Illinois State Water Survey Division

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CHANNEL SCOUR INDUCED BY SPILLWAY FAILURE
AT LAKE CHARLESTON, ILLINOIS

by
Misganaw Demissie, William C. Bogner,
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INTRODUCTION

Lake Charleston serves as the sole source of water supply to the City of Charleston and Eastern Illinois University. The lake was initially created by construction of a dam across the Embarras River in 1947. The dam consists of two earth embankments and a 420-foot-long overflow spillway. The original capacity of the lake was estimated to be 2,283 acre-feet (ac-ft). One-foot wooden flashboards were installed on top of the spillway crest in 1964 to replace some of the lake capacity lost due to sediment accumulation. In 1982 the City of Charleston constructed a dike across the lake to form a side channel reservoir which could hold water at an elevation 8 feet higher than the old lake. This increased the available water supply. At the same time, the flow of the Embarras River was diverted away from the side channel reservoir in order to reduce the sedimentation rate.

Water is pumped from the old lake bottom outside of the dike to the side channel reservoir to maintain adequate water supply for the city. The diversion channel was built from the upper reaches of the lake to the lower reaches just upstream of the spillway to reroute the Embarras River flow past the side channel reservoir.

In November 1985 parts of the spillway which controlled the water level in the old lake outside the side channel reservoir failed during a moderate flood event. The spillway failure resulted in formation of a large "sinkhole" (scour hole) at the upstream side of the spillway. The sinkhole gradually migrated upstream from the spillway to the diversion channel, which is very close to the dike in several places. The upstream migration of the scour hole formed a deep channel much lower than the original lake bottom. The portion of the old lake outside the side channel reservoir was drained, with the streamflow confined in the new channel.

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This exposed the pump intake used to supply water to the side channel reservoir.

As a result of the spillway failure and the subsequent events the city was faced with two major problems. The first and most immediate was the possible instability of the dike which forms the side channel reservoir. The second problem was the replacement or rebuilding of the spillway to maintain sufficient water depth to submerge the water intake structure that replenishes the side channel reservoir with water.

If the dike fails, the city will be without water and will have to find an alternative source of water to maintain minimum service. The stability of the dike is threatened by the deepened channel created by the upstream migration of the sinkhole, and by high flows in the Embarras River. As the new channel deepens and migrates upstream, the stability of the dike will be reduced if the deep channel approaches the foundation of the dike. Since the dike was designed to hold water about 8 feet above the old lake level, any additional hydraulic head differential induced by the lowering of the Embarras River bed will increase the hydrostatic pressure on the dike and increase seepage through and under the dike.

Another hydraulic force that will threaten the stability of the dike is the flow of the Embarras River, which flows directly towards the dike near the lower end of the diversion channel. During low flow periods this should present little problem. However, during high flows it should be expected that the dike will be subjected to higher flow velocities. If the high flows in the Embarras River generate high enough velocities to scour the dike embankment and its foundation, the stability of the dike will be affected significantly and dike failure may result.

This project was initiated to investigate the physical processes that took place after the spillway failure and to evaluate the potential for further problems associated with the scour channel development. It was believed that the problems facing the City of Charleston could happen at other places; therefore the need exists to evaluate and document what occurred at Lake Charleston to provide a scientific basis for emergency planning at places with potential similar problems.

Realizing that the situation at Lake Charleston provided a rare opportunity to investigate such phenomena under natural conditions, Water Survey engineers prepared a proposal to the Department of Energy and

Natural Resources for partial funding to conduct the study. Funding for the project was approved immediately, resulting in the study and this report.

<div align="center">BACKGROUND</div>

Location

Lake Charleston is located on the Embarras River in Coles County in east-central Illinois. The main stem of the Embarras River originates in the City of Champaign and flows in a southerly direction to Lake Charleston (figure 1). The drainage area of the Embarras River upstream of the Riverview Dam at Lake Charleston is 786 square miles and drains parts of the predominantly agricultural counties of Champaign, Vermilion, Douglas, Edgar, and Coles in Illinois. There are no other major lakes or dams in the watershed upstream of Lake Charleston.
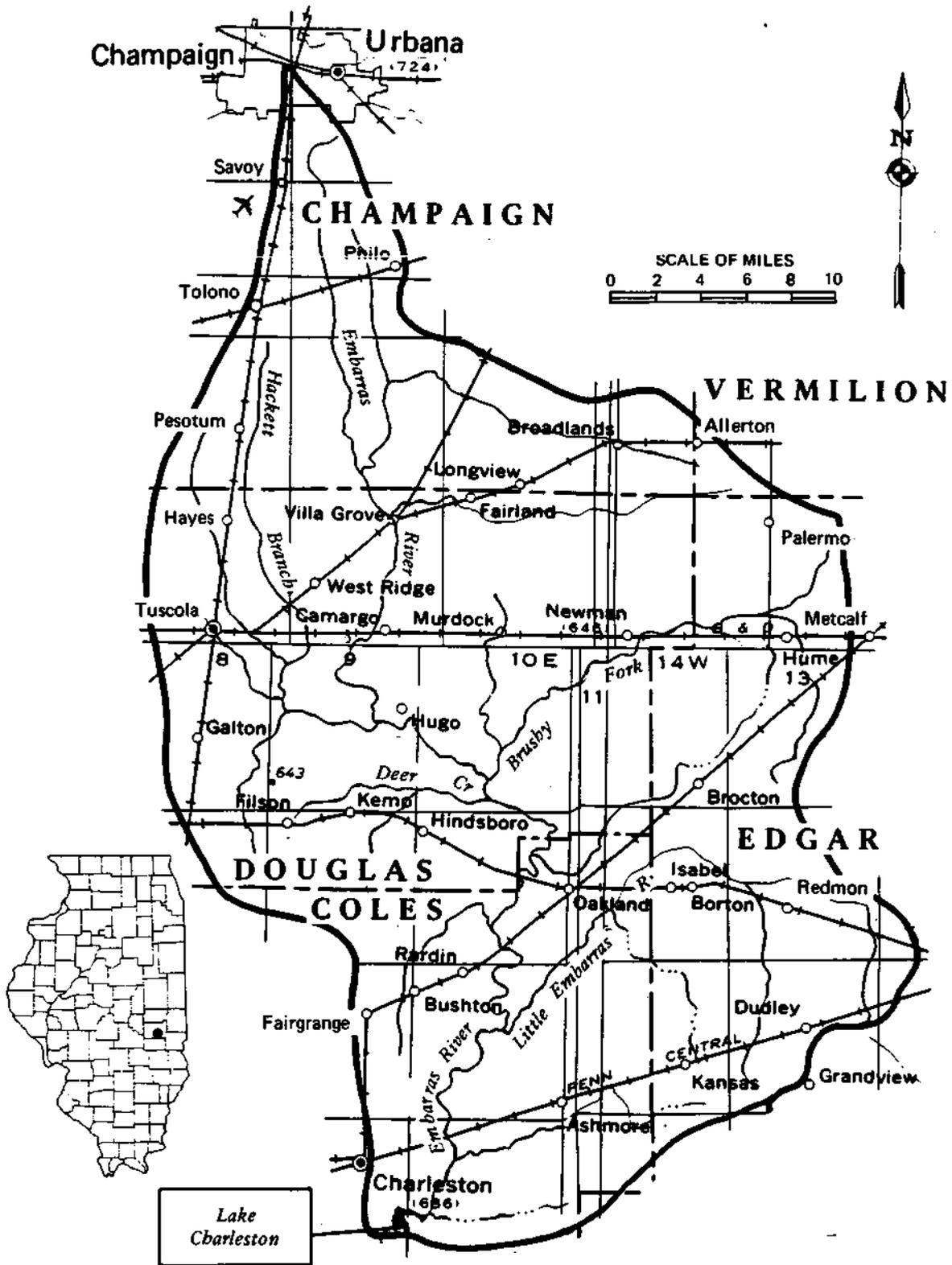
Figure 1.   Embarras River watershed upstream of Lake Charleston

Dam and Spillway

Lake Charleston was created by construction of the Riverview Dam across the Embarras River (R.M. 118.8) in 1947. The Riverview Dam consists of two separate segments known as "Dam A" and "Dam B" as shown in figure 2. The two dams are separated by natural high ground in the middle of the Embarras River valley. They are earth embankment dams with a total length of 1,190 feet. "Dam A" is about 370 feet long and 29 feet high while "Dam B" is about 820 feet long with a maximum height of 36 feet (USCOE, 1980).

A 420-foot-long uncontrolled overflow spillway is located along "Dam B" as shown in figure 2. A typical cross-sectional view of the spillway is shown in figure 3. The spillway structure consists of an approach apron on the upstream side, a concrete weir at the middle, and 27- and 18-inch-thick slabs on the downstream slope of the spillway. The crest of the spillway at the concrete weir is 579.6 ft msl, and the stilling basin at the downstream end of the spillway is at 557 ft msl. The downstream slope of the spillway below the weir structure is 2.5 to 1 (horizontal to vertical). Under the concrete weir, sheet pilings were driven to an impervious stratum to prevent seepage under the spillway. In 1964 1-foot-high wooden flashboards were installed on top of the weir crest to increase the capacity of the lake. The normal water elevation was thereby increased to 580.6 msl.

Since the lake is primarily used as the sole source of water supply for the city of Charleston, the operation of the spillway is not affected by any other requirements. Water is allowed to freely overflow the wooden flashboards whenever the lake level gets above mean pool elevation.

A 1980 field investigation report for the national dam safety program by the U.S. Army Corps of Engineers (1980) indicated several possible problems related to the spillway. These included cracks on the spillway slope wall and seepage through some of these cracks. A prior report by Hanson Engineers (1977) stated that "significant voids" were detected under the spillway slabs at several core locations in 1966. Displacement of spillway slabs was also detected at several joints on the spillway below the concrete weir which forms the spillway crest. The displacement of the joints and the cracks on the spillway slabs were thought to be related to the voids beneath the spillway slabs (USCOE, 1980; Hanson Engineers, 1977). Partial attempts were made to fill the voids underneath the slopewall with grout after the 1966 inspection. Five- to seven-foot voids were reported
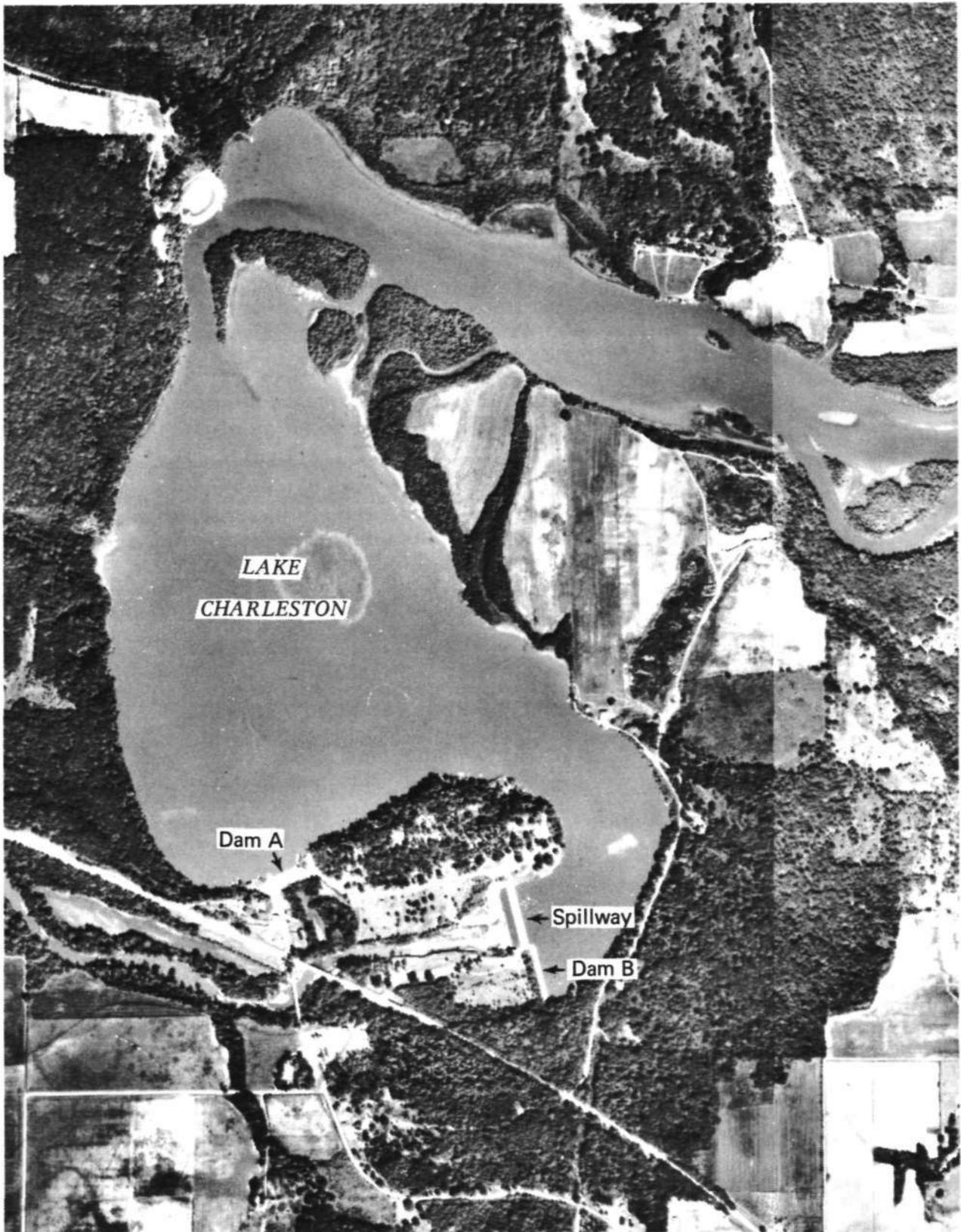
Figure 2. The Riverview Dam and Lake Charleston
before construction of side channel reservoir and dike
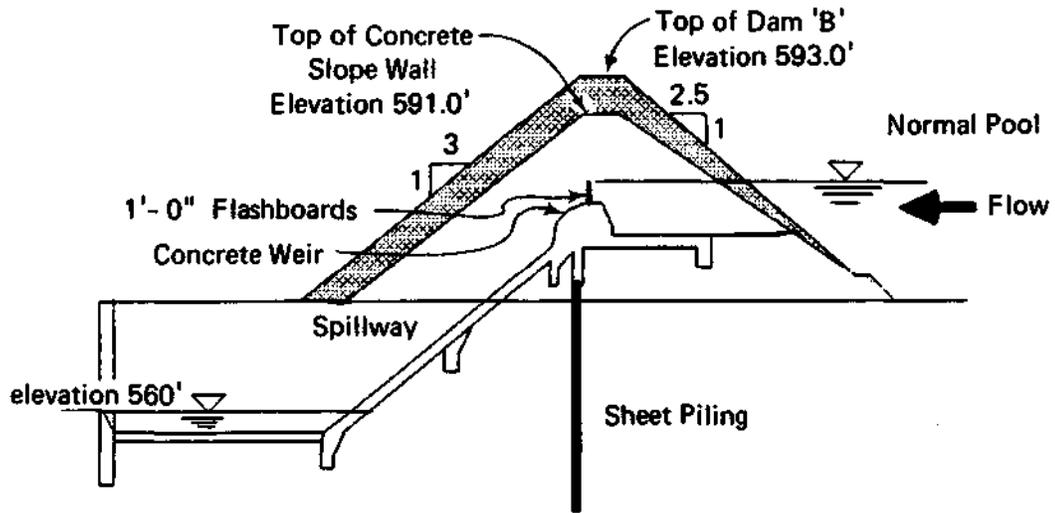
Figure 3. Cross-sectional view of spillway and "Dam B"

to have been encountered during the grout operation; however, the voids were not completely filled with grout at that time.

Lake Sedimentation

Lake sedimentation has been a major problem in managing Lake Charleston. The loss of lake capacity due to sedimentation and increased demand for water led to the installation of the 1-foot wooden fiashboards in 1964 and the construction of a dike to create a side channel reservoir in 1982.

The Illinois State Water Survey conducted two lake sedimentation surveys of Lake Charleston, one in 1960 and another one in 1974 (Yang, 1974). A limited survey of the side channel reservoir was also conducted by the Water Survey just prior to the completion of the dike in 1982. The rate of sedimentation based on the two complete surveys (1960 and 1974) is summarized in table 1.

In 27 years, from 1947 to 1974, the lake lost 1,418 ac-ft of its original capacity at an average annual rate of 53 ac-ft. By 1974 the lake had lost about 62 percent of its original volume. The sedimentation rate was therefore very high and the storage capacity was being depleted very rapidly.

In spring 1986 a series of cross sections in Lake Charleston were surveyed. Further analyses and observations can be made about sedimentation rates and patterns in Lake Charleston on the basis of the previous

Table 1. Sedimentation Survey Results Up to 1974

Year	Lake capacity* (ac-ft)	Capacity loss (ac-ft)	Sedimentation rate (ac-ft/vear)
1947	2,283		
1960	1,395	888	68
1974	865	530	38
1947-1974		1418	53

* Note: Lake capacity was computed for a mean pool elevation of 580 ft msl. The installation of the 1-foot wooden fiashboards increased the mean pool elevation by 1 ft to 581 ft msl and the lake capacity by 404 ac-ft. The 1974 capacity with the fiashboards was therefore 1269 ac-ft.

surveys and the survey conducted in 1986. This is important since the history of sedimentation in the lake might otherwise be misinterpreted because of the relatively small amount of sediment left in the lake after the spillway failure. Much of the unconsolidated sediment which accumulated in the lower reach of the lake just upstream of the spillway was washed out during and after the spillway failure. Thus the sediment which remained on the old lake bottom after the spillway failure (figure 4) is not a true measure of the past sedimentation rate in the lake.

The variations of the sedimentation rates within the lake and the lower sedimentation rate in the lower reaches of the lake are illustrated by comparing the historical sedimentation rate of three segments of the lake. The three segments selected for this comparison are segment 2 in the lower lake, segment 4 in the middle of the lake, and segment 8 in the upper reaches of the lake (figure 5). Information on these three segments, taken from Illinois State Water Survey files, is given in table 2. Segments 2 and 8 each made up about 8 percent of the original lake volume, while segment 4 accounted for 24 percent of the volume. When comparing sediment accumulation in the three segments, shown in table 2, it is obvious that segment 4, which accounted for the larger portion of the lake volume, accumulated much more sediment than the other two segments. However, the sediment accumulation rate in segment 8, which is in the upper part of the lake, was almost twice that of segment 2, which is in the lower part of the lake, even though they had about equal original volume. In terms of average sediment thickness, the rate was the highest for the upper segment (segment 8) and the lowest for the lower segment (segment 2), with that of the middle segment being somewhere between the two.

In summary, the accumulation of sediment in the lake gradually decreased in the downstream direction, resulting in the lowest sedimentation rate at the lower-end segment. The middle section of the lake, whose sedimentation rate is represented by segment 4, made up about 70 percent of the lake's volume and the rate of sedimentation was high for that portion of the lake.

Therefore, the old Lake Charleston, as a whole, suffered extremely high sedimentation rates with the major sedimentation impacts occurring in the middle lake segments from 1947 to 1974. The area now occupied by the

Figure 4. Lake sediment remaining on the old lake bottom
after spillway failure
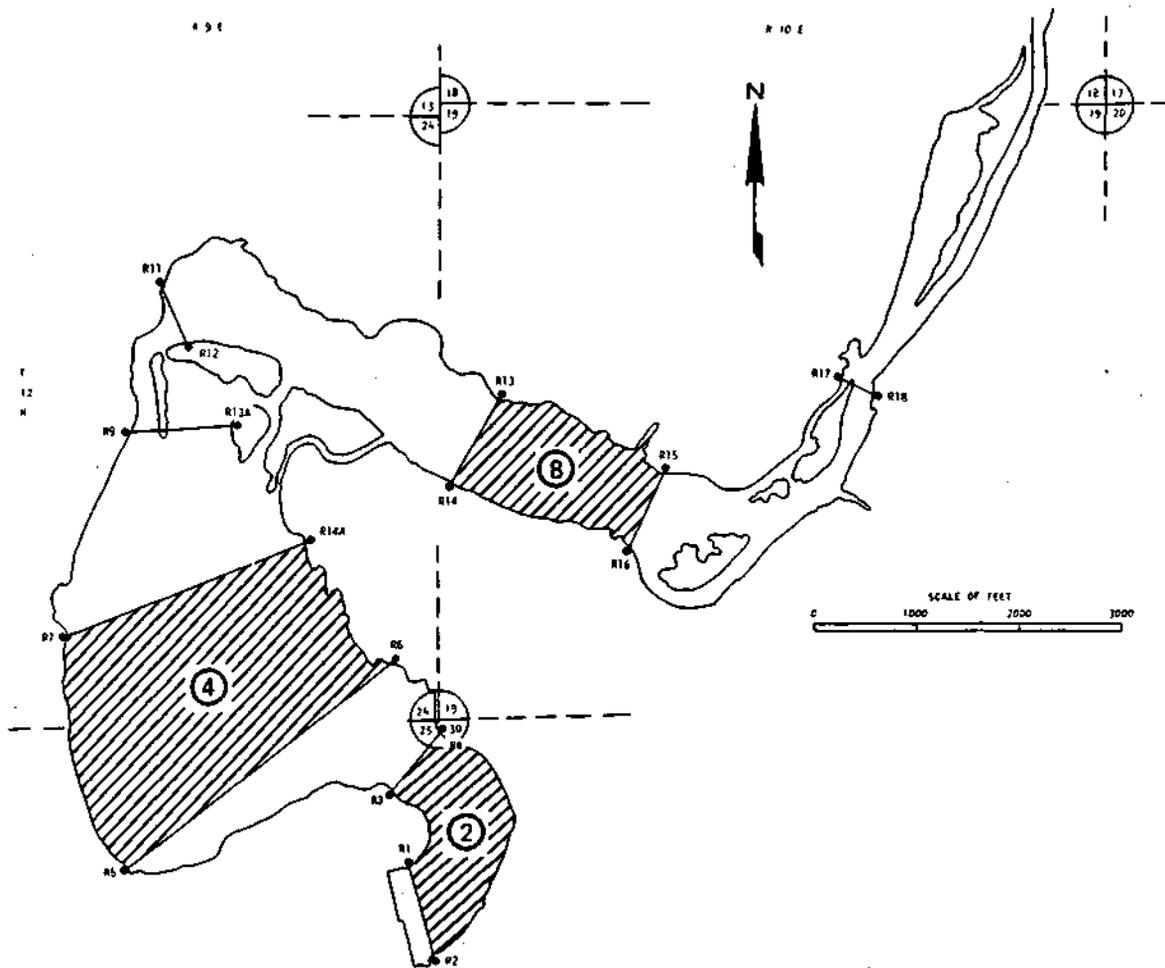
Figure 5. Locations of segments 2, 4, and 8 in the old Lake Charleston

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Table 2. Sediment Distribution in Lake Charleston

Segment number	Surface area (acres)	Original volume (ac-ft)	Percent of original lake volume	Sediment deposition Volume (ac-ft) 1960	1974	Thickness (ft) 1960	1974
2	25.8	172.3	7.5	34.4	73	1.3	2.8
4	102.1	553.7	24	181.3	371.3	1.8	3.6
8	32.3	183.7	8	89.7	132.4	2.8	4.1
Lake total	381	2283		888	1,418	2.3	3.7

scour channel would have had 2 to 3 feet of sediment accumulations when the spillway failure occurred. This sediment was washed out as the channel formed. Sediment scour also occurred in a diminishing pattern with lateral distance from the new channel.

Side Channel Reservoir

Because of lake capacity loss due to sedimentation and increased water demand, from 1975 to 1980 the city investigated various alternatives to increase the water storage capacity available to the city. These alternatives included building new reservoirs at several other locations in the area, and the idea of a side channel reservoir at the site of the old lake, which was finally selected and implemented in 1980-1982. The creation of the side channel reservoir involved building a dike across the lake as shown in figure 6 to isolate the western portion of the lake from the rest of the old lake, and constructing a diversion channel on the eastern side of the dike to divert the Embarras River flow directly to the spillway area. A photograph of the side channel reservoir, dike, and diversion channel is shown in figure 7.

The side channel reservoir was designed to hold water up to an elevation 8 feet higher than the mean pool elevation of the old lake as shown in figure 8. Water from the Embarras River is pumped to the reservoir to keep the water level at a desirable elevation (588 ft msl). The design capacity of the side channel reservoir is estimated to be 3680 ac-ft. With the expected reduction of the sedimentation rate in the side

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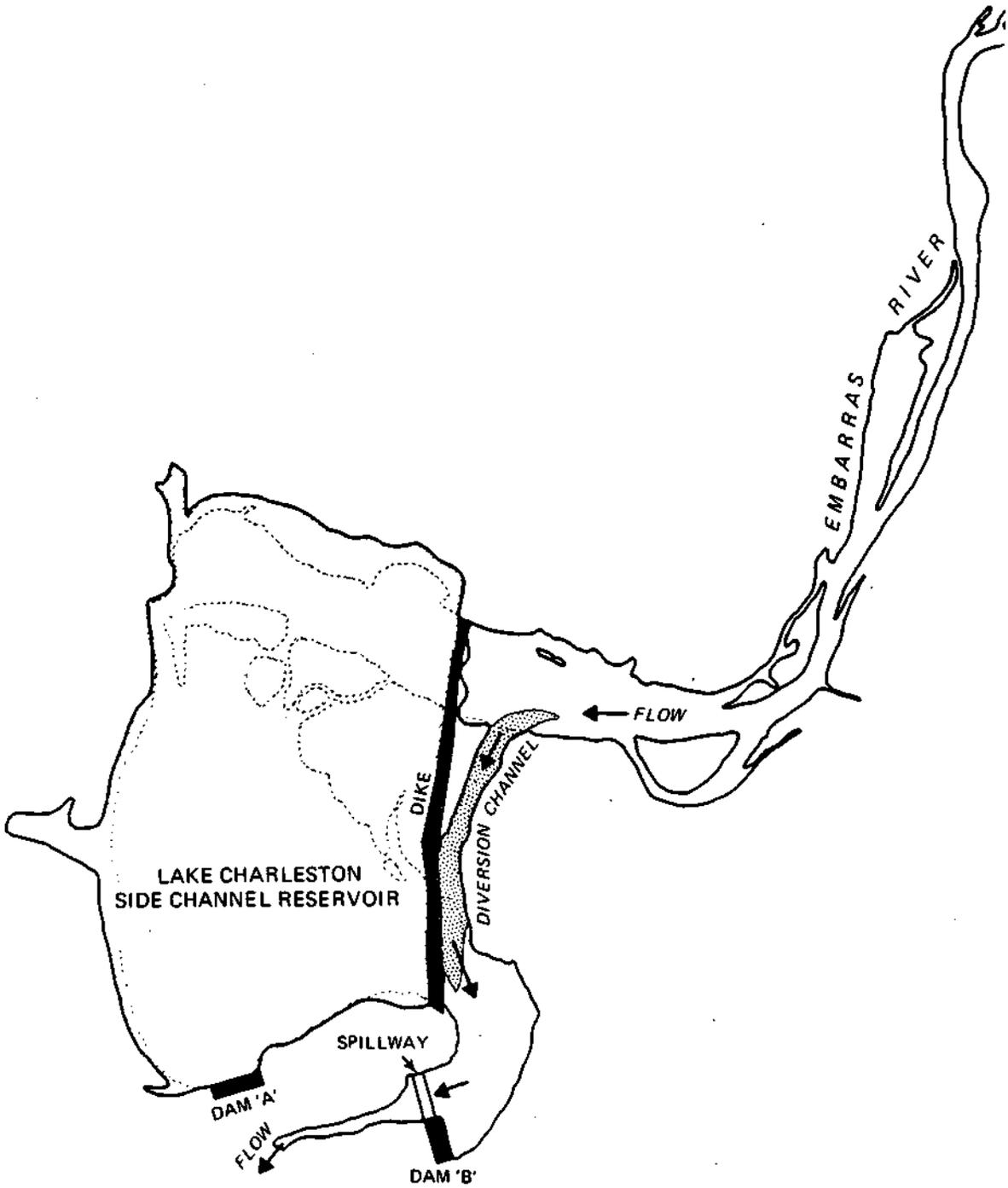
Figure 6. Side channel reservoir at Lake Charleston
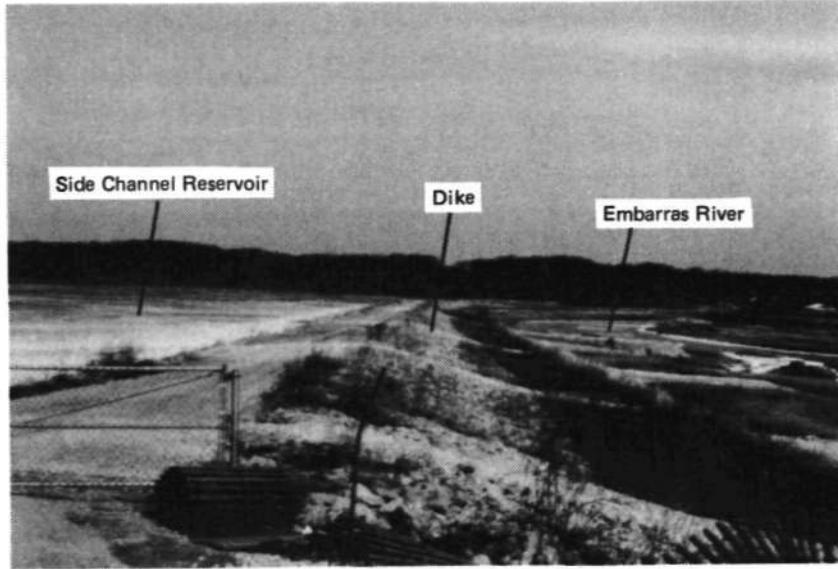
Figure 7. View of side channel reservoir, dike,
and diversion channel
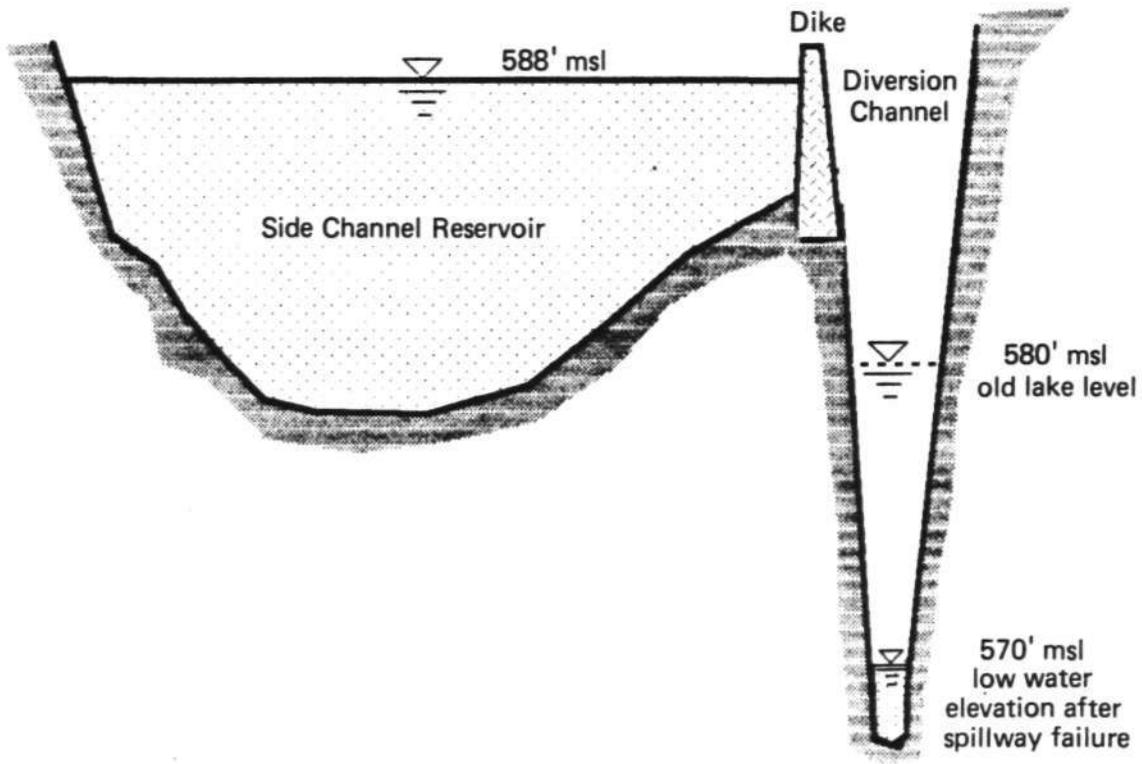
Figure 8. Cross-sectional view of side channel reservoir, dike,
and diversion channel

channel reservoir, the present capacity may provide adequate water supply for the city for an extended period of time.

Diversion Channel

The diversion channel shown in figure 6 was constructed to divert the flow of the Embarras River away from the dike and the side channel reservoir. It routes the stream flow towards the spillway. The profile of the diversion channel as built is shown in figure 9. The channel was not extended as far as the spillway but terminated close to the downstream end of the dike.

An important physical feature along the channel is the presence of a rock ledge for approximately 840 feet. The rock ledge played an important role in reducing the upstream migration of the scouring in the channel. During low and medium flows the rock ledge forms a similar flow pattern to that in riffles in a natural stream (figure 10). Even though some erosion of the rock ledge may have taken place as evidenced by rock fragments on the ledge, it has for the most part remained stable.

Spillway Failure

In late November 1985, parts of the spillway located along Dam B failed during a moderate flood. The initial sign of the spillway failure was the displacement of slopewall panels on the downstream slope of the spillway. The panels were eventually moved further downstream and the fill material underneath the spillway at the breach was eroded, exposing the sheet piling below. Figure 11 shows the spillway before and after failure. The scour hole which developed just upstream of the spillway, referred to as a "sinkhole," is shown in figure 12. As the scouring moved further upstream, the deep channel shown in figure 13 was formed.

A detailed record of the spillway failure, the development of the "sinkhole," and the upstream migration of the scour channel was kept by Dan England, Assistant City Engineer, City of Charleston. The record is presented below in its entirety. To assist in the interpretation of the record, figure 14 was prepared to indicate the location of the headcut at different times (shown by dates on the figure). The headcut is the upstream extent of the scour channel as it migrated from the spillway towards the diversion channel.
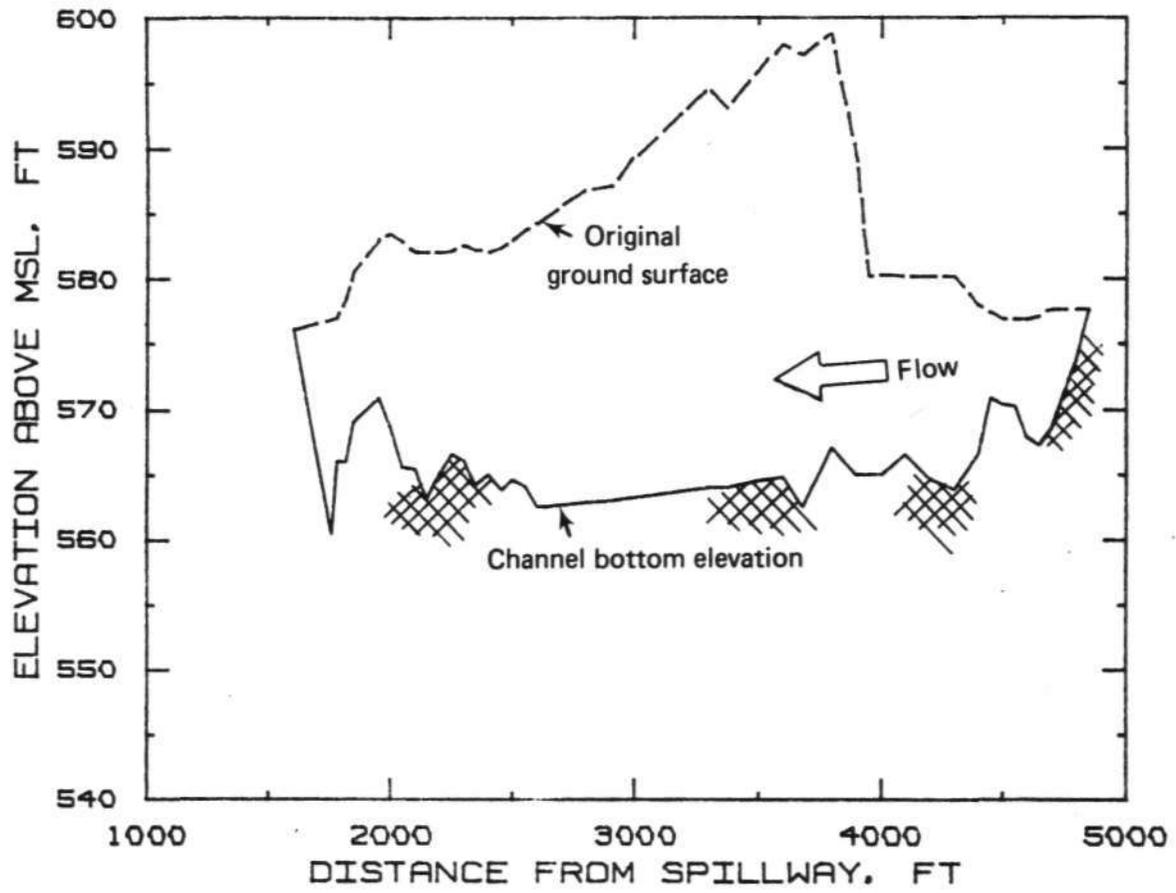
Figure 9.   Profile of the diversion channel in Lake Charleston
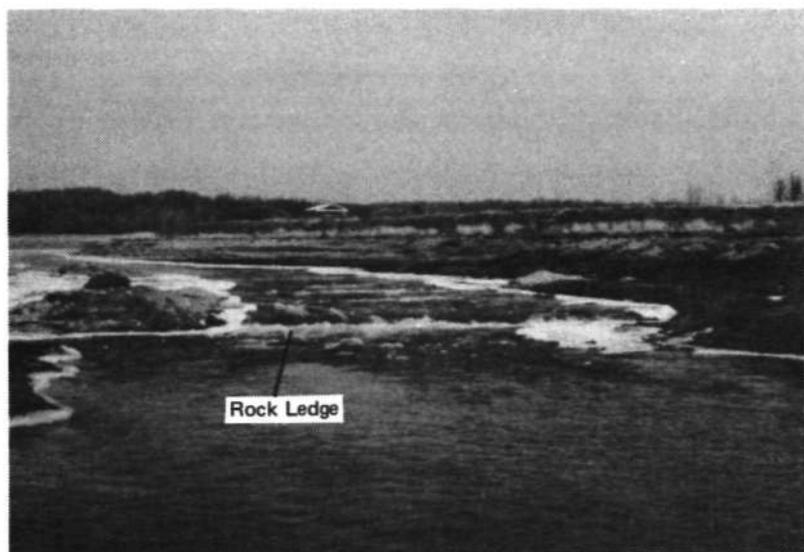
Figure 10.   Flow over the rock ledge in the diversion channel
at Lake Charleston (note riffles at rock ledge)

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SPILLWAY BEFORE FAILURE

SPILLWAY AFTER FAILURE
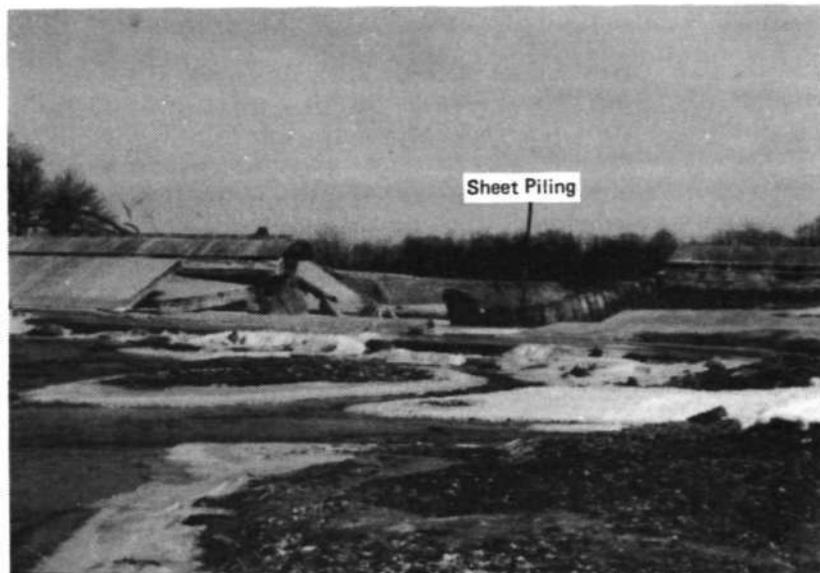
SPILLWAY AFTER FAILURE

Figure 11. Lake Charleston spillway before and after failure
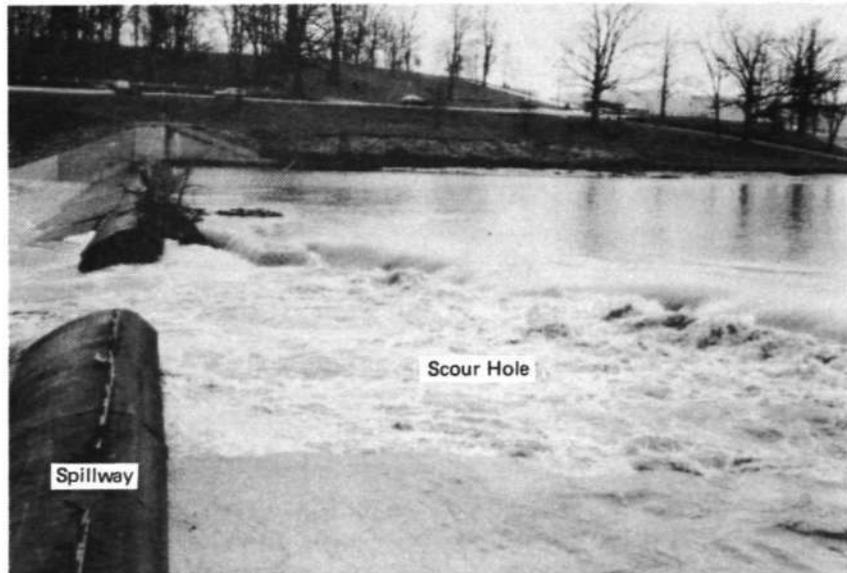
Figure 12. Scour hole just upstream of Lake Charleston spillway
after its failure
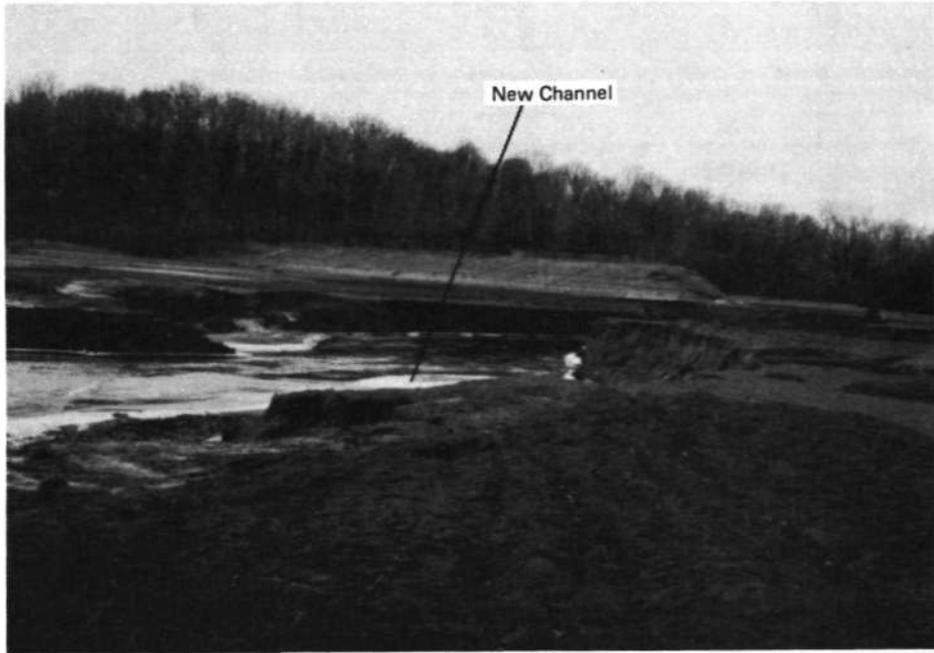
Figure 13. New stream channel which developed after the spillway failure at Lake Charleston
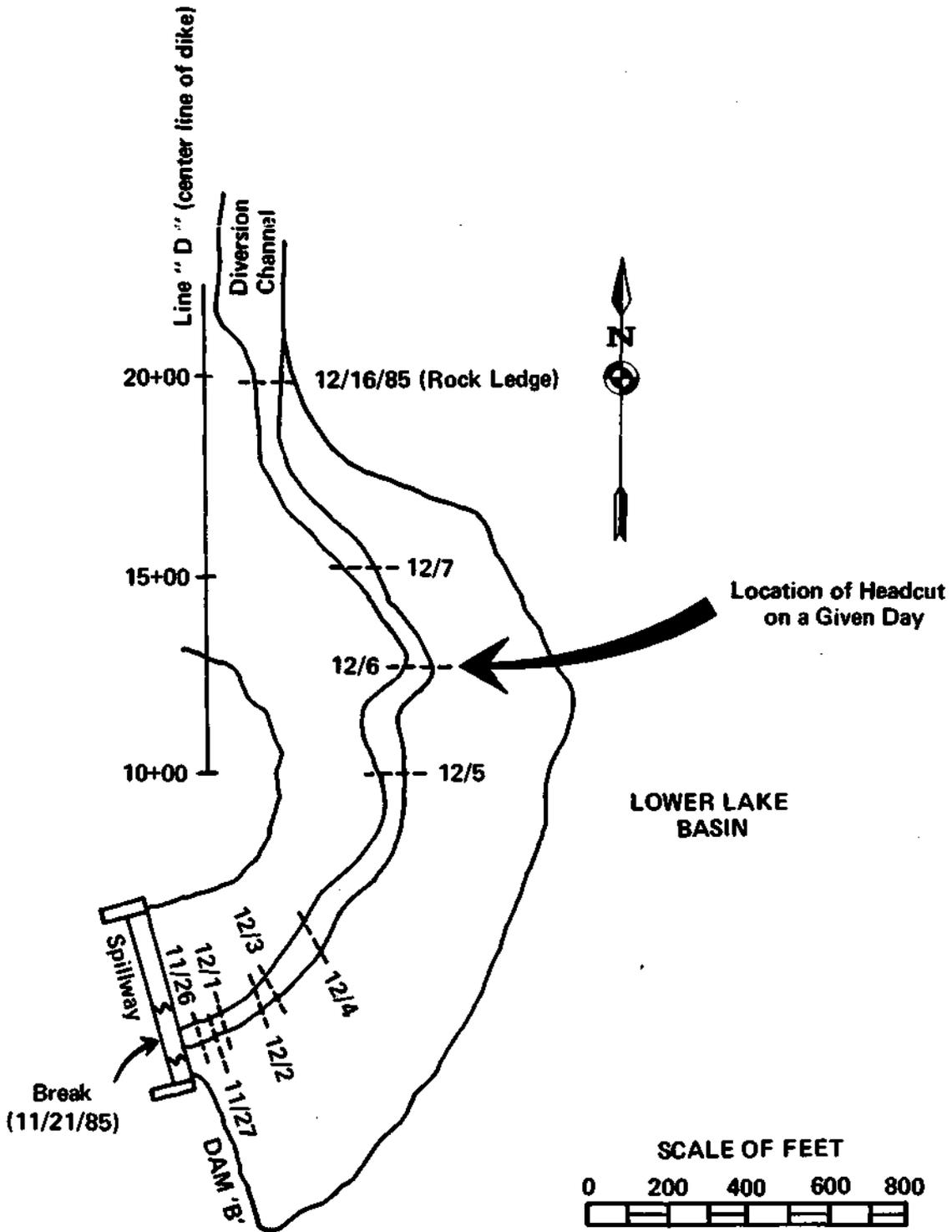
Figure 14. Location of headcut upstream of Lake Charleston spillway
at different times

Progression of Readout at Lake Charleston Spillway Collapse

11/21/85 - Slopewall panels (4) on downstream side of cap of spillway were displaced and standing erect in river current.

11/24/85 - Additional slopewall panel displaced

11/25/85 - 7:30 p.m. Sinkhole appeared behind crest of spillway approx. 80' in width and 30' upstream

11/26/85 - Headcut had moved to 39' upstream at 7:30 p.m.

11/27/85 - 2:10 a.m. - Crest spillway panels 3 & 4 from south end collapsed and sheet piling beneath opened up

8:10 a.m. - Headcut moved to 42' upstream of spillway

4:30 p.m. - Headcut moved to 75' upstream but had taken a south easterly course and was not following the river current flow. This measurement, the preceding measurements, and those to follow until otherwise noted are perpendicular to crest of spillway structure.

11/28/85 - 10:05 a.m. - Headcut at 77' from spillway and coned back to the north and traveling parallel with river current

4:10 p.m. - Headcut at 78' upstream of spillway

11/29/85 - 12:20 p.m. - Headcut at 88.5' upstream of spillway

3:30 p.m. - Headcut at 93' upstream of spillway

11/20/85 - 7:30 a.m. - Headcut at 94' upstream of spillway

3:00 p.m. - Headcut - same location

12/1/85 - 9:00 a.m. - Headcut still at 94' upstream of spillway

3:30 p.m. - Headcut 97' upstream of spillway

12/2/85 - 7:00 a.m. - Headcut moved to 179' upstream of spillway with new finger headed off toward pumaphouse. (This finger's growth eventually stopped because as the main area of headcut moved upstream there was no water or current to increase the new finger area.)

4:30 p.m. - Headcut 199' upstream of spillway

12/3/85 - Headcut at 250' upstream of spillway (4:30 p.m.)

        Note: river current upstream in river channel near station 24+00 on Line "D" is 1 to 2 feet per second/at Station 12+00 it is 5 to 6 feet per second.

12/4/85 - Began construction of finger dikes to keep fast moving current away from side channel dike and to attempt to keep headcut as far away from dike side or river as possible.

        4:30 p.m. - Headcut 370' from spillway

12/5/85 - 5:00 p.m. - Headcut at 1000' from spillway

        This measurement at right angle to spillway - measurements beyond this point will be referenced to distance from spillway but measured along the centerline of the headcut or referenced to Line "D" stationing - 1000' from spillway equates to Station 10+00 on Line "D".

12/6/85 - 4:30 p.m. - Headcut at right angle to Station 12+65, Line "D"

12/7/85 - 3:30 p.m. - Headcut at right angle to Station 15+10, Line "D"

12/8/85 - 3:00 p.m. - Headcut at right angle to Station 15+20, Line "D" and remained at this same location until 9:00 p.m. 12/16/85. During this 8 or 9 day time period while the headcut did not travel upstream, the resulting channel site did grow in width from around 30 to 40' wide at its mouth near Station 15+20 to 200+. Also on 12/18/85 construction of the 5 finger dikes and the stone ledge dam at Station 19+85, Line "D" were completed.

        Between 9:00 p.m. 12/16/85 and 7:30 a.m. 12/17/85 the headcut progressed to its present (2/5/86) location at the stone ledge dam Station 19+85, Line "D".

The rock ledge has effectively stopped the upstream migration of the scouring. There is evidence of fragmented rocks at the rock ledge indicating some erosion at the rock ledge. There is approximately 7 feet of drop in water surface elevation through the rock ledge.

FIELD DATA COLLECTION AND ANALYSIS

The collection of field data to analyze the upstream migration of the sinkhole was essential because of the dynamic nature of the problem and the need to quantify important physical and hydraulic parameters. The field data collection program included making regular field inspections of the

site, surveying the new channel with respect to existing lake sedimentation monuments and old surveying notes, measuring velocities and discharges under different flow conditions, and collecting and analyzing bed material samples. The results of the field data collection effort will be presented and discussed in this section.

Surveying

   A field survey was conducted of a series of 29 cross sections of the river/lake system from the Route 130 bridge to 500 feet upstream of the East Harrison Road bridge. The locations of the surveyed cross sections are shown in figure 15. These cross sections all followed the present river flow with the exception of cross section 14, which paralleled the side channel reservoir dike northwest of the diversion channel.

   In all cases, an attempt was made to locate these cross sections to correspond either to Water Survey lake sedimentation ranges which were surveyed in 1960 and 1974 (4 cross sections), or to the Embarras River cross sections surveyed by the Illinois Department of Transportation-Division of Water Resources (IDOT-DOWR) for a 1962 Interim Report on Flood Control and Drainage Development of the Embarras River (15 cross sections). Five cross sections of the diversion channel were located for comparison with design plans. Five cross sections had no previous history.

   The four lake sedimentation ranges were located by recovering existing survey markers and repeating the 1960 and 1974 surveys. Cross section 6 in the downstream lake basin was also used to locate the two new lines (cross sections 7 and 8) run in that basin as angles to the north of cross section 1. Cross section 14 required a slight shift in line on the south end due to interference with the side channel dike. Cross sections 17 and 21 were run between recovered monuments.

   The 15 cross sections used to repeat the 1962 DOWR Flood Control survey were relocated by eye in the field by using base maps prepared at the time of the survey. These cross sections were roughly located at the time of the 1986 survey by a stadia survey.

   The five cross sections within the diversion channel (cross sections 9 to 13) were surveyed by re-establishing the construction baseline of the side channel dike. Cross sections were surveyed between stations 22+00 and
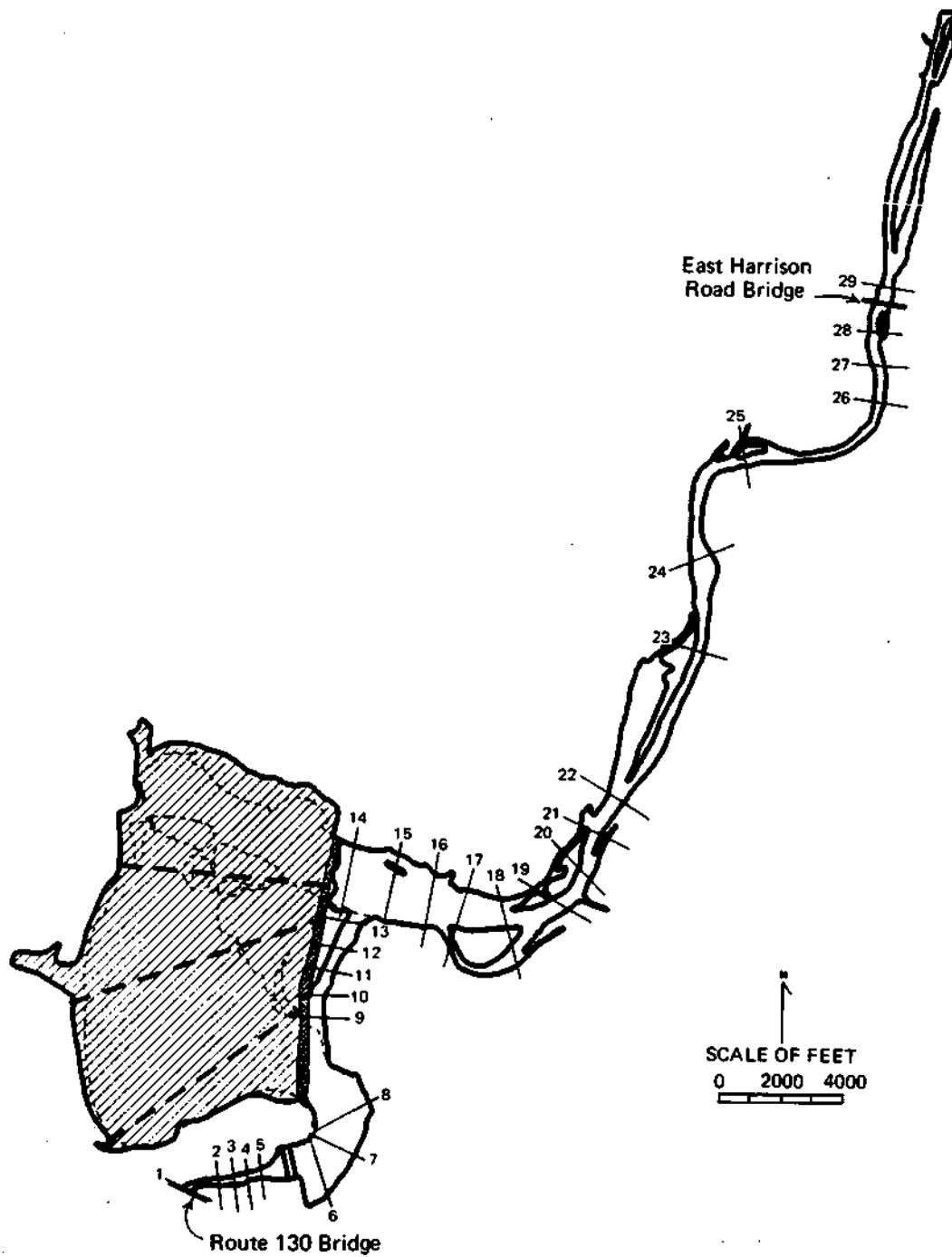
Figure 15. Locations of 1986 survey cross sections
along the Embarras River

38+00 on the design survey plans at 400-foot intervals. The survey lines were run perpendicular to the baseline in an easterly direction.

The five cross sections which did not have previous records included cross sections 7 and 8 just upstream of the spillway and three cross sections (cross sections 2 to 4) downstream of the spillway. These three cross sections were roughly tied with two of the DOWR survey lines in the area between Route 130 and the spillway. Cross section 1 was run immediately upstream of the highway bridge.

Similar survey methods were used for all cross sections. Cross sections were surveyed from the westerly bank of the river/lake system. Upstream of the diversion channel and downstream of the spillway, temporary elevation benchmarks were set and stadia traverses placed to locate survey lines. In the lower lake basin, a lake sedimentation survey monument was used for both line location and benchmark elevation. Along the diversion channel, the levee construction traverse was recovered and used for location.

Cross sections were surveyed as spurs off of the traverse line in two phases. The first phase consisted of a stadia and level survey from the traverse line to the river channel where a pin was placed to locate line stationing and serve as a temporary benchmark. The second phase consisted of a water depth survey in the channel and any complementary leveling necessary on the easterly side of the channel.

The profiles of the 29 cross sections that were surveyed are given in Appendix A.

Velocity and Discharge Measurements

Velocity measurements were needed to evaluate the potential of the Embarras River in eroding the dike embankment and its foundation. Since the Embarras River diversion channel is adjacent to the dike and there is a gradual bend around the site, it was felt that high velocities might be generated next to the dike and threaten its stability. Previous studies on velocity distribution in a natural channel (Kaskaskia River) conducted by Bhowmik (1979) showed that the maximum velocity in a channel cross section tends to migrate towards the outer bank of a stream channel during high and low flows. Similar tendencies are also expected in the diversion channel near the dike that forms the side channel reservoir.

Velocity and discharge measurements were made in May and June 1986 to investigate the velocity distribution pattern in the diversion channel. The initial plan was to measure velocity and discharge during low, medium, and high flows. However, because of the short duration of the proj ect and the absence of a major flood (which is fortunate for the city), flow measurements were taken only during medium flows.

Five velocity measurement cross sections were established on the diversion channel. The cross sections were set to be perpendicular to the traverse of the dike as shown in figure 16. The five cross sections are identified in figure 16 as 2300, 2400, 2600, 2800, and 2900. On May 2, 1986, velocity measurements were made at all five cross sections. The average discharge during the measurements was calculated to be 848 cfs. On June 11, 1986 velocity measurements were made at four of the five cross sections. The average discharge for this set of measurements was computed to be 1258 cfs. The positions of these flows in the flow duration curve for the Embarras River at Charleston are shown in figure 17. The 848 cfs is exceeded 21 percent of the time while the 1258 cfs is exceeded 14 percent of the time on the average. Both of the discharges are greater than the mean flow, which is expected to be exceeded 25 percent of the time.

The velocity data were collected according to the procedures described by Buchanan and Somers (1969) for stream gaging. The instrument used was a standard Price-type current meter with a 30-pound weight suspended on a cable from an A-reel, which was mounted on a work boat. The boat was positioned at different distances from shore along the cross section and held in position by a cable strung across the channel. The distance across the channel was measured from markings on the cable. Velocities were measured at 0.2 and 0.8 of the total depth at each vertical. The average velocity at the vertical was then determined by finding the mean of the two readings.

The velocity distributions determined from the measurements made on May 2 and June 11 are presented in figures 18 to 26. The maximum veloci-ties range from 2.3 to 3.3 fps for the flow ranges measured during the field trips. A complete listing of the measured velocities is presented in Appendix B. During the measurements in May the velocities were generally evenly distributed across the flow area without a very distinct lateral

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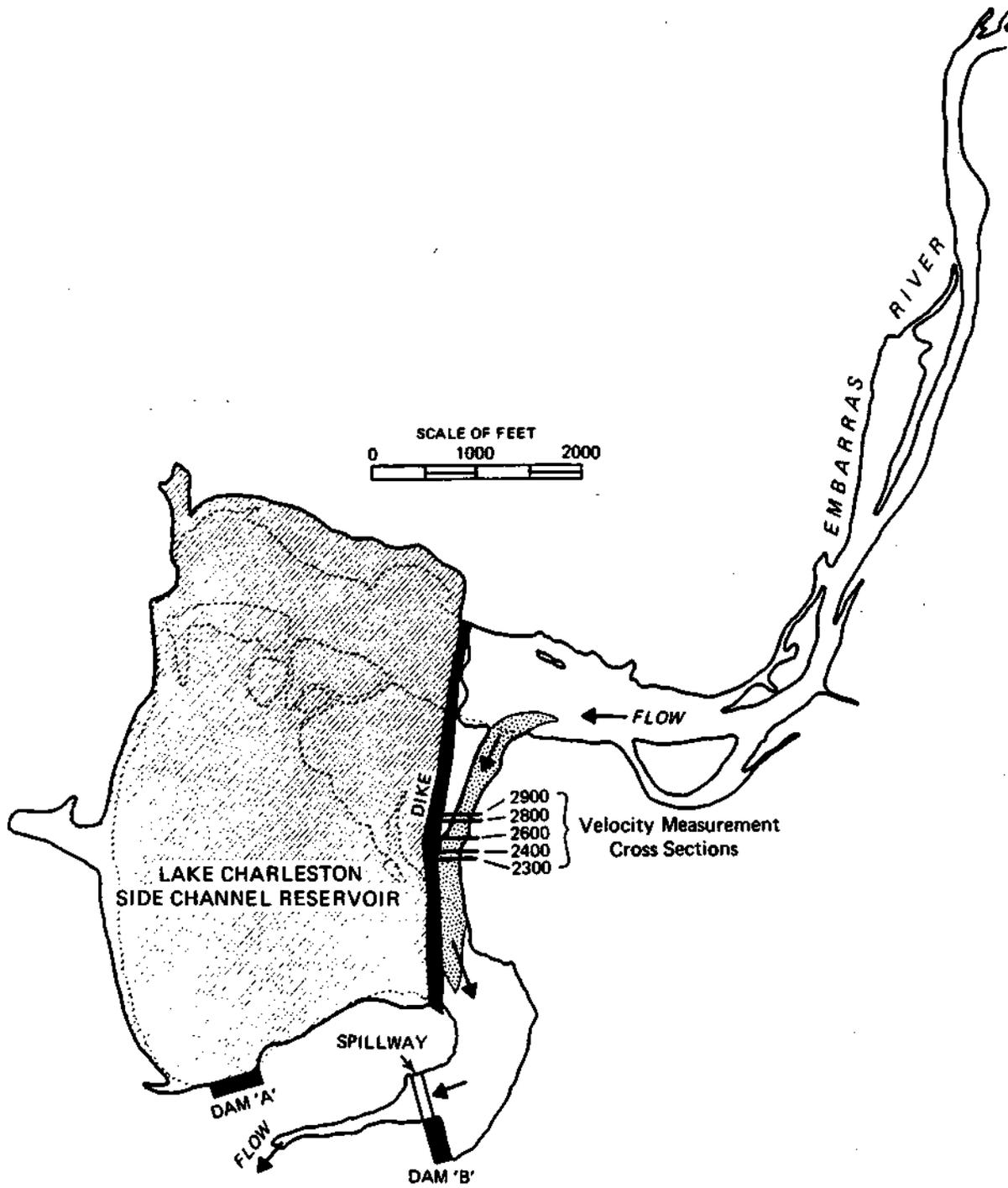
Figure 16. Locations of cross sections where velocity measurements
were made in 1986
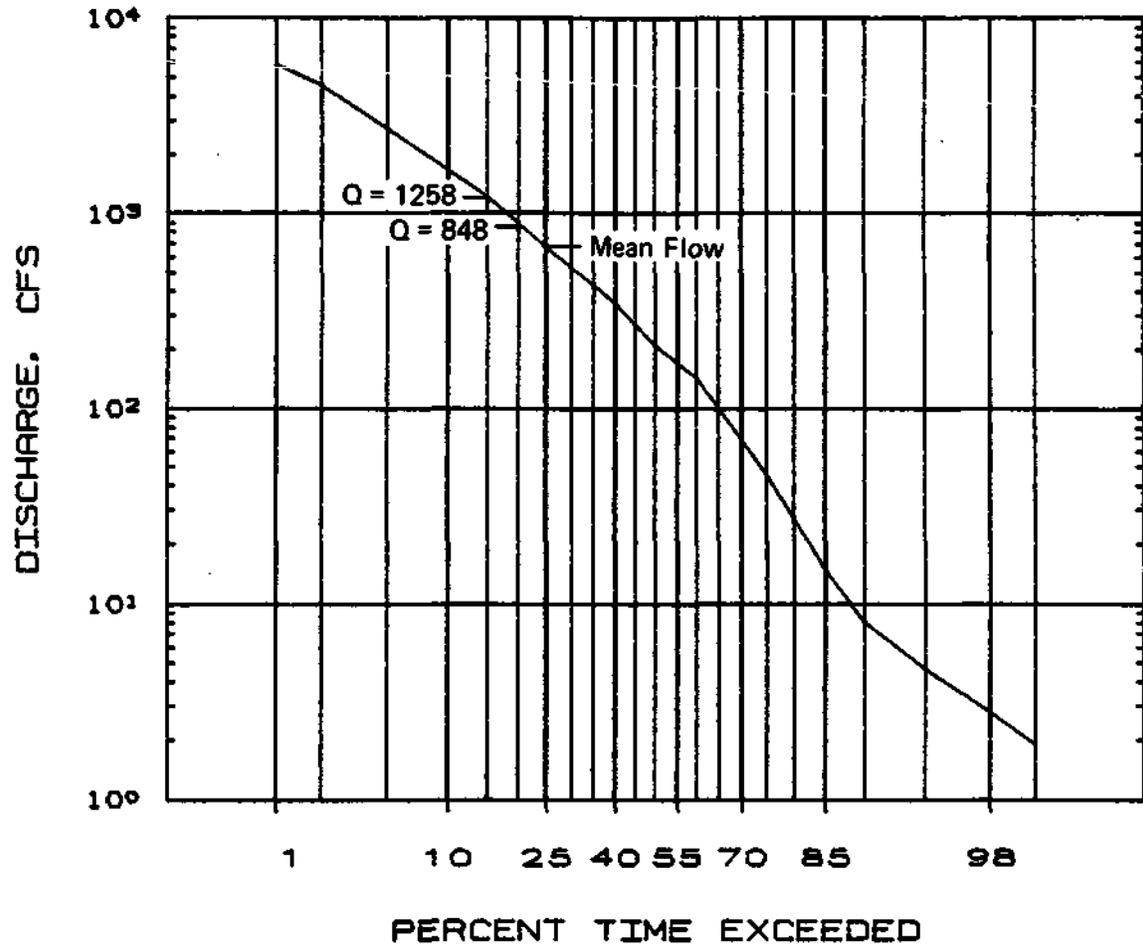
Figure 17. Flow duration curve for Embarras River at Lake Charleston
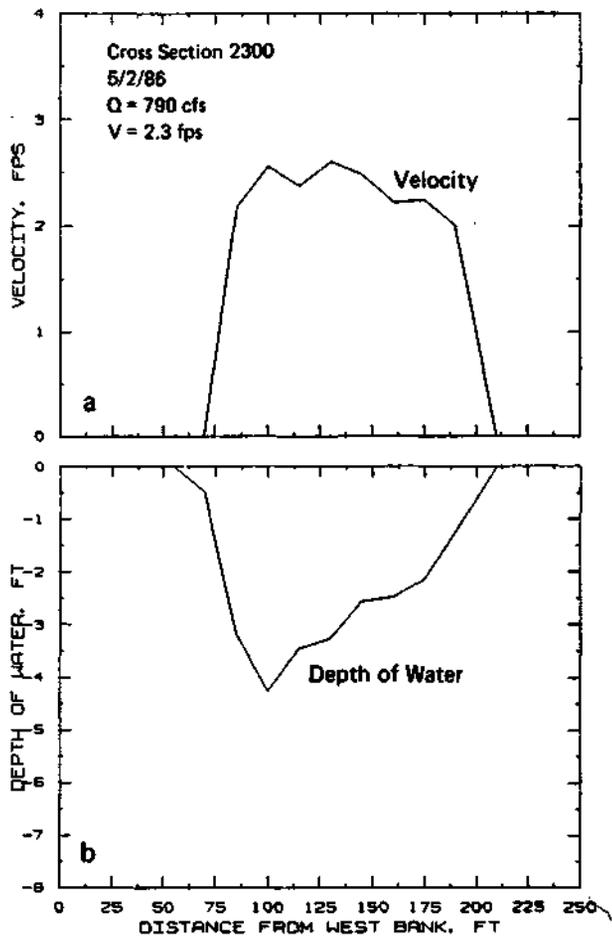
Figure 18. Velocity and water depth
at cross section 2300, May 2, 1986
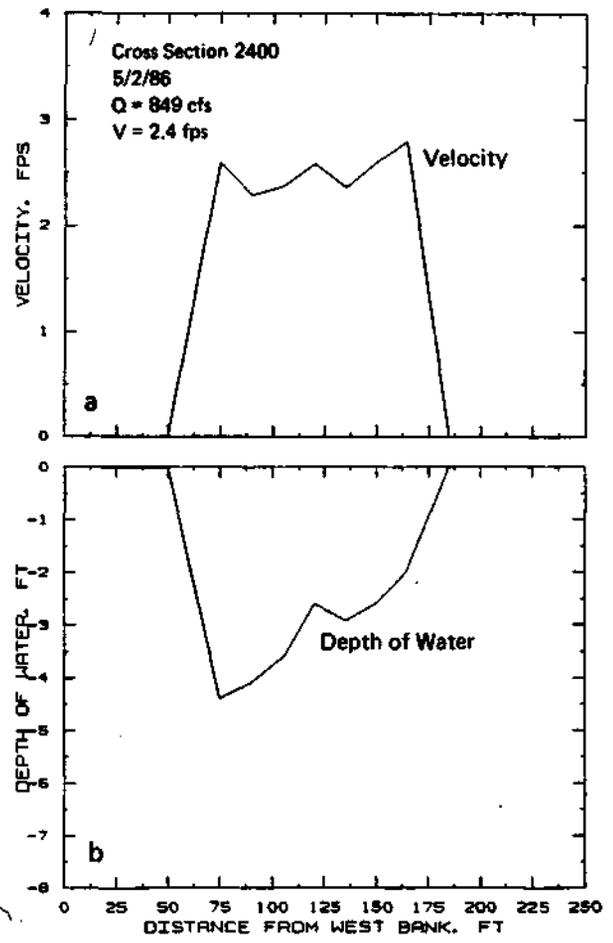
Figure 19. Velocity and water depth
at cross section 2400, May 2, 1986
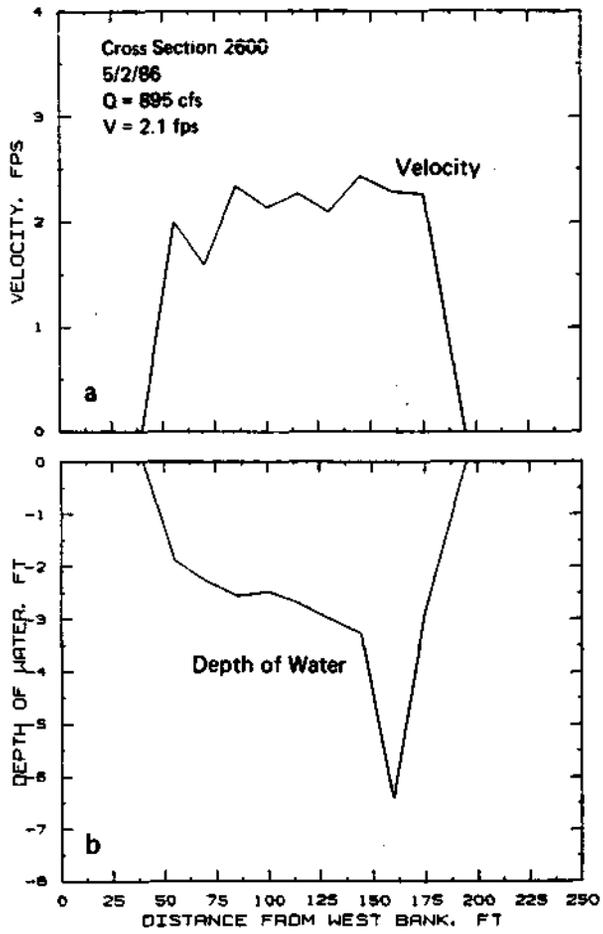
Figure 20. Velocity and water depth at cross section 2600, May 2, 1986
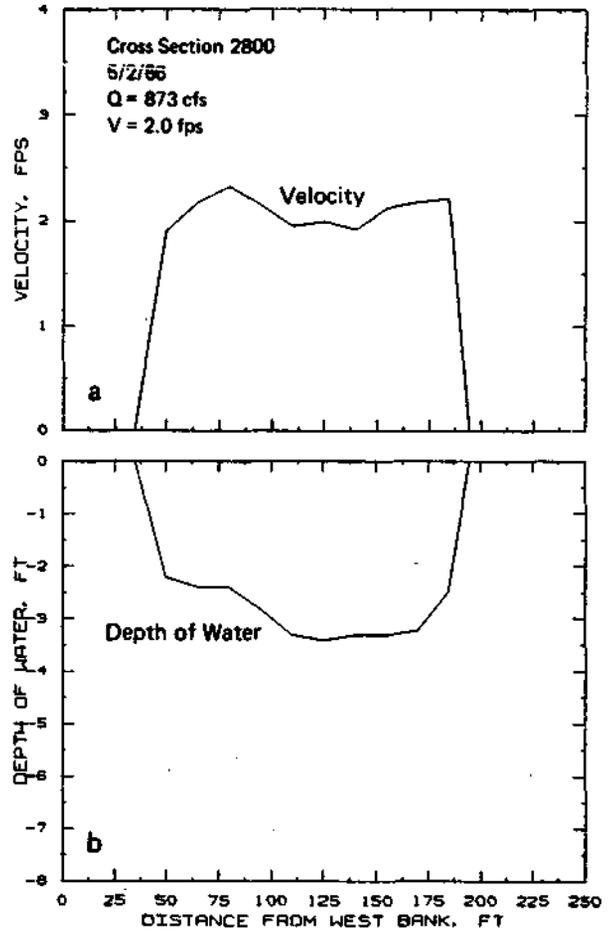
Figure 21. Velocity and water depth at cross section 2800, May 2, 1986
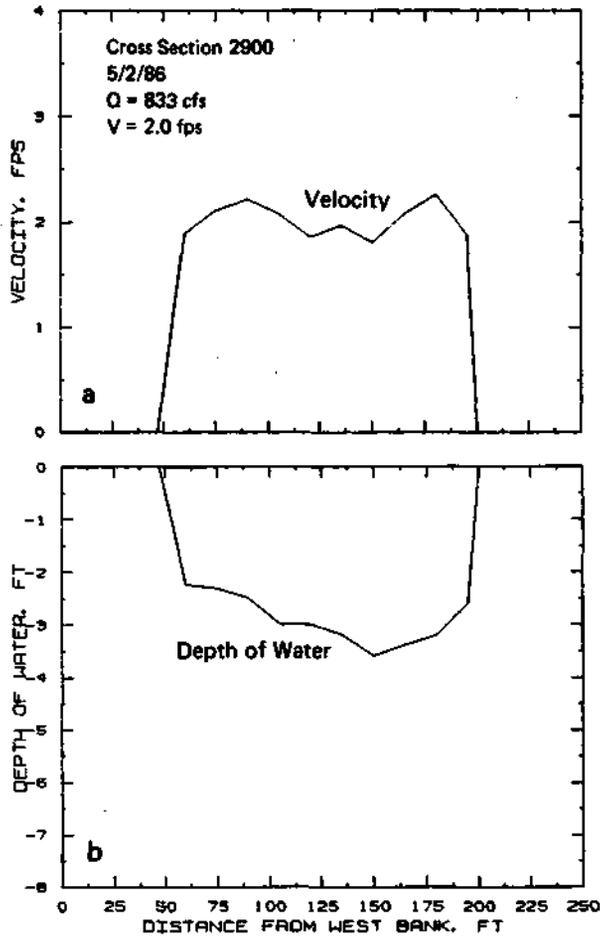
Figure 22. Velocity and water depth at cross section 2900, May 2, 1986
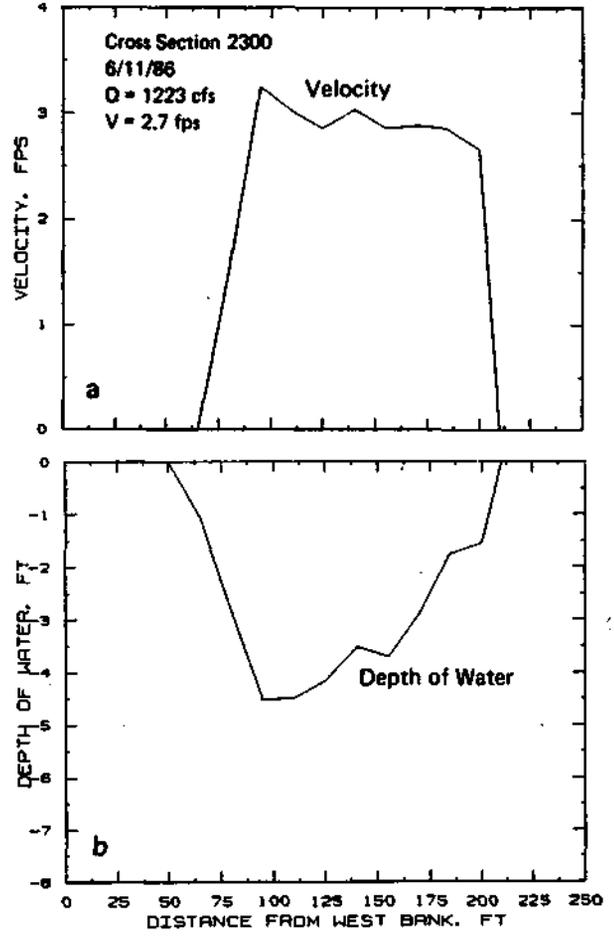
Figure 23. Velocity and water depth at cross section 2300, June 11, 1986
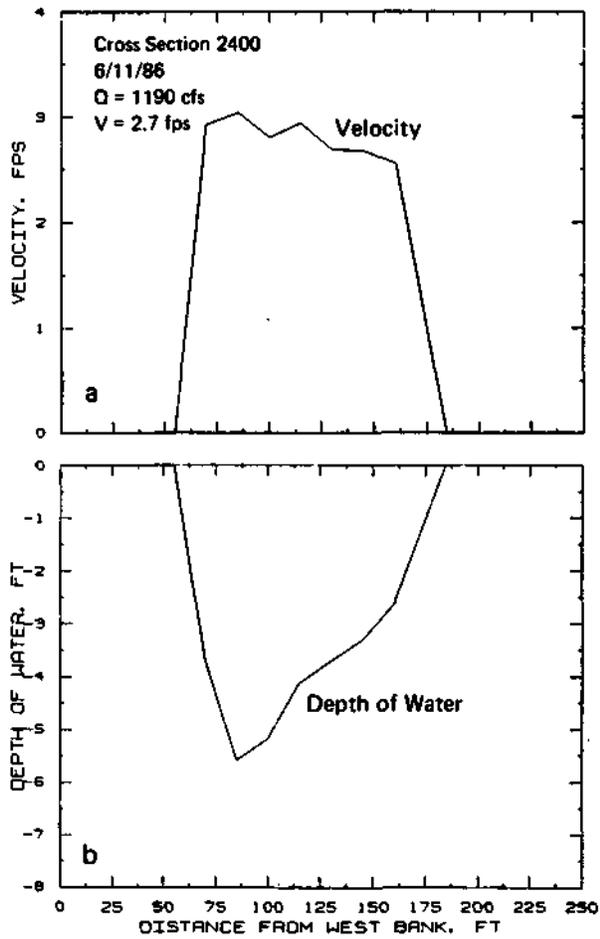
Figure 24.   Velocity and water depth at cross section 2400, June 11, 1986
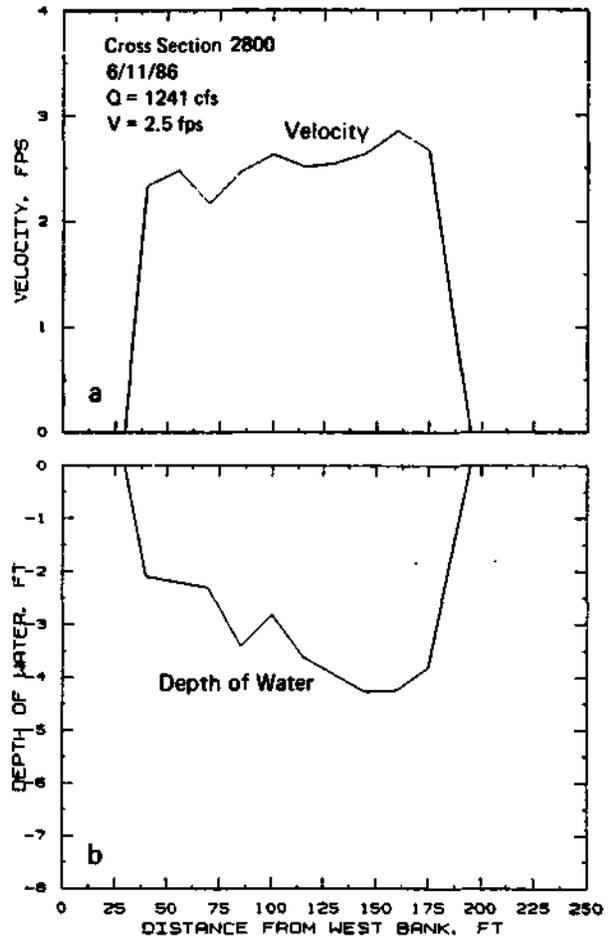
Figure 25.   Velocity and water depth at cross section 2800, June 11, 1986
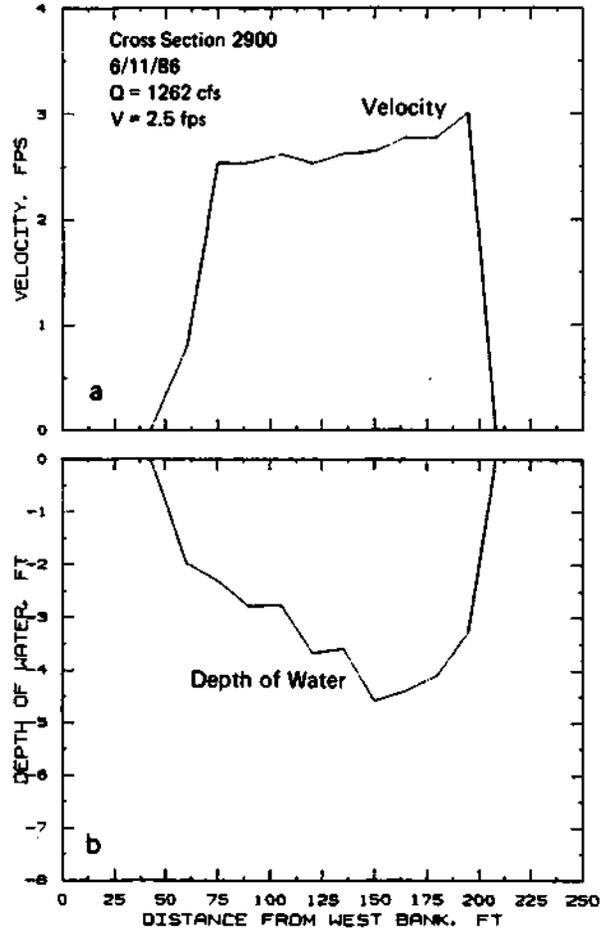
Figure 26.   Velocity and water depth
at cross section 2900, June 11, 1986

migration of the maximum velocity. During the measurements on June 11, however, the location of the maximum velocity is shown to migrate from shore to shore. In the upper reaches at cross sections 2800 and 2900 the maximum velocity is closer to the eastern bank of the channel, while at cross sections 2300 and 2400 in the downstream reaches the maximum velocity is located closer to the western bank, along the side of the dike. During higher flows a similar pattern of velocity distribution and migration of maximum velocity from side to side as indicated by the June measurements will be expected. At the same time the maximum velocities will be higher as the discharge gets higher than those measured for this project.

Particle Size and Unit Weight Analyses

A total of 14 samples were collected to determine physical character-istics of the native soil and accumulated lake sediments. Three samples were analyzed to determine unit weight and 11 samples were analyzed to determine particle size distribution. Of these samples, the 3 unit weight samples and 4 of the particle size samples were native materials. Of the other 7 particle size samples, 2 were from exposed lake sediments, 2 were from near-channel sand deposits, and 3 were samples of unexposed sediments from the side channel reservoir. Unit weight samples were collected with a 2-inch-diameter coring sampler, and particle size samples were scraped from the surface with a small scoop.

Analyses of the 3 unit weight samples give an average unit weight of 100 pounds per cubic foot for the native soils.

Due to drying and consolidation of the lake sediments following exposure, it was decided that the in situ sediment condition was not representative of conditions immediately following spillway failure. Instead, samples from the 1974 sedimentation survey which indicated unit weights of 43 pounds per cubic foot were used.

The results for the 11 particle size samples collected during the study were compared with the results for 9 samples collected during the 1974 sedimentation survey. The results of 14 of these analyses (9 from the present study and 5 from the 1974 survey) were combined into three general material types: lake sediments, native soil, and stream bed.

The lake sediment particle size comparison for the lower lake basin is shown in figure 27. Figure 27 shows the comparison between two sample

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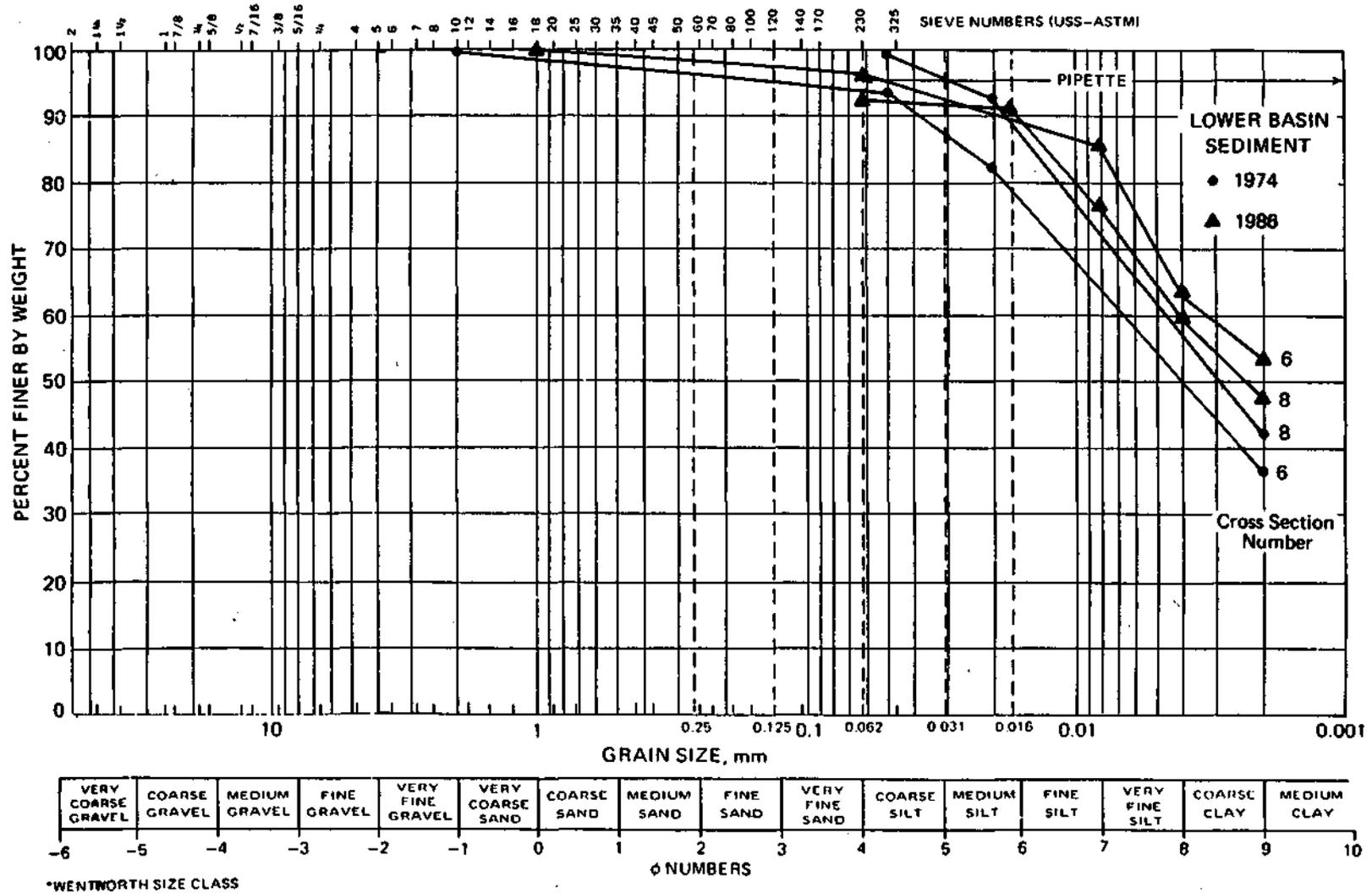
Figure 27. Particle size distribution of 1974 and 1986 samples
in lower reaches of Lake Charleston

sets, one from cross section 6 and the other from cross section 8, from the lower basin. These analyses indicate a fairly good correlation between the 1974 and 1986 samples, with the 1986 samples representing generally finer particle sizes. These results support the hypothesis that significant erosion has occurred beyond the limits of the scour channel. Sediments in the lower end of a reservoir generally accumulate sediments with coarser materials on top of finer materials due to loss of trap efficiency in the upper end of the reservoir with time. Thus the finer sediments collected during the 1986 sampling were most likely deposited prior to the 1974 survey, with later sediment deposits having been scoured after the spillway failure. Because of the change in reservoir hydraulics in 1982, it cannot be determined from this analysis whether some or all of this sediment was reworked prior to the spillway failure.

The native soil samples (figure 28) indicate generally loamy soils with high silt and sand content. In the area of the scour channel, samples indicate loam and silty clay overlying sandy loam.

The stream bed samples from both the 1974 and 1986 sampling (figure 29) indicate a predominance of medium to coarse sand. These materials are readily transported by high flows in the Embarras but will redeposit in lower flows or slack water. The 1986 samples were both collected from channel edge deposits. The 1974 sampling would represent center channel deposits. The influence of the lake can be seen on the 1974 sample from cross section 14, which shows much higher silt and clay fractions than the other samples. Since all of these samples exhibit some lake influence, they should not be considered representative of general Embarras River bed materials.

MATHEMATICAL MODELING

Mathematical models are useful tools for investigating future conditions under various assumed scenarios. In the present case, it is important to simulate the development of the scour channel and its migration upstream under different flow and physical assumptions. Before using a mathematical model to predict future conditions, it is important to evaluate the performance of the model for observed situations. For this project a sediment transport model was selected and used to simulate what
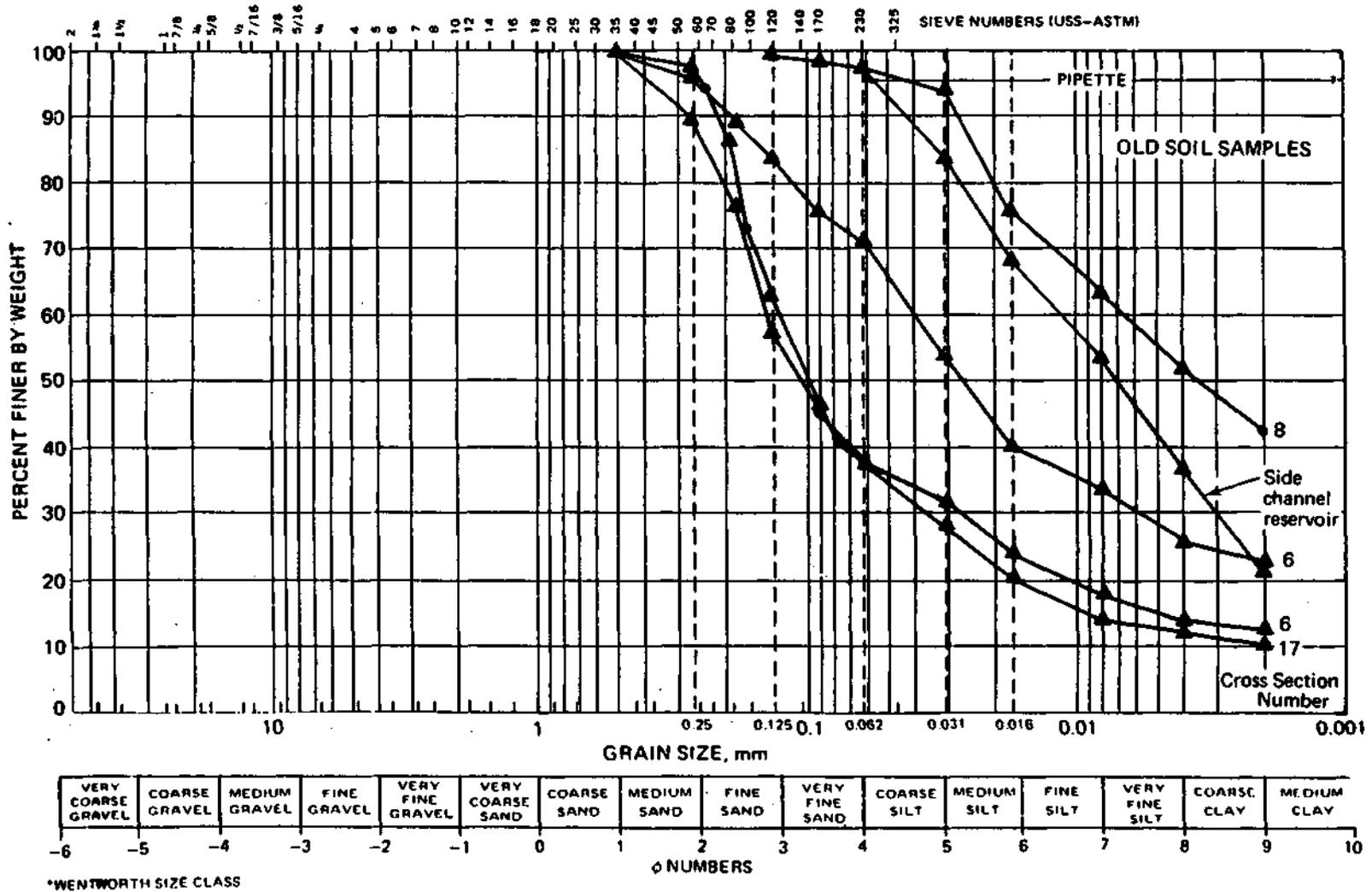
Figure 28.   Particle size distribution of native soil
scoured by the formation of the new channel
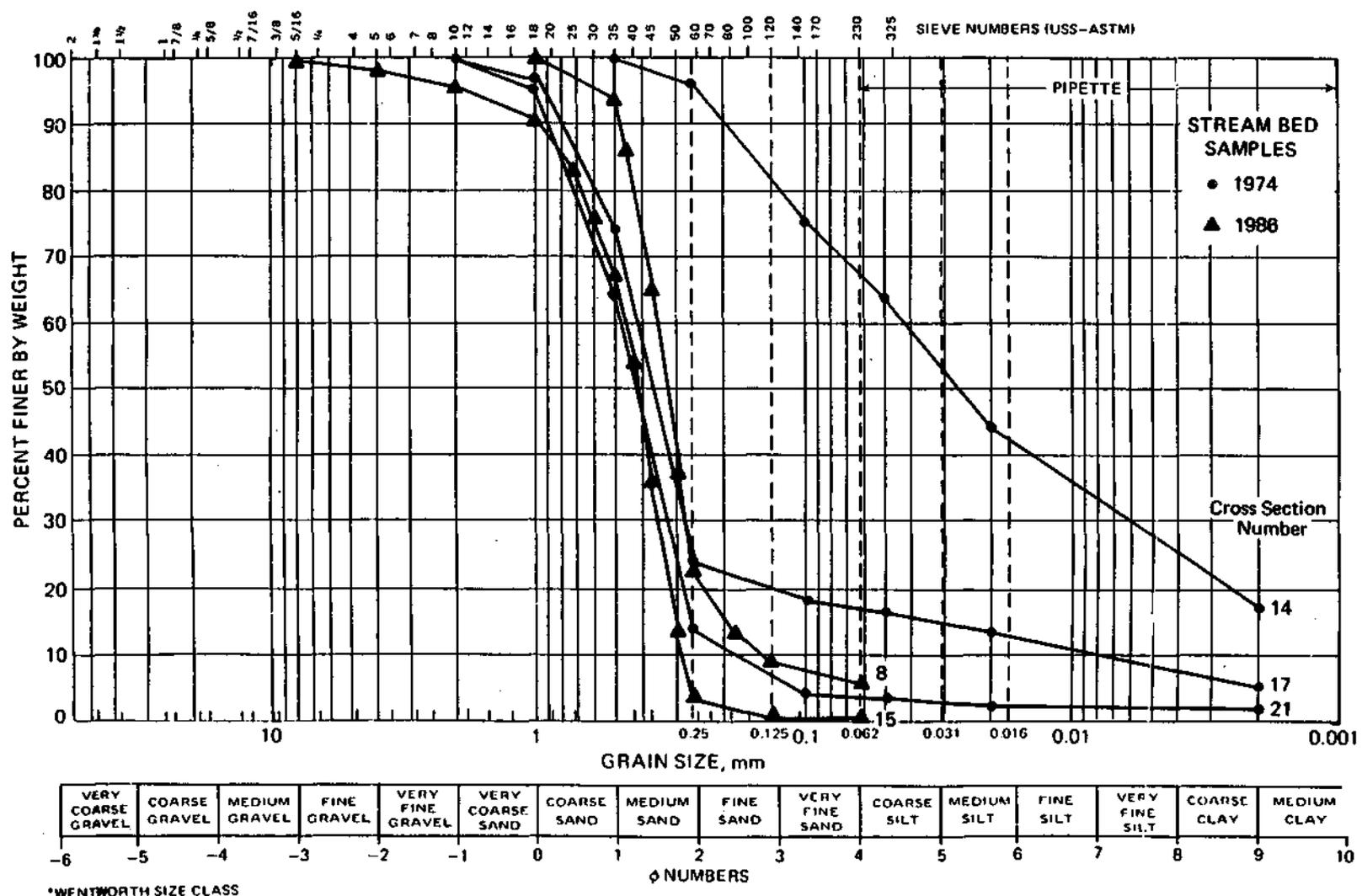
Figure 29. Particle size distribution of stream bed material samples
in Embarras River at Lake Charleston

happened in Lake Charleston after the spillway failure and then was used to investigate what might happen under different assumptions.

Sediment transport models can be one-, two-, or three-dimensional. At the present time, application of three-dimensional models, which simulate sediment transport in the longitudinal, lateral, and vertical directions, is limited to very special cases because such models are complex and expensive to run and require extensive data which are difficult and expensive to obtain. Two-dimensional models are generally used in situations where lateral velocities and sediment transport are important, such as estuaries and very wide rivers. One-dimensional models are generally adequate to simulate flow and sediment transport in rivers where the transport in the transverse and vertical directions is small compared to the transport in the longitudinal direction.

In the present study, the HEC-6, a one-dimensional sediment routing model, was selected to be used in modeling sediment transport in the Embarras River and the development of the scour channel after the spillway failure at Lake Charleston.

HEC-6 Model

The HEC-6 model is a one-dimensional simulation program designed to analyze scour and deposition of sediment in rivers and reservoirs. It simulates the ability of the stream to transport sediment and encompasses the full range of conditions embodied in Einstein's Bed Load Function, plus the transport and deposition of silt and clay, armoring, and the destruc- ' tion of the armor layer in a stream channel.

The basis for the water surface profile calculations is essentially Method II given by USCOE (1959). The transport capacity is calculated by using the method given by Laursen (1958) or the method of Toffaleti (1966).

The basic equations are the energy equation:

$$\frac{\partial H}{\partial x} + \frac{\partial}{\partial x}\left(\frac{\alpha V^2}{2g}\right) = S_e \tag{1}$$

and the continuity-of-sediment-material equation:

$$\frac{\partial G}{\partial x} + B \frac{\partial y_s}{\partial t} = q_s \qquad\qquad (2)$$

where:

     H = water surface elevation

     V — average flow velocity

    S_e = friction and other losses

     x = distance in the flow direction

     g = gravitational acceleration

      = energy correction coefficient, accounting for the shape of
        the velocity profile

     B = width of deposit or scour area (movable bed)

     G = sediment load

    q_s = lateral inflow of sediment

     y = depth of sediment deposit or scour above a stable layer

     t = time

The above equations incorporate a number of assumptions and approximations about water flow and sediment transport, such as hydrostatic pressure distribution in the water column and uniformity of velocity within a section. Sediment transport is also assumed to be a one-dimensional phenomenon despite the fact that it is actually a three-dimensional process.

Equations 1 and 2 contain three unknown parameters -- $H(x,t)$, $V(x,t)$, and $y_s(x,t)$ -- and two independent variables, x and t. In order to solve the equations for the unknowns, there is need of a third closure equation. Such an equation should relate the rate of sediment movement to the hydraulic parameters. It is usually empirical and has the form

      G = fnt(V,D,W,S,T,df/dt,d)   (3)

where:

    D = water depth in a cross section

    W = water width in a cross section

    S = water surface slope

    f = fraction of bed material in a certain size class, such as silt,
       clay, fine sand, etc.

    d = sediment particle size

    All other variables are as defined earlier.

Equation 3 is a symbolically stated sediment discharge formula. It may be a simple one that expresses the rate of sediment movement, G, explicitly as a function of the flow and material characteristics, or it may be a system of equations such as those developed by Einstein (1950).

Equations 1 to 3 form a non-linear system of partial differential - equations. The system is quite complex and there is no analytical solution available. However, it can be solved by using numerical methods on digital computers.

The HEC-6 model uses an implicit finite difference scheme to discretize the domain to solve equations 1 to 3 numerically. The computational proce-dure in the model is as follows:

Step 1: The energy and the continuity equations are solved to deter-mine the water surface profile and all the pertinent hydraulic parameters (elevation, slope, velocity, depth, and width) at each cross section along the study reach. The water surface profile is calculated by using the backward step method to solve the energy equation in the same way as in the HEC-2 water surface profile computations (USCOE, 1979).

Step 2: Using the hydraulic data obtained during the calculations of water surface profiles, the program calculates the inflowing sediment load, armoring, equilibrium depth, gradation of material in active layer, trans-port capacity, etc., for each cross section. The transport capacity is determined analytically from empirical relations incorporated into the model. The available options of such relations are: 1) Toffaleti's application of Einstein's Bed Load Function (Toffaleti, 1966); 2) Laursen's relationship as modified by Madden for small rivers (Laursen, 1958; USCOE, 1977); 3) the DuBois relationship (Vanoni, 1977); 4) Yang's streampower equation (Garde and Raju, 1985); or 5) any relationship developed by the user for a particular study. The relationship has to be specified in a form where the transport capacity per unit width is a function of the product of the water depth and the energy slope. For the form of the different sediment transport equations and detailed discussions of them, the reader is referred to Vanoni (1977), Graf (1971), Garde and Raju (1985), and Simons and Senturk (1977).

Step 3: The program calculates the sediment load leaving the study reach, and then changes the volume of bed material to reflect scour or deposition. The depth of deposit or scour is adjusted to reflect the new

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volume. The above procedure is repeated for a sequence of water discharges (derived from the discretized hydrograph) and the corresponding sediment loads. The changes are calculated with respect to time for each reach and with respect to distance along the stream for the different reaches within the study area.

Input Data Requirements of HEC-6

    The input data needed to run HEC-6 can be grouped into four categories, as described in USCOE (1977):

    1. Geometric data. Cross section coordinates, reach lengths, and Manning's n-values are required for water surface calculations. In addition, the movable bed portion of each cross section and the depth of sediment material have to be inputted.

    2. Sediment data. Inflow sediment load data, gradation of bed material in the stream bed, and fluid and sediment properties are needed.

    3. Hydrologic data. Water discharges, temperatures, and durations have to be inputted.

    4. Operating rule. A relationship between discharge and water surface elevation at the downstream boundary of the study reach (where calculations start) has to be supplied. This relationship can be a rating curve or else can be derived from Manning's equation or from a critical depth assumption.

    The procedures to prepare all the input data are described in USCOE (1977) in detail. For the geometric data input, two formats are available. One is the standard format which is also used in the HEC-2 "Water Surface Profiles" program (USCOE, 1979). The other is an optional format, called the alternate format, used only in the HEC-6 program. The alternate format is designed so that a pseudo two-dimensional approach can be implemented for solving sediment transport problems. The alternate format uses hydraulically similar strips in the direction of flow to compute hydraulic variables in the lateral direction. Up to seven strips can be used for the same problem. The final step in implementing a pseudo two-dimensional sediment transport technique would require transferring water and sediment from one strip to another in such a manner that continuity and momentum are preserved. However, this step has not yet been incorporated in the HEC-6 program.

Limitations and Potential Uses of HEC-6

HEC-6 is a one-dimensional sediment transport model. Thus it does not simulate a lateral distribution of sediment load across a cross section. The cross section is divided into two parts: the movable bed part and the stable bed. For each reach the entire movable bed part is moved vertically up or down depending on whether deposition or scour occurs in this reach. The stable bed is not allowed to change. The program also cannot account for density or secondary currents since it is one-dimensional. Bed forms are not simulated except that n-values can be introduced as functions of the discharge. This indirectly introduces an approximate consideration of bed forms.

The HEC-6 model has many potential applications, however, and has been successfully applied to the following or related problems:

-Reservoir sediment deposition to determine volume and location of sediment

-Degradation of stream bed downstream of a dam

-Long-term trends of scour or deposition in channels

-Influence of dredging on the rate of deposition

-Scour during floods

-Impact of changing water-sediment mixture in natural streams, or changing the stream's boundary and hydraulics of flow

-Impact of dams on a stream

-Impact of channel contraction required to maintain navigation depths

Application of HEC-6 for Embarras River at Lake Charleston

Geometric Data

The initial geometric data for the application of HEC-6 in the Embarras River at Lake Charleston consisted of 21 cross sections taken from old and new surveys. The cross sections used for the model are identified by the letter M followed by a cross section number from 0 to 20. The cross sections and the thalweg are shown in Appendix C. Cross section MO is the cross section at the Route 130 bridge downstream of the spillway. It was taken from the 1986 survey. Cross sections Ml to M4 and M18 to M20 are taken from the 1962 DOWR survey. 'Cross section M4 is the cross section at the spillway. Cross sections M5 to M8 are taken from the 1974 lake sedi-mentation survey. Cross sections M9 to M17 are taken from the cross

sections of the diversion channel that was excavated in 1982. The loca-
tions of the cross sections used in the model are shown in figure 30.

The alternate format of HEC-6 was used in modeling the deposition and
scour in the Embarras River at Lake Charleston. The reaches were sub-
divided into five strips, four strips for the overbanks and one for the
main channel. Different Manning's n values were used for the overbanks and
main channel. Values of n - 0.060 for the overbanks and n - 0.040 for the
main channel were used in the model. At the spillway a value of n - 0.020
was used. Different longitudinal distances between adjacent cross sections
were specified for each strip when appropriate in order to take the sinuos-
ity of the river into account.

Initially, the old cross-sectional data and the spillway before
failure were used to simulate the pre-failure conditions and to generate
cross-sectional data that were assumed to have existed just before the
failure of the spillway. The cross sections resulting from the pre-failure
runs were then used as input data to the second stage of simulations, where
the failure of the spillway is incorporated. The spillway cross section
(cross section M4) was modified by a triangular opening 16 feet deep and 76
feet wide which represents the breach in the spillway as shown in Appendix
C.

Sediment Data

The sediment load data used in the model were taken from Bonini et
al. (1983). The suspended sediment rating curve is shown in figure 31.
The regression equation between the suspended sediment load and water
discharge is given as:

$$Q_s = 0.019 \ Q_W^{1.51} \tag{4}$$

where:

   Q_S = sediment load in tons per day

   Q_w = water discharge in cfs

The suspended sediment load was increased by 15 percent to account
for the bed load. The total sediment load equation is therefore given by
equation 5:

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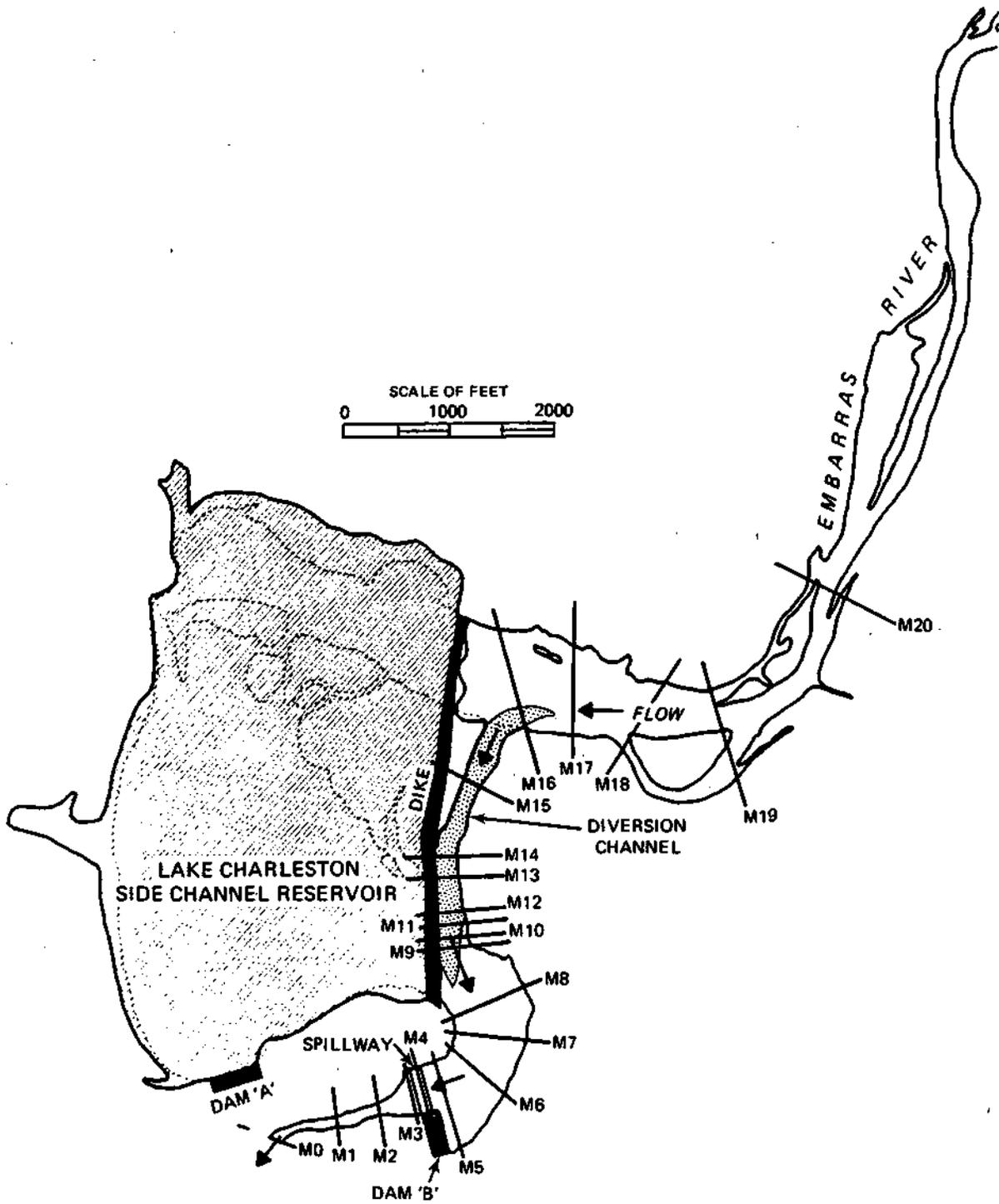
Figure 30. Locations of cross sections used in the HEC-6 model
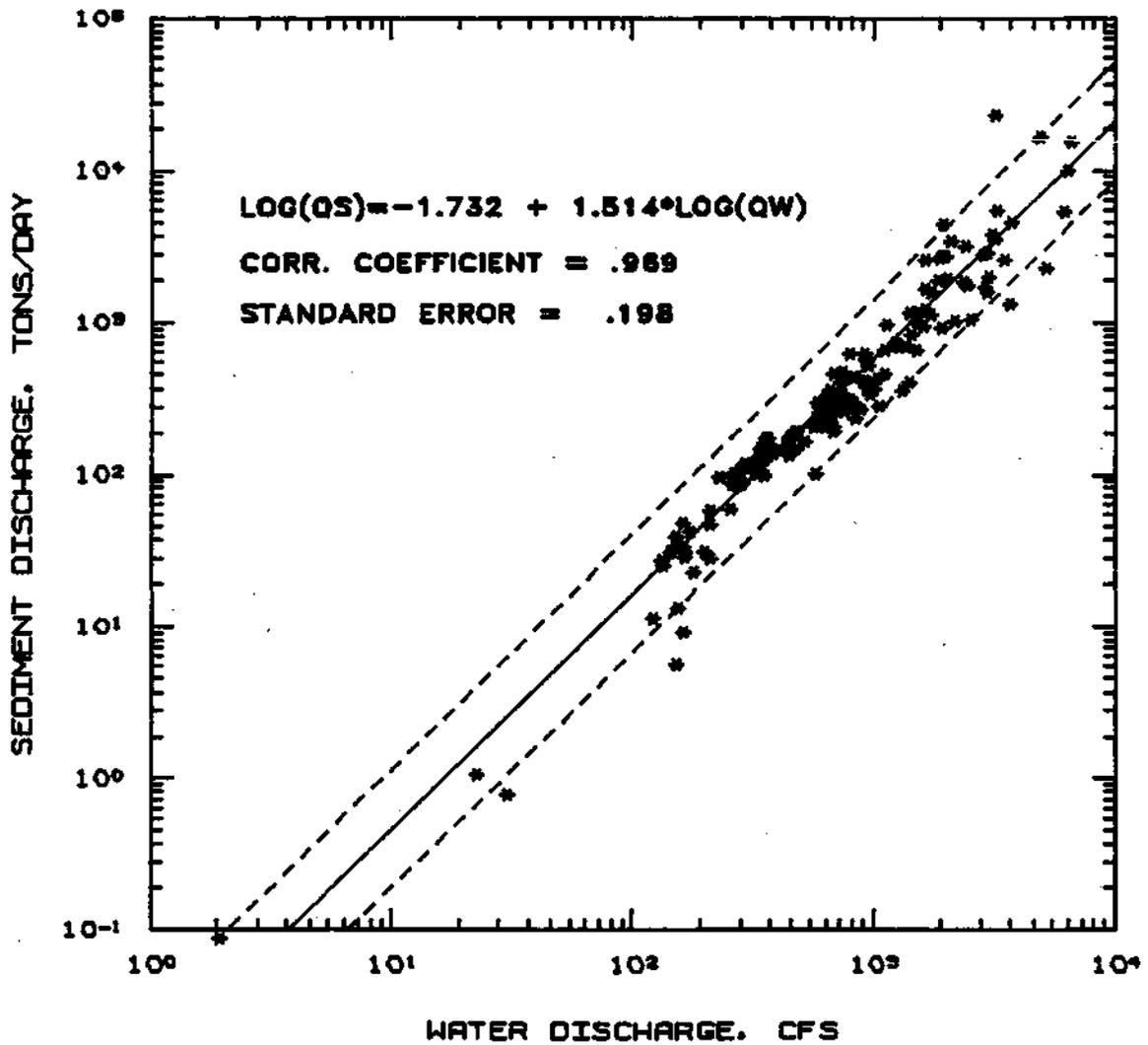
Figure 31. Suspended sediment rating curve for the Embarras River
near Diona, 1L

$$Q_s - 0.022\ Q_w^{1.51} \tag{5}$$

The percentages of clay, silt, and sand are estimated on the basis of data from Bonini et al. (1983) as 45.2 percent clay, 37.4 percent silt, and 17.4 percent sand, resulting in the following rating curves for the different sediment fractions:

Clay: $Q_{sc} = 0.010\ Q^{1.51}$

Silt: $Q_{ssi} = 0.008\ Q_w^{1.51}$ $\tag{6}$

Sand: $Q_{ssa} = 0.004\ Q_w^{1.51}$

where Q_{sc} = clay load in tons per day, Q_{ssi} = silt load in tons per day, and Q_{ssa} = sand load in tons per day.

The bed material gradation data were taken from <u>Water Resources Data</u> <u>for Illinois</u> for Water Year 1980 (USGS, 1980). Those data are given in table 3.

Hydrologic Data

The input hydrograph used was the Water Year 1982 flow hydrograph for the Embarras River near Diona, Illinois, located downstream of Lake Charleston. The data were obtained from the 1982 <u>Water Resources Data for</u> <u>Illinois</u> (USGS, 1982). The hydrograph, shown in figure 32, was adjusted for drainage area by multiplying all the discharges by a factor of 0.85, which is the ratio of the drainage area at Lake Charleston to that at

Table 3. Bed Material Gradation Curve Data

Diameter, mm	% finer
0.062	1
0.125	2
0.250	9
0.500	52
1.000	70
2.000	80
4.000	85
8.000	89
32.000	100
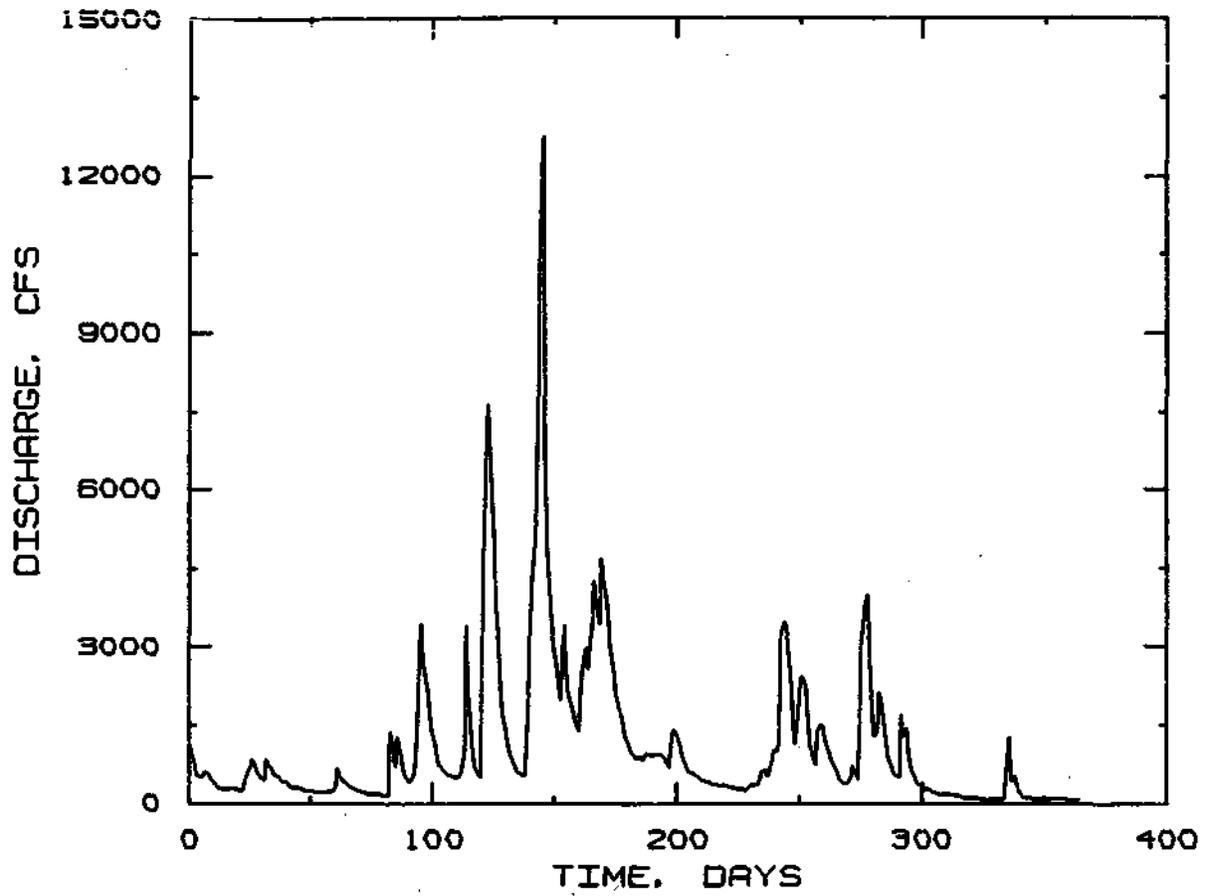
Figure 32.   Input hydrograph used in HEC-6 simulations

Diona. For simulation purposes and qualitative evaluation of the model,
the 1982 hydrograph at Diona, which is the latest hydrograph available at
that station, provides sufficient variation and similar flow conditions to
those experienced during the spillway failure. The flows represented by
this hydrograph were routed through the river twice to simulate the flow
and sediment transport before the spillway failure, and once to simulate
conditions after the spillway failure.

Operating Rule

The operating rule specified at the downstream end of the study reach
(Route 130 bridge downstream of the spillway) is a rating curve that
resulted from application of Manning's equation. Critical depth was
specified at the spillway for the runs before the spillway failure with a
head loss of 0.5 ft.

Simulation Results

As discussed earlier, the HEC-6 was run under the pre-spillway failure
conditions to estimate the lake geometry just prior to failure. Both
Toffaleti's and Laursen's sediment transport equations were used in the
simulation runs to check whether there were any significant differences in
the model outputs depending on the equation used. The bed profiles
resulting from such simulations are shown in figure 33. The two sets of
results are very similar, and both show sediment deposition for much of the
study reach except for the area just downstream of the rock ledge where
there is some scour. For the area immediately upstream of the spillway,
the results show some sediment deposition above the 1974 sedimentation
survey cross-sectional profiles. Most of the sediment deposition, however,
is in the upper reaches of the old lake.

The channel geometry outputs of the pre-failure runs were then used
as input data to the HEC-6 simulation of the scour after spillway failure.
Both Toffaleti's and Laursen's equations were used. The profiles of the
scour channel generated by the HEC-6 as compared to the pre-failure profile
are shown in figure 34 for both equations. There was no significant
difference in the profiles generated by the two sediment transport equa-
tions, even though Laursen's equation gives slightly more scour (figure
34b).
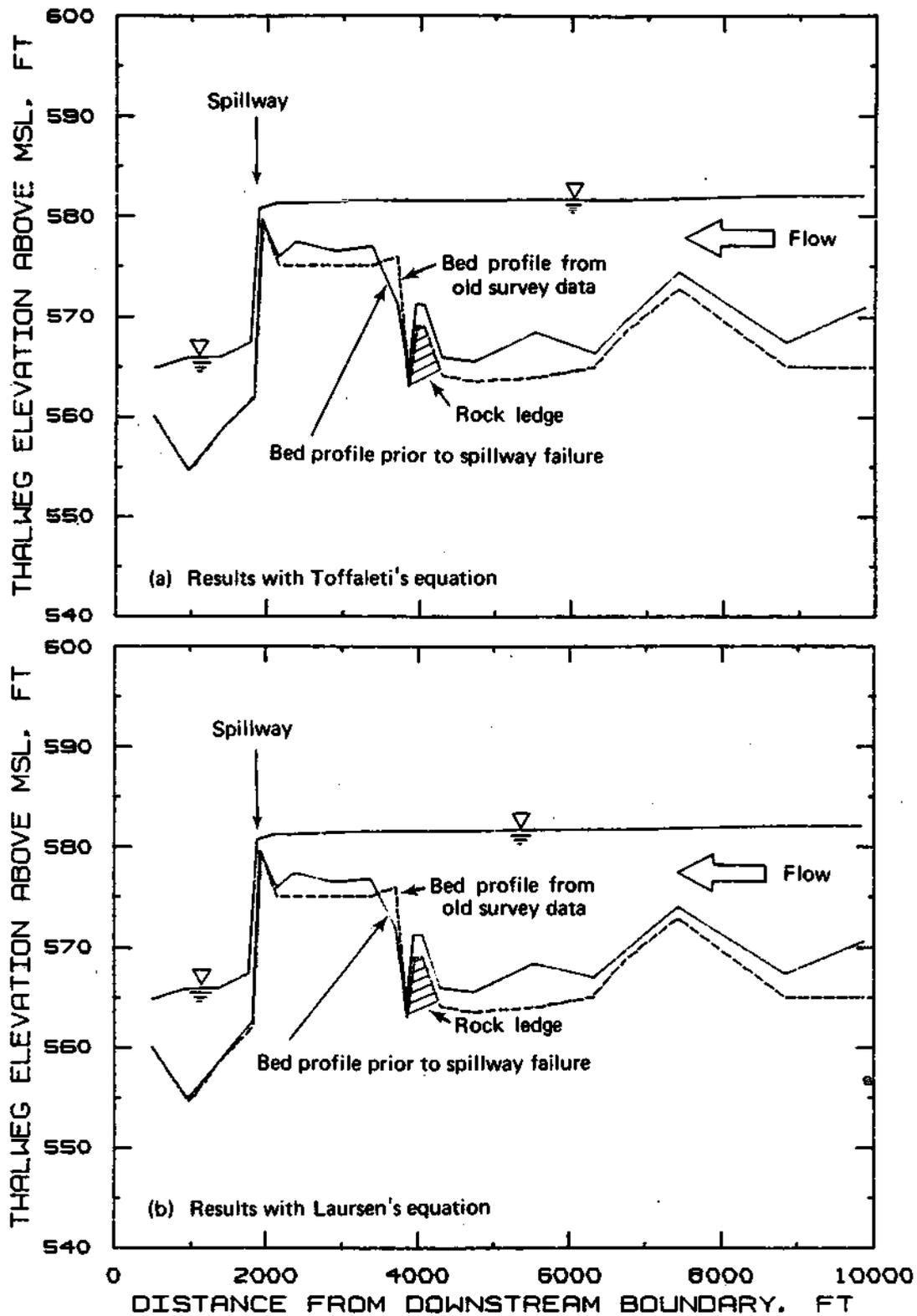
Figure 33. HEC-6 simulation results for conditions prior to spillway failure

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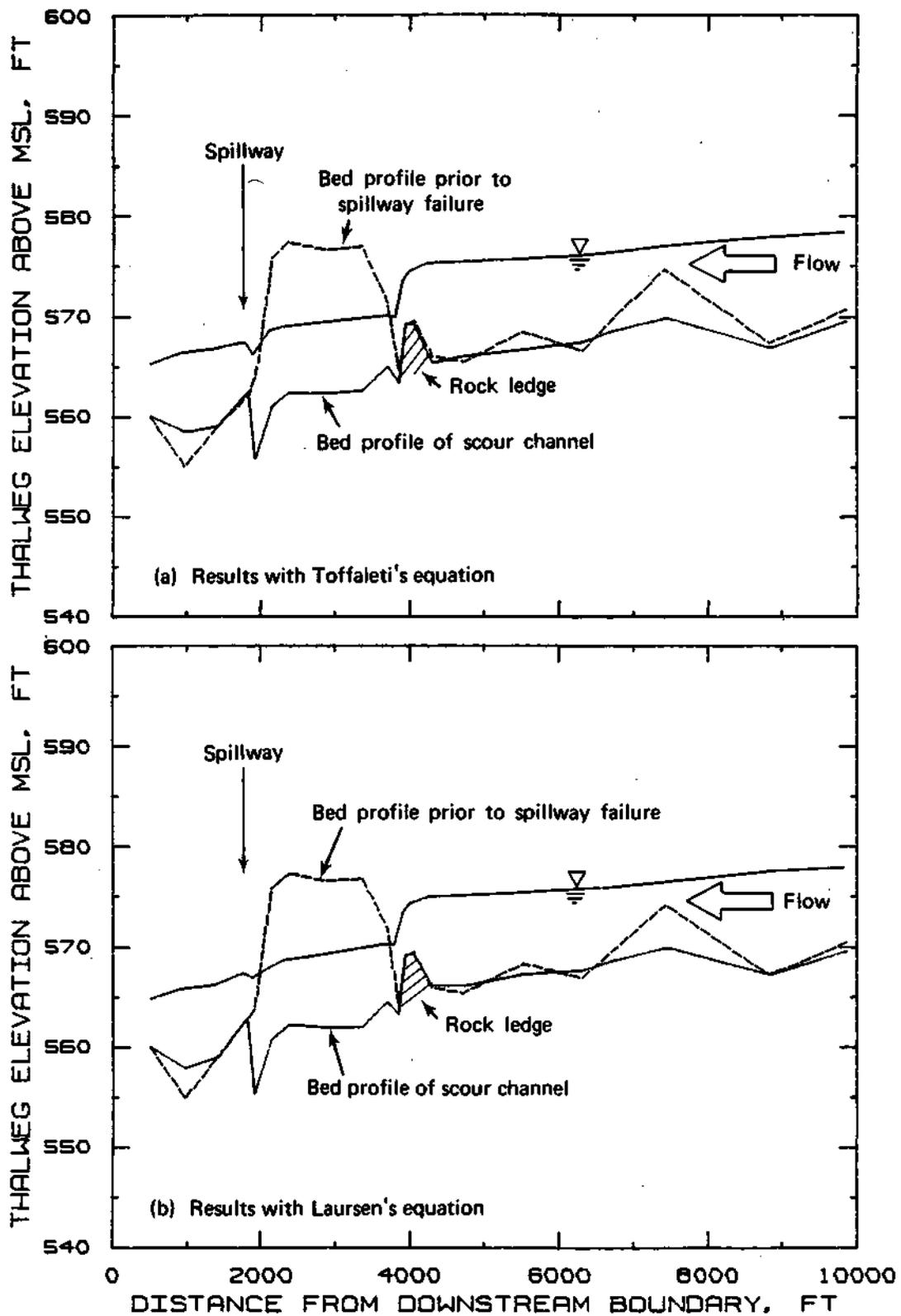
Figure 34. HEC-6 simulation results after spillway failure

As shown in figure 34, the HEC-6 simulation results show over 15 feet
of scour for the reach just upstream of the spillway. Most of the extreme
scour was confined between the spillway and the rock ledge, which is about
2,000 feet upstream. There is also an area of scour further upstream, just
beyond the diversion channel.

One of the significant observations in the field is the influence of
the rock ledge in stopping the upstream migration of the scour channel. It
was also observed that the rock ledge, which is primarily shale, is being
scoured slowly, especially during high flows. Thus it was judged important
to investigate how much more scour might have taken place if the rock ledge
had not been present, or how much scour might occur in the future if the
rock ledge is gradually eroded away. The result of such an analysis using
the HEC-6 is shown in figure 35, in which the bed elevations of the scour
channel with and without the rock ledge are compared. Both Toffaleti's and
Laursen's equations were used. As shown in the figures, the scouring would
have progressed much further upstream if the rock ledge had not been pres-
ent. The development of the channel downstream of the rock ledge would
have been similar either with or without the presence of the rock ledge.
The channel upstream of the rock ledge, however, would have been about 2
feet deeper without the rock ledge than it is with the rock ledge.

DISCUSSION

Comparison of Field Data with Model Results

In figure 36, the simulation results from the HEC-6 are compared with
the field survey data for the profile of the scour channel. In general,
the agreement between the simulated profile and that based on the field
data is very good except for the areas just upstream of the rock ledge and
just downstream of the spillway. In the area of major scour, upstream of
the spillway and downstream of the rock ledge, the agreement between the
two is very good.

In the areas where the simulation results do not match very well with
the survey data, it is possible that prior sedimentation and channel armor-
ing had taken place. Since there are only limited data on the bed material
characteristics from those areas, refinement of the bed material data to
improve model performance was not attempted.

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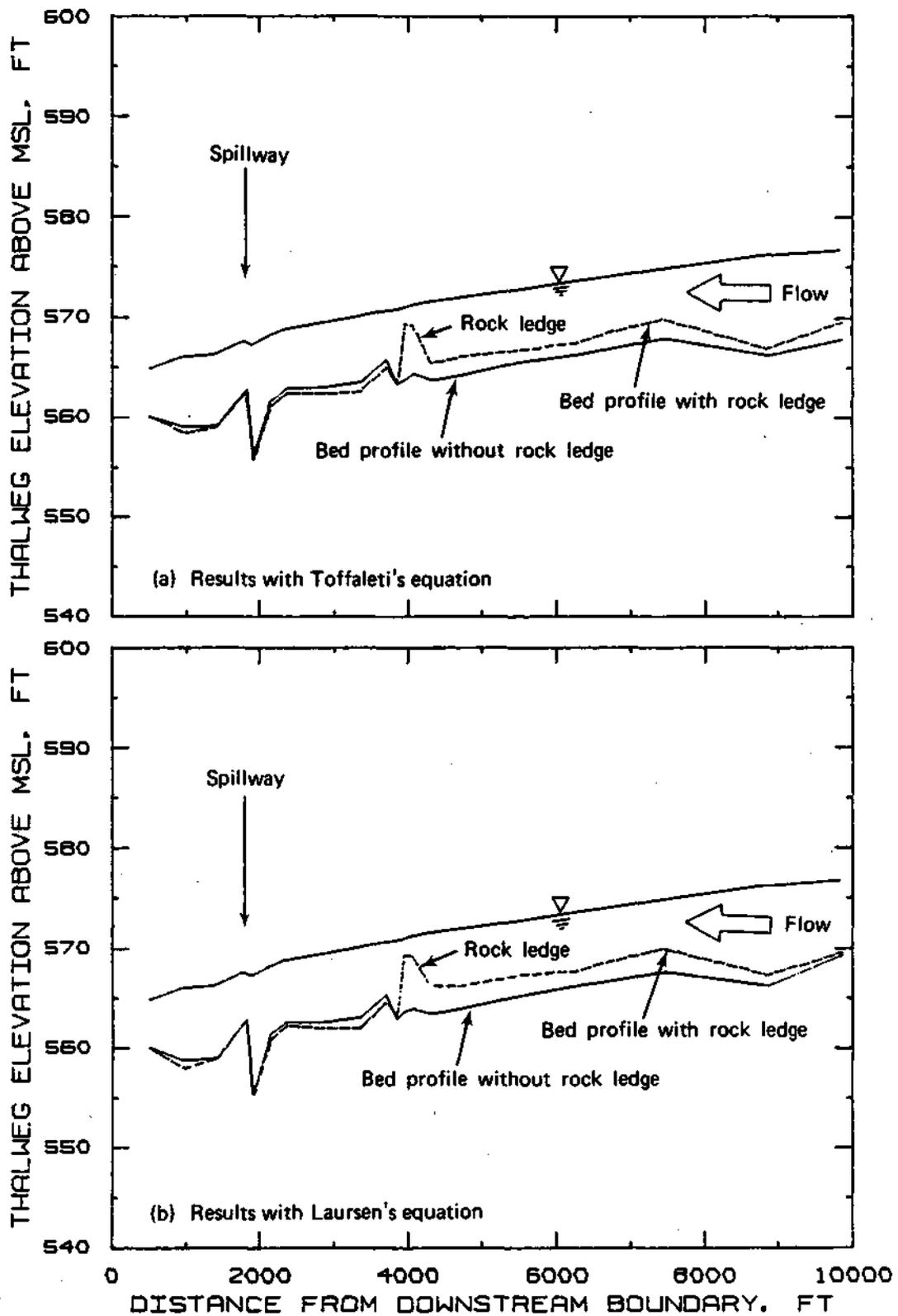
Figure 35. HEC-6 simulation results with and without rock ledge
in the diversion channel

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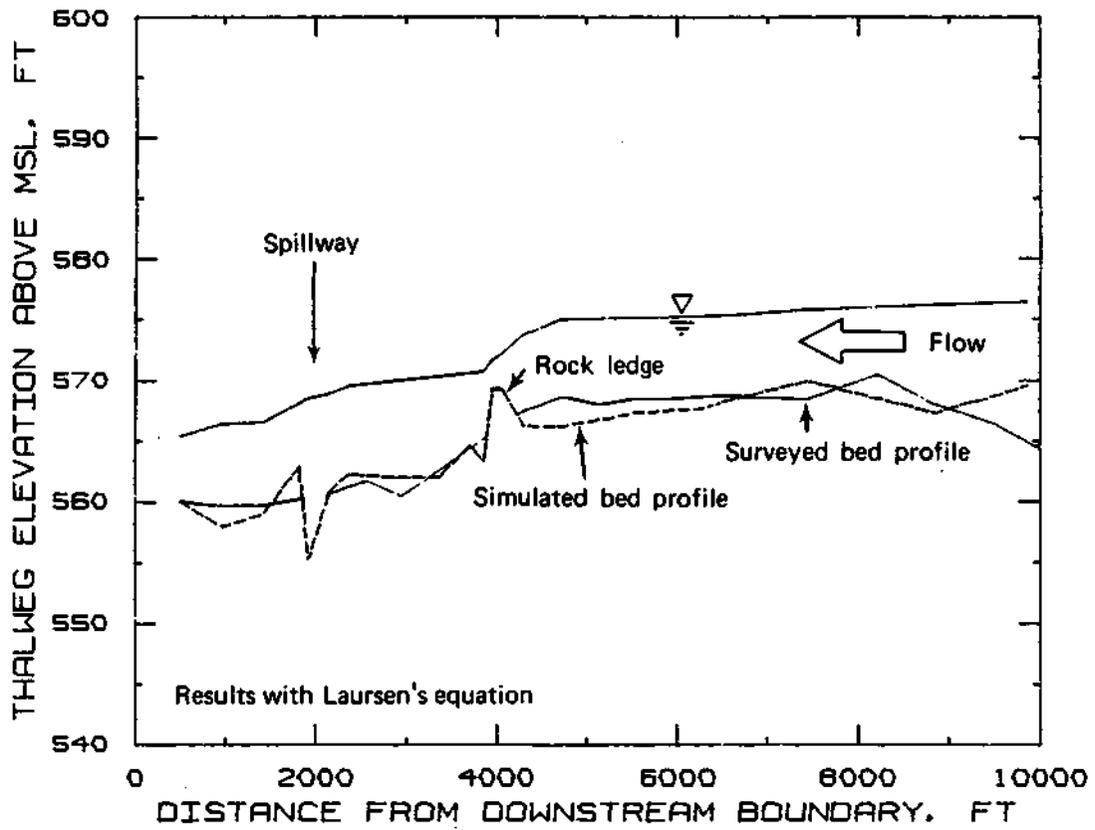
Figure 36. Comparison of field-measured and simulated channel profiles

Sediment and Soil Scoured after Spillway Failure

To determine the volume of sediment scoured out of the lake, the volumes of the segments between cross sections for the assumed initial conditions, the 1986 survey, and the cross sections of the HEC-6 simulation results were calculated. The results of these computations are presented in tables 4, 5 and 6. Volumes were computed below 580 ft msl for the three cases. Since the scouring patterns downstream and upstream of the rock ledge are different, two sets of comparisons are appropriate.

For the initial conditions, the lake and channel volume downstream of the rock ledge (cross section M13) and upstream of the spillway was calculated to be 91.1 ac-ft (total volume between cross sections M5 and M13 in table 4). The volume for the same segment based on the 1986 survey is 206.4 ac-ft (total volume between cross sections 6 and 9 in table 5). Similarly the volume for the same segment based on the HEC-6 simulation results is 178.1 ac-ft (total volume between cross sections M5 and M13 in table 6). The volume of sediment and soil scoured is determined by subtracting the initial volume of 91.1 ac-ft from the 1986 survey volume of 206.4 ac-ft and the HEC-6 volume of 178.1 ac-ft. Thus the volumes of sediment and soil scoured are 115.3 ac-ft (206.4 minus 91.1) and 87 ac-ft (178.1 minus 91.1) based on the 1986 survey and the HEC-6 results, respectively. The scouring simulated by the HEC-6 is less than that obtained from the 1986 survey.

For the channel upstream of the rock ledge, the 1986 survey shows a net scour of 25.6 ac-ft of sediment, while the HEC-6 results show a scour of 25.7 ac-ft of sediment. The volumes were calculated from tables 4, 5, and 6 in a similar fashion as for the segments below the rock ledge. The HEC-6 reproduced the net scour upstream of the rock ledge very well, although the profiles shown in figure 36 did not match exactly.

SUMMARY AND CONCLUSIONS

Summary

The development of a scour channel after the failure of the spillway at Lake Charleston was investigated by collecting field data and by using a mathematical model. The field data collection program included surveying

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Table 4. Segment Volumes and Total Volumes below 580' msl Based on
Initial Cross Sections Used in HEC-6

Cross section no.	Cross-sectional area (ft^2)	Volume (ac-ft) Segment	Total	Surface width (ft)
M5	2537.2	0.0	0.0	735.0
M5	2537.2	12.8	12.8	735.0
M6	1476.0	10.1	22.9	725.6
M7	1421.5	16.6	39.6	582.5
M8	1190.2	15.0	54.6	571.1
M9	1458.0	10.3	64.9	210.0
M10	2085.1	6.1	71.0	225.3
M11	1986.6	4.7	75.7	277.2
M12	1657.0	4.2	79.9	265.0
M13	2248.4	11.2	91.1	249.0
M14	2155.2	20.2	111.3	213.2
M15	1604.2	34.5	145.8	213.8
M16	2445.7	37.2	183.0	314.6
M17	1807.5	19.5	202.5	1306.0
M18	899.0	22.4	224.9	733.8

Table 5. Segment Volumes and Total Volumes below 580' msl
Based on 1986 Survey

Cross section no.	Cross-sectional area (ft^2)	Volume (ac-ft)		Surface width (ft)
		Segment	Total	
6	3878.3	0.0	0.0	743.3
6	3878.3	19.6	19.6	743.3
7	3652.3	34.6	34.6	687.1
8	5057.6	40.0	94.2	847.3
9	2130.6	112.2	206.4	266.0
10	1972.5	18.8	225.2	221.2
11	2034.7	18.4	243.6	230.9
12	1934.4	18.2	261.8	229.4
13	1935.9	17.8	279.6	224.9
15	2825.8	32.8	312.4	889.5
16	2654.0	21.4	333.8	1363.3
17	2153.4	32.0	365.8	696.1

Table 6. Segment Volumes and Total Volumes below 580' msl Based on
HEC-6 Outputs after Spillway Failure

Cross section no.	Cross-sectional area (ft^2)	Volume (ac-ft)		Surface width (ft)
		Segment	Total	
M5	4057.2	0.0	0.0	735.0
M5	4057.2	20.5	20.5	735.0
M6	3771.0	19.8	40.3	725.6
M7	3568.7	42.1	82.4	579.6
M8	3425.2	40.1	122.5	571.1
M9	2790.0	24.3	146.8	210.0
M10	2359.9	8.9	155.6	237.9
M11	2254.3	5.3	160.9	320.3
M12	1944.3	4.8	165.8	291.3
M13	2341.2	12.3	178.1	267.8
M14	2043.3	20.1	198.2	232.7
M15	1816.4	35.4	233.6	213.8
M16	2200.1	36.9	270.5	306.2
M17	2513.1	21.6	292.2	1313.2
M18	2985.6	45.4	337.6	746.7

the size and location of the new and old channels, conducting velocity and discharge measurements at selected locations, and analyzing soil and sediment characteristics. The mathematical model simulation involved preparing data for the HEC-6 sediment transport model and evaluating its capability to reproduce what happened in the field. The model was also used to investigate certain hypothetical situations that could have happened or might happen in the future.

Conclusions

The failure of a spillway produces a sudden discontinuity in the stream channel profile from the upstream to the downstream channel, resulting in major scouring upstream of the spillway. The scouring of the channel migrates upstream until it encounters a non-erodible segment of channel or until the bed profile reaches a state of dynamic equilibrium. In the case of Lake Charleston, the scour channel migrated upstream until it encountered a rock ledge which was not eroded.

The rock ledge in the diversion channel played an important role in retarding the upstream migration of the scour channel and preventing a possible failure of the side channel reservoir.

The HEC-6 model simulates the scouring which results from a spillway failure with reasonable accuracy and thus could be used in emergency cases and in investigating possible impacts of a spillway failure.

The stability of the side channel reservoir dike at Lake Charleston will continue to be threatened by seepage and the flow of the Embarras River until the spillway is replaced.

In general, the lesson learned from the Lake Charleston spillway failure is the need for careful evaluation regarding possible locations of a scour channel, such as the one that developed in Lake Charleston, in relation to the siting of important structures upstream of a spillway or dam.

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Appendix A
Cross-Sectional Profiles (1986 Survey)

CROSS SECTION 1

CROSS SECTION 2

CROSS SECTION 3

CROSS SECTION 4

65

CROSS SECTION 5

CROSS SECTION 6

66

CROSS SECTION 7

CROSS SECTION 8

CROSS SECTION 9

CROSS SECTION 10

CROSS SECTION 11

CROSS SECTION 12

CROSS SECTION 13

CROSS SECTION 14

70

CROSS—SECTION 15

CROSS SECTION 16

71

CROSS SECTION 17

CROSS SECTION 18

CROSS SECTION 19

CROSS SECTION 20

73

CROSS SECTION 21

CROSS SECTION 22

CROSS SECTION 23

CROSS SECTION 24

75

CROSS SECTION 25

CROSS SECTION 26

76

CROSS SECTION 27

DISTANCE FROM LEFT EDGE OF WATER, FT

CROSS SECTION 28

DISTANCE FROM LEFT EDGE OF WATER, FT

CROSS SECTION 29

Plot: ELEVATION ABOVE MSL, FT (vertical axis, 540 to 600) vs DISTANCE FROM LEFT EDGE OF WATER, FT (horizontal axis, 0 to 1600)

Appendix B
Velocity and Discharge Data

```
CROSS  SECTION NUMBER        2300
DATE OF DATA COLLECTION     5/ 2/86
TOTAL CROSS SECTION AREA    349 sq.ft.
TOTAL DISCHARGE             790 cfs
AVERAGE VELOCITY            2.3 fps
```

DISTANCE	DEPTH	VELOCITY	AREA	DISCHARGE
(ft)	(ft)	(fss)	(sq.ft.)	(cfs)
0.0	0.00	0.000	0.00	0.00
55.0	0.00	0.000	0.92	0.00
70.0	0.49	0.000	11.50	0.00
85.0	3.18	2.181	44.72	97.56
100.0	4.27	2.553	60.41	154.24
115.0	3.44	2.376	52.84	125.53
130.0	3.25	2.603	47.80	124.42
145.0	2.56	2.477	39.49	97.81
160.0	2.46	2.213	36.48	80.73
175.0	2.13	2.243	30.94	69.40
190.0	1.25	1.997	20.36	40.66
210.0	0.00	0.000	3.12	0.00

```
CROSS  SECTION NUMBER        2400
DATE OF DATA COLLECTION     5/ 2/86
TOTAL CROSS SECTION AREA    360 sq.ft.
TOTAL DISCHARGE             849 cfs
AVERAGE VELOCITY            2.4
```

DISTANCE	DEPTH	VELOCITY	AREA	DISCHARGE
(ft)	(ft)	(fps)	(sq.ft.)	(cfs)
0.0	0.00	0.000	0.00	0.00
50.0	0.00	0.000	13.74	0.00
75.0	4.40	2.596	73.63	191.14
90.0	4.10	2.289	61.15	139.95
105.0	3.61	2.371	53.15	126.00
120.0	2.59	2.580	41.40	106.83
135.0	2.92	2.353	42.57	100.18
150.0	2.59	2.591	38.32	99.29
165.0	1.97	2.797	30.70	85.86
185.0	0.00	0.000	4.92	0.00

```
CROSS SECTION NUMBER        2600
DATE OF DATA COLLECTION     5/ 2/86
TOTAL CROSS SECTION AREA    419 sq.ft.
TOTAL DISCHARGE             895 cfs
AVERAGE VELOCITY            2.1 fps
```

DISTANCE	DEPTH	VELOCITY	AREA	DISCHARGE
(ft)	(ft)	(fps)	(sq.ft.)	(cfs)
0.0	0.00	0.000	0.00	0.00
40.0	0.00	0.000	3.51	0.00
55.0	1.87	1.997	25.28	50.48
70.0	2.26	1.585	33.77	53.52
85.0	2.56	2.341	37.65	88.15
100.0	2.46	2.136	37.52	80.14
115.0	2.69	2.271	40.48	91.92
130.0	2.99	2.089	44.78	93.55
145.0	3.28	2.432	54.56	132.72
160.0	6.43	2.287	83.91	191.86
175.0	2.89	2.258	49.95	112.81
195.0	0.00	0.000	7.22	0.00

```
CROSS SECTION NUMBER        2800
DATE OF DATA COLLECTION     5/ 2/86
TOTAL CROSS SECTION AREA    427 sq.ft.
TOTAL DISCHARGE             873 cfs
AVERAGE VELOCITY            2.0 fps
```

DISTANCE	DEPTH	VELOCITY	AREA	DISCHARGE
(ft)	(ft)	(fps)	(sq.ft.)	(cfs)
0.0	0.00	0.000	0.00	0.00
35.0	0.00	0.000	4.12	0.00
50.0	2.20	1.903	29.22	55.61
65.0	2.39	2.169	35.56	77.13
80.0	2.39	2.317	36.72	85.08
95.0	2.82	2.145	42.45	91.03
110.0	3.31	1.944	48.97	95.20
125.0	3.41	1.988	50.81	101.04
140.0	3.31	1.908	49.89	95.18
155.0	3.31	2.118	49.52	104.87
170.0	3:22	2.184	47.06	102.79
185.0	2.49	2.216	29.40	65.17
195.0	0.00	0.000	3.12	0.00

```
CROSS SECTION NUMBER        2900
DATE OF DATA COLLECTION     5/ 2/86
TOTAL CROSS SECTION AREA    418 sq.ft.
TOTAL DISCHARGE             833 cfs
AVERAGE VELOCITY            2.0 fps
```

DISTANCE	DEPTH	VELOCITY	AREA	DISCHARGE
(ft)	(ft)	(fps)	(sq.ft.)	(cfs)
0.0	0.00	0.000	0.00	0.00
47.0	0.00	0.000	3.63	0.00
60.0	2.23	1.893	27.73	52.49
75.0	2.30	2.110	34.69	73.21
90.0	2.49	2.216	37.95	84.12
105.0	2.99	2.075	43.86	91.00
120.0	2.99	1.855	45.15	83.76
135.0	3.18	1.968	48.10	94.68
150.0	3.58	1.800	52.53	94.58
165.0	3.38	2.078	50.69	105.33
180.0	3.18	2.257	47.00	106.09
195.0	2.59	1.874	25.41	47.60
200.0	0.00	0.000	1.62	0.00

```
CROSS SECTION NUMBER        2300
DATE OF DATA COLLECTION     6/11/86
TOTAL CROSS SECTION AREA    454 sq.ft.
TOTAL DISCHARGE             1223 cfs
AVERAGE VELOCITY            2.7 fps
```

DISTANCE	DEPTH	VELOCITY	AREA	DISCHARGE
(ft)	(ft)	(fps)	(sq.ft.)	(cfs)
0.0	0.00	0.000	0.00	0.00
50.0	0.00	0.000	2.03	0.00
65.0	1.08	0.000	17.53	0.00
80.0	2.85	1.506	42.63	64.21
95.0	4.53	3.250	64.71	210.30
110.0	4.49	3.028	66.87	202.45
125.0	4,17	2.861	61.88	177.05
140.0	3.51	3.040	54.26	164.96
155.0	3.71	2.860	53.70	153.61
170.0	2.89	2.887	42.69	123.24
185.0	1.74	2.856	27.87	79.59
200.0	1.54	2.665	17.72	47.22
210.0	0.00	0.000	1.93	0.00

```
CROSS SECTION NUMBER        2400
DATE OF DATA COLLECTION     6/11/86
TOTAL CROSS SECTION AREA    436 sq.ft.
TOTAL DISCHARGE             1191 cfs
AVERAGE VELOCITY            2.7 fps
```

DISTANCE (ft)	DEPTH (ft)	VELOCITY (fps)	AREA (sq.ft.)	DISCHARGE (cfs)
0.0	0.00	0.000	0.00	0.00
55.0	0.00	0.000	6.89	0.00
70.0	3.67	2.921	51.80	151.30
85.0	5.58	3.041	79.35	241.34
100.0	5.18	2.803	76.52	214.51
115.0	4.13	2.940	63.18	185.75
130.0	3.71	2.686	55.67	149.53
145.0	3.31	2.676	49.15	131.54
160.0	2.62	2.557	45.58	116.56
185.0	0.00	0.000	8.20	0.00

```
CROSS SECTION NUMBER        2800
DATE OF DATA COLLECTION     6/11/86
TOTAL CROSS SECTION AREA    494 sq.ft.
TOTAL DISCHARGE             1241 cfs
AVERAGE VELOCITY            2.5 fps
```

DISTANCE (ft)	DEPTH (ft)	VELOCITY (fps)	AREA (sq.ft.)	DISCHARGE (cfs)
0.0	0.00	0.000	0.00	0.00
30.0	0.00	0.000	2.62	0.00
40.0	2.10	2.335	23.81	55.60
55.0	2.20	2.489	32.97	82.08
70.0	2.30	2.170	36.36	78.88
85.0	3.41	2.474	47.98	118.71
100.0	2.82	2.637	44.91	118.44
115.0	3.61	2.520	53.27	134.27
130.0	3.94	2.557	59.05	151.00
145.0	4.27	2.656	63.30	168.12
160.0	4.23	2.863	62.75	179.64
175.0	3.81	2.673	57.89	154.72
195.0	0.00	0.000	9.51	0.00

```
CROSS SECTION NUMBER        2900
DATE OF DATA COLLECTION     6/11/86
TOTAL CROSS SECTION AREA    500 sq.ft.
TOTAL DISCHARGE             1262 cfs
AVERAGE VELOCITY            2.5 fps
```

DISTANCE	DEPTH	VELOCITY	AREA	DISCHARGE
(ft)	(ft)	(fps)	(sq.ft.)	(cfs)
0.0	0.00	0.000	0.00	0.00
43.0	0.00	0.000	4.18	0.00
60.0	1.97	0.782	27.93	21.83
75.0	2.30	2.535	34.76	88.11
90.0	2.79	2.543	40.85	103.86
105.0	2.76	2.617	43.12	112.83
120.0	3.67	2.531	53.21	134.67
135.0	3.58	2.625	55.73	146.28
150.0	4.59	2.644	66.62	176.17
165.0	4.40	2.778	65.76	182.71
180.0	4.10	2.780	60.53	168.29
195.0	3.28	3.010	42.14	126.83
208.0	0.00	0.000	5.33	0.00

Appendix C
Cross-Sectional Profiles from HEC-6 Simulations

CROSS SECTION M0

CROSS SECTION M1

CROSS SECTION M2

CROSS SECTION M3

CROSS SECTION M4

CROSS SECTION M5

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CROSS SECTION M6

CROSS SECTION M7

CROSS SECTION M8

CROSS SECTION M9

90

CROSS SECTION M10

CROSS SECTION M11

CROSS SECTION M12

CROSS SECTION M13

92

CROSS SECTION M14

CROSS SECTION M15

CROSS SECTION M16

CROSS SECTION M17

94

CROSS SECTION M18

CROSS SECTION M19

CROSS SECTION M20