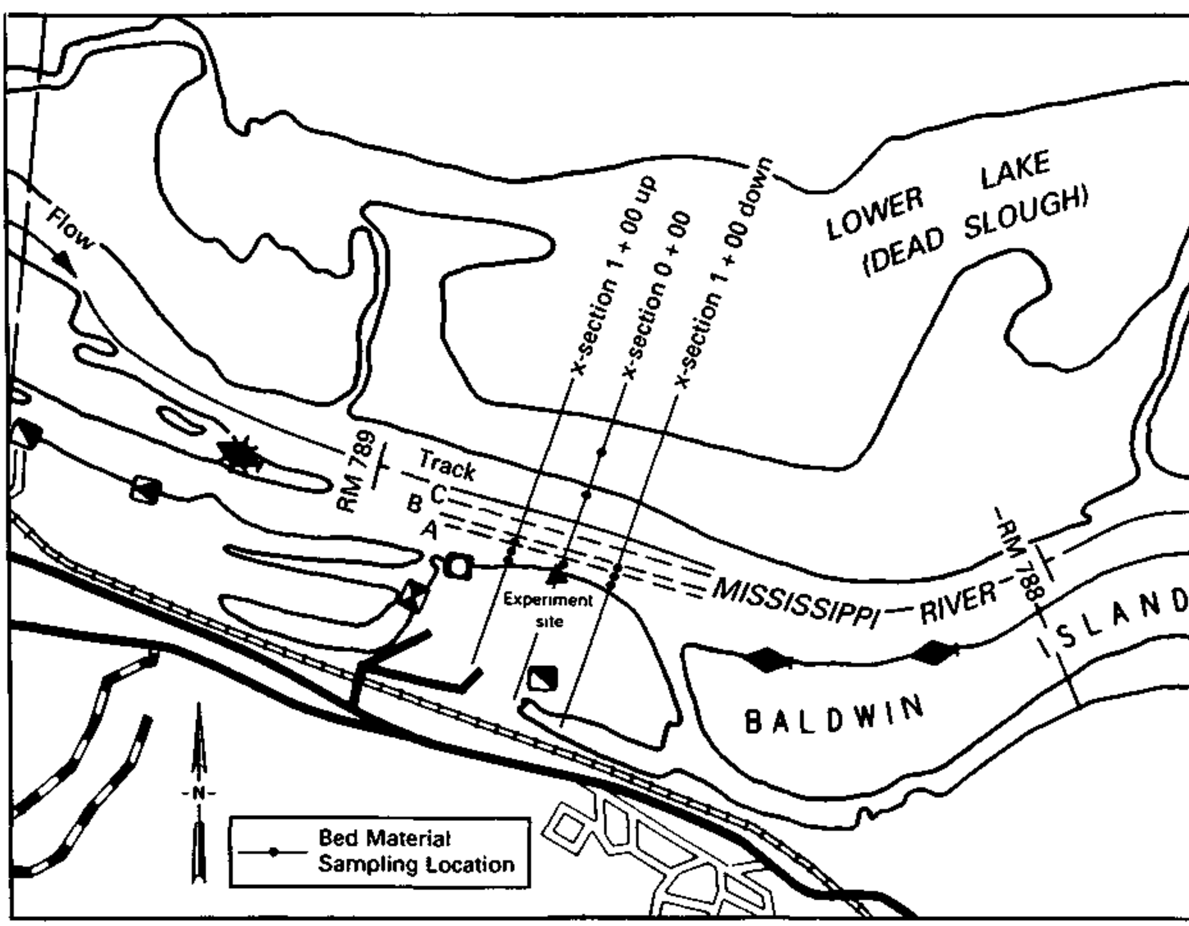

Figure 16. General plan view of the Mississippi River site near Red Wing, Minnesota

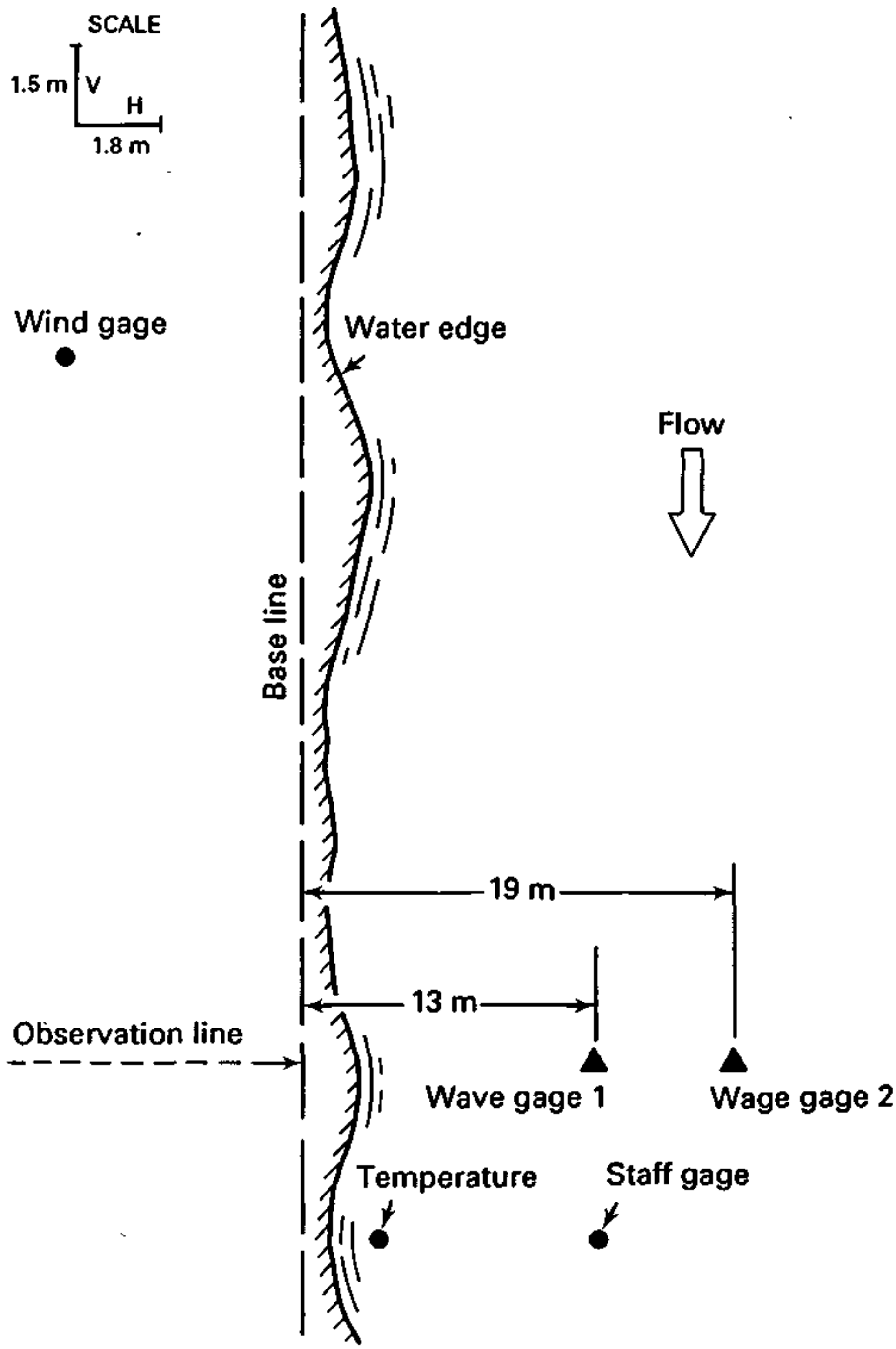

Figure 17. Plan view of the field setup, Red Wing site

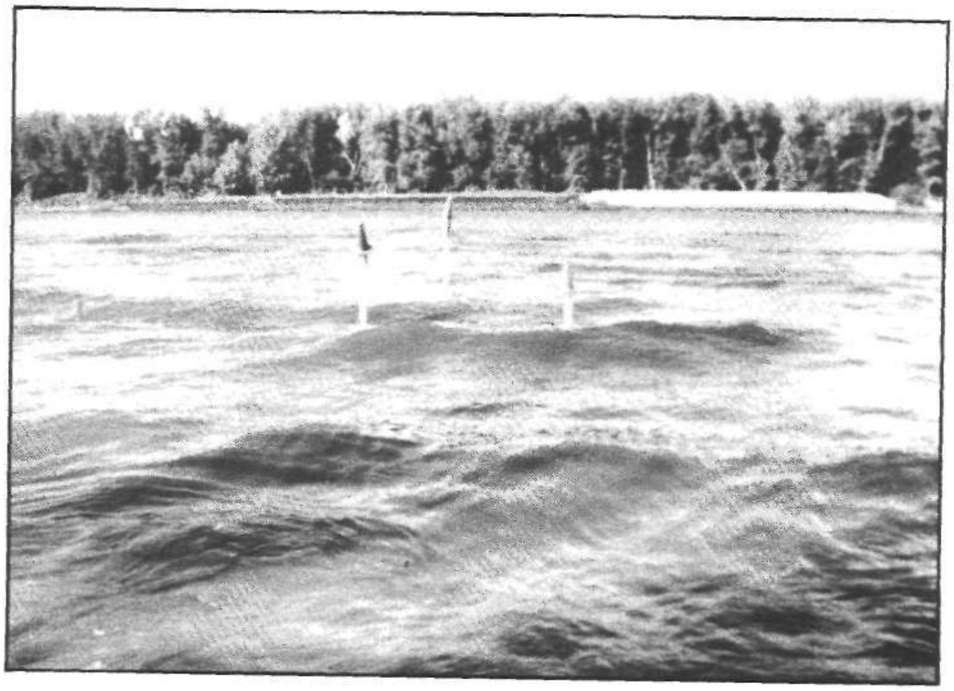

Figure 18. Wave gages and staff gage setup at the Red Wing site

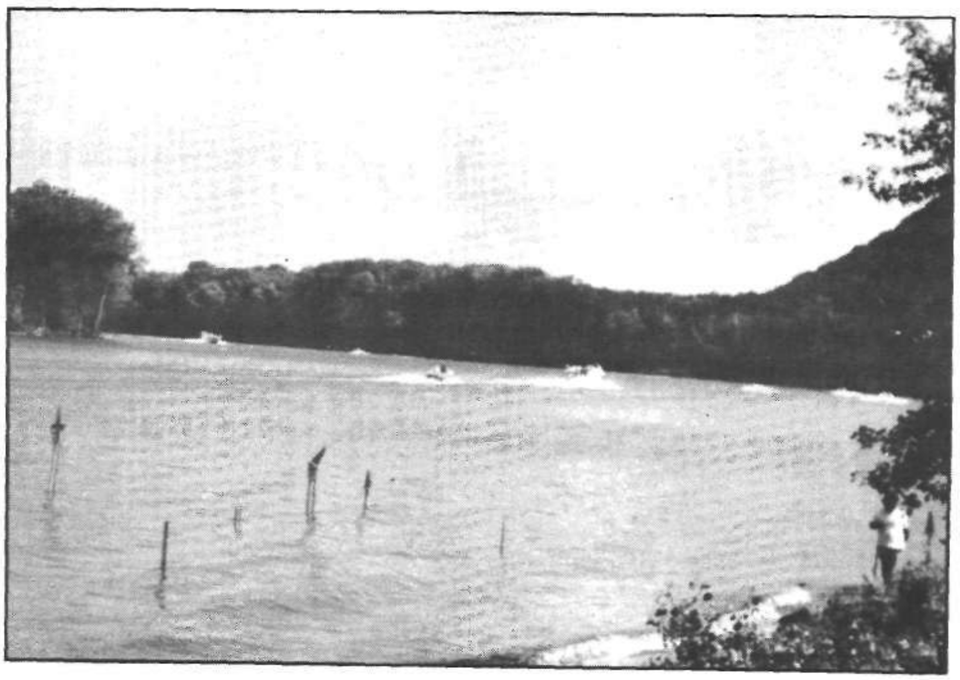

Figure 19. General data collection arrangement for the controlled runs at the Red Wing site

with recreational craft, and no other data could have been collected even if an attempt had been made to do so.

At the beginning of the data collection period for uncontrolled boating event data, information on name, registration number, size, shape, speed, direction, sailing angle, and distance from the shore was sought for every boat passing the site. This information could then be correlated with wave data collected by the two wave gages. The boat speed, sailing angle, and distance were measured by using two theodolites 100 meters apart on one side of the river and three fixed markers, spaced 100 meters apart, on the other side of the river.

However, after about 9:00 a.m. on Friday, so many recreational craft passed the site that it became apparent that it would be impossible to keep track of this information. Thus after vain attempts were made to determine the speed of each boat passing the site for about an hour or so on Friday, this portion of the data collection was abandoned because of the unreliability and difficulty of collecting the boat speed data and correlating those data with the individual wave trains traversing the wave gages.

At this time it was decided to count all the boats passing the test site, record their approximate distances from the wave gages (on the basis of floats located near the gages), and note their direction of movement. The type of boats and their approximate lengths were also noted. On the basis of the controlled runs with boats of known characteristics, the authors were able to estimate the lengths of these boats. Notations were also made as to whether each boat was moving at a high, medium, or low speed.

Wave data collection followed a similar procedure to that described for the controlled runs. Even though it became impossible to correlate the individual waves with any particular boat, a general type of correlation existed between the frequency of the boat passages and the waves measured at the gages. At many times, the river took on the appearance of a busy interstate highway through a large metropolitan city, with six or seven boats passing at the same time in each direction.

Figure 20 shows some of the typical boats that passed this site and the waves generated by those boats during the 1989 Labor Day weekend. All the data were checked regularly in the field to ensure that the wave gages were working properly and the necessary wave data were accurately recorded. Subsequently, all the data were taken to the office for analyses.

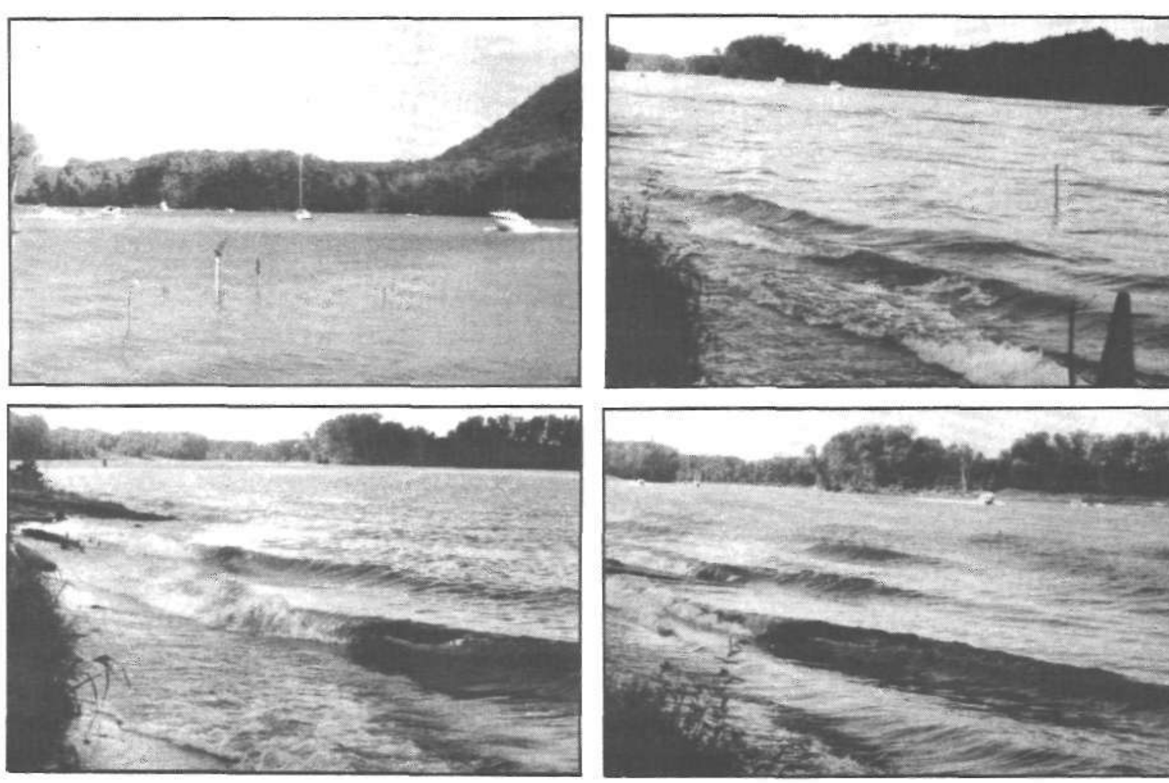

Figure 20. Typical recreational boat movement and waves generated at the Red Wing site on the Mississippi River

ANALYSIS OF DATA

This section has five major parts, in which the following topics are discussed:

1) The background data collected at the two sites.
2) Various wave characteristics that were considered in the analysis of the data.
3) Data on waves generated by the controlled runs.
4) A regression-type model for predicting wave heights generated by recreational boat movement in a waterway, developed on the basis of the data for the controlled runs. This relationship is recommended for future use in developing management alternatives to protect streams and rivers against waves generated by recreational craft. The proposed equation can also be used for lake environments because the blocking ratios for recreational craft at both sites are above 20.
5) Data on waves generated by uncontrolled boating events during the 1989 Labor Day weekend.

Background Data

Background data collection included measurements of discharges, bed material sampling, suspended sediment sampling, and characterization of the field site. These data are presented separately for the two sites.

Havana Site (Illinois River)

For the Havana site, located between River Miles 121 and 122 on the Illinois River, the closest gaging station is at Meredosia (River Mile 71.3) with a drainage area of 67,467 square kilometers (sq km). The drainage area of the Illinois River at Havana is 47,433 sq km. Since discharge data are not available for the Havana site, the discharge data for Meredosia were used to develop an approximate flow-duration curve for the Havana site.

Figure 21 shows the flow-duration curve for Meredosia. An approximate flow-duration curve for Havana was derived from the Meredosia data on the assumption that the discharge is directly proportional to the drainage area, and this curve is also shown in figure 21. The discharge for the period July 17-20,1989, is also indicated on this plot.

Flow conditions during the field experiments were obtained from the U.S. Geological Survey, Urbana, IL. For July 17-20, the average discharge was 150 cubic meters per second (cms). The flow-duration curve in figure 21 indicates that this discharge had a frequency of occurrence of about 80 percent, i.e., 80 percent of the time the flow on the Illinois River near Havana will be equal to or greater than this value. Thus the streamflow at this site when the data were collected was in the lower ranges of the long-term average flows.

Background data collection also included data on velocity distribution across the whole width of the river. Figure 22a shows the lateral distribution of the vertically averaged velocity measured at the Havana site on July 17, 1989. The velocity data were used to develop isovel plots for the site, which are shown in figure 22b. An examination of these plots shows that the velocity distribution across the channel is not exactly symmetrical, but that the maximum velocity occurs somewhat to the east side of the channel. The locations of the wave gages are shown in figure 22c.

Nine bed material samples were collected. Three were collected at the data collection site, three were collected 30 meters upstream from the site, and three were collected 30 meters downstream from the site. All the samples were analyzed for particle size dis-

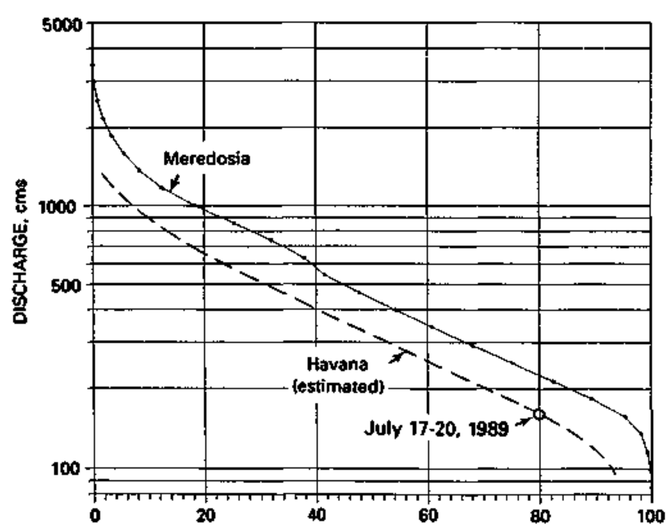

Figure 21. Flow-duration curve for the Illinois River at Meredosia, at River Mile 71.3

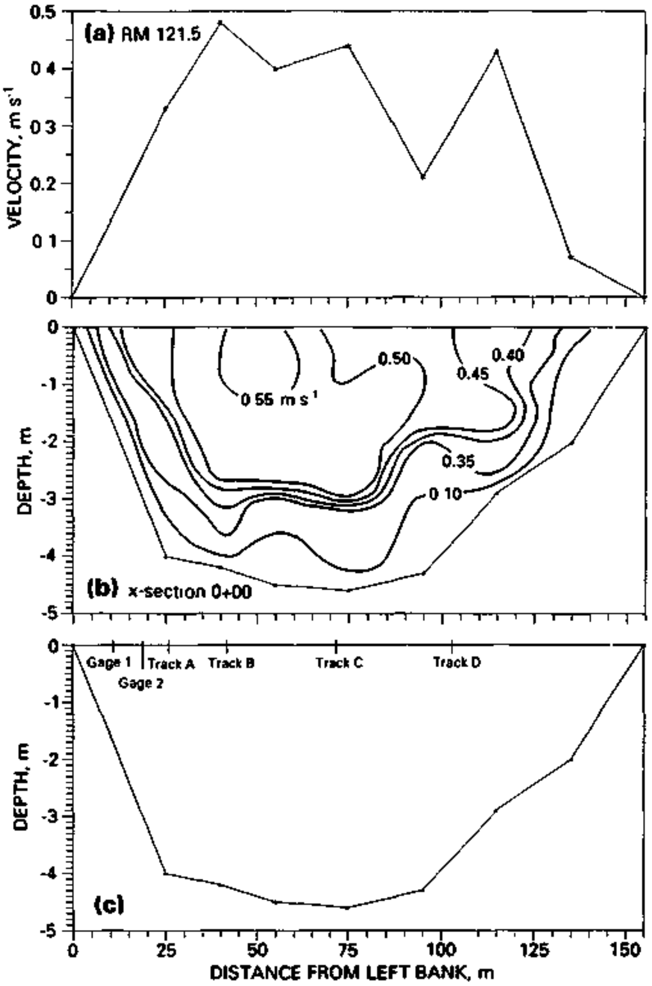

Figure 22. (a) Lateral velocity distribution and (b) isovel plots, Havana site, July 17, 1989, and (c) wave gage and track locations at the site

tributions. Appendix A-1 shows the particle size distributions of these samples.

The bed material analyses show that sand is the major type of material in the river bed. At this reach, coarser particles exist near the shore area, especially within a distance of 18 meters. Finer particles quickly dominate with increased distance from the shore. By using D_{50} (median of the distribution) as an index, one finds that:

a) Within 18 meters from the shoreline, the materials vary from very coarse sand to gravel.
b) Beyond 18 meters but less than 25 meters, the materials are mostly fine sand.
c) Beyond 25 meters from the shoreline, the bed materials are dominated by very fine sand.

Appendix A-2 shows some of the characteristics of the bed materials at this site. The standard deviation (a) and uniformity coefficient (U) are used to measure the gradation of the particles. Higher values of a and U indicate a very well-graded material, whereas lower values of σ and U indicate uniform particle size. From Appendix A-2 it can be seen that upstream of the survey site the bed materials are well-graded; at the survey site the distribution becomes fairly uniform; and some gradation again occurs downstream from the survey site.

Wind data were collected on July 18 through 20. These data show that the wind was quite nominal with an average (arithmetic mean) magnitude of about 2.22 meters per second. Table 2 summarizes the wind data from this site.

Table 2. Daily Wind Characteristics at the Havana Site, July 18-July 20, 1989

Date	*Speed (m/s)*	*Direction (degrees)*
July 18	**1.33**	**203**
July 19	**1.35**	**15**
July 20	**3.98**	**27**

Red Wing Site (Mississippi River)

Background data similar to those collected at the Havana site were collected at the Red Wing site in Minnesota. In addition, suspended sediment concentration samples were collected. The Red Wing site is located between River Miles 788 and 789 on the Mississippi River, and the closest gaging station where long-term discharge records are available is at Lock and Dam 3 at River Mile 797.

The flow-duration curve developed for Lock and Dam 3 is shown in figure 23. During the field experiments, the discharge showed an increase from 146 cms on August 30 to 332 cms on September 5. These two flows had approximate frequencies of occurrence of 92 percent and 54 percent, respectively (figure 23).

Isovels developed from the velocity distribution data are given in figure 24c. The lateral velocity distribution is depicted in figure 24b. The channel is somewhat deeper near the left-hand side of the river (looking downstream), but the velocity distribution is quite symmetrical. The velocities were less than those at the Havana site.

Eleven suspended sediment samples were collected from this site. The lateral distribution of these suspended sediment samples is shown in figure 24a.

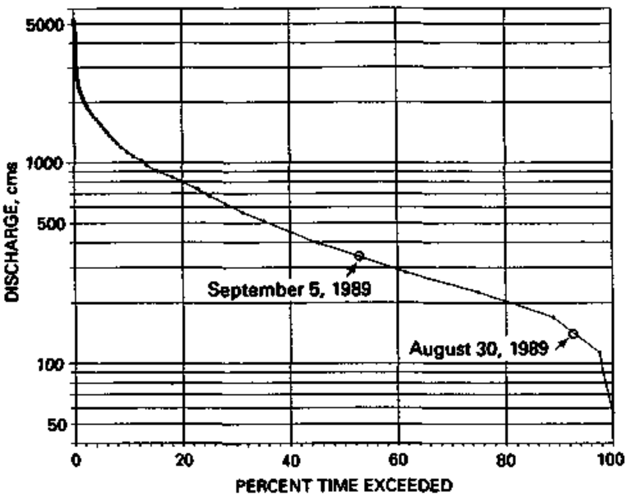

Figure 23. Flow-duration curve for the Mississippi River at Lock and Dam 3, at River Mile 797

Except for a single sample near the center of the river, the suspended sediment concentrations appear to vary symmetrically from the center line. At this site on the Mississippi River, the sediment load was calculated as 574 tons per day on the basis of the discharge and the sediment concentration data for September 5, 1989.

Fifteen bed material samples were collected in the field. Of these, nine samples were taken at the data collection site, three were collected 30 meters downstream, and three were collected 30 meters upstream. The size distributions of these bed material samples are given in Appendix A-3. Appendix A-4 shows the characteristics of these materials.

The analysis in Appendix A-4 indicates that the bed materials are coarser near the shore area and become finer toward the center of the river. However, the range in the distribution is narrow, and all D_{50} values are in the sand classification. It can be observed that:

a) Within 12 meters of the shore zone, the bed materials are mostly coarse sand.
b) Beyond 12 meters, the bed materials are in the medium sand classification.
c) In the range of 140 to 210 meters, the bed materials are in the fine sand classification.

Again, the a and U values were calculated for these bed material samples. In contrast to the Havana site, this reach of the Mississippi River has a fairly uniform distribution of particle sizes, and they are all in the sand classification.

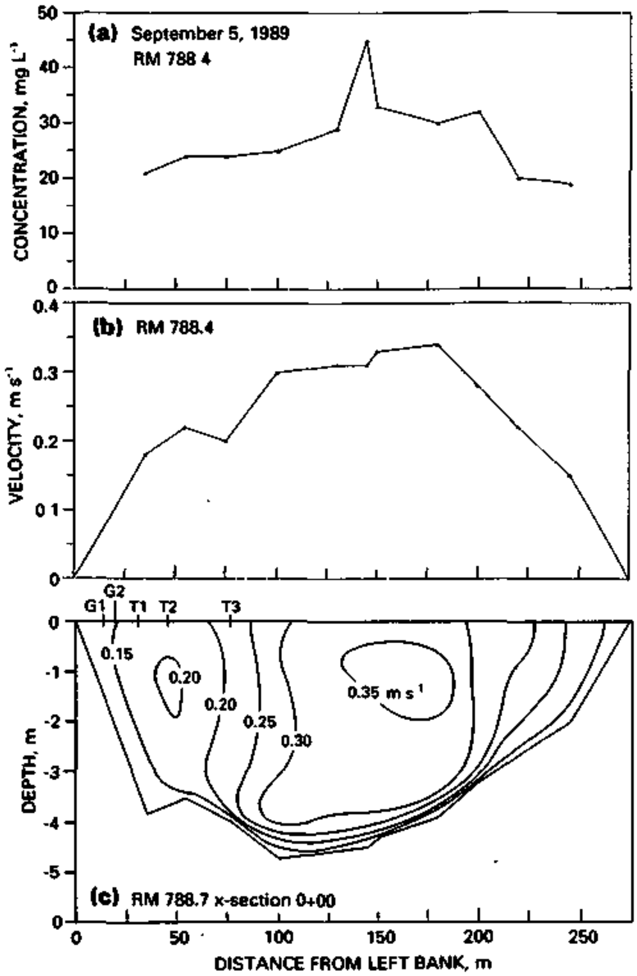

Figure 24. (a) Lateral distribution of suspended sediment concentrations, (b) lateral velocity distribution, and (c) isovel plots, Red Wing site, September 5, 1989

The wind characteristics at this site were monitored from September 1 through September 5, 1989. Table 3 shows the variations in wind speed and direction in daily average values (arithmetic mean). During these five days, the average wind speed was 1.8 meters per second.

Wave data collected during the period when recreational boats were not passing the site showed that wave heights were generally less than 0.03 meter. These waves were generated by wind only. If the wind was blowing from the west (flow direction) with substantial speed, wind-generated waves at this site probably would be substantial.

The maximum wave height that can be generated at this site for a specified wind speed and direction can be computed only by using the equation reported by Bhowmik (1976), which was based on research conducted by many researchers. During the field experiments, the wind speed was generally low and no significant wave activity was generated by the wind at this site.

In evaluating the stability of a streambank against water waves, it must be kept in mind that any of the causes of waves (recreational boats, commercial navigation traffic, or winds), or a combination of causes, could play a dominant role in the stability of a bank. In analyzing the streambank stability, one must consider all these factors in addition to the geotechnical properties of the bank and bank materials, and the ambient flow conditions in the river.

Table 3,. Daily Wind Characteristics at the Red Wing Site, September 1-5, 1989

Date	*Speed (m/s)*	*Direction (degrees)*
September 1	3.85	326
September 2	0.55	83
September 3	1.84	171
September 4	1.97	263
September 5	0.76	181

Determination of Wave Characteristics

The techniques followed in collecting wave data for controlled and uncontrolled boating events were described previously. Wave data for waves generated by wind and recreational boats were collected 10 times per second, and the data were logged continuously. Figure 25 shows a typical wave event generated by the controlled run of a recreational boat. Such plots have been used to determine maximum and average wave heights, total wave duration, number of waves in an event, and period and steepness of each individual wave.

Wave energy and power spectra can also be derived from these data. Wave speed and attenuation of waves could not be derived because the wave directions are not uniform for all controlled runs, and exact information on the separation and superimposition of the waves as they travel between two wave gages is not available.

Definitions of some of the wave parameters (figure 25) are as follows. Some modifications were made in the definitions of these parameters since boat waves do not have regular sinusoidal profiles, the classic shape used in defining various wave parameters.

Maximum Wave Height: Maximum wave height is the highest wave that occurs within a wave event. Wave height is measured from the trough to the crest.

Period: Wave period is the elapsed time from crest to crest or trough to trough of a continuous wave. For maximum wave heights generated by recreational craft, only half of the period is determined (figure 25).

Steepness: Steepness for the maximum wave is derived by dividing wave height by the corresponding period.

Duration: Duration of an event is the total time that has elapsed from the beginning to the end of a wave train.

Number of Waves: Total number of waves in an event.

Average Wave Height: Average wave height is calculated by averaging all the wave heights in a wave event.

Significant Wave Height: Significant wave height is the average wave height of the highest one-third of the waves during a measuring interval.

Wave Energy: Wave energy consists of kinetic and potential energy. These energies are usually calculated by concentrating on a controlled water column for one wave crest over the wave length. Therefore these energies are generally described as the average energy per unit surface area.

The average total energy per unit surface area is. the sum of the average potential energy and the average kinetic energy densities (Ippen, 1966). For

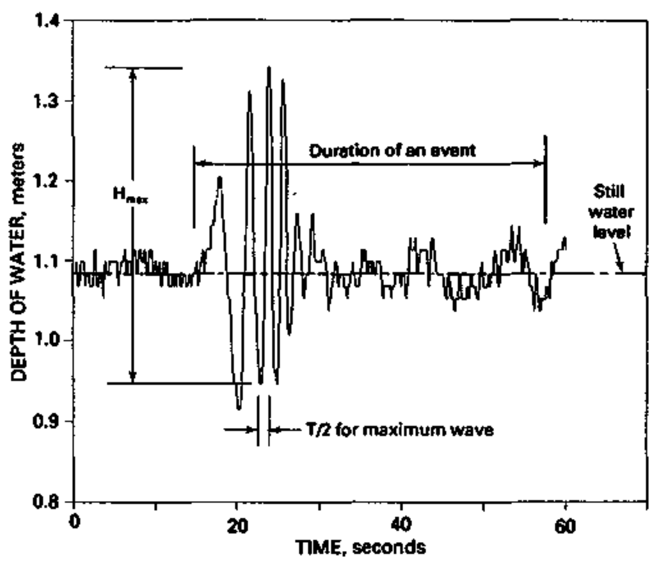

Figure 25. Typical wave event generated by the controlled run of a recreational boat at the Red Wing site

simple harmonic, small-amplitude waves, the average kinetic energy density (KE) and average potential energy density (PE) are equal and they are:

$$KE = PE = \gamma a^2/4 \tag{3}$$

where *y* is the unit weight of water and a is the wave amplitude, which is one-half of the wave height. Therefore the total energy per unit area equals

$$E = KE + PE = \gamma a^2/2 \tag{4}$$

Boat waves are generally not simple harmonic waves. When calculating wave energy for complex wave forms, the Fourier transform (Brigham, 1974) is used. Harmonic components in each complex wave are sought and the total energy, E, per unit area is the sum of energy calculated for each harmonic (Ippen, 1966).

$$E = \sum e_i = \gamma (\sum a_i^2)/2 \tag{5}$$

Power Spectrum: The power spectrum is the Fourier transform of the covariance function. Assuming that the time series of wave profiles is stationary and made up of mixtures of cosine waves, its variance can be decomposed into components of average power or variance at various frequencies.

$$\sigma^2 = \int_{-\infty}^{\infty} \Gamma(f)\, df \tag{6}$$

where a^2 is the variance and $\Gamma(f)$ is called the power spectrum of the stochastic process. Thus $\Gamma(f)$ df is an approximate measure of the average power or variance in the frequency band f to f + 8f.

Waves Generated by Controlled Runs

Wave data were collected by using two wave gages installed about 8 meters apart at the Havana site (figure 5) and 6 meters apart at the Red Wing site (figure 17). Some typical waves generated by the controlled runs are plotted in figures 26 and 27. An examination of 24 plots (12 from the figures and 12 others) shows that the wave train generated by an individual recreational craft generally lasts less than one minute and has a discrete number of waves. Also in general, the duration of wave trains is directly related to the distances between the wave gages and the boats. Longer-duration wave trains were associated with distant boats rather than closer boats.

Obviously other parameters also affect the shape, duration, and magnitudes of the waves produced by various recreational craft. An examination of many plots similar to those in figures 26 and 27 indicates that, for similar circumstances, downbound boats produce waves having a slightly longer duration than those produced by upbound boats; and with an increase in speed, the duration of the waves also increases. It appears that for smaller boats, there exists an optimum distance at which the duration of the waves is the longest. For all the runs, the planing speed approximately equaled the highest speed at which field data were collected.

At the Havana site, four tracks were used and boats were run at three different speeds. Table 4 lists the boats used; their types, lengths in meters, and sitting drafts in meters; number of tracks used by each boat; range of speeds; and total number of runs for each boat. As can be seen, 126 runs were conducted at this site. Table 5 presents similar data for the Red Wing site, where 120 runs were conducted. Thus the total number of controlled runs was 246.

The data collected for these runs are summarized in Appendix B-1 for the Havana site and in Appendix B-2 for the Red Wing site. Generalized analyses were made to determine the characteristics of the waves generated by controlled runs of typical recreational boats moving on the UMRS.

It has been mentioned that each individual recreational boat generated a discrete number of waves normally lasting less than one minute. The number of waves and the total durations of these waves, including the maximum and average wave heights, are important parameters that can be used in developing management alternatives for shorelines and near-shore zones or "wave-wash zones" of streams and rivers experiencing heavy recreational traffic.

Figure 28 shows frequency histograms of the number of waves generated by recreational boats in the controlled runs. This figure illustrates that on the average, about 12 to 15 waves were generated by individual boats travelling at various distances from the wave gages at three different speeds. The total durations of waves generated by controlled runs was also analyzed. Figure 29 shows histograms for the durations of the waves measured by the wave gages at both field sites. The median duration of all the waves was about 20 to 26 seconds, with some extending about 50 seconds or more.

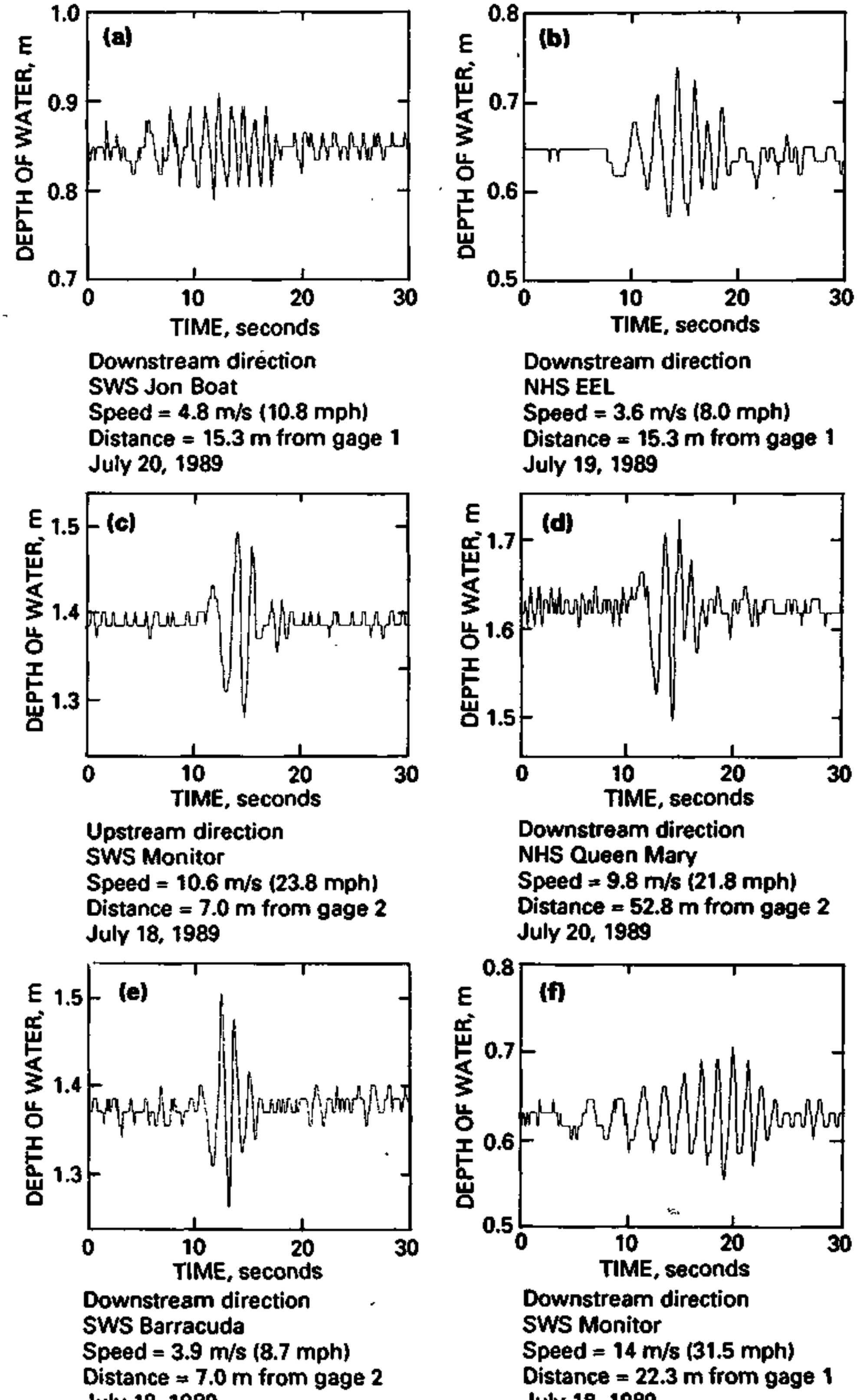

Figure 26. Typical waves generated by recreational boats during controlled runs, Havana site

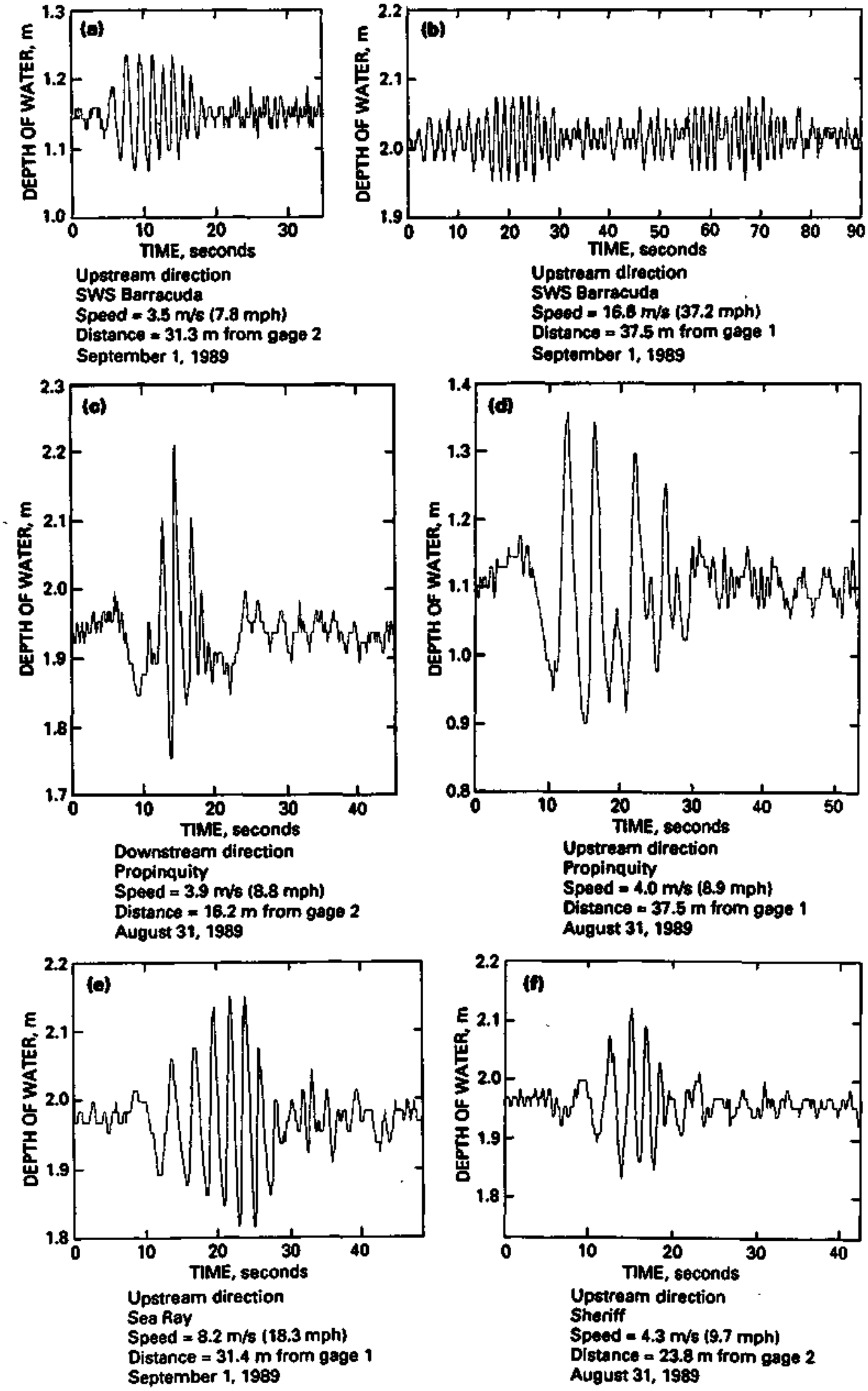

Figure 27. Typical waves generated by recreational boats during controlled runs, Red Wing site

Table 4. Controlled Runs at the Havana Site, July 17-20, 1989

Name of boat	*Type (length, m)*	*Sitting draft, m*	*No. of tracks*	*Range of speed*	*No. of runs*
Barracuda	**V-Hull (5.8)**	**0.15**	**4**	**4 speeds**	**28**
EEL	**Flat Bottom (5.5)**	**0.15**	**4**	**4 speeds**	**26**
Jon Boat	**Flat Bottom (3.7)**	**0.1**	**3**	**3 speeds**	**12**
Monitor	**Tri-Hull (5.8)**	**0.3**	**4**	**4 speeds**	**24**
Queen Mary	**V-Hull (5.5)**	**0.2**	**3**	**4 speeds**	**24**
River Diver	**Pontoon (7.3)**	**0.2**	**3**	**3 speeds**	**12**

Total no. of runs = 126

Table 5. Controlled Runs at the Red Wing Site, August 30 - September 6, 1989

Name of boat	*Type (length, m)*	*Sitting draft, m*	*No. of tracks*	*Range of speed*	*No. of runs*
Barracuda	**V-Hull (5.8)**	**0.15**	**3**	**3 speeds**	**24**
Propinquity	**Cabin Cruiser (14.3)**	**0.76**	**3**	**2 speeds**	**18**
Scorpion	**Chris Craft (11.9)**	**0.61**	**3**	**4 speeds**	**18**
Sea Ray	**Cabin Cruiser (11.3)**	**0.61**	**3**	**3 speeds**	**18**
Lund Baron	**V-Hull (6.4)**	**0.67**	**3**	**4 speeds**	**18**
Trojan	**Cabin Cruiser (7.9)**	**0.61**	**2**	**3 speeds**	**12**
Aluminum Craft	**V-Hull (5.8)**	**0.23**	**2**	**4 speeds**	**12**

Total no. of runs = 120

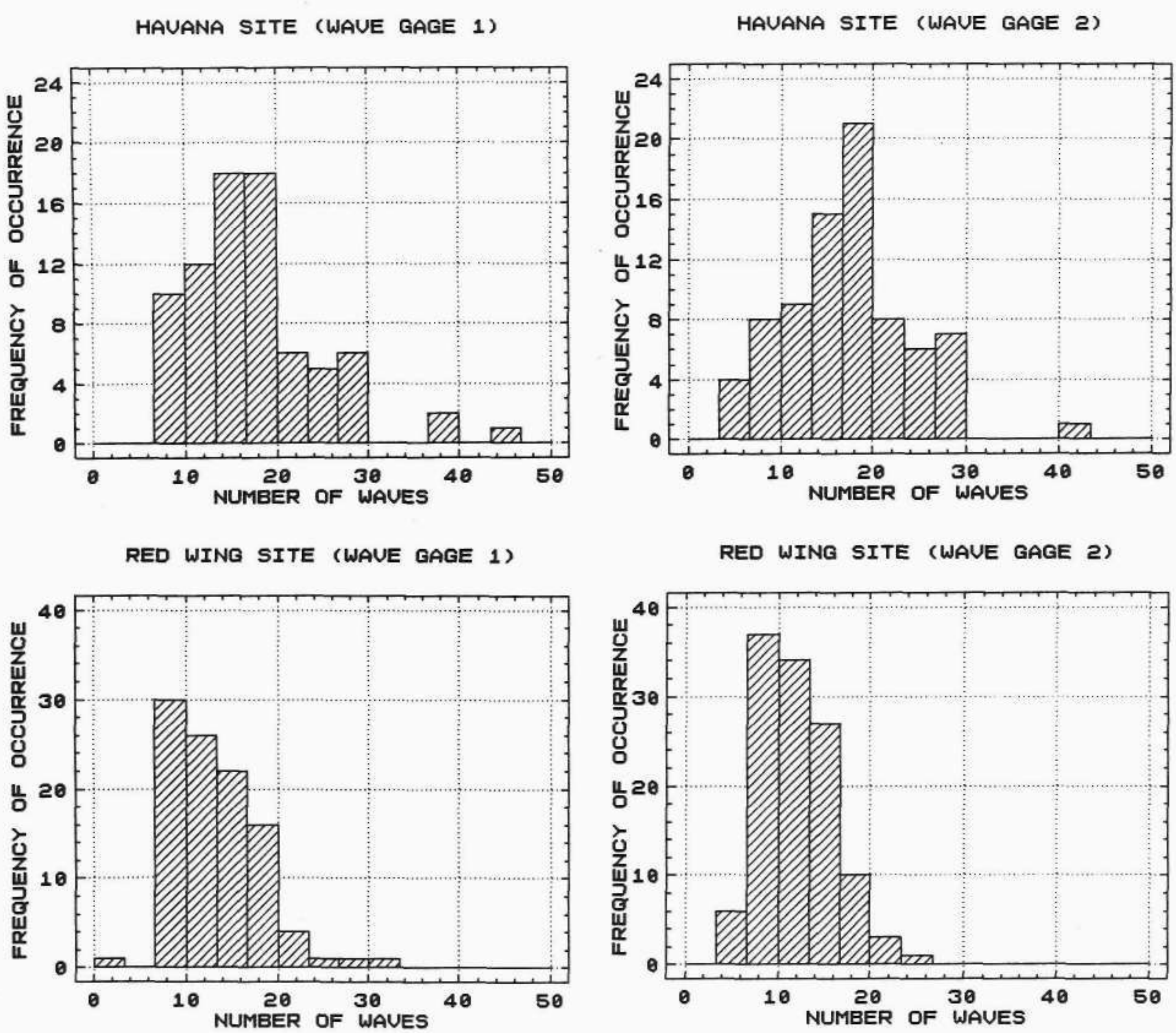

Figure 28. Frequency distributions of the number of waves generated by controlled runs

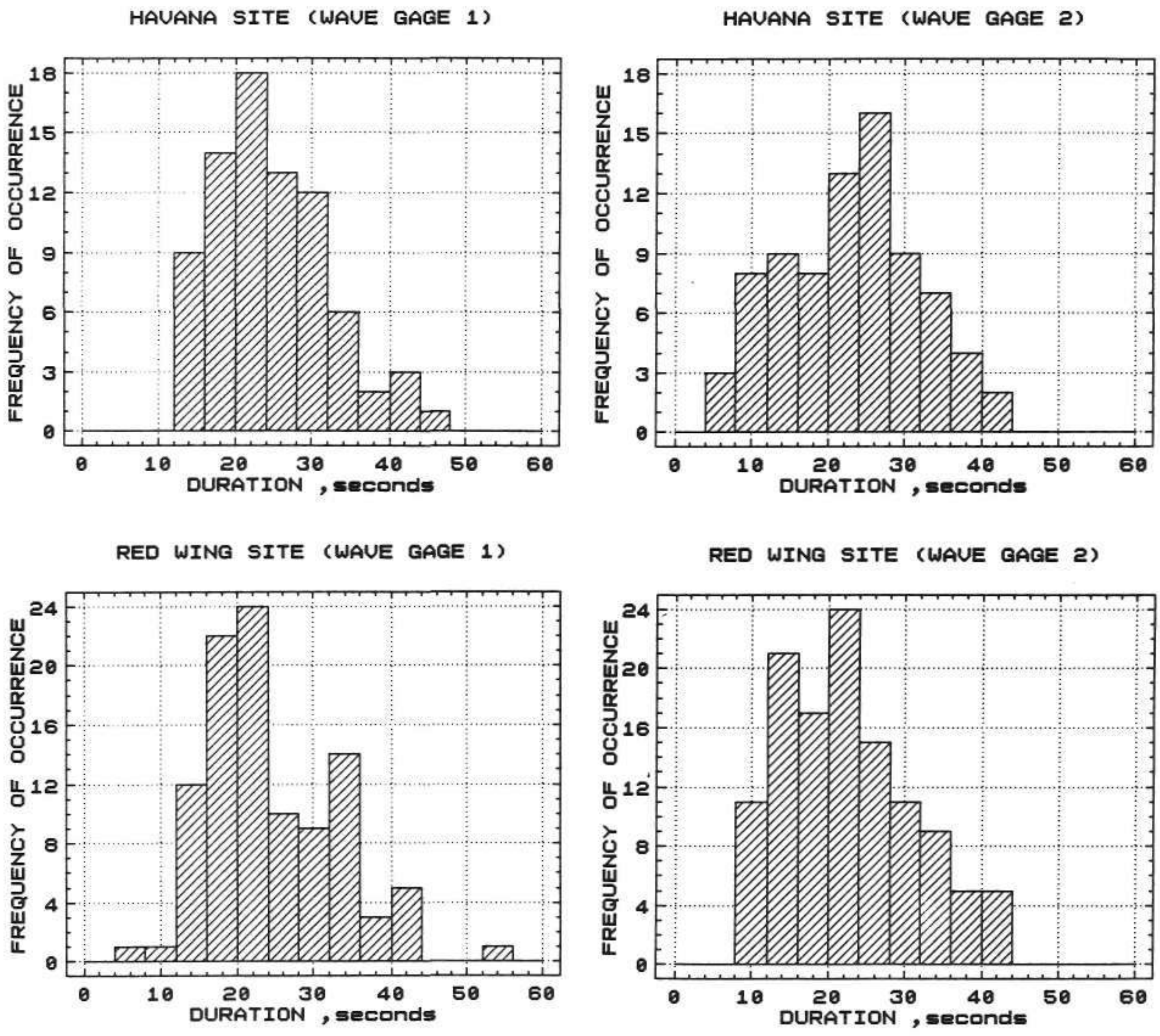

Figure 29. Frequency distributions of the duration of waves for all controlled runs

The average wave heights for all the runs at the two sites are shown in figure 30. This illustration indicates that the average wave height varied from about 0.02 to about 0.25 meter, with the majority of wave trains having an average height of about 0.05 meter. However, the average wave height is a different measure from the maximum wave height. When a similar analysis of the maximum wave heights was made, it was observed that a substantial amount of the waves had maximum wave heights in the range of 0.12 to 0.2 meter, and some of them were as high as 0.6 meter (figure 31).

The analyses presented in figures 28 through 31 are extremely important in analyzing and estimating the bank erosion potential of waves generated by recreational boats. Assuming that a recreational boat can generate an average of about 15 waves with an average duration of 22 seconds and an average height of 0.05 meter, the average energy that must be dissipated by the shoreline can be determined. Using equation 4, the total energy of each wave in this example becomes:

$$E = [(9.81)(0.05/2)^2]/2$$
$$= 0.0031 \text{ kilo-newtons per meter}$$

where γ, the unit weight of water in equation 4, is 9.81 kilo-newtons per cubic meter. Therefore for an average of 15 waves, the total energy becomes about (0.0031 x 15) 0.0465 kilo-newtons per meter. If recreational boats are moving continuously, it is expected that similar wave energy needs to be dissipated or reflected by the shoreline per unit time. In this analysis, it was assumed that all the energy will be dissipated at the water and land surface interface, although it is known that wave heights decrease as waves break on a sloping bank, which will also dissipate some of the wave energy.

The erosion threshold for a streambank can be reached if a series of waves having amplitudes less than the maximum wave heights impinge on the banks in quick succession. This can be illustrated by considering a series of 10 waves having an average amplitude of about 0.05 meter in a wave train where the maximum wave height is 0.12 meter. The energy content of these waves is 0.0465 kilo-newtons per meter, compared to the energy content of 0.0177 kilo-newtons per meter for a single wave 0.12 meter high. Thus a series of small-amplitude waves could have as much impact on a bank as a single large wave, even though a single large wave could exceed the threshold value for the initiation of bank failure at a specific site with certain types of bank characteristics.

The computation shown above is for simple harmonic wave patterns. Since boat waves are not simple harmonics, computations are also made to determine the total energy (E) per unit area as the sum of the energies for each harmonic. This analysis is presented in the section "Wave Characteristics."

Predictive Relationship for Maximum Wave Heights

Maximum wave height is an important parameter for run-up and riprap stability. It is the most important parameter in the wave profiles.

Data collected from the controlled runs were used to develop a predictive relationship for the estimation of maximum wave heights generated by recreational traffic. Before this analysis was initiated, the contributing factors and parameters that may be responsible for generating waves were determined. This evaluation showed that the following variables should be considered in the development of a predictive equation:

$$(v, h_p, W_6, g, x, H_m, d, v, L, D)$$

where

v = relative speed of the boat, m/s, $v = v_b - v_f$
where
v_b = absolute speed of the boat, m/s
v_f = water velocity, m/s
h_p = horsepower, newton meters/s
W_B = weight of the boat, newtons
g = acceleration due to gravity, m/s^2
x = distance of the boat from the wave gage, m
H_m = maximum wave height generated by each boat for each run, m
d = draft of the boat, m
v = kinematic viscosity, meter^2/second
L = length of the boat, m
D = water depth, m

In the selection of the variables, maximum wave height was selected rather than significant wave height. In a wave train of 10 to 15 waves generated by recreational boats, it will not be feasible to conduct a statistical analysis to determine the significant wave heights. Consequently, in all of the subsequent analyses of waves generated by recreational boats, maximum heights will be used as the main parameter.

Moreover, it should be noted that the maximum wave height has the highest potential to cause bank erosion. Also, recreational boat waves have a dis-

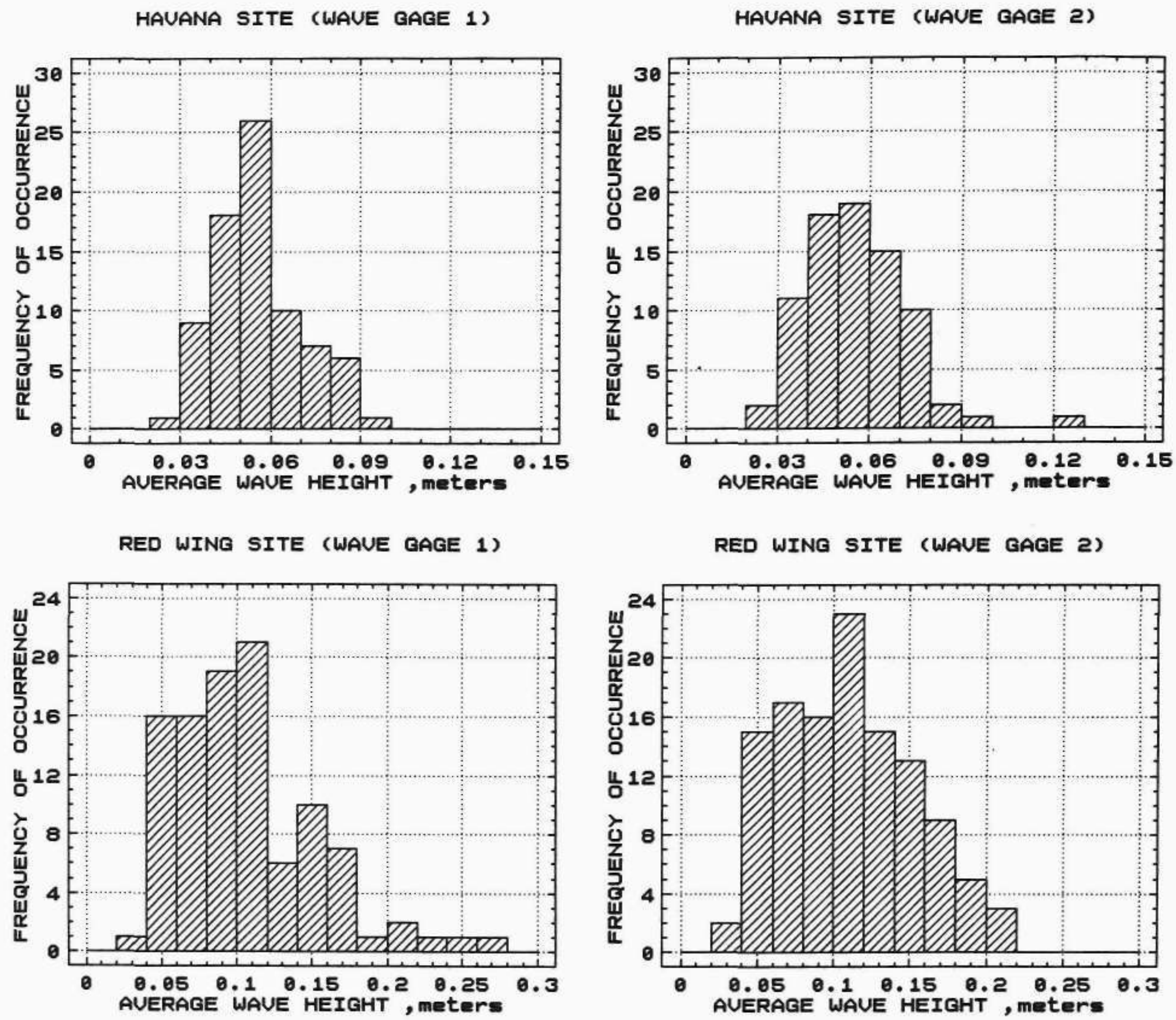

Figure 30. Frequency distributions of average wave heights for all controlled runs

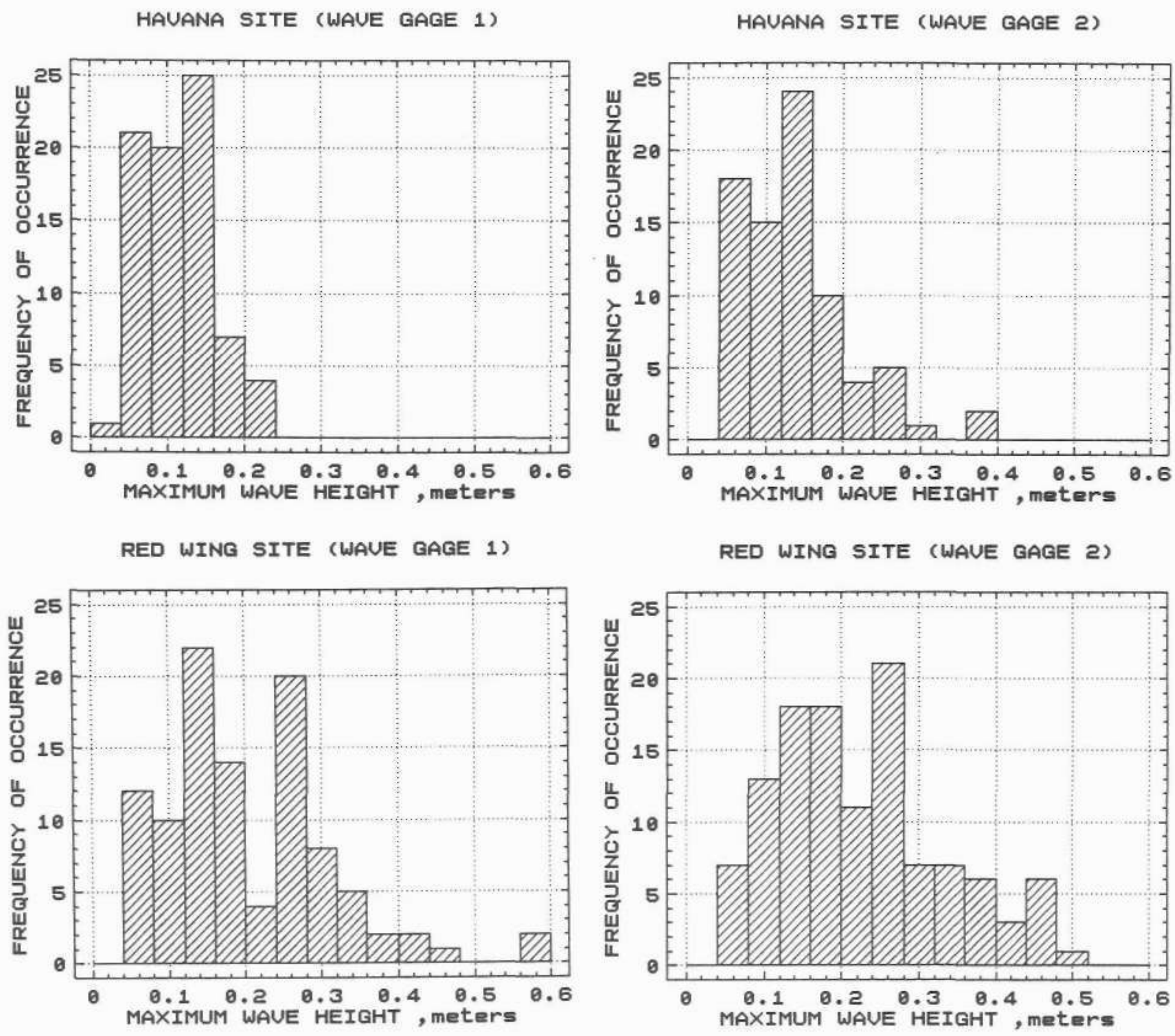

Figure 31. Frequency distributions of maximum wave heights for all controlled runs

tinct maximum H_m and do not resemble wind-generated waves, which generally have recurrent peak values.

Next a dimensional analysis was performed to determine dimensionless parameters to be used in the subsequent evaluation. This analysis indicated that the above parameters can be grouped as follows:

$$(H_m/x) = f[(h_p/vW_s), (gx/v^2), (L/x), (d/x), (v/xv), gD/v^2] \qquad (7)$$

A stepwise regressional analysis was then performed to determine the relative importance of each of the above dimensionless parameters. This analysis involved substituting the numerical value of each parameter in the general relationship and then determining the predictability of H_m as each additional parametric value was added.

Predictive equations based on data for each class of boats would certainly be better than a single predictive equation for all classes of boats. However, to determine such a series of predictive equations, data would have to be collected for all classes of boats for approximately 100 runs. The improvement in the prediction of boat waves with a series of five to seven equations each based on 100 or more runs would not be that significant and would require much greater investment in the field experiments. Because of the numerous limitations and the limited potential for improving the predictive relationships, it was decided to conduct the field experiments as stated previously and to develop a single predictive equation.

Numerical values of draft and horsepower vary in real conditions, and these values may not be easily obtainable. Draft changes with the speed of the boat, and determining the draft for each speed and also for different boats is almost impossible in the field. Therefore draft is represented by the sitting draft of the boat, which can be obtained from the manufacturer's manual or by reading the water mark when the boat is idle on the water. The horsepower is determined on the basis of the manufacturer's specifications.

The above analysis showed that parameters such as h_p and W_8 do not improve the predictability of the equation needed to estimate the wave heights. The final equation that was developed is:

$$(H_m g/V^2) = e^{4.996} (L/x)^{0.560} (gx/v^2)^{0.215} (gv/v^3)^{0.402} (gd/v^2)^{0.355} \qquad (8)$$

This is the original equation in nondimensional form. With numerical values of g equal to 9.81 m/s^2, and v equal to 10^{-6} m^2/s for temperature equal to 24˚C (75˚F), equation 8 can be simplified as:

$$H_m = (0.537)(v^{-0.346})(x^{-0.345})(L^{0.56})(d^{0.355}) \qquad (9)$$

Equations 8 and 9 can be used to estimate the wave heights generated by recreational boats within the UMRS.

An examination of equation 9 reveals that the parameters found to be important for predicting maximum wave heights are essentially the same ones previously found to be important and used by Bhowmik (1975, 1976) in developing his predictive relationship equation (equation 2) for determining maximum wave heights generated by recreational traffic. However, the coefficients and exponents of equation 9 are slightly different from those previously proposed by Bhowmik (equation 2).

To compare the validity of this equation with that of Bhowmik's 1975 equation, field data collected from the controlled runs at Havana and Red Wing were used to compare the measured maximum wave heights with the maximum wave heights predicted by both equations. Initially equation 2 (Bhowmik, 1975, 1976) was used to compute the maximum wave heights for all the controlled runs of the present study.

Figure 32 shows a comparison between the measured maximum wave heights and those predicted by equation 2. The square of the correlation coefficient for this analysis is 0.63. It should be pointed out that equation 2 proposed by Bhowmik (1975, 1976) was developed on the basis of laboratory data collected by Das (1969) and only 13 sets of field data collected by Bhowmik (1976) from Carlyle Lake in Illinois, compared to the 246 sets of data collected for the present study. Thus the database for the development of equation 2 was extremely limited. However, even with this limited database, the parameters that Bhowmik found to be important in 1975 and 1976 were the same parameters that yielded the maximum correlation between the measured and computed wave heights in this 1989 study.

Figure 33 shows a comparison between measured maximum wave heights and those predicted by equation 9. As can be seen, the fit is excellent, and the square of the correlation coefficient has increased from 0.63 for equation 2 to 0.86 for this equation. The scatter of the data from the line of perfect agreement is significantly less than that in figure 32. Equation 9 can be used for predicting maximum wave heights generated by recreational craft on a river. This equation also can be used for predicting wave heights in a lake environment since the block-

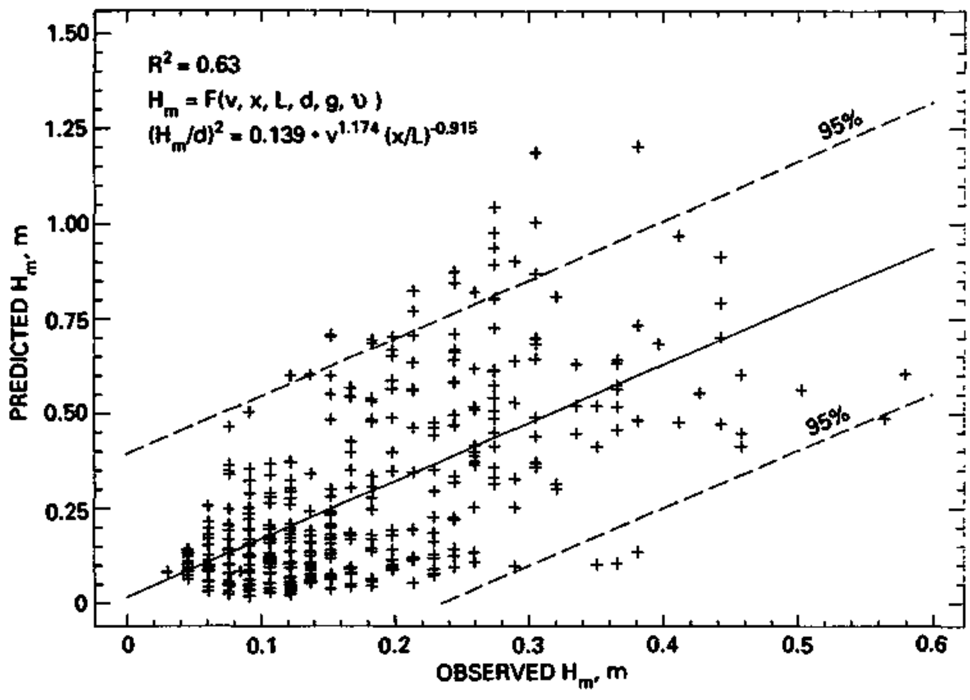

Figure 32. Relationship between measured maximum wave heights and those predicted by equation 2

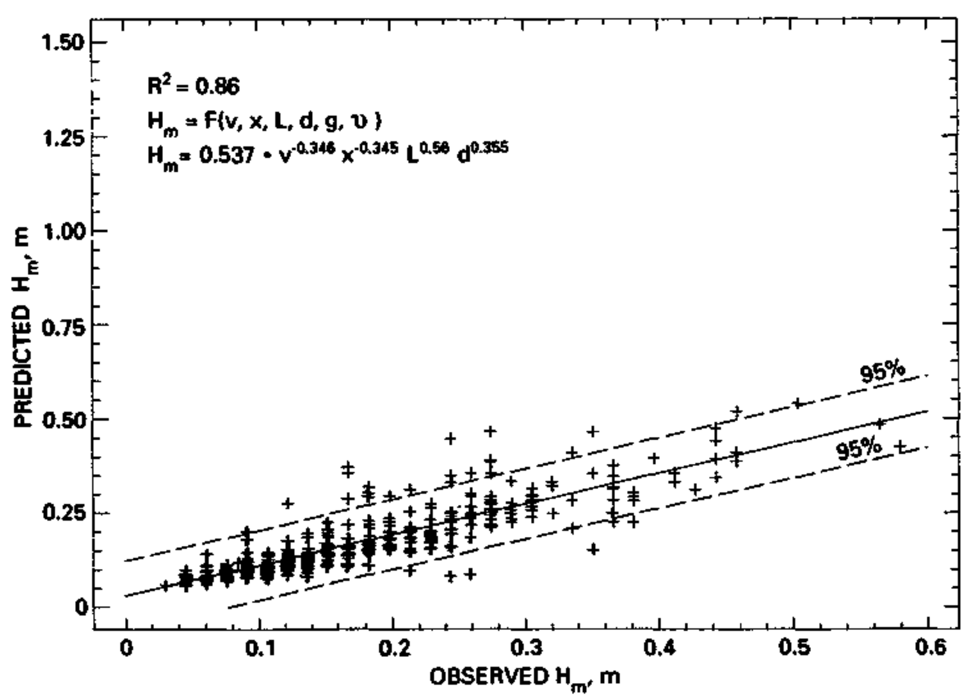

Figure 33. Relationship between measured maximum wave heights and those predicted by equation 9

ing ratios for all these controlled runs were above 20, which should also be the case for lake environments.

At this time readers must be made aware of some of the limitations of equation 9. An examination of the equation will indicate that boat speed is the controlling factor for generation of the maximum wave heights for a single boat with fixed length and draft, moving at a fixed distance from the shore.

Field experience has shown that the draft does not stay constant as the speed of the boat is increased. This has been observed by the present researchers and by other hydraulic and biological scientists working on large rivers and lakes that have heavy recreational traffic (S. Johnson, Minnesota Department of Natural Resources, personal communication, 1991).

Normally as the speed of a boat increases, the submerged volume of the boat decreases, with an associated decrease in the effective draft and in the effective length of the boat that will stay in contact with the water. At some point, most recreational boats will plane out, keeping minimum contact with the water through which they are moving.

Waves generated by recreational boats moving at various speeds with different effective lengths and drafts are not the same. There is an optimum speed for each boat at which the waves generated by the boat will be at the maximum.

To develop a relationship that will show the maximum wave-making characteristics of a boat, parameters such as draft and length used in equation 9 must be considered for each individual boat, and functional relationships between speed and effective length, and between speed and effective draft, have to be developed and used in equation 9 to compute the maximum wave heights. Unfortunately, these types of relationships can be developed only with extensive laboratory and field simulations completely beyond the scope of the present investigation. Moreover, the improvement in the predictability of the wave heights resulting from these changes may not warrant such an extensive investment of time and effort.

Therefore the present investigators opted to keep the boat length and draft constant for each boat at all speeds for use in equation 9. This decision corresponds with the basic goal of the project: to develop an easy-to-use predictive relationship for boats of various sizes and shapes. The alternative would require personnel entrusted with the development of strategies for recreational boat waves to know the specific hydrodynamic characteristics of each boat before they could estimate the wave heights for each boat.

Waves Generated by Uncontrolled Boating Events

Traffic Characteristics

As mentioned previously, data were collected on waves generated by uncontrolled recreational boat movement near the city of Red Wing, Minnesota, on the Mississippi River. These data were analyzed to determine various wave characteristics and to evaluate traffic movement during Labor Day weekend in 1989.

The number of recreational boats passing the test site showed an initial increase from Wednesday, August 30, through Saturday, September 2, and then a steady decrease through Tuesday, September 5. The distribution of boats per day during daylight hours is shown in figure 34. On Saturday, September 2, 704 boats passed the test site.

The frequency distribution of boat passages on an hourly basis was also determined (figure 35). This distribution shows a steady increase in boat traffic from the early morning hours until around 3:00 or 4:00 p.m., after which a steady decrease was observed. By about 6:00 p.m., the number of boats passing the test site decreased to about the same number as observed in the early morning hours. On the average, the number of boat passages in the upstream and downstream directions was approximately the same except that on Saturday, September 2, 263 boats moved in the upstream direction and 441 boats in the downstream direction (figure 36).

The distributions shown in figures 34 and 35 can be exemplified further by showing the distribution of boat passages on an hourly basis on Sunday, September 3, when 557 boats passed the test site. This distribution is shown in figure 37. As can be seen, the distribution is almost symmetrical, with the maximum number of boats passing the test site from about 12:00 noon through 2:00 or 3:00 p.m.

Information on the sizes and shapes of boats was also collected. Since so many boats passed the test site at the same time, especially from September 2 through September 4, data were noted only in general terms as to the size, length, and hull shape of the boats, and their approximate speed and distance from the shore.

Figures 38 and 39 show the frequency distributions of the boats in terms of boat length and boat shape, respectively. Figure 38 shows that in general the majority of boats passing the test site were between 6.1 and 9.2 meters (20 and 30 feet) long. This of course was expected for a riverine environment

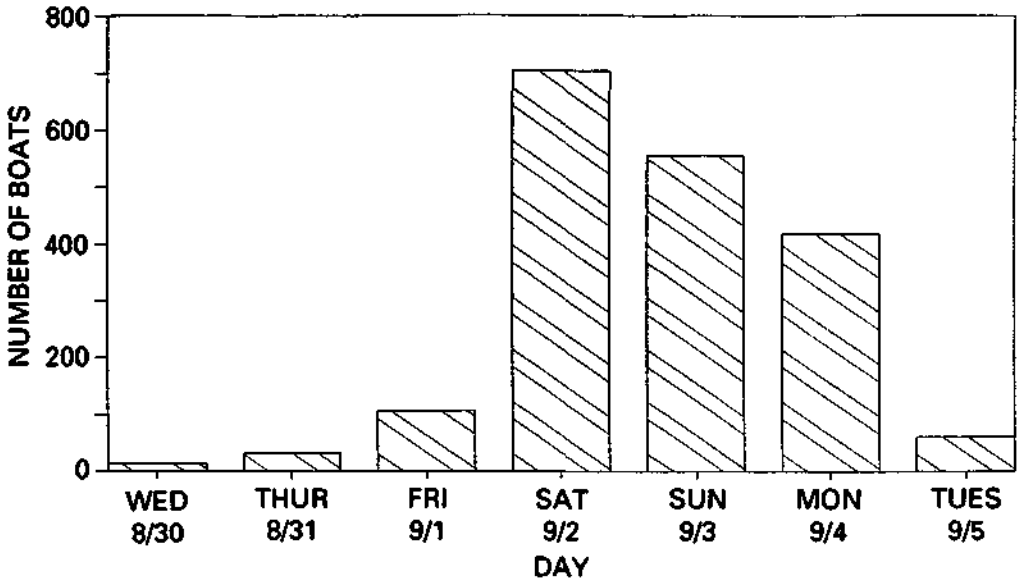

Figure 34. Frequency analysis of boat passages, August 30,1989, through September 5,1989, Red Wing site

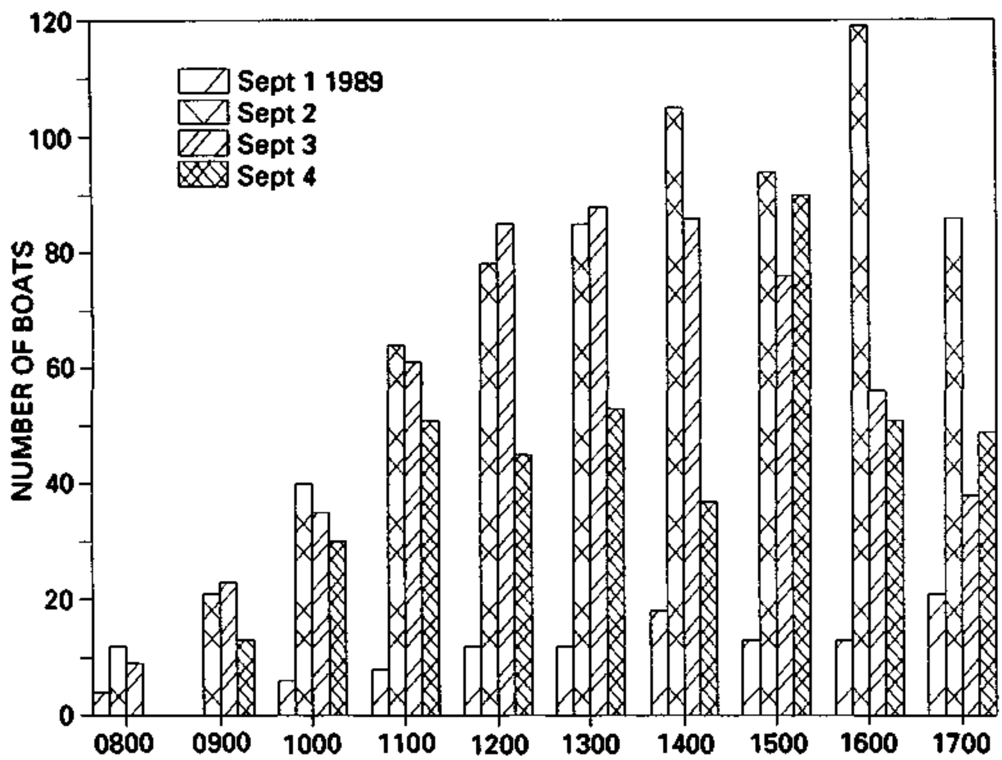

Figure 35. Frequency distribution of boat passages on an hourly basis, Red Wing site

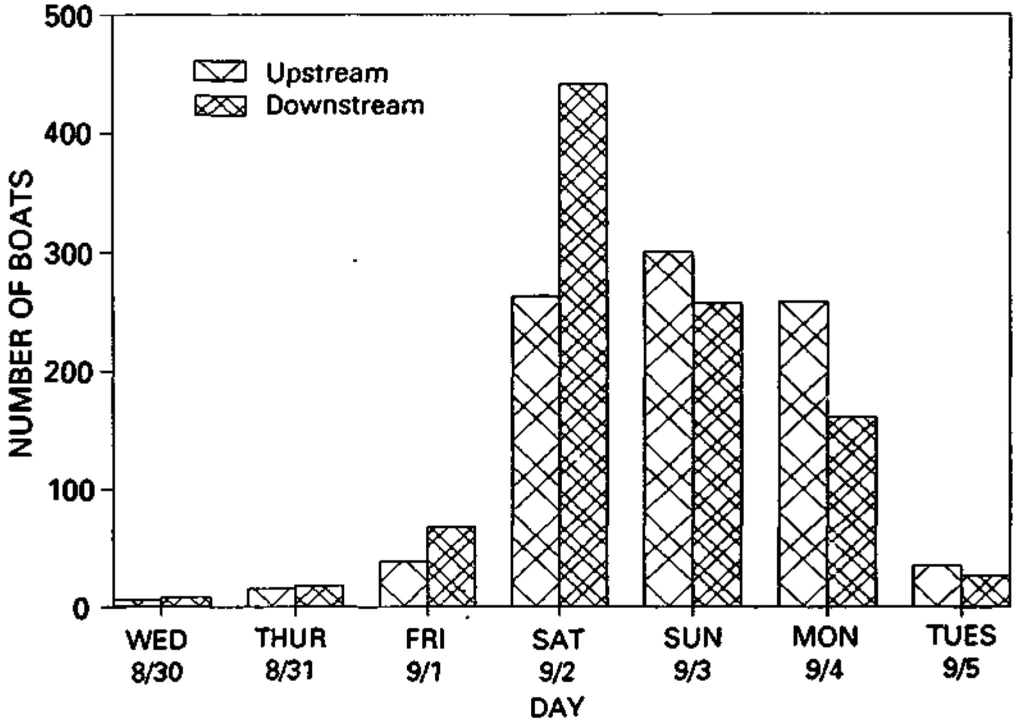

Figure 36. Frequency distribution of boat passages in the upstream and downstream directions, August 30,1989, through September 5,1989, Red Wing site

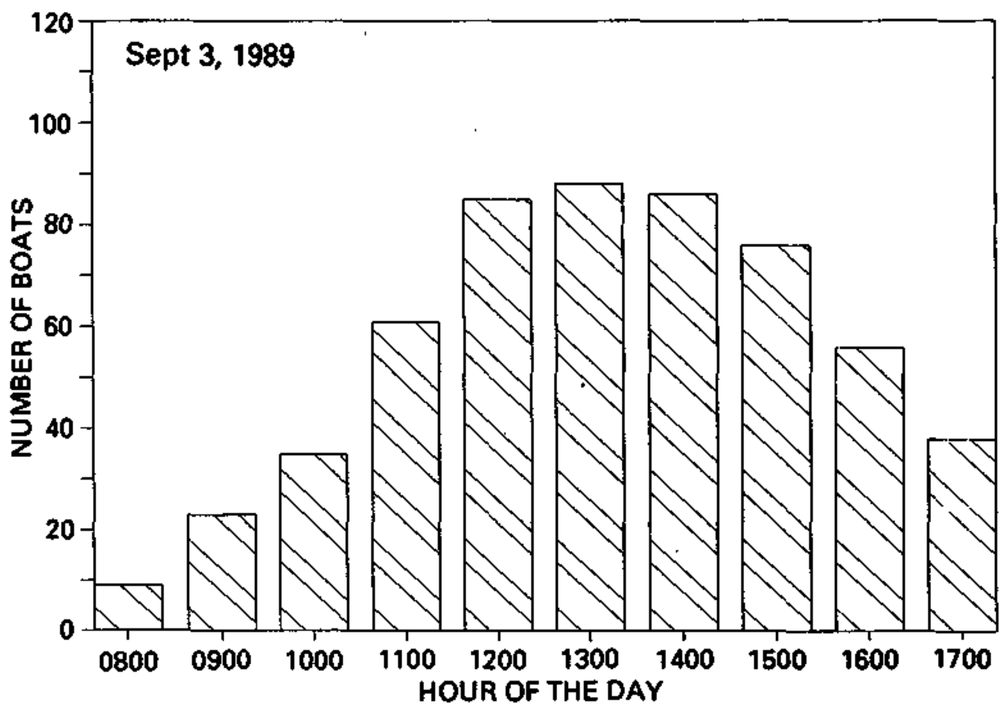

Figure 37. Frequency distribution of boat passages on an hourly basis for Sunday, September 3, 1989

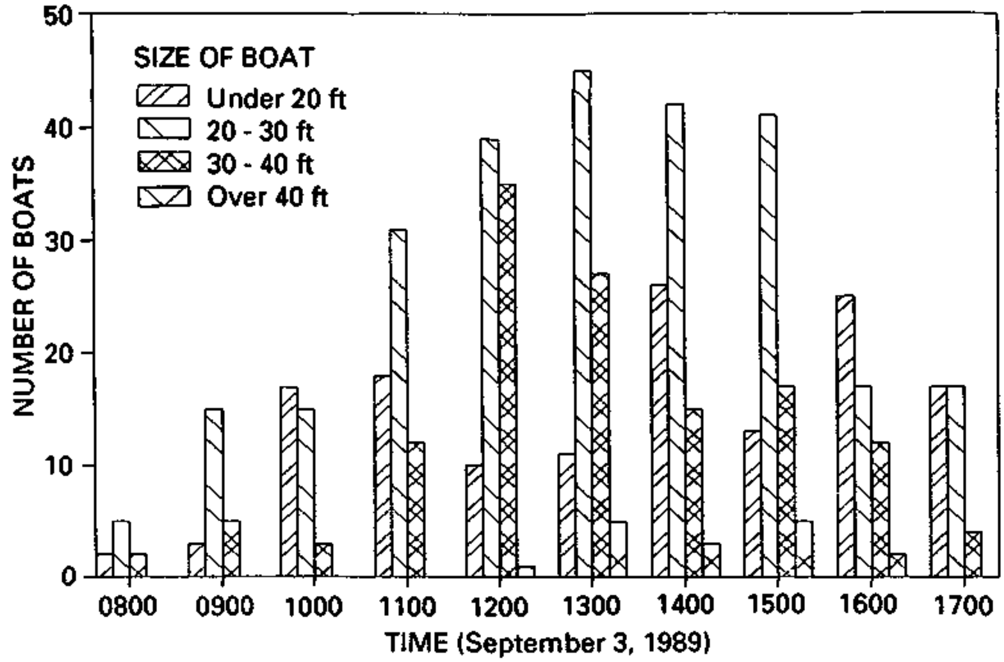

Figure 38. Frequency distribution of boat lengths on an hourly basis, Sunday, September 3,1989

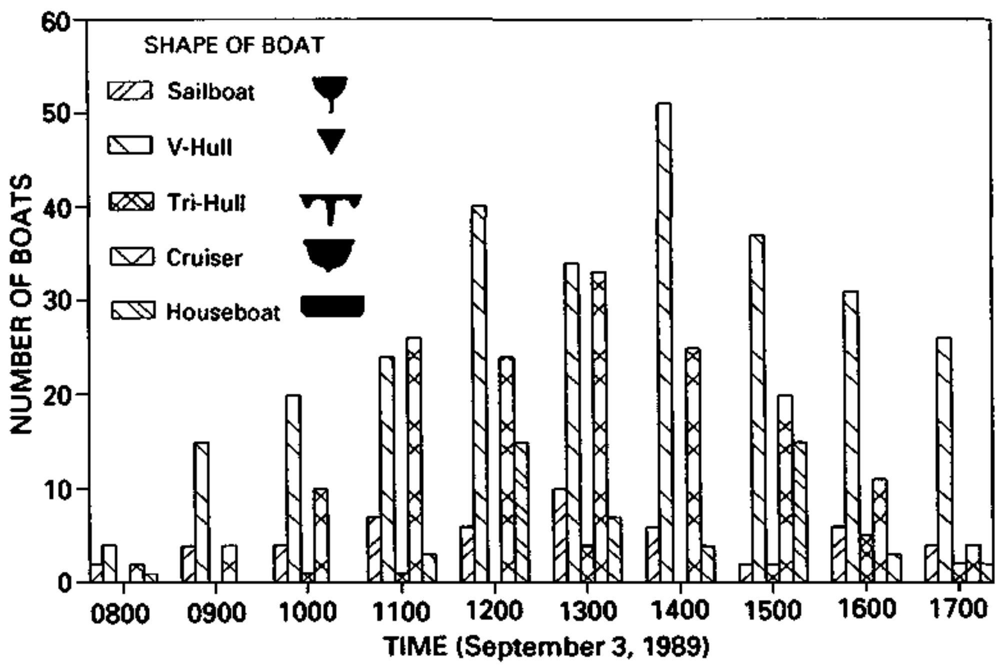

Figure 39. Frequency distribution of boat shapes on an hourly basis, Sunday, September 3, 1989

where not too many houseboats or sailboats are expected. Most of the pleasure craft passing the test site were either V-hull boats or cruisers as can be seen in figure 39.

Other analyses showed that, in general, a majority of the boats passed within the middle one-third of the river width and that the speeds ranged from medium to high. The speeds and distances of individual boats could not be measured because of the high frequency of boat movements, so plots could not be generated for these characteristics.

In summary, an extremely high amount of recreational boat activity was present at this site. On the busiest day of the weekend, more than 700 boats passed this section of the river, with a maximum of 120 boats passing the site in a single hour. The distributions of boat passages in the upstream and downstream directions were similar, although on Saturday, September 2, more boats moved in the downstream direction. Heavy rains occurred that day in the late afternoon, so some boats may have traveled back upstream after the monitoring stopped in the evening, and some may have been towed back from the road. The busiest time of the day was from about noon to 3:00 or 4:00 p.m. The majority of boats were from 20 to 30 feet in length and fell in the categories of V-hulls and cruisers.

It should be noted that in developing management alternatives for the control of recreational boat movements, consideration must be given to site-specific concerns. If the main concern is the stability of the river banks against repeated onslaughts of waves, then an evaluation must be made to determine the critical wave height that will render a specific streambank unstable for the specified bank material composition and bank slopes.

A determination can then be made regarding the river banks that will be unstable against a specified wave height, and corrective measures (such as reduction in boat speeds or establishment of boating lanes at sufficient distance from the shore) can be implemented to reduce the wave action on the shore. At the same time, river banks can also be stabilized with artificial bank stabilization works. Such analyses will be dependent upon the physical characteristics of the river reach and the expected frequency of boat movement.

Research on commercial traffic, conducted by the same authors, has shown that sediment particles and other organic matter are normally resuspended within the channel border areas, especially near the wave-wash zones, in sufficient magnitude to increase the turbidity of the water in this area. Even though no sediment data were collected for the present project, field observations indicated an increase in suspended sediment concentrations near the shores as a result of the breaking waves. Also, boat-induced waves cause breaker zones to move away from the shore. If this increase is of sufficient magnitude, management decisions are needed to decrease the effects of increased sediment concentrations on the organisms living close to the shore or the wave-wash zones.

In developing management alternatives to reduce possible impacts on the banks resulting from the movement of recreational boats, consideration could be given to limiting the frequency of movement from noon to early afternoon and also probably to maintaining the traffic lanes near the middle of the river. It is not possible to limit the size and length of boats, but obviously some control on boat speed can be instituted.

Wave Characteristics

Wave data were collected from two wave gages at the Red Wing site (figure 17), located 13 and 19 meters from the shoreline. The river is 275 meters wide at this location (figure 24), and the sampling was done at the right-hand side of the river looking downstream. During the early part of the data collection, such as on August 30-31 and September 1, it was possible to correlate the passage of an individual boat with a single wave train intersecting the wave gages. However, starting on September 1–4, it was impossible to relate any of the wave trains to a specific boat. Several examples of the variability of the wave structures with time are illustrated in figure 40.

Figure 40 was developed by selecting three five-minute intervals of data from September 1-3. It demonstrates wave characteristics generated by continuous, uncontrolled recreational boat movement Figures 40a and 40b are plots of waves generated by the passage of three boats, interspersed by wind-generated waves. The water surface profiles between the recreational boat wave trains represent the ambient wind-generated waves, which have smaller magnitudes. The boat-generated waves can be clearly identified, and their distributions are similar to those shown for controlled runs in figures 26 and 27.

Figures 40c and 40d show similar variations, except that at around 3:00 p.m. on September 2, boat passages became more frequent and waves were generated almost continuously. The effects of continuous boat traffic become more apparent in figures 40e and 40f, which show that at about 11:00 a.m. on September 3, boat passages at the site were more or

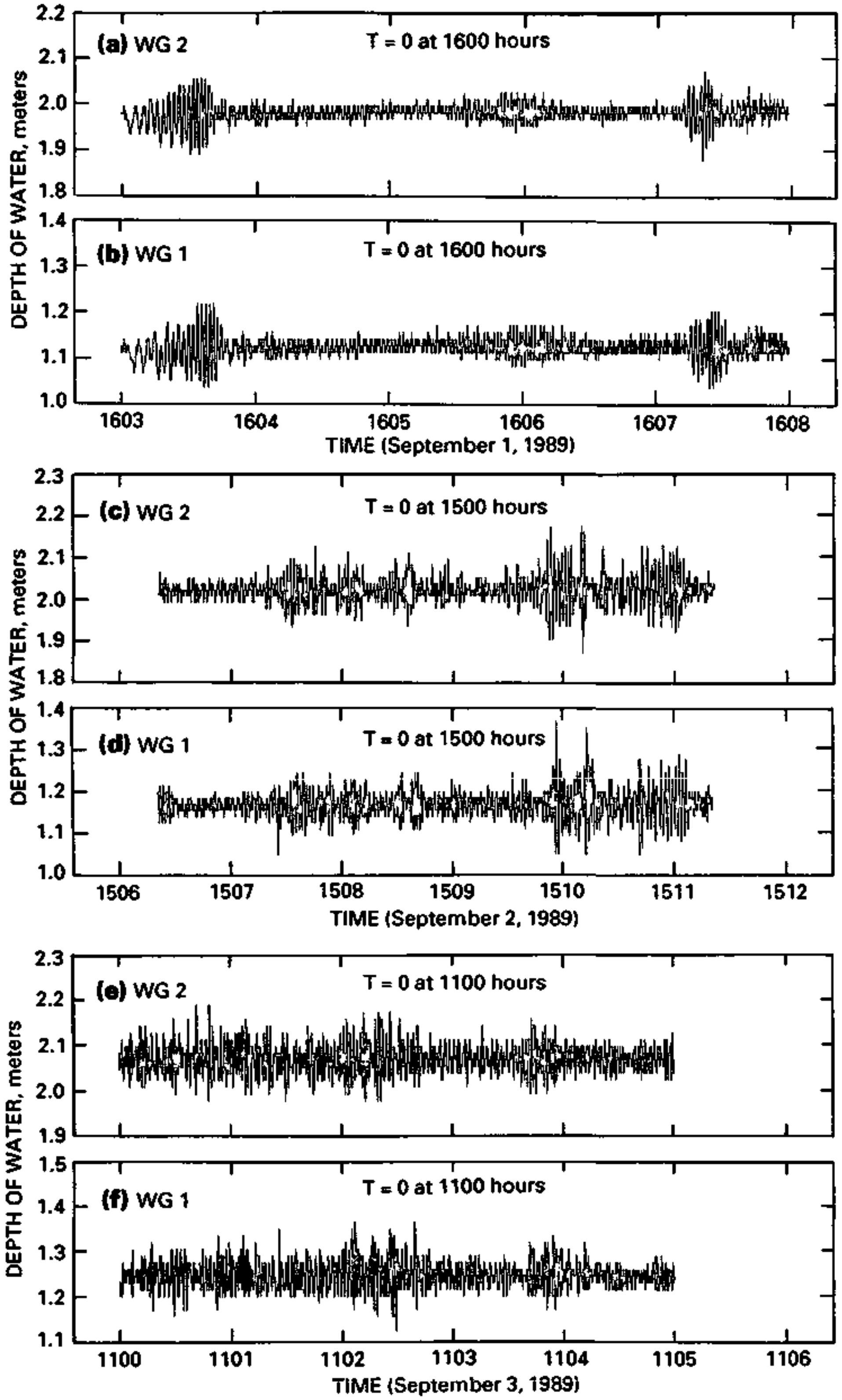

Figure 40. Typical waves generated by uncontrolled boating events

less continuous and the waves reaching the gages were also continuous. The pattern of these waves appears to be similar to that of waves generated by wind except that they are much larger.

An examination of figure 40 indicates that when boat traffic was heavy, no separation of individual wave trains was possible, and that the waves were crossing the wave gages in a manner similar to that of random waves generated by sustained wind activity on an open body of water. Thus it was decided to analyze these data as random waves.

Refraction and reflection of waves occur at the shoreline, and some of the waves measured at the wave gages would have been amplified or decreased as a result of these processes. Readers should be warned against such resonant behavior of the waves measured during this experiment. However, the data that were collected could not be used to determine the magnitudes of these effects. Moreover, the basic objectives of this project were to qualify the waves at these sites and show how to analyze the data to obtain a reasonable resolution. The analyses presented here were considered the best alternative for this specific case.

The data were analyzed for two time intervals: five minutes and one hour. It should be noted that the analyses of the traffic characteristics were done on an hourly basis (figures 35 through 39). All the wave data for each wave gage were partitioned on an hourly basis. The hourly wave data were then assumed to be individual events lasting for an hour only, even though the waves generally passed the gages throughout the whole day, irrespective of the hour of the day. These waves are only a function of the frequency of boat passages.

The wave data were analyzed to determine the significant wave heights for each individual hour. Significant wave height is a statistical parameter that is used extensively in the evaluation of wind-wave characteristics. Figures 41a and 41b show the significant wave heights measured at wave gage 2 (19 m from the shore) and wave gage 1 (13 m from the shore).

An examination of these plots shows that the variations in significant wave heights for September 1-4 are similar to the variations observed in the frequency of boat movement at the test site for the same four-day period (figures 34 and 35). At both wave gages, hourly significant wave heights varied from a minimum of about 0.1 meter to a maximum of 0.45 to 0.48 meter. In general, as far as the hourly significant wave heights are concerned, not much variation existed between these two gages even though they were installed 6 meters apart.

Figure 42 shows the hourly distribution of maximum wave heights at both gages at the Red Wing site for uncontrolled boating events. The distribution of maximum wave heights is similar to the distribution of significant wave heights, and some correlation exists between this distribution and the distribution of traffic at this location observed during the same period (figures 34 and 35).

Wave gage 2, located 19 meters from the shore, measured maximum wave heights of more than 0.5 meter on several occasions. Wave gage 1, located 13 meters from the shore, measured maximum wave heights of 0.5 meter or more only once. Thus it appears that the amplitude of the waves decreased within a distance of about 6 meters as the waves moved from wave gage 2 to wave gage 1. However, field observations did not indicate a significant reduction in amplitudes from wave gage 1 to the shoreline.

Figures 41 and 42 imply that the waves generated by frequent movement of recreational craft were almost continuous. Such waves will break near the shore at a more or less continuous rate. Therefore the analysis of the waves as random phenomena is quite appropriate and affords an opportunity to determine wave parameters from a random and more or less continuous event.

To determine the relative importance and significance of waves generated by recreational boats compared to those generated by wind, techniques suggested by Bhowmik (1976, 1978) were used to compute the equivalent wind velocity that would generate waves the size of those observed in the field (figures 41 and 42). The following equation (after Bhowmik, 1976, 1978) was used for this exercise:

$$g\ Hs/Ve^2 = (3.0) \times (10^{-4})\ (g\ Fe/Ve^2)^{0.435} \qquad (10)$$

where g is the acceleration due to gravity in m/s^2, Hs is the significant wave height in meters, Ve is the effective wind velocity in m/s, and Fe is the effective fetch length in meters. For the computations at the Red Wing site, the wind was assumed to blow at a constant speed straight toward the wave gage from across the river. With these assumptions, the numerical value of effective fetch became the same as the width of the river, and effective wind velocity and actual wind velocity also became identical.

With these assumptions, it was found that sustained winds of 58 and 45 miles per hour would be needed to generate significant waves of 0.4 and 0.3 meter, respectively, at the wave gages at this site. This simple analysis indicates that recreational boats can probably generate waves of sufficient magnitude

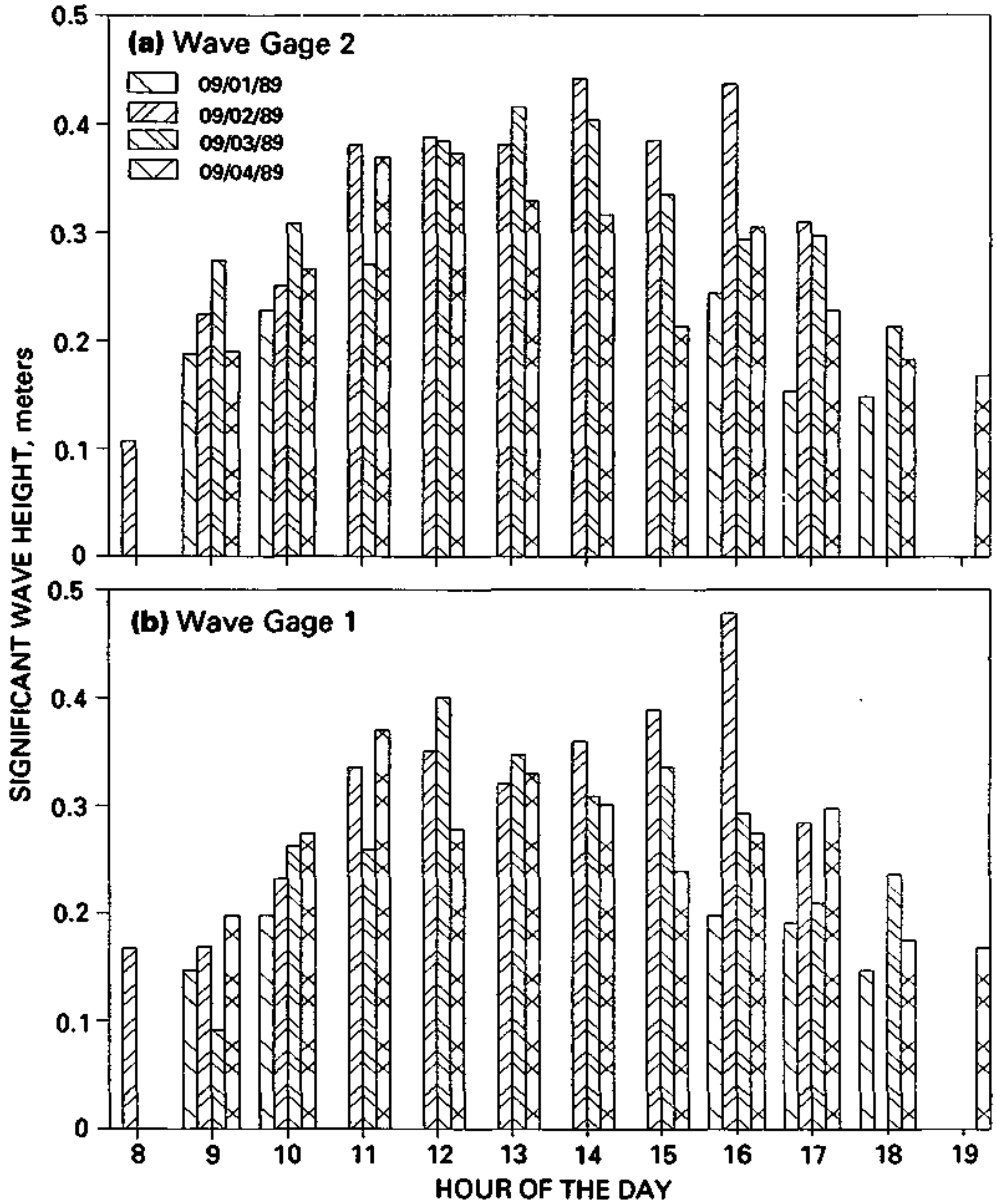

Figure 41. Distribution of significant wave heights on an hourly basis for uncontrolled boating events

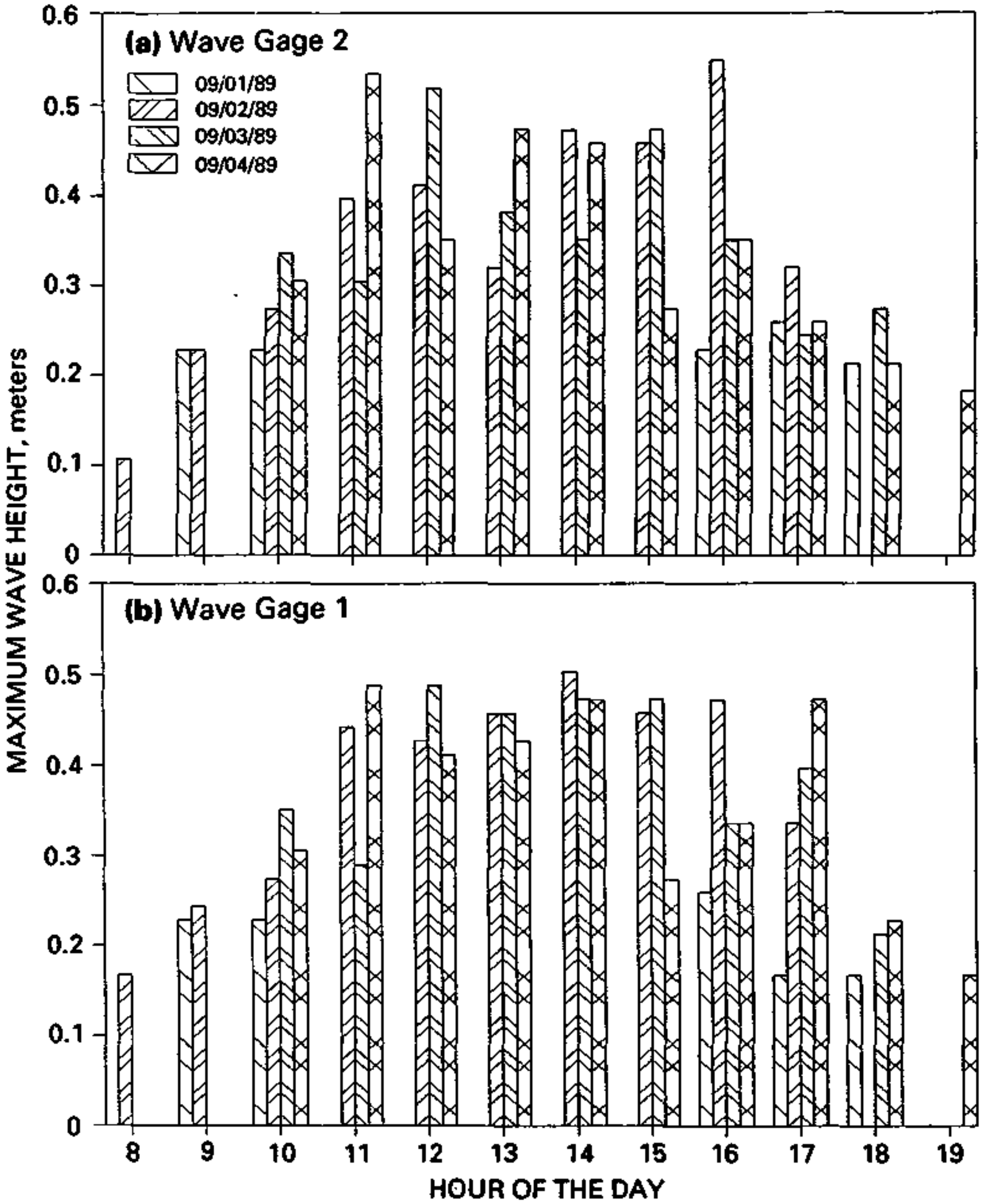

Figure 42. Distribution of maximum wave heights on an hourly basis for uncontrolled boating events

that an evaluation should be made to determine if they might initiate bank erosion or shoreline instability.

Another measure of the vulnerability of the shoreline to erosion is the amount of energy contained within each wave train that needs to be dissipated or reflected from the shoreline. For this particular analysis, all the wave data were partitioned into five-minute intervals. In each interval, each individual wave was isolated and its harmonics were found by Fourier analysis. Then equation 5 was used to compute its energy content. The total wave energy in a five-minute interval was the sum of the energy content of all the individual waves. These calculated values of wave energies were used to develop frequency histograms of wave energies at both wave gages at the Red Wing site.

Figure 43 shows the distribution of wave energies at the Red Wing site. This figure shows that wave gage 2 endured much higher magnitudes of "wave energy" impacts than wave gage 1. Obviously wave gage 2 experienced waves of much higher amplitudes than those observed at the closer wave gage 1. Wave energy of 150 newtons/meter^2 occurred approximately 120 times at wave gage 1, while wave energy of about 425 newtons/meter^2 occurred about 70 times at wave gage 2. It can be postulated that wave energies near the shoreline would be smaller in magnitude than those shown in figure 43 for wave gage 1, but the frequency of occurrence would be higher.

The analyses presented in the last few paragraphs are important with regard to the stability of shorelines against wave actions. Frequent movement of recreational traffic would certainly maintain a steady train of waves that would impinge on the shorelines, and the significant and maximum wave heights might be greater than those shown in figures 41 and 42. With these increases in wave heights, the frequencies of waves impinging on the shorelines would also increase, which in turn would impact the shorelines. Thus in the case of noncohesive banks or banks having no vegetation, the probability of erosion would certainly be increased if the frequency of boat passages increased.

The data collected and the analyses performed so far do not lend themselves to an evaluation of wave height distribution due to changes in boat passage frequency. However, an analysis was performed to obtain an estimate of the wave heights with a change in the frequencies of boat passages.

Data on significant wave heights shown in figure 41 were used to develop figure 44, in which changes in the hourly significant wave heights are plotted against the number of boats passing the site each hour. An increase in significant wave heights is associated with an increase in the number of boats passing the test site, as would normally be expected from such random events.

As noted before, wave gage 2 measured higher significant wave heights than wave gage 1, which is farther from the boat tracks. One interesting point should be mentioned here. With an increase in the frequency of boat passages, not only did the significant wave heights at both gages increase, but the differences in the magnitudes of the wave heights measured at the two gages diminished. Thus it appears that above a certain frequency of boat passages, the magnitudes of the significant wave heights measured at these two wave gages installed 6 meters apart would be identical.

It is therefore reasonable to postulate that when more than about 190 boats pass a site per hour, the waves that will impinge on the shoreline may be almost identical to those observed at a distance of 13 to 19 meters from the shore. Here again it should be pointed out that the magnitudes of the wave heights measured at a certain distance from the shore could be modified at the water and shoreline interface by the effects of water depths, slope of the shoreline, and roughness of the bottom materials.

Two linear equations were developed on the basis of the plots shown in figure 44. These are:

$$Hs_1 = [(2.1)\,N + 166.8] \times 10^{-3} \qquad (11)$$

$$Hs_2 = [(1.9)\,N + 205.6] \times 10^{-3} \qquad (12)$$

where Hs_1 and Hs_2 are the significant wave heights in meters measured at wave gage 1 and wave gage 2, respectively, and N is the number of boats passing per hour at this site. These two equations must not be extrapolated beyond the limits of the data shown in figure 44. Also, it should be noted that these two equations were developed from the data collected at the Red Wing site. At other locations, water depths, composition of bed and bank materials, slopes of the banks, and other physical factors could alter the waves generated by wind and recreational or commercial traffic movements.

The analyses that have been presented for uncontrolled boating events indicate that for an area with extremely heavy recreational boat usage such as the Red Wing area in Minnesota, the heights of waves produced by the boats can be as much as 0.5 meter or more. A comparison of figures 41 and 42 for uncontrolled boating events with figures 30 and 31 for controlled runs shows that the distributions of wave

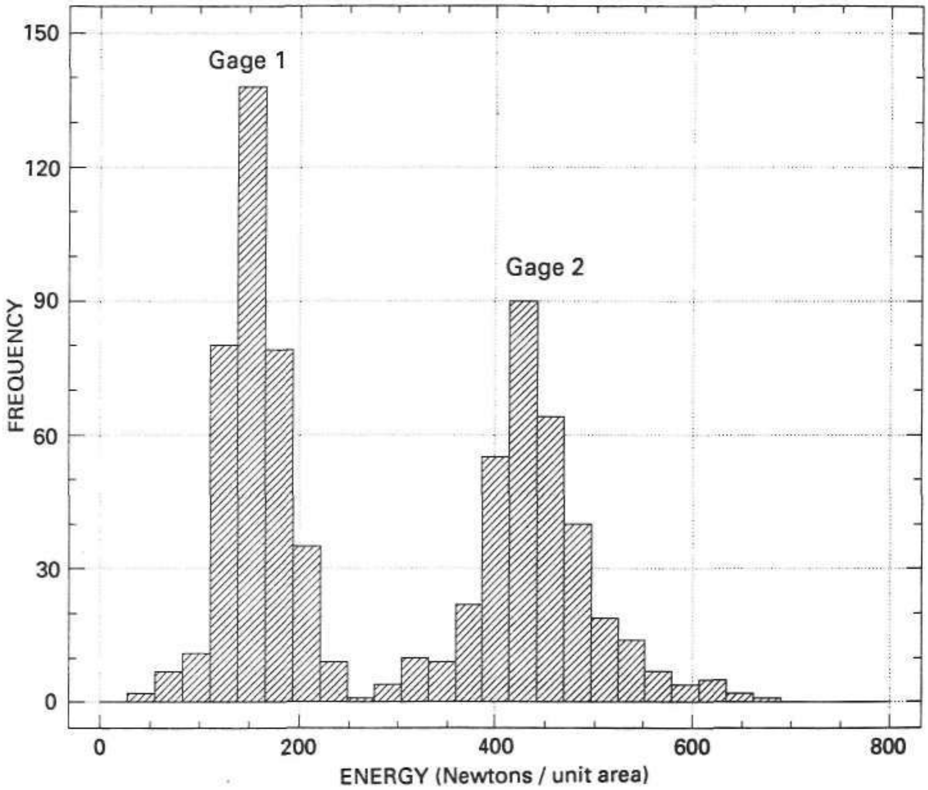

Figure 43. Frequency distribution of wave energies for 5-minute intervals. Red Wing site

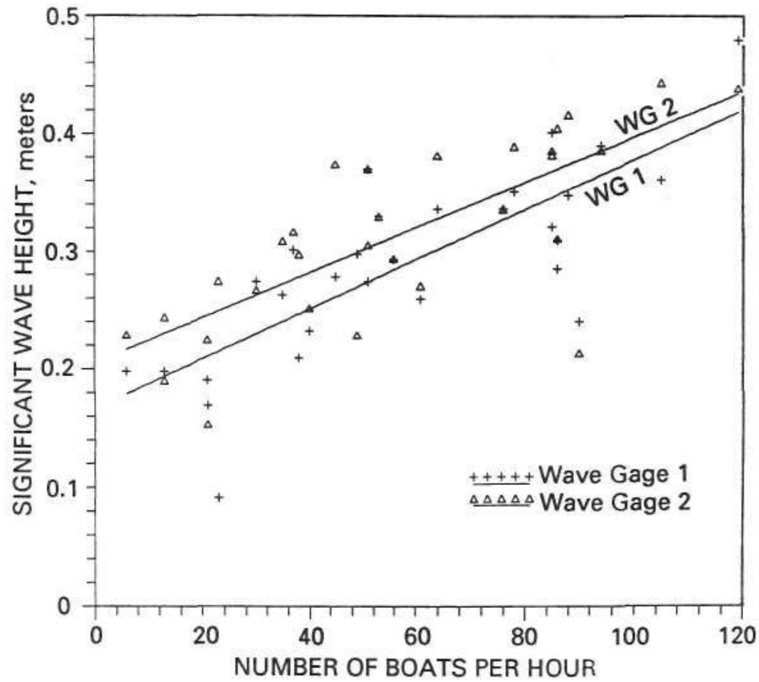

Figure 44. Relationship between significant wave heights and number of boats passing every hour

heights for both these cases are in the same general range. However, higher waves are caused more frequently by uncontrolled boating events than by controlled runs.

This is quite reasonable when one considers that at times from 5 to 12 boats passed the test site simultaneously, and consequently waves measured at the gages are the end products of all the waves produced by individual boats. Field observations have also shown that such a heavy recreational use of the river can produce waves with heights of about 0.4 to 0.5 meter for a prolonged period of time.

DISCUSSION

The objectives of this research project were to determine the characteristics of waves generated by recreational craft, to develop a regression-type relationship for the estimation of wave heights, and to gather wave data during a busy weekend when the recreational use of the river was very high. Those data were then analyzed to determine the variability of the waves.

The project was not designed to measure the resuspension of sediments by the waves or the way the waves might impact the stability or instability of the shorelines or the vegetation in the shore zones. Additional research is needed to determine the resuspension of sediments and the stability of banks against boat- generated waves.

The data collected for this project and the analyses presented so far indicate that heavy recreational use of a river will generate substantial wave activity that could be detrimental to the shore vegetation and the stability of the banks. However, an analysis of the stability of the banks must take into account not only the waves generated by recreational craft, wind, and other traffic, but also such factors as the hydraulics of flow in the river and the geotechnical characteristics of the bank materials and the bank slope.

A comprehensive analysis of the wave data from the controlled runs was used to develop the regression relationship given in equation 9. This equation is recommended for use in determining the maximum wave heights generated by a boat moving a certain distance from the shore.

For example, suppose someone wants to determine the maximum wave height for the following parameters:

Speed, v = 8.94 m/s (20 mph)
Distance, x = 45.7 m (150 ft)
Length, L = 5.5 m (18 ft)
Draft, d = 0.76 m (2.5 ft)

Using equation 9, the maximum wave height for the above parameters becomes

H_m = 0.16 meter

A note of caution should be included in this discussion for readers who plan to use equations 11 and/or 12 and figure 44 to estimate significant wave heights from a knowledge of the hourly frequency of boat passages. These two equations and figure 44 were developed from data on the movement of uncontrolled boats.

Thus at some times during the day, boats passed the site at somewhat regular intervals, generating waves with similar frequencies, but at other times in the day, boat passages were irregular, generating waves with irregular intervals and amplitudes. In a one-hour period, 20 to 30 boats passing within a very short time could produce waves as high as those produced by 60 to 80 boats passing at regular intervals during the entire hour. Readers need to understand these limitations in the development of this figure and the associated equations.

Even though a relationship was not developed for estimating the stability of a bank or bank materials against wave action, equation 13, developed by Bhowmik (1976), can be used for estimating the stable median diameter of a lakeshore against an estimated wave action.

$$W_{50} = (6.2\, S_s\, H_s^{3})/[(S_s-1)^3\, (\mathrm{Cos}\alpha - \mathrm{Sin}\alpha)^3] \qquad (13)$$

where W_{50} is the weight of the median diameter of the bank materials in kilograms, S_B is the specific gravity of the bank materials, H_B is the significant wave height in meters, and a is the slope of the bank in degrees and is less than 45 degrees. In equation 13, significant wave height H_B can be replaced with maximum wave height H_m for computational purposes. Thus with known values of H_m or H_B, S_B, and the bank slope, the stable median diameter of the bank materials can be estimated.

However, it must again be cautioned here that equation 13 was developed on the assumption that banks will be stabilized with riprap materials completely *noncohesive* in nature. Cohesive materials are much more stable than noncohesive materials.

The stabilization of streambanks with riprap against an anticipated wave action will require the use of appropriate filter blankets to prevent the washout of the fine bank materials (Bhowmik, 1976). Normally crushed stones about 1.3 to 1.9 millimeters in diameter and about 0.02 to 0.025 meter thick are used as filter blankets.

Recommendations for Future Studies

As a result of this investigation and the comments received from 17 reviewers of the draft copy of this report, the following recommendations are made for further study of waves generated by recreational boats and their potential effects:

1) Effects of recreational boat waves on the stability of streambanks composed of a variety of bank materials and slopes should be investigated.

2) The resuspension characteristics of sediments due to boat waves should be determined, especially in streams and rivers where the bed materials are composed primarily of silt, clay, and fine sediments. Analyses should also be performed to determine the effects of boat waves on turbidity increases due to the resuspension of sediments and organic matter.

3) Effects of recreational boats on sediment resuspension and bank stability within the shallow backwater areas away from the main channel should be investigated.

4) Further study should be conducted to determine the effects of various boat characteristics (for example, different drafts) on the wave-making capability of boats. This should include the development of functional relationships between speed and effective length, and between speed and effective draft, including the submerged area and volume of the boat. These types of relationships are needed for each category of boats. Such information would be useful in the development of regulations for recreational boating activity.

5) The effects of sloping banks, characteristics of the bottom materials, run-up, and refraction, reflection, and resonance effects of waves should be investigated in detail to obtain a reasonable quantification of the effects of these factors on the overall characteristics of the waves generated by recreational boats within the UMRS.

SUMMARY

This report presents the results of a research project undertaken to determine the characteristics of waves generated by recreational craft within the UMRS. The specific objective of the project was to collect wave data from both controlled and uncontrolled events, analyze the data, and propose a functional relationship for determining wave heights generated by recreational craft.

To meet the goals of the project, 246 controlled runs were made with 12 different boats at two sites, one on the Illinois River and the other on the Mississippi River. Data from the controlled runs indicated that recreational boats can generate from 4 to 40 waves per event, with a mean of about 10 to 20 waves. These waves can last from 6 to 40 seconds or more. Average wave heights for these controlled events varied from 0.01 to 0.25 meter, with a median of about 0.06 to 0.12 meter. The maximum wave height was as much as 0.6 meter.

The wave data from the controlled runs were used to develop a regression equation for estimating maximum wave heights on the basis of the speed, draft, and length of the boats, and their distance from the measuring point. This relationship is now recommended for use in determining wave heights generated by recreational boats. No other relationship exists for determining wave heights generated by recreational boats except the relationship proposed by the first author in the mid-1970s.

However, this equation should not be used beyond the limits of the data sets for which it was developed. Also, this equation should be used with caution for other sites because the differences in water width and depth, bank slope and permeability, and roughness of bed materials may generate somewhat different wave activity.

The data from uncontrolled boating events indicated that as many as 704 boats passed a highly used area of the UMRS in a single day on a busy weekend. Up to 120 boats passed the site in a single hour. Sustained movement of recreational boats can generate essentially continuous waves, giving the appearance of random waves at or near the shoreline.

During a busy weekend, the majority of the recreational craft passed the study site from around

2:00 to 4:00 p.m. Boats in the range of 6.1 to 9.2 meters (20 to 30 feet) in length were most common. During the heavy boating activity at the Red Wing site on Saturday, September 2, 1989, the maximum wave height measured was 0.52 meter, and the average for the whole day was 0.065 meter.

Analyses were also performed by partitioning the wave heights on an hourly basis. These analyses indicated that significant wave height can reach a magnitude of 0.4 meter or higher, and maximum wave height can reach 0.5 meter or higher. Calculations were performed to show that for waves of 0.4 meter in height to develop at the Red Wing site from wind alone, the wind would have to be blowing at a speed of about 26 meters per second (58 mph) across the measuring point. Wave energies were computed by partitioning the waves into five-minute intervals. These analyses showed that the shorelines are subjected to wave activity of fairly high intensity.

To obtain an estimate of the wave heights produced by more frequent passage of boats, an analysis was performed to relate significant wave heights to the hourly frequency of boat passages. Relationships developed from this analysis can be used to approximate the heights of waves generated by the frequent passage of boats. However, this equation should not be used beyond the limits of the data sets for which it was developed.

No analyses were performed to determine the bank erosion potential or sediment resuspension characteristics of the waves generated by recreational boats. However, existing mathematical formulations can be used to analyze the stability of banks composed of noncohesive bank materials. Additional research should be initiated to determine the effects of recreational boats on the stability of cohesive and noncohesive banks, and the way in which wave activity resuspends bed materials.

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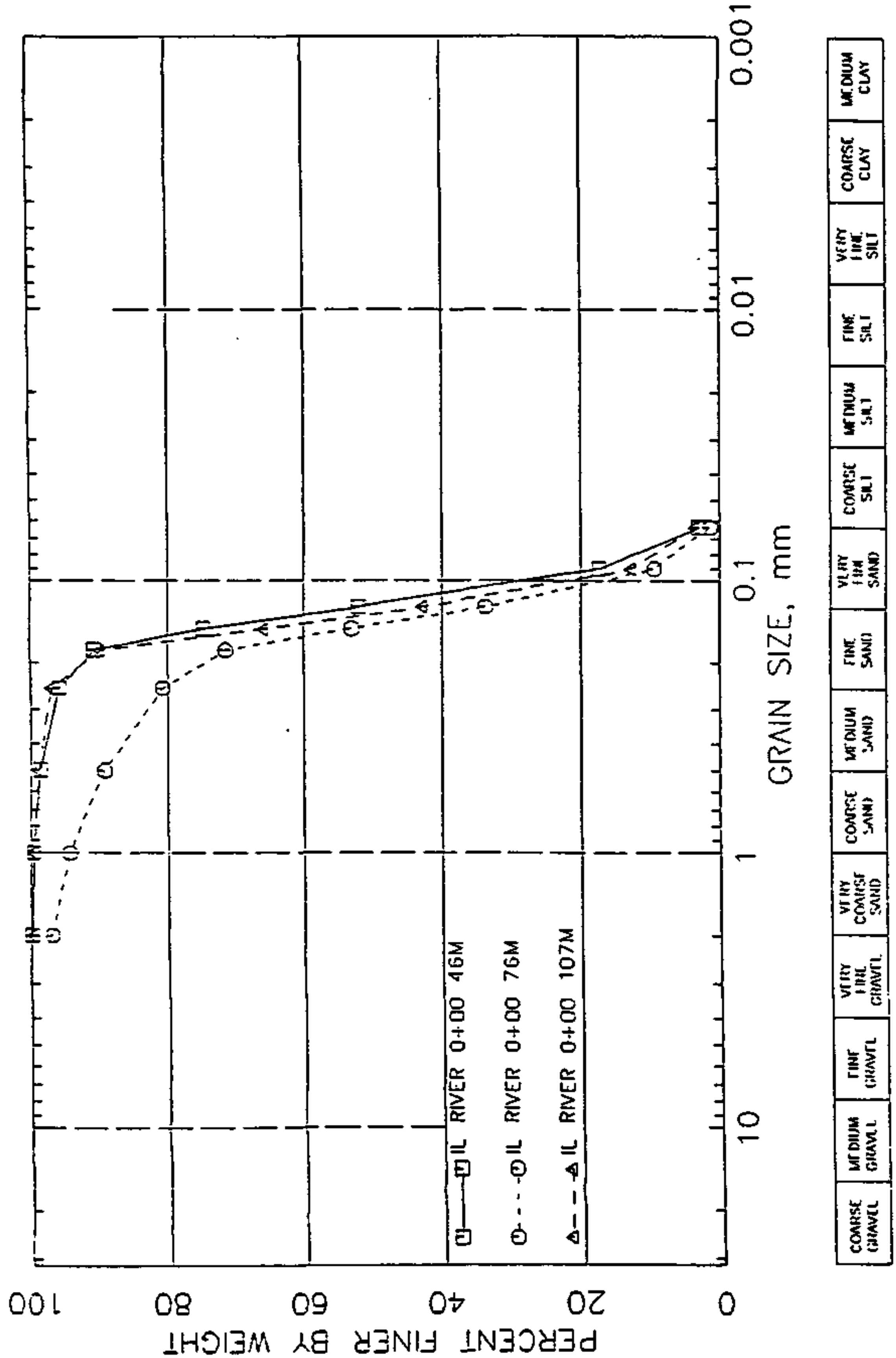
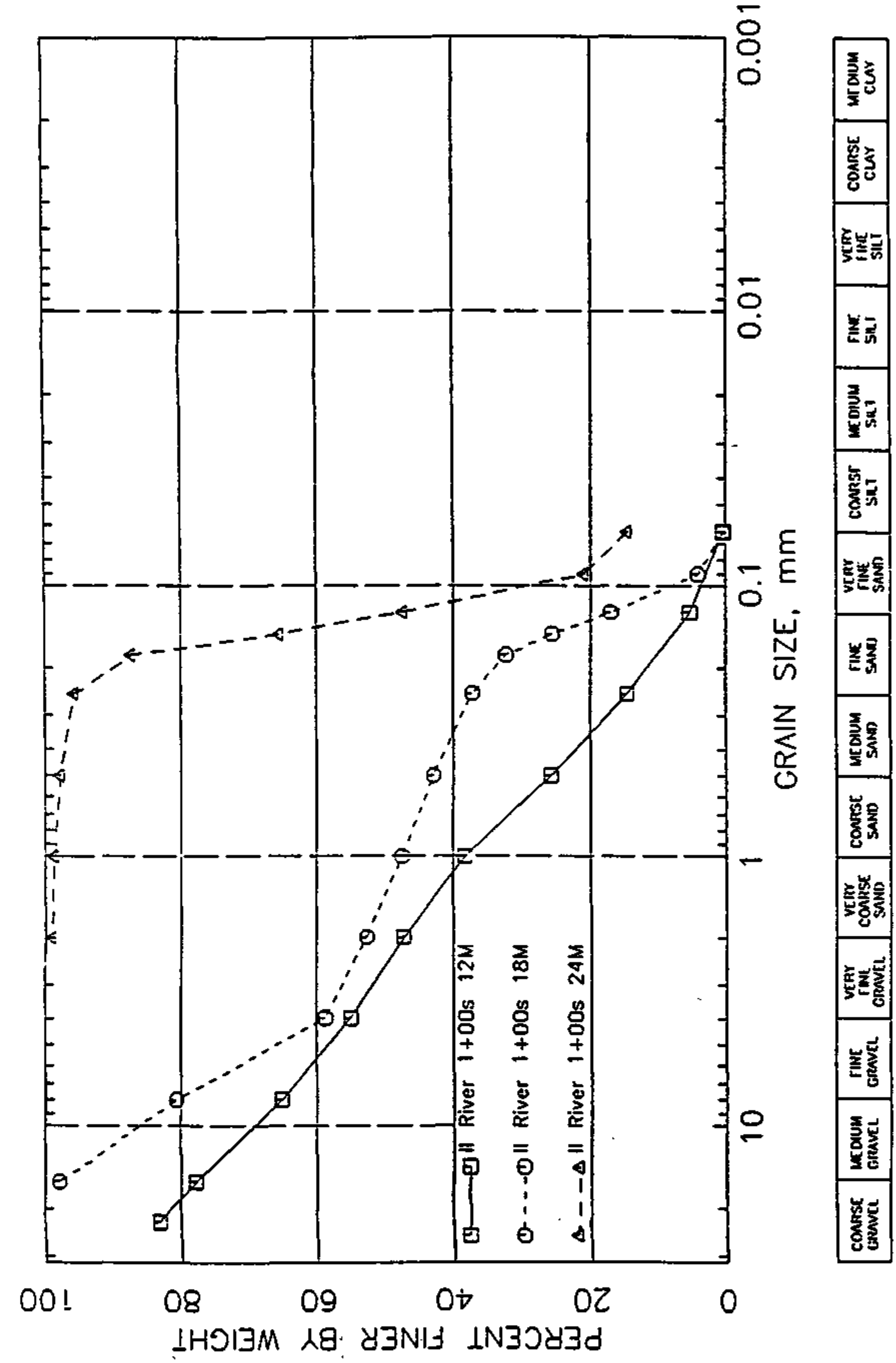
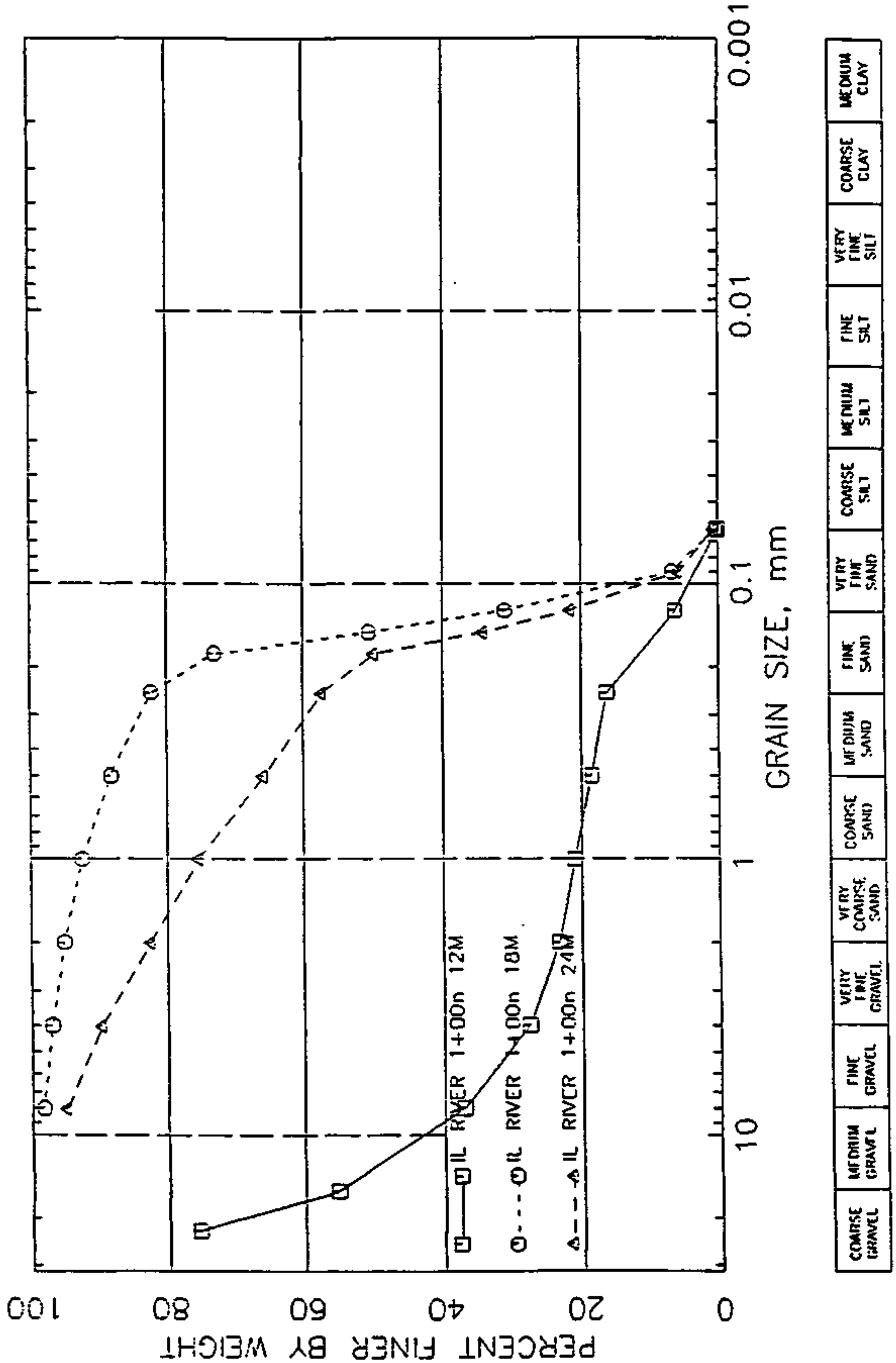
Appendix A-1. Particle Size Distributions of the Bed Material Samples at the Havana Site


PERCENT FINER BY WEIGHT
0
20
40
60
80
100
GRAIN SIZE, mm
10
1
0.1
0.01
0.001
IL RIVER 0+00 46M
IL RIVER 0+00 76M
IL RIVER 0+00 107M
COARSE GRAVEL
MEDIUM GRAVEL
FINE GRAVEL
VERY FINE GRAVEL
VERY COARSE SAND
COARSE SAND
MEDIUM SAND
FINE SAND
VERY FINE SAND
COARSE SILT
MEDIUM SILT
FINE SILT
VERY FINE SILT
COARSE CLAY
MEDIUM CLAY


PERCENT FINER BY WEIGHT
100
80
60
40
20
0
GRAIN SIZE, mm
10
1
0.1
0.01
0.001
II River 1+00s 12M
II River 1+00s 18M
II River 1+00s 24M
COARSE GRAVEL
MEDIUM GRAVEL
FINE GRAVEL
VERY FINE GRAVEL
VERY COARSE SAND
COARSE SAND
MEDIUM SAND
FINE SAND
VERY FINE SAND
COARSE SILT
MEDIUM SILT
FINE SILT
VERY FINE SILT
COARSE CLAY
MEDIUM CLAY



PERCENT FINER BY WEIGHT
0
20
40
60
80
100
GRAIN SIZE, mm
10
1
0.1
0.01
0.001
IL RIVER 1+00n 12M
IL RIVER 1+00n 18M
IL RIVER 1+00n 24M
COARSE GRAVEL
MEDIUM GRAVEL
FINE GRAVEL
VERY FINE GRAVEL
VERY COARSE SAND
COARSE SAND
MEDIUM SAND
FINE SAND
VERY FINE SAND
COARSE SILT
MEDIUM SILT
FINE SILT
VERY FINE SILT
COARSE CLAY
MEDIUM CLAY

Appendix A-2. Characteristics of the Bed Material Samples at the Havana Site

Sample location	d_{50}	d_{85}	σ	*U*
30 meters north, 24.4 meters	2.3	22.4	9.3	3.1
30 meters south, 18.3 meters	1.2	10.5	9.4	4.1
30 meters south, 24.4 meters	.13	.18	1.6	2
Survey site, 45.8 meters	.13	.17	1.5	1.4
Survey site, 76.3 meters	.14	.40	2.1	1.7
Survey site, 106.8 meters	.16	.23	1.6	1.5
30 meters north, 12.2 meters	15.0	22.4	30.7	100.0
30 meters north, 18.3 meters	.15	.35	1.9	1.9
30 meters north, 24.4 meters	.18	3.0	9.2	3.3

Remarks:

σ is the standard deviation and it is calculated as

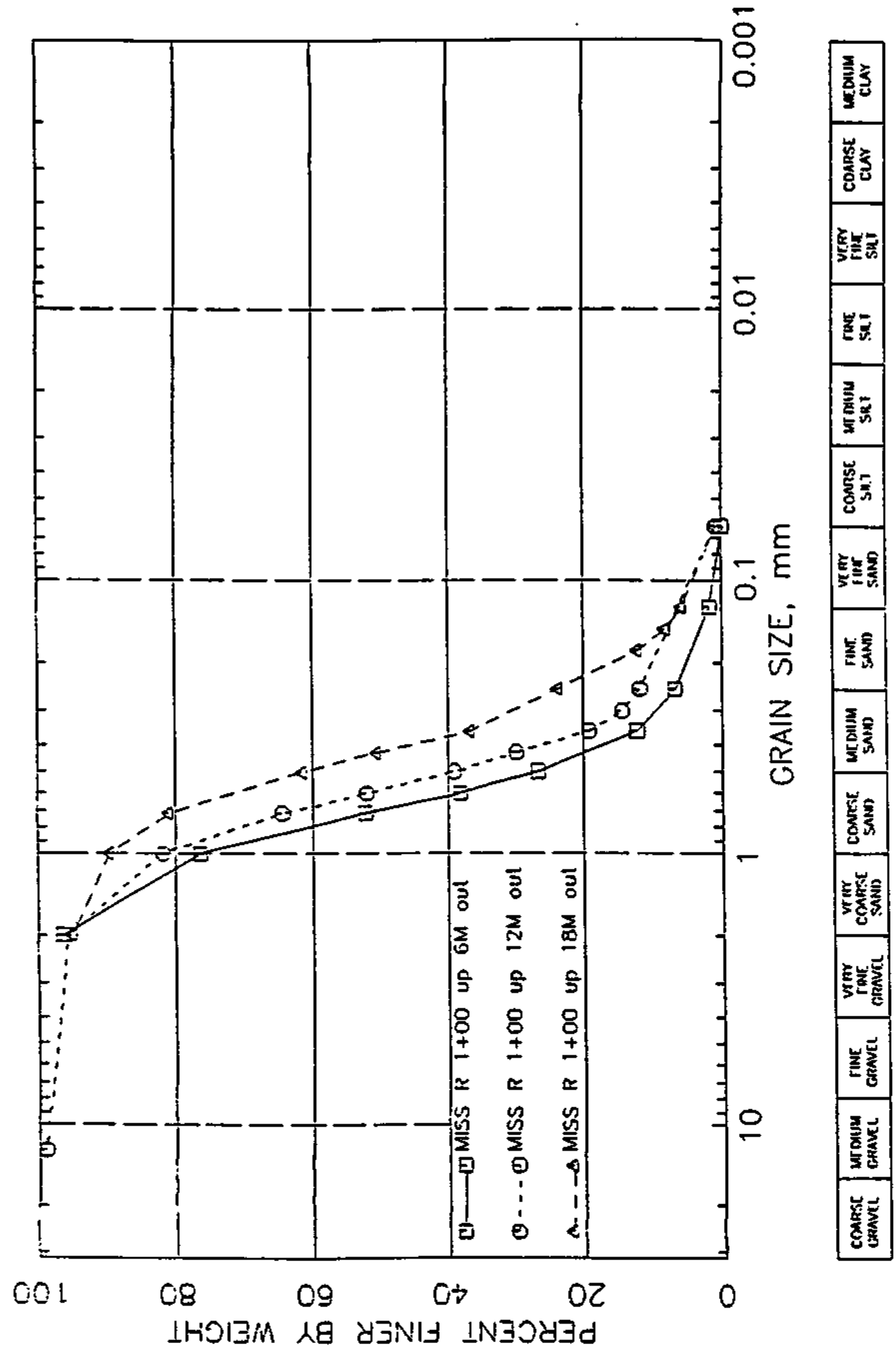
$$\sigma = \frac{1}{2} \left[\frac{d_{84.1}}{d_{50}} + \frac{d_{50}}{d_{15.9}} \right]$$

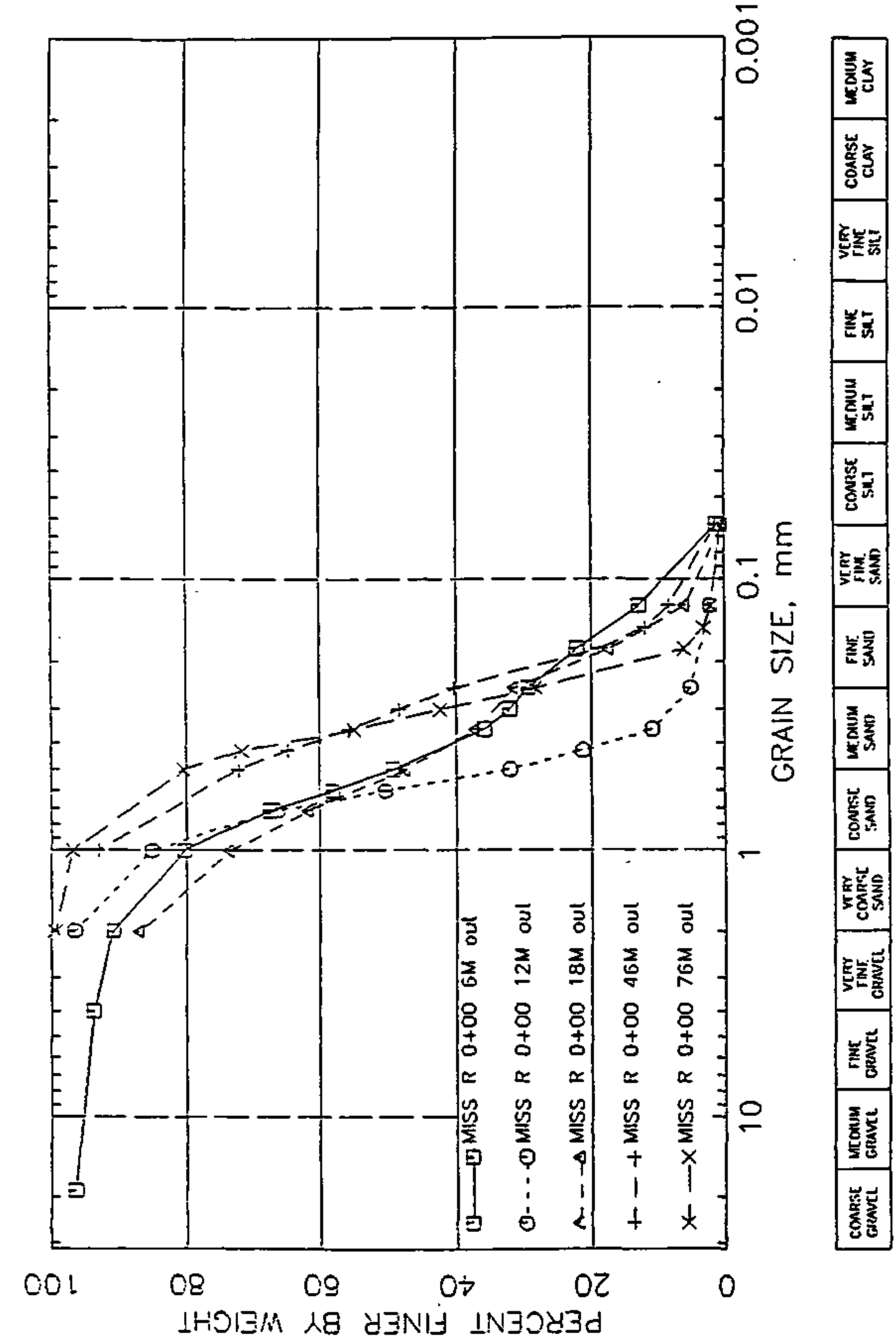
U is the uniformity coefficient and it is calculated as

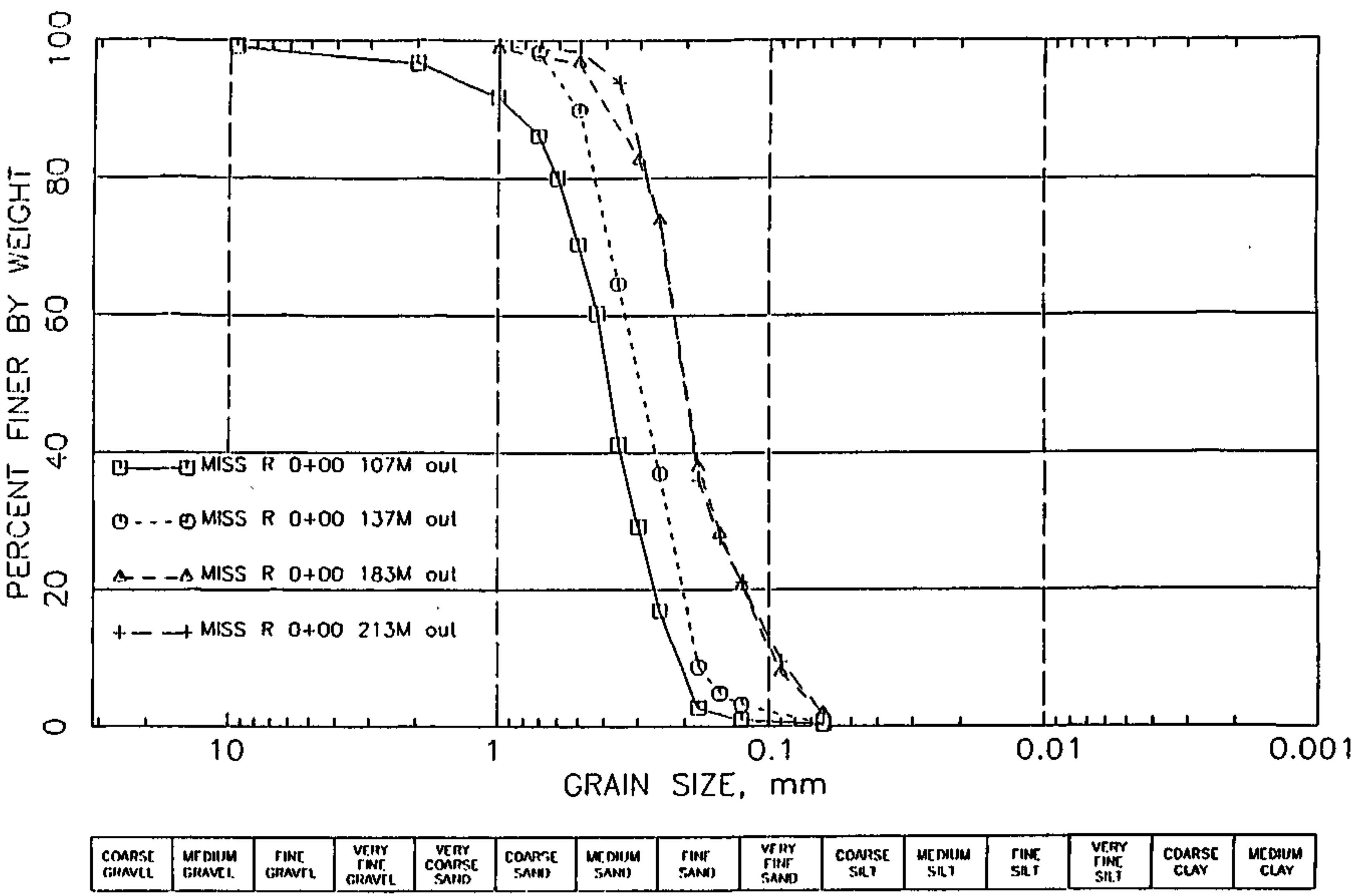
$$U = d_{60}/d_{10}$$

where d_{aa} indicates the equivalent diameters (in millimeters) for which aa percent of the particles are finer in diameter (in millimeters).

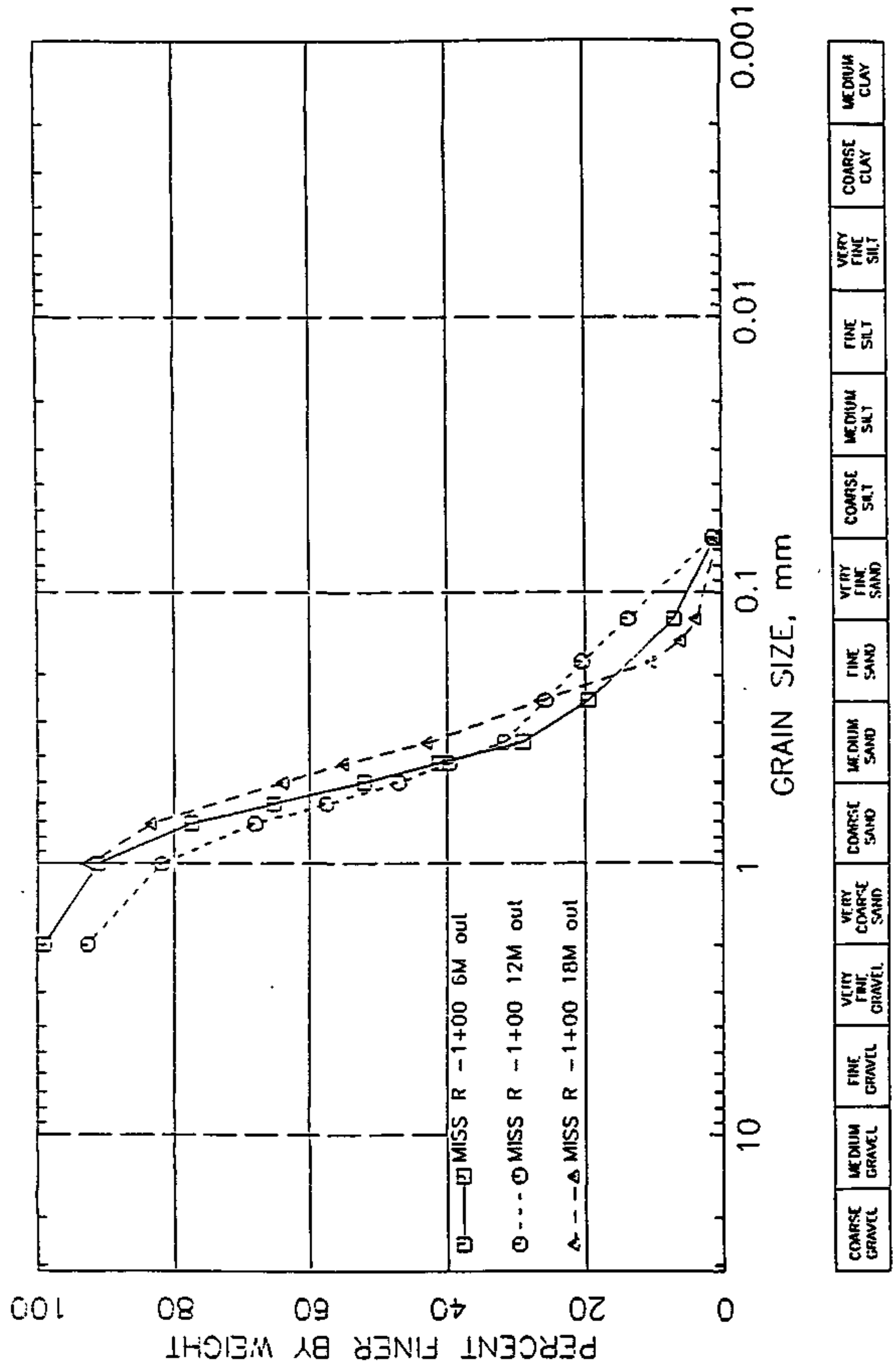
Appendix A-3. Particle Size Distributions of the Bed Material Samples at the Red Wing Site


PERCENT FINER BY WEIGHT
100
80
60
40
20
0
GRAIN SIZE, mm
10
1
0.1
0.01
0.001
MISS R 1+00 up 6M out
MISS R 1+00 up 12M out
MISS R 1+00 up 18M out
COARSE GRAVEL
MEDIUM GRAVEL
FINE GRAVEL
VERY FINE GRAVEL
VERY COARSE SAND
COARSE SAND
MEDIUM SAND
FINE SAND
VERY FINE SAND
COARSE SILT
MEDIUM SILT
FINE SILT
VERY FINE SILT
COARSE CLAY
MEDIUM CLAY


PERCENT FINER BY WEIGHT
0
20
40
60
80
100
GRAIN SIZE, mm
10
1
0.1
0.01
0.001
MISS R 0+00 6M out
MISS R 0+00 12M out
MISS R 0+00 18M out
MISS R 0+00 46M out
MISS R 0+00 76M out
COARSE GRAVEL
MEDIUM GRAVEL
FINE GRAVEL
VERY FINE GRAVEL
VERY COARSE SAND
COARSE SAND
MEDIUM SAND
FINE SAND
VERY FINE SAND
COARSE SILT
MEDIUM SILT
FINE SILT
VERY FINE SILT
COARSE CLAY
MEDIUM CLAY



PERCENT FINER BY WEIGHT
100
80
60
40
20
0
10
1
0.1
0.01
0.001
GRAIN SIZE, mm
MISS R 0+00 107M out
MISS R 0+00 137M out
MISS R 0+00 183M out
MISS R 0+00 213M out
COARSE GRAVEL
MEDIUM GRAVEL
FINE GRAVEL
VERY FINE GRAVEL
VERY COARSE SAND
COARSE SAND
MEDIUM SAND
FINE SAND
VERY FINE SAND
COARSE SILT
MEDIUM SILT
FINE SILT
VERY FINE SILT
COARSE CLAY
MEDIUM CLAY



MISS R -1+00 6M out
MISS R -1+00 12M out
MISS R -1+00 18M out
PERCENT FINER BY WEIGHT
0
20
40
60
80
100
GRAIN SIZE, mm
10
1
0.1
0.01
0.001
COARSE GRAVEL
MEDIUM GRAVEL
FINE GRAVEL
VERY FINE GRAVEL
VERY COARSE SAND
COARSE SAND
MEDIUM SAND
FINE SAND
VERY FINE SAND
COARSE SILT
MEDIUM SILT
FINE SILT
VERY FINE SILT
COARSE CLAY
MEDIUM CLAY

Appendix A-4. Characteristics of the Bed Material Samples at the Red Wing Site

Sample location	d_{50}	d_{85}	σ	U
30 meters u/s, 6.1 meters	.68	1.2	1.8	2.6
30 meters u/s, 12.2	.58	1.2	1.9	3.1
30 meters u/s, 18.3 meters	.42	.80	2.1	3.1
Survey site, 6.1 meters	.50	1.5	3.3	5.7
Survey site, 12.2 meters	.60	1.0	1.6	1.8
Survey site, 18.3 meters	.53	1.9	3.3	4.5
Survey site, 45.8 meters	.30	.75	2.1	2.6
Survey site, 76.3 meters	.33	.61	1.7	2.0
Survey site, 107 meters	.39	.69	1.7	1.9
Survey site, 137 meters	.29	.43	1.5	1.7
Survey site, 183 meters	.20	.32	1.7	2.3
Survey site, 213 meters	.20	.30	1.7	2.3
30 meters d/s, 6.1 meters	.47	.81	2.0	4.1
30 meters d/s, 12.2 meters	.50	1.2	2.9	6.0
30 meters d/s, 18.3 meters	.40	.73	1.9	2.5

Remarks:

σ is the standard deviation and it is calculated as

$$\sigma = \frac{1}{2}\left[\frac{d_{84.1}}{d_{50}} + \frac{d_{50}}{d_{15.9}}\right]$$

U is the uniformity coefficient and it is calculated as

$$U = d_{60}/d_{10}$$

where d_{aa} indicates the equivalent diameters (in millimeters) for which aa percent of the particles are finer in diameter.

Appendiix B-1. Summary of Field Data from Controlled Runs, Havana Site

Name or manufacturer	*Type*	*Length (meters)*	*Gagel (meters)*	*Gage2 (meters)*	*Direction*	*Speed (m/s)*	*Speed (mph)*
Monitor	Work Boat	5.8	15.3	7.0	up	3.2	7.1
Monitor	Work Boat	5.8	15.3	7.0	down	3.5	7.8
Monitor	Work Boat	5.8	15.3	7.0	up	10.6	23.8
Monitor	Work Boat	5.8	15.3	7.0	down	10.9	24.5
Monitor	Work Boat	5.8	15.3	7.0	up	13.5	30.2
Monitor	Work Boat	5.8	15.3	7.0	down	13.6	30.3
Monitor	Work Boat	5.8	30.5	22.3	up	3.3	7.3
Monitor	Work Boat	5.8	30.5	22.3	down	3.5	7.9
Monitor	Work Boat	5.8	30.5	22.3	up	10.2	22.8
Monitor	Work Boat	5.8	30.5	22.3	down	10.7	23.9
Monitor	Work Boat	5.8	30.5	22.3	up	13.6	30.4
Monitor	Work Boat	5.8	30.5	22.3	down	13.9	31.1
Monitor	Work Boat	5.8	61.0	52.8	up	3.0	6.8
Monitor	Work Boat	5.8	61.0	52.8	down	3.8	8.5
Monitor	Work Boat	5.8	61.0	52.8	up	9.9	22.2
Monitor	Work Boat	5.8	61.0	52.8	down	10.0	22.4
Monitor	Work Boat	5.8	—	52.8	up	12.6	28.2
Monitor	Work Boat	5.8	—	52.8	down	13.8	30.8
Monitor	Work Boat	5.8	—	83.3	up	3.2	7.2
Monitor	Work Boat	5.8	—	83.3	down	3.8	8.6
Monitor	Work Boat	5.8	—	83.3	up	10.1	22.5
Monitor	Work Boat	5.8	—	83.3	down	11.2	25.0
Monitor	Work Boat	5.8	—	83.3	up	13.0	29.0
Queen Mary	Runabout	5.5	61.0	52.8	up	3.4	7.5
Queen Mary	Runabout	5.5	61.0	52.8	down	9.8	21.8
Queen Mary	Runabout	5.5	61.0	52.8	up	10.8	24.1
Queen Mary	Runabout	5.5	61.0	52.8	down	12.4	27.7
Queen Mary	Runabout	5.5	61.0	—	up	15.6	35.0
Queen Mary	Runabout	5.5	61.0	—	down	17.5	39.1
Queen Mary	Runabout	5.5	30.5	22.3	up	3.1	6.9
Queen Mary	Runabout	5.5	30.5	22.3	down	4.3	9.5
Queen Mary	Runabout	5.5	30.5	22.3	up	4.0	8.9
Queen Mary	Runabout	5.5	30.5	22.3	down	12.0	26.7
Queen Mary	Runabout	5.5	30.5	22.3	up	15.5	34.6
Queen Mary	Runabout	5.5	30.5	22.3	down	17.2	38.5
Queen Mary	Runabout	5.5	61.0	52.8	up	3.4	7.5
Queen Mary	Runabout	5.5	61.0	52.8	down	9.8	21.8
Queen Mary	Runabout	5.5	61.0	52.8	up	10.8	24.1
EEL	Jon Boat	5.5	15.3	7.0	up	3.1	6.9
EEL	Jon Boat	5.5	15.3	7.0	down	3.6	8.0
EEL	Jon Boat	5.5	15.3	7.0	up	9.1	20.4

Appendix B-1 (continued)

Name or manufacturer	*Type*	*Length (meters)*	*Gagel (meters)*	*Gage2*	*Direction*	*Speed (m/s)*	*(mph)*
EEL	Jon Boat	5.5	15.3	7.0	down	9.8	1.9
EEL	Jon Boat	5.5	15.3	7.0	up	14.4	32.2
EEL	Jon Boat	5.5	15.3	7.0	down	15.9	35.5
EEL	Jon Boat	5.5	30.5	22.3	up	6.2	13.8
EEL	Jon Boat	5.5	30.5	22.3	down	3.8	**8.4**
EEL	Jon Boat	5.5	30.5	22.3	up	10.5	23.5
EEL	Jon Boat	5.5	30.5	22.3	down	12.2	27.3
EEL	Jon Boat	5.5	30.5	22.3	up	14.2	31.8
EEL	Jon Boat	5.5	30.5	22.3	down	16.3	36.3
EEL	Jon Boat	5.5	61.0	52.8	up	3.0	6.7
EEL	Jon Boat	5.5	61.0	52.8	down	4.0	8.9
EEL	Jon Boat	5.5	61.0	52.8	up	10.3	23.1
EEL	Jon Boat	5.5	—	52.8	down	11.4	25.5
EEL	Jon Boat	5.5	—	52.8	up	16.2	36.1
EEL	Jon Boat	5.5	—	52.8	down	18.5	41.3
EEL	Jon Boat	5.5	91.5	83.3	up	2.8	6.2
EEL	Jon Boat	5.5	91.5	83.3	down	3.6	8.0
EEL	Jon Boat	5.5	91.5	83.3	up	9.3	20.8
EEL	Jon Boat	5.5	—	83.3	down	10.4	23.2
EEL	Jon Boat	5.5	91.5	83.3	down	16.9	37.8
EEL	Jon Boat	5.5	—	83.3	down	20.3	45.3
Barracuda	Runabout	5.8	15.3	7.0	up	3.3	7.4
Barracuda	Runabout	5.8	15.3	7.0	down	3.9	8.7
Barracuda	Runabout	5.8	15.3	7.0	up	12.2	27.2
Barracuda	Runabout	5.8	15.3	7.0	down	12.4	27.7
Barracuda	Runabout	5.8	15.3	7.0	up	17.5	39.1
Barracuda	Runabout	5.8	15.3	—	down	17.3	38.7
Barracuda	Runabout	5.8	30.5	—	up	3.9	8.7
Barracuda	Runabout	5.8	30.5	—	down	5.2	11.6
Barracuda	Runabout	5.8	30.5	—	up	12.1	27.0
Barracuda	Runabout	5.8	30.5	—	down	13.0	29.0
Barracuda	Runabout	5.8	30.5	—	up	17.7	39.6
Barracuda	Runabout	5.8	61.0	52.8	up	4.5	10.0
Barracuda	Runabout	5.8	61.0	52.8	up	5.2	11.6
Barracuda	Runabout	5.8	61.0	52.8	down	12.0	26.8
Barracuda	Runabout	5.8	61.0	52.8	up	12.1	27.1
SWS Jon Boat	Jon Boat	3.7	15.3	7.0	up	3.7	8.2
SWS Jon Boat	Jon Boat	3.7	30.5	22.3	up	4.6	10.2
SWS Jon Boat	Jon Boat	3.7	15.3	7.0	up	7.8	17.5
SWS Jon Boat	Jon Boat	3.7	30.5	22.3	up	2.5	5.6
SWS Jon Boat	Jon Boat	3.7	30.5	22.3	down	4.8	10.8

Appendix B-1 (concluded)

Name or manufacturer	*Type*	*Length (meters)*	*Gagel*	*Gage2*	*Direction*	*Speed (m/s)*	*Speed (mph)*
			(meters)				
SWS Jon Boat	Jon Boat	3.7	30.5	22.3	up	7.7	17.3
SWS Jon Boat	Jon Boat	3.7	30.5	22.3	down	8.7	19.4
SWS Jon Boat	Jon Boat	3.7	—	52.8	up	3.1	6.8
SWS Jon Boat	Jon Boat	3.7	—	52.8	down	4.6	10.2
River Diver	Pontoon	7.3	15.3	7.0	up	2.6	5.8
River Diver	Pontoon	7.3	15.3	7.0	down	3.2	7.2
River Diver	Pontoon	7.3	15.3	7.0	up	6.3	14.2
River Diver	Pontoon	7.3	15.3	7.0	down	7.4	16.4
River Diver	Pontoon	7.3	30.5	22.3	up	3.4	7.6
River Diver	Pontoon	7.3	30.5	22.3	down	4.4	9.8
River Diver	Pontoon	7.3	—	22.3	up	7.0	15.7
River Diver	Pontoon	7.3	—	22.3	down	8.2	18.3
River Diver	Pontoon	7.3	61.0	52.8	up	3.3	7.3
River Diver	Pontoon	7.3	61.0	52.8	down	4.4	9.9
River Diver	Pontoon	7.3	61.0	52.8	up	6.9	15.5
River Diver	Pontoon	7.3	61.0	52.8	down	8.5	19.0

Appendix B-2. Summary **of Field Data from Controlled Runs, Red** Wing Site

Name or manufacturer	*Type*	*Length (meters)*	*Gagel*	*Gage2*	*Direction*	*Speed*[1]	
			(meters)			*(m/s)*	*(mph)*
Propinquity	Houseboat	14.3	22.3	16.2	down	3.9	8.8
Propinquity	Houseboat	14.3	22.3	16.2	up	3.5	7.8
Propinquity	Houseboat	14.3	22.3	16.2	down	5.1	11.4
Propinquity	Houseboat	14.3	22.3	16.2	up	6.3	14.0
Propinquity	Houseboat	14.3	22.3	—	down	8.6	19.2
Propinquity	Houseboat	14.3	22.3	16.2	up	9.0	20.1
Propinquity	Houseboat	14.3	37.5	31.4	down	4.0	8.9
Propinquity	Houseboat	14.3	37.5	31.4	up	4.0	8.9
Propinquity	Houseboat	14.3	37.5	31.4	down	5.1	11.5
Propinquity	Houseboat	14.3	37.5	31.4	up	6.1	13.6
Propinquity	Houseboat	14.3	37.5	31.4	down	8.5	19.1
Propinquity	Houseboat	14.3	37.5	31.4	up	8.7	19.5
Propinquity	Houseboat	14.3	68.0	61.9	down	3.8	8.5
Propinquity	Houseboat	14.3	68.0	61.9	up	3.7	8.3
Propinquity	Houseboat	14.3	68.0	61.9	down	5.3	11.9
Propinquity	Houseboat	14.3	68.0	61.9	up	5.0	11.1
Propinquity	Houseboat	14.3	68.0	61.9	down	8.7	19.4
Propinquity	Houseboat	14.3	68.0	61.9	up	8.4	18.8
Scorpion	Power Boat	11.9	23.8	17.7	up	4.7	10.6
Scorpion	Power Boat	11.9	22.3	16.2	down	7.4	16.6
Scorpion	Power Boat	11.9	23.8	17.7	up	10.0	22.3
Scorpion	Power Boat	11.9	22.3	16.2	down	14.2	31.7
Scorpion	Power Boat	11.9	22.3	16.2	up	16.0	35.7
Scorpion	Power Boat	11.9	22.3	16.2	down	17.0	38.0
Scorpion	Power Boat	11.9	37.5	31.4	up	6.8	15.3
Scorpion	Power Boat	11.9	37.5	31.4	down	7.6	16.9
Scorpion	Power Boat	11.9	37.5	31.4	up	13.4	29.9
Scorpion	Power Boat	11.9	37.5	31.4	down	13.1	29.4
Scorpion	Power Boat	11.9	37.5	31.4	up	17.0	38.1
Scorpion	Power Boat	11.9	37.5	31.4	down	14.3	31.9
Scorpion	Power Boat	11.9	68.0	61.9	up	7.5	16.8
Scorpion	Power Boat	11.9	68.0	61.9	down	6.7	15.1
Scorpion	Power Boat	11.9	68.0	61.9	up	12.1	27.1
Scorpion	Power Boat	11.9	68.0	61.9	down	13.1	29.3
Scorpion	Power Boat	11.9	68.0	61.9	up	18.4	41.0
Scorpion	Power Boat	11.9	68.0	61.9	down	17.2	38.5
Sea Ray	Cabin Cruiser	11.3	—	31.4	down	4.2	9.3
Sea Ray	Cabin Cruiser	11.3	—	31.4	up	3.7	8.3
Sea Ray	Cabin Cruiser	11.3	—	31.4	down	8.2	18.3
Sea Ray	Cabin Cruiser	11.3	—	31.4	up	8.2	18.3
Sea Ray	Cabin Cruiser	11.3	—	31.4	down	14.8	33.1

Appendix B-2 (continued)

Name or manufacturer	*Type*	*Length (meters)*	*Gagel*	*Gage2*	*Direction*	*Speed (m/s)*	*Speed (mph)*
			(meters)				
Sea Ray	Cabin Cruiser	11.3	—	31.4	up	14.2	31.8
Sea Ray	Cabin Cruiser	11.3	—	61.9	down	4.5	10.0
Sea Ray	Cabin Cruiser	11.3	—	61.9	up	4.0	8.9
Sea Ray	Cabin Cruiser	11.3	—	61.9	down	10.0	22.4
Sea Ray	Cabin Cruiser	11.3	—	61.9	up	8.0	17.9
Sea Ray	Cabin Cruiser	11.3	—	61.9	down	15.3	34.3
Sea Ray	Cabin Cruiser	11.3	—	61.9	up	14.1	31.4
Sea Ray	Cabin Cruiser	11.3	—	16.2	down	4.2	9.5
Sea Ray	Cabin Cruiser	11.3	—	16.2	up	3.8	8.4
Sea Ray	Cabin Cruiser	11.3	—	16.2	down	10.1	22.6
Sea Ray	Cabin Cruiser	11.3	—	16.2	up	9.2	20.5
Sea Ray	Cabin Cruiser	11.3	—	16.2	down	14.6	32.7
Sea Ray	Cabin Cruiser	11.3	—	16.2	up	14.3	32.0
Trojan	Cabin Cruiser	7.9	20.7	14.6	down	6.4	14.3
Trojan	Cabin Cruiser	7.9	20.7	14.6	up	6.2	13.8
Trojan	Cabin Cruiser	7.9	20.7	14.6	down	8.6	19.3
Trojan	Cabin Cruiser	7.9	20.7	14.6	up	9.5	.21.3
Trojan	Cabin Cruiser	7.9	20.7	14.6	down	11.3	25.3
Trojan	Cabin Cruiser	7.9	20.7	14.6	up	11.8	26.5
Trojan	Cabin Cruiser	7.9	36.0	29.9	down	5.1	11.5
Trojan	Cabin Cruiser	7.9	36.0	29.9	up	5.0	11.3
Trojan	Cabin Cruiser	7.9	36.0	29.9	down	9.1	20.4
Trojan	Cabin Cruiser	7.9	36.0	29.9	up	9.8	22.0
Trojan	Cabin Cruiser	7.9	36.0	29.9	down	11.6	25.9
Trojan	Cabin Cruiser	7.9	36.0	29.9	up	12.0	26.8
Aluminum Craft	V-Hull	5.8	25.3	19.2	down	5.2	11.6
Aluminum Craft	V-Hull	5.8	20.7	14.6	up	5.0	11.2
Aluminum Craft	V-Hull	5.8	22.3	16.2	down	13.2	29.5
Aluminum Craft	V-Hull	5.8	20.7	14.6	up	11.5	25.6
Aluminum Craft	V-Hull	5.8	20.7	14.6	down	17.1	38.2
Aluminum Craft	V-Hull	5.8	20.7	14.6	up	16.7	37.2
Aluminum Craft	V-Hull	5.8	36.0	29.9	down	4.9	11.0
Aluminum Craft	V-Hull	5.8	36.0	29.9	up	4.8	10.8
Aluminum Craft	V-Hull	5.8	36.0	29.9	down	13.1	29.3
Aluminum Craft	V-Hull	5.8	36.0	29.9	up	11.7	26.3
Aluminum Craft	V-Hull	5.8	36.0	29.9	down	17.4	38.9
Aluminum Craft	V-Hull	5.8	36.0	29.9	up	16.4	36.7
Lund-Baron	V-Hull	6.4	23.8	17.7	up	4.3	9.7
Lund-Baron	V-Hull	6.4	20.7	14.6	down	4.9	10.9
Lund-Baron	V-Hull	6.4	20.7	14.6	up	14.7	32.9
Lund-Baron	V-Hull	6.4	20.7	14.6	down	14.5	32.3

Appendix B-2 (concluded)

Name or manufacturer	Type	Length (meters)	Gagel (meters)	Gage2	Direction	Speed (m/s)	(mph)
Lund-Baron	V-Hull	6.4	20.7	14.6	up	21.4	47.9
Lund-Baron	V-Hull	6.4	20.7	14.6	down	21.3	47.5
Lund-Baron	V-Hull	6.4	37.5	31.4	up	4.0	8.9
Lund-Baron	V-Hull	6.4	37.5	31.4	down	4.3	9.7
Lund-Baron	V-Hull	6.4	37.5	31.4	up	12.7	28.5
Lund-Baron	V-Hull	6.4	37.5	31.4	down	12.9	28.8
Lund-Baron	V-Hull	6.4	37.5	31.4	up	21.9	48.9
Lund-Baron	V-Hull	6.4	68.0	61.9	down	4.1	9.1
Lund-Baron	V-Hull	6.4	68.0	61.9	up	4.5	10.0
Lund-Baron	V-Hull	6.4	68.0	61.9	down	14.4	32.2
Lund-Baron	V-Hull	6.4	68.0	61.9	up	13.3	29.7
Lund-Baron	V-Hull	6.4	68.0	61.9	down	21.4	47.7
Lund-Baron	V-Hull	6.4	68.0	61.9	up	20.4	45.5
Barracuda	Runabout	5.8	37.5	31.4	up	3.5	7.8
Barracuda	Runabout	5.8	37.5	31.4	down	4.3	9.7
Barracuda	Runabout	5.8	37.5	31.4	up	11.6	25.8
Barracuda	Runabout	5.8	37.5	31.4	down	11.3	25.3
Barracuda	Runabout	5.8	37.5	31.4	up	16.6	37.2
Barracuda	Runabout	5.8	37.5	31.4	down	16.2	36.1
Barracuda	Runabout	5.8	65.0	58.9	up	4.0	9.0
Barracuda	Runabout	5.8	65.0	58.9	down	4.1	9.3
Barracuda	Runabout	5.8	65.0	58.9	up	11.6	25.8
Barracuda	Runabout	5.8	65.0	58.9	down	11.4	25.6
Barracuda	Runabout	5.8	65.0	58.9	up	16.4	36.7
Barracuda	Runabout	5.8	65.0	58.9	down	16.3	36.3
Barracuda	Runabout	5.8	36.0	29.9	up	3.7	8.3)
Barracuda	Runabout	5.8	36.0	29.9	down	4.4	9.8
Barracuda	Runabout	5.8	36.0	29.9	up	10.1	22.6
Barracuda	Runabout	5.8	36.0	29.9	down	9.9	22.2
Barracuda	Runabout	5.8	36.0	29.9	up	16.5	36.8
Barracuda	Runabout	5.8	36.0	29.9	down	17.0	37.9
Barracuda	Runabout	5.8	20.7	14.6	up	3.6	8.1
Barracuda	Runabout	5.8	20.7	14.6	down	4.6	10.2
Barracuda	Runabout	5.8	20.7	14.6	up	12.4	27.6
Barracuda	Runabout	5.8	20.7	14.6	down	11.9	26.5
Barracuda	Runabout	5.8	20.7	14.6	up	17.1	38.2
Barracuda	Runabout	5.8	20.7	14.6	down	17.1	38.2

[redacted]