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REPORT OF INVESTIGATION 92
STATE OF ILLINOIS
ILLINOIS INSTITUTE OF NATURAL RESOURCES

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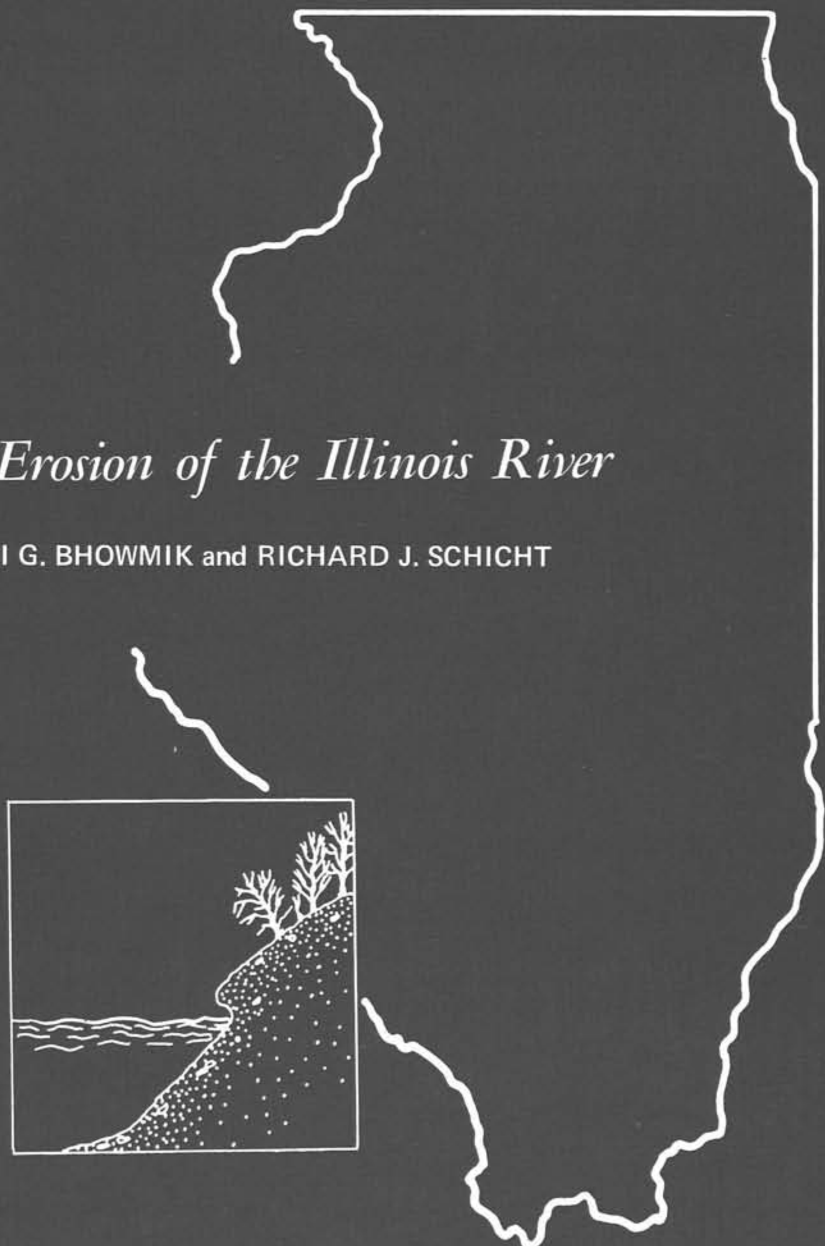
Bank Erosion of the Illinois River

by NANI G. BHOWMIK and RICHARD J. SCHICHT



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ILLINOIS STATE WATER SURVEY
URBANA
1980

REPORT OF INVESTIGATION 92

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Abstract: Banks of the Illinois River have been eroding because of natural and man-made acts. In many places the erosion is very severe, in other places the banks are stable. The bank erosion of the river was investigated in detail and the results are presented in this report. Field inspection of the river from Joliet to Grafton was made. Extensive bed and bank material samples were collected and grain size distributions were determined. Plan views of 20 selected reaches were developed and the bank slopes at these reaches were determined. Hydraulic parameters were either computed or estimated, and the stability of the banks at all 20 locations was tested following accepted methods and techniques in hydraulics. The stability analysis was done for discharges with and without additional Lake Michigan diversions for three typical water years. In general, the silty, sandy, and clayey materials of these severely eroded banks should be stable against the action of tractive force and flow velocity. However, preliminary computations indicated that the banks are unstable as far as the wind-generated wave action is concerned. It is suspected that river traffic-generated wave action also has a similar effect. A monitoring program is outlined, and a future research project related to the wave action on the banks is suggested.

Reference: Bhowmik, Nani G., and Richard J. Schicht. Bank Erosion of the Illinois River. Illinois State Water Survey, Urbana, Report of Investigation 92, 1980.

Indexing Terms: Bank erosion, bank stability, bank protection, hydraulics, resistance, Illinois River, velocity, waves (water), barges, tractive forces, shear stress, particle size.

URBANA
1980

Printed by authority of the State of Illinois
(20741-7-80-1000)

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ABSTRACT

Banks of the Illinois River have been eroding because of natural and man-made acts. In many places the erosion is very severe; in other places the banks are stable. The bank erosion of the river was investigated in detail to ascertain the probable effects of increased Lake Michigan diversion on bank stability or erosion. Field inspection of the river from Joliet to Grafton was made. Extensive bed and bank material samples were collected and grain size distributions were determined. Plan views of 20 selected reaches' were developed and the bank slopes at these reaches were determined. Hydraulic parameters were either computed or estimated, and the stability of the banks at all 20 locations was tested following accepted methods and techniques in hydraulics.

The stability analysis was done for discharges with and without additional Lake Michigan diversions for three typical water years. In general, the silty, sandy, and clayey materials of these severely eroded banks should be stable against the action of tractive force and flow velocity. However, preliminary computations indicated that the banks are unstable as far as the wind-generated wave action is concerned. It is suspected that river traffic-generated wave action also has a similar effect. A monitoring program is outlined, and a future research project related to the wave action on the banks is suggested.

INTRODUCTION

A 5-year study and demonstration program to determine the effects of increased Lake Michigan diversion on water quality of the Illinois Waterway and on the susceptibility of the Illinois Waterway to additional flooding was authorized in Section 166 of the Water Resources Development Act of 1976 (P.L. 94-587). It was planned during the 5-year demonstration program to increase Lake Michigan diversion from the presently authorized 3200 cubic feet per second (cfs) to a maximum of 10,000 cfs.

The incremental flow may or may not have any effect on the regime of the river. In order to get a better understanding of the effects of increased flow on the hydraulics of flow and its effect on bank erosion, the U. S. Army Corps of Engineers through the Illinois Division of Water Resources funded the Illinois State Water Survey to study the present bank erosion areas of the Illinois River. This preliminary study provides some answers as to the probable effects of the increased diversion on the stability or erosion of the banks of the Illinois River.

This report presents the objectives of the study, a description of afield trip on the river, the method of analysis, and the results of the study. A recommended program for monitoring the bank erosion areas of the Illinois River and a possible future research program are discussed.

The material in this report was originally prepared in draft form by Bhowmik and Schicht (1979) for the Illinois Division of Water Resources, and that document contains the surveyors' monument locations and the raw grain size analyses for bank and bed material samples.

Acknowledgments

This research was conducted by the authors as part of their regular duties at the State Water Survey under the general supervision of Dr. William C. Ackermann, now Chief Emeritus of the Illinois State Water Survey. The U. S. Army Corps of Engineers supplied the boat and the pilot for the field trip on the river. Sam Nakib of the Corps of Engineers accompanied the data collection crew during the field trip. Ms. Karen Kabbes and Mike Diedrichsen of the Illinois Division of Water Resources helped in the collection of the field data during the boat trip. Water Survey employees Bill Bogner, Jim Gibb, Ken Smith, Keu K. Kim, and Misganaw DeMissie assisted in the field data collection program. Misganaw DeMissie, Rose Mary Roberts, and Katalin Bajor helped in the analysis of the field data. Kurt Peterson and William Motherway, Jr., prepared the illustrations. Mrs. J. Loreena Ivens and Mrs. Patricia A. Motherway edited the report. Mrs. Marilyn J. Innes prepared the camera copy.

A & H Engineering of Champaign, analyzed the grain-size distribution of the bank and bed materials. Dodson-Van Wie Engineering and Surveying, Ltd., of Mattoon, Illinois performed the detailed surveying.

BACKGROUND AND DATA COLLECTION

The Illinois River and its main tributaries stretch from Milwaukee, Wisconsin, and South Bend, Indiana, to Grafton, Illinois. It is one of the main waterways in Illinois. The tributaries of this river basically drain farmlands. Figure 1 shows the drainage basin of the Illinois River. The drainage area of the Illinois River is 28,906 square miles.

Physiographically, the river basin is located in the till plains section of the central United States (Fenneman, 1928). Large scale relief features are absent within Illinois; however, there are some local features which effectively change the physiographic features of the basin from one location to another.

On the basis of the topography of the bedrock surface, glaciations, age of the drift, and other factors, the state of Illinois was divided into a number of physiographic divisions by Leighton et al. (1948). The Illinois River flows through about five of these physiographic divisions characterized by broad till plains which are in the youthful stages of erosion.

The river in its upper part above the big bend near DePue has a broad flat bottom valley with steep walls. Between DePue and Peoria, the floodplains of the river are rather narrow; downstream from Peoria, the floodplains of the river are rather wide. This is especially true for the length of the river from Pekin to Meredosia. Downstream from Meredosia, the floodplain of the river gradually narrows until it meets with the Mississippi River near Grafton.

The Illinois River in its present form consists of a series of pools created by eight locks and dams. The water surface profiles and the average depths of flow are maintained by these locks and dams. The U. S. Army Corps of Engineers maintains a 9-foot navigational channel along the length of the river. This major waterway has carried a tremendous amount of barge traffic since the opening of the locks and dams in 1933. Presently over 40 million tons of traffic traverse the river in a year (Carlisle, 1977). Tows operating on the river may be composed of as many as 15 barges (carrying 1500 tons each) pushed by a 5000 horsepower tow boat. This size tow, nearly 105 feet wide and 1200 feet long, can move at a speed in excess of 8 miles per hour with a draft of 9 feet and could move 11,000 cubic feet of water per second.

The banks of any stream or river that flows through noncohesive or partly cohesive materials will erode unless there is natural or artificial protection. The causative factors of bank erosion along the Illinois River, either in combination of all or in part, are: the normal flow of the river, waves generated by the wind and/or waterway traffic, increase in flow velocity because of the passage of barge traffic, and/or a variety of other reasons including prop wash.

Objectives

The main objectives of this research project are to:

1) Document present bank erosion areas
2) Develop present plan views of severely eroded banks at about 20 selected reaches
3) Make bank stability analyses for each reach
4) Attempt to assess the effect of the increase in the Lake Michigan diversion on bank erosion

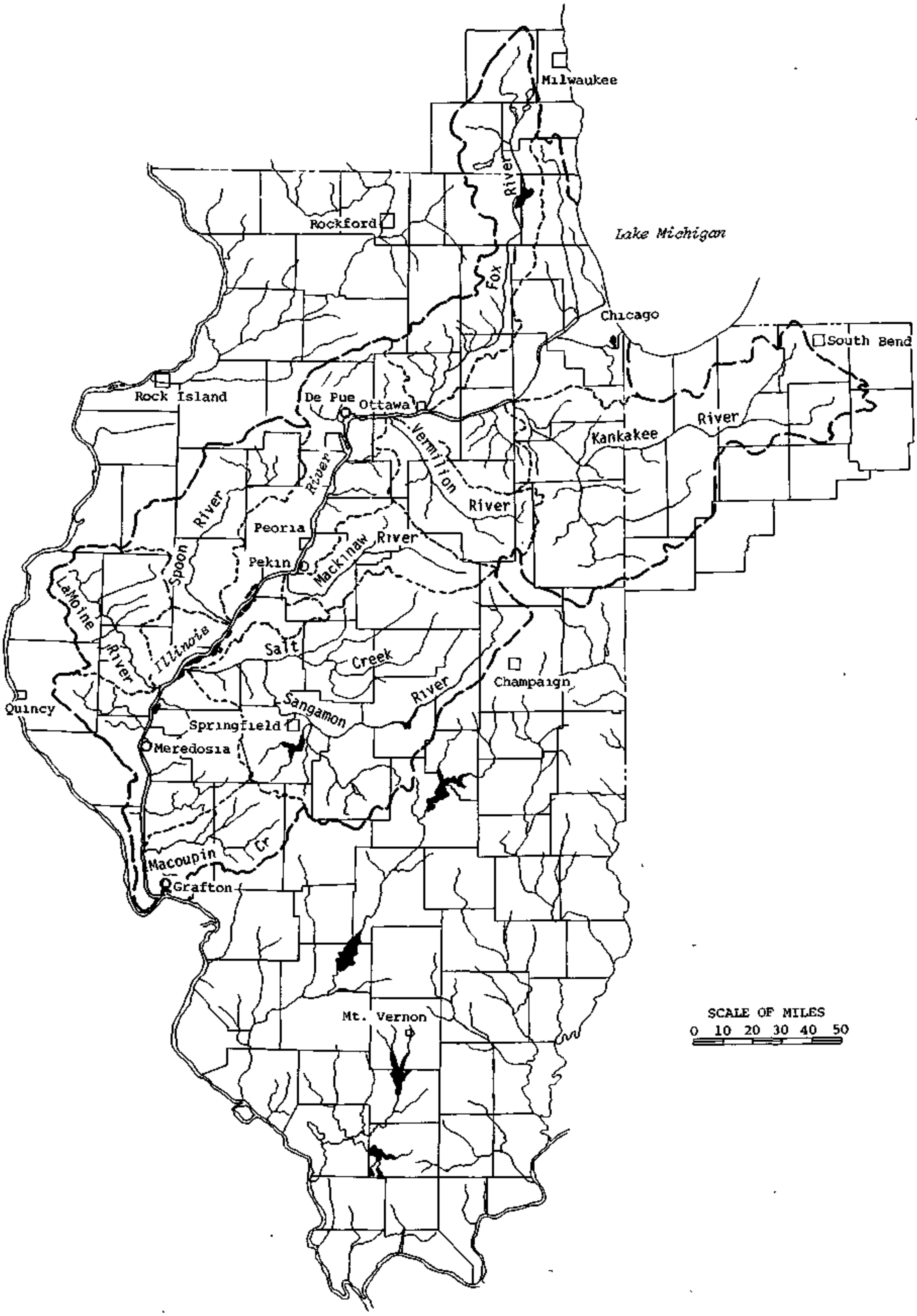


Figure 1. Drainage basin of the Illinois River

5) Propose a monitoring system to document any future changes in bank conditions
6) Suggest future research areas that should be undertaken to better identify the causes of the bank erosion of the Illinois River.

Data Collection

A 5-day boat trip on the Illinois River was taken from July 17 through 21, 1978, to document the severity of bank erosion. The U. S. Army Corps of Engineers supplied the boat and a pilot for the trip. The trip started at Joliet and ended at Pere Marquette State Park near Grafton. Photographs of the boat are shown in figure 2.

During the trip, severely eroded banks were photographed and soil samples from the eroded banks and the river bed were collected at intervals of 3 to 4 miles. Figure 3 shows the location of the 24 river reaches, each consisting of only one side of the river, selected during the field trip for initial analysis and further study.

Whenever a portion of the river bank appeared to be severely eroded, the main boat was anchored and a flat bottom metal boat was used to land at the site of the eroded bank. First, photographs of the eroded banks were taken and then a few representative areas of the banks were selected for collection of bank material samples. Photographs of banks at Reaches 6 and 18 are shown in figure 4. A 2-foot by 2-foot grid with mesh points at 0.1-foot intervals was placed on top of the undisturbed soil samples, and a photograph was taken to show the areal distribution of the undisturbed bank materials (figure 5). Subsequently, the top layer of the bank material was scraped, bagged, and analyzed at the Water Survey. This procedure was repeated for each selected reach.

The bed material samples were collected with either an Ekman dredge, a Ponar sampler, or a Shipwek sampler depending upon the condition of the flow and the effectiveness of the sampler. However, most of the bed material samples were collected by using the Ponar sampler shown in figure 6. Figure 7 shows the Shipwek sampler ready for use. Figure 8 shows locations where bank and bed material samples were collected. (The sample numbers coincide with those on tables 2 and 3.)

During the course of this boat trip, no other field data were collected. Hydraulic and flow data that were needed for further analysis were obtained either from the Chicago District Office of the U. S. Army Corps of Engineers or from the files of the U. S. Geological Survey.

The Army Corps of Engineers supplied the sounding data, the stage and discharge data for 17 locations with and without increased diversion, and geometric data at about 0.3 to 5.0-mile intervals along the river.

DATA ANALYSIS

Geometric and Hydraulic Characteristics of the Eroded Banks

There were numerous reaches of the river bank where erosion was present. The severely eroded reaches were marked on the charts of the Illinois Waterway (U. S. Army Corps of Engineers, 1974) during the course of the boat trip. Twenty of these reaches of the river were later selected for analysis and further investigation. Figure 9 shows these reaches as they were traced from the charts of the Illinois Waterway and shows the flow direction, river mile, north direction, and active channel width. The bank of the river that was selected for detailed analysis is also shown.

A detailed survey was made of each of the reaches to determine the plan view and the bank slope at about 3 to 6 sections for each reach. A permanent concrete monument was installed at or near each of the reaches. These monuments will be useful in the future to facilitate surveying the change or changes in the plan view of the selected eroded banks.

Figure 10 shows the plan views of the selected reaches along the Illinois River. The plan views, locations of the measured bank slope sections, and the direction of flow were taken from the original plan and sectional view of the reach as submitted by the surveying firm. The locations where the

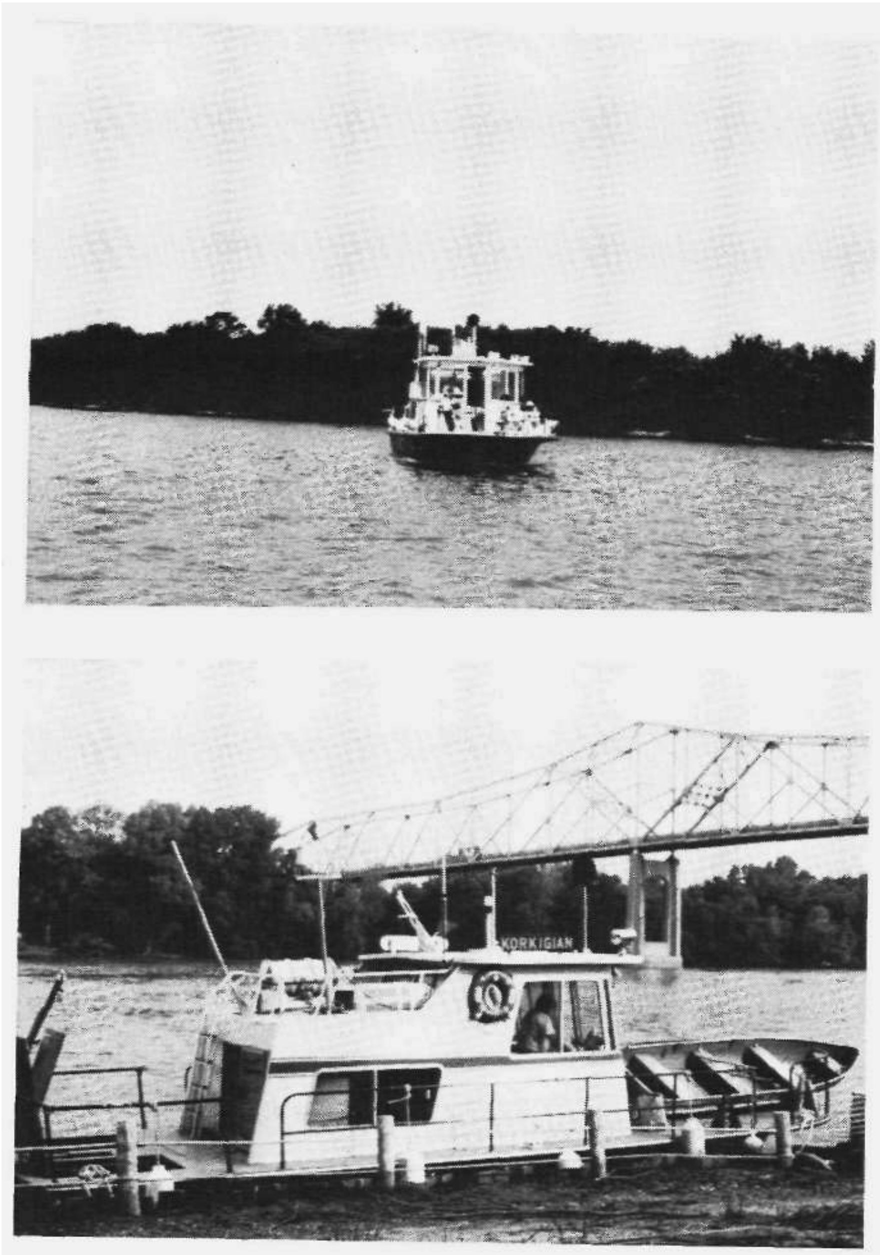


Figure 2. Photographs of the boat used in the data collection

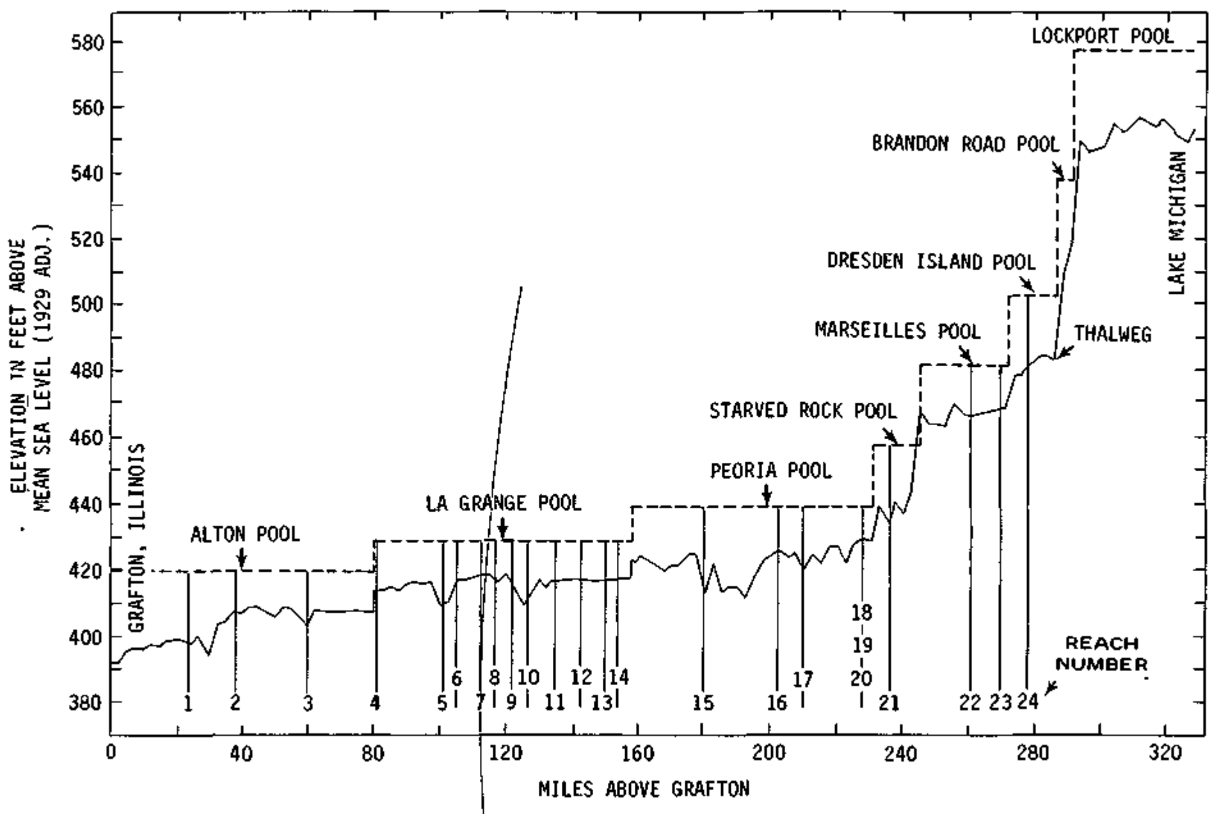


Figure 3. Profile of the Illinois River and the location of the reaches selected for further bank erosion investigations

bank material samples were collected are also shown in this figure.

The upstream part of Reach 1, figure 10, is just downstream of a bend and constitutes the outside bank of this bend. The radius of curvature, R, of this bend is 4700 feet with a deflection angle, Δ, of 41 degrees. The rest of the reach constitutes the outside bank of another bend with reverse characteristics. For the second bend the value of R is 3100 feet and A is 37.5 degrees. Close to River Mile 24, near the upstream part of the reach, the high velocity flow stayed close to the eroded bank and may be partially responsible for the erosion of the bank at this location. The deflection angle, A, in degrees, of a bend is defined as the included angle between the centerlines of the upstream and downstream reaches of the bend.

Reach 2, located on a straight portion of the river, constitutes one side of a low lying island.

Reach 3 is along a straight portion of the river just downstream of a bend with a long radius of curvature and a small deflection angle.

The upstream part of Reach 4 constitutes the outside downstream bank of a bend with a radius of curvature of 3200 feet and A of 67 degrees. The downstream part of the reach constitutes the inside bank of a bend with R equal to 4800 feet and A equal to 41 degrees. The high velocity flow and the sailing line stays close to this bank, especially near the upstream part of the reach.

Reach 5 is the outside bank of a bend with R equal to 13,000 feet and A equal to 22.5 degrees. This is an extremely flat bend at a point where the river is relatively narrow.

Reach 6 is located outside of an extremely flat bend with a long radius. For all practical purposes, this reach can be assumed to be a straight reach. Here the river is relatively narrow and the sailing line is close to the eroded bank.

Reach 7 is the outside downstream bank of a bend. The lower part of this reach forms the inside bank of the next bend. Again, the river is narrower at this location.

Reach 8 is the outside bank of a bend with R

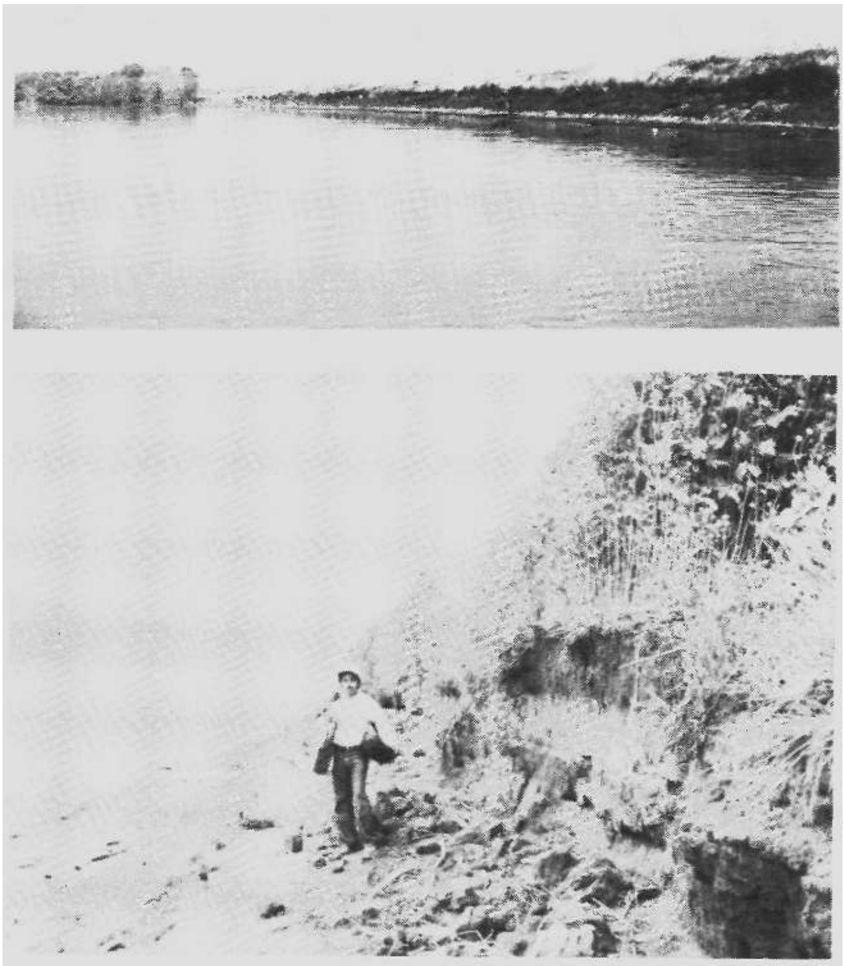


Figure 4. Photographs of Reach 6 (top) and Reach 18 (bottom)

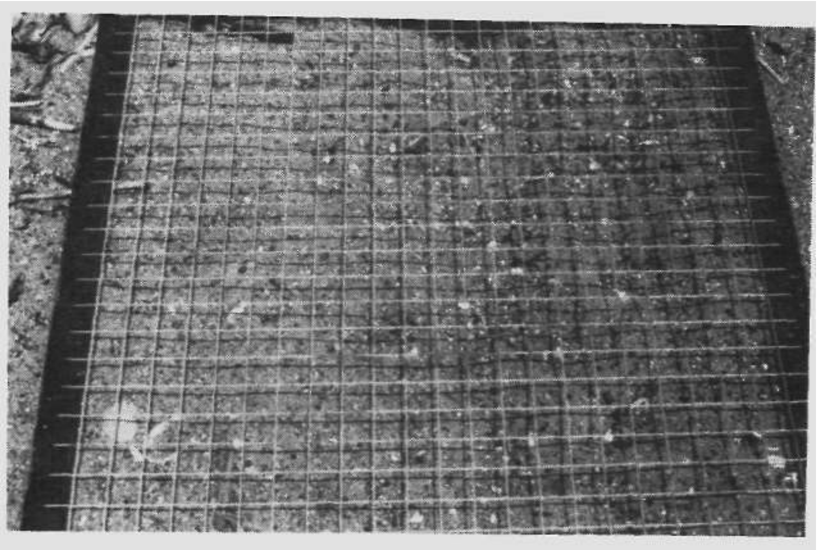


Figure 5. Undisturbed bank material

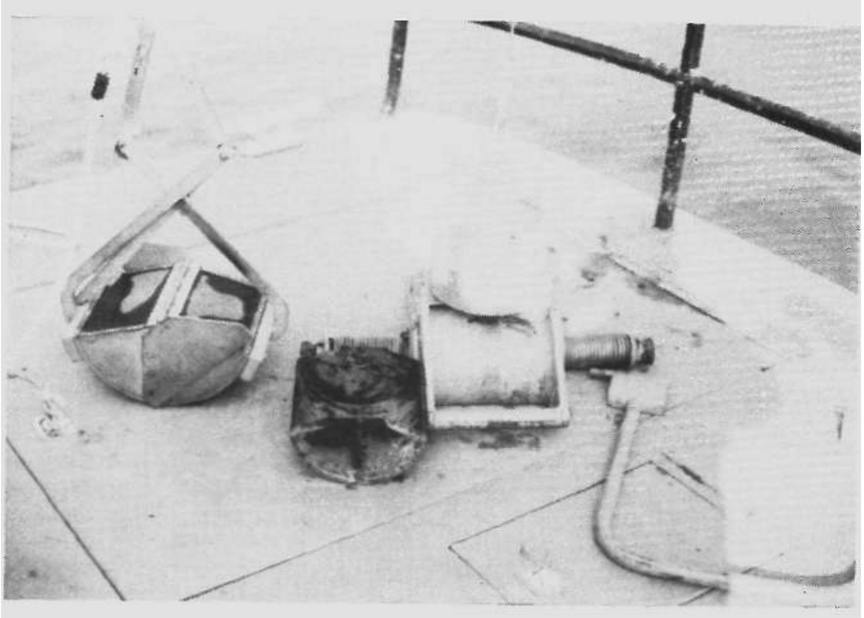


Figure 6. Photograph of the Ponar (left) and Shipwek (right) samplers

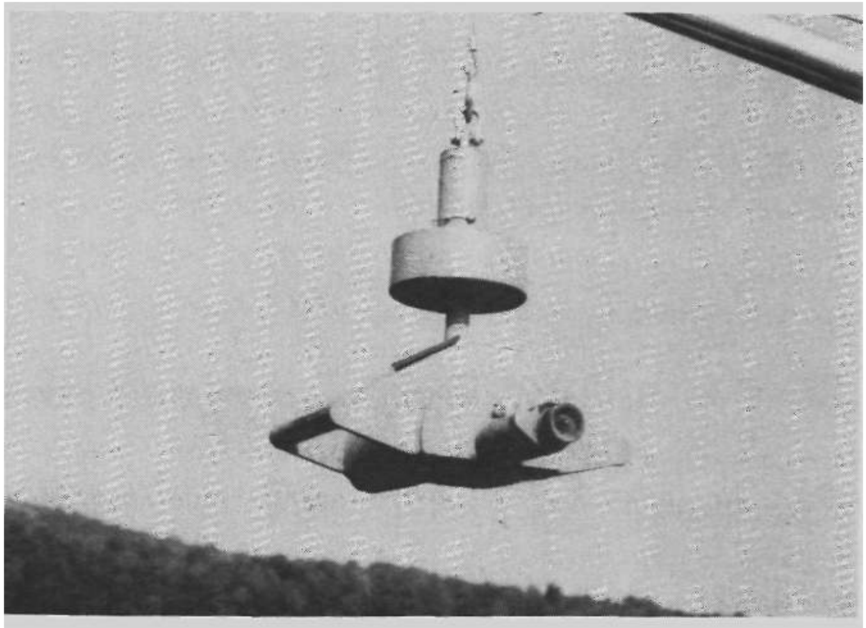


Figure 7. Photograph of the Shipwek sampler

equal to 7500 feet and A equal to 44 degrees. This is a rather sharp bend where the effect of the bend on the flow hydraulics may be a prime factor in the erosion of this bank.

Reach 9 is also the outside bank of a bend with R equal to 4900 feet and A equal to 55.5 degrees. The sailing line for this location is rather close to the bank.

Reach 12 is the outside bank of a very flat bend with R equal to 19,000 feet and A equal to 23 degrees. This reach can be assumed to be a straight reach.

On the other hand, Reach 13 is the outside bank of a very sharp bend with R equal to 2500 feet and A equal to 97 degrees. The bank erosion at this location is being accelerated by the effects of the bend on flow characteristics and possibly by the increased wave activity caused by barge traffic around such a sharp bend.

Reach 14 constitutes the inside bank just downstream of a bend with R equal to 8400 feet and A equal to 43 degrees. The bank erosion at this location is possibly the result of the barge traffic and wind wave action.

Reach 15, the left bank just upstream of Peoria Lake, can be considered to be a straight reach.

Reach 17 is basically a straight reach on the right hand side of the river. Here the river is relatively wide and the bank erosion is probably due to the wave action.

Reaches 18, 19, and 20 can be assumed to be straight reaches. There is an extremely flat bend with a very long radius of curvature just upstream of these reaches. Note that Reach 18 is located just upstream of Reach 19 and is on the same side of the river. River banks at Reaches 18 and 19 are very low and extensive erosion is present at these locations. It is suspected that the main cause of the erosion may be the wave action in the river.

Reach 22 is the inside downstream bank of a bend with R equal to 12,000 feet and A equal to 30.5 degrees. Here the cause of bank erosion is probably a combination of flow velocity and wave action in the river.

Reach 23 is on a straight segment of the river. Bank erosion is not very severe at this location. The sailing line is very close to this side of the river, and possibly wave action plays an important role in the instability of the bank.

Reach 24 is near the confluence with the Du Page River. This reach constitutes the left bank of the river. There is a very large rectangular lake just northwest of this reach. The lake is about 0.5 mile by 1 mile in size. Because the sailing line is very close to this reach, bank erosion is suspected to be caused by traffic-generated wave action in the river.

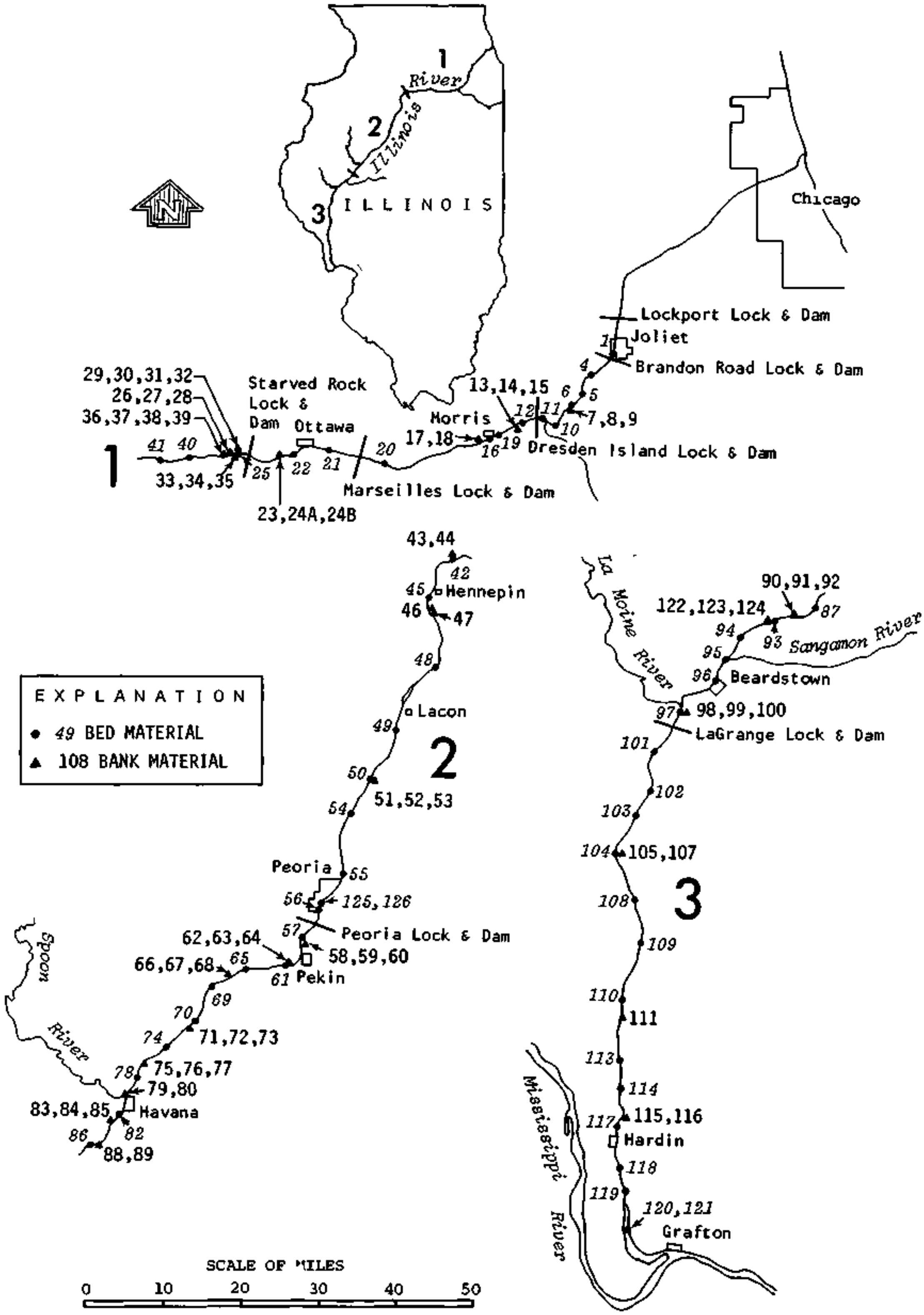


Figure 8. Locations where bed and bank material samples were collected

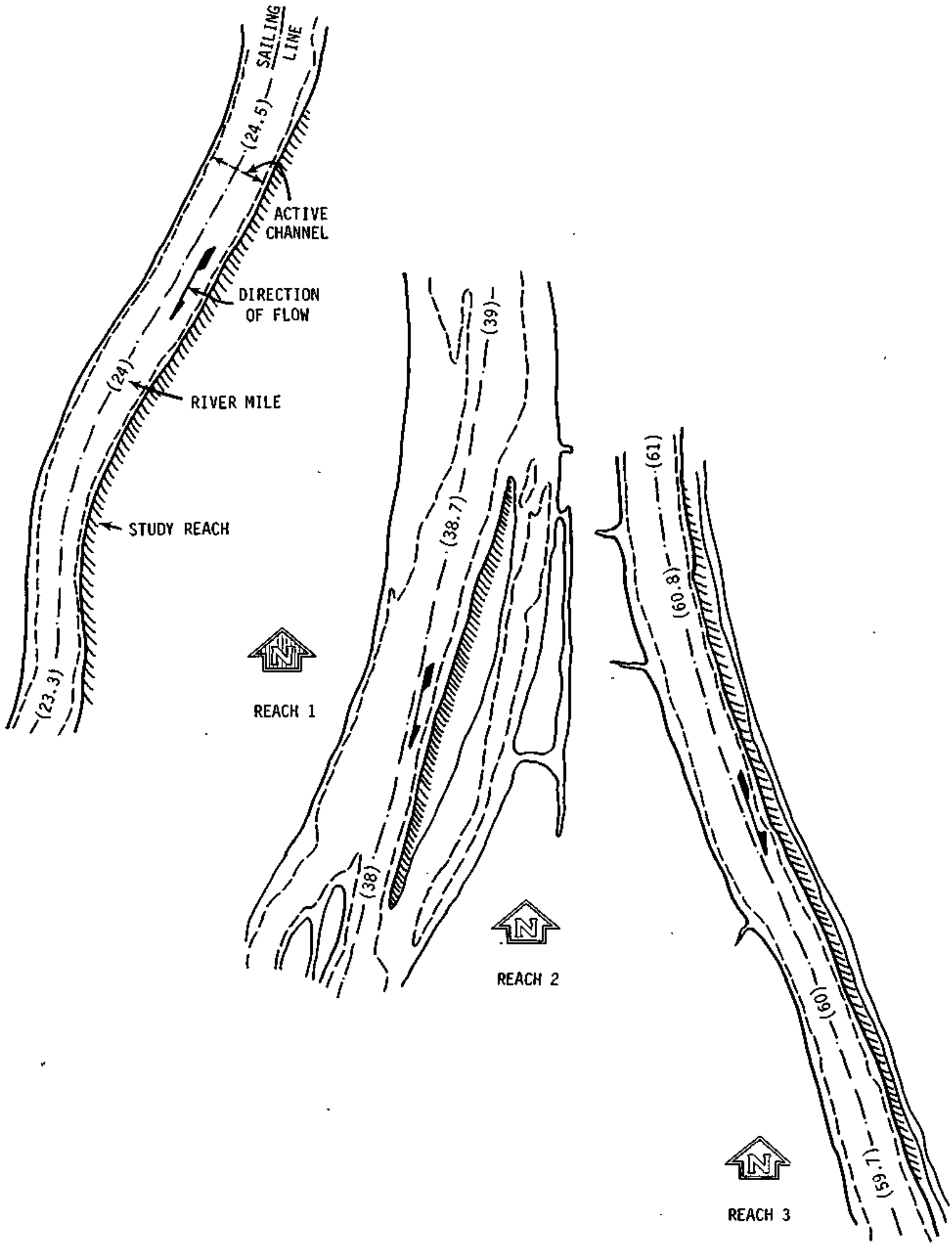
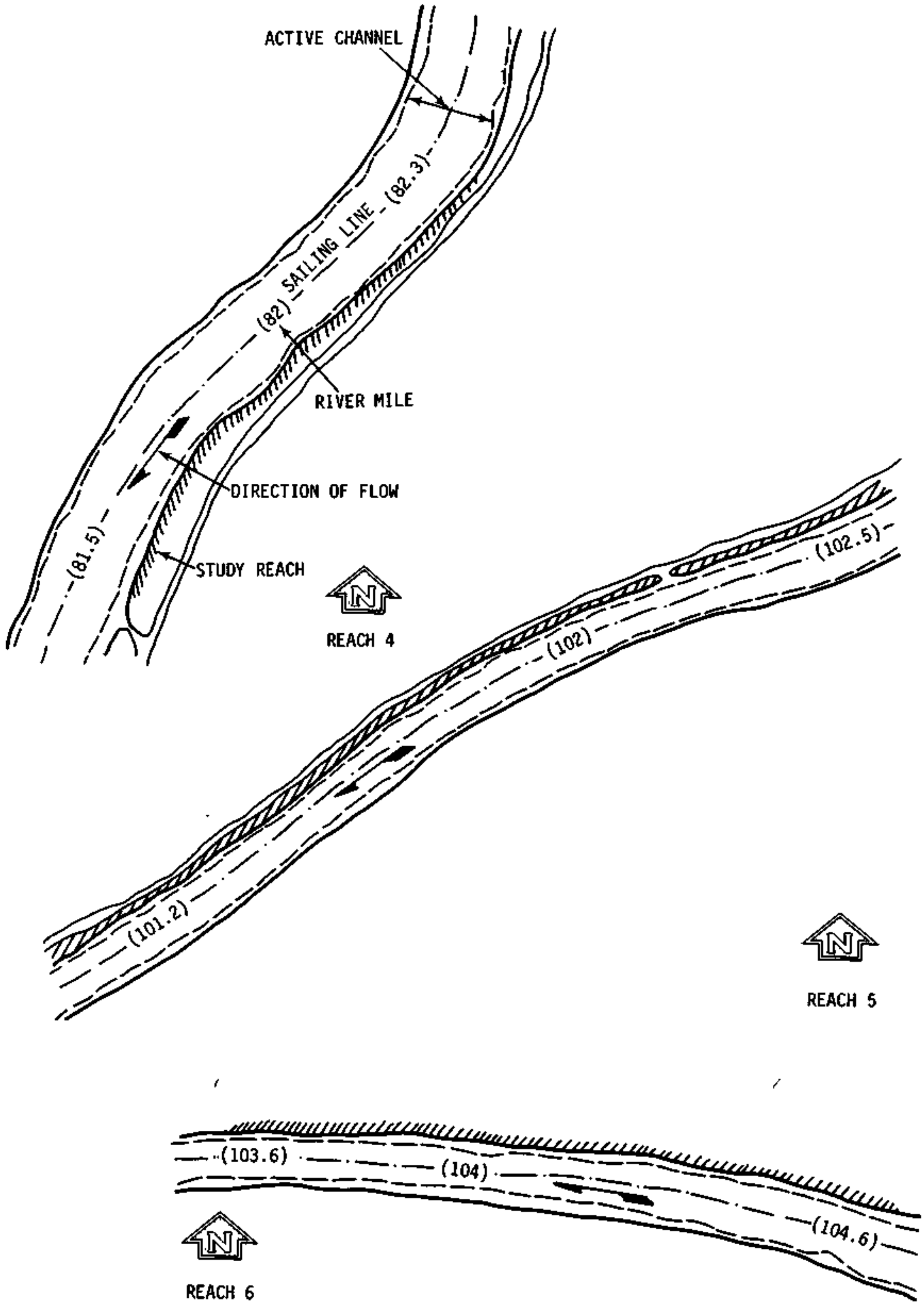


Figure 9. Severely eroded banks at 20 reaches along the Illinois River



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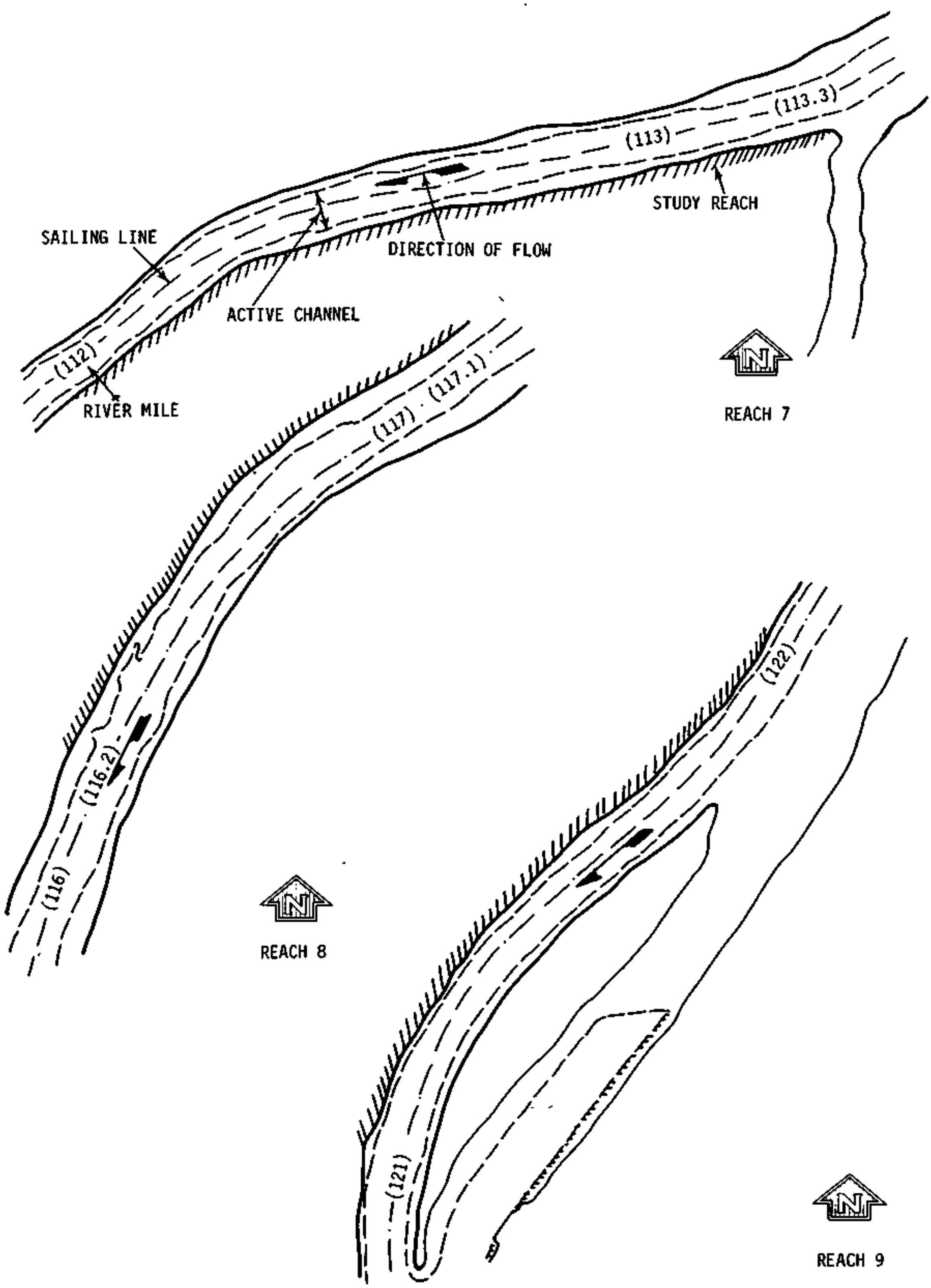

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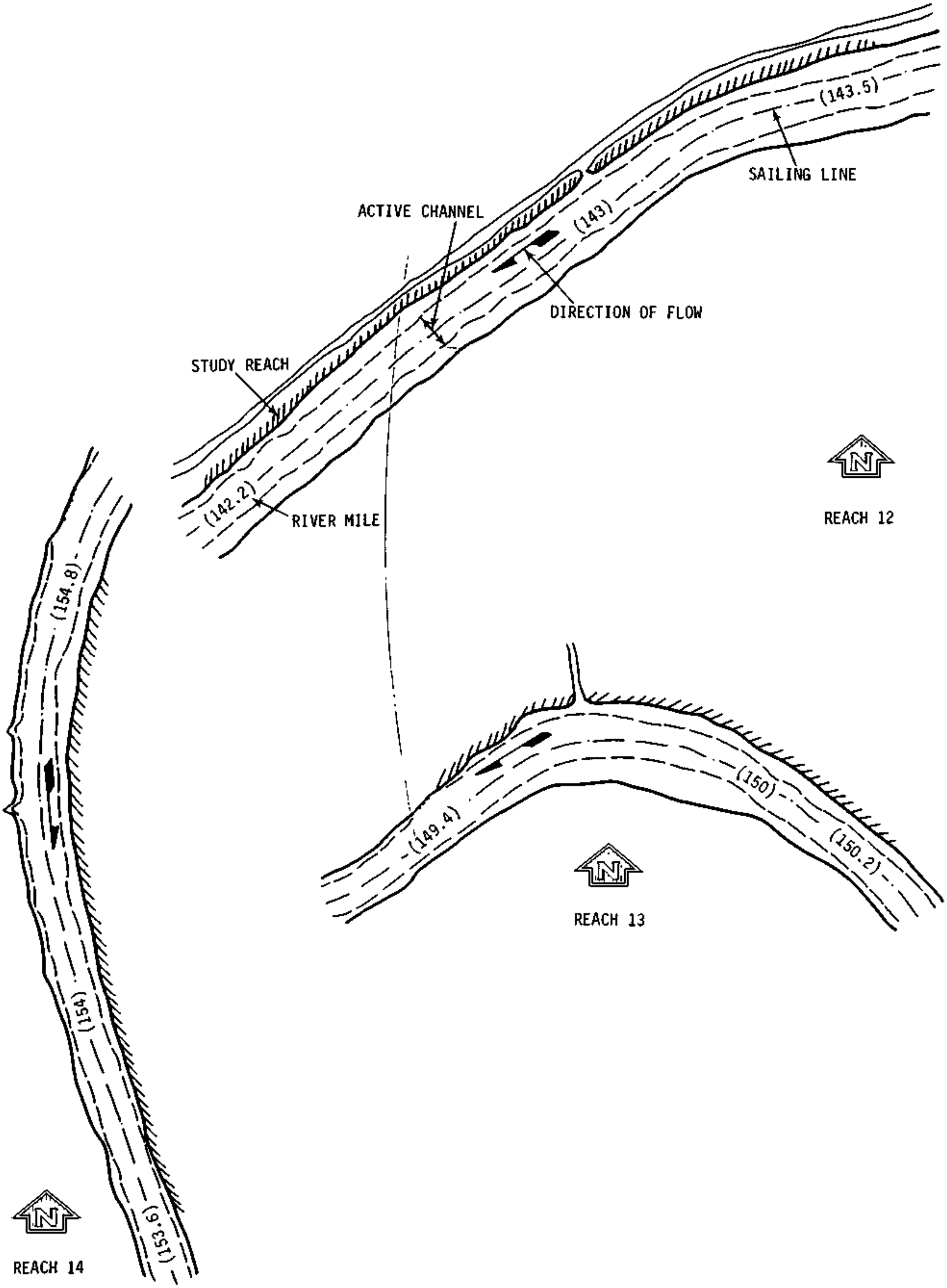

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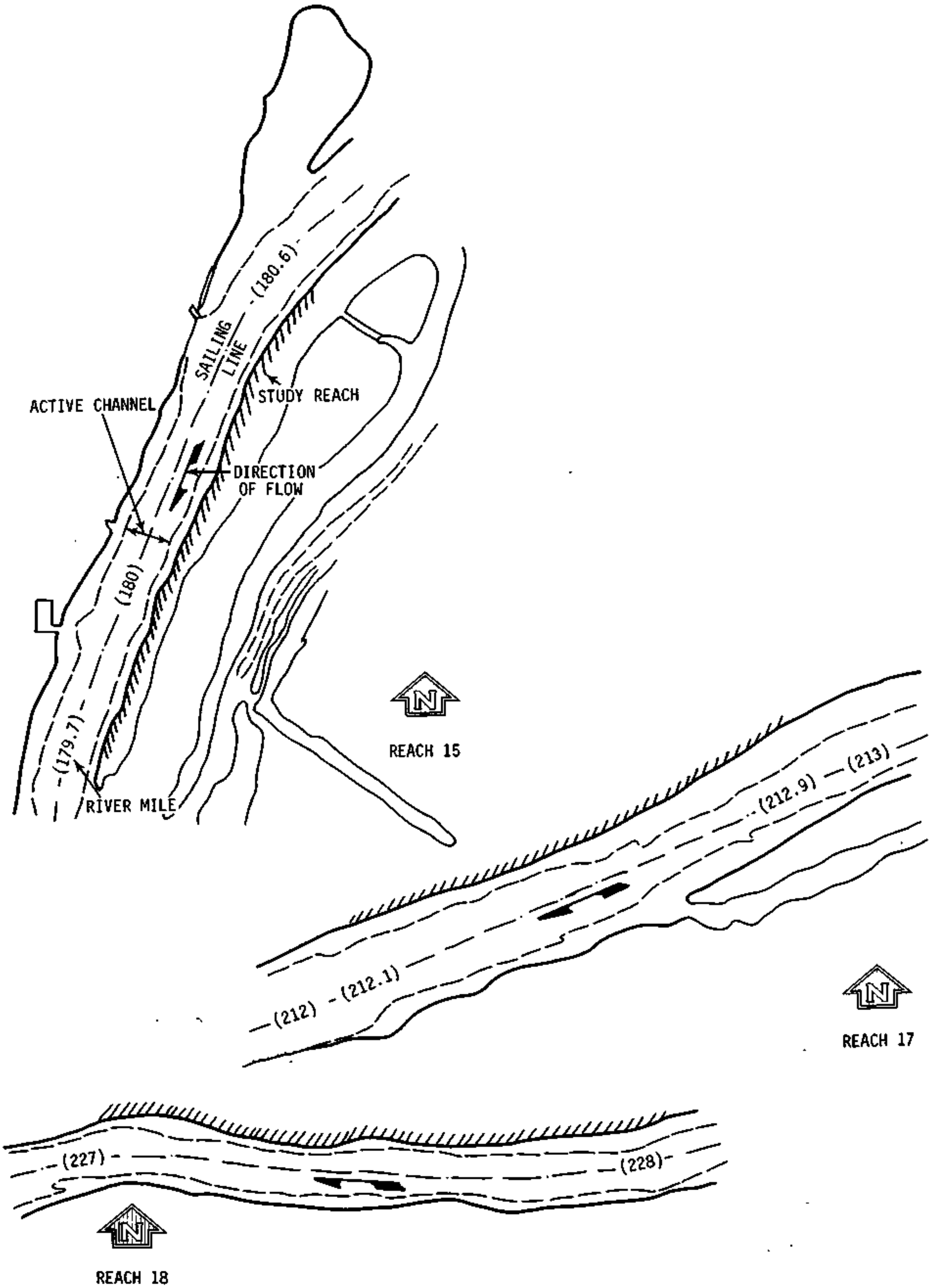

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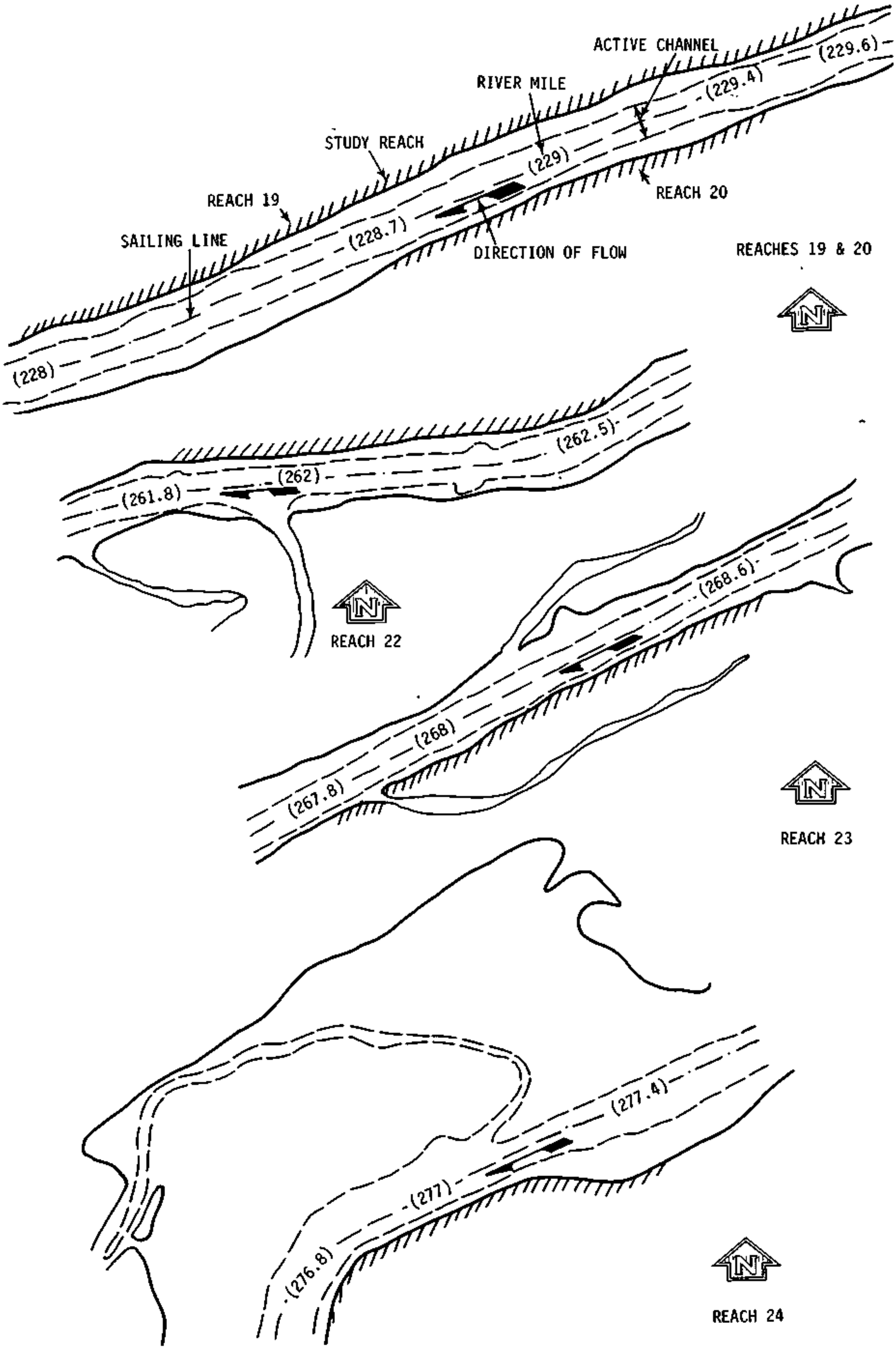

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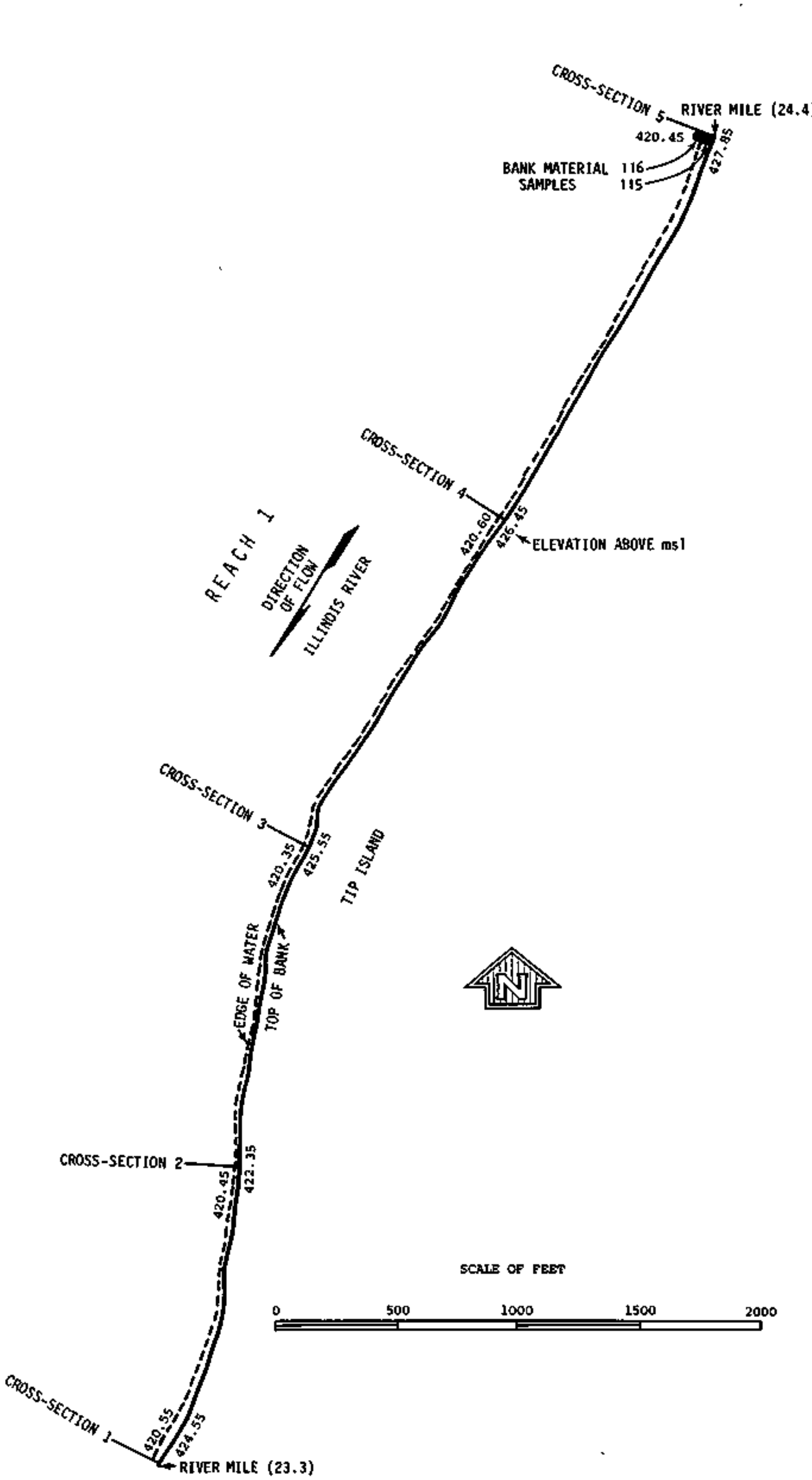
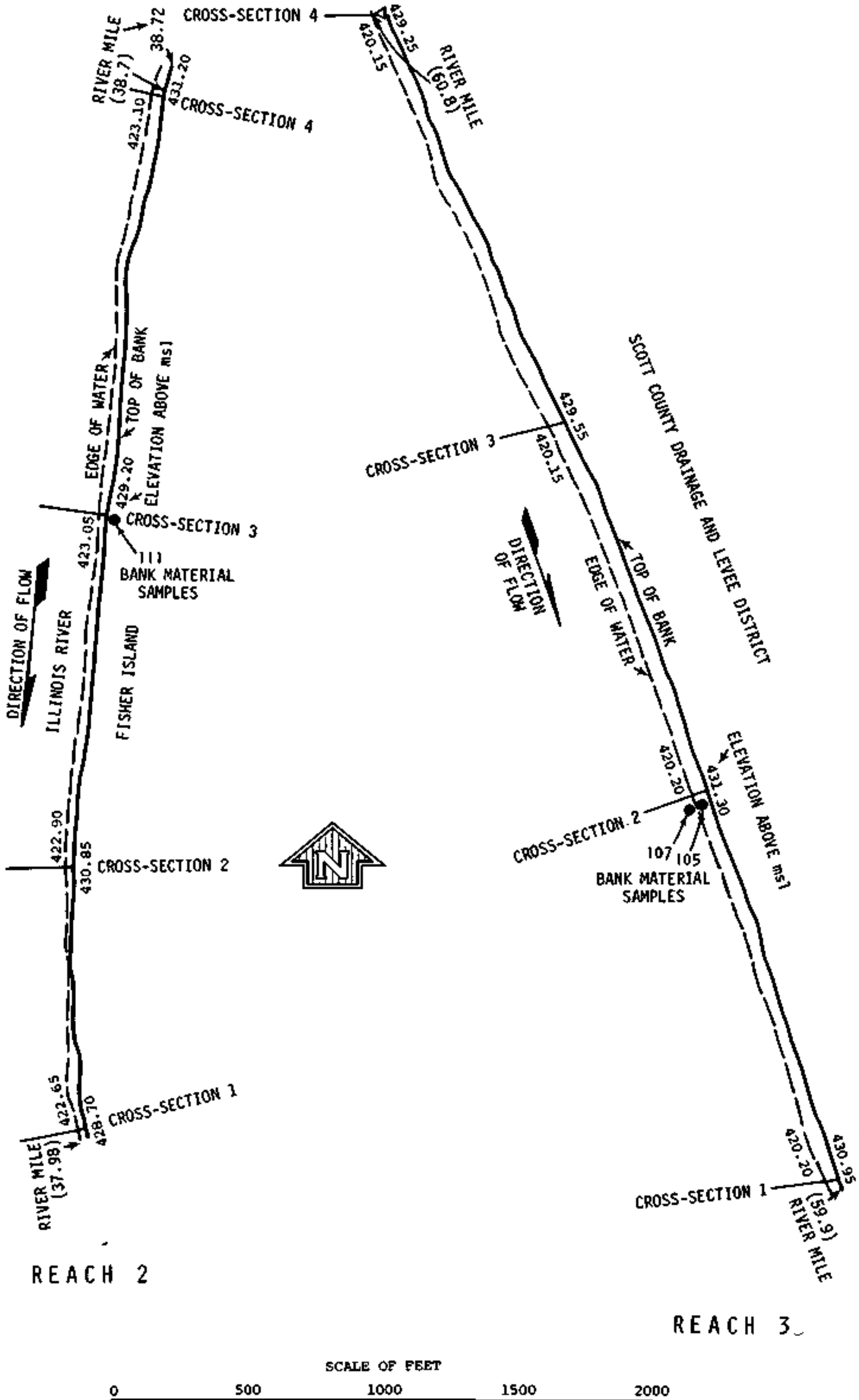


Figure 10. Plan views of 20 reaches along the Illinois River



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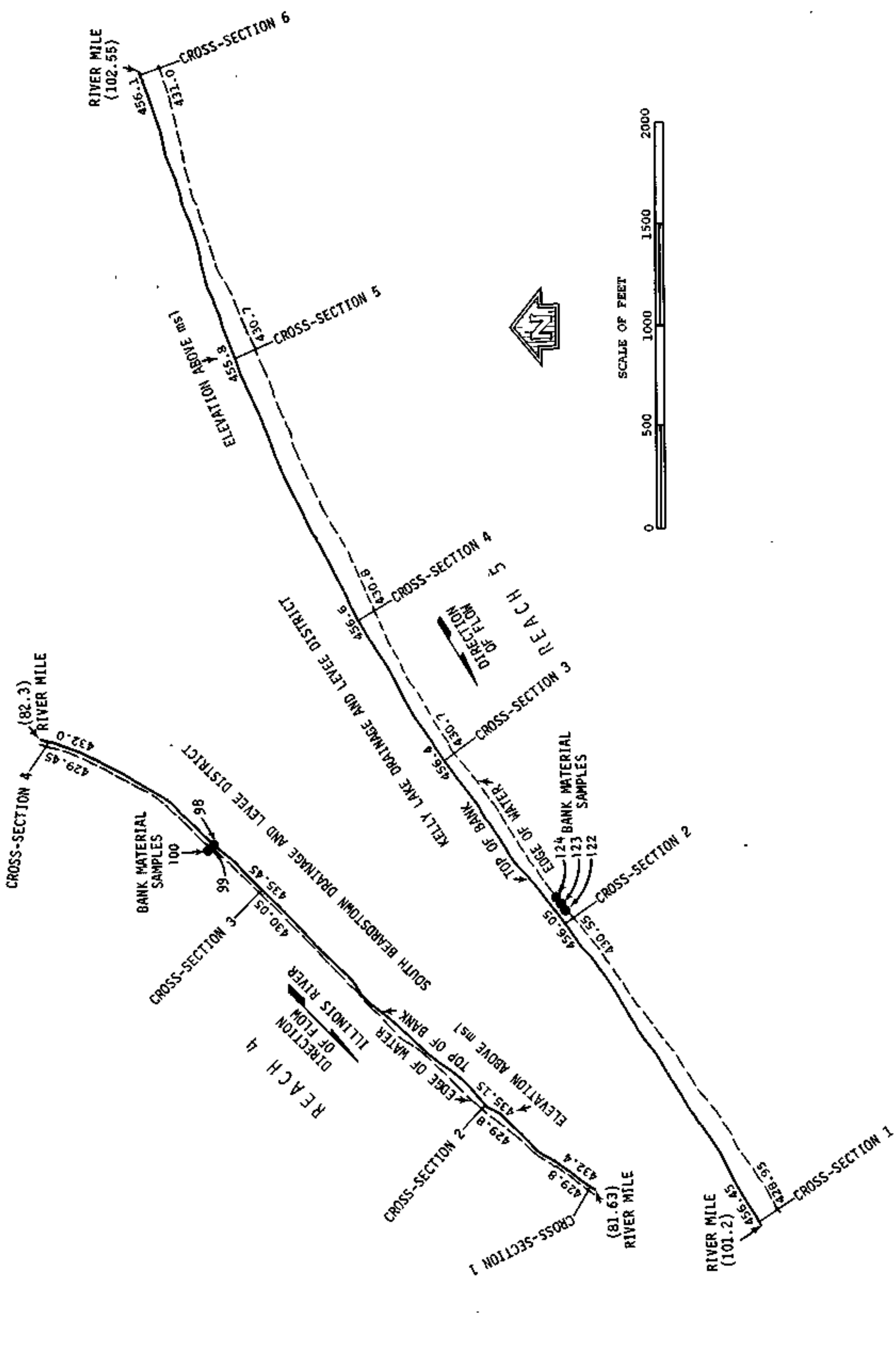

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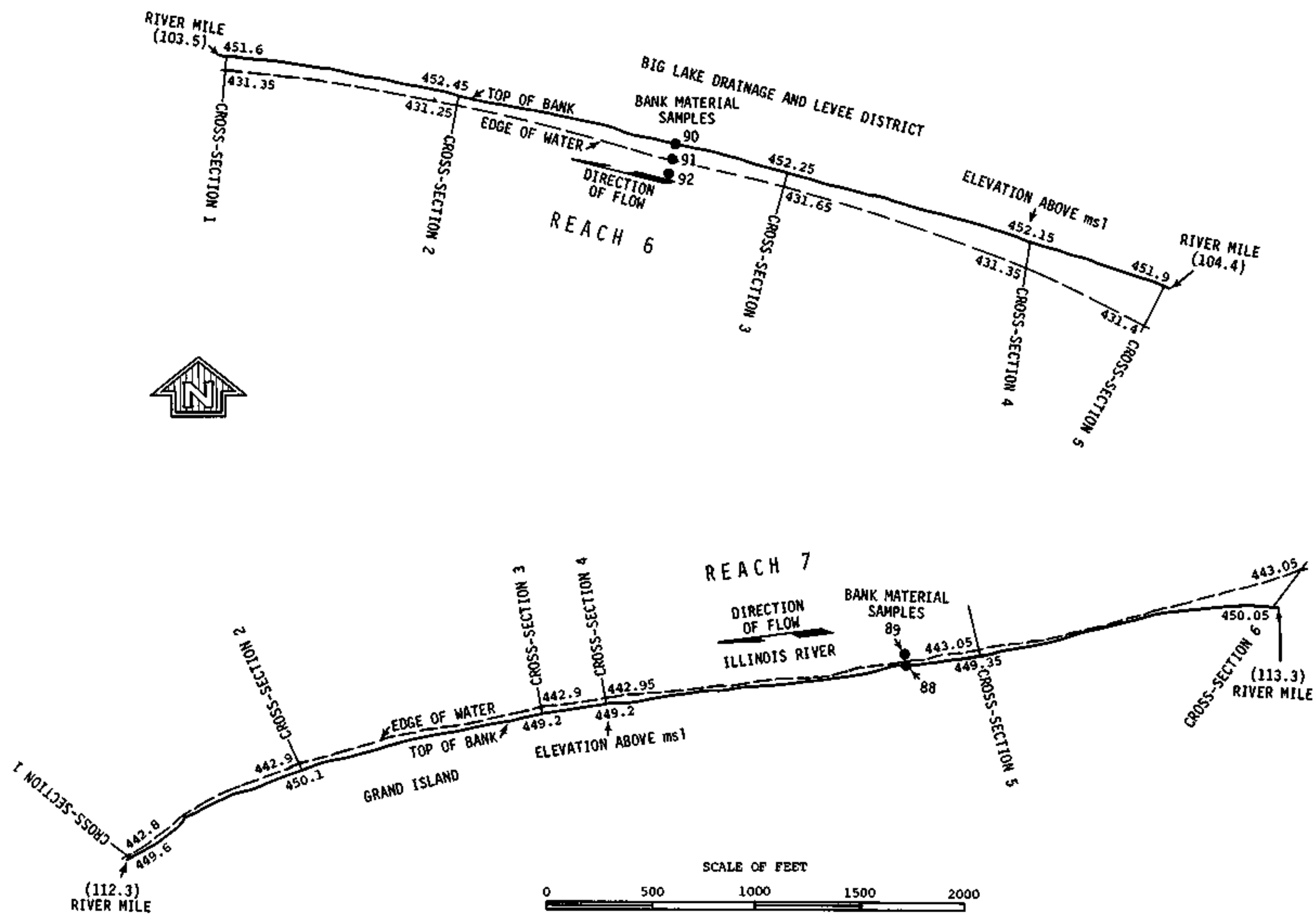

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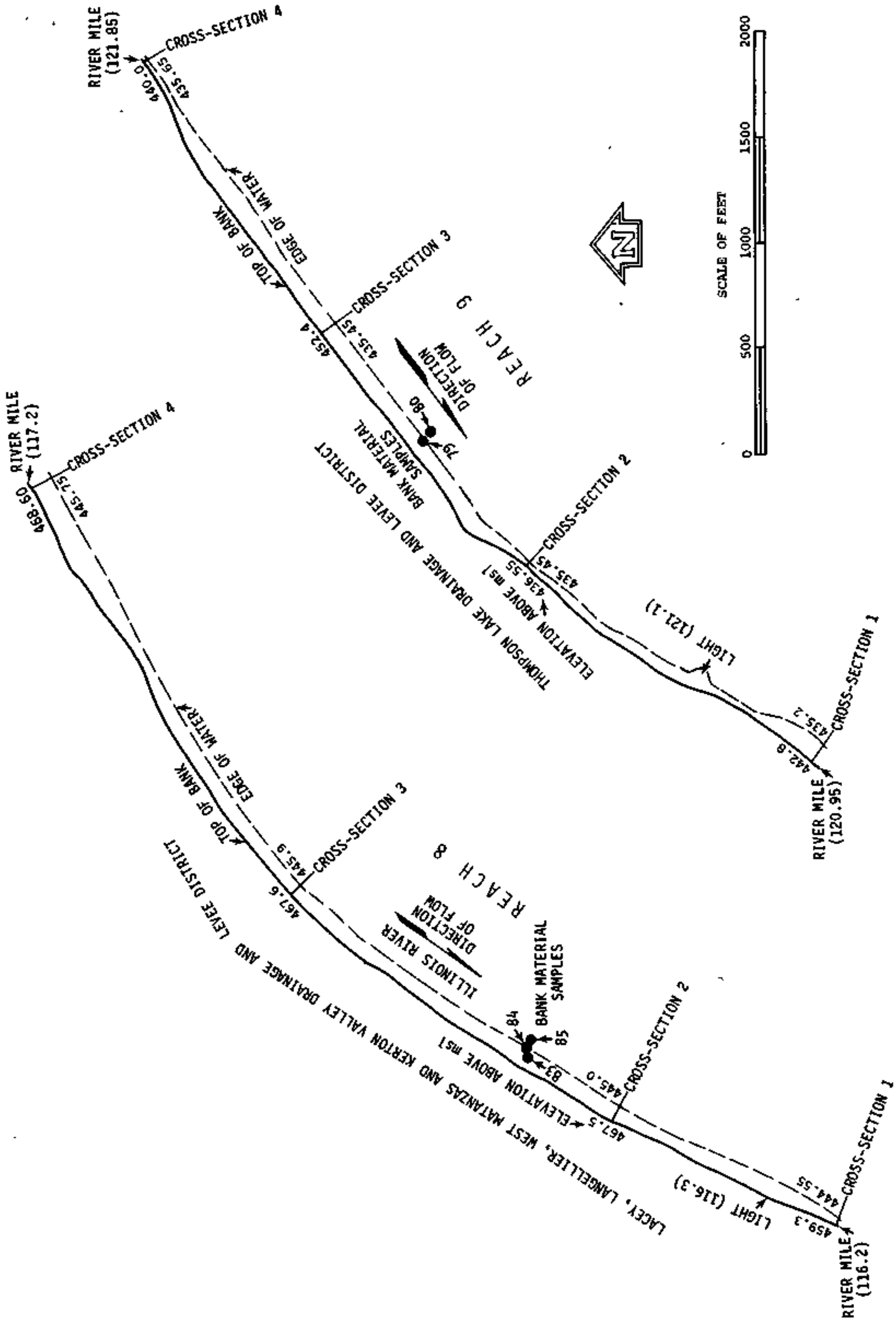

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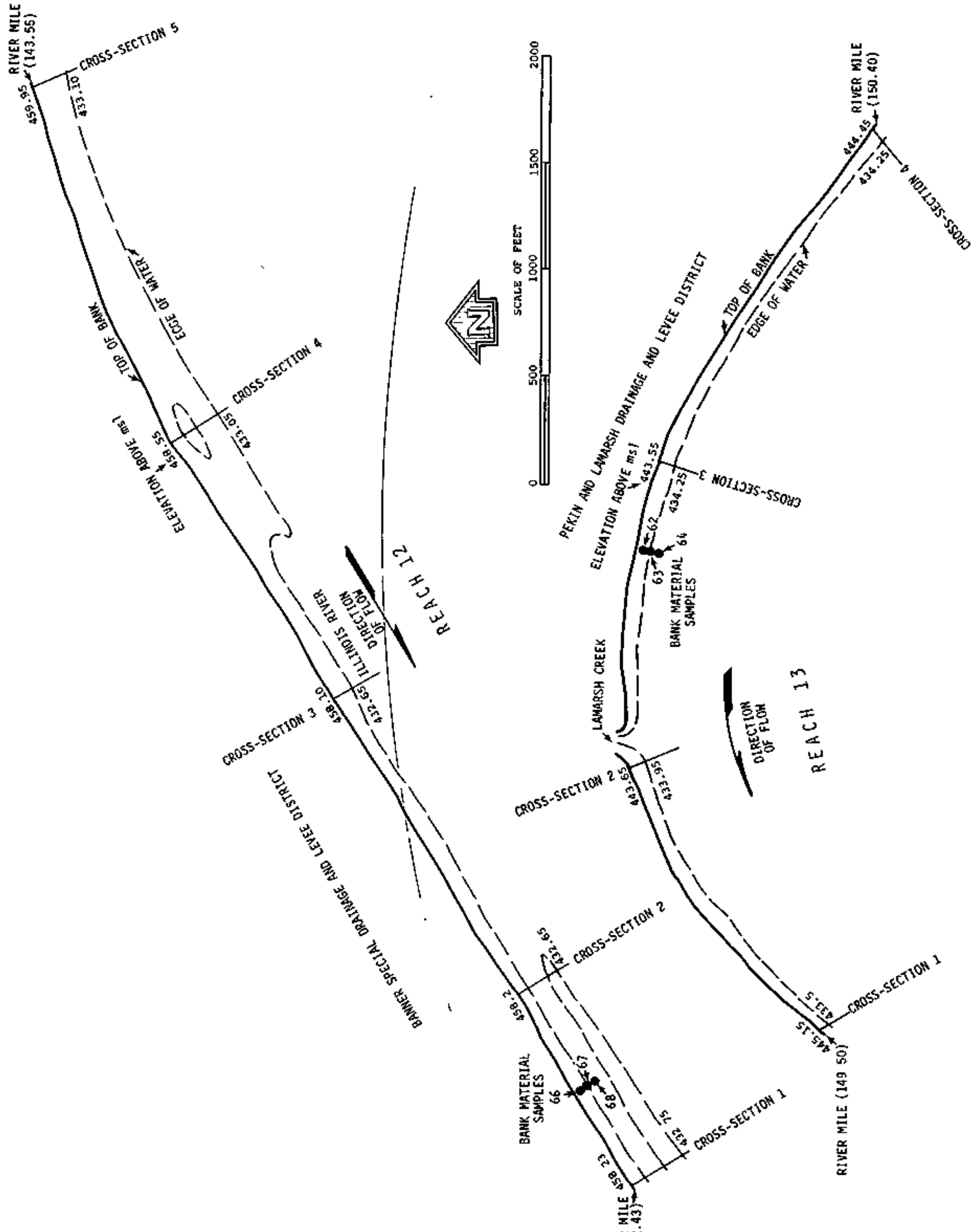

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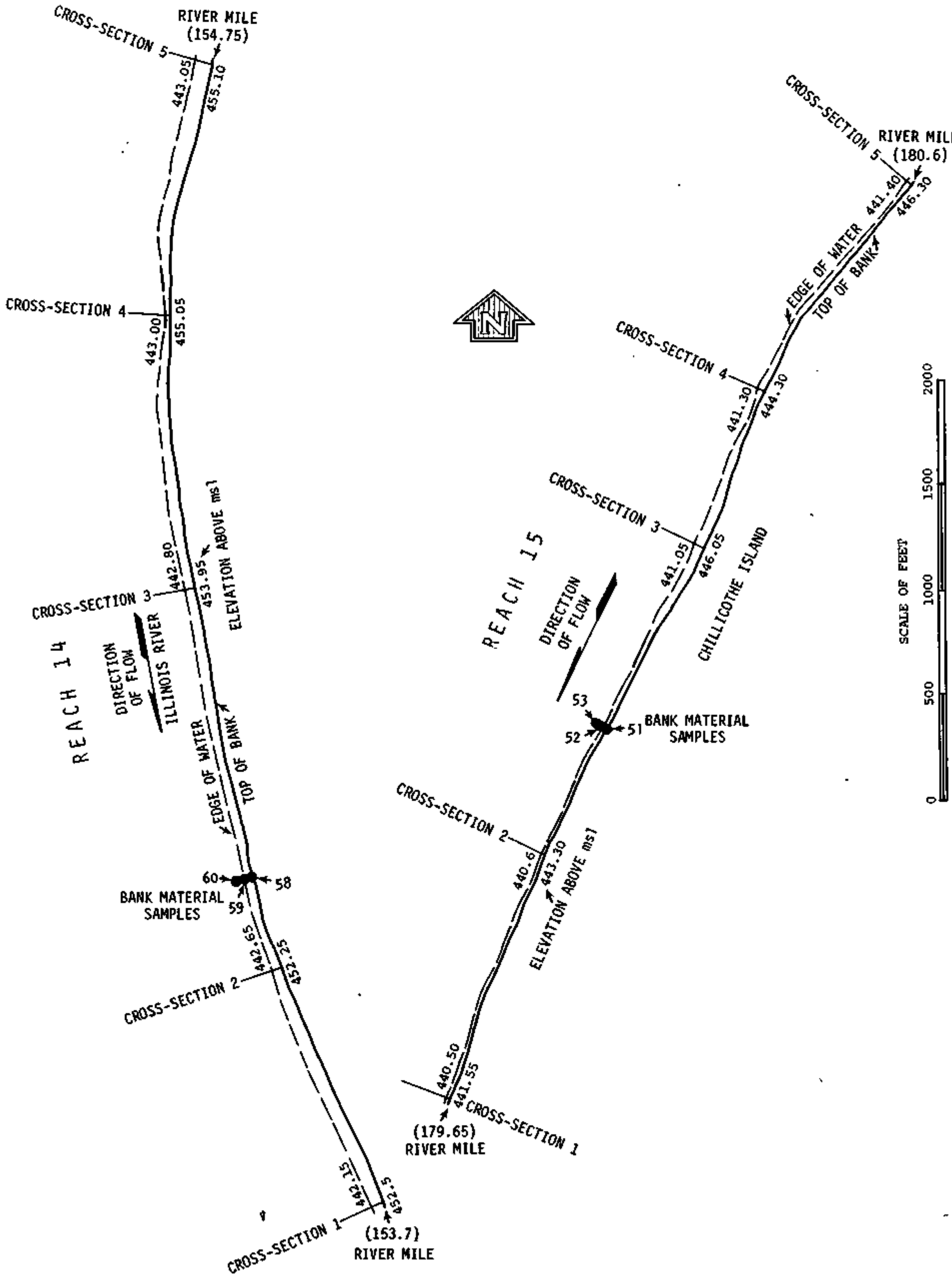

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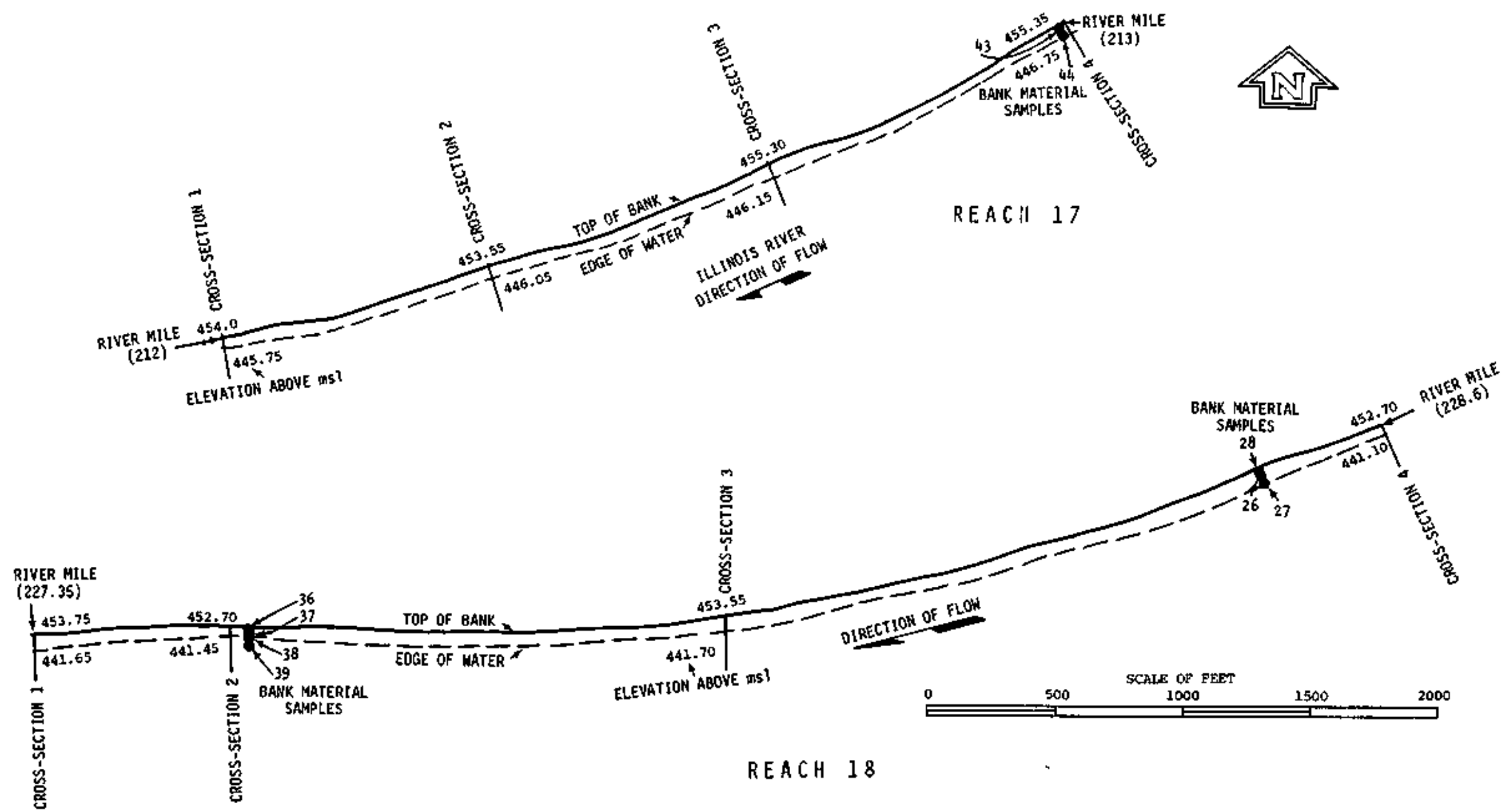

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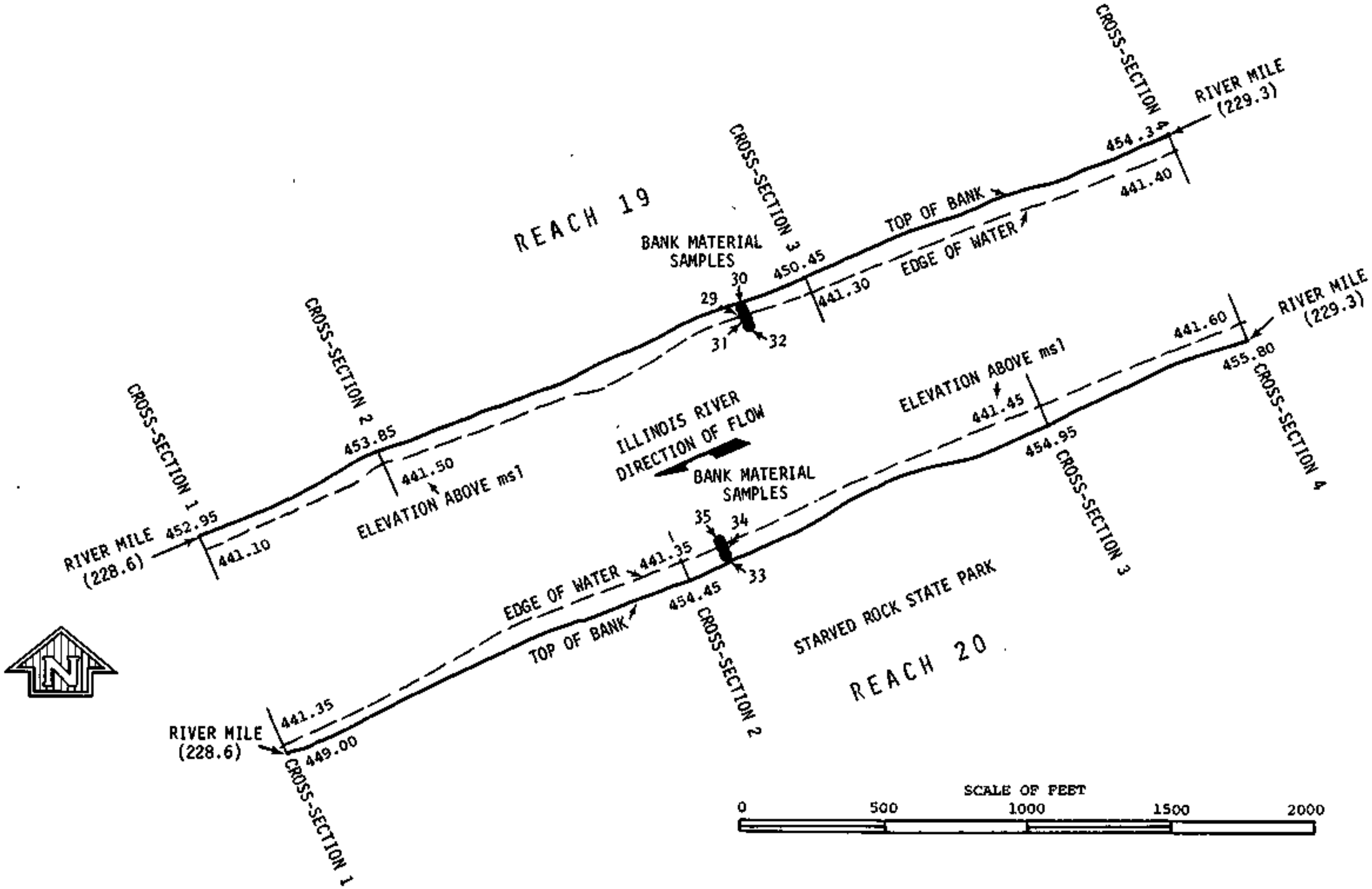

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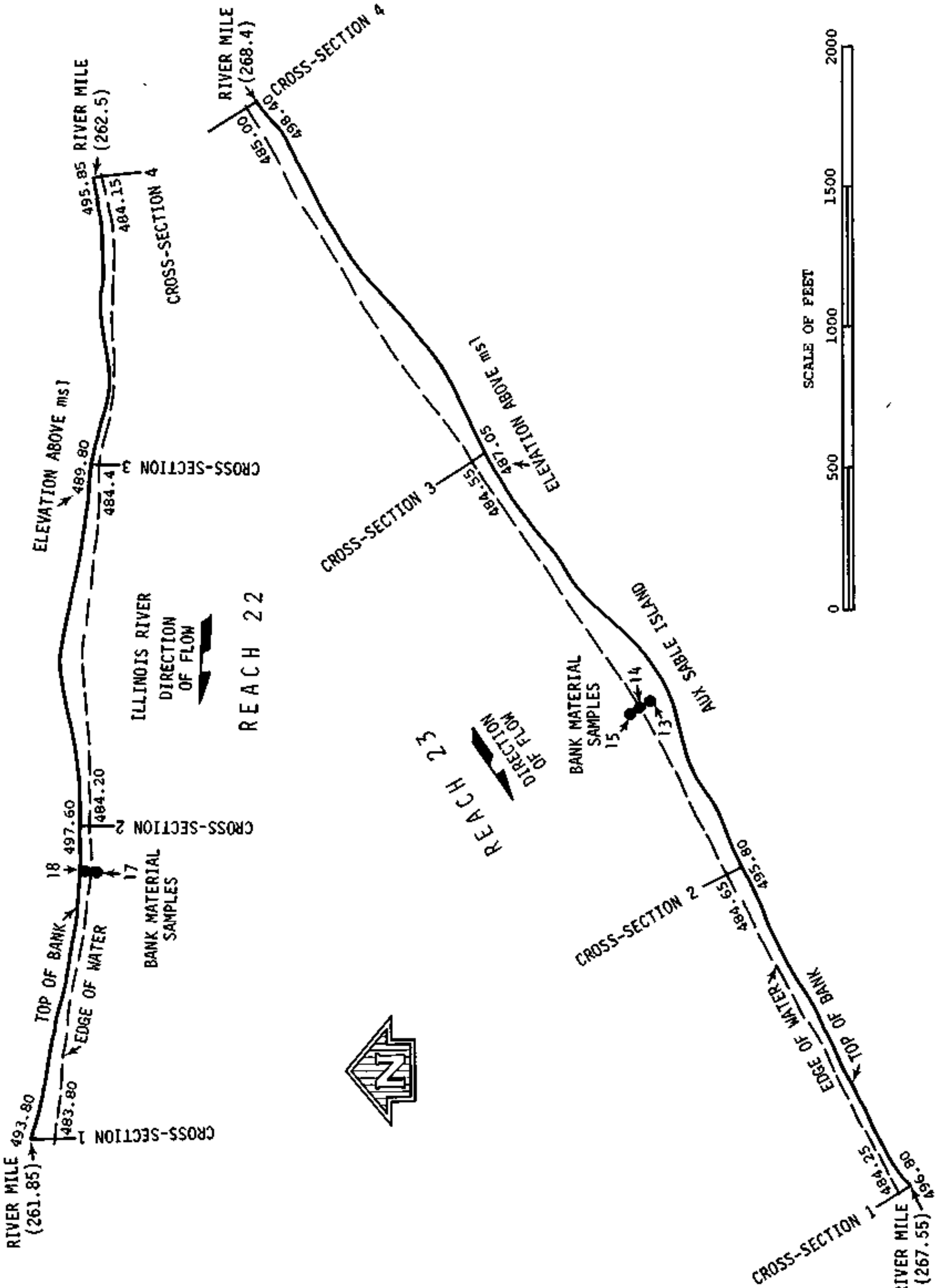

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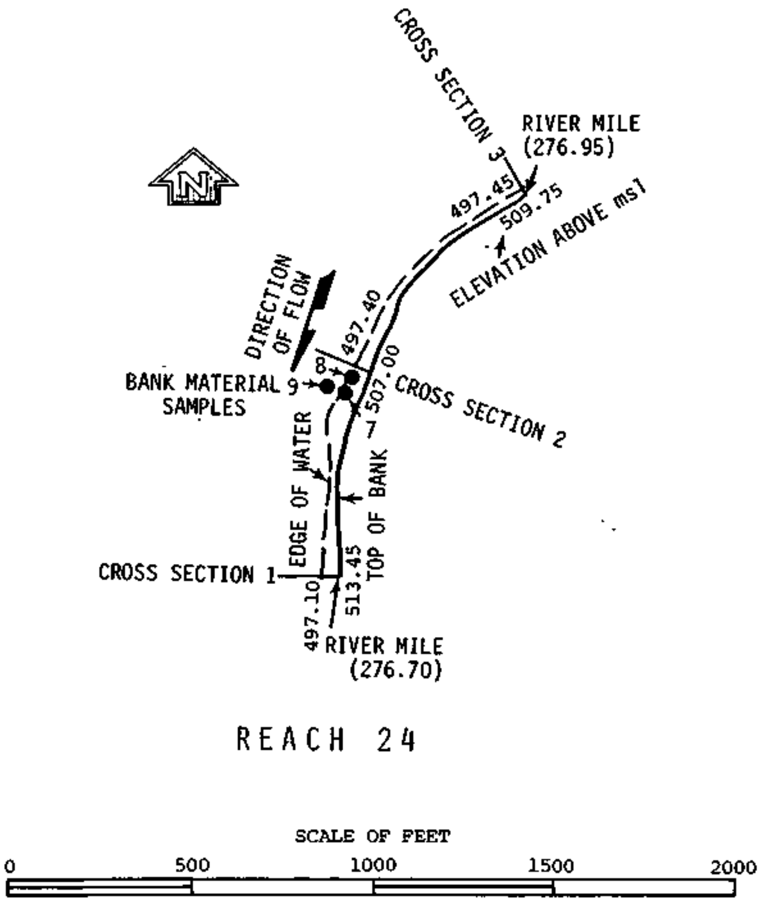


Figure 10. Concluded

The geometric parameters described above are summarized in Table 1.

The first and second columns in table 1 show the reach number and the extent of each reach of the river selected for a detailed survey. The river miles for some of the reaches were repeated for two reasons. Reach 1 extends through two river bends of opposite curvature. The Reach 1 river miles were repeated in the second column so that the numerical values of the radius of the curvatures corresponding to those two bends could be shown in the fourth column. For all other reach and river mile repetitions the variable is the bank slope present in those reaches. The bank slope was not constant over the entire length of some of the reaches. The slope variability within each reach - is shown in the last column.

The fourth column in table 1 gives the radius of curvature, R, of the bends, and the fifth column shows the corresponding deflection angle, A, in degrees. Column six shows the average bankfull width, W, of the river for each reach. The ratio of R/W is shown in the seventh column. The value of R/W varies anywhere from less than 1 to 31.7.

The reaches described here were selected to be representative bank erosion areas along the Illinois River. There are numerous other segments of the river where bank erosion is severe. This was not meant to be an all-inclusive investigation showing all the bank erosion areas with detailed analysis. It is the contention of the researchers that an analysis of these selected reaches should shed some light on the causative factors that contribute to bank erosion along the Illinois River.

Table 1. Characteristics of Twenty Selected Reaches along the Illinois River

Reach	*River mile from - to*	*Subreach shape*	*Radius of curvature, R (feet)*	*Deflection angle, A (degrees)*	*Average top width at bankfull stage, W (feet)*	*R/W*	*Bank slope*
1	23.3 - 24.4	Curved	4,700	41.0	700	6.7	1:7
1	23.3 - 24.4	Curved	3,100	37.5	500	6.2	1:7
2	37.98- 38.72	Straight			900		1:5.5
3	59.9 - 60.8	Straight			600		1:6.5
4	81.63- 82.3	Curved	3,200	67.0	800	4.0	1:7.5
4	81.63- 82.3	Curved	4,800	41.0	800	6.0	1:4
5	101.2 -102.55	Curved	13,000	22.5	420	31.0	1:2.1
5	101.2 -102.55	Curved	13,000	22.5	420	31.0	1:53
6	103.5 -104.4	Straight			500		1:3.5
6	103.5 -104.4	Straight			500		1:9
6	103.5 -104.4	Straight			500		1:18
7	112.3 -113.3	Curved	11,150	51.5	500	22.3	1:10
8	116.2 -117.2	Curved	7,500	44.0	650	11.5	1:6
9	120.95-121.85	Curved	4,900	55.5	500	9.8	1:7
12	142.43-143.55	Curved	19,000	23.0	600	31.7	1:4
12	142.43-143.55	Curved	19,000	23.0	600	31.7	1:26
13	149.5 -150.4	Curved	2,500	97.0	600	4.2	1:7.5
14	153.7 -154.75	Curved	8,400	43.0	480	17.5	1:5
14	153.7 -154.75	Curved	8,400	43.0	480	17.5	1:100
15	179.65-180.6	Straight			700		1:12.5
17	212.0 -213.0	Straight			900		1:7
18	227.35-228.6	Straight			650		1:6.5
19	228.6 -229.3	Straight			650		1:8
20	228.6 -229.3	Straight			650		1:8
22	261.85-262.5	Curved	12,000	30.5	500	24.0	1:9
22	261.85-262.5	Curved	12,000	30.5	500	24.0	1:3.5
23	267.55-268.4	Straight			500		1:7.5
23	267.55-268.4	Straight			500		1:7.5
23	267.55-268.4	Straight			500		1:6
23	267.55-268.4	Straight			500		1:2.5
24	276.7 -276.95	Curved	1,400	50.0	2400	0.6	1:5

Bank Slope

The bank slope is an important parameter in the stability analysis of any river bank. The surveying crew determined the bank slope at each selected reach for a minimum of three to a maximum of six sections. The data were plotted individually for each reach taking the bed of the river as the datum. The plot shows the lateral displacements of the bank with each foot of drop from the top of the bank. Figures 11 and 12 show two typical plots that were developed for Reaches 3 and 14, respectively. Data from Reaches 1, 2, 3, 4, 7, 8, 9, 13, 15, 17, 18, 19, 20, and 24 indicated that a single average bank slope determined from plots similar to figure 11 ,can be used as the representative bank slope for each one of these reaches. However, data analyzed from Reaches 5, 6, 12, 14, 22, and 23 indicated that either two distinct slopes exist in the same reach, similar to the one shown in figure 12, or different parts of the same reach have different slopes. The bank slopes for all the reaches vary anywhere from 1:3.5 to 1:9. The first number stands for the vertical drop, the second for the horizontal displacement.

Bed Slope

Figure 3 shows the profile of the thalweg for the length of the Illinois River. This figure shows the

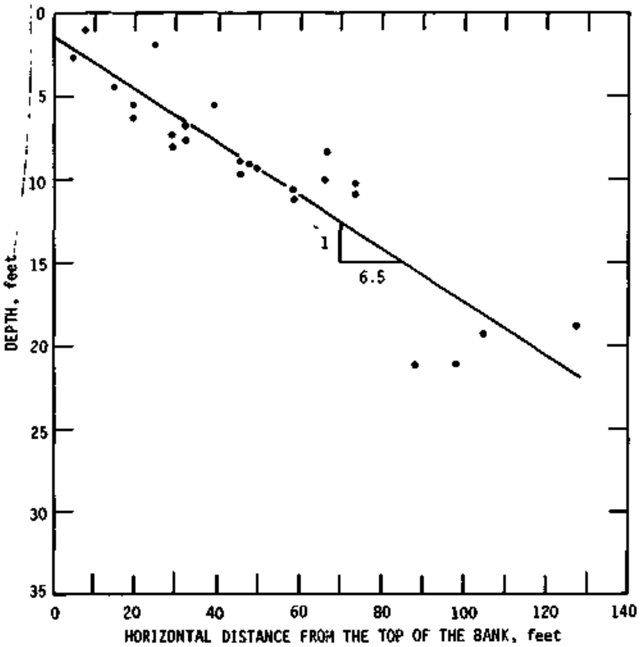


Figure 11. Typical plot showing the bank slope for Reach 3

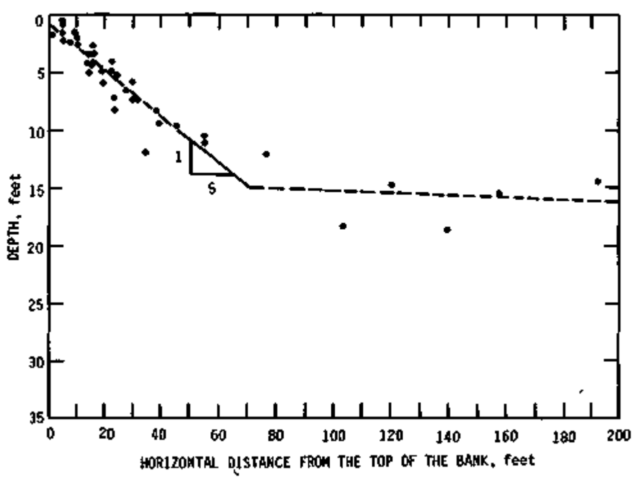


Figure 12. Typical plot showing the bank slope for Reach 14

elevation of the lowest points along the river; however, it is quite apparent that no uniform bed slope exists for the entire river length. The U. S. Army Corps of Engineers supplied a set of computer printouts showing the sounding data at various locations along the river. These sounding data were plotted and an average bed elevation was determined for each location. The average bed elevations were used to develop plots showing the bed elevation versus distances for each pool. Figure 13 shows such a plot for two segments of the Illinois River. Similar plots were also developed for other segments of the river covering all of the reaches under investigation.

One of the hydraulic parameters needed to perform a stability analysis of the river bank, or to find the erosion potential of the bed is the hydraulic gradient of the river. Since data related to the water surface profiles at each reach for various discharges are not available, the average bed slope determined for each reach (similar to figure 13) was used as the hydraulic gradient of the river.

Bank Material Sizes

Altogether, 67 bank material samples were collected from different locations (figure 8) along the Illinois River. The exact locations for most of these bank material samples are shown in figure 10. The rest of the bank material samples were collected from other reaches that were not selected for further investigation.

All of the samples were analyzed by both sieve and hydrometer techniques to determine the particle size distribution. Plots were developed showing the percent by weight versus the particle size for each one of the samples.

Table 2 shows geometric parameters that are used in describing and identifying the particle size and distribution. The d_{50} and d_{95} indicate the equivalent particle diameters for which 50 percent and 95 percent, respectively, of the particles are finer in diameter. The standard deviation, a, is defined in equation 1.

$$\sigma = \frac{1}{2} [(d_{84.1}/d_{50}) + (d_{50}/d_{15.9})] \quad (1)$$

Here $d_{84.1}$ and $d_{15.9}$ indicate the equivalent particle diameters for which 84.1 percent and 15.9 percent, respectively, of the particles are finer in diameter.

The other parameter shown in table 2 is the uniformity coefficient, U, and it is defined by the ratio given in equation 2.

$$U = d_{60}/d_{10} \quad (2)$$

The numerical values of the standard deviation and the uniformity coefficient indicate a measure of the gradation of the particles. Higher values of σ and U will indicate a very well graded material, whereas a lower value of σ and U will demonstrate

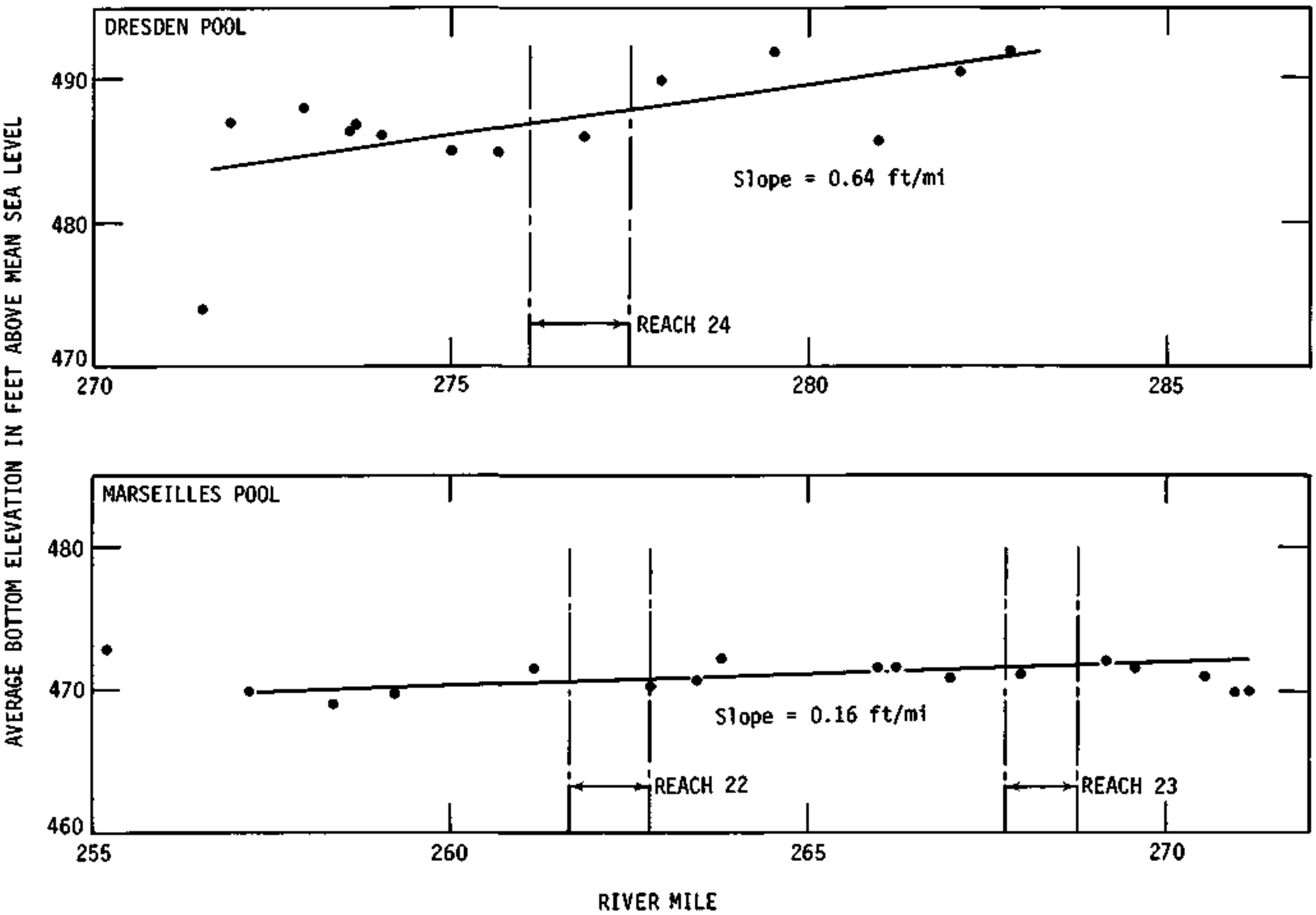


Figure 13. Bed slope of the Illinois River at two different locations

the uniformity of these particles. The last column in table 2 gives the general nature of the bank materials.

In order to determine if the bank material particle sizes for different samples are similar, frequency distribution analyses of the d_{50} and d_{95} sizes were made. Figures 14 and 15 show the frequency distribution for d_{50} and d_{95} sizes, respectively. From figure 14 it is obvious that 63 of the 67 bank material samples have their median diameter smaller than 2 mm. The middle insert in figure 14 shows that out of these 63 samples, 38 have d_{50} values less than 0.1 mm. The top insert in figure 14 shows that 15 of the samples have d_{50} sizes within the range of 0.01 to 0.02 mm indicating that these materials are in the clay to silty ranges.

As shown in figure 15, 60 out of 66 samples have d_{95} values less than 11 mm. The middle insert in figure 15 indicates that 53 out of 60 samples have a d_{95} value of less than 1 mm. The top insert shows that 20 of the samples have d_{95} values in the range of 0.2 to 0.3 mm indicating that they are basically sandy materials.

Figure 16 shows the frequency distribution for σ and U. Although no definitive statement can be made as to the uniformity characteristics of these materials, they are basically well-graded materials, though some of the samples consist of uniform materials for almost 60 to 70 percent of their volumes.

Data analyzed for the bank materials definitely indicate that wherever serious bank erosion exists on the Illinois River, the bank materials are usually composed of fine-grained sands to silts having very little resistance against relatively high flow velocity and the onslaught of waves generated either by wind or by waterway traffic. This may explain to some extent why severe bank erosion exists on the Illinois River waterway wherever the bank lacks any natural or artificial protection.

Table 2. Particle Size Characteristics of Bank Material Samples

Sample number	d_{50} *(mm)*	d_{95} *(mm)*	σ	*U*	*Remarks*
Reach 1, river mile 24 4					
116	0.013	0.13			Clayey silt
115	0.014	0.065			Clayey silt
Reach 2, river mile 38.4					
111	0.021	0.19			Silt
Reach 3, river mile 60.2					
107	0.04	0.175	5.88		Sandy silt
105	0.063	0.19	4.74	30.40	Sandy silt
Reach 4, river mile 82.1					
100	0.012	0.20			Clayey silt
99	0.15	0.24	1.59	2.83	Fine sand
98	0.17	0.32	4.60	23-75	Fine-to-medium sand
Reach 5, river miles 101.0 to 102.0					
124	0.018	0.51			Sandy clayey silt
123	0.017	0.26			Sandy clayey silt
122	0.014	0.27			Sandy clayey silt
Reach 6, river mile 104.0					
92	0.01	0.30			Clayey silt
91	0.0084	0.065			Clayey silt
90	0.0034	0.042			Silty clay
Reach 7, river mile 113.0					
89	0.016	0.17			Silt
88	0.027	0.20			Silt
Reach 8, river mile 116.5					
85	0.52	10.0	6.23	5.0	Fine-to-coarse sand
84	0.27	0.44	1.75	3.29	Fine sand
83	0.008	0.19			Silty clay
Reach 9, river mile 121.4					
80	0.75	13.0	5.14	4.31	Fine-to-coarse sand
79	2.40	36.0	7.07	16.07	Fine-to-coarse sand and gravel
Reach 10, river mile 126.0					
77	0.019	1.0			Silt
76	0.0115	0.24			Silt
75	0.034	0.25			Silt
Reach 11, river mile 134.0					
73	0.24	0.55	1.50	1.80	Medium-to-fine sand
72	0.23	0.70	1.87	3.43	Medium-to-fine sand
71	0.0074	0.075			Clayey silt
Reach 12!, river mile 142.5					
68	0.035	0.12	2.63		Mottled gray silt
67	0.0073	0.14			Clayey silt
66	0.013	0.49			Clayey silt
Reach 13, river mile 150.0					
64	0.0073	0.26			Clayey silt
63	0.17	0.42	15.14	115.0	Silty fine-to-coarse sand
62	0.032	0.40	17.83		Sandy silt
Reach 14, river mile 154.0					
60	0.14	0.24	2.98	15.0	Fine-to-medium sand
59	0.04	0.20	8.04		Sandy silt
58	0.05	0.15	6.10		Sandy silt

Sample number	d_{50} *(mm)*	d_{95} *(mm)*	σ	*U*	*Remarks*
Reach 15i, river mile 180.0					
53	0.26	5.0	4.48	40.0	Fine-to-coarse sand
52	0.19	0.38	10.26	80.0	Silty fine-to-medium sand
51	0.017	0.24			Clayey silt
Reach 16i, river miles 204.0-204.5					
47	0.005	0.27			Clayey silt
46	0.0033	0.20			Clayey silt
Reach 17, river mile 213.0					
44	0.17	0.26	1.11	2.25	Fine sand
43	0.042	0.23	12.40		Sandy silt
Reach 18, river mile 227.5					
39	0.29	0.94	2.56	34.0	Fine-to-coarse sand
38	0.08	0.27	11.65	105.0	Silty fine sand
37	0.12	0.27	10.19	80.0	Silty fine sand
36	0.011	0.13			Clayey silt
Reach 18, river mile 228.5					
28	0.024	0.24	12.77		Sandy silt
27	0.23	0.40	1.57	3.0	Fine-to-medium sand
26	0.12	0.35	11.08	62.96	Silty fine-to-medium sand
Reach 19, river mile 229.0					
32	0.27	0.45	4.56	25.45	Fine-to-medium sand
31	0.06	0.24	11.58		Sandy silt
30	0.07	0.28	10.46		Fine-to-medium sand
29	0.20	0.39	1.29	1.4	Fine sand
Reach 20, river mile 228.9					
35	0.08	8.0	25.0		Sandy silt
34	0.29	0.57	1.67	3.16	Medium-to-fine sand
33	0.39	0.53	1.44	2.15	Medium-to-fine sand
Reach 21, river mile 235.6					
24B	0.23	1.10	29.82		Silty fine-to-coarse sand
24A	0.18	0.5	20.94		Silty fine-to-coarse sand
23	0.40	15.0	1.80	1.96	Fine-to-coarse sand
Reach 22, river mile 262.0					
18	0.02	0.18			Little clay and fine sand
17	0.24	0.47	1.42	1.63	Fine-to-medium sand
Reach 23, river mile 267.9					
15	0.35	7.0	4.83	4.50	Fine-to-coarse sand
14	2.0		30.13	427.27	Fine-to-coarse sand
13	0.075	0.38			Silty fine-to-medium sand
Reach 24, river mile 276.8					
9	:20.0	67.0	1667.92		Fine-to-coarse gravel
7,8	:14.0	103.0	6.52	28.57	Sandy fine-to-coarse gravel

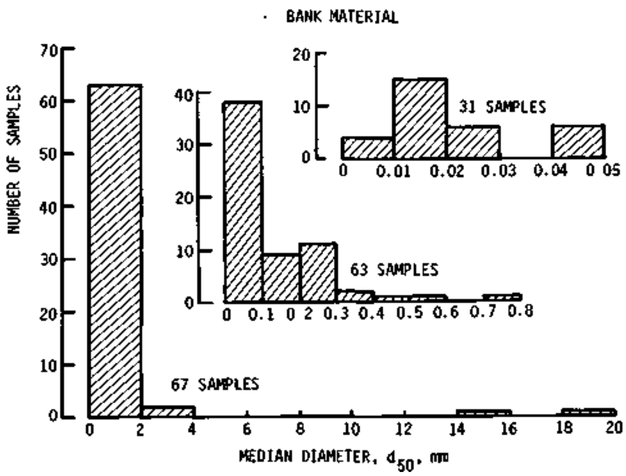


Figure 14. Frequency distribution of the median diameter of the bank materials

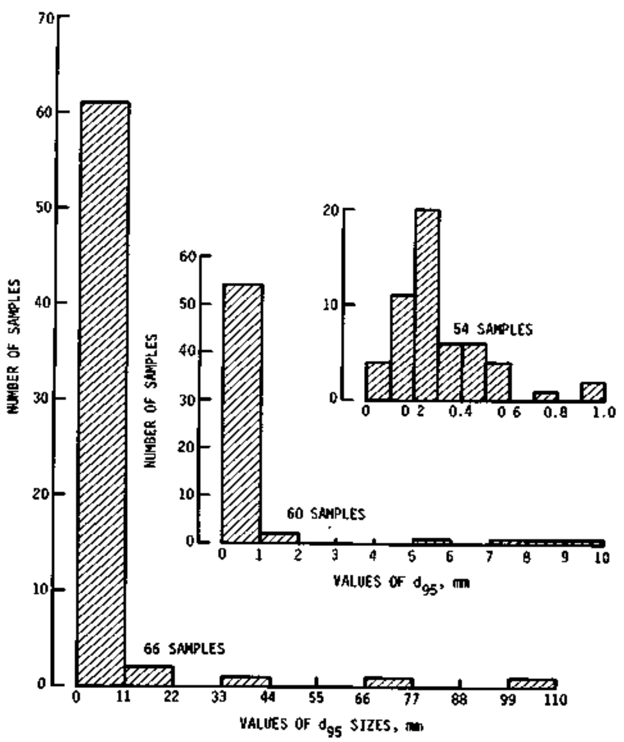


Figure 15. Frequency distribution of the d_{95} sizes of the bank materials

Bed Material Sizes

A total of 54 bed material samples were collected and analyzed. Table 3 shows the values of d_{50}, d_{95}, σ, and U, and a description of the materials. Other information shown are river mile locations, sample numbers, and general comments on the type of materials.

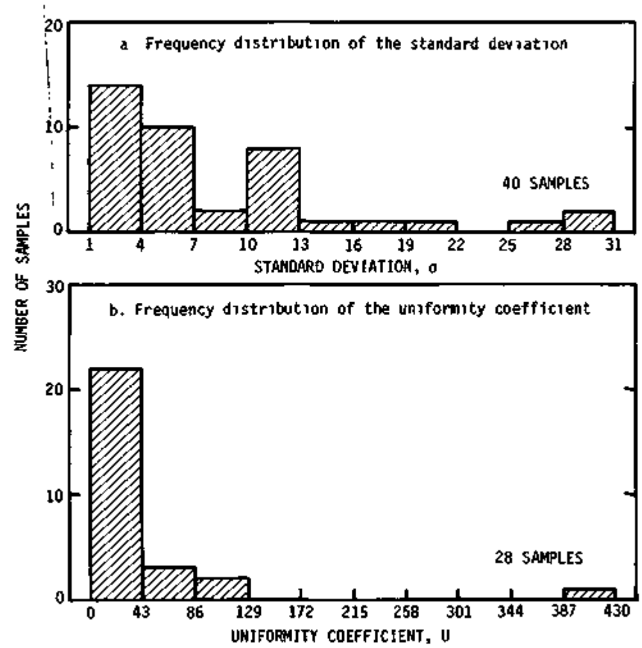


Figure 16. Frequency distribution of standard deviation (0) and uniformity coefficient (U)

Figure 17 shows the frequency distribution of the median diameter, d_{50}, of the bed materials. Out of 53 samples plotted, 49 had d_{50} sizes less than 5 mm. However, the insert in the figure indicates that 14 of the 49 samples with d_{50} less than 5 mm had d_{50} values less than 0.1 mm, whereas the rest of the d_{50} values follow a distribution similar to a normal distribution function with a mean value somewhere in the range of 0.3 and 0.4 mm. However, when all of the samples are considered, it is obvious that the bed material of the Illinois River is basically composed of fine-to-medium sands with the occasional presence of gravel size and larger particles.

Figure 18, where the frequency distributions of the d_{95} sizes of the bed materials are shown, indicates that 44 of the 54 samples had d_{95} values less than 6.6 mm. The inserts indicate that most of these 44 samples had d_{95} values less than 1.2 mm.

The frequency distribution of the standard deviation, *a*, and uniformity coefficient, U, are shown in figures 19 and 20, respecitvely. They indicate that the bed materials of the Illinois River are basically well graded.

The bank and bed material data presented so far and the various parameters computed from the particle size distribution will be used later for the stability analysis of the banks. This should be an ex-

Table 3. Particle Size Characteristics of Bed Material Samples

River mile	*Sample number*	*d_{50} (mm)*	*d_{95} (mm)*	*σ*	*U*	*Remarks*
8.0	121		0.014			Clay
8.0	120	0.24	0.70	1.59	1.59	Fine-to-medium sand
13.2	119	0.42	6.0	3.49	2.45	Fine-to-coarse sand
17.0	118	0.23	32.0	8.0	46.67	Fine-to-medium sand
22.8	117	0.019	0.070			Silt
28.9	114	0.33	0.65	1.49	1.85	Fine-to-medium sand
33.0	113	0.024	0.49			Sandy silt
41.4	110	0.37	1.4	1.56	1.78	Fine-to-coarse sand
48.5	109	0.28	23.0	1.54	1.88	Fine-to-coarse sand
54.2	108	0.47	1.0	1.48	2.13	Fine-to-coarse sand
60.2	104	0.0125	0.32			Silt
65.8	103	0.35	0.52	1.56	2.62	Fine-to-medium sand
69.3	102	0.30	1.0	1.60	1.79	Fine-to-medium sand
76.0	101	0.33	0.61	1.40	1.68	Fine-to-medium sand
82.1	97	0.40	0.80	1.43	1.91	Fine-to-medium sand
88.2	96	0.38	0.75	1.54	2.15	Fine-to-medium sand
92.0	95	0.38	1.0	1.54	2.0	Fine-to-coarse sand
95.8	94	0.42	1.20	1.61	2.19	Fine-to-medium sand
101.7	93	0.012	0.18			Silt
107.0	87	0.30	1.50	1.80	2.19	Fine-to-coarse sand
112.6	86	0.32	1.10	1.71	2.25	Fine-to-coarse sand
118.0	82	0.38	1.50	1.78	2.20	Fine-to-medium sand
124.0	78	0.40	10.0	3.68	3.57	Fine-to-coarse sand
129.9	74	0.090	0.30	1.45	1.28	Fine sand
135.0	70	0.18	2.20	2.49	1.31	Fine-to-coarse sand
140.0	59	0.36	1.70	1.66	2.10	Fine-to-coarse sand
145.0	65	0.19	1.05	3.30	5.40	Fine-to-medium sand
150.0	61	0.43	1.50	2.15	3.57	Fine-to-medium sand
154.4	57	0.013	0.45			Silt
160.2	56	20.0	55.0	6.38	31.94	Sandy shells
161.0	125	0.045	0.25	5.10		Sandy silt
161.0	126	0.17	0.46	2.17	3.50	Fine-to-medium sand
166.0	55	0.0045	0.55			Clayey silt
174.9	54	0.0054	0.52			Clayey silt
180.0	50	0.30	0.62	1.54	2.0	Fine-to-medium sand
186.4	49	0.025	0.20	5.77		Sandy silt
196.4	48	27.0	60.0	1.96	103.33	Fine gravel and shells
206.0	45	0.32	0.80	1.33	1.38	Fine-to-medium sand
213.0	42	0.36	1.80	1.83	1.91	Fine-to-coarse sand
218.0	41	0.40	4.0	2.02	1.50	Fine-to-coarse sand
222.0	40	0.33	1.15	1.46	1.23	Fine-to-medium sand
229.0	25	0.35	3.0	2.05	2.05	Fine-to-coarse sand
238.0	22	0.71	5.5	2.58	2.79	Fine-to-coarse sand
242.9	21	30.0	66.0	1.80	3.50	Fine-to-coarse sand
250.0	20	0.48	25.0	4.92	2.0	Fine-to-coarse sand
263.4	16	0.51	32.0	26.24	2.26	Fine-to-coarse sand
265.0	19	0.54	60.0	46.21	3.17	Fine-to-coarse sand
269.0	12	0.38	2.0	1.63	1.83	Fine-to-coarse sand
272.4	11	0.011	0.75			Silt
274.0	10	0.01	0.20			Silt
277.0	6	0.08	0.90	7.98	46.67	Silty sand
279.4	5	50.0	65.0	4.72	33.33	Fine-to-coarse gravel
282.3	4	0.275	0.75	1.86	2.41	Fine-to-medium sand
286.9	1	0.024	0.40	7.13	26.92	Sandy silt

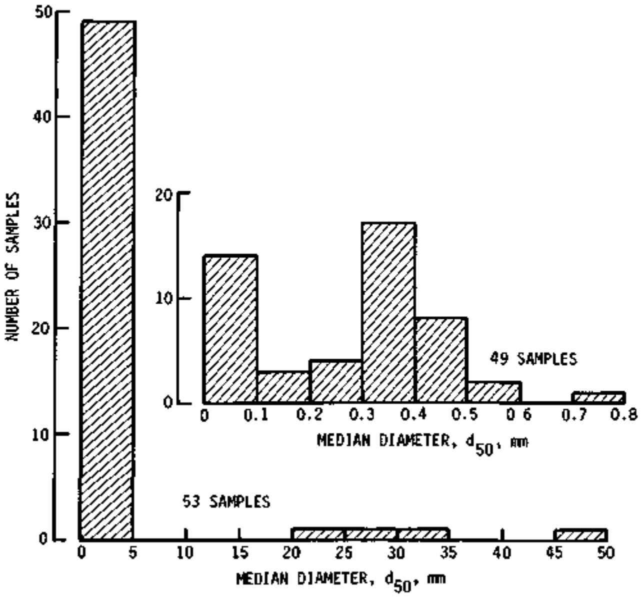


Figure 17. Frequency distribution of the median diameter of the bed materials

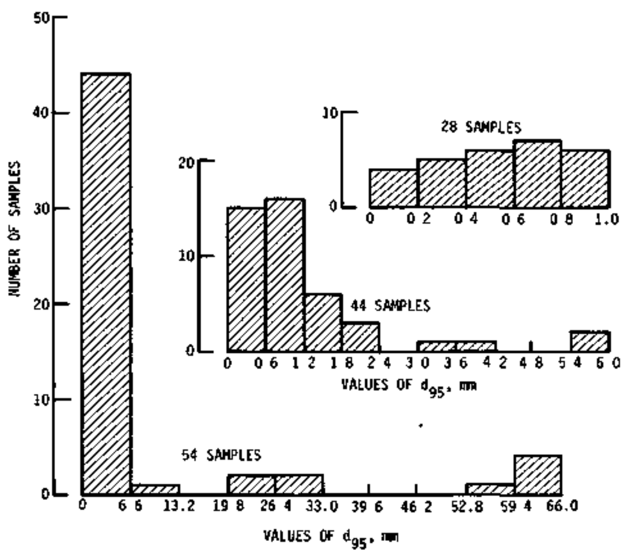


Figure 18. Frequency distribution of the d_{95} sizes of the bed materials

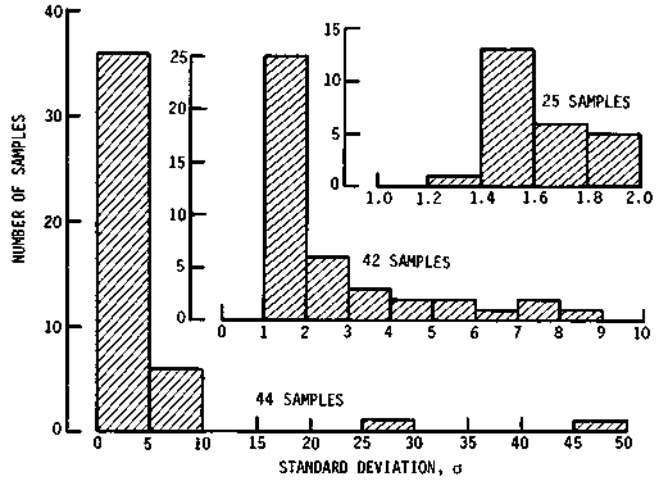


Figure 19. Frequency distribution of the standard deviation *(σ)* of the bed materials

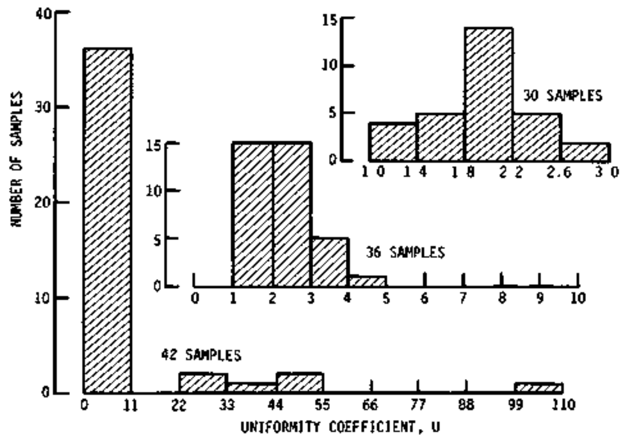


Figure 20. Frequency distribution of the uniformity coefficient (U) of the bank materials

cellent data base that could be used in the future for further hydraulic analysis of the Illinois River. Knowledge of the size distribution of the bed materials is needed in the study and investigation of sediment transport in any open channel flow problem. To the best of the authors' knowledge, this is the first time that a comprehensive set of bed and bank material sample data from the Illinois River were collected and analyzed systematically.

Hydraulic Geometry of the River

In the stability analysis of the banks at various selected reaches, some hydraulic geometric parameters must be determined on the basis of historical data. The parameters that are needed are: the discharge, Q, for some specified frequency; the corresponding cross-sectional area, A; top width, W; depth, D; and the river stage. These data are needed for two different cases, e.g., with the present diversion (3200 cfs) and with increased diversion discharges.

Most of the flow data that were needed for the stability analysis of the banks were supplied by the Corps of Engineers. The Corps also supplied the plots showing the average daily stages and the average daily discharges versus time for the water years of 1971, 1973, and 1977. These data were given for the conditions based on present and increased diversion practices. Data were available for only 17 locations along the whole length of the river. Since the 20 selected reaches were scattered along the river from Joliet to Grafton, quite a bit of interpolation had to be made to estimate the stage and discharge at or near any one of these selected reaches.

The stability of any bank depends upon many hydraulic and geometric factors. But whenever the stage in the river is relatively high, it is suspected that the banks of the river will be vulnerable to the erosive action of the flow as compared with the low flow regime of the river. Therefore, in all subsequent analyses, it was assumed that the critical condition related to the bank erosion potential of the river will exist whenever the stage in the river is the highest. The stability of each reach was checked against this selected maximum stage and discharge for present and increased diversion practices.

Two of the water years, 1971 and 1973, were years with relatively high stage conditions. For these water years, the maximum stage and discharge at all selected reaches did not show any variation or change between the conditions of the present diversion of 3200 cfs and increased diversions of 6600 and 10,000 cfs. Therefore, for these water years, the stability of the banks was tested for only one set of conditions. On the other hand, water year 1977 was a relatively dry year. The maximum stages for the conditions of present diversion and increased diversions of 6600 and 10,000 cfs did show some changes at all selected locations, so the stability of the banks were tested for the three different conditions.

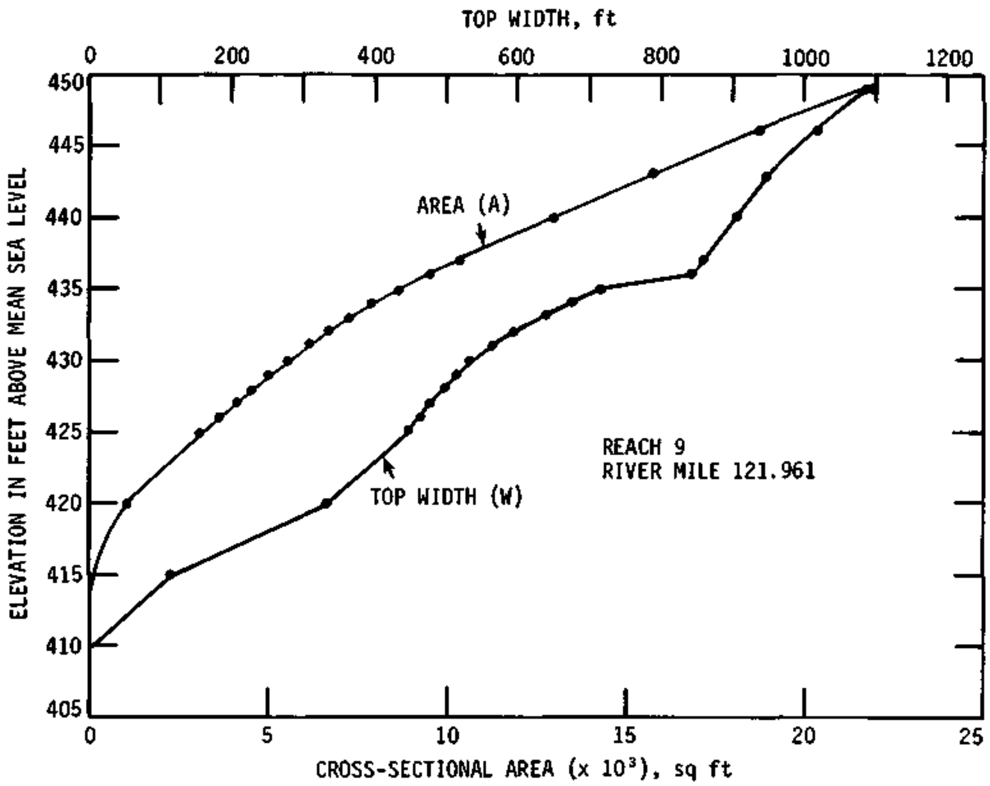
The values of A, D, and W for selected maximum stages for each reach were computed from the sounding data supplied by the Corps of Engineers. All of the sounding data for each reach were plotted as elevations above mean sea level versus A and W. Figure 21 shows such relationships for Reach 9 for two cross sections. Once the maximum stages for. various conditions were selected, values of A and W were determined from plots similar to those shown in figure 21. Whenever the sounding data were available at more than one cross section in any reach, an average of the values of A and W were computed. With known discharge, Q, cross-sectional area, A, and top width, W, the values of average depth, D, and average velocity, V, were computed.

In some instances, the floodplain of the Illinois River is broad and wide. In such cases, it is probable that the floodplain is not fully effective in conveying an equal amount of discharge proportional to its cross-sectional area (Bhowmik and Stall, 1979b). Therefore, in a few instances the effective cross-sectional areas were modified, and the values of W and A were computed on the basis of this modified shape of the river. Figure 22 shows such a typical case for Reach 23 near River Mile 268. Here it was assumed that the effective cross-sectional area of the river varies similarly to the cross-sectional area shown by the broken line. The relationships between elevations above msl in feet versus top width, W, and area, A, were developed on the basis of this modified cross-sectional shape of the river at this location.

Stability Analysis

On the basis of the particle size distribution analysis presented thus far, the Illinois River essentially flows through alluvial materials composed of gravel to rock near its upper part to sand, silt, and clay near its lower part. Most of the major rivers of the world also flow through alluvial materials with a sand bed. Streams and rivers flowing in a sand bed channel become altered by changes in the bed forms (Simons and Richardson, 1971). In some instances, these changes in bed form can alter flow resistance and the concentration of the suspended sediments dramatically. In some cases, an increase in resistance to flow can increase the flow depths quite significantly.

In testing the stability of any river bank one must consider the various factors that may make a bank unstable. Among the various forces that can cause a bank to erode are the force developed by the flowing water and the action of waves generated by either the wind or waterway traffic. Among physical parameters that will affect the bank stability are the bank material sizes, the bank slope, natural or artificial protective measures, orientation of the exposed bank toward the prevailing wind direction, the proximity of the bank to the main waterway traffic, frequency and physical characteristics of the waterway traffic, climatic changes



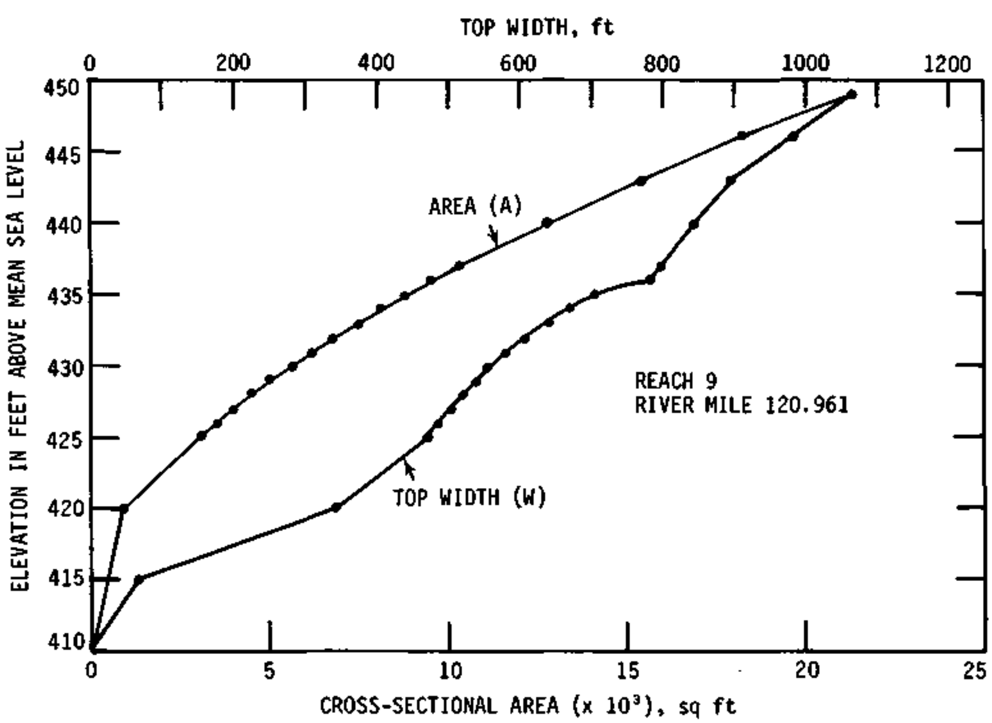


Figure 21. Typical hydraulic geometry relationships for Reach 9

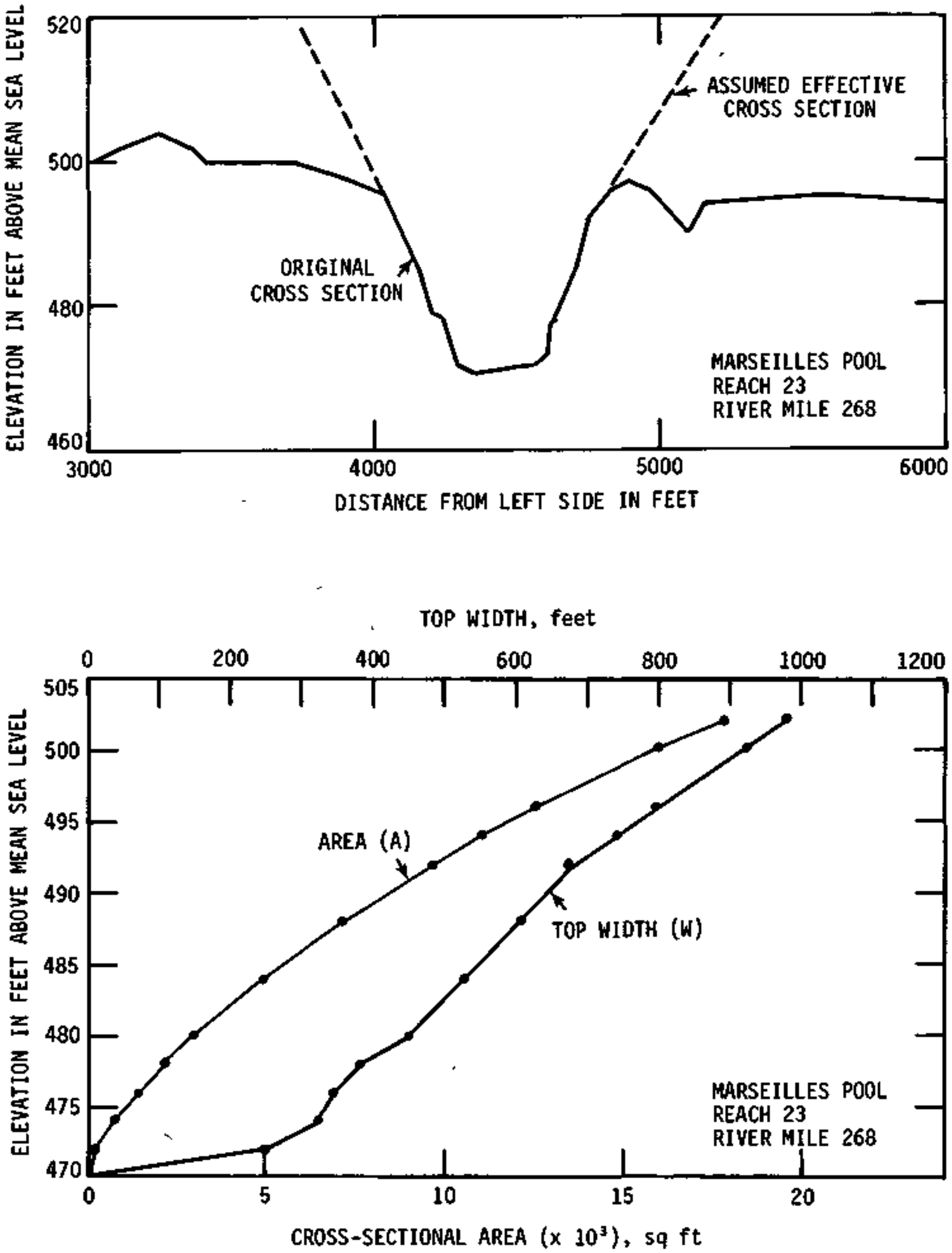


Figure 22. Typical hydraulic geometry relationships for Reach 23

which may account for rapid changes in the viscosity of the water, and ice action.

From observations, it appears that a combination of flow characteristics and wave action is responsible for the bank erosion of the Illinois River. The segment or segments of the river banks that are being eroded consist of materials from sand to silt to clay particle sizes. Unless these materials are on a very flat slope, their natural resistance against erosion in a high velocity stream is negligible. Moreover, wave action or flow may undercut the bank. The cantilevered bank will either fall because of its own weight or because of the effects of the next high flow. Figure 23 shows two such hypothetical cases.

In many places along the Illinois River the banks are stable. Usually at all of these places, the bank materials consist of larger particles or dense vegeta-

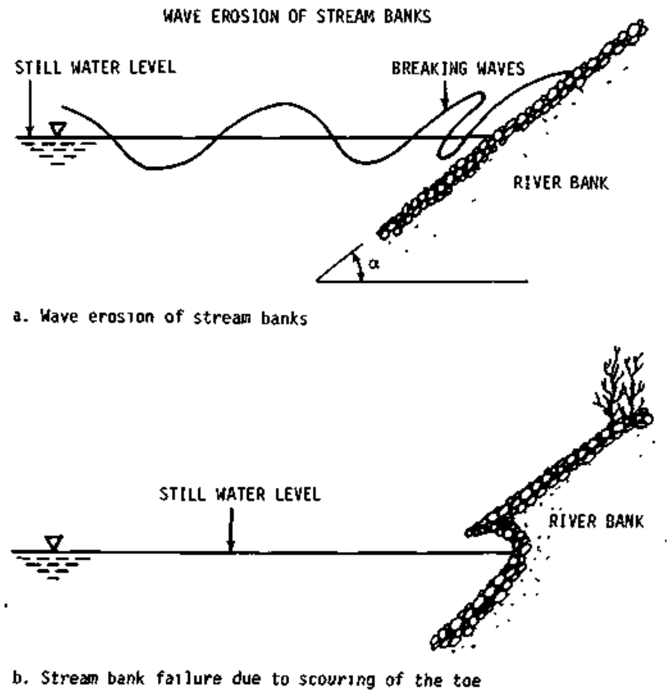


Figure 23. Initiation of bank erosion

tion, or the tree roots are well developed and help to protect the banks.

The stability analyses of the bank slopes for 20 selected reaches are discussed next in this report. The bank stability was analyzed by a number of different methods, namely, Lane's critical tractive force method (Lane, 1955), the critical velocity or permissible velocity method for various bank material sizes (Lane, 1955; Chow, 1959), and the Shields' criteria (ASCE, 1975). In addition to these methods, the stability of the banks was also tested against wave action generated by prevailing winds.

Theoretically, the flow velocity in a confined waterway should increase during the passage of a large tow with barges, especially so underneath a barge with a 9-foot draft. The increased flow velocity may accelerate the scour of the bed and the erosion of the banks.

Stability Analysis of the Individual Reaches

Tables 4 and 5 show the parameters that were computed and/or estimated to test the stability of the banks. Data are shown for the water years 1971, 1973, and 1977, and the various parameters are explained below.

The maximum discharge, Q, in cfs, was estimated on the basis of the maximum stage at all selected locations. Cross-sectional area, A, in square feet, top width, W, in feet, and the average depth, D, in feet, were estimated from the sounding data supplied by the Corps of Engineers. The effective cross-sectional shape of the river for Reaches, 1, 2, 3, 12, 13, 14, 15, 22, and 23 was assumed to be different from that given by the actual sounding data similar to figure 22 for Reach 23.

The average velocity, V, in feet per second (fps), was computed on the basis of discharge, Q, and cross-sectional area, A. The average bed slope, So, in feet per mile (ft/mi) was computed from actual field data as described previously. The shear force, T_O, was computed by the equation given below

$$\tau_o = \gamma D S_o \qquad \sqrt{\gamma D S_o} = \sqrt{\tau} \qquad (3)$$

where γ is the unit weight of water in pounds per cubic foot and T_o is in pounds per square foot. The shear velocity, V*, was computed by

$$V_* = \sqrt{g D S_o} \qquad (4)$$

where g is the acceleration due to gravity in feet per second squared and V* is in fps. The median diameter of the bank materials in tables 4 and 5 has been converted to inches. The boundary Reynolds number, R*, is defined by

$$R^* \approx V_* d_{50} / \nu \qquad (5)$$

where V* is the shear velocity in fps, d_{50} is the median diameter of the bank materials in feet, and v is the kinematic velocity of water in square feet per second. For the computations shown in tables 4 and 5, the values of v are based on a water temperature of 65° F. The dimensionless shear stress was computed by the equation

$$\tau_* = \tau_o / [(\gamma_s - \gamma) d_{50}] \qquad (6)$$

where γ_s is the unit weight of the bank materials assumed to be 165 pounds per cubic foot, 7 is the unit weight of water equal to 62.4 pounds per cubic foot. Values of the boundary Reynolds number and the dimensionless shear stress were needed to test the stability of banks with the Shields' relationship (ASCE, 1975).

Lane's tractive force (Lane, 1955) shown as τ_L and the maximum permissible velocity shown as V_p were based on the relationships and tables given by Lane.

All of the parameters discussed thus far are given

Table 4. Stability Analyses for Water Year 1977 for Given Diverted Flows

Reach	Maximum discharge, Q (cfs)	Cross-sectional area, A (sq ft)	Top width, W (ft)	Average depth, D (ft)	Average velocity, V (fps)	Average bed slope, S_o (ft/mile)	Bed shear stress, To (lb/sq ft)
Diverted flow = 3200 cfs							
1	38,800	11,300	895	12.6	3.4	0.0715	0.011
2	37,900	15,400	1310	11.8	2.5	0.0715	0.010
3	36,400	14,500	1200	12.1	2.5	0.0715	0.010
4	37,800	16,900	1093	15.5	2.2	0.057	0.010
5	35,000	11,400	743	15.3	3.1	0.057	0.010
6	35,000	11,500	928	12.4	3.0	0.057	0.008
7	35,000	10,800	825	13.1	3.2	0.057	0.009
8	35,000	14,750	1085	13.6	2.4	0.057	0.009
9	30,500	12,700	870	14.6	2.4	0.057	0.010
12	30,500	12,600	655	19.2	2.4	0.057	0.013
13	26,200	14,200	860	16.5	1.9	0.057	0.011
14	26,200	12,700	685	18.5	2.1	0.057	0.012
15	25,400	11,900	915	13.0	2.1	0.0107	0.002
17	27,300	14,650	1380	10/6	1.9	0.0107	0.001
18	27,000	14,800	820	18.1	1.8	0.0107	0.002
19	27,000	14,800	820	18.1	1.8	0.0107	0.002
20	27,000	14,800	820	18.1	1.8	0.0107	0.002
22	39,700	13,300	675	19.7	3.0	0.155	0.036
23	30,300	13,600	840	16.2	2.2	0.155	0.030

Reach	Shear velocity, v_* (fps)	Median diameter of the bank material, d_{50} (inches)	Boundary Reynolds number, R_*	Dimensionless shear stress, T_*	Lane's limiting tractive force, T_L (lb/sq ft)	Maximum permissible velocity, V (fps)
Diverted flow = 3200 cfs						
1	0.074	0.0005	0.27	2.56	0.048	5.5
2	0.072	0.0008	0.42	1.46	0.047	5.5
3	0.073	0.002	1.08	0.58	0.048	5.5
4	0.073	0.0044	2.37	0.26	0.044	3.0
5	0.073	0.0006	0.32	1.94	0.023	5.5
6	0.066	0.0003	0.15	3.11	0.042	5.5
7	0.067	0.0009	0.44	1.17	0.049	5.5
8	0.069	0.010	5.09	0.10	0.049	3.0
9	0.071	0.062	32.5	0.019	0.062	6.5
12	0.082	0.0007	0.42	2.2	0.044	5.5
13	0.076	0.0028	1.57	0.46	0.049	5.5
14	0.080	0.003	1.77	0.47	0.048	5.5
15	0.029	0.0061	1.30	0.038	0.049	3.0
17	0.026	0.0042	0.81	0.028	0.048	3.0
18	0.034	0.0049	1.23	0.048	0.048	3.0
19	0.034	0.0059	1.48	0.039	0.049	3.0
20	0.034	0.010	2.51	0.023	0.049	3.0
22	0.136	0.0051	5.12	0.82	0.042	3.0
23	0.124	0.032	29.3	0.11	0.058	3.0

Table 4. Continued

Reach	Maximum discharge, Q (cfs)	Cross-sectional area, A (sq ft)	Top width, W (ft)	Average depth, D (ft)	Average velocity, V (fps)	Average bed slope, S_O (ft/mile)	Bed shear stress, T_O (lb/sq ft)
Diverted flow = 6600 cfs							
1	41,700	12,300	940	13.1	3.4	0.0715	0.011
2	40,700	16,800	1360	12.4	2.4	0.0715	0.010
3	39,000	16,000	1260	12.7	2.4	0.0715	0.011
4	40,500	17,550	1100	16.0	2.3	0.057	0.011
5	36,200	11,500	750	15.3	3.2	0.057	0.010
6	36,200	11,800	945	12.5	3.1	0.057	0.008
7	36,200	11,000	843	13.1	3.3	0.057	0.009
8	36,200	14,950	1103	13.6	2.4	0.057	0.009
9	32,500	12,950	875	14.8	2.5	0.057	0.010
12	32,500	13,100	660	19.9	2.5	0.057	0.013
13	28,300	14,800	890	16.6	1.9	0.057	0.011
14	28,300	13,000	690	18.8	2.2	0.057	0.013
15	27,100	12,400	930	13.3	2.2	0.0107	0.0017
17	28,500	15,500	1470	10.5	1.8	0.0107	0.0013
18	27,800	14,950	838	17.8	1.9	0.0107	0.0023
19	27,800	14,950	838	17.8	1.9	0.0107	0.0023
20	27,800	14,950	838	17.8	1.9	0.0107	0.0023
22	39,700	13,300	675	19.7	3.0	0.155	0.036
23	30,300	13,600	840	16.2	2.2	0.155	0.030

Reach	Shear velocity, v_* (fps)	Median diameter of the bank material, d_{50} (inches)	Boundary Reynolds number, R_*	Dimensionless shear stress, T_*	Lane's limiting tractive force, T_L (lb/sq ft)	Maximum permissible velocity, V_P (fps)
Diverted flow = 6600 cfs						
1	0.076	0.0005	0.28	2.58	0.048	5.5
2	0.074	0.0008	0.43	1.53	0.047	5.5
3	0.074	0.002	1.10	0.63	0.048	5.5
4	0.075	0.0044	2.42	0.29	0.044	3.0
5	0.073	0.0006	0.32	2.0	0.023	5.5
6	0.066	0.0003	0.15	3.27	0.042	5.5
7	0.067	0.0009	0.45	1.14	0.049	5.5
8	0.069	0.010	5.07	0.11	0.049	3.0
9	0.072	0.062	32.80	0.019	0.062	6.5
12	0.083	0.0007	0.43	2.2	0.044	5.5
13	0.076	0.0028	1.57	0.47	0.049	5.5
14	0.081	0.003	1.79	0.49	0.048	5.5
15	0.029	0.0061	1.33	0.032	0.049	3.0
17	0.026	0.0042	0.81	0.037	0.048	3.0
18	0.034	0.0049	1.23	0.054	0.048	3.0
19	0.034	0.0059	1.48	0.044	0.049	3.0
20	0.034	0.010	2.51	0.026	0.049	3.0
22	0.136	0.0051	5.13	0.82	0.042	3.0
23	0.124	0.032	29.20	0.11	0.058	3.0

Table 4. Continued

Reach	Maximum discharge, Q (cfs)	Cross-sectional area, A (sq ft)	Top width, W (ft)	Average depth, D (ft)	Average velocity, V (fps)	**Average** bed slope, S_O (ft/mile)	Bed shear stress, T_O (lb/sq ft)
Diverted flow = 6600 cfs							
1	41,700	12,300	940	13.1	3.4	0.0715	0.011
2	40,700	16,800	1360	12.4	2.4	0.0715	0.010
3	39,000	16,000	1260	12.7	2.4	0.0715	0.011
4	40,500	17,550	1100	16.0	2.3	0.057	0.011
5	36,200	11,500	750	15.3	3.2	0.057	0.010
6	36,200	11,800	945	12.5	3.1	0.057	0.008
7	36,200	11,000	843	13.1	3.3	0.057	0.009
8	36,200	14,950	1103	13.6	2.4	0.057	0.009
9	32,500	12,950	875	14.8	2.5	0.057	0.010
12	32,500	13,100	660	19.9	2.5	0.057	0.013
13	28,300	14,800	890	16.6	1.9	0.057	0.011
14	28,300	13,000	690	18.8	2.2	0.057	0.013
15	27,100	12,400	930	13.3	2.2	0.0107	0.0017
17	28,500	15,500	1470	10.5	1.8	0.0107	0.0013
18	27,800	14,950	838	17.8	1.9	0.0107	0.0023
19	27,800	14,950	838	17.8	1.9	0.0107	0.0023
20	27,800	14,950	838	17.8	1.9	0.0107	0.0023
22	39,700	13,300	675	19.7	3.0	0.155	0.036
23	30,300	13,600	840	16.2	2.2	0.155	0.030

Reach	Shear velocity, v_* (fps)	Median diameter of the bank material, d_{50} (inches)	Boundary Reynolds number, R_*	Dimensionless shear stress, T_*	Lane's limiting tractive force, T_L (lb/sq ft)	Maximum permissible velocity, V_P (fps)
Diverted flow = 6600 cfs						
1	0.076	0.0005	0.28	2.58	0.048	5.5
2	0.074	0.0008	0.43	1.53	0.047	5.5
3	0.074	0.002	1.10	0.63	0.048	5.5
4	0.075	0.0044	2.42	0.29	0.044	3.0
5	0.073	0.0006	0.32	2.0	0.023	5.5
6	0.066	0.0003	0.15	3.27	0.042	5.5
7	0.067	0.0009	0.45	1.14	0.049	5.5
8	0.069	0.010	5.07	0.11	0.049	3.0
9	0.072	0.062	32.80	0.019	0.062	6.5
12	0.083	0.0007	0.43	2.2	0.044	5.5
13	0.076	0.0028	1.57	0.47	0.049	5.5
14	0.081	0.003	1.79	0.49	0.048	5.5
15	0.029	0.0061	1.33	0.032	0.049	3.0
17	0.026	0.0042	0.81	0.037	0.048	3.0
18	0.034	0.0049	1.23	0.054	0.048	3.0
19	0.034	0.0059	1.48	0.044	0.049	3.0
20	0.034	0.010	2.51	0.026	0.049	3.0
22	0.136	0.0051	5.13	0.82	0.042	3.0
23	0.124	0.032	29.20	0.11	0.058	3.0

Table 4. Concluded

Reach	Maximum discharge, Q (cfs)	Cross-sectional area, A (sq ft)	Top width, W (ft)	Average depth, D (ft)	Average velocity, V (fps)	Average bed slope, S_O (ft/mile)	Bed shear stress, T_O (lb/sq ft)
Diverted flow = 10,000 cfs			***Table 4***				
1	44,700	13,100	975	13.4	3.4	0.0715	0.011
2	43,600	18,100	1400	12.9	2.4	0.0715	0.011
3	41,800	16,800	1320	12.7	2.5	0.0715	0.011
4	43,300	18,200	1113	16.4	2.4	0.057	0.011
5	38,800	12,000	783	15.3	3.2	0.057	0.010
6	38,800	12,350	935	13.2	3.1	0.057	0.009
7	38,800	11,550	878	13.2	3.4	0.057	0.009
8	38,800	15,750	1125	14.0	2.5	0.057	0.009
9	35,200	13,600	888	15.3	2.6	0.057	0.010
12	35,200	13,600	670	20.3	2.6	0.057	0.014
13	30,800	15,500	940	16.5	2.0	0.057	0.011
14	30,800	13,600	695	19.6	2.3	0.057	0.013
15	29,300	13,200	950	13.9	2.2	0.0107	0.002
17	30,900	16,450	1585	10.4	1.9	0.0107	0.001
18	29,700	15,400	855	18.0	1.9	0.0107	0.002
19	29,700	15,400	855	18.0	1.9	0.0107	0.002
20	29,700	15,400	855	18.0	1.9	0.0107	0.002
22	39,700	13,300	675	19.7	3.0	0.155	0.036
23	30,300	13,600	840	16.2	2.2	0.155	0.030

Reach	Shear velocity, v_* (fps)	Median diameter of the bank material, d_{50} (inches)	Boundary Reynolds number, R_*	Dimensionless shear stress, T_*	Lane's limiting tractive force, T_L (lb/sq ft)	Maximum permissible velocity, V_P (fps)
Diverted flow = 10,000 cfs						
1	0.076	0.0005	0.28	2.64	0.048	5.5
2	0.075	0.0008	0.44	1.59	0.047	5.5
3	0.074	0.002	1.10	0.63	0.048	5.5
4	0.076	0.0044	2.45	0.29	0.044	3.0
5	0.073	0.0006	0.32	2.0	0.023	5.5
6	0.068	0.0003	0.15	3.45	0.042	5.5
7	0.068	0.0009	0.45	1.15	0.049	5.5
8	0.070	0.010	5.14	0.11	0.049	3.0
9	0.073	0.062	33.34	0.019	0.062	6.5
12	0.084	0.0007	0.43	2.28	0.044	5.5
13	0.076	0.0028	1.56	0.46	0.049	5.5
14	0.083	0.003	1.83	0.51	0.048	5.5
15	0.030	0.0061	1.35	0.034	0.049	3.0
17	0.026	0.0042	0.81	0.036	0.048	3.0
18	0.034	0.0049	1.24	0.054	0.048	3.0
19	0.034	0.0059	1.49	0.045	0.049	3.0
20	0.034	0.010	2.53	0.027	0.049	3.0
22	0.136	0.0051	5.13	0.82	0.042	3.0
23	0.124	0.032	29.00	0.108	0.058	3.0

Table 5. Stability Analyses for Water Years 1971 and 1973*

Reach	Maximum discharge, Q (cfs)	Cross-sectional area, A (sq ft)	Top width, W (ft)	Average depth, D (ft)	Average velocity, V (fps)	Average bed slope, S_O (ft/mile)	Bed shear stress, T_O (lb/sq ft)
Water Year 1971							
1	47,000	14,300	1020	14.0	3.3	0.0715	0.012
2	46,900	19,600	1455	13.5	2.4	0.0715	0.011
3	46,800	18,300	1400	13.1	2.6	0.0715	0.011
4	44,300	15,150	1048	14.5	2.9	0.057	0.010
5	31,600	10,700	700	15.3	3.0	0.057	0.010
6	31,600	10,700	870	12.3	3.0	0.057	0.008
7	31,600	10,150	755	13.4	3.1	0.057	0.009
8	31,600	13,850	1015	13.7	2.3	0.057	0.009
9	29,200	11,900	858	13.9	2.5	0.057	0.009
12	29.200	12,300	650	18.9	2.4	0.057	0.013
13	27,300	13,800	840	16.4	2.0	0.057	0.011
14	27,300	12,900	690	18.7	2.1	0.057	0.013
15	27,700	12,600	935	13.5	2.2	0.0107	0.002
17	30,300	16,050	1550	10.4	1.9	0.0107	0.001
18	29,700	15,600	850	18.4	1.9	0.0107	0.002
19	29,700	15,600	850	18.4	1.9	0.0107	0.002
20	29,700	15,600	850	18.4	1.9	0.0107	0.002
22	28,600	11,700	660	17.7	2.4	0.155	0.032
23	23,400	10,100	700	14.4	2.3	0.155	0.026

Reach	Shear velocity, v_* (fps)	Median diameter of the bank material, d_{50} (inches)	Boundary Reynolds number, R_*	Dimensionless shear stress, T_*	Lane's limiting tractive force, T_L (lb/sq ft)	Maximum permissible velocity, V_p (fps)
Water Year 1971						
1	0.078	0.0005	0.29	2.76	0.048	5.5
2	0.077	0.0008	0.45	1.66	0.047	5.5
3	0.076	0.002	1.11	0.64	0.048	5.5
4	0.071	0.0044	2.30	0.26	0.044	3.0
5	0.073	0.0006	0.32	2.0	0.023	5.5
6	0.065	0.0003	0.14	3.21	0.042	5.5
7	0.068	0.0009	0.45	1.17	0.049	5.5
8	0.069	0.010	5.09	0.11	0.049	3.0
9	0.070	0.062	31.8	0.018	0.062	6.5
12	0.081	0.0007	0.42	2.12	0.044	5.5
13	0.076	0.0028	1.56	0.46	0.049	5.5
14	0.081	0.003	1.78	0.49	0.048	5.5
15	0.030	0.0061	1.34	0.033	0.049	3.0
17	0.026	0.0042	0.81	0.036	0.048	3.0
18	0.035	0.0049	1.25	0.055	0.048	3.0
19	0.035	0.0059	1.51	0.046	0.049	3.0
20	0.035	0.010	2.56	0.027	0.049	3.0
22	0.13	0.00:1	4.86	0.74	0.042	3.0
23	0.12	0.032	27.53	0.096	0.058	3.0

Table 5. Concluded

Reach	Maximum discharge, Q (cfs)	Cross-sectional area, A (sqft)	Top width, W (ft)	Average depth, D (ft)	Average velocity, V (fps)	Average bed slope, S_O (ft/mile)	Bed shear stress, T_O (lb/sq ft)
Water Year 1973							
1	98,800	25,800	1420	18.2	3.8	0.0715	0.015
2	97,900	35,500	1940	18.3	2.8	0.0715	0.015
3	96,300	34,500	2060	16.8	2.8	0.0715	0.014
4	98,300	29,850	1215	24.6	3.3	0.057	0.017
5	67,400	19,100	908	21.0	3.5	0.057	0.014
6	67,400	21,050	1075	19.6	3.2	0.057	0.013
7	67,400	19,550	1078	18.1	3.5	0.057	0.012
8	67,400	25,950	1378	18.8	2.6	0.057	0.013
9	65,000	21,650	1075	20.1	3.0	0.057	0.014
12	65,000	18,400	740	24.9	3.5	0.057	0.017
13	55,000	22,800	1110	20.5	2.4	.0.057	0.014
14	55.000	18,800	745	25.2	2.9	0.057	0.017
15	51,400	19,600	1120	17.5	2.6	0.0107	0.002
17	56,700	27,200	2880	9.4	2.1	0.0107°	0.001
18	51,200	21,400	1088	19.7	2.4	0.0107	0.002
19	51,200	21,400	1088	19.7	2.4	0.0107	0.002
20	51,200	21,400	1088	19.7	2.4	0.0107	0.002
22	77,800	16,600	715	23.2	4.7	0.155	0.042
23	55,000	17,400	965	18.0	3.2	0.155	0.032

Reach	Shear velocity, v_* (fps)	Median diameter of the bank material, d_{50} (inches)	Boundary Reynolds number, R_*	Dimensionless shear stress, T_*	Lane's limiting tractive force, T_L (lb/sq ft)	Maximum permissible velocity, V_p (fps)
Water Year 1973						
1	0.089	0.0005	0.33	3.58	0.048	5.5
2	0.089	0.0008	0.53	2.25	0.047	5.5
3	0.086	0.002	1.26	0.83	0.048	5.5
4	0.092	0.0044	3.00	0.44	0.044	3.0
5	0.085	0.0006	0.38	2.75	0.023	5.5
6	0.083	0.0003	0.18	5.13	0.042	5.5
7	0.079	0.0009	0.53	1.58	0.049	5.5
8	0.081	0.010	5.96	0.15	0.049	3.0
9	0.084	0.062	38.22	0.025	0.062	6.5
12	0.093	0.0007	0.48	2.79	0.044	5.5
13	0.084	0.0028	1.74	0.57	0.049	5.5
14	0.094	0.003	2.07	0.66	0.048	5.5
15	0.034	0.0061	1.52	0.042	0.049	3.0
17	0.025	0.0042	0.77	0.033	0.048	3.0
18	0.036	0.0049	1.30	0.059	0.048	3.0
19	0.036	0.0059	1.56	. 0.049	0.049	3.0
2*0	0.036	0.010	2.64	0.029	0.049	3.0
22	0.15	0.0051	5.57	0.97	0.042	3.0
23	0.13	0.032	30.78	0.12	0.058	3.0

** Maximum stages and discharges remained the same for all diversion cases (Data from U. S. Army Corps of Engineers)* in tables 4 and 5 for 19 reaches. Computations are not shown for Reach 24. Because of the broad and wide exposure of Reach 24 to the water surface (figure 9), it is obvious that the bank erosion at this location basically resulted from the wave action of the water.

If it is assumed that the tractive force on the bank is the dominant force against which the stability of the banks must be checked, then the values of T_O must be less than the values of T_L. The tabulated values shown in tables 4 and 5 indicate that in all cases, T_O is less than the value of T_L . Thus the banks at all locations should be stable as far as the tractive force is concerned.

On the other hand, if we assume that the stability of the banks depends upon the permissible velocity, V_p, that the bank materials can withstand, then the values of V should be less than the values of V . In tables 4 and 5, this is found to be true for all cases except for Reach 22 for the water year 1973. For this reach, the permissible velocity is more than the computed average velocity. The permissible velocities were estimated on the basis of the composition of the existing bank materials (Lane, 1955) at different locations.

The above comparison can be refined by estimating and using the bottom velocity V_b rather than the average velocity V. Further refinements can be made by taking into consideration the hydraulic effect of the river bend on flow velocity. Research results from Bhowmik and Stall (1978, 1979a) show that the value of the flow velocity at 0.5 foot above the bed can vary anywhere from 70 to 95 percent of the average velocities in the individual verticals in a cross section. The average of these values can be taken to be about 90 percent. Thus it is assumed that

$$V_b = 0.9\, V_v \qquad (7)$$

where V_v is the average velocity in any vertical in a cross section. On the other hand, the maximum average velocity in a vertical inside a bend was found to be about 28 percent more than the average velocity in the cross section. These data were collected from the Kaskaskia River, which is smaller than the Illinois River. If it is assumed that the relationships developed for the Kaskaskia River are also valid for the Illinois River, then the average maximum bottom velocity in the Illinois River in a bend can be assumed to be 15 percent more than the average velocity in the cross section as shown in equation 8.

$$V_b = 0.9\, V_v = (0.9)\,(1.28)\, V = 1.15\, V \qquad (8)$$

Reaches 1, 4, 5, 6, 7, 8, 9, 12, 13, 16, 18, and 19 are located either on the concave bank of a bend or on the bank that is a continuation of the concave bank of the upstream bend. If the average velocity is increased by 15 percent at all of these locations for all five conditions given in tables 4 and 5, then only the maximum bottom velocity at Reach 22 will exceed the maximum permissible velocity. This was found to be true for the water year 1977 with diversions of 6600 cfs and 10,000 cfs. Except for this location, in all other cases the banks should be stable as far as the maximum permissible velocities in the river for the existing bank material compositions are concerned.

When the stability of the banks was tested with Shields' relationship (ASCE, 1975), it was observed that in a few instances, the banks were shown to be unstable. In the Shields' relationship, the values of R_* and T_* are computed from equations 5 and 6, respectively, and these values are plotted in a figure similar to figure 24. However, it must be pointed out that the Shields' diagram was developed for noncohesive materials and that the value of the hydraulic gradient is needed to compute both the abscissa and the ordinate of figure 24. In almost all cases, the plotted points were found to be clustered around the particular bed slope that was used in the computation of U_* and T_O. Since in all the computations bed slope was assumed to be equal to the hydraulic gradient, and field data are not available for the magnitude of the hydraulic gradients, the stability analysis following the Shields' diagram may or may not be valid for the above cases.

The computed average velocities shown in tables 4 and 5 were based on the estimated stage, the discharge, and the cross-sectional area of the river at respective reaches. In order to check whether or not these computed average velocities corresponding to certain discharges are anywhere close to the measured average velocities, the gaging data from the U. S. Geological Survey files were compared with the computed velocities. Data were gathered from the gaging stations at Kingston Mines, Meredosia, and Marseilles.

The discharge measurement data from Kingston

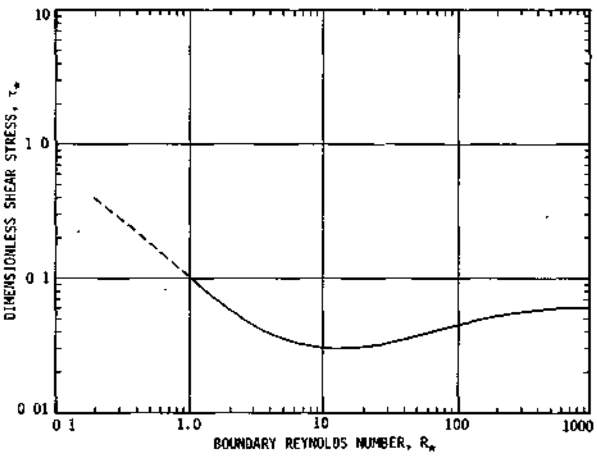


Figure 24. Shields' diagram (ASCE, 1975)

Mines resulted in average velocities of 2.03, 1.97, 2.40, and 3.33 fps corresponding to discharges of 20,500, 26,800, 37,000, and 61,600 cfs, respectively.

The computed velocities for Reaches 12, 13, and 14, which are close to the Kingston Mines gage, varied from 1.9 to 3.5 fps for discharges of 26,200 and 65,000 cfs, respectively.

The discharge measurement data at the Meredosia gage resulted in average velocities of 2.04 and 2.47 fps for discharges of 29,200 and 70,300 cfs, respectively. Computed velocities for Reaches 2, 3, and 4, which are in the proximity of the Meredosia gage, varied from 2.3 to 3.3 fps for discharges of 37,800 and 98,300 cfs, respectively.

The discharge measurement data at the Marseilles gage resulted in average velocities of 3.11 and 4.20 fps for discharges of 11,100 and 39,600 cfs, respectively. The computed velocities for Reach 22, which is about 20 miles upstream of the gage, varied from 2.4 to 4.7 fps for discharges of 28,600 and 77,800 cfs, respectively.

These computations indicated that the procedure followed in the analysis and estimation of different parameters shown in tables 4 and 5 should yield a reasonable approximation of the actual field condition for the anticipated flow condition in the river.

Stability of the Banks against Wind-Generated Waves

Banks exposed to the direct action of waves will erode if they lack protection, and to a certain degree, almost all reaches of the Illinois River are exposed to wave action.

An analysis, using methodology given in detail by Bhowmik (1976, 1978), was made to compute the wave height and the stable size of the bank materials. The methodology suggested in the Shore Protection Manual by the U. S. Army Corps of Engineers (1977) can also be used to compute wave height and the stable size of the bank materials.

In the computation of the wave height, it was assumed that wind blowing for a duration of 6 hours having a return period of 50 years will be the critical wind velocity that may develop significant wave action. Historical data related to wind velocity and duration were analyzed by Bhowmik (1976, 1978) for five climatological stations in and around Illinois. The design wind velocity was selected for each reach on the basis of its proximity to the climatological station for which data have been analyzed. The wind data analyses also included the variability of the prevailing wind directions.

Once the wind velocity and direction were selected, the maximum fetch, F, facing the exposed bank was measured from the charts of the Illinois Waterway (U. S. Army Corps of Engineers, 1974). Here fetch, F, is defined as the maximum length of the water surface over which the wind blows before it is deflected by the bank. In any confined waterway, the maximum fetch is usually much larger than the width, W, of the waterway normal to the direction of the fetch. In all the theoretical relationships that have been developed by various researchers to compute the wave heights thus far, fetch is used as a parameter, provided the value of the width of the waterway normal to the direction of the fetch is also as long as the fetch itself. In order to make corrections for the effects of the confined waterway, the following equation was utilized to compute the effective fetch, designated as F_e.

$$F_e = 1.054\, W^{0.6}\, F^{0.4} \qquad (9)$$

This equation is valid whenever the ratio of W/F is between 0.05 to 0.6. However, when the value of W/F is more than 0.6, the total length of the fetch was used to compute the wave height.

The wave height exceeded by one-third of the waves in the wave profile and designated as the significant wave height was computed by the following equation (Bhowmik, 1976).

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

where H_s is the significant wave height in feet, g is the acceleration due to gravity in feet per second squared, and V_w is the wind velocity in fps. With the computed value of H_s, the measured value of bank slope, a, and an assumed value of the specific gravity, the median weight of the stable riprap particle, W_{50}, was computed by the following equation.

$$W_{50} = (0.388\ S_s\ H_s^{\ 3})/(S_s - 1)^3\ (\cos \alpha - \sin \alpha)^3 \quad (11)$$

where W_{50} is the median weight of the riprap particle in pounds, S is the specific gravity of the particle, and α is the bank slope. For all computations, the value of S_s was assumed to be 2.65.

Two sets of computations based on the two methods to determine the fetch length were made to estimate the significant wave heights for each reach. Techniques for determining the fetch lengths for each method are shown in figure 25. For the first computation, fetch (a) was assumed to be the maximum length of the water surface over which the wind can blow based on the prevailing wind direction. Here, the measured fetch, F, was modified to estimate the effective fetch, F_e, from equation 9 to account for the constricted nature of the waterway. This value of F_e was then used to compute H_s from equation 10. In the computation of W_{50} from equation 11, the bank slope, α, had to be modified to account for the directional orientation of the fetch, F.

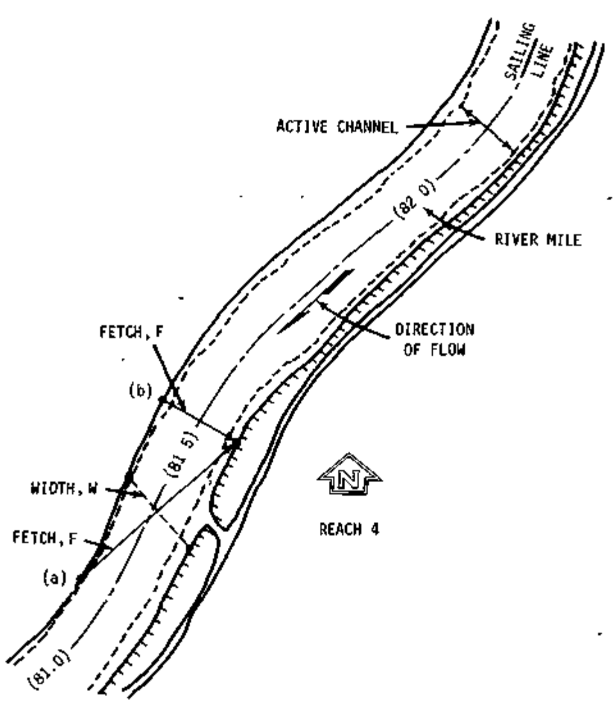


Figure 25. A typical reach showing the direction of wind and fetches utilized to compute the wind-generated wave height

For the second computation the wind and fetch (b) in figure 25 were taken in a direction normal to the exposed bank. Here no correction was used to account for the constriction of the waterway.

The computational procedure outlined above was followed for each reach of the river. For a detailed step by step procedure, the reader is referrred to the original publication by Bhowmik (1976).

The procedure outlined above was used to estimate the stable size of the bank materials against an anticipated wave action. These results are given in table 6. The computed values of the median diameter of the stable particles and the existing and measured median diameter of the bank materials are given in the last two columns. A comparison between these two sets of sizes of the median diameters will show that in all instances the estimated stable particle size is much higher than the existing size of the bank material.

Table 7 shows the computed values of the stable median diameter of the particles for selected reaches when the prevailing wind direction normal to the bank is considered. For these cases where the fetch is much smaller than for case (a) (figure 25), the estimated d_{50} is also always higher than the existing d_{50}.

Bank materials along the Illinois River are basically sandy to silty with some clay content. Any material with clay will be cohesive and hence may be more stable than the purely noncohesive materials. Therefore, in some cases, although the numerical differences between the computed and existing d_{50} sizes are very high, the effective size difference, considering the stability of the sand, silt, and clay mixture, may not be that high. Even though we can assume that this clayey mixture is more stable than noncohesive materials of fine-to-median size sands, still, considering wind-generated wave action alone, it is unmistakably clear that the stable sizes of the bank material must be much higher than the existing bank material at those selected 20 reaches of the Illinois River.

Waterway Traffic-Generated Waves

Commercial or pleasure crafts traveling in any waterway may generate waves which may be detrimental to the banks of the waterway. The Illinois

Table 6. Measured and Computed Median Diameter of the Bank Materials Considering Wave Action Generated by Wind in the Direction of Maximum Fetch

Reach	Wind characteristics: Climatological station	Month	Wind velocity, * V_w (fps)	Wind direction	Fetch in the direction of wind, F, (ft)	Width, normal to the direction of fetch, (ft)
1	St. Louis	March	67.42	40 SW	2700	580
2	St. Louis	March	67.42	30 SW	3800	900
3	Springfield	March	95.32	45 NW	1100	700
4	Springfield	March	95.32	45 SW	2000	850
5	Springfield	March	95.32	52 SW	5700	420
6	Springfield	March	95.32	0 W	6000	500
7	Springfield	March	95.32	0 W	1900	500
8	Springfield	March	95.32	30 SW	4850	600
9	Springfield	March	95.32	40 SW	4000	500
12	Springfield	March	95.32	50 SW	8200	600
13	Springfield	March	95.32	50 SW	3600	500
14	Springfield	March	95.32	30 SW	1100	550
15	Moline	May	84.01	30 SW	1300	700
17	Moline	May	84.01	60 SW	4800	900
18	Moline	May	84.01	75 SW	4000	570
19	Moline	May	84.01	60 SW	2800	680
20	Moline	May	84.01	80 SW	1800	650
22	Urbana	March	61.0	75 SW	2300	500
23	Urbana	March	61.0	75 SW	4000	550
24	Urbana	March	61.0	60 NW	2800	3000

Reach	Effective fetch, F_e, (ft)	Significant wave height, H_s, (ft)	Bank slope along the direction of fetch, (degrees)	Median weight of the stable riprap, W_{50} (pounds)	Equivalent median diameter of the stable riprap, d_{50} (inches)	Average existing median diameter of the bank materials, d_{50} (inches)
1	1131	1.13	1.7	0.36	1.9	0.00053
2	1688	1.34	3.7	0.68	2.4	0.00083
3	884	1.50	3.8	0.95	2.7	0.0020
4	1262	1.75	3.0	1.45	3.1	0.0044
5	1256	1.75	6.4	1.77	3.3	0.00064
6	1424	1.84	3.2	1.71	3.2	0.00029
7	899	1.51	1.5	0.85	2.6	0.00085
8	1459	1.86	3.8	1.83	3.3	0.010
9	1211	1.72	2.6	1.33	3.0	0.062
12	1800	2.04 .	5.1	2.61	3.7	0.00072
13	1161	1.69	5.1	1.47	3.1	0.0027
14	765	1.41	4.2	0.81	2.5	0.0030
15	945	1.34.	0.8	0.57	2.2	0.0061
17	1853	1.79	1.7	1.44	3.1	0.0042
18	1310	1.54	4.5	1.08	2.8	0.0051
19	1262	1.52	1.0	0.85	2.6	0.0056
20	1030	1.39	1.5	0.67	2.4	0.010
22	970	0.94	1.2	0.20	1.6	0.0051
23	1282	1.06	1.0	0.29	1.8	0.032
24	2800	1.49	9.5	1.38	3.0	0.67

** 6-hour duration and 50-year return period*

Table 7. Measured and Computed Median Diameter of the Bank Materials Considering Wave Action Generated by Wind in the Direction Normal to the Bank

	Wind Characteristics				
Reach	*Cltmatological station*	*Month*	*Wind velocity,* υ_w (fps)*	*Wind direction (degrees)*	*Fetch in the direction of wind, F (ft)*
1	St. Louis	March	67.42	62 NW	580
2	St. Louis	March	67.42	73.5 NW	750
3	Springfield	March	95.32	71.5 SW	600
4	Springfield	March	95.32	63 NW	800
6	Springfield	March	95.32	30 SW	600
13	Springfield	March	95.32	30 SW	600
14	Springfield	March	95.32	80 NW	520
15	Moline	May	84.01	67 NW	700
18	Moline	May	84.01	30 SW	600
22	Urbana	March	61.0	0 S	550
24	Urbana	March	61.0	72 NW	1550

Reach	*Significant wave height, H_s (ft)*	*Bank slope along the direction of fetch, (degrees)*	*Median weight of the stable riprap, W_{50} (pounds)*	*Equivalent median diameter of the stable riprap, d_{50} (inches)*	*Average existing median diameter of the bank materials, d_{50} (inches)*
1	0.84	8.4	0.23	1.7	0.00053
2	0.94	10.4	0.37	2.0	0.00083
3	1.27	9.0	0.81	2.5	0.0020
4	1.43	7.7	1.08	2.8	0.0044
6	1.27	15.1	1.32	3.0	0.00029
13	1.27	7.5	0.72	2.4	0.0027
14	1.19	11.5	0.81	2.5	0.0030
15	1.17	4.5	0.48	2.1	0.0061
18	1.10	8.4	0.50	2.2	0.0051
22	0.74	6.4	0.13	1.4	0.0051
24	1.15	11.6	0.75	2.5	0.67

** 6-hour duration and 5-year return period*

River is one of the major waterways of the Midwest, and it carries a tremendous amount of barge traffic in addition to the pleasure craft.

As far as it is known to the authors, no field data have been published related to the distribution and magnitudes of waves generated by barge traffic in a waterway. Some laboratory data have been reported by Das (1969) and Sorensen (1973). Bhowmik (1976) collected a very limited amount of boat-generated wave data from a lake and has developed a relationship for computing the maximum wave height.

Karaki and vanHoften (1974) described the various principles involved in the generation of waves by passing river traffic especially in the Illinois and Upper Mississippi Rivers. For that report, no theoretical analysis was made and no field data were collected. A number of color aerial photographs were shown depicting the pattern and the type of waves generated by waterway traffic.

Johnson (1976) and Karaki and vanHoften(1974) discussed the effect of barge traffic on the resuspension of the sediment particles with an associated increase in turbidity and its effect on the dissolved oxygen concentrations in the Illinois and Upper Mississippi Rivers. Liou and Herbich (1977) developed a numerical model to study the sediment movement in a restricted waterway induced by a ship's propeller.

Figure 26 shows what happens to the velocity distribution in a river just upstream, underneath, and downstream of a moving boat. The hydraulic forces that a channel bank and bed must withstand

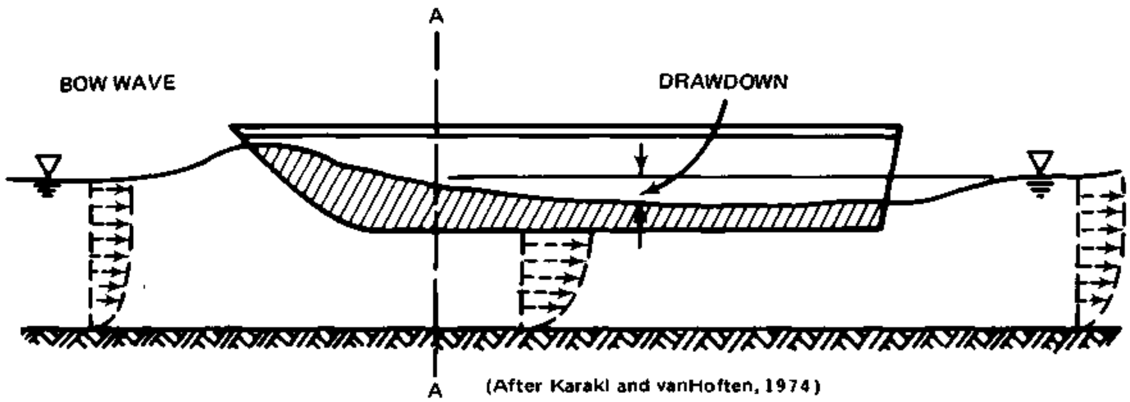


Figure 26. Surface disturbances created by boats

during the passage of a barge for deep, normal, and shallow channel depths are shown in figure 27. For shallow water flow, the lateral and longitudinal flow velocity underneath a moving barge must increase tremendously causing even more scouring of the bed and erosion of the banks. However, field data are needed before any definitive type of analysis or statement can be made regarding the potential of barge traffic on the scouring of the bed or the erosiveness of the banks.

RECOMMENDED MONITORING AND RESEARCH PROGRAMS

Monitoring Program

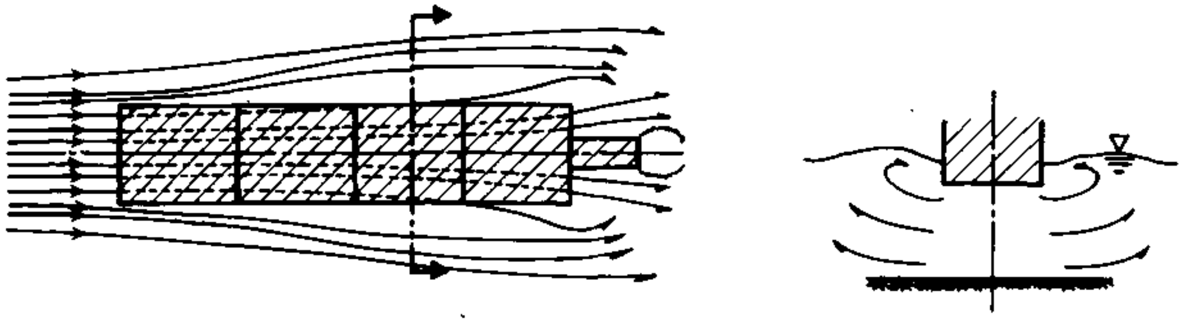
The authors recommend that a monitoring program be undertaken to document any future changes in bank erosion along the Illinois River. Locations recommended for monitoring are the 20 locations selected for this study (see figure 3). These reaches represent a set of severely eroded banks along the river and have already been well documented with permanent concrete monuments. Baseline data, such as plan view and bank slope, are available for the 1978 conditions. The proposed monitoring program would entail the following.

1) Resurvey all 20 reaches selected for the present investigation, determine the plan view and bank slopes for each reach, and collect representative bank material samples from each reach.
2) Compare the newly developed plan views and the measured bank slopes with the original set of data collected in 1978, determine the rate of erosion, and compare samples of bank material composition to check for any changes or variations.
3) Reanalyze the stability of the banks at selected reaches showing marked changes.
4) Make an attempt to postulate the probable changes in the rate or nature of bank erosion along the Illinois River from the original (1978) and the new data.
5) If new information or data are available relating to the characteristics and nature of waves generated by the waterway traffic, incorporate these data with the stability analyses.

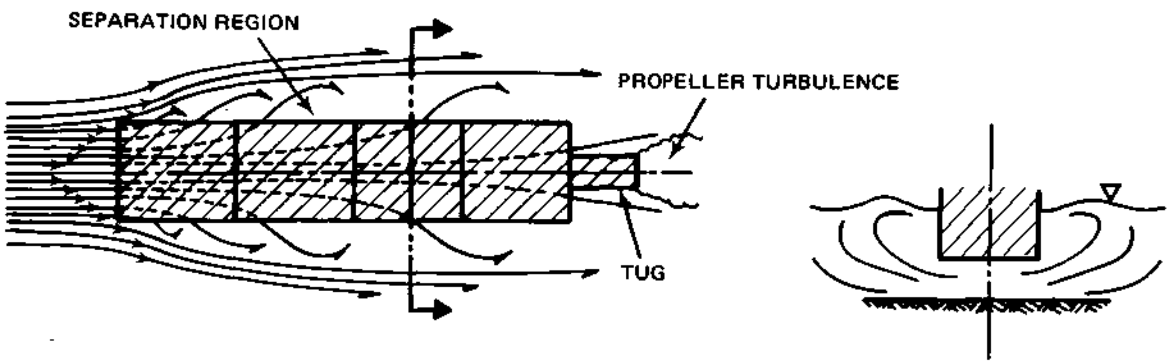
Future Research

Data presented in this report document that severe bank erosion occurs along the Illinois River. The normal flow characteristics of the river may or may not be responsible for the bank erosion. Present analysis of the data indicates that wave action generated by wind and/or waterway traffic, may be the main cause of erosion.

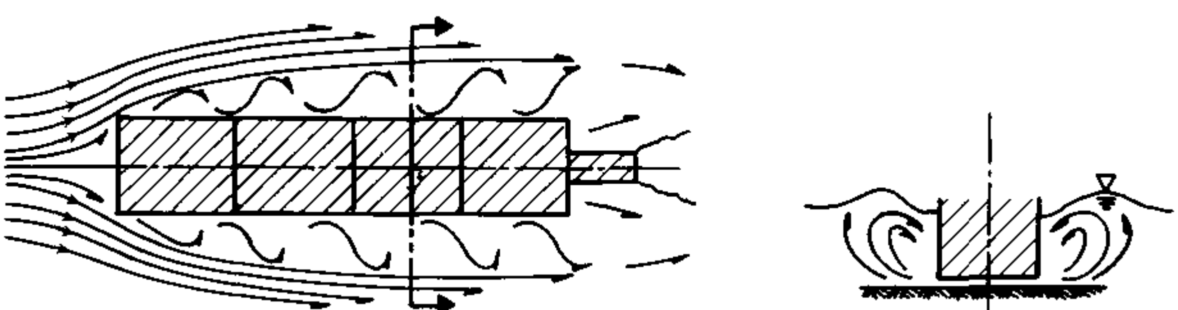
The nature and characteristics of waves generated by these two factors are not necessarily the same. An extensive literature search tells us that very little basic information exists regarding waves generated by waterway traffic and its potential for



CASE 1 -- DEEP WATER



CASE 2 -- NORMAL DEPTH



(After Karaki and vanHoften, 1974)

CASE 3 - SHALLOW DEPTH

Figure 27. Acceleration of flow and turbulence created by tow boats

river bank erosion. Furthermore, waves generated by wind in an inland stream, their interaction with flow velocity, confinement of the waterway, and relative interdependence between these parameters are not well understood.

Future research objectives should be 1) to collect data on waves generated by river traffic and winds on the Illinois River and 2) to determine bank erosion potential of these waves and to suggest preventive measures against destructive action of the traffic- and wind-generated waves.

It is recommended that four representative reaches of the Illinois River be selected for study. Wave data at each reach should be collected and analyzed to determine amplitudes, periods, energy spectrum, and other relevant parameters. Correlations between speed of the river traffic; distance of the sailing line from the bank; the width, length, and draft of the vessels; and wave parameters such as maximum wave height or significant wave height should be developed. Consideration of the wave characteristics, mechanics of flow in the river, sediment transport, nature of the bed and bank materials, geology, and other pertinent parameters are essential in the development of a methodology for protecting and/or preventing future stream bank erosion. Results of this future study could have a broad spectrum of application relating to waterway traffic-generated waves in inland waterways, intercoastal waterways, and in some cases in lakes.

SUMMARY AND CONCLUSIONS

Erosion of the stream bank attracts public attention, reduces property value, results in permanent loss of real estate, increases the turbidity of the stream, and accelerates the silting of reservoirs or backwater lakes along the stream course. Banks of any stream or river flowing through noncohesive or partly cohesive materials will erode if natural or artificial protection is lacking. Bank erosion along the Illinois River ranges from negligible to severe. The normal flow characteristics, changes in the flow regime,-and water wave action in the river all initiate and sustain the bank erosion.

The present investigation of bank erosion along the Illinois River was initiated to study the probable effects of increased diversion from Lake Michigan. A boat trip was taken to document and select 20 eroded reaches of the Illinois River for study. A total of 67 bank material samples and 54 bed material samples were collected and analyzed to determine the particle size distribution of the materials. Present plan views and bank slopes were surveyed and a permanent concrete monument was installed at each reach for future monitoring.

On the basis of present and anticipated flow conditions and of measured and estimated hydraulic parameters, bank stability analyses at each study reach were made following different accepted procedures. Stability analyses indicate that as far as the flow hydraulics are concerned, bank erosion along the Illinois River will not be affected by the proposed increase in diversion. In all probability, the main cause of the bank erosion of the Illinois River is the wave action caused by the wind and/or waterway traffic.

A future monitoring program is proposed to document and monitor areas of bank erosion along the river at a few selected locations.

A research project is also suggested to investigate the effects of waves on the stability of the banks. The two types of waves that should be studied are the wind-generated and waterway traffic-generated waves.

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